

Evaluation of Rigid Pavement Rehabilitation Methods on I-86

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Abstract

Experimental sections on Interstate 86 (I-86) in New York State were constructed and instrumented to evaluate the effectiveness of using various fracturing treatments applied to existing pavement before constructing unbonded concrete overlays. The treatments were untreated pavement, rubbilized pavement, and cracked and sealed pavement. In each treatment, four slabs were instrumented with Linear Variable Differential Transformers (LVDTs), Vibrating Wire (VW) strain gauges, and Thermocouples (TCs) to collect long-term environmental data, and micromerement strain gauges to collect dynamic response data. A Weigh-in-Motion (WIM) unit and a weather station provided supplemental data on traffic and environmental conditions. Environmental data were recorded hourly and used to determine thermal effects on slab shape, strain, and stress. Dynamic response data were collected during nondestructive testing with falling weight deflectometers (FWD) in July 2006, November 2006, and March 2007.

A comparison of the environmental effects on the slabs over the three treatments suggests that the untreated section experiences the largest strains and stresses, and is thus more likely to be subject to future distress in the form of top-down cracking. The dynamic response data were comparable for all sections, with the strains the least in the untreated section. However, the maximum dynamic strains were generally much lower than the maximum environmental strains, suggesting that the environmental factors will be more important in determining the relative longevity of the pavement over each treatment.

Introduction

In the United States, the interstate highway system is continuously undergoing maintenance, repair, and reconstruction. When complete reconstruction of a highway is unnecessary, other various rehabilitation techniques are used to renew pavements more economically. Among other factors, the condition of the distressed pavement, the traffic demands, and the environment often dictate the most appropriate rehabilitation treatment for a given roadway. An experimental section on Interstate 86 (I-86) in New York State was constructed to evaluate the effectiveness of using various fracturing treatments before constructing unbonded concrete overlays. This

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The rehabilitation of I-86 in New York State’s southern tier (formerly NYS Route 17) is divided into four projects, totaling about 26km of highway. The experimental concrete overlay sections lie in Cattaraugus County between the towns of Hinsdale and Cuba – east of exit 27 and west of exit 28. Aside from the experimental sections and bridges, the project consists of rubblization and resurfacing of the highway with an HMA overlay. I-86 was originally constructed in 1972 with 229 mm of jointed reinforced concrete pavement (JRC) (Swart, 2006). After over 30 years of service, major distresses were present in the concrete, and rehabilitation was necessary. While most of the project was rehabilitated by rubblizing the existing JRC and overlaying it with 205mm Hot Mix Asphalt (HMA), a 2.54 km stretch of I-86 was divided into three experimental concrete sections. In the first section the existing pavement was left untreated, in the second section the pavement was rubblized, and in the third it was cracked and sealed (C/S). The first section runs 820m long and the other two are each 860m long. A typical cross section of the concrete overlay system is presented in Figure 1, and a detail of the driving lane is shown in Figure 2.

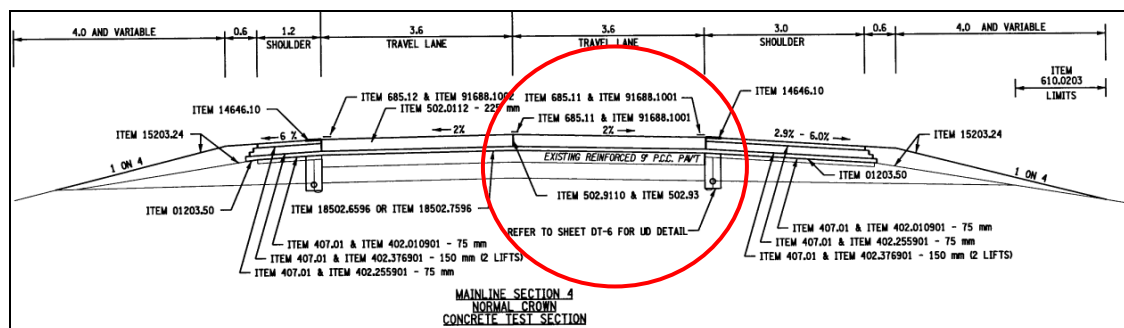


Figure 1 – Cross section of typical concrete overlay section

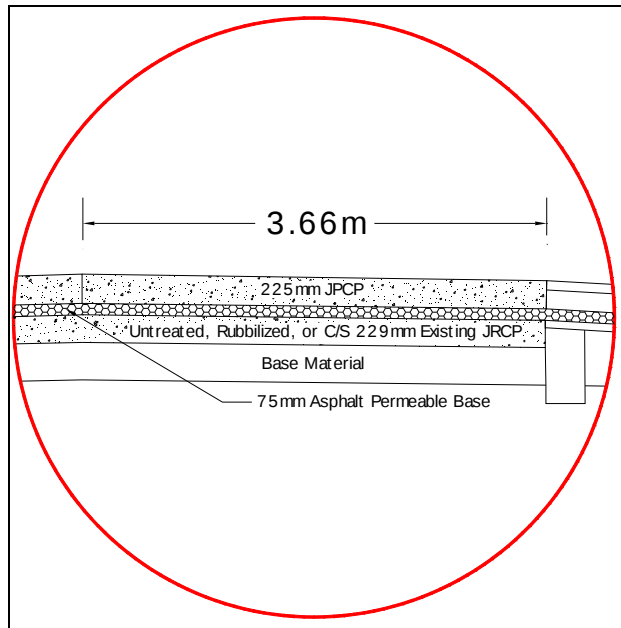


Figure 2 – Detail of overlay system in driving lane

The concrete overlay system consists of a 75mm separation layer of asphalt-treated base on top of the existing pavement, topped by 225mm of jointed plain concrete pavement (JPCP). Slabs are 4.75m long, 3.66m wide, with doweled transverse joints and tie bars in the longitudinal joints. In each section, four slabs in the driving lane were instrumented to measure strain, temperature, vertical displacement, and subgrade soil moisture. A Weigh in Motion (WIM) detector was installed to record traffic data, and a weather station monitored air temperature, precipitation, relative humidity, wind speed and direction, and solar radiation.

Background Information. Before a proper rehabilitation technique can be chosen, a careful assessment of surface distresses, structural adequacy, and functional conditions must be performed. Some common reconditioning techniques include diamond grinding, joint sealing, dowel bar retrofitting, subgrade repair or replacement, partial removal and replacement, bituminous overlay, bonded portland cement concrete (PCC) overlay, unbonded PCC overlay, and complete removal and reconstruction.

In this project, two different fracturing techniques were applied to the existing JRCP before the overlays were constructed: cracking & seating, and rubbilization. One test section was left untreated, and serves as the control section for the experiment. The objective of using cracking & seating, and rubbilization fracturing techniques were to reduce the Modulus of the base to increase the load carrying capacity of the unbonded PCC overlay. For C&S the modulus was less than 3450 MPa and for the Rubbilized layer less than 1034 MPa.

Practical Significance of Research

This research examined the differences in behavior between rubbilized, cracked and seated, and untreated JRCP bases with an unbonded concrete overlay and presents

recommendations for design of unbonded jointed plain concrete pavements. The recommendations are based on in-situ instrumentation, mechanical responses of the pavement in non-destructive testing, field results and analyses of use for concrete pavement designers.

Instrumentation

On the I-86 experimental project, three experimental JPCP overlay sections were constructed. In one section the existing pavement structure was untreated (a conventional unbonded overlay), in another section it was rubbilized, while in a third section the existing JRCF was cracked & seated. In each experimental section four concrete slabs were instrumented with embedded sensors. The typical instrumentation plan is presented in Figure 3 which shows the alphabetical labels of each sensor location. The plan is also shown with dimension details in Figure 4.

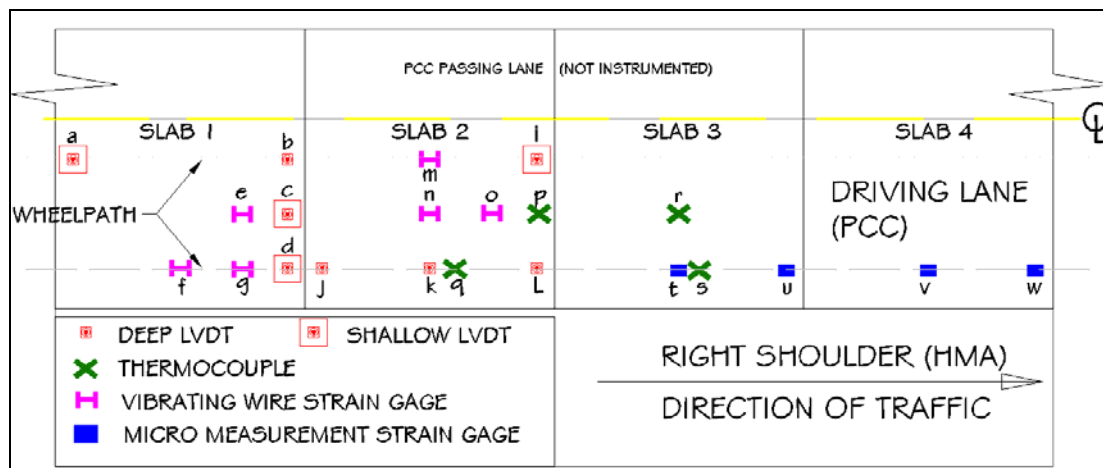


Figure 3 – Typical PCC instrumentation plan. The lower case letters indicate the locations of sensors embedded in the pavement.

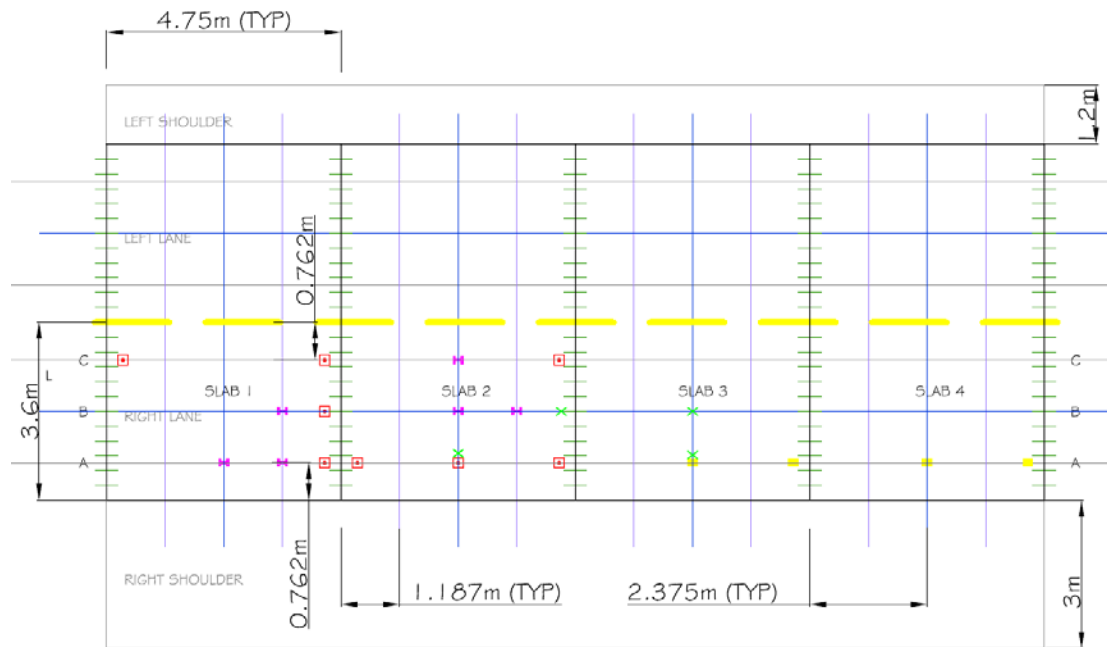


Figure 4 – Dimensioned instrumentation plan

Sensor Types. The types of sensors used in the study include:

1. Honeywell Sensotec S3C Linear Variable Differential Transformers (LVDTs) to measure the displacement of slabs;
2. Geokon Model VCE-4200 Vibrating Wire Strain Gauges (VW);
3. Vishay EGP-5-120 Micro-Measurement Strain Gauges (MM); and
4. Thermocouples (TC).

In this project, depending on the location of the LVDT in the slab, the reference plates were installed differently. In each section, four LVDTs are deep reference, and four are shallow reference LVDTs, shown in Figure 5. For a deep reference LVDT, the reference plates were mounted atop 3.6 m long rods, anchored deep in the subsurface soils. With a deep reference LVDT, the movement of the pavement is measured with respect to a point beyond the zone of influence from a load, so as to obtain an absolute deflection of the entire pavement-subgrade system. Shallow reference LVDTs measure the deflection of the pavement and overlay system with respect to the subgrade. The reference plates for the shallow reference LVDTs were installed at the interface between the subgrade and the “treated” pre-existing PCC pavement.

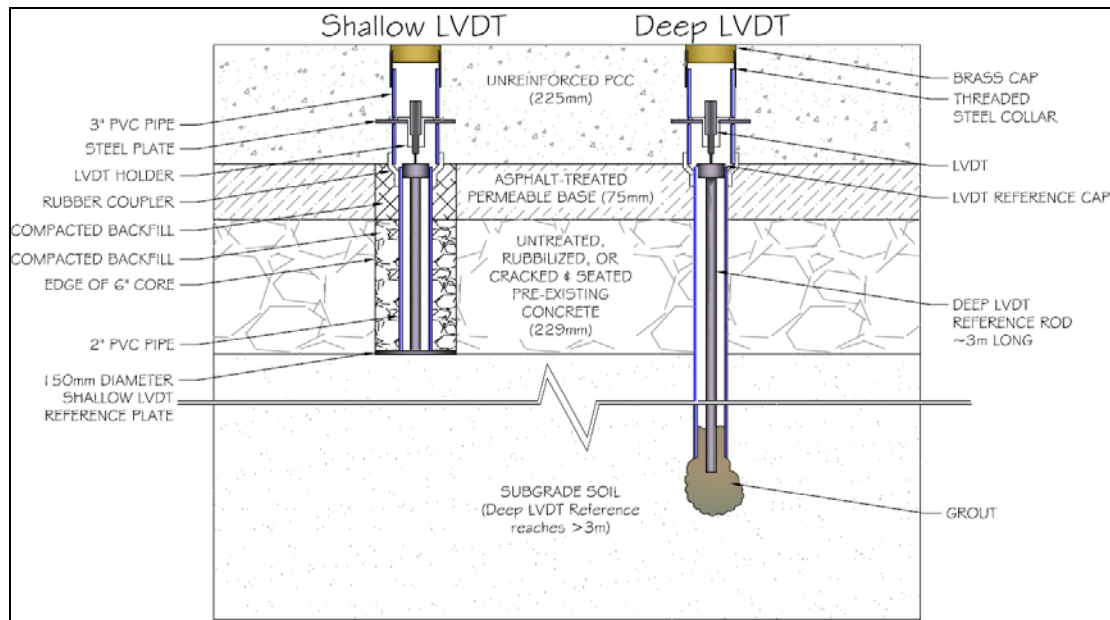


Figure 5 – Schematic of shallow and deep LVDT reference rods

Geokon Model VCE-4200 vibrating wire strain gauges were used in this study to measure long-term strains in the concrete caused by environmental effects, temperature, and moisture conditions. VW strain gauges provide long-term, stable strain readings and are directly embedded into the concrete slab. Vishay EGP-5-120 Micro-Measurement strain gauges were used to measure dynamic mechanical strains in response to Falling Weight Deflectometer (FWD) testing and other applied loads.

Pavement temperature was measured at four depths throughout the thickness of the concrete with a profile probe consisting of four T-type thermocouples, spaced 55mm apart on a 190mm rod in order to measure a temperature profile through the depth of the slab.

Types of Data Collected. Data collected on this project may be classified into two main categories: long-term or environmental data and dynamic response data. Long-term data are quasi-static quantities that were recorded periodically (typically hourly) over several months. These data are typically responses to environmental stimuli. Examples of long-term data are strains in the concrete due to temperature effects, and climatic data like temperature and precipitation. Dynamic data are measurements collected over short periods of time (several seconds at a time), at a very high sampling rate, during impact load testing like falling weight deflectometer (FWD) testing. Collected environmental data included slab deflection (LVDTs), environmental strain (VW strain gauges), pavement temperature (TCs), and climatic information from the weather station. LVDTs were used for both long-term and dynamic data acquisition. The MM gauges measured dynamic strains only.

Long-term Data Collection. Data collection from LVDTs, TCs and VWs began one hour before concrete placement in each section, with data recorded at 30 minute intervals for first three days, and then at 1 hour intervals thereafter. Paving occurred

on June 6th 2006 in the cracked & sealed (C/S) section and on June 8th 2006 in the rubbilized and untreated sections. Hourly readings of slab deflections, strains at the top and bottom of each slab, and slab temperatures were used since these are not rapidly changing quantities. Hourly readings provided sufficient information to deduce the shapes of warped/curled slabs, and environmentally induced stresses. All climatic data from the weather station were collected hourly beginning May 3rd 2006. Data collection was occasionally interrupted during periods when batteries were depleted before scheduled site visits.

Dynamic Response Data Collection During Nondestructive Testing. Nondestructive testing using a Falling Weight Deflectometer (FWD) was performed three times. The first round of testing occurred when the concrete was two weeks old, on June 20-21, 2006, using a KUAB 120 SP1G model FWD. A Dynatest 8000 FWD, depicted in Figure 6, was used on subsequent testing on November 28th – 29th 2006 and March 20th – 21st 2007.



Figure 6 – Dynatest FWD device in March 2007

The deflections read by the geophones on the FWD were used to calculate load transfer efficiencies (LTE), spreadability ratios, and normalized displacements at the joints, and deflection spreadabilities normalized displacements at midslab. In June, nearly all concrete slabs were tested at the joints, but no midslab drop sequences were performed. In November and March not all slabs were tested due to time constraints; only 10 to 12 slabs in each section were tested in November, and 20 to 24 slabs were tested in each section in March, but midslab drops were performed on all instrumented slabs. Since the forces of each drop were recorded by the FWD device, the responses measured by the embedded LVDTs and MM gauges on the instrumented slabs were matched with the loads. Knowing the loads that caused the measured deflections and strains allowed for normalized deflections and normalized strains in the slabs to be determined and for the displacement readings by the FWD geophones and the LVDTs to be compared. Comparing dynamic response strains and deflections from gauges in similar locations in different experimental sections

allowed for conclusions to be drawn about support conditions, structural integrity, and ultimately about predicted failure modes in the sections with different treatments.

Compressive and Flexural Strength of Field-Cured Cylinders and Beams

Cylinders were cast during construction to obtain representative samples of the product being placed. Cylinders were prepared in 4 and 6 inch standard molds according to ASTM C 192, and cured in a lime bath near the experimental sections. Since the cylinder specimens could not be transported to Ohio University before 28 days, SJB Inc., of Hamburg NY, was hired to perform compressive and flexural strength testing on the field-cured cylinders. Specimens were picked up from the site at 7 days, 14 days, 28 days, and 90 days, and transported according to FHWA standards to SJB's laboratory. Additionally, cylinders were transported to the Ohio University materials testing laboratory at an age of 30 days for determination of static modulus, Poisson's ratio, unit weight, and verification of ultimate strength.

The cylinders achieved strengths of 31 MPa by 28 days, and increased to 33 MPa by 90 days. The design strength for the mix was 27.58 MPa, which was clearly met. The laboratory testing conducted at Ohio University determined a modulus of elasticity value of 23.24 GPa. This value was used for the calculation of developed stresses after the ultimate strength of the concrete was achieved. An average Poisson's ratio of 0.181, a unit weight of 2315 kg/m³, and the 28 day compressive strength was determined to be 27.9 MPa. The ultimate compressive strength determined in this testing met the design requirement, however was somewhat lower than the strengths determined by SJB.

SJB Services, Inc. was also contracted to perform flexural testing of the field-cured beams. Beams were tested at ages of 7, 14, 28, and 90 days. A peak modulus of rupture value of about 3800kPa occurred at 28 days. Flexural strengths achieved by the field-cured beam specimens did not approach the design specification of 4826kPa. The flexural strength peaked at about 3792kPa at 28 days and then decreased slightly to 3620kPa at 90 days. These results may be cause for concern, as low tensile strength in the concrete may initiate the first major distresses that appear in the experimental concrete sections of I-86.

Curing Strains

Environmental conditions during the first hours after concrete placement can dictate early strain development. Plots of the strain developed in the first 48 hours after paving for each test section are presented in Figure 7, Figure 8, and Figure 9. The ambient air temperature is plotted on the secondary Y-axis so that the general environmental conditions during initial curing can be observed. Negative strain values represent compressive strains, and positive strain values represent tensile strains.

Paving of the C/S section occurred at around 12:00pm on June 6th 2006, and the untreated and rubblized sections were paved on June 8th 2006 at around 6:20pm and 2:20 pm, respectively. The strains were zeroed at the earliest time the readings became stable.

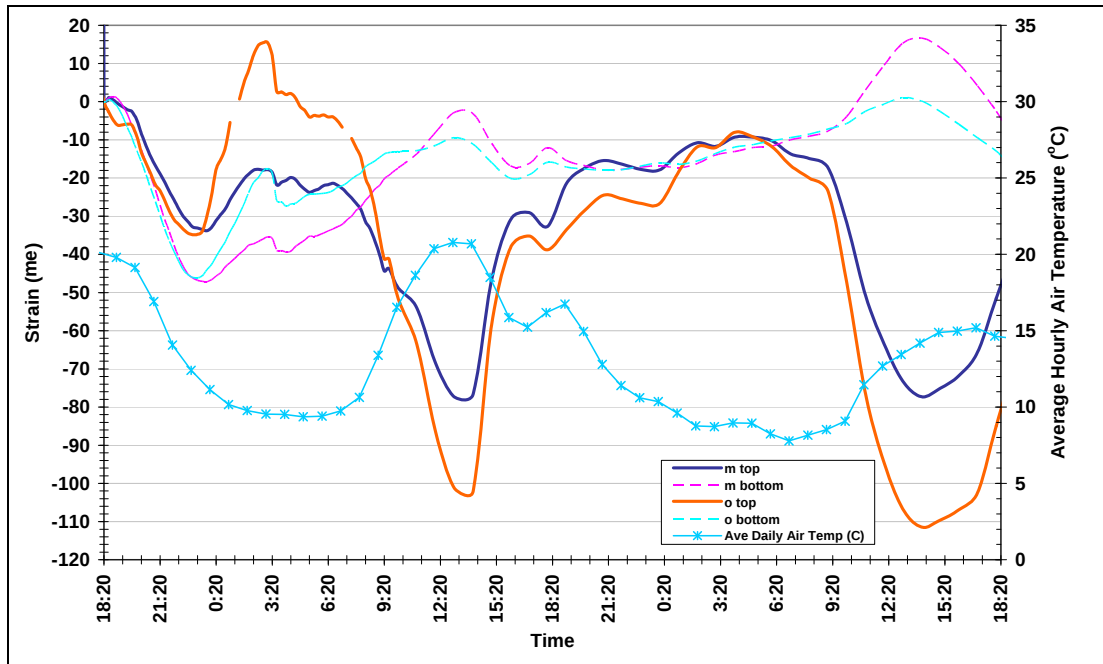


Figure 7 – Early strain development in the untreated section at sensor locations m and o

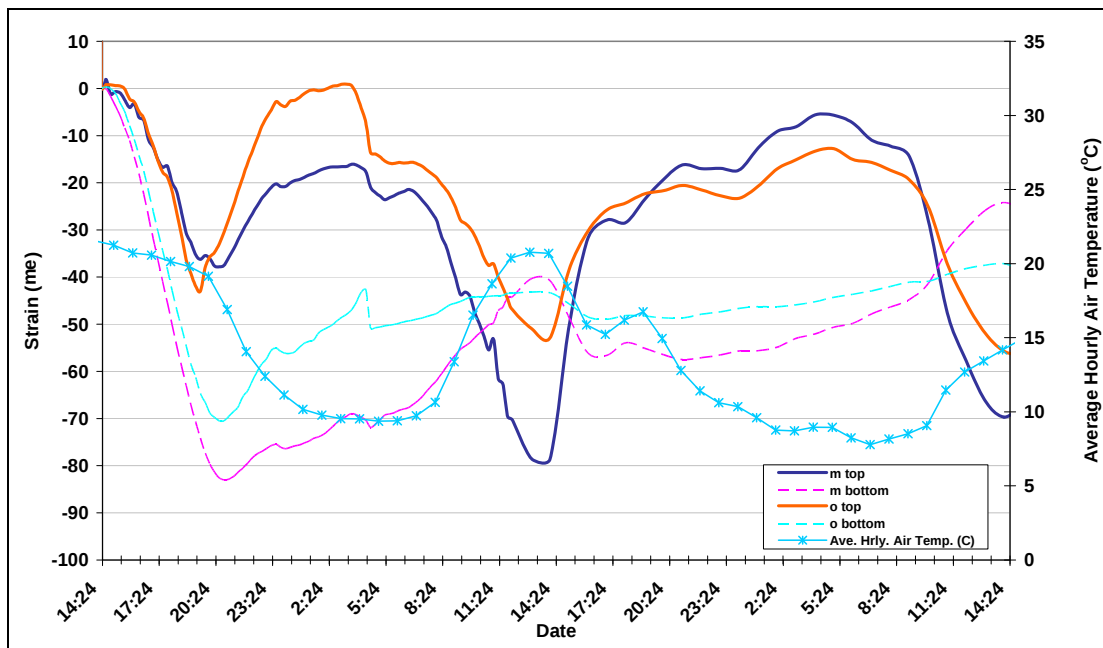


Figure 8 – Early strain development in the rubblized section at sensor locations m and o

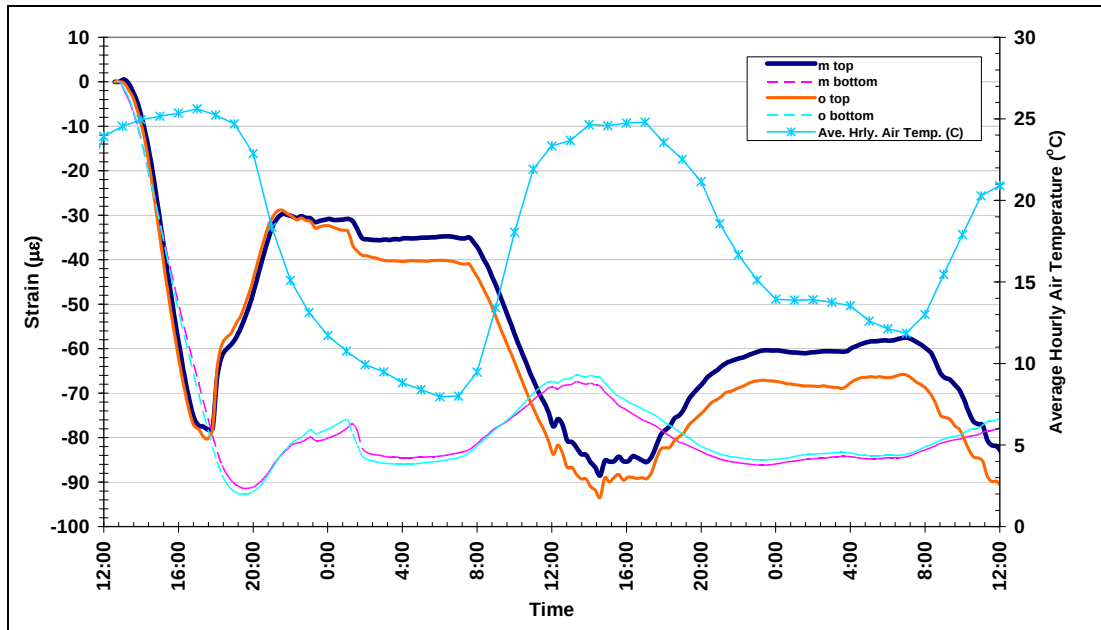


Figure 9 – Early strain development in the C/S section at sensor locations m and o

Temperature and Temperature-induced Deflections

With readings from TCs at four depths in the thickness of the concrete, temperature profiles within the slab were determined. A typical summer temperature profile plot is shown in Figure 10 on the following page. It is apparent from this plot that at night the surface of the slab is colder than the bottom, since the concrete retains heat from the sun. After the sun rises the surface begins to increase in temperature at a much faster rate than at the slab bottom, causing the temperatures to cross at about 9:30 AM. The opposite held true in the evening when the sun went down, and the surface cooled faster than the rest of the slab, with a second temperature crossing at around 20:30 (8:30 PM).

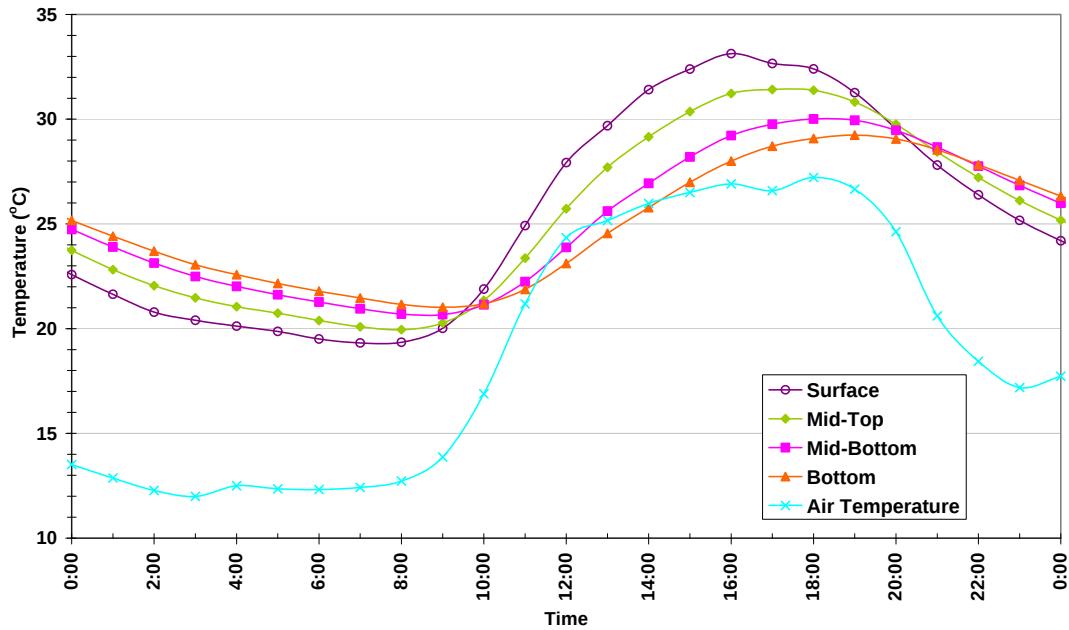


Figure 10 – Pavement temperature profile over 24 hours, C/S section July 24th, 2006

The LVDTs continuously measured the deflections in each instrumented slab as a function of time, and these were correlated with the temperature gradients determined by taking the temperature difference between the top and bottom thermocouples in the slab and dividing by the distance between them, 15.62 cm. Changes in the deflection were monitored over a 24 hour or 48 hour period containing one or two temperature reversal cycles. To do this, first the temperature data were inspected to locate a time of zero temperature gradient. In Figure 10, for example, this point occurred at about 9:30AM. All the deflection readings are zeroed relative to those at this point, and the change in displacement calculated for the next 24-48 hours. A sample plot of center-slab deflection and slab-end deflection is shown in Figure 11 for the dates of July 11-12, 2006; the temperature gradient is superimposed with the corresponding axis on the right. Similarly, the changes in strains and stresses over the monitoring period, relative to the time when the deflections were zeroed, were recorded and plotted, as in Figure 12 and Figure 13, which show typical strain and stress response behavior.

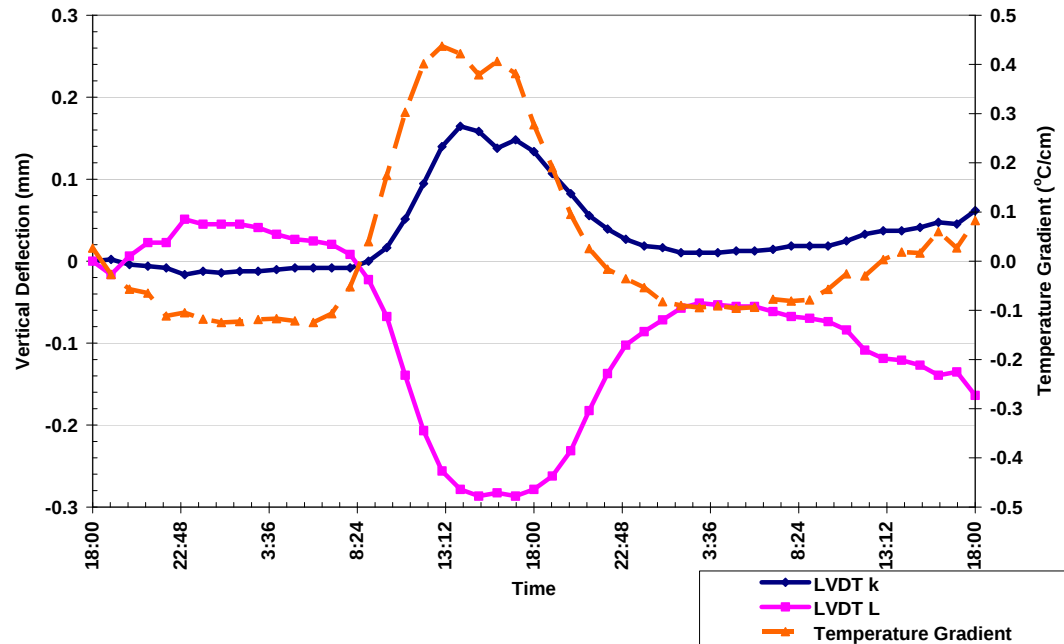


Figure 11 – Vertical deflection at slab-center and slab-end vs. time, C/S section July 10th-11th 2006. LVDTs k and l refer to the sensor locations in Figure 3.

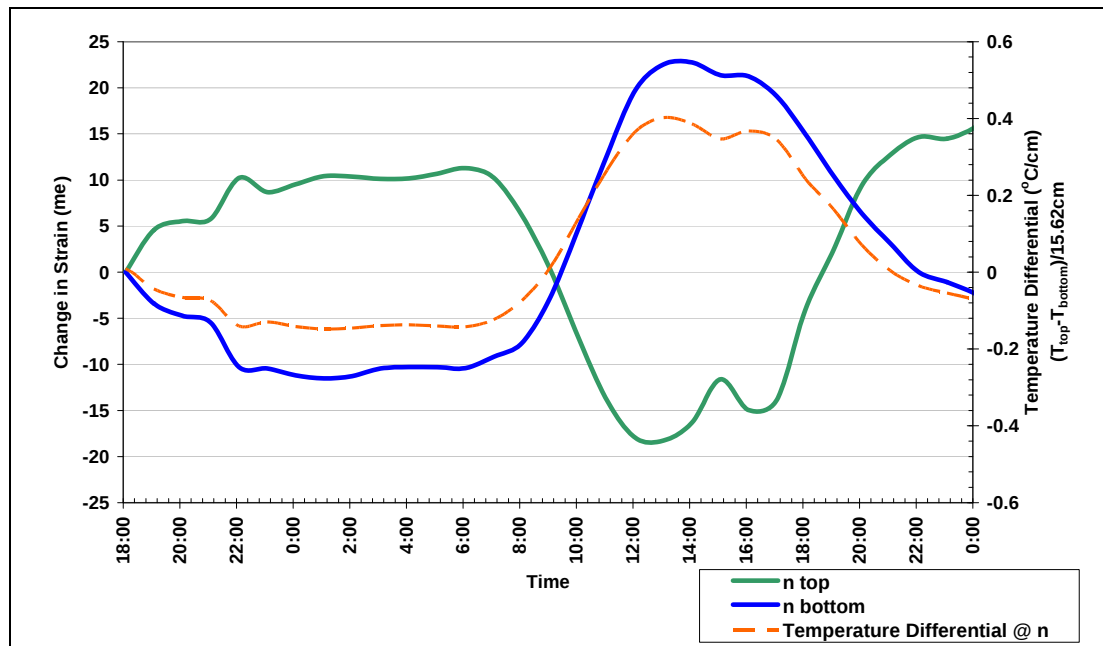


Figure 12 – Change in strain vs. time at slab center (sensor location n in Figure 3), C/S section July 11th – 12th 2006

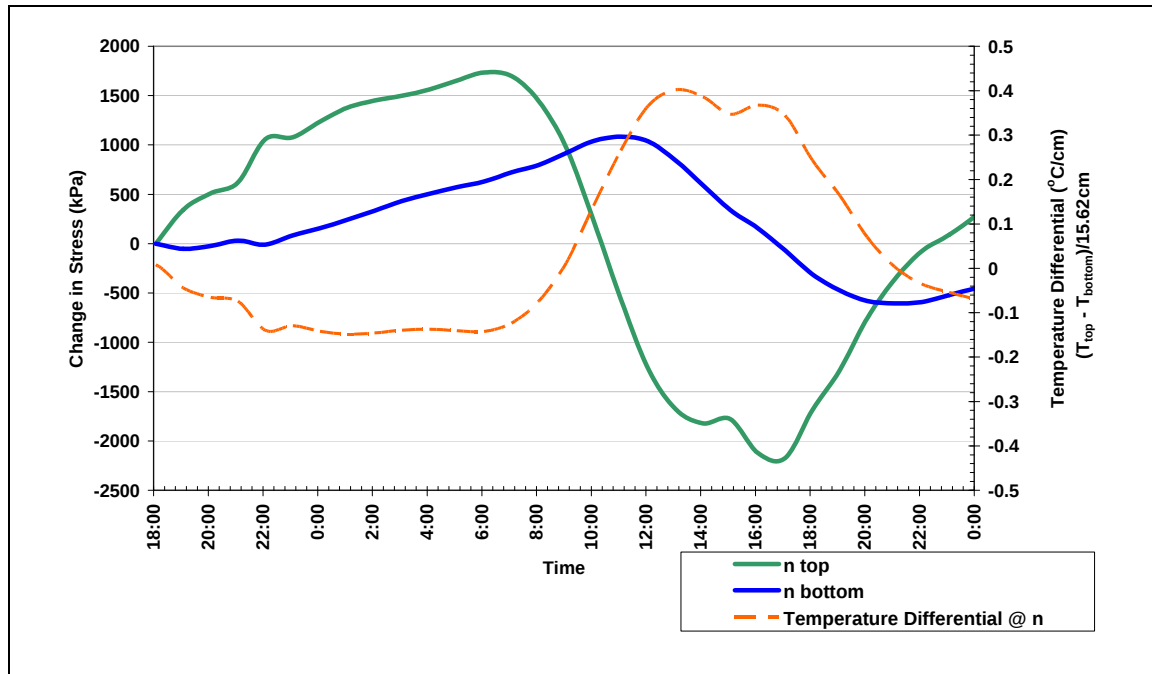


Figure 13 – Change in stress vs. time in the C/S section at sensor location n, July 11th - 12th 2006

Changes in Strain and Stress at Slab Center (Sensor Location n). In the following figures, the changes in strain at slab center (sensor location n) are presented over 48 hours for each experimental section. The data were recorded starting July 18, 2006, 12-14 days after the initial 28-day curing period was completed. Change-in-strain plots for the untreated section are presented first, followed by those for the rubblized section, and finally those for the C/S section. It is generally expected that the structural response from thermal curling would be more severe in the slab center location than the wheelpath location. Furthermore, the trends between the three experimental sections are expected to remain the same as when comparing the strains and stresses at sensor location n. For all plots of strain and stress, positive values represent tension, and negative values represent compression. Figure 14, Figure 15, and Figure 16 show that the environmentally induced strains were significantly higher in the untreated section than in either the rubblized section or the C/S section. The same trend was identified in the developed stresses.

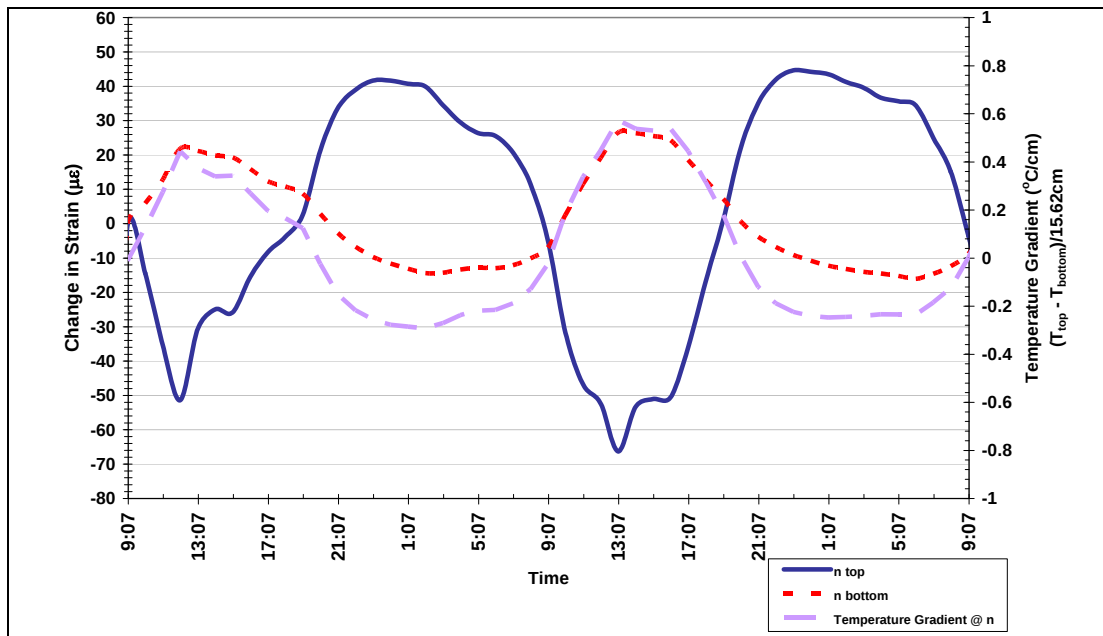


Figure 14 – Change in strain vs. time over 48 hours: untreated section July 18th-19th 2006 at location n

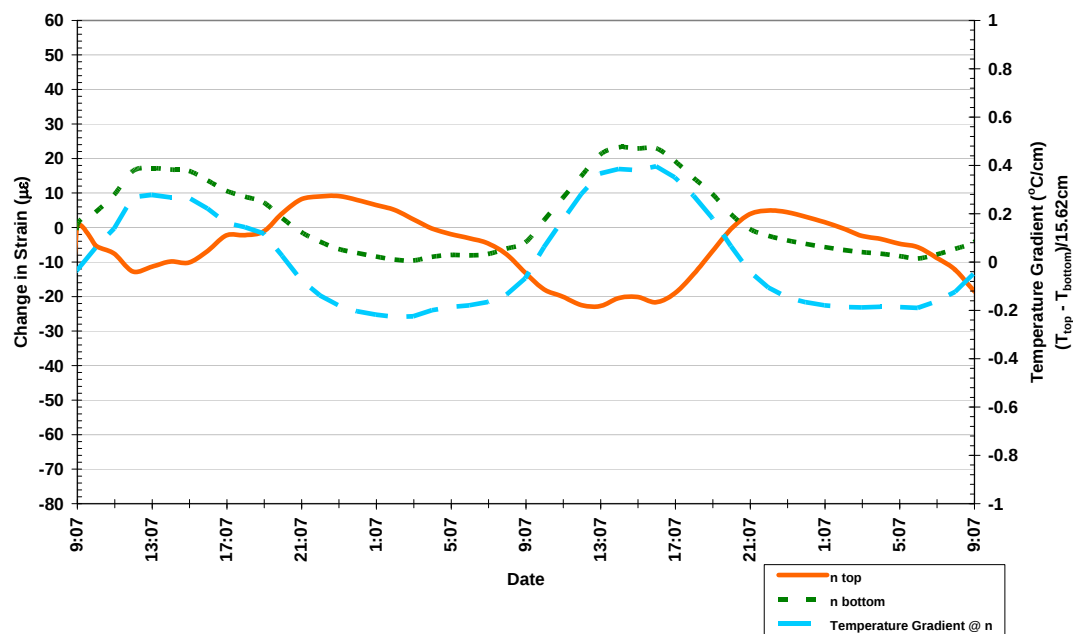


Figure 15 – Change in strain vs. time over 48 hours: rubblized section July 18th-19th 2006 at location n

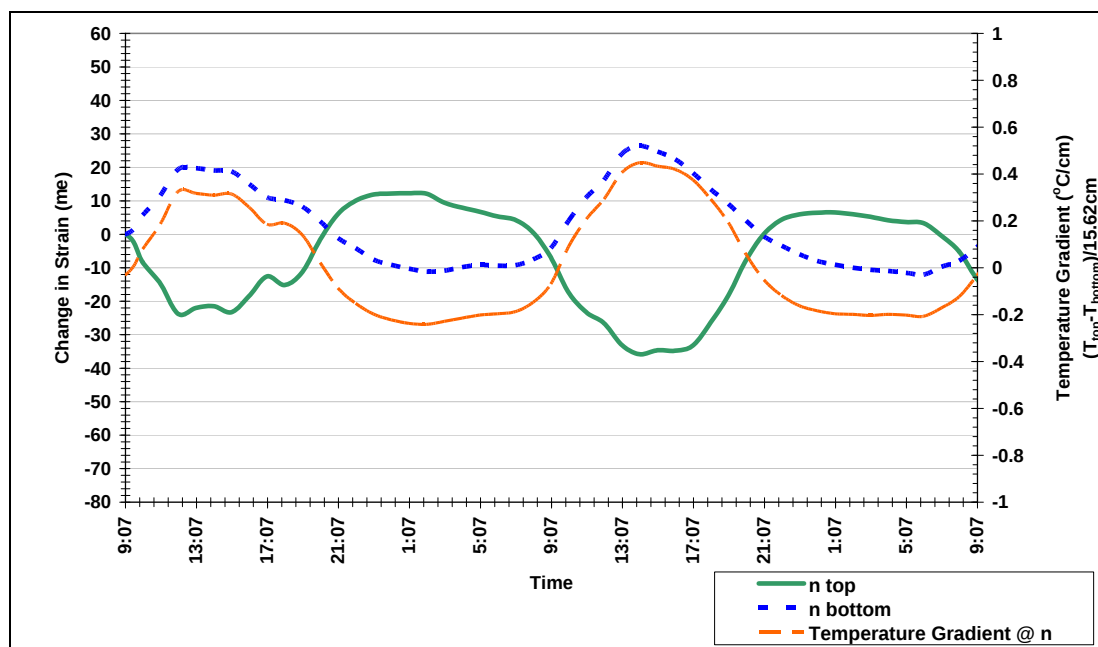


Figure 16 – Change in strain vs. time over 48 hours: C/S section July 18th-19th 2006 at location n

The maximum change in compressive and tensile strains and stresses for the top and bottom of the slabs are tabulated in Table 1, including the values from the December data sets.

Table 1 – Maximum changes in strain and stress for each time period at slab center (Location n)

Location n		Untreated				Rubbilized				Crack & Seat			
		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)	
July 18-19	Maximum	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
	Compression	-66.3	-16.1	-4312.3	-1454.2	-22.8	-9.5	-2711.7	-936.9	-35.8	-12.0	-3266.1	-1058.9
	Tension	44.6	26.5	1988.5	793.1	9.1	23.2	959.1	790.6	12.3	26.4	1210.4	769.7
Dec 1-2													
	Compression	-21.8	-6.7	-215.5	0.0	-9.5	-8.7	0.0	0.0	-11.2	-6.1	0.0	0.0
	Tension	22.2	7.3	3627.2	2273.5	9.1	5.8	3068.1	2067.6	3.8	9.9	2947.4	1887.2
Dec 5-6													
	Compression	-31.9	-9.8	-1760.1	-380.6	-11.6	-11.9	-1074.7	-335.3	-9.4	-12.6	-1172.7	-212.2
	Tension	24.3	9.3	1846.1	388.8	12.9	6.7	1439.0	258.1	8.5	10.1	1578.0	439.3

In the untreated section, the change in tensile stress in the top of the slab on December 1-2 2006 approached the modulus of rupture determined with laboratory testing of 3800kPa, yet this does not necessarily indicate that failure should be expected since the initial sense and magnitude of stress is unknown. The results from the flexural strength testing indicate that the concrete has a significantly lower flexural strength than the required 4726kPa, with peak strengths of almost 3800kPa and ultimate strength of only 2750kPa. If tensile stresses consistently exceed the capacity of the concrete, top-down tensile cracks should be considered a likely distress mode in the future.

As at location n, at location f the highest strains and stresses occurred in the untreated section, as can be seen in the results tabulated in Table 2. The rubbilized section, however, exhibited the next highest strains and stresses, while the C/S section

experienced the lowest strains and stresses. This relationship is different from the slab center during the same time period, in which the rubbilized section experienced the least severe response of all three sections, seemingly invalidating the expectation that the relationships between the sections are independent of slab location. The differences between the rubbilized and C/S section at location f, however, were not significant; rather the responses were quite similar. In the following section, the differences between the responses of each section for a given slab location are compared and discussed in more detail.

Table 2 – Maximum strains and stresses for each time period in the wheelpath midslab (Location f)

		Untreated				Rubbilized				Crack & Seat			
		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)	
		Maximum	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	
July 18-19	Compression	-46.54	-12.41	-3468	-1122	-25.89	-7.261	-3241	-1125	-27.03	-6.512	-3134	-886.7
	Tension	18.409	22.584	1194.5	737.93	3.8878	21.732	956.54	758.27	3.9163	20.317	1037.5	611.82

Table 3 was created by taking direct ratios of peak strains and stresses between each section. The tables illustrate how much greater the values of strain and stress were in the untreated section than the other two sections, and that the rubbilized and C/S sections had somewhat similar structural responses. Quantities in bold/red are the ratios between two sections greater than 1.15 or less than 0.85; exceeding a 15% threshold.

Table 3 – Ratios of strains and stresses between sections at slab center (location n)

		Strains and stresses between sections at slab center (location 1)											
		Untreated over Rubbilized				Untreated over C/S				C/S over Rubbilized			
		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)	
July 18-19	Maximum	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
	Compression	2.9	1.7	1.6	1.6	1.9	1.3	1.3	1.4	1.6	1.3	1.2	1.1
	Tension	4.9	1.1	2.1	1.0	3.6	1.0	1.6	1.0	1.3	1.1	1.3	1.0
Dec 1-2													
	Compression	2.3	0.8			2.0	1.1			1.2	0.7		
	Tension	2.4	1.3	1.2	1.1	5.8	0.7	1.2	1.2	0.4	1.7	1.0	0.9
Dec 5-6													
	Compression	2.8	0.8	1.6	1.1	3.4	0.8	1.5	1.8	0.8	1.1	1.1	0.6
	Tension	1.9	1.4	1.3	1.5	2.9	0.9	1.2	0.9	0.7	1.5	1.1	1.7

The peak strains at location n in the untreated section were typically several times greater than the other sections. In July, the tensile strain at the top gauge in the untreated section was almost 5 times greater than in the rubbilized, and 3.6 times more than in the C/S section. The tensile stresses in the bottom, however, were found to be nearly equal in all sections for July 18-19 and December 1-2. On December 5-6, the rubbilized section had the lowest tensile stress in the bottom of the slab, compared to the untreated and C/S sections. At the locations of the top gauges, the tensile stresses in the untreated section were calculated to be 2.1 times more than the rubbilized section and 1.6 times greater than the C/S section. In the warm temperatures of July, the C/S section exhibited 30% higher tensile strains and stresses in n_{top} than the rubbilized section, but in the cooler weather of December, the strains

at n_{top} in the rubbilized section were greater. In both December periods, the stresses in the top gauges were equal in the rubbilized and C/S sections.

In compression, the untreated section experienced significantly higher stresses than the other sections as well, suggesting a consistently more severe structural response; however these differences are fairly trivial since compressive failure is not a realistic concern in these applications.

Table 4 shows the ratios of the peak strains and stresses in location f. The rubbilized section exhibited equal strains and stresses in the top of the slab as the C/S section, and slightly lower strains and stresses in the bottom of the slab. This behavior is similar to location n in both of the December time periods, where the strains and stresses in the rubbilized section often exceeded those in the C/S section.

Table 4 – Ratios of strain and stresses between sections in the wheelpath midslab (location f)

	Untreated over Rubbilized				Untreated over C/S				C/S over Rubbilized			
	Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
July 18-19												
Maximum	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
Compression	1.8	1.7	1.1	1.0	1.7	1.9	1.1	1.3	1.0	0.90	1.0	0.79
Tension	4.7	1.0	1.2	1.0	4.7	1.1	1.2	1.2	1.0	0.93	1.1	0.81

In the midslab wheelpath locations, very similar trends between the strains and stresses were observed as in the slab center. The tensile strains in the top of the slabs were 4.7 times greater in the untreated section than in the other two, while the tensile stresses were only 20% higher. In all three sections, the peak compressive and tensile stresses were approximately equal, with the greatest differences only 20% to 30%. The bottom of the slab in the rubbilized section experienced 20% lower compressive and tensile stresses than the C/S section, and the bottom of the slab in the C/S section experienced 30% lower compressive stress and 20% lower tensile stress than the untreated section.

Table 5 shows the relative differences in peak strain and stress between location n and location f during the same period for each section. Only three values in the table are less than unity – in the rubbilized section, for compressive strains and stresses.

Table 5 – Ratio between values at slab center (location n) and outer wheelpath midslab (location f)

	Untreated				Rubbilized				Crack & Seat			
	Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)		Strain (me)		Stress (kPa)	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
July 18-19												
Maximum	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
Compression	1.43	1.29	1.24	1.30	0.88	1.31	0.84	0.83	1.32	1.85	1.04	1.19
Tension	2.42	1.18	1.66	1.07	2.35	1.07	1.00	1.04	3.13	1.30	1.17	1.26

Since all other values are greater than or equal to one, it may be generally concluded that the structural responses at the center of the slab were generally equal or greater than those in the outer wheelpath at midslab, meeting the expectation that the slab center is a more critical location. In the untreated section, almost 2.5 times more tensile strain occurred in the top of the slab at the center, and 1.66 times more tensile stress resulted. In the rubbilized section, 2.35 times more strain occurred in the top of the slab than in the wheelpath, but the stresses were found to be equivalent.

A similar relationship was determined in the C/S section, with over 3 times more tensile strain in the top at slab center than in the wheelpath but only slightly higher tensile stresses.

Contribution from the Subgrade. The difference between a deep reference and a shallow reference LVDT in similar locations is the contribution from the subgrade. LVDT b and LVDT i are in corresponding locations within Slab 1 and Slab 2, respectively. LVDT b is deep reference and LVDT i is shallow reference, so the readings from i were subtracted from b to obtain the subgrade contribution at the inner wheelpath location. The other paired locations are in the outer wheelpath at the leave-end of the slab, with LVDT l being a deep reference in Slab 2, and LVDT d a shallow reference in Slab 1. The difference between the two represents the subgrade contribution at this location. For each section, the differences between these two pairs were plotted for the July 18–19 2006 time period in Figure 17, Figure 18, and Figure 19.

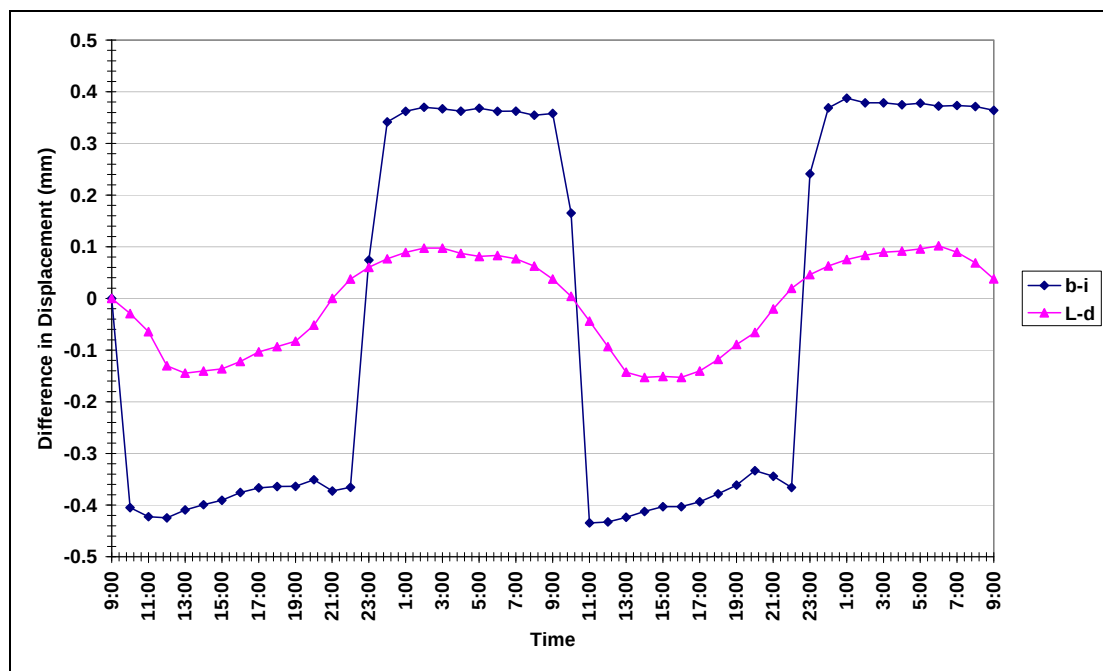


Figure 17 – Subgrade contribution to vertical displacement in untreated section, locations b/i and l/d

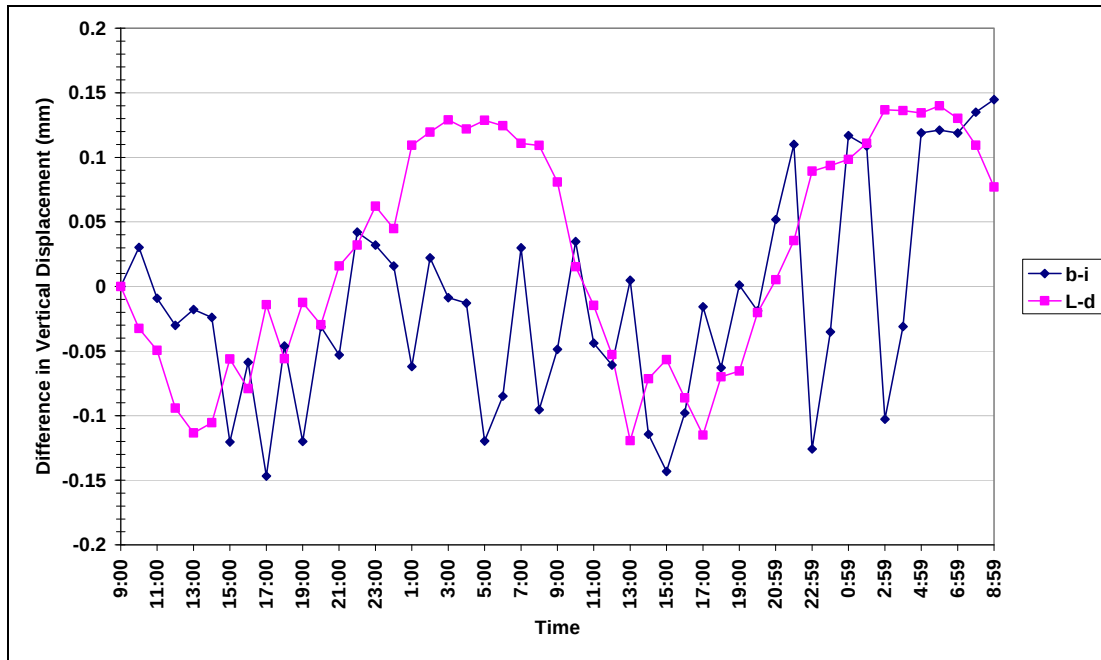


Figure 18 – Subgrade contribution to vertical displacement in rubblized section, locations b/i and l/d

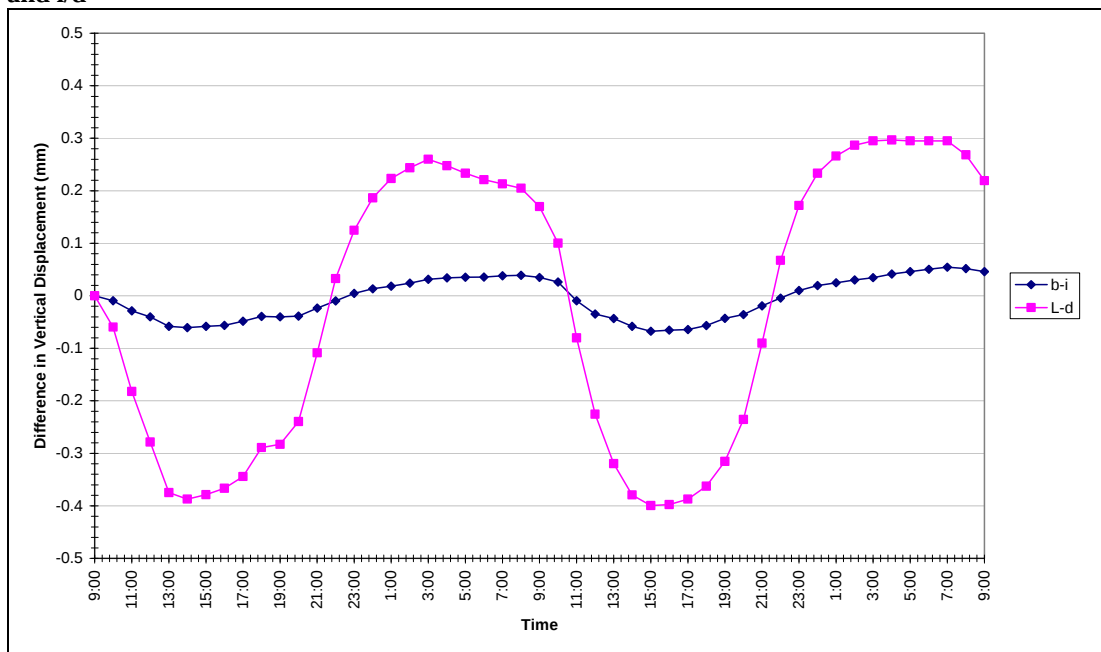


Figure 19 – Subgrade contribution to vertical displacement in C/S section, locations b/i and l/d

FWD Testing

Figure 20 and Table 6 show the peak deflection data for each test in each test section from the three FWD test sessions, in June 2006, November 2006, and March 2007. From Figure 20 it is evident that the normalized strains generally increased from June to November to March in each experimental section, though there were some

exceptions. The rubbilized section exhibited peak strain values that were relatively consistent across all three seasons, and some individual gauges in the C/S and untreated sections measured lower strains in November than in March.

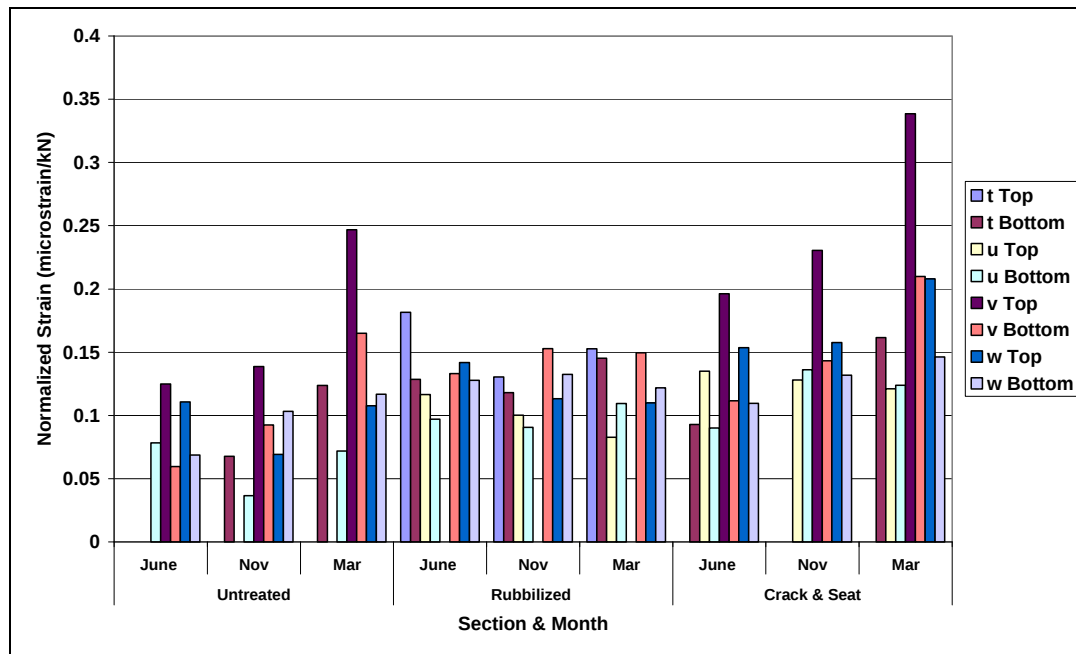


Figure 20 – Peak dynamic strain responses organized by section

Table 6 – Peak dynamic strains and normalized strains from FWD testing – all available data

June 20th 2006		Untreated			Rubbilized			C/S		
MM	Location	Load (kN)	Strain me	Norm. Strain me/kN	Load (kN)	Strain me	Norm. Strain me/kN	Load (kN)	Strain me	Norm. Strain me/kN
t	Top	71.03	ND	ND	69.97	12.7	0.181	71.06	ND	ND
	Bottom		ND	ND		9.0	0.129		6.6	0.093
u	Top	71.52	ND	ND	72.13	8.4	0.116	71.12	9.6	0.135
	Bottom		5.6	0.078		7.0	0.097		6.4	0.090
v	Top	70.52	8.8	0.125	70.67	ND	ND	70.84	13.9	0.196
	Bottom		4.2	0.060		9.4	0.133		7.9	0.112
w	Top	71.40	7.9	0.111	71.96	10.2	0.142	70.30	10.8	0.154
	Bottom		4.9	0.069		9.2	0.128		7.7	0.110
November 28th 2006		Untreated			Rubbilized			C/S		
MM	Location	Load (kN)	me	me/kN	Load (kN)	me	me/kN	Load (kN)	me	me/kN
t	Top	73.77	ND	ND	72.90	9.5	0.130	73.38	ND	ND
	Bottom		5.0	0.068		8.6	0.118		ND	ND
u	Top	73.68	ND	ND	72.80	7.3	0.100	73.48	9.4	0.128
	Bottom		2.7	0.037		6.6	0.091		10.0	0.136
v	Top	73.58	10.2	0.139	72.65	ND	ND	73.34	16.9	0.230
	Bottom		6.8	0.092		11.1	0.153		10.5	0.143
w	Top	73.63	5.1	0.069	72.46	8.2	0.113	73.58	11.6	0.158
	Bottom		7.6	0.103		9.6	0.132		9.7	0.132
March 20th 2007		Untreated			Rubbilized			C/S		
MM	Location	Load (kN)	me	me/kN	Load (kN)	me	me/kN	Load (kN)	me	me/kN
t	Top	65.49	ND	ND	67.49	10.3	0.153	67.49	ND	ND
	Bottom		8.1	0.124		9.8	0.145		10.9	0.162
u	Top	65.39	ND	ND	67.63	5.6	0.083	66.95	8.1	0.121
	Bottom		4.7	0.072		7.4	0.109		8.3	0.124
v	Top	64.86	16	0.247	67.59	ND	ND	65.30	22.1	0.338
	Bottom		10.7	0.165		10.1	0.149		13.7	0.210
w	Top	65.93	7.1	0.108	67.29	7.4	0.110	66.37	13.8	0.208
	Bottom		7.7	0.117		8.2	0.122		9.7	0.146

*ND indicates no usable data

Table 7 includes all available data from LVDTs and from the FWD device to date. Table 8 displays the normalized data only, which are also plotted in Figure 21. Due to limited data, relationships in Figure 21 are more evasive when comparing the response of any section between seasons. From the untreated and C/S sections, the data show that midslab deflections (LVDT location k – outer wheelpath midslab) were higher in June than in November, while at the joint, the deflections were greater in November, albeit only slightly. In June, the very high displacement measured in the untreated section by LVDT k may be justified by the temperature gradient at the time of testing. The drop occurred at about 6:00 PM, when the temperature gradient in the concrete was at a high positive value, meaning that the top of the slab was several degrees warmer than the bottom. This induced a concave down curled shape, causing the center of the slab to lift. In the untreated section, this lift was not supported while in the rubbilized and C/S sections, more support was maintained by the base and the AC interlayer.

Table 7 - Peak dynamic deflections and normalized deflections from FWD testing – all data

June 20th 2006		Untreated			Rubblized			C/S		
LVDT		Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)
a	d _{FWD}	71.07	0.100	1.404	70.72	0.144	2.040	71.14	0.136	1.910
	d _{LVDT}		0.019	0.270		0.032	0.447		0.027	0.385
b	d _{FWD}	71.17	0.098	1.374	71.89	0.162	2.254	71.32	0.126	1.774
	d _{LVDT}		0.068	0.961		0.126	1.754		0.074	1.035
c	d _{FWD}	71.17	0.083	1.171	71.01	0.140	1.978	68.30	0.099	1.443
	d _{LVDT}		ND	ND		0.026	0.362		0.020	0.288
d	d _{FWD}	71.38	0.086	1.199	71.00	0.131	1.839	71.46	0.107	1.500
	d _{LVDT}		ND	ND		ND	ND		ND	ND
i	d _{FWD}	71.07	0.103	1.447	70.79	0.175	2.476	71.32	0.121	1.699
	d _{LVDT}		0.025	0.355		0.055	0.770		0.020	0.280
J	d _{FWD}	71.85	0.088	1.223	71.11	0.131	1.839	70.60	0.112	1.587
	d _{LVDT}		ND	ND		0.087	1.221		0.068	0.969
k	d _{FWD}	70.94	0.087	1.225	71.36	0.130	1.819	71.93	0.061	0.855
	d _{LVDT}		0.214	3.018		0.087	1.225		0.068	0.951
L	d _{FWD}	71.35	0.087	1.225	71.31	0.136	1.913	71.57	0.116	1.622
	d _{LVDT}		0.058	0.813		0.093	1.301		0.085	1.182
November 28th 2006		Untreated			Rubblized			C/S		
LVDT		Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)
a	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
b	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
c	d _{FWD}	73.77	0.073	0.988	74.60	0.105	1.410	73.53	0.107	1.451
	d _{LVDT}		0.013	0.182		0.021	0.279		0.017	0.231
d	d _{FWD}	77.38	0.086	1.106	73.77	0.098	1.322	73.92	0.135	1.821
	d _{LVDT}		ND	ND		ND	ND		0.013	0.173
i	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
J	d _{FWD}	74.89	0.084	1.123	73.04	0.109	1.488	73.68	0.167	2.262
	d _{LVDT}		ND	ND		ND	ND		ND	ND
k	d _{FWD}	74.46	ND	ND	73.63	0.101	1.366	75.14	0.152	2.028
	d _{LVDT}		0.057	0.764		ND	ND		0.058	0.765
L	d _{FWD}	73.92	0.066	0.887	74.07	0.103	1.396	73.34	0.269	3.668
	d _{LVDT}		0.067	0.909		ND	ND		0.098	1.334
March 21st 2007		Untreated			Rubblized			C/S		
LVDT		Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)	Load (kN)	d (mm)	Norm. Disp. (micron/kN)
a	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
b	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
c	d _{FWD}	65.85	0.098	1.492	67.93	0.168	2.479	68.32	0.164	2.398
	d _{LVDT}		ND	ND		ND	ND		ND	ND
d	d _{FWD}	65.15	0.128	1.968	68.41	0.124	1.819	66.66	0.142	2.138
	d _{LVDT}		ND	ND		ND	ND		ND	ND
i	d _{FWD}	ND	ND	ND	ND	ND	ND	ND	ND	ND
	d _{LVDT}		ND	ND		ND	ND		ND	ND
J	d _{FWD}	65.51	0.091	1.393	68.07	0.120	1.761	66.22	0.133	2.006
	d _{LVDT}		ND	ND		ND	ND		ND	ND
k	d _{FWD}	64.95	0.204	3.143	67.83	0.137	2.015	65.93	0.237	3.591
	d _{LVDT}		ND	ND		ND	ND		ND	ND
L	d _{FWD}	66.04	0.105	1.586	67.78	0.098	1.443	67.20	0.143	2.128
	d _{LVDT}		ND	ND		ND	ND		ND	ND
Shaded rows represent deep reference LVDTs					*ND indicates no usable data					

Table 8 – Peak dynamic normalized deflection response data

micron/kN		Untreated			Rubbilized			Crack & Seat		
Location		June	Nov	Mar	June	Nov	Mar	June	Nov	Mar
a	d _{FWD}	1.40	ND	ND	2.04	ND	ND	1.91	ND	ND
	d _{LVDt}	0.27	ND	ND	0.45	ND	ND	0.39	ND	ND
b	d _{FWD}	1.37	ND	ND	2.25	ND	ND	1.77	ND	ND
	d _{LVDt}	0.96	ND	ND	1.75	ND	ND	1.03	ND	ND
c	d _{FWD}	1.17	0.99	1.49	1.98	1.41	2.48	1.44	1.45	2.40
	d _{LVDt}	ND	0.18	ND	0.36	0.28	ND	0.29	0.23	ND
d	d _{FWD}	1.20	1.11	1.97	1.84	1.32	1.82	1.50	1.82	2.14
	d _{LVDt}	ND	ND	ND	ND	ND	ND	ND	0.17	ND
i	d _{FWD}	1.45	ND	ND	2.48	ND	ND	1.70	ND	ND
	d _{LVDt}	0.35	ND	ND	0.77	ND	ND	0.28	ND	ND
j	d _{FWD}	1.22	1.12	1.39	1.84	1.49	1.76	1.59	2.26	2.01
	d _{LVDt}	ND	ND	ND	1.22	ND	ND	0.97	ND	ND
k	d _{FWD}	1.22	ND	3.14	1.82	1.37	2.01	0.85	2.03	3.59
	d _{LVDt}	3.02	0.76	ND	1.22	ND	ND	0.95	0.77	ND
l	d _{FWD}	1.22	0.89	1.59	1.91	1.40	1.44	1.62	3.67	2.13
	d _{LVDt}	0.81	0.91	ND	1.30	ND	ND	1.18	1.33	ND

*ND indicates no usable data

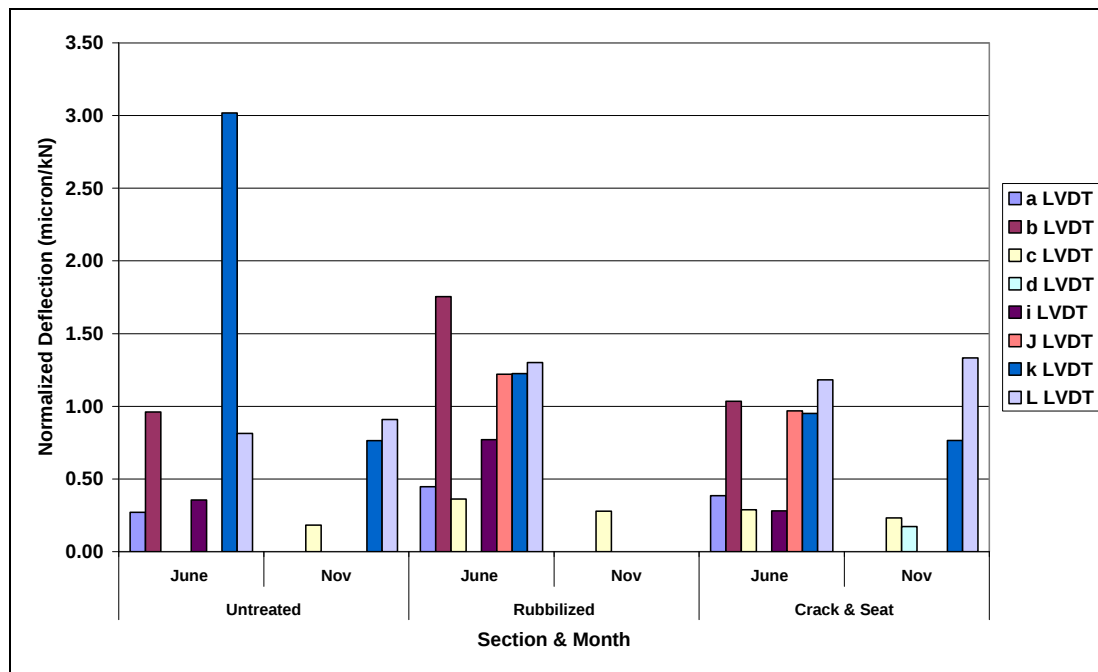


Figure 21 – Peak dynamic deflection response from LVDt's organized by section

The differences between the responses in each section are quantified in Table 9 by dividing the normalized displacement values of the untreated and C/S sections into those of the rubbilized section for each LVDt location.

Table 9 – Ratios of LVDT displacements in rubbilized section to untreated and C/S sections, June 20th 2006

LVDT	June 20th, 2006	
	Ratio of Rubbilized to Untreated	C/S
a	1.65	1.16
b	1.83	1.7
c	ND	1.25
d	ND	ND
i	2.17	2.75
j	ND	1.26
k	0.41	1.29
l	1.6	1.1

Aside from the relative rank of the rubbilized section typically having highest deflections, the differences were variable. From the shallow reference LVDTs, only LVDT a and LVDT i have data in all three sections. At LVDT a, the deflection in the rubbilized section was 1.65 times greater than the untreated section, and 1.16 times more than C/S. At LVDT i, also in the inner wheelpath, but at the opposite end of the slab from location a, the rubbilized section values were 2.17 and 2.75 times greater than those of the untreated or C/S sections, respectively. The vertical displacement readings from the shallow LVDTs represent the deflection in the pavement-subbase system only. The data show that the rubbilized JRCF deflected significantly more than the untreated or C/S JRCF, but the amount of difference to expect in this system is inconclusive.

With very limited data from the LVDTs in November 2006, few conclusions can be drawn about the differences between the sections at this time. In location c, however, it can be observed that the shallow displacement in the rubbilized section was again greater than in the other two sections. Table 10 summarizes the relationships between the FWD displacements in the three sections at each LVDT location. While other valuable information can be taken from the LVDT readings themselves, the FWD device tended to be more consistent, and more complete data sets were obtained. From the table, it can be generalized that in June, the displacements in the rubbilized section were distinctively greatest in the rubbilized section, and distinctively smallest in the untreated section. In November, the C/S section exhibited larger displacements in each location than the rubbilized section, but the untreated section still tended to displace the least. In March 2007, the data were more scattered, but the C/S section and the untreated section tended to displace more than the rubbilized section, and the C/S section again had the highest normalized displacements.

Table 10 – Ratios of FWD displacements in rubbilized section to untreated and C/S sections

LVDT	June		November		March	
	Ratio of Rubbilized to Untreated	C/S	Ratio of Rubbilized to Untreated	C/S	Ratio of Rubbilized to Untreated	C/S
a	1.45	1.07	ND	ND	ND	ND
b	1.64	1.27	ND	ND	ND	ND
c	1.69	1.37	1.43	0.97	1.66	1.03
d	1.53	1.23	1.2	0.73	0.92	0.85
i	1.71	1.46	ND	ND	ND	ND
j	1.5	1.16	1.33	0.66	1.26	0.88
k	1.49	2.13	ND	0.67	0.64	0.56
l	1.56	1.18	1.57	0.38	0.91	0.68

Discussion of Results and Conclusions

Environmental Response. The untreated section exhibited the highest environmental strains and stresses in each season. For the July 18-19 2006 time period, as in other periods examined, the environmentally induced changes in strains and changes in stresses in the center of the slab were significantly higher in the untreated section than in either the rubbilized section or the C/S section. In the wheelpath midslab, the same ordering of the responses between the sections held, but was not as pronounced.

The large variances in strain and stress between the sections occurred during times of high temperature gradients, as expected. Temperature gradients influenced the interaction between the overlay and the existing JRCP differently in each section. When the temperatures of the surface and the subsurface were different, differential thermal movement of the JRCP transferred forces through frictional bonds to the overlay. In the untreated section especially, the relative expansion or contraction of the original JRCP intensified the bending stresses in the top of the slab. For example, when the slab was curled up, the bottom was warm so the JRCP expanded more than the overlay and added axial tension to the slab. The uniform axial tension superimposed on the bending stresses, increasing the tension in the top of the slab while reducing the compression in the bottom of the slab. For a slab that was curled down, this action amplified the compression in the top and reduced the tension in the bottom. Again, this effect was strongest in the untreated section, moderate in the C/S section, and non-existent in the rubbilized section.

In addition to thermal effects of the existing JRCP, loss of support caused strains and stresses to develop, most severely affecting the untreated section, and affecting the C/S section to a lesser degree. The rubbilized base tended to flex and maintain contact with the pavement, offering more evenly distributed support than the untreated section. The support provided by the C/S base was better than the untreated section, but was more comparable to that provided by the rubbilized base. The experimental data upheld the expectations that the stresses in the untreated section would be greatest due to thermal action of the JRCP and loss of support. The relationship between the environmental responses from the rubbilized and C/S sections changed over time, but the responses were generally similar.

Dynamic Test Responses. The relative severity of dynamic strains among the three sections was opposite of the environmental strains. On June 20th 2006, the rubbilized section exhibited the largest dynamic strains on average, closely followed by the C/S section, and the untreated section showed the smallest strains. While some strains in the rubbilized section were over twice as large as the untreated section, on average the strain response was 65% greater in the rubbilized section than in the untreated section, and 10% larger than the C/S section. On November 28th 2006 and March 20th 2007, the C/S experienced the largest strains, then the rubbilized, and again the smallest strains occurred in the untreated section. On these dates, the C/S section exhibited normalized strains that were 75% and 50% larger than the untreated section, and 18% and 32% larger than the rubbilized section, respectively.

The gauges in the top of the slabs typically measured higher strains than the bottom of the slabs in the same location. This effect was most pronounced in the untreated section. Frictional interlock with the AC interlayer caused the neutral axis to deepen in the slab due to the additional bending capacity of the JRCP. In the rubbilized section, the strains in the top and bottom gauges were approximately equal since the rubbilized base offers very little bending capacity.

In June, the dynamic displacements were distinctively greatest in the rubbilized section, and distinctively smallest in the untreated section in wheelpath and lane-center locations. Measurements from deep reference LVDTs, shallow reference LVDTs, and FWD geophones confirmed the highest displacements occurring in the rubbilized section. The summarized data show that in November and March, the rubbilized section still tended to experience the highest displacements along the center of the lane, but in the wheelpath, the deflections were highest in the C/S section. Exceptions occurred when loss of support during positive temperature gradients in the slab centers of the C/S section allowed large displacements in certain slabs. The dynamic deflections measured in the wheelpath completely correspond with the trends in dynamic strain previously discussed.

General Conclusions. Rubbilization treatment was very effective for negating thermal action of existing JRCP, maintaining contact/support with overlay, significantly reducing environmentally induced strains and stresses compared to the conventional overlay. The section with the rubbilized base exhibited relatively low structural stiffness, and was ineffective for reducing lift and limiting high early dynamic strains. The AC interlayer did not necessarily act as a bond breaking layer, and the data show that the overlay in the untreated section acted compositely with the existing JRCP.

The C/S treatment was effective for reducing thermal action of existing JRCP, significantly reducing environmentally induced strains and stresses, and maintaining more load transfer capability than the untreated control section. C/S treatment was only partially effective for eliminating loss-of-support and controlling dynamic strains. The structural stiffness of the system (its resistance to displacement) was quite low.

The higher dynamic stresses in the C/S and rubbilized sections were much less critical than the larger environmental stresses determined for the untreated section,

since the differences in the environmental stresses far exceeded those between the dynamic stresses. Because of the more severe thermally induced strains and stresses, it is expected that the untreated section will fail first. Top-down tensile cracks, and eventually full depth transverse cracks, are a likely mode of future distresses and are most likely to occur in winter months.

Since cracking and seating procedures are faster and less labor intensive than rubbilization while the benefits are comparable, C/S is probably a more economically effective rehabilitation technique. However, there are no significant distresses in any of the three sections at a pavement age of 11 months. Without examining a complete lifespan of the pavement, making predictions of economic worth of these treatments is premature.

Reference

Swart, C. S. (2006). A forensic investigation of pavement performance on Interstate 86 in Olean, New York. (Master of Science, Ohio University), 24, 28, 59, 141.