EARLY PERFORMANCE OF CONCRETE PAVEMENT
CONTAINING GROUND GRANULATED BLAST FURNACE SLAG

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CHAPTER 1
INTRODUCTION

1.1 GENERAL STATEMENT

A significant number of highway systems in the United States are constructed with rigid pavements. These highways should provide safe and efficient transportation of people and goods from one location to another. The pavements used must be able to withstand the repeated loading from high traffic volumes and heavy trucks. Portland Cement Concrete (PCC) provides characteristics such as strength, durability, and economy to highway pavements. However, more efficient pavement designs and construction methods are needed due to the high frequencies of rigid pavement deterioration. The performance of concrete pavements is affected by a number of factors including concrete properties, subgrade properties, environmental conditions, construction methods, and the repeated cycles of loading and unloading by high traffic volumes and heavy vehicles.

Premature deterioration of rigid pavements has led pavement designers to explore the use of admixtures to improve the durability of concrete. Many different types of slags can be used as admixtures, but only ground granulated blast furnace slag (GGBFS) is suitable for use as a cementitious material. Blast furnace slag is made from the molten iron collected at the bottom of the furnace and the liquid iron slag floating on the pool of
iron. When rapidly cooled, this iron forms a glassy granular material known as granulated blast furnace slag. It is then ground to a specified fineness thus deriving the name ground granulated blast furnace slag (GGBFS). Structural engineers have recommended the use of GGBFS as an admixture in the construction of bridge decks and columns for many years. The American Association of State Highways and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM) have approved the use of ground granulated blast furnace slag in specifications AASHTO M302 and ASTM C989. The addition of GGBFS to a portland cement concrete mixture has various effects on fresh concrete including an improvement of workability and finishing characteristics, a decrease in the rate and quantity of bleeding, and a delayed time of setting at low temperatures. The effects of GGBFS on hardened concrete include an increase in strength, decrease in permeability, and an increase in resistance to sulfate attack. Concrete is often described as “high performance” concrete when admixtures are used in portland cement concrete designs to attain improvements of normal concrete properties. The influence of GGBFS on concrete pavement performance has not been fully explored and much information is still needed. The concrete design for the U. S. Route 50 project specifies that 25% of the portland cement should be substituted with GGBFS. This provides an excellent research project for the investigation of GGBFS in rigid pavement.

The focus of the U. S. Route 50 project is to develop an instrumentation plan for three high performance concrete sections and one normal concrete pavement section. Once instrumented, field data can be collected and processed from the high performance
mixture and compared to the standard Ohio Department of Transportation Class-C concrete mixture. The GGBFS influences on the curing process and the performance of pavement containing GGBFS due to environmental factors will be monitored. Non-destructive tests from the Falling Weight Deflectometer (FWD) will be performed to obtain dynamic data on pavement response. The slab shapes will also be monitored. From this instrumentation, the data collected can be used to determine if it is economical to use GGBFS as an admixture in portland cement concrete slabs. Also, improvements in pavement design could originate from these test sections.

1.2 LITERATURE REVIEW

An extensive amount of research has been performed on rigid pavements in the past and a large volume of information has been published on the response of pavements to environmental factors. However, a small amount of information has been published on the effects of ground granulated blast furnace slag used as a cementitious substance in pavement mix designs.

Geiseler, Kollo, and Lang [1] presented results from laboratory work and long-term experience in practice on the effects of blast furnace slag used in concrete mix design. Blast furnace slag has been used extensively in Germany for the past 40 years. High contents of blast furnace slag, ranging from 36% to 80%, have been substituted for cement in concrete designs. The use of blast furnace slag in cements is shown to affect the strength development. In their study, concrete using standard portland cement generally showed higher compressive strengths during the first 28 days. After 28 days,
the blast furnace slag cements continued to gain strength and have higher compressive strengths. It was also noted that the use of granulated blast furnace slag provided high resistance against alkali-aggregate reaction, sulfate attack, and diffusion of chlorides into concrete. Research has shown that the electrolytic resistance of concrete increases with higher amounts of granulated blast furnace slag, which causes a decrease in the rate of corrosion of the steel reinforcement in the concrete.

ACI Committee 226 [2] reported various effects on the properties of concrete mixtures that use ground granulated blast furnace slag as a cementitious material in combination with portland cement. Concrete containing GGBFS showed improvements in workability and placeability when compared to concrete without GGBFS. The time of setting required for concrete mixtures with ground granulated blast furnace slag was also significantly increased at low temperatures. At temperatures greater than 85°F, the time of setting was not changed. In order for concrete to attain its potential strength and durability, proper moisture and temperature conditions must be maintained during early stages. Testing showed that the rate at which concrete gains strength was inversely proportional to the amount of GGBFS used in the mix. Blends with GGBFS generally showed higher flexural strengths at ages beyond 7 days. The heat of hydration and the permeability of concrete decreased for all mixtures incorporating the use of GGBFS. While resistance to sulfate attack increased with the use of GGBFS, no change was found in its freeze/thaw resistance. The modulus of elasticity for blends with ground granulated blast furnace slag was found to be essentially the same as concrete mixes without GGBFS. Concrete containing GGBFS appears bluish-green in color between the
second and fourth days due to the sulfide sulfur in the GGBFS reacting with other compounds in the cement. After curing, the GGBFS concrete becomes a lighter gray color than normal portland cement concrete. The three strength grades for GGBFS specified by ASTM standard C989 are grades 120, 100, and 80. The grade is determined by comparing the strength relation of a 50/50 percent GGBFS/portland cement blend to a blend using only portland cement. The relationship of these two blends is known as the slag activity index.

In the paper “Validation of Concrete Pavement Responses Using Instrumented Pavements,” Barenburg and Zollinger [3] compared data from two instrumented pavement sections to a theoretical mathematical model. The two test sections in Illinois were instrumented with PML-60 and PML-120 model strain gages that were manufactured by Tokyo Sokki Kenkyujo and purchased from Texas Measurements Incorporated. These strain gages were placed one-half inch below the top surface and one-half inch above the bottom surface of the pavement at various locations. Thermocouples were also embedded to monitor temperature changes in the concrete and temperature gradients throughout the slab depth. A detailed description of the installation methods used to place the strain gages was also given. Both static and dynamic loadings were used on the test pavement sections. The bond between the slab and the subbase played an important role in the results of the data. The results from the bonded condition were well represented by the mathematical model, but the calculated values were significantly higher when compared to the unbonded conditions. Curling and warping strains from varying climatic conditions were found to be much higher than anticipated.
The authors concluded that pavement responses from test sections could be used to verify mathematical models.

Armaghani, Larsen, and Smith [4] present data that describes the displacements of a concrete pavement slab due to temperature variation and weather. The test site consisted of six slabs that were each 20 feet long, 12 feet wide, and 9 inches thick. Thermocouples were placed in the concrete at 1”, 2.5”, 4.5”, 6.5”, and 8 inches from the top surface of the pavement. Another thermocouple was used to measure the ambient temperature. LVDT’s (Linear Variable Differential Transformers) were installed in both horizontal and vertical directions to monitor slab displacement. Information collected from the thermocouples was processed into average pavement temperatures, temperature differentials, and temperature gradients throughout the slab thickness. The pavement temperature was found to follow a pattern similar to the air temperature pattern, with the maximum and minimum pavement temperature occurring one to two hours later than the ambient temperature’s maximum and minimum. Slab curling was described from the correlation between the temperature differential and the vertical displacements measured by the LVDT’s at the center and edges of the slab. The maximum daily displacements occurred at the same time the maximum temperature differentials were recorded in the slab. The largest displacements were found to occur when weather conditions were clear and sunny. When the pavement surface was exposed to moisture and shade, the temperature distributions changed drastically.

In their paper “Field Instrumentation and Performance Monitoring of Rigid Pavements,” Rollings and Pittman [5] studied the effects of load and temperature-induced
stresses. Static loads were found to be more severe than dynamic loads, and therefore were used in the analytical model. Temperature gradients in the slab produced changes in the volume of the top and bottom of the slab. These volume changes caused stresses to develop in the concrete. The temperature-induced stresses were found to account for 16% to 60% of the total stresses (load and temperature-induced) in the pavement. Slab curling problems were analyzed with slabs of varying size, thickness, and joint spacing. The authors concluded that more data was needed to determine the combined effects of load and temperature-induced stresses on the performance of rigid pavements.

Choubane and Tia [6] conducted an experimental and analytical study to determine the effects of thermal-induced stresses in concrete pavements. In their paper “Analysis and Verification of Thermal-Gradient Effects on Concrete Pavement,” the authors confirmed that the temperature distributions throughout the depth of the slab was nonlinear and could be represented fairly well by a quadratic equation. Falling Weight Deflectometer (FWD) loads were used to induce strains at various locations on the test slab. The test section consisted of six concrete slabs, each 20 feet long, 12 feet wide, and 9 inches thick. The temperatures of the concrete slabs were measured by five thermocouples positioned at 1”, 2.5”, 4.5”, 6.5”, and 8 inches below the slab surface. When a nonlinear temperature distribution was used, the maximum computed tensile stresses in the slab were lower for the daytime condition and higher for the nighttime condition when compared to the calculated stresses from an assumed linear temperature distribution. The authors concluded that it was essential to consider the simultaneous
effects of load and temperature gradients in the design and analysis of concrete pavements.

1.3 OBJECTIVES

The objectives of this thesis are somewhat different than the overall objectives of the U. S. Route 50 project. Only two of the high performance concrete sections have been instrumented at this time. The third high performance concrete section and the normal concrete section will be constructed in the fall of 1998. In addition to the data presented in this thesis, data will be collected from the remaining two sections and the feasibility of GGBFS in pavement design will be determined. The objectives for the instrumentation of the first two sections are as follows:

- Develop a computer program to monitor the gages in the instrumented test sections.
- Successfully install the instrumentation to ensure that the location and orientation of the gages remain in the exact position after the slip-form paving operation passes through the test sections.
- Present test procedures required for evaluation of the performance of concrete slabs containing GGBFS due to fluctuating environmental conditions and dynamic loading from the Falling Weight Deflectometer.
- Determine deflections at the pavement surface caused by the application of loads from the Falling Weight Deflectometer.
- Develop a method to monitor the shape of the rigid pavement.
- Present strains and temperatures developed in the rigid pavement caused by the curing process of the concrete and changes in the environmental conditions. The influence of the GGBFS on the curing process can be determined from this data.
1.4 OUTLINE

This thesis contains information on the instrumentation of the first two high performance sections and detailed information on the various types of strain gages embedded in the pavement. The methods used to collect and process this data will also be discussed.

CHAPTER 2 presents a description of the project location, a detailed description of the sensors used in the test slabs, the preparation and layout for the instrumented sections, and the installation procedures used to place the sensors in the test sections.

CHAPTER 3 describes the data acquisition systems and testing procedures used to monitor the dynamic, environmental, and Dipstick® 2000 testing.

CHAPTER 4 presents a detailed list of all the formulas required to process the raw data, a description of the figures located in the Appendices, and a discussion of the data collected from the two high performance concrete sections.

CHAPTER 5 summarizes the data and presents conclusions and recommendations for future research projects involving rigid pavement.
CHAPTER 2

PROJECT DESCRIPTION, INSTRUMENTATION SELECTION, PREPARATION, LAYOUT, AND WIRE LABELING

2.1 PROJECT DESCRIPTION

The high performance rigid pavement site is located east of Athens, Ohio on U.S. Route 50. A total of seven miles of highway will be reconstructed and upgraded to four lanes of rigid pavement. There are two instrumented pavement sections located in the eastbound driving lane with 135 various embedded gages. The test sections begin approximately 300 feet east of the intersection of U. S. Route 50 and Athens County Road 24A. Personnel from the Ohio Research Institute for Transportation and the Environment (ORITE) at Ohio University and the Ohio Department of Transportation (ODOT) specified what types of instruments were to be used. The two instrumented sections were named High Performance Section One and Section Two. Each section consisted of 3 slabs that were 21 feet long, 12 feet wide, and 10 inches thick. Figure 2.1 shows the layout for the two test sections.
Figure 2.1 Layout of HP Sections 1 and 2
The majority of the instruments were installed between station 155+57 and station 156+83. Three Time Domain Reflectometry (TDR) probes were placed under an unsealed joint at station 160+19. Preparations to instrument the two test sections began on Tuesday, October 7th and continued until the concrete was placed on Thursday, October 16th.

The same section composition was used throughout the project. The base consisted of six inches of Dense Graded Aggregate Base (DGAB) and four inches of Non-Stabilized Drainage Base (NSDB), Type NJ. The ODOT item 304 aggregate base was used for the DGAB mixture. This mixture of crushed carbonate stone conformed to the sieve specifications given in Table 2.1. The New Jersey base is also a mixture of crushed carbonate stone that consists of a smaller amount of fines. The sieve specifications for the NJ base are also given in Table 2.1. A layer of bituminous prime coat was applied at 0.4 gallons per square yard in between the two types of base. The plots in Appendix A present a cross-section of this pavement.

**Table 2.1 Sieve Analysis Specifications for Aggregate Base**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Total % Passing</th>
<th>Sieve Size</th>
<th>Total % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 mm (2 in)</td>
<td>100</td>
<td>38 mm (1 ½ in)</td>
<td>100</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>70 ~ 100</td>
<td>25 mm (1 in)</td>
<td>95 ~ 100</td>
</tr>
<tr>
<td>19 mm (3/4 in)</td>
<td>50 ~ 90</td>
<td>12.5 mm (1/2 in)</td>
<td>60 ~ 80</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>30 ~ 60</td>
<td>4.75 mm (No. 4)</td>
<td>40 ~ 55</td>
</tr>
<tr>
<td>600 μm (No. 30)</td>
<td>90 ~ 33</td>
<td>2.36 mm (No. 8)</td>
<td>5 ~ 25</td>
</tr>
<tr>
<td>75 μm (No. 200)</td>
<td>0 ~ 13</td>
<td>1.18 mm (No. 16)</td>
<td>0 ~ 8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>300 μm (No. 50)</td>
<td>0 ~ 5</td>
</tr>
</tbody>
</table>
All the wires from the concrete test slabs were routed in trenches to concrete pull boxes that were placed adjacent to the driving lane shoulder. Each test section was equipped with its own concrete pull box to store and protect the wires. A hole was cut into one side of the pull box before it was placed on a drainable bed of aggregate. After all the wires were placed inside the boxes, fill was placed around the pull boxes so the tops would be level with the ground surface. A 120-volt electric outlet supplied power to the data collection systems when testing was required.

2.2 INSTRUMENTATION SELECTION

The sensors were selected based on cost, durability, sensitivity, and previous success of various gages used by Ohio University personnel. The sensors’ durability was a major concern since they had to survive the construction process, environmental changes, and withstand heavy traffic loads in the future. The gages also had to be sensitive enough to record changes in the concrete caused by fluctuations in the environment and various types of loading. The instruments used in the U.S. Route 50 project were able to measure changes in strain, temperature, moisture, and slab shape.

2.2.1 Strain Measurement

The gage length was the major criteria used to select the strain gages for concrete pavement. The basic composition of concrete is large aggregate, fine aggregate, cement, and water. A gage length larger than the maximum aggregate size had to be chosen to accurately measure the strain in the aggregate / mortar matrix. There were three types of strain gages used in this project; the Micro Measurements EGP-5-120 embedment strain
gage, the TML KM-100B strain transducer, and the Geokon Model VCE-4200 vibrating wire strain gage.

Micro Measurements Division of Measurements Group, Inc. of Raleigh, North Carolina manufactures and distributes the Micro Measurement EGP-5-120 embedment strain gages. The outer casing of the gage is composed of a concrete/polymer composite which measures 5 inches long, 0.7 inch wide, and 0.4 inch thick (Figure 2.2). This composite casing provides protection against corrosion and mechanical damage during installation. Although the recommended operating temperature is from -5°C to +50°C, the gage will still function between the temperatures of -30°C to +65°C. The sensing grid of the strain gage is made of modified Karma foil that is placed on a polyimide backing. Each strain gage has a nominal resistance of 120 ± 0.8% ohms, a gage factor of 2.05 ± 1.0%, and an active gage length of 4 inches. The Micro Measurement strain gage is only capable of measuring dynamic strains.

The TML KM-100B strain transducer is an embedment transducer that is designed to measure temperature and curing strains as well as dynamic strains of concrete. It is manufactured by the Tokyo Sokki Kenkyujo Corporation and distributed by Texas Measurements, Inc. of College Station, Texas. The KM-100B operates as a 350-ohm full Wheatstone bridge configuration and has a gage factor of approximately 2.00. The strain limit is ± 0.5%, and the operating temperature is from -20°C to +80°C. The casing of the transducer is constructed of a rubber-coated sleeve and two metal flanges. It is 4 inches long and has a ¾-inch diameter (Figure 2.3). Four active strain gages are used to measure strain. Changes in temperature are measured using one active
gage and three external resistors. The TML KM-100B strain transducer can be used to measure both static and dynamic strains.

The Geokon Model VCE-4200 vibrating wire strain gage is designed primarily to measure long-term static strains and temperature changes in concrete, and it is not suitable for measurement of dynamic strains. It is manufactured and distributed by Geokon, Inc. of Lebanon, New Hampshire. The gage is made up of two parts: 1) a rubber-coated steel tube that protects a steel wire held in tension between two end flanges and 2) a small plastic block 0.75 inches in diameter that contains an electromagnetic coil and a thermistor. The steel tube is 6 inches long and has a flange diameter of 0.75-inches (Figure 2.4). This gage will operate between the temperatures of -20°C and +80°C. The vibrating wire strain gage has a strain limit of ± 0.3% and a sensitivity of ± 1.0 microstrain. Strains are measured in the concrete by the vibrating wire principle. A steel wire is held in tension between the two end flanges. As deformations occur in the concrete, the two flanges move relative to one another causing changes in the steel wire’s tension. The electromagnetic coil “plucks” the wire and measures the change in resonant frequency of vibration in the steel wire. This change in frequency is converted to strain through a simple equation. Changes in temperature can also be measured with the vibrating wire strain gage by means of an internal thermistor. The thermistor measures changes in resistance. These resistance readings are then converted to temperatures by a polynomial equation.
Figure 2.2 Micro Measurement Strain Gage
Figure 2.3  KM-100B Strain Transducer
2.2.2 Temperature Measurement

The KM-100B and Vibrating Wire strain gages were used to measure temperature changes in the concrete pavement. In addition to these strain gage measurements, thermocouples were placed at various depths in order to obtain the temperature gradient throughout the concrete. A thermocouple is constructed of two dissimilar metals joined together at one end and protected in a stainless steel tube. When the two metals are twisted together, a small voltage is induced because each metal has a different number of free electrons at different temperatures [7]. Copper and constantan were the two types of metals used in the thermocouples.

Thermocouples only measure the temperature at a specific location. In order to obtain the variation in temperature throughout the slab depth, an arrangement of thermocouples was used. Five single thermocouples were tied to a steel rod that held them in place at one inch, two inches, five inches, eight inches, and nine inches below the surface of the concrete pavement. Single thermocouples were also tied to four of the strain gage chairs.

2.2.3 Subgrade Moisture Measurement

Campbell Scientific, Inc. of Logan, Utah developed a moisture measurement system that utilizes Time Domain Reflectometry (TDR) to obtain the moisture content of in-situ soils. An electric pulse is sent through a coaxial cable to the probes. The condition of the soil medium around the probes influences the velocity of the reflected wave. The propagation velocity measured across the TDR probes is used to calculate the moisture content of the soil. The dielectric constants of many types of dry soils are
known. The wave velocity is inversely proportional to the soil’s dielectric constant. Since water has a very high dielectric constant, a wet soil will cause a decrease in the velocity of the reflected wave.

2.2.4 Slab Shape

The Face Construction Technology Dipstick® 2000 was used to monitor changes in the shape of the slab. This device was designed to obtain profile measurements at a rate and accuracy greater than that of traditional rod and level survey procedures. A battery, LCD display, and an inclinometer (pendulum) are located inside the main body of the Dipstick®. The sensor inside the Dipstick® is positioned so that its axis and the line passing through the contact points of the footpads are co-planar. As the Dipstick® is pivoted from one leg to the other, the sensor becomes unbalanced and the LCD display turns blank. When the Dipstick® reaches a state of equilibrium, the difference in elevation between the two footpads is measured and displayed on the LCD panel. A box eleven feet wide and twenty feet long was drawn on the pavement along with a diagonal between two corners. The Dipstick® was traversed around the box and along the diagonal to measure the changes in elevation of the slab surface. All the data was logged in the palmtop computer that was attached to the handle of the Dipstick®.

2.3 PREPARATION AND LAYOUT

In order for the test sections to have power, a location had to be chosen where a power drop from existing lines could be made. OU personnel decided that a pole could be placed along Athens County Road 24A and power could be routed to the test sections
from this pole. For this reason, the first slab was placed at station 155+57. At this section, the 6” layer of 304 aggregate base was already in place and compacted to meet ODOT specifications. A 4-inch shallow underdrain was also buried and surrounded by pea gravel and filter fabric throughout the test sections.

The first step was to stake out the six test slabs that were to be instrumented. A small pen mark was made on top of the driving lane stringline at station 155+57. A five-second transit was then set up directly over top of the stringline at this station. After the transit was backsighted and zeroed in on the stringline, a 90° angle was turned to locate the other end of the transverse joint. A plumb bob was used to mark the passing lane stringline so a stake could be placed at this location. Twenty-one feet was measured to locate the next transverse joint. This procedure of backsighting and turning 90° was repeated for the final seven transverse joints. A steel pin that was ¾-inch in diameter and 24 inches long was hammered approximately 12 inches into the base. These pins were painted with a fluorescent paint so the contractors would not remove them.

The TDR probes were the first instruments placed. These probes were placed at five locations in the subgrade layer beneath the 304 aggregate base. Three TDR probes were buried at each location. The contractor’s stringline in the 10-foot shoulder was positioned exactly five feet from the edge of the driving lane. The center of the driving lane was located by measuring eleven feet parallel to the contractor’s stringline. At this distance, a string was held in tension over the 21’ length and 10’-6” was measured from the approach end of the slab to locate the slab’s center. A 4-inch underdrain pipe was already buried in the subgrade beneath the joint between the driving lane and the 10-foot
shoulder, therefore TDR positions two and three were moved six inches inside of the joint under the driving lane. TDR position 2 (TDR-2) was placed in the center of slab #5 and TDR-3 was located under the transverse joint between slab #5 and slab #6. TDR position 4 was placed under the center of the last transverse joint of slab #6. The Ohio Department of Transportation was investigating the differences between joints with sealant and joints without sealant. It was decided that the last TDR position should be placed under an unsealed joint. The unsealed transverse joints were to begin at station 160+00; therefore, the last TDR location (TDR-5) was placed 11 feet from the stringline under the transverse joint at station 160+19. The locations of the Time Domain Reflectometry probes are given in Table 2.2. After all the positions were located, an ODOT drill rig excavated holes, and the TDR probes were placed. A more detailed description of this process is given in section 2.4.1. The layout for the TDR probes is given in Figure A.1.

Table 2.2 TDR Probe Locations

<table>
<thead>
<tr>
<th>TDR Location #</th>
<th>Station</th>
<th>Distance from Shoulder Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>156+51.5</td>
<td>6 feet</td>
</tr>
<tr>
<td>2</td>
<td>156+51.5</td>
<td>6 inches</td>
</tr>
<tr>
<td>3</td>
<td>156+62</td>
<td>6 inches</td>
</tr>
<tr>
<td>4</td>
<td>156+83</td>
<td>6 feet</td>
</tr>
<tr>
<td>5</td>
<td>160+19</td>
<td>6 feet</td>
</tr>
</tbody>
</table>

In order to evaluate changes in elevations of the concrete slabs, a reference rod was placed adjacent to each slab in the 4-foot berm. A distance of 29 feet was measured along the simulated transverse joint from the stringline in the 10-foot shoulder. At these positions, a nail was placed and string was connected to these nails to denote the outside edge of the passing lane. The contractor for this project slip-formed the two 12-foot lanes
at the same time. They returned at a later date to pour the 4-foot berm and the 10-foot shoulder. Since the distance between the footpads of the Dipstick® 2000 is 12 inches, the reference pins were placed six inches from the edge of the passing lane so the Dipstick® could be used from the reference pin. Once these positions were marked, a 3” core bit was used to drill a hole approximately 10 feet deep. Due to inconsistencies in the subgrade, some of the reference holes had to be repositioned. Once all the holes were drilled, a 2” diameter PVC pipe was placed into the holes. This 2” pipe was adjusted so the top would be approximately 3” above the compacted 304 aggregate base. The opening of the pipe was taped closed to prevent any New Jersey base from falling into the reference pipes. The locations of the reference pipes were recorded so they could be relocated. The remaining 4” layer of Non-Stabilized Drainage Base (NSDB), Type NJ, was placed and compacted to meet ODOT specifications. The reference pipes were found and a 2” to 4” expander was attached to the top of the 2-inch pipe. These expanders were also taped closed until the reference rods were ready to be placed in the pipes. After the two 12’ lanes were poured, the reference rods were placed in the PVC pipes. The reference rods measured 12’ long and ¼-inch in diameter and were painted for protection against rust. After the rods were placed in the holes, grout was poured to the bottom of the pipe to prevent settlement and spacers were used to center the rods in the 2” pipe.

The layout of the strain gages could not begin until the 4” layer of New Jersey base was placed and compacted. Markings were made on the NJ base to denote the positions of the strain gages. The driving lane was the only lane instrumented. To locate
the edges of the driving lane, measurements of 5 feet and 17 feet were made parallel to
the contractor's stringline along the simulated transverse joints and nails were placed.
Strings were connected to these nails to simulate the 12' driving lane. Distances of 30'',
6', and 9'-6" were measured parallel to the edge of the driving lane to mark the
wheelpaths and the centerline of the driving lane. In slab #1, marks were made on the
New Jersey base along the 30" and 6' distances at 1'-6", 4', and 10'-6" parallel to the
approach end of the slab. Slab #2 was laid out the same as slab #1 with an additional row
of markings along the 9'-6" line at the same distances. In slab #3, one measurement of
10'-6" was made along the 30" and 9'-6" distances and three measurements of 1'-6", 4',
and 10'-6" were marked along the centerline of the driving lane. In order to layout the
KM-100B strain gages, a distance of 18" was measured parallel to the outside edge of the
driving lane and along the simulated transverse joint between slabs #2 and #3. At the
intersection of these distances, a mark was made on the aggregate base at a 45° angle.
This procedure was repeated to mark the last position in slab #3 for a total of three KM-
100B locations per section. These same procedures were used to layout the second
instrumented section due to the similarity of the two sections. The layouts of these
sections are presented in Figures A.2 through A.7 in Appendix A.
2.4 INSTRUMENTATION INSTALLATION

One of the most critical components of the project was the installation of the instruments in the test sections. The procedures used to install the measuring devices were based on past experiences of the Center for Geotechnical and Environmental Research (CGER) projects at Ohio University. A detailed description of the various installation techniques used in this project is given below.

2.4.1 Soil Moisture Probes

The TDR probes were installed before the 4" layer of New Jersey base was placed to avoid disturbing the required elevation tolerances of the base. An ODOT drill rig used a 12" core bit to excavate the TDR holes to the correct depth. This bit was placed inside a 14" diameter hole that was cut out of a sheet of cardboard. The cardboard was used to catch the displaced soil and prevent it from falling back into the hole. The displaced soil was set aside in containers until it was needed later. After the hole was finished, a straight edge was placed across the top of the hole to measure the correct depths for the TDR positions. The five TDR locations each consisted of three TDR probes. The TDR probes were placed parallel to one another at 24", 12", and 6" below the layer of Dense-Graded Aggregate Base (Item 304). The subgrade material from the containers was placed and compacted in between the probes in 1 ½" lifts. The cables from the TDR probes were routed in a shallow trench in the 6" layer of Dense-Graded Aggregate Base to the 10-foot shoulder. A diagram of the TDR probe positions is given in Figure A.1. A list of the wire numbers corresponding to the TDR locations is given in Table 2.3.
Table 2.3 TDR Probe Wire Numbers and Locations

<table>
<thead>
<tr>
<th>TDR Location</th>
<th>Station</th>
<th>Depth</th>
<th>Wire Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>156+51.5</td>
<td>6&quot;</td>
<td>7</td>
</tr>
<tr>
<td>1</td>
<td>156+51.5</td>
<td>12&quot;</td>
<td>8</td>
</tr>
<tr>
<td>1</td>
<td>156+51.5</td>
<td>24&quot;</td>
<td>9</td>
</tr>
<tr>
<td>2</td>
<td>156+51.5</td>
<td>6&quot;</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>156+51.5</td>
<td>12&quot;</td>
<td>14</td>
</tr>
<tr>
<td>2</td>
<td>156+51.5</td>
<td>24&quot;</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>156+62</td>
<td>6&quot;</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>156+62</td>
<td>12&quot;</td>
<td>17</td>
</tr>
<tr>
<td>3</td>
<td>156+62</td>
<td>24&quot;</td>
<td>18</td>
</tr>
<tr>
<td>4</td>
<td>156+83</td>
<td>6&quot;</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>156+83</td>
<td>12&quot;</td>
<td>20</td>
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<tr>
<td>4</td>
<td>156+83</td>
<td>24&quot;</td>
<td>21</td>
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<tr>
<td>5</td>
<td>160+19</td>
<td>6&quot;</td>
<td>4</td>
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<td>5</td>
<td>160+19</td>
<td>12&quot;</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>160+19</td>
<td>24&quot;</td>
<td>6</td>
</tr>
</tbody>
</table>

2.4.2 Strain Gages

The two goals for the installation of the gages were to guarantee that there were no voids between the concrete and the gages, and to maintain the precise position of the gage within the concrete. The procedures used to attain these goals were based on previous instrumentation projects. To ensure that the gages remained at their correct elevations and positions, steel chairs were fabricated and painted for rust protection. These stands consisted of two separate legs that allowed the gages to be unrestrained. The stands were designed to hold the strain gages at 1 ½-inches and 8 ½-inches below the surface of the pavement. Two different types of legs were made for the Micro Measurement, Vibrating Wire, and KM-100B strain gages. The main difference between
the two types of stands was the contact point between the gage and the legs of the stand. A flat surface was made for the Micro Measurement gages and a half-circular surface was fabricated into the stand legs for the Vibrating Wire and KM-100B strain gages. The strain gages were placed on the contact surfaces of the legs and held in place with plastic tie straps. These steel chairs were placed in the positions given in Appendix A and anchored to the base with 6” steel hooks. The chairs for the Micro Measurement and Vibrating Wire strain gages were placed in the approach half of the test slab due to the symmetry of the developed strains in the slab. After the gages were in place, the top elevation of the strain gages was measured relative to the elevation of the paving stringline to ensure that the top gage was 1 $\frac{1}{2}$-inches below the surface of the concrete.

The first slab of each section was instrumented with Micro Measurements EGP-5-120 strain gages. These gages were placed along the centerline and right-wheelpath of the driving lane and oriented along the longitudinal axis of the slab. The locations of these gages are given in Figure A.2. The EGP-5-120 strain gages were attached to the stands with plastic tie straps.

The Vibrating Wire Strain Gages were placed in two different orientations in slabs #2 and #3. A longitudinal orientation was used in slab #2 along the center and both wheelpaths of the driving lane. This orientation is given in Figure A.3. In slab #3, gages were placed transverse to the slab at the positions given in Figure A.4. After the chairs were anchored in the correct locations, the gages were secured to the stands with plastic tie straps. The Vibrating Wire Strain Gage layout for slabs #2 and #3 are given in Figures 2.5 and 2.6.
Figure 2.5  VWSG Placement in Slab #2

Figure 2.6  VWSG Placement in Slab #3
The locations of the KM-100B strain transducers are given in Figures A.2 and A.3 in Appendix A. These gages were oriented on a 45° angle to monitor the strains developed in the corners of the slabs. The KM-100B gages were attached to the steel chairs with plastic tie straps and positioned so approximately ¾” extended past the stand. This allowed the metal flanges of the KM-100B to be completely surrounded in concrete.

A four-sided steel box fabricated from sheet metal was assembled around the strain gages. These boxes were 9” tall, 12” long, and 6” wide. The boxes consisted of four separate pieces that pulled apart in the vertical direction with relative ease. Two 4” long pins were welded to a ½” angle on the 12” by 9” metal sides to anchor the box into the base layer. The 6” by 9” sides slid into the ½” angle to complete the formation of the box. After the boxes were in position, concrete was placed by hand in the boxes and vibrated to eliminate air pockets between the gage and the concrete. The height of the boxes was designed to provide one inch of clearance as the paving machinery passed through the test sections. These boxes were strong enough to provide resistance against the flowing forces of the concrete developed by the paver, yet weak enough in the vertical direction to be pulled apart after the pavement was in place. A layout of the sections is given in Figure 2.7. Pictures of the assembled steel boxes surrounding the three types of strain gages are given in Figures 2.8, 2.9, and 2.10.
Figure 2.7  Route 50 Strain Gage Layout

Figure 2.8  Micro Measurement Strain Gage in Steel Box
Figure 2.9  KM-100B Strain Transducer in Steel Box

Figure 2.10  Vibrating Wire Strain Gage in Steel Box
2.4.3 Thermocouples

The placement of the thermocouples was not performed until the sections were completely ready for paving. The thermocouples were placed in two different arrangements as shown in Figures A.8 and A.10. The single thermocouples that were monitored by the maturity meter were attached to strain gage stands at 1 ½-inches and 8 ½-inches below the surface of the pavement. The locations of the single thermocouples are given in Table 2.4.

Table 2.4 Location of Single Thermocouples

<table>
<thead>
<tr>
<th>Wire #</th>
<th>Section</th>
<th>Location</th>
<th>Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>HP1</td>
<td>VW-12</td>
<td>Bottom</td>
</tr>
<tr>
<td>7</td>
<td>HP1</td>
<td>VW-12</td>
<td>Top</td>
</tr>
<tr>
<td>8</td>
<td>HP1</td>
<td>KM-3</td>
<td>Bottom</td>
</tr>
<tr>
<td>9</td>
<td>HP1</td>
<td>KM-3</td>
<td>Top</td>
</tr>
<tr>
<td>10</td>
<td>HP2</td>
<td>MM-7</td>
<td>Bottom</td>
</tr>
<tr>
<td>11</td>
<td>HP2</td>
<td>MM-7</td>
<td>Top</td>
</tr>
<tr>
<td>12</td>
<td>HP2</td>
<td>MM-6</td>
<td>Bottom</td>
</tr>
<tr>
<td>13</td>
<td>HP2</td>
<td>MM-6</td>
<td>Top</td>
</tr>
</tbody>
</table>

For the second arrangement, five thermocouples were affixed to a steel rod with plastic ties. The steel rod was placed into the base and tied to the wire mesh so the thermocouples would remain at 1”, 2”, 5”, 8”, and 9” below the pavement surface. The multi-depth thermocouple arrangements were placed in the center slab of each section. Colored tape was placed on the thermocouple wires to denote their positions in the test slabs. The location of these multi-depth thermocouple stakes is shown in Table 2.5. The layouts for the thermocouples are given in Figures A.8 through A.10.
Table 2.5  Multi-Depth Thermocouple Locations

Note: Wire #1 is 1” from the surface and #5 is 1” from the base.

<table>
<thead>
<tr>
<th>Wire #</th>
<th>Tape Color</th>
<th>Section &amp; Slab #</th>
<th>Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-5</td>
<td>Green</td>
<td>HP Section 1-Slab 2</td>
<td>center of slab</td>
</tr>
<tr>
<td>1-5</td>
<td>Red</td>
<td>HP Section 1-Slab 2</td>
<td>in front of KM-1</td>
</tr>
<tr>
<td>1-5</td>
<td>White</td>
<td>HP Section 2-Slab 2</td>
<td>center of slab</td>
</tr>
<tr>
<td>1-5</td>
<td>Yellow</td>
<td>HP Section 2-Slab 2</td>
<td>in front of KM-4</td>
</tr>
</tbody>
</table>

2.5  PLACEMENT OF CONCRETE

The procedure the contractor used to place the pavement consisted of pouring the first six inches of concrete, placing the wire mesh on top of the six-inch layer, and finishing with a layer of four inches for a total of ten inches of concrete pavement. This procedure could not be used through the test sections because the gages were held in place at 1 ½-inches and 8 ½-inches above the Non-Stabilized Drainage Base layer. Therefore, one layer of concrete ten inches thick was placed throughout the instrumented slabs. Steel racks were used to hold the wire mesh at five inches above the DGAB layer. An instrumented slab with wire mesh in place is given in Figure 2.11.

Concrete was carefully placed by hand in the boxes so the gages would remain in their correct positions. It was then vibrated to eliminate any air pockets in the box. Concrete was also placed around the multi-depth thermocouple stakes for protection against the flowing forces of the paver. After the paver and spreader passed, the boxes had to be removed from the concrete. A scaffolding set up on wheels spanned the freshly paved concrete. This scaffolding allowed ORITE personnel to position themselves directly above the steel boxes. A tape measure was used to locate the positions of the
boxes from the edge of the driving lane. Channel-Lock pliers were used to remove the boxes from the pavement. Fresh concrete was placed and vibrated where the boxes were removed. The contractor's finishers used floats to provide a smooth driving surface throughout the test sections.

![Figure 2.11 Instrumented Test Slab with Wire Mesh](image)

### 2.6 INSTRUMENTATION WIRES AND LABELS

The wires were all routed in trenches to temporary wooden boxes located approximately 25' from the edge of the driving lane. Shallow trenches were dug in the compacted New Jersey base. Trenches approximately 10'' deep were excavated with a pick and shovel through the Dense-Graded Aggregate Base. The trenching machine in
Figure 2.12 was used to make trenches ranging from 24” to 36” deep. The wires were placed in the trenches and covered with the excavated soil. The soil and aggregate base was then compacted with a small mechanical vibrating compactor.

The wires were labeled in the ORITE laboratory at Ohio University before they were taken out to the test site. The labels consisted of the section type, gage type, gage position, and wire number. The serial numbers of the KM-100B and Micro Measurement gages were also written on the labels. The labels were written on yellow heat-shrink tube and placed on the wires. A clear heat-shrink was placed over top of the labels for protection. The heat-shrink prevented the labels from being removed from the wires. Label examples are given in Figure 2.13.
<table>
<thead>
<tr>
<th>Geokon VCE-4200 Vibrating Wire Strain Gage</th>
<th>Back</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>Back</td>
</tr>
<tr>
<td>HP1 – VWL – 3B</td>
<td>WIRE # 15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TML KM-100B Strain Transducer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
</tr>
<tr>
<td>KM100B - 422</td>
</tr>
<tr>
<td>Back</td>
</tr>
<tr>
<td>HP1 – WIRE 7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Micro Measurement EGP-5-120 Strain Gage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
</tr>
<tr>
<td>HP1 – MM4T</td>
</tr>
<tr>
<td>13 – 0416 – 1 - BR</td>
</tr>
<tr>
<td>Back</td>
</tr>
<tr>
<td>HP1 – MM4B</td>
</tr>
<tr>
<td>13 – 0419 – 2 - BL</td>
</tr>
</tbody>
</table>

where: **Geokon VCE-4200 Vibrating Wire Strain Gage**
- HP1 = High Performance section 1
- VWL = Vibrating Wire Longitudinal
- 3B = Bottom gage at Location 3

**TML KM-100B Strain Transducer**
- 1B = Bottom gage at Location 1
- 422 = Serial Number of gage

**Micro Measurement EGP-5-120 Strain Gage**
- MM4T = Micro Measurement top gage at location 4
- 13 = wire #13
- 0416-1 = Serial Number of gage
- BR = Brown cable wire scheme (all top gages were connected to brown)
- BL = Black cable wire scheme

**Figure 2.13 Instrumentation Labels**

The wiring schemes for the Micro Measurement and the KM-100B strain gages are given in Figure 2.14. Two Micro Measurement gages were connected to one field cable to use the wires in the cables more efficiently.
**Figure 2.14  Wire Schemes for Micro Measurement EGP-5-120 and KM-100B**

The serial numbers and locations of the KM-100B and Micro Measurement strain gages are given in Table 2.6. The Vibrating Wire Strain Gages did not have serial numbers.
Table 2.6 Serial Numbers of KM-100B and Micro Measurement Gages

**TML KM-100B**

<table>
<thead>
<tr>
<th>Location</th>
<th>Wire #</th>
<th>Serial Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Bottom</td>
<td>7</td>
<td>4KD-422</td>
</tr>
<tr>
<td>1 Top</td>
<td>8</td>
<td>4KD-378</td>
</tr>
<tr>
<td>2 Bottom</td>
<td>9</td>
<td>4KD-369</td>
</tr>
<tr>
<td>2 Top</td>
<td>10</td>
<td>4KD-338</td>
</tr>
<tr>
<td>3 Bottom</td>
<td>11</td>
<td>4KD-337</td>
</tr>
<tr>
<td>3 Top</td>
<td>12</td>
<td>4KD-414</td>
</tr>
<tr>
<td>4 Bottom</td>
<td>13</td>
<td>4KD-424</td>
</tr>
<tr>
<td>4 Top</td>
<td>14</td>
<td>4KD-368</td>
</tr>
<tr>
<td>5 Bottom</td>
<td>15</td>
<td>4KD-387</td>
</tr>
<tr>
<td>5 Top</td>
<td>16</td>
<td>4KD-444</td>
</tr>
<tr>
<td>6 Bottom</td>
<td>17</td>
<td>4KD-453</td>
</tr>
<tr>
<td>6 Top</td>
<td>18</td>
<td>4KD-412</td>
</tr>
</tbody>
</table>

**Micro Measurement EGP-5-120**

*High Performance Section 1*

<table>
<thead>
<tr>
<th>Location</th>
<th>Wire #</th>
<th>Serial Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 Top</td>
<td>13</td>
<td>0416 – 1</td>
</tr>
<tr>
<td>4 Bottom</td>
<td>13</td>
<td>0419 – 2</td>
</tr>
<tr>
<td>5 Top</td>
<td>14</td>
<td>0474 – 1</td>
</tr>
<tr>
<td>5 Bottom</td>
<td>14</td>
<td>0464 – 4</td>
</tr>
<tr>
<td>6 Top</td>
<td>15</td>
<td>0482 – 4</td>
</tr>
<tr>
<td>6 Bottom</td>
<td>15</td>
<td>0469 – 2</td>
</tr>
<tr>
<td>7 Top</td>
<td>16</td>
<td>0471 – 4</td>
</tr>
<tr>
<td>7 Bottom</td>
<td>16</td>
<td>0420 – 2</td>
</tr>
<tr>
<td>8 Top</td>
<td>17</td>
<td>0416 – 2</td>
</tr>
<tr>
<td>8 Bottom</td>
<td>17</td>
<td>0475 – 2</td>
</tr>
<tr>
<td>9 Top</td>
<td>18</td>
<td>0416 – 3</td>
</tr>
<tr>
<td>9 Bottom</td>
<td>18</td>
<td>0419 – 4</td>
</tr>
</tbody>
</table>
**Micro Measurement EGP-5-120**

*High Performance Section 2*

<table>
<thead>
<tr>
<th>Location</th>
<th>Wire #</th>
<th>Serial Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 Top</td>
<td>22</td>
<td>0466 – 2</td>
</tr>
<tr>
<td>4 Bottom</td>
<td>22</td>
<td>0464 – 1</td>
</tr>
<tr>
<td>5 Top</td>
<td>23</td>
<td>0463 – 2</td>
</tr>
<tr>
<td>5 Bottom</td>
<td>23</td>
<td>0465 – 3</td>
</tr>
<tr>
<td>6 Top</td>
<td>24</td>
<td>0463 – 4</td>
</tr>
<tr>
<td>6 Bottom</td>
<td>24</td>
<td>0419 – 1</td>
</tr>
<tr>
<td>7 Top</td>
<td>25</td>
<td>0469 – 1</td>
</tr>
<tr>
<td>7 Bottom</td>
<td>25</td>
<td>0421 – 1</td>
</tr>
<tr>
<td>8 Top</td>
<td>26</td>
<td>0462 – 3</td>
</tr>
<tr>
<td>8 Bottom</td>
<td>26</td>
<td>0472 – 1</td>
</tr>
<tr>
<td>9 Top</td>
<td>27</td>
<td>0479 – 4</td>
</tr>
<tr>
<td>9 Bottom</td>
<td>27</td>
<td>0426 – 2</td>
</tr>
</tbody>
</table>
CHAPTER 3

DATA ACQUISITION SYSTEMS
AND TESTING PROCEDURES:
DYNAMIC, ENVIRONMENTAL,
AND DIPSTICK® 2000 TESTING

3.1 DYNAMIC TESTING

The use of Non-Destructive Testing to impose stresses in concrete pavements has been a well-established method for experimentation. The Falling Weight Deflectometer (FWD) has been used in the past by ORITE personnel with good results. The advantage of using FWD is that the strains recorded by the data acquisition systems can be compared to the known applied loads. The point of contact of the FWD pad is designed to simulate traffic loading on the pavement. Different magnitudes of load can be applied to the FWD pad to simulate varying degrees of traffic loading.

3.1.1 FWD Data Acquisition System

For dynamic testing, the data acquisition system needed to have the ability to collect and store a large number of data points for short test duration. The Megadac 5108AC system manufactured by Optim Electronics Corporation of Germantown, Maryland was utilized for the Falling Weight Deflectometer testing. This system has been used extensively by ORITE personnel on the U. S. Route 23 project in Delaware, Ohio. The Megadac system operates as a 16-bit system and provides the ability to collect, filter,
and store up to 4 megabytes of memory. An interactive IEEE-488 communications bus was placed in the 486-66DX IBM compatible computer for communication between the computer and the Megadac. Test Control Software (TCS) was provided by Optim to allow the transfer of commands from the computer to the Megadac. This software provided the ability to balance the strain gages, start and stop data collection, and view data directly after it was collected. The setup of this system is shown in Figure 3.1.

One Analog Input Module in the Megadac was required to collect data from the U.S. Rout 50 project. The AD-1 808FB-1 module was used to interact with the KM-100B and Micro Measurement EGP-5-120 strain gages. This analog input module provided eight channels for measuring quarter, half, and full bridge strain gages. Jumper settings in the module could be set to provide an excitation and calibration voltage, gain, and filter frequency for each set of channels. The Megadac was set up to collect 1200 data points per second and filter at 100 hertz.

Both dynamic strain gages required a full bridge completion between the strain gage and the Megadac. The KM-100B strain gage needed a 350-ohm full bridge and the EGP-5-120 required a 120-ohm full bridge setup. The use of a Screw Terminal Block (STB) was utilized to provide these bridge completions. The STB provided eight channels for interfacing between the gage and the Megadac. Each channel consisted of five screw terminals for connecting the gages to the STB. Jumpers were arranged to provide a full bridge completion. The STB’s were designated STB 808FB/120 for the EGP-5-120 strain gage and STB 808FB/350 for the KM-100B strain gage. Plugs were assembled for the KM-100B strain gages. This provided a quick and error-free method
for the connection between the strain gage and the Screw Terminal Block. The KM-100B plug detail is given for the field end of the plug in Table 3.1.

<table>
<thead>
<tr>
<th>Field Wire #</th>
<th>Section</th>
<th>Field Wire Color</th>
<th>Pin #</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>19</td>
</tr>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>37</td>
</tr>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>18</td>
</tr>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>36</td>
</tr>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>17</td>
</tr>
<tr>
<td>7 / 13</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>35</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>16</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>34</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>15</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>33</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>14</td>
</tr>
<tr>
<td>8 / 14</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>32</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>13</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>31</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>12</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>30</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>11</td>
</tr>
<tr>
<td>9 / 15</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>29</td>
</tr>
<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>10</td>
</tr>
<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>28</td>
</tr>
<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>9</td>
</tr>
<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>27</td>
</tr>
<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>8</td>
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<tr>
<td>10 / 16</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>26</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>7</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>25</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>6</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>24</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>5</td>
</tr>
<tr>
<td>11 / 17</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>23</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>Red</td>
<td>4</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>Black</td>
<td>22</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>White/Red</td>
<td>3</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>White/Brown</td>
<td>21</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>Brown</td>
<td>2</td>
</tr>
<tr>
<td>12 / 18</td>
<td>HP1 / HP2</td>
<td>Shield</td>
<td>20</td>
</tr>
</tbody>
</table>
3.1.2 FWD Testing Procedure

The locations of the Micro Measurement and KM-100B strain gages were marked before the testing began. An X was made directly on the pavement above each stain gage location. The Megadac was set up in the back of the Ohio University box truck and connected to the STB for the KM-100B and EGP-5-120 strain gages. The KM-100B plugs were connected to the STB and the Micro Measurement wires were connected to the screw terminals of the STB. The strain gages were then balanced and testing was ready to begin.

ODOT personnel applied the load to the pavement using the Dynatest Model 8000 Falling Weight Deflectometer (Figure 3.2). The FWD was controlled inside the ODOT suburban by a laptop computer and the applied loads and measured deflections were recorded. Loads ranging from 9,920 to 18,584 lbs were applied to an 11.82-inch diameter plate. The base plate was designed to simulate heavy tire loads to the pavement. The applied pressures to the pavement ranged from 90.4 to 169.4 psi. Deflections were measured by geophones in the direction of traffic at -12", 0", 12", 18", 24", 36", and 60" from the applied load.

The FWD test on November 6, 1997 was only performed at four locations in HP Section 1. Test #2 was conducted on December 3, 1997 on both High Performance Sections. There was no traffic on the new pavement during these test dates. The TCS software on the computer allowed the operator to start and stop the collection of data by pressing the corresponding function key.
Figure 3.1  Megadac Data Acquisition Setup

Figure 3.2  Dynatest Model 8000 Falling Weight Deflectometer
3.2 ENVIRONMENTAL TESTING

Temperature differences throughout the depth of the slab cause curling to occur. Warping in a concrete slab is caused by differences in moisture throughout the slab. Moisture is lost as concrete cures thus causing shrinkage to occur. The data acquisition system had to have the ability to invoke a command to collect data for an extended period of time. The CR7 and CR10 data acquisition systems manufactured by Campbell Scientific, Inc. of Logan, Utah were utilized. These systems were programmed to collect and store data every 30 minutes. The CR7 system was used to collect data from the KM-100B gages and thermocouples. The CR10 system was the only Campbell Scientific system capable of collecting and storing data from the Vibrating Wire Strain Gages.

3.2.1 CR10 Data Acquisition System

The CR10 was used to collect and store temperature and strain values from the Vibrating Wire Strain Gages. The CR10 unit consisted of a thermocouple reference, cover, wiring panel, and base (Figure 3.3) [8]. The CR10 wiring panel is given in Figure 3.4 [8]. The 9-pin serial I/O port was used with the SC32A interface device. This device connected the laptop computer to the CR10. The SC32A allowed the transfer of programs and data between the computer and CR10. Campbell Scientific provided the software needed to program the CR10. The PC208 software consisted of two sections. The “Edlog” function was used to write the programs for the CR10. The “Graphterm” function was required to download the programs to the CR10 and collect data from the CR10.
Figure 3.4   CR10 Wiring Panel
Since each test section consisted of 28 Vibrating Wire Strain Gages, multiplexers were needed. The AM416 multiplexer provided 16 channels of four lines. These lines were labeled H1, H2, L1, and L2. A shield was shared between the four lines. The AM416 multiplexer provides the bridge completion for several gages. Sixteen differential strain gages could be monitored by one AM416 multiplexer. Therefore, two multiplexers were needed per section. An AVW vibrating wire interface was required to complete the circuitry needed to interface Geokon’s VCE-4200 series vibrating wire strain gage to the CR10. The CR10 required a constant 12-volt power source. An environmentally sealed box housed the CR10, AVW interface, and the AM416 multiplexers. The data acquisition circuit for the VCE-4200 vibrating wire is given in Figure 3.5.

3.2.2 CR7 Data Acquisition System

The CR7 system was used to collect and store data from the KM-100B strain transducers and the thermocouples. The CR7 wiring panel is given in Figure 3.6 [9]. The CR7 system (Figure 3.7) consisted of a control module, I/O module, battery supply, and sensor ports [9]. The control module consisted of a display and keyboard for direct programming of the CR7. A 9-pin serial I/O interface was also on the control module for connecting to the SC32A interface. An I/O processor card, 16-bit analog interface card, and 7 open slots for any combination of I/O cards made up the I/O module. Personnel set up the two CR7’s in the ORITE laboratory at Ohio University. The bridge completion circuits, AM416 multiplexers, and the power supply were housed in an environmentally sealed box and connected to the CR7 through the sensor ports.
Figure 3.5 Geokon VCE-4200 Data Acquisition Circuit
Figure 3.6  CR7 Wiring Panel Programming Instructions
The wires for the female end of the KM-100B plug were connected to the multiplexers in the sealed box. The data acquisition circuit for the KM-100B strain transducers is given in Figure 3.8.

The multi-depth thermocouple lead wires were connected directly to the 723-T I/O card in the CR7. This card provided enough screw terminals to collect the temperatures of the 10 thermocouples. Two maturity meters were used to collect temperatures from the 8 single thermocouples placed in HP Section 1-slab #3 and HP Section 2-slab #1.

Figure 3.7 CR7 Measurement and Control System
Figure 3.8 KM-100B Data Acquisition Circuit
3.2.3 Environmental Testing Procedure

The PC208 software was specially designed by Campbell Scientific to program commands for the CR7 and CR10. These programs were written and checked in the ORITE laboratory at Ohio University. The data acquisition systems were taken to the project site after the programs were verified to be working properly. The Vibrating Wire Strain Gage field wires were then connected to the multiplexers in the CR10 box. The KM-100B plugs were attached to the plugs from the CR7 and the thermocouples were connected to the 723-T analog input card in the CR7. The programs were then downloaded through the SC32A interface to the CR7 and CR10 systems. The program was set up to collect and store data from the test slabs every 30 minutes. This data was collected at a regular interval to prevent the stored data from being overwritten. At this time, the researcher could stop the collection process, download a new program, or restart the same program. The data from the two systems was stored in ASCII comma delimited format. This data was then transported back to the ORITE laboratory for processing. The CR7 and CR10 systems collected data from the instrumented slabs beginning on October 16th, 1997 and commencing on November 20th, 1997.

3.3 DIPSTICK® 2000 TESTING

A data acquisition system capable of measuring changes in elevation to a very high degree of accuracy was needed to monitor the slab shape. Traditional surveying methods were not accurate enough to measure such small changes in elevation. Face Construction Technologies of Norfolk, Virginia manufactured the Dipstick® 2000 to
combat this problem. The Dipstick® 2000 was capable of collecting profile measurements at a much higher rate and accuracy than rod and level survey procedures. This instrument provided the ability to traverse the diagonal and edges of a 12’ wide by 21’ long slab in approximately 25 minutes.

3.3.1 Dipstick® 2000 Data Acquisition System

The Dipstick® 2000 was stored disassembled in a protective aluminum case. This case provided a safe and easy method for transporting the DS2000 to and from the project site. The components of the Dipstick® 2000 are shown in Figure 3.9 [10]. A description of each part is given in Table 3.2 [10].

<table>
<thead>
<tr>
<th>Item #</th>
<th>Description</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dipstick® unit with moonfeet attached</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Palmtop computer and clamp assembly</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>Handle assembly (2 pieces)</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>Tape measure</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>AC adapters for Dipstick® and Palmtop</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>Pointed feet</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>Allen wrenches, 1/8”, 3/16”, 1/4&quot;, and 5/32”</td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>Open-end wrench, 7/16” x 3/8”</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>Calibration Shim, 0.125”</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>Marking crayon, “keel”</td>
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</tr>
<tr>
<td>11</td>
<td>Alkaline batteries, 1.5V, AA size</td>
<td>4</td>
</tr>
<tr>
<td>12</td>
<td>Leg extensions for rough terrain</td>
<td>2</td>
</tr>
<tr>
<td>13</td>
<td>Variable foot spacer</td>
<td>1</td>
</tr>
<tr>
<td>14</td>
<td>Computer connector cable</td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>RoadFace™ data collection software disk</td>
<td>1</td>
</tr>
<tr>
<td>16</td>
<td>Dipstick® Road Profiler Manual</td>
<td>1</td>
</tr>
<tr>
<td>17</td>
<td>RoadFace™ software disk</td>
<td>1</td>
</tr>
<tr>
<td>18</td>
<td>RoadFace™ software manual</td>
<td>1</td>
</tr>
<tr>
<td>19</td>
<td>Palmtop computer manual</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>Cables, 1 serial and 1 parallel</td>
<td>2</td>
</tr>
<tr>
<td>21</td>
<td>9-25 Serial cable adapter</td>
<td>1</td>
</tr>
</tbody>
</table>
The Dipstick® 2000 Case

Figure 3.9 Dipstick® 2000 Components
The RoadFace™ data collection program software was provided by Face Construction Technologies on a PCMCIA memory card. This card was placed in the palmtop computer. The software provided the ability to view plots of the data collected by the DS2000 on the palmtop computer. This IBM compatible palmtop computer provided 1 megabyte of RAM, 1 megabyte of internal storage, and an external card capable of storing 1 megabyte of memory. Directories were created for each test slab and data was stored in the subdirectories for each traverse. The Dipstick® unit needed to be turned on to warm up for approximately 5 minutes. There were two handle sections for the DS2000. The lower handle housed a Ni-Cad battery and was connected to the hub on the Dipstick® unit. This battery would run for approximately 10 hours on a full charge. The computer clamp assembly was placed in between the two Dipstick® handles. This assembly held the palmtop computer in place while traversing a slab. The handle section with the ball and trigger was the top piece. Once all the cables were connected properly, the Dipstick® was ready to collect data.

A pendulum was placed inside the Dipstick® unit. The axis of the sensor inside the Dipstick® unit was arranged so it would be co-planar with the axis of the Dipstick’s feet. When the Dipstick® is pivoted from one leg to the other, the sensor becomes unbalanced and the LCD display on the unit becomes blank. When the two legs are in contact with the surface of the pavement and the sensor reaches a state of equilibrium, the elevation difference of the two feet is displayed. It is important not to apply any pressure to the handle while the unit is attempting to reach equilibrium. This unit was calibrated
at the factory for an accuracy of 0.001" per reading. Accumulated errors were adjusted for bias after the traverse was completed [11].

3.3.2 Dipstick® 2000 Testing Procedure

Paint was used to mark a rectangular box twenty feet long and eleven feet wide. The four sides of this box were monitored with the Dipstick® unit and changes in elevations were recorded. The eleven-foot lengths of the box were measured parallel to the approach transverse joint at six inches and twenty feet-six inches. The two twenty-foot lengths were measured six inches and eleven feet-six inches from the edge of the driving lane. A diagonal was also painted across the center of the slab.

The first step was to pivot the Dipstick® from the reference rod to the starting corner of the test slab and back to the reference rod. This procedure presented a closed traverse of which any error could be corrected. The Dipstick® was then pivoted clockwise around the 11’ × 20’ box and the diagonal between the two opposite corners. To initiate the data collection, the trigger on the handle was pushed. The start end of the Dipstick® must be pointed in the forward direction at this time. When the traverse was completed, the trigger was pushed and data collection commenced. This data was stored in the appropriate subdirectories of the palmtop computer attached to the handle. The data was then transferred via a link cable to a computer in the ORITE laboratory for processing.
CHAPTER 4
DATA ANALYSIS PROCEDURES, DATA PRESENTATION, AND DISCUSSION OF RESULTS

4.1 DATA ANALYSIS PROCEDURES

The majority of the data stored in the data acquisition systems was in the form of raw data. The formulas used to process this data into strains and temperatures are given in the following sections.

4.1.1 Geokon VCE-4200 Vibrating Wire Strain Gage

Strain and temperature changes were recorded by the VCE-4200 vibrating wire strain gages. The data required to compute strain was stored in the CR10 as resonant frequency squared. The temperature data was stored in degrees Celsius.

The change in apparent strain developed in the concrete can be expressed by the following formula [12]:

\[
\Delta \varepsilon_a = \Delta \varepsilon_i + \Delta \varepsilon_t
\]

(4.1)

where: \( \Delta \varepsilon_a \) = change in apparent concrete strain (\( \mu \varepsilon \))

\( \Delta \varepsilon_i \) = change in indicated concrete strain (\( \mu \varepsilon \))

\( \Delta \varepsilon_t \) = strain correction change for thermal expansion of gage (\( \mu \varepsilon \))
The change in indicated concrete strain can be derived from the frequency of vibration of the wire given in Equation 4.2 [12].

\[ f = \frac{101,142}{L_w} \sqrt{\varepsilon_w} \]  

(4.2)

where: \( f \) = frequency (cycles/second)  
\( L_w \) = length of the wire (in.) = 5.875 inches  
\( \varepsilon_w \) = wire strain (in./in.)

The deformation of the wire must be equal to the deformation of the gage. This relationship is given in Equation 4.3.

\[ \varepsilon_w L_w = \varepsilon L_g \]  

(4.3)

where: \( \varepsilon \) = strain in the gage (in./in.)  
\( L_g \) = length of the gage (in.) = 6 inches

Substituting Equation 4.3 into Equation 4.2 and solving for the strain in the gage yields the following:

\[ \varepsilon = 3.304 \times 10^{-9} (f^2) \]  

(4.4)

The CR10 measures the period, T, of the vibrating wire in milliseconds. Therefore the frequency squared term needs to be multiplied by \( 1 \times 10^6 \).

\[ \varepsilon = 3.304 \times 10^{-3} (f^2) \]  

(4.5)

Due to the input frequency range, the output from the CR10 was nine times larger.

Equation 4.6 can be substituted into Equation 4.5 to solve for the indicated strain (in./in.) in the concrete [8].
\[ f^2 = \left(\frac{1}{\sqrt{g}}\right)y \]  

(4.6)

where: \( y \) = output from the CR10

The change in indicated strain (\( \mu \varepsilon \)) measured in the concrete can be expressed by

Equation 4.7.

\[ \Delta \varepsilon_i = \left(\frac{3,304}{9}\right)[y_j - y_o] \]

(4.7)

where: \( \Delta \varepsilon_i \) = change in indicated strain (\( \mu \varepsilon \))

\( y_j \) = CR10 reading at time \( j \)

\( y_o \) = initial CR10 reading

The correction for thermal expansion of the gage wire is given in the following equation [12]:

\[ \Delta \varepsilon_t = C_w \ast (T_j - T_o) \]

(4.8)

where: \( \Delta \varepsilon_t \) = change in strain correction for thermal expansion of gage (\( \mu \varepsilon \))

\( C_w \) = coefficient of expansion of the steel wire = 12.2 \( \mu \varepsilon / ^\circ C \)

\( T_j \) = temperature at time \( j \) (\(^\circ C\))

\( T_o \) = initial temperature \(^\circ C\)

Substitution of Equations 4.7 and 4.8 into Equation 4.1 yields:

\[ \Delta \varepsilon_a = \left(\frac{3,304}{9}\right) \ast (y_j - y_o) + 12.2 \ast (T_j - T_o) \]

(4.9)

The CR10 readings were substituted into Equation 4.9 to obtain the apparent strains developed in the concrete test slabs.
Since the vibrating wire strain gage recorded relative temperatures, an initial thermocouple reading was required to calculate actual temperatures in the concrete at the vibrating wire locations. These actual temperatures were calculated from Equation 4.10.

\[ T_a = T_i + (T_j - T_o) \]  

(4.10)

where: 

- \( T_a \) = actual temperature of concrete
- \( T_i \) = initial temperature of thermocouple = 18.5°C
- \( T_j \) = temperature at time \( j \) (°C)
- \( T_o \) = initial temperature (°C)

### 4.1.2 TML KM-100B Strain Transducer

The KM-100B strain transducers were used in this project to measure both static and dynamic strain and temperature. The strain data collected by the Megadac from the FWD tests was not ready at this time. Therefore, only the environmental strains and temperatures will be discussed. The KM-100B utilizes a full bridge arrangement to measure strain. An excitation voltage is supplied across the Red and Black lead wires of the gage. The differential output voltage is measured across the Green and White lead wires. A full bridge arrangement is also used to measure temperature. An excitation voltage is supplied across the Red and Black lead wires and the differential output voltage is measured across the Red and Yellow wires. The wiring diagram for the KM-100B is given in Figure 4.1.

In order to calculate the change in strain in the gage, the internal change in temperature is needed. The CR7 stores the temperature output from the KM-100B as [9]:

- **CR7 stores the temperature output from the KM-100B as [9]:**
\[ y_j = 1000 \left( \frac{\Delta V}{V} \right) \]  

(4.11)

where: \( y_j \) = temperature voltage ratio reading of CR7 at time \( j \) (mV/V)

\( \Delta V \) = output voltage (V)

\( V \) = excitation voltage (V)

The thermal strain is related to this voltage ratio by the following equation [13]:

\[ \frac{\Delta V}{V} = \frac{1000 \times S_g \varepsilon_t}{4 + 2S_g \varepsilon_t} \]  

(4.12)

where: \( \Delta V/V \) = voltage ratio (mV/V) = \( y_j \)

\( S_g \) = common gage factor

\( \varepsilon_t \) = thermal strain (\( \mu \varepsilon \))

Substituting the CR7 reading into Equation 4.12 and solving for the thermal strain yields Equation 4.13.

\[ \varepsilon_t = \frac{4y_j}{S_g (1000 - 2y_j)} \]  

(4.13)

Equation 4.13 is then reduced and divided by \( 1 \times 10^{-6} \) so the thermal calibration coefficient, \( C_t \), can be used in the format \( ^\circ \text{C}/1 \times 10^{-6} \).

\[ \varepsilon_t = \left( \frac{1}{1 \times 10^{-6}} \right) \left( \frac{4}{2S_g} \right) \left[ \frac{y_j}{500 - y_j} \right] \]  

(4.14)

The internal changes in temperature for the KM-100B can be calculated from the following formula [14]:

\[ \Delta T = C_t \times (\varepsilon_{to} - \varepsilon_j) \]  

(4.15)
Substituting Equation 4.14 into Equation 4.15 yields the following:

\[ \Delta T = \left( \frac{C_t}{1 \times 10^{-6}} \right) \left( \frac{4}{2S_g} \right) \left( \frac{y_{ro}}{500 - y_{ro}} - \frac{y_{ij}}{500 - y_{ij}} \right) \]  \hspace{1cm} (4.16)

where: \( \Delta T = \) internal change in temperature at gage \(^\circ\)C

\( C_t = \) thermal calibration coefficient = 0.0176 \(^\circ\)C/1 \times 10^{-6}

\( y_{ro} = \) initial CR7 voltage ratio reading (mV/V)

\( y_{ij} = \) CR7 reading at time \( j \) (mV/V)

This \( \Delta T \) was required to calculate the apparent strain in the KM-100B. The actual temperature at the strain transducer positions can be calculated from Equation 4.17.

\[ T_a = T_i + \Delta T \]  \hspace{1cm} (4.17)

where: \( T_a = \) actual temperature of concrete \(^\circ\)C

\( T_i = \) initial temperature of thermocouple = 18.5\(^\circ\)C

The strain differential voltage ratio is stored in the CR7 as [9]:

\[ y_j = 1000 \times \left( \frac{\Delta V}{V} \right) \]  \hspace{1cm} (4.18)

where: \( y_j = \) CR7 strain voltage ratio (mV/V)

The indicated strain for a full bridge arrangement is given by Equation 4.19 [15].

\[ \varepsilon_i = \frac{1}{S_g} \left[ \frac{\Delta V_1}{V_1} + \frac{\Delta V_2}{V_2} + \frac{\Delta V_3}{V_3} + \frac{\Delta V_4}{V_4} \right] \]  \hspace{1cm} (4.19)

where: \( \varepsilon_i = \) indicated strain (in/in)

\( \Delta V_n = \) voltage remaining after gage \( n \)

\( V_n = \) voltage before gage \( n \)
When the temperature increases in the pavement, the voltage in each gage will also increase. The KM-100B is compensated for this change in voltage. When deformations occur in the pavement, the two sets of resistors will read opposite each other; \( \Delta V_1 = -\Delta V_2 = \Delta V_3 = -\Delta V_4 = \Delta V \), and \( V_1 = V_2 = V_3 = V_4 = V \). Substituting these values into Equation 4.19 yields indicated strain in the form of (in/in) [15].

\[
\varepsilon_i = \frac{4}{S_g} \left( \frac{\Delta V}{V} \right)
\]  

(4.20)

The strain voltage ratio from the CR7 can then be substituted to yield change in indicated strain in the form of (\( \mu \)in/in).

\[
\varepsilon_i = \frac{4000}{S_g} (y_j)
\]  

(4.21)

The indicated strain and the strain due to temperature fluctuations constitute the apparent strain in the concrete. Coefficients are needed for each of these terms. The formula for apparent strain is given in Equation 4.22 [13].

\[
\varepsilon_a = C_\varepsilon \varepsilon_i + C_\beta (\Delta T)
\]  

(4.22)

where: \( \varepsilon_a \) = apparent strain (\( \mu \)\( \varepsilon \))

\( C_\varepsilon \) = strain correction coefficient (\( \mu \)\( \varepsilon \) / \( \mu \)\( \varepsilon \))

\( C_\beta \) = temperature correction coefficient (\( 1 \times 10^{-6} \) / °C)

\( \Delta T \) = internal change in temperature at gage (°C)

Assuming a common gage factor of 2.0 and substituting Equation 4.21 into Equation 4.22 yields the change in apparent strain [13]:

\[
\Delta \varepsilon_a = 2000C_\varepsilon (y_j - y_o) + C_\beta (\Delta T)
\]  

(4.23)
Equation 4.23 was used to calculate the apparent strains in the rigid pavement. The manufacturers of the KM-100B strain transducers supplied the coefficient values. These values are listed in Table 4.1.

**Table 4.1 Correction Coefficients for TML KM-100B Strain Transducers**

<table>
<thead>
<tr>
<th>Location</th>
<th>Wire #</th>
<th>( C_\theta (1 \times 10^{-6} / ^\circ C) )</th>
<th>( C_e (\mu e / \mu e) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Bottom</td>
<td>7</td>
<td>9.6</td>
<td>0.814</td>
</tr>
<tr>
<td>1 Top</td>
<td>8</td>
<td>10.0</td>
<td>0.812</td>
</tr>
<tr>
<td>2 Bottom</td>
<td>9</td>
<td>10.3</td>
<td>0.820</td>
</tr>
<tr>
<td>2 Top</td>
<td>10</td>
<td>9.7</td>
<td>0.821</td>
</tr>
<tr>
<td>3 Bottom</td>
<td>11</td>
<td>9.5</td>
<td>0.816</td>
</tr>
<tr>
<td>3 Top</td>
<td>12</td>
<td>9.7</td>
<td>0.828</td>
</tr>
<tr>
<td>4 Bottom</td>
<td>13</td>
<td>10.0</td>
<td>0.825</td>
</tr>
<tr>
<td>4 Top</td>
<td>14</td>
<td>10.1</td>
<td>0.820</td>
</tr>
<tr>
<td>5 Bottom</td>
<td>15</td>
<td>9.8</td>
<td>0.826</td>
</tr>
<tr>
<td>5 Top</td>
<td>16</td>
<td>9.1</td>
<td>0.814</td>
</tr>
<tr>
<td>6 Bottom</td>
<td>17</td>
<td>9.6</td>
<td>0.818</td>
</tr>
<tr>
<td>6 Top</td>
<td>18</td>
<td>9.6</td>
<td>0.821</td>
</tr>
</tbody>
</table>

**Figure 4.1 KM-100B Wiring Diagram**
4.1.3 *Thermocouples*

Since the Maturity Meters store temperature in degrees Celsius, no further calculations were required to process this data. A program was downloaded to the CR7 to convert voltage readings from the thermocouples to temperatures in degrees Celsius. These temperatures were stored in the memory of the CR7.

4.1.4 *Dipstick® 2000 Elevations*

The palmtop computer attached to the Dipstick® handle recorded each Dipstick® measurement. A closed loop survey was used so any accumulated errors could be distributed equally among the total number of readings. The last value of the running sum of readings theoretically should be zero since the Dipstick® is traversed from same start and finish position. The final elevation was corrected by dividing the total accumulated error by the number of readings to find the error per reading. This value was then subtracted from each reading so the beginning and ending elevations were equal to zero. The start point for the closed loop survey was then adjusted to the elevation of the start point relative to the reference rod elevation. The 11’ by 20’ traverse readings were also adjusted from the reference rod elevation.

4.1.5 *FWD Deflections*

Seven geophones were positioned along the length of the slab at –12”, 0”, 12”, 18”, 24”, 36”, and 60” from the point of load application. Three different loading magnitudes were applied at each dynamic test location. The laptop computer used to operate the FWD stored the applied load, pressure, and corresponding geophone
deflections in milli-inches for each test location. No further processing of this data was required.

4.1.6 Raw Data Processing

All the instrumentation in this project was initialized when the concrete was placed through the test sections. This provided the ideal reference point for measurements of strain and temperature fluctuations induced in the test slabs. Absolute values of strain can be calculated since the slabs were monitored from the first day the concrete was placed.

Microsoft EXCEL '97 was the program used to process the raw data. The equations from section 4.1 of this thesis were programmed into the cells to produce values of strain, temperature, and elevation. Plots of these calculated values were made to represent the response of the gages during the curing process in the concrete. Deflection and slab elevation graphs were also plotted from the early test data.

4.2 DATA PRESENTATION

The data processed from the Vibrating Wire Strain Gages, KM-100B Strain Transducers, thermocouples, Dipstick® 2000 readings, and FWD deflections are given in Appendix B, Appendix C, Appendix D, Appendix E, and Appendix F, respectively. In Appendix B and Appendix C, the strain values were plotted on the primary y-axis, the temperatures were plotted on the secondary y-axis, and the time of data collection was plotted on the x-axis. The responses from the top and bottom gages were given on each graph to show how the strain and temperature varied with the concrete depth and the
location in the slab. The main title of the plots in Appendix B and Appendix C represent the section number, gage orientation and type, measured responses, and the gage location. The section number, slab number, and the location of the chair that the thermocouple was tied to were given in the main title for the plots in Appendix D. For the Dipstick\textsuperscript{®} 2000 plots, the section and slab numbers were indicated in the main title. The elevation changes were represented on the y-axis and the distance from the start point was given on the x-axis. The section number, location of applied load, and the testing date were included in the FWD deflection plots’ main titles. The vertical deflections in milli-inches were given on the y-axis and the locations of the geophones were represented on the x-axis.

Some data was lost during the data collection of the two high performance concrete sections. Two vibrating wire strain gages located 1 $\frac{1}{2}$-inches from the pavement surface were damaged during the paving process. One other VWSG did not collect for the first two hours of testing and therefore could not be adjusted from the same reference time as the other sensors. Due to time constraints, a previous program was used to collect thermocouple temperatures in the CR7. This program was set up to read only the reference temperature and six thermocouples. A new program was downloaded to the CR7 dataloggers two days after the pavement was placed.
4.3 DISCUSSION OF RESULTS

The plots in Appendices B through F represent the data collected during the October-1997 testing. A discussion of this data is given for the strain, temperature, slab shape elevations, and deflections measured in the test sections.

4.3.1 Strain Discussion

Strain plots are presented in Appendices B and C for the Vibrating Wire Strain Gages and the KM-100B Strain Transducers, respectively. Stresses were not calculated from these strains because the concrete’s modulus of elasticity was constantly changing during curing. The electrical noise present in the dynamic strain data was not filtered at this time and therefore could not be presented in this thesis. The Vibrating Wire Strain Gages were oriented in the test slabs to monitor both longitudinal and transverse strains. The longitudinal strains in the concrete slab are represented at locations 1 through 9. Locations 10 through 14 represent the transverse strains developed in the slab during the curing process. Both sets of strains followed a pattern similar to the temperature fluctuations at the corresponding depths in the concrete. During the first 24 hours, the maximum magnitudes of strain were slightly larger for the gages oriented along the transverse direction. Figure 4.2 represents a strain comparison between longitudinal and transverse gages located in the same positions for the first 24 hours. Locations 6 and 13 were positioned ten feet – six inches from the approach end of the slab along the centerline of the driving lane in slab numbers 2 and 3, respectively. These locations were chosen because the temperature differences between locations 6 and 13 were very small (<0.3 °C). Although the data from the KM-100B strain transducers was affected by
electrical noise, the trend of the strains could still be analyzed. The maximum positive and negative values of strain for all three gages occurred at approximately the same time and seemed to follow the temperature trends.

4.3.2 Temperature Discussion

The temperature data collected during the first 96 hours of testing is given in Appendixes B, C, and D. The trends of the temperature data collected from the vibrating wire, KM-100B, and thermocouples all followed the same pattern. The overall downward trend of the temperature data is representative of the decreasing daily air temperature during data collection. The minimum daily concrete temperatures at 1 ½-inches below the pavement surface generally occurred between 7:00 a.m. and 9:00 a.m. and the maximum daily temperatures occurred between 3:00 p.m. and 5:00 p.m. The temperatures measured 8 ½-inches below the surface of the pavement generally reached their peaks 2 to 4 hours later than the temperatures measured at 1 ½-inches below the pavement surface. The maximum positive and negative temperature differentials were noted to occur during the daytime hours. The maximum temperatures recorded at 8 ½-inches below the pavement surface occurred 9 to 10 hours after the pavement was placed. The temperature data recorded during the first 24 hours was used to represent the curing of the concrete. After 24 hours, the temperature trends followed the everyday heating and cooling air temperature trends.

Figure 4.3 represents temperature data collected from the same location in two different slabs. The temperature in degrees Celsius is on the y-axis and the time of data collection is on the x-axis. Vibrating wire location 6 represents a longitudinal gage in the
center of slab #2 and vibrating wire location 13 represents a transverse gage in the center of slab #3. The temperature differences between gages positioned in the same locations were very small (< 0.3 °C). The small temperature differences are a testament to the accuracy capabilities of the vibrating wire strain gage. The temperatures recorded near the center of the two-twelve feet wide lanes were generally warmer than the temperatures near the pavement edge. A comparison of temperatures at two different locations is given in Figure 4.4. Vibrating wire location 3 was positioned 10 ½-feet from the approach end of the slab in the left-wheelpath and location 7 was 1 ½-feet from the approach end in the right-wheelpath. The temperatures at location 3 were warmer than the recorded temperatures at location 7 during the first 24 hours. The combination of losing heat through the pavement surface and driving lane edge probably caused lower temperatures near the edge. The cold nighttime temperatures affected location 7 more than location 3 due to the closer edge distance at location 7 (less concrete for insulation). During curing, colder temperatures generally were recorded by the sensors in the concrete towards the pavement edge.
High Performance Section 1 - Curing Strain Comparison
Vibrating Wire Location 6 and Location 13

Figure 4.2 Strain Comparison of Longitudinal and Transverse Vibrating Wire Strain Gages
High Performance Section 1 - Curing Temperature Comparison
Vibrating Wire Location 6 and Location 13

Figure 4.3 Curing Temperature Comparison, HP Section 1, VW Location 6 and 13
High Performance Section 1 - Curing Temperature Comparison
Vibrating Wire Location 3 and Location 7

Figure 4.4 Curing Temperature Comparison, HP Section 1, VW Locations 3 and 7
The dashed line in Figure 4.5 represents the temperature difference between the top gages at locations 3 and 7. The solid line represents the temperature difference between the bottom gages at these locations. Generally, a larger temperature difference between the gages at 1 ½-inches below the pavement surface was recorded. This shows that the top gages were affected more by the cold air temperature than the gages positioned closer to the base. The combined effects of the cold air temperatures and less concrete insulation near the edges attributed larger temperature differences at the top locations. A difference of approximately 1.5 °C between the two top locations was present at the time of maximum curing temperature. The maximum temperature difference at 8 ½-inches below the pavement surface was around 0.9 °C.

The solid line in Figure 4.6 represents the temperature difference between the top and bottom gages at location 3. The dashed line represents the temperature difference at these positions near the edge of the pavement. The concrete at 8 ½-inches below the pavement surface was approximately 8 °C warmer than the concrete at 1 ½-inches below the surface. This shows that the concrete cured in the bottom half of the slab before curing was reached in the top half of the slab. The temperature difference between the top and bottom gages near the pavement edge (location 7) was larger due to the combined effects of the cold air temperature and the smaller edge distance.
High Performance Section 1 - Curing Temperature Difference
Vibrating Wire Location 3 and Location 7

Figure 4.5 Curing Temperature Difference (Same Elevation), HP Section 1, VW Location 3 and 7
High Performance Section 1 - Curing Temperature Difference
Vibrating Wire Location 3 and Location 7

Figure 4.6 Curing Temperature Difference (Same Location), HP Section 1, VW Locations 3 and 7
The cold air temperature trend during the evening of October 16th and early morning of October 17th can be seen in Figure D.1. The air temperature rises for approximately two hours and then drops sharply for the next sixteen hours to 2 °C. The presence of the GGBFS combined with the cold air temperatures caused the concrete in the bottom half of the slab to cure before the concrete in the top. This prevented the contractors from cutting the expansion joints on time, thus allowing cracks to form on the pavement surface. The temperature data collected from the Vibrating Wire and KM-100B strain gages appears to represent this phenomenon. The temperatures monitored at 8 1/2-inches below the surface of the pavement generally increased for the first nine hours. This increasing temperature trend does not follow the air temperature trend thus representing the generation of heat from the hydration of the cement. This shows that the concrete in the bottom half of the slab was curing during the first nine hours. The downward temperature and strain trend represented by the gages at 1 1/2-inches below the surface of the pavement seemed to follow the air temperature trend. The slight rise in temperature at approximately nine hours could possibly represent an attempt by the concrete in the upper half of the slab to cure. The continuing drop in temperature eventually caused the temperature in the top half of the slab to fall thus delaying the time of setting. The curing of the slab from the bottom-up caused cracks to form in the pavement surface. The delayed time of setting at cold temperatures for concrete mixtures containing ground granulated blast furnace slag is well represented by the data collected from this project.
4.3.3 **Slab Shape Elevations**

The laptop computer attached to the handle recorded the changes in elevations between the footpads of the Dipstick®. These values were adjusted to represent changes in elevations from the reference rod elevation. Plots were made from this data are presented in Appendix E.

Five plots are given for each slab number. The first plot represents the complete traverse of the slab. Plots 2 through 5 represent the changes in elevations along each length of the slab. A drawing with the typical slab dimensions is included on every plot. The design slope of the pavement is well represented by the two transverse lengths of the slab. In general, the elevations along the longitudinal lengths seem to show that the slab was raising relative to the reference rod. This could actually represent the collection of loose aggregate on the footpads of the Dipstick®. The changes in elevations of the slab caused by curling and warping have not been determined from the Dipstick® data at this time.

4.3.4 **FWD Vertical Deflections**

The vertical deflections measured by the geophones are represented with plots in Appendix F. These deflections were measured at distances in the direction of traffic at –12”, 0”, 12”, 18”, 24”, 36”, and 60” from the applied load. The Falling Weight Deflectometer was only used over top of the dynamic strain gage locations. These plots clearly represent an increase in deflection with larger applied loads, as would be expected. In general, larger vertical deflections were measured near the locations of the transverse joints and the shoulder edge of pavement. The larger deflections represent an
area of less contact (gap) between the pavement base and the surface of the DGAB layer. Figures F.13 and F.14 indicate this increase in deflection as the location of the applied load is moved towards the edge of pavement.
CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The procedures used to instrument the concrete slabs have produced excellent early pavement response data. The strain gage stands provided an unrestrained condition, therefore allowing the gages to respond to changes in the pavement. Of the two types of strain gages used to monitor the environmental changes, the Vibrating Wire strain gages seemed to produce the clearest data. The strain trends of all the gages seemed to follow the temperature trends. Generally, the strains measured by the transverse vibrating wire gages were larger than the longitudinal gages. Although electrical noise was evident in the KM-100B data during the environmental testing, the strain and temperature trends were still recognizable. The KM-100B strain gages also offer the ability to monitor strains during dynamic loading. Stresses were not presented in this thesis because the modulus of elasticity was continuously changing. All of the Micro Measurement and KM-100B strain gages survived and should produce excellent dynamic data in future tests.

The temperature data seemed to represent the problems associated with placing concrete containing ground granulated blast furnace slag in cold weather. Concrete temperatures at 1 ½-inches and 8 ½-inches below the surface of the pavement were generally colder near the edge of the driving lane. A delayed time of setting was clearly
monitored by the sensors in the pavement during the first 24 hours. During curing, the concrete temperature at 8 ½-inches below the pavement surface was approximately 8 degrees Celsius warmer than the temperatures at 1 ½-inches below the pavement surface. The warmer concrete temperature generated at the lower gage elevation showed that curing occurred first at this position in slab depth. The contractors were unable to cut expansion joints because the curing of the pavement surface was delayed. This allowed cracks to form and eventually led to the replacement of some concrete sections near the test site. This phenomenon was well represented by the temperature data presented in this thesis.

The data collected from the Dipstick\textsuperscript{®} 2000 seemed to represent the design slope of the pavement very accurately but changes in elevations due to curling and warping were not as clear. The curing problems monitored by the strain gages may have introduced problems during the collection of the Dipstick\textsuperscript{®} 2000 data. The vertical deflections recorded from the FWD testing were generally larger at the locations near the transverse joints and edges of the test slab. The larger deflections at these locations represent a gap between the bottom of the pavement and the aggregate base.

The data collected during the time these sections were monitored produced significant levels of strain in the concrete and also presented answers to curing problems present in concrete mixes containing GGBFS. To improve the pavement’s durability, the magnitudes of strain monitored by the gages and the curing problems identified in this project should be incorporated into future pavement designs.
5.2 RECOMMENDATIONS

For future research projects, the use of LVDT’s should be incorporated into the section layouts to monitor the deflections caused by the curling and warping phenomenon. Based on the early environmental strain and curing problems presented in this thesis, the placement of concrete containing ground granulated blast furnace slag should be avoided during cold air temperatures.

A third high performance test section and a normal concrete test section will be instrumented on U.S. Route 50 in the near future. A comparison of the two sections in this thesis with the future data from the new test sections should provide valuable information on the effects of substituting ground granulated blast furnace slag for cement in the concrete mix design. These tests could help determine if it is feasible to use such an admixture in pavement design.
LIST OF REFERENCES


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Figure A.5 Location of Micro Measurement Strain Gages, HP Section 2, Slab #1
Figure A.6 Location of KM-100B & Vibrating Wire Strain Gages, HP Section 2, Slab #2
Figure A.7 Location of KM-100B & Vibrating Wire Strain Gages, HP Section 2, Slab #3
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HP Section 2, Transverse Vibrating Wire Strain and Temperature, Location 13

Oct. 16, 1997
2:00 PM

Oct. 20, 1997
2:00 PM
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TML KM-100B Strain Transducer Data
Figure C.1 KM-100B Strain and Temperature Plot, HP Section 1, Location 1

KM-100B Strain and Temperature, Location 1

Strain

Temperature (deg C)

HP Section 1, 45° Angle

Oct. 16, 1997
2:30 PM

Oct. 18, 1997
2:30 PM

Time (hrs)

Strain (µε)

--- Strain (Top)
--- Temp. (Top)
HP Section 1, 45° Angle
KM-100B Strain and Temperature, Location 3

Figure C.3 KM-100B Strain and Temperature Plot, HP Section 1, Location 3
Figure C.4 KM-100B Strain and Temperature Plot, HP Section 2, Location 4
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KM-100B Strain and Temperature, Location 5

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Thermocouple Temperature
Micro Measurement Location 6

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Dipstick® 2000 Data
Slab Shape
HP Section 1, Slab #1

Distance From Start Point (ft)

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Slab Shape
HP Section 1, Slab #2

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Slab Shape
HP Section 1, Slab #2

Figure E.10 Slab Shape Plot, HP Section 1, Slab #2
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Slab Shape
HP Section 1, Slab #3

Figure E.13 Slab Shape Plot, HP Section 1, Slab #3
Slab Shape
HP Section 1, Slab #3

Distance From Start Point (ft)

Elevation (m)

-2.400
-2.200
-2.000
-1.800
-1.600
-1.400
-1.200
-1.000
-0.800
-0.600
-0.400
-0.200
0.000
0.200

10/17/97
10/19/97
10/21/97
10/23/97
10/30/97

Start Point
Traffic

Figure E.14 Slab Shape Plot, HP Section 1, Slab #3
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Slab Shape
HP Section 2, Slab #1

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Slab Shape
HP Section 2, Slab #2

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Slab Shape
HP Section 2, Slab #2

Figure E.24 Slab Shape Plot, HP Section 2, Slab #2
Slab Shape
HP Section 2, Slab #2

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Slab Shape
HP Section 2, Slab #3

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Figure F.2 Vertical Deflection Plot, HP Section 1, Micro Measurement Location 5
Figure F.3 Vertical Deflection Plot, HP Section 1, Micro Measurement Location 6

Distance from Applied Load (in.)

Deflection (1 x 10^3 in.)

- 95.5 psi
- 117.4 psi
- 161.4 psi

HP Section 1, Micro Measurement Location 6
FWD Vertical Deflections, 12-3-97
Figure F.4 Vertical Deflection Plot, HP Section 1, Micro Measurement Location 7

Distance from Applied Load (in.)

-97.2 psi
-119 psi
-164.1 psi

Distance (1 x 0.3 in.)

-12 -6 0 6 12 18 24 30 36 42 48 54 60 66
HP Section 1, Micro Measurement Location 8
FWD Vertical Deflections, 12-3-97

Figure F.5 Vertical Deflection Plot, HP Section 1, Micro Measurement Location 8
Figure F.6: Vertical Deflection Plot, HP Section 1, Micro Measurement Location 9

Distance from Applied Load (in.)

Deflection (1 x 10^3 in.)

- 95.2 psi
- 118 psi
- 164.1 psi
Figure F.7 Vertical Deflection Plot, HP Section 1, KM-100B Location 1
Figure F.8 Vertical Deflection Plot, HP Section 1, KM-100B Location 2

FWD Vertical Deflections, 12-3-97

Distance from Applied Load (in.)

Deflection (1 x 10^-3 in.)

- 96.8 psi
- 119.6 psi
- 165.7 psi

HP Section 1, KM-100B Location 2
Figure F.9  Vertical Deflection Plot, HP Section 1, KM-100B Location 3
Figure F.10  Vertical Deflection Plot, HP Section 2, Micro Measurement Location 4
Figure F.11 Vertical Deflection Plot, HP Section 2, Micro Measurement Location 5

Distance from Applied Load (in.)

Distance from Applied Load (in.)

- 90.7 psi
- 112.5 psi
- 156.5 psi
Figure F.12 Vertical Deflection Plot, HP Section 2, Micro Measurement Location 6
Figure F.13  Vertical Deflection Plot, HP Section 2, Micro Measurement Location 7
Figure F.14 Vertical Deflection Plot, HP Section 2, Micro Measurement Location 7 (Offset)
Figure F.15 Vertical Deflection Plot, HP Section 2, Micro Measurement Location 8
Figure F.16 Vertical Deflection Plot, HP Section 2, Micro Measurement Location 9

Distance from Applied Load (in.)

Deflection (1 x 10^3 in.)

- 92.6 psi
- 116.8 psi
- 132.3 psi
Figure F.17: Vertical Deflection Plot, HP Section 2, KM-100B Location 4

Distance from Applied Load (in.)

Deflection (1 X 10^3 in.)

- 93.8 psi
- 116.8 psi
- 168.8 psi
Figure F.18 Vertical Deflection Plot, HP Section 2, KM-100B Location 5

Distance from Applied Load (in.)

(Deflection (\( \times 10^{-3} \) in.))
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