Implementation: Mitigation of High Sulfate Soils in Texas

Development of Design and Construction Guidelines

Final Report 5-6618

Prepared for

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1. INTRODUCTION

Texas Department of Transportation (TxDOT) continues to experience pavement failures which are particularly evident in sites where high sulfate soils of 8000 ppm or higher predominate. Many of the recent pavement failures are attributed to sulfate-induced soil heave where an expansive mineral called Ettringite is formed from calcium-based stabilizers reacting with water, clay, and sulfates in the soil. Current chemical stabilization practices for high sulfate soils have resulted in high maintenance costs and safety concerns due to the increasing roughness and distress that these pavements have experienced. Many districts have to partially or completely rehabilitate these pavements built on high sulfate soils within a few months to three to four years after original construction. In many cases, the repairs will include a complete restoration of the pavement lanes for several miles, which can be translated to losses of entire pavement infrastructure for those miles of pavement.

Implementation project 5-6618 is initiated to evaluate the pavement performance at US 82 highway near Bells, TX, where novel construction techniques with two different stabilizers and extended mellowing periods are adopted. Two treated pavement test sections and one control unpaved test section were constructed and monitored periodically during the course of two years and the details of the results are presented. Detailed analysis of Ettringite formation assessments and non-destructive pavement monitoring studies including FWD and profiler are presented. This final report summarizes the results obtained for all the Tasks performed for project 5-6618-001 which include future recommendations for design and construction on high sulfate expansive clayey soils.

2. FIELD IMPLEMENTATION

At the conclusion of TxDOT research project 0-6618, three test sections comprising of one control and two treated test sections were constructed on US 82 highway near Bells in Grayson County, Texas. These test sections aided in screening and evaluation of stabilizers for sulfate soil conditions. Control Section (Test Section 3) consisted of a lime treatment subgrade section with a 3 day mellowing period, which is a commonly practiced stabilization technique by TxDOT. Test sections 1 and 2 are composed of lime-fly ash treated and lime treated sections, respectively with varying mellowing time durations. Layout of these test sections is presented in Figure 1. As a part of the implementation project, 5-6618, monitoring of these test sections was conducted using elevation surveys, surface profiler studies and Falling Weight Deflectometer investigations.

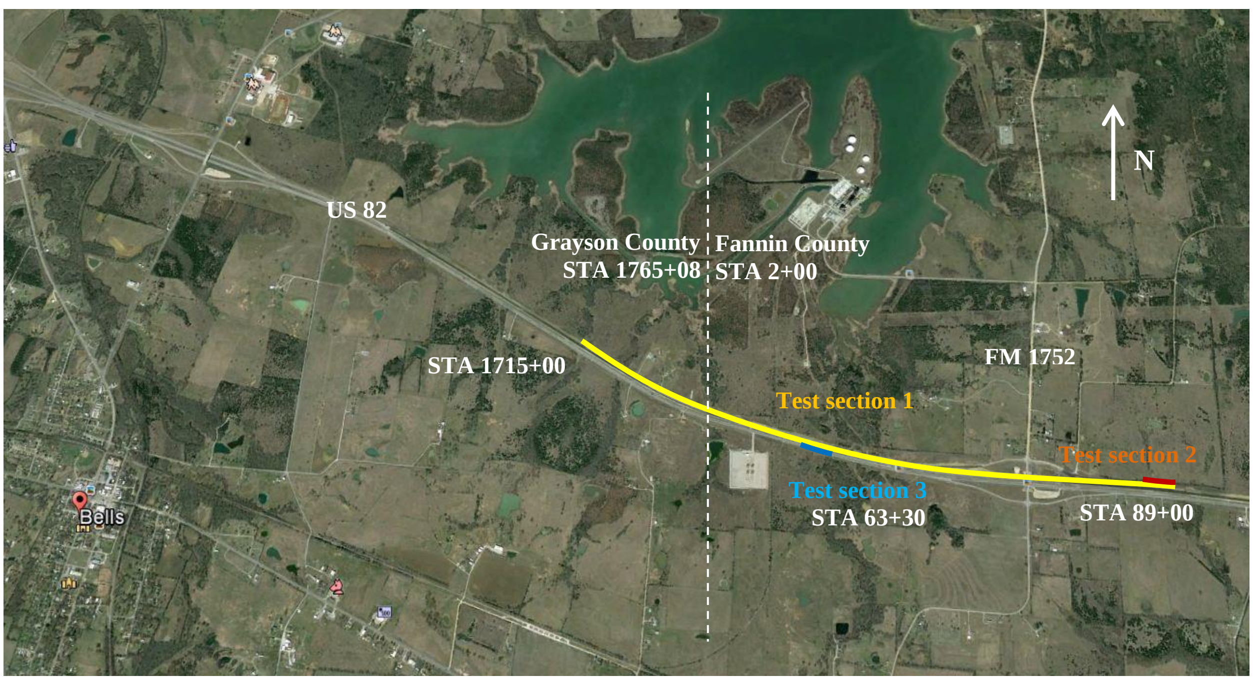


Figure 2: Layout of test sections at US 82, Bells, TX

An overview of each test section along with the construction steps followed in the field is presented in Table 1. Test sections 1 and 2 were constructed with extended mellowing periods of 10 and 7 days, respectively. Test section 1 is 2.1 miles long and is part of the east bound US 82 pavement lanes in STA 89+00 Fannin County to STA 1715+00 in Grayson County. Test section 2 is in the north bound shoulder from STA 88+00 to 89+00, extending to 100 feet. Test section 3 was constructed in the median and away from both pavement lanes of US 82 highway. The following sections present the field monitoring studies and the analyses of field monitoring results.

Table 1: Construction phase followed for different test sections

Days	**Treatment**		
	Lime + Fly Ash Extended mellowing (Test Section 1)	**Lime Extended mellowing (Test Section 2)**	**Lime 3 day mellowing (Test Section 3)**
1	Lime Treated subgrade (6%) light compact	Lime Treated subgrade (6%) light compact	Lime Treated subgrade (6%) light compact
2-3	Mellowing period	Mellowing period	Mellowing & Final Compact
4	Recut & Light Compact	Recut & Light Compact	-
5	Mellowing period	Mellowing period	-
6	Recut & Light Compact	Recut & Light Compact	-
7	Mellowing period	Remix & Final Compaction	-
8	Fly ash treatment (3%) & Light Compact	-	-
9	Mellowing period	-	-
10	Remix & Final Compaction	-	-

Construction of US 82 pavement section near Bells, TX is completed in May 2014. Typical section of the constructed test pavement is presented in Figure 2. The details of the three test sections are provided in the following sections.

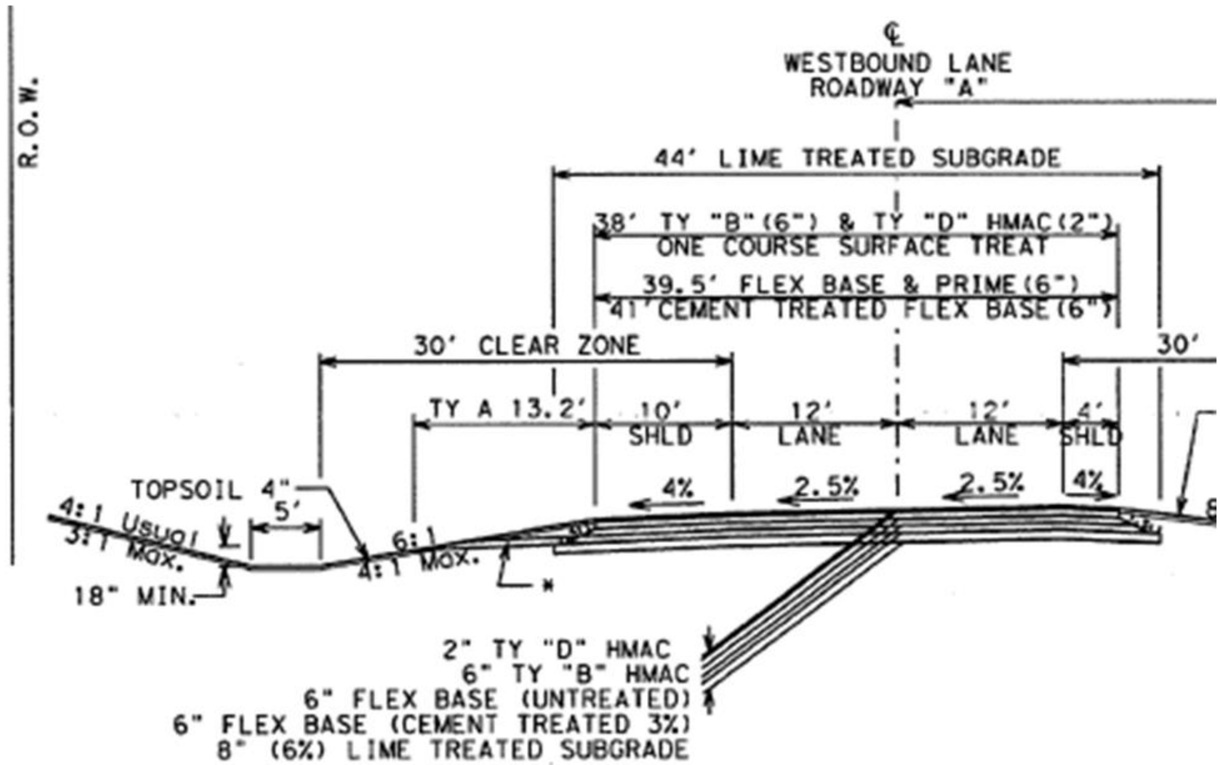


Figure 2: Typical section of US82 pavement

2.0.1 Test Section 1:

Test section 1 comprised of base course with lime + fly ash with extended mellowing from STA 1715+00 in Grayson County to 89+00 in Fannin County as shown in Figure 3. In this region, the main lanes and shoulders were treated with lime and fly ash with extended mellowing. Paving of the test section was completed in May 2014 and Falling Weight Deflectometer (FWD) and surface profiler investigations are followed.

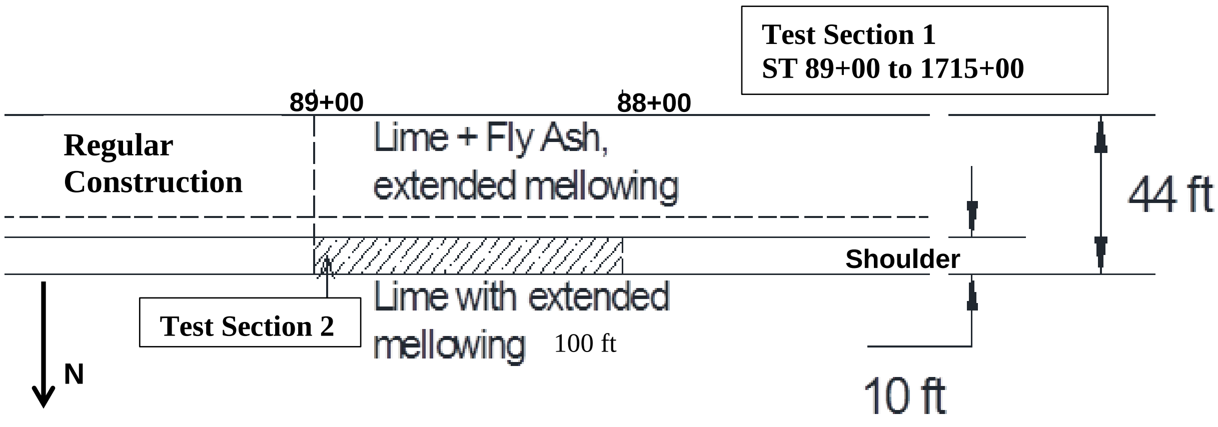


Figure 3: Schematic of test sections 1 and 2

2.0.2 Test Section 2:

This test section was treated with lime followed by extended mellowing from STA 88+00 to STA 89+00 in Fannin County as shown in Figure 3. Treatment was performed on the north shoulder of the pavement only as shown in Figure 4. FWD and surface profiler investigations are conducted regularly in order to evaluate pavement performance. An overview of test section 2 is presented in Figure 4.

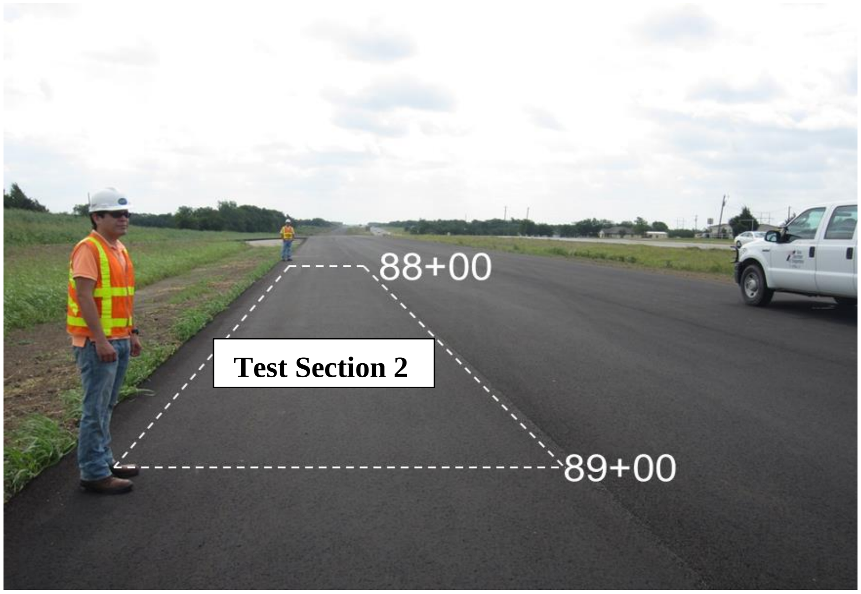


Figure 4: Schematic of test sections 1 and 2

2.0.3 Test Section 3:

Test section 3 was constructed using Lime with 3 day mellowing (typically followed for construction and is currently a control section) as shown in Figure 5 extending from STA 63+00 to STA 63+60 in Fannin County. This test section is separate from the pavement structure and is constructed in the pavement median. Test section was completed and monitoring studies are conducted at this location. This section is not paved and hence only surface elevations surveys are being conducted.

All field sections are monitored for a period of two years and pavement performance assessments of the test sections is reported.

2.1 Task 1: Collect Field Data and Monitor Performance: Test Section 3

Once the test sections were compacted and pavement is constructed, monitoring of the test section was initiated in August, 2014. Available bench mark on the opposite road side 500 feet away from Test section 3, is utilized in determining the elevation of data points in test section. Along the test section, 12 data points were marked with the help of 1 foot long rebar and paint markings as shown in Figure 5. These data points were established to study possible heave mechanisms occurring in field with time and seasonal variations. Figure 5 presents the elevation data of the 12 test points located at Test section 3.

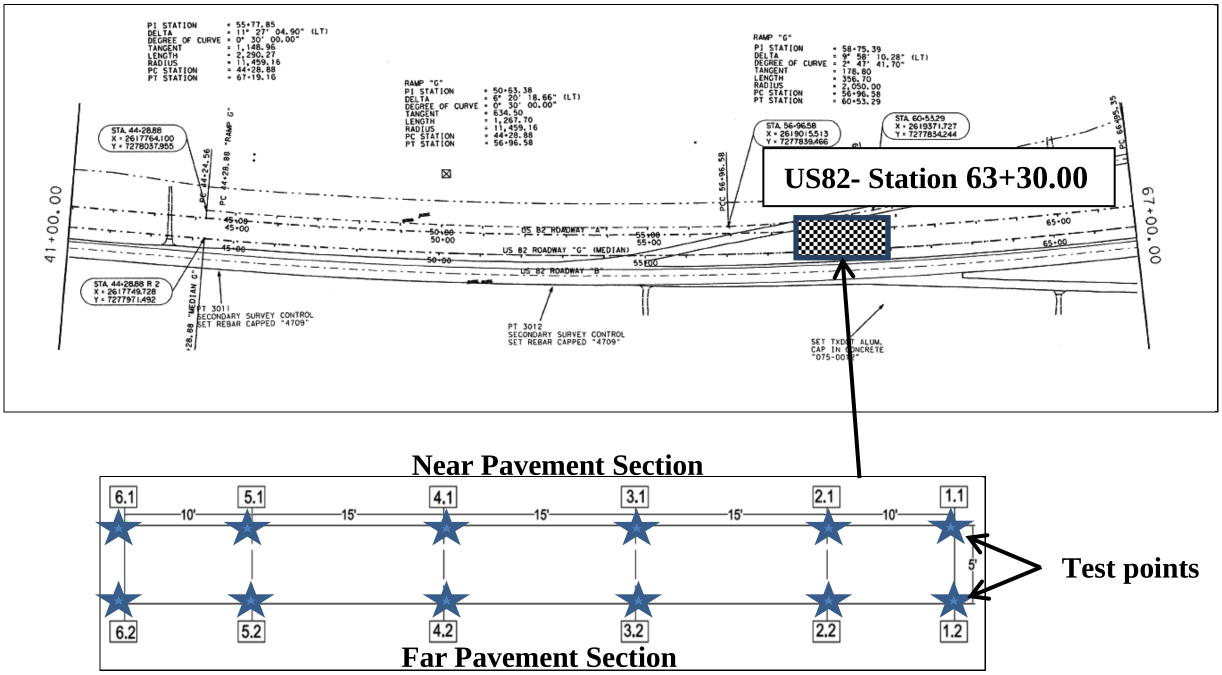


Figure 5: Layout of Section 63+00 to 63+60 (Test Section 3) at US82, TX

2.1.1 Elevation Survey at Test Section 3:

Test section 3 also known to be the control section is monitored for deformations occuring due to the formation of ettringite mineral. The variation of elevations of the test pins located at test section 3 are highlighted in Figure 6 which are recorded during the fiscal year of 2014, Figure 7 during fiscal year of 2015 and Figure 8 during the fiscal year of 2016. Deformations associated with elevation change are captured and analyzed.

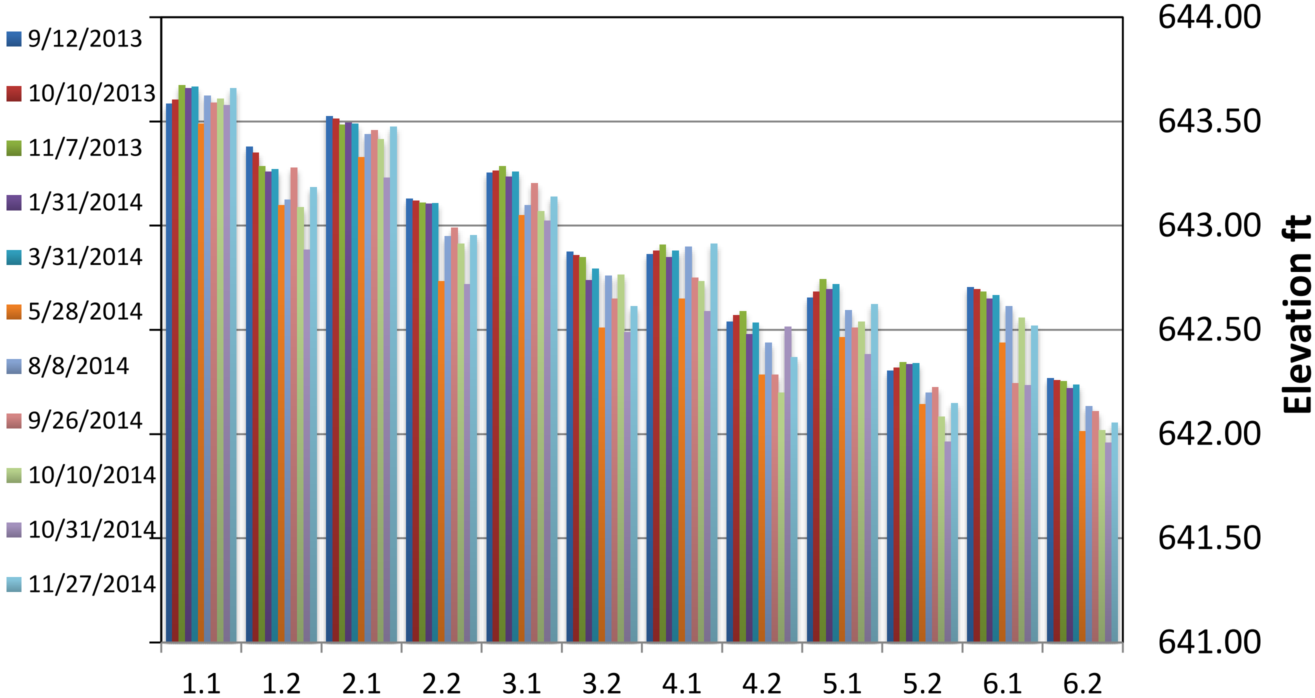


Figure 6: Elevations at test section 3 during the year of 2014

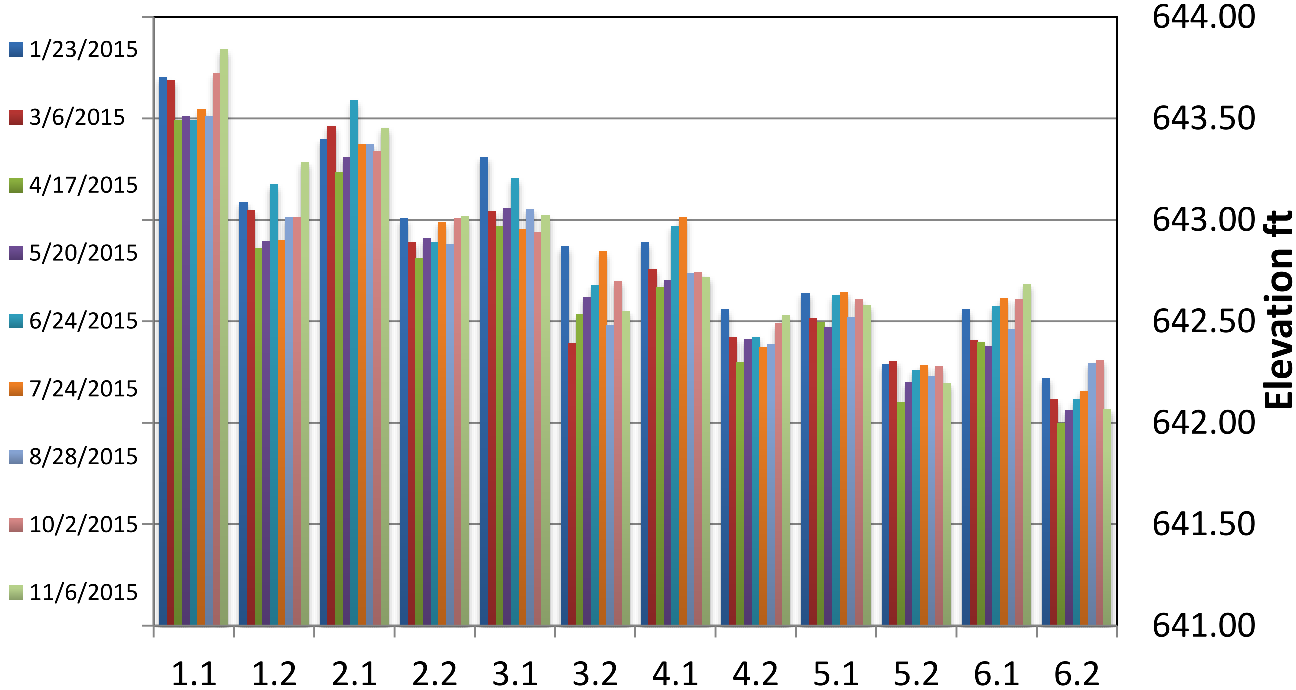


Figure 7: Elevations at test section 3 during the year of 2015

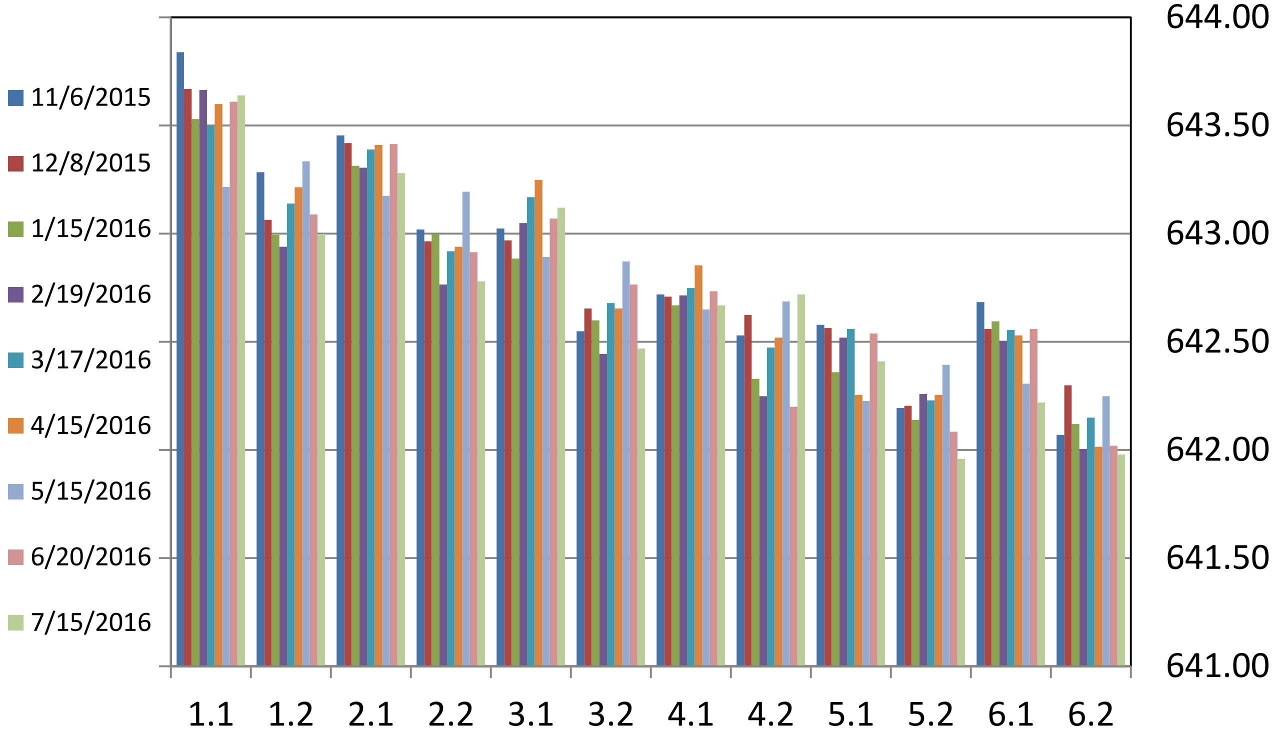


Figure 8: Elevations at test section 3 during the year of 2016

Details of the deformation trends are presented in Figure 9 for near pavement control section and Figure 10 for far pavement control section. A maximum swell of 3 inches is observed at near pavement control section and 0.48 inches for far pavement control section. A maximum shrinkage of 4.8 and 5.8 inches was observed for top and bottom control sections, respectively.

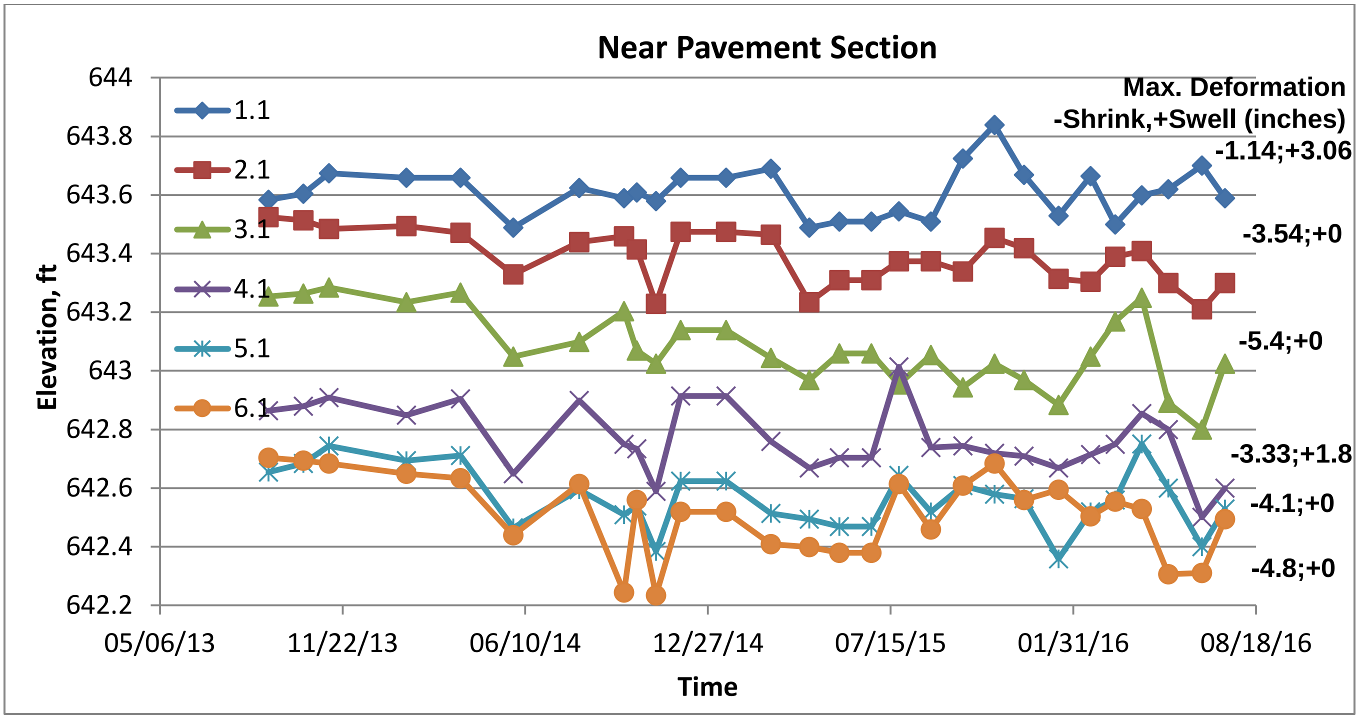


Figure 9: Deformations recorded at "Near" pavement section of test section 3

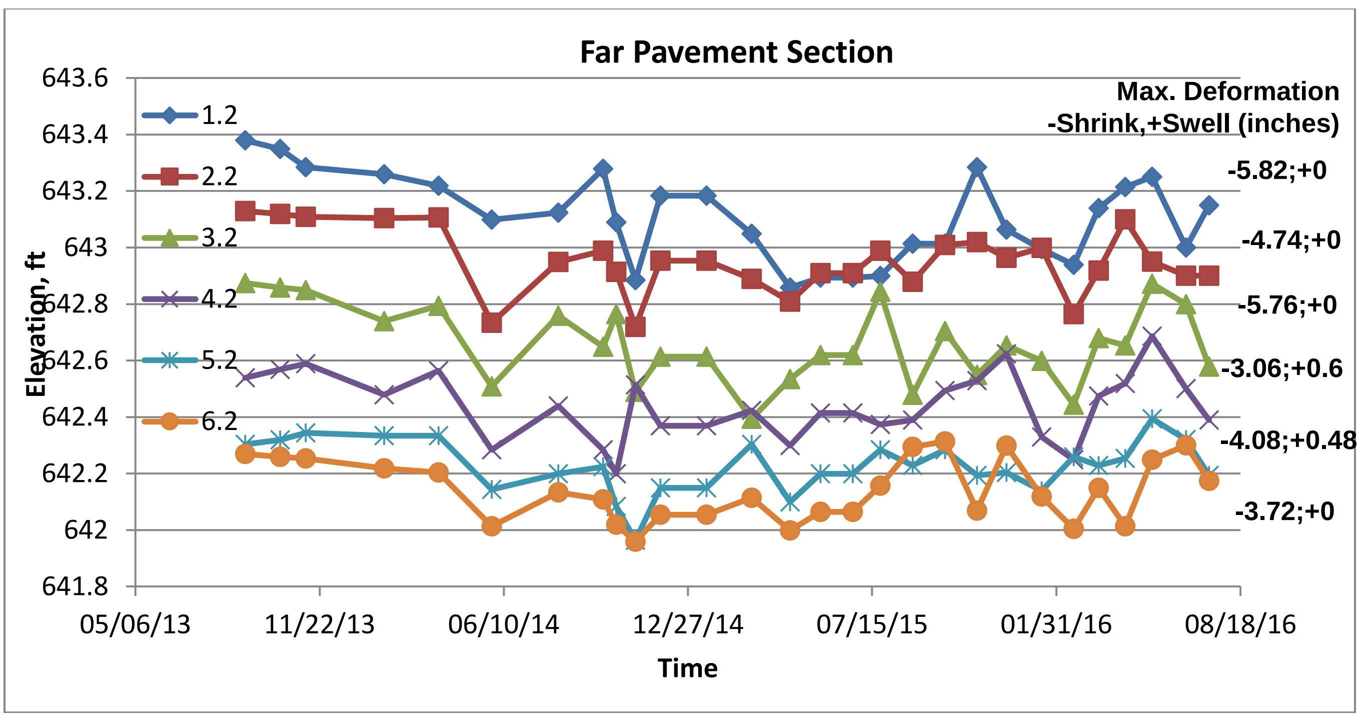


Figure 10: Deformations recorded at "Far" pavement section of test section 3

Much of the deformations recorded are associated due to the severe moisture fluctuation associated from varying climatic conditions. Rainfall data collected from Savoy, Texas is presented in Figure 11. A peak rainfall close to 12 inches has been observed during the month of May, 2015. Similarly, maximum and average temperatures recorded have been presented in Figure 12. It is observed that the far pavement section has been subjected to significant wash out due to the tremendous downpour of rainfall in June, 2015 and in the following recent months.

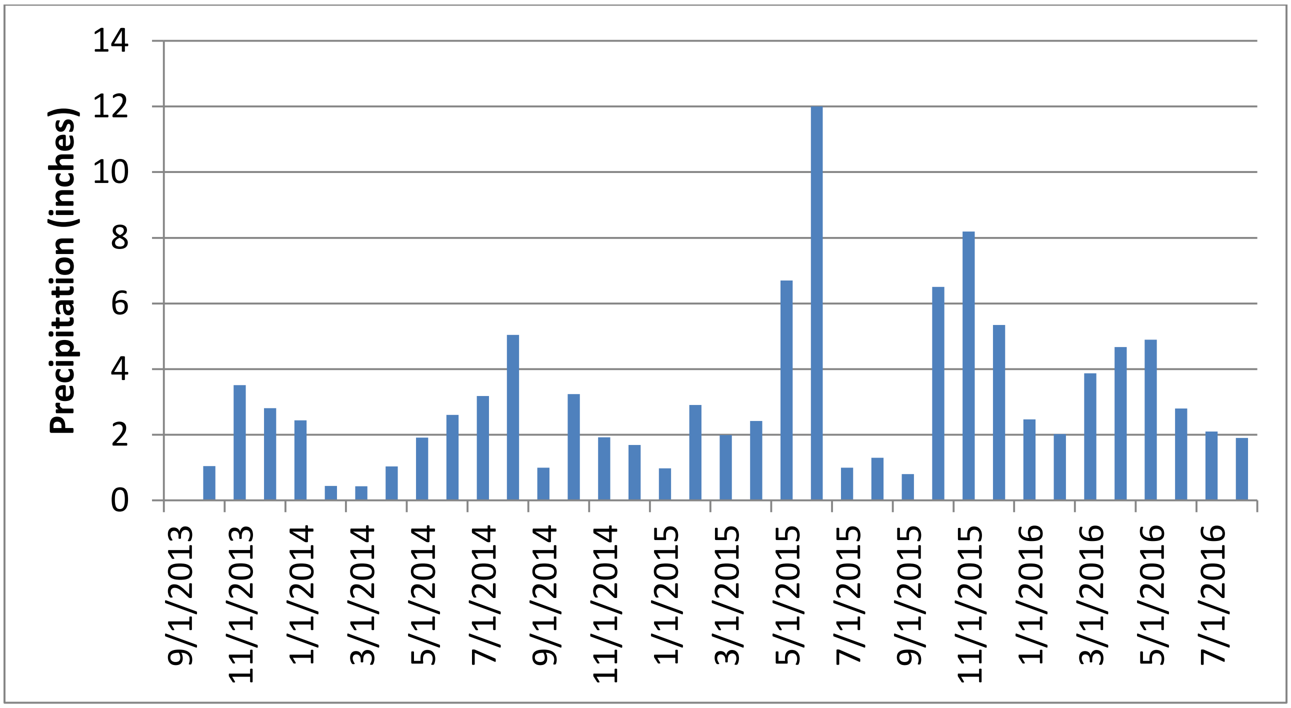


Figure 11: Precipitation at Savoy, Texas (near US 82 Highway)

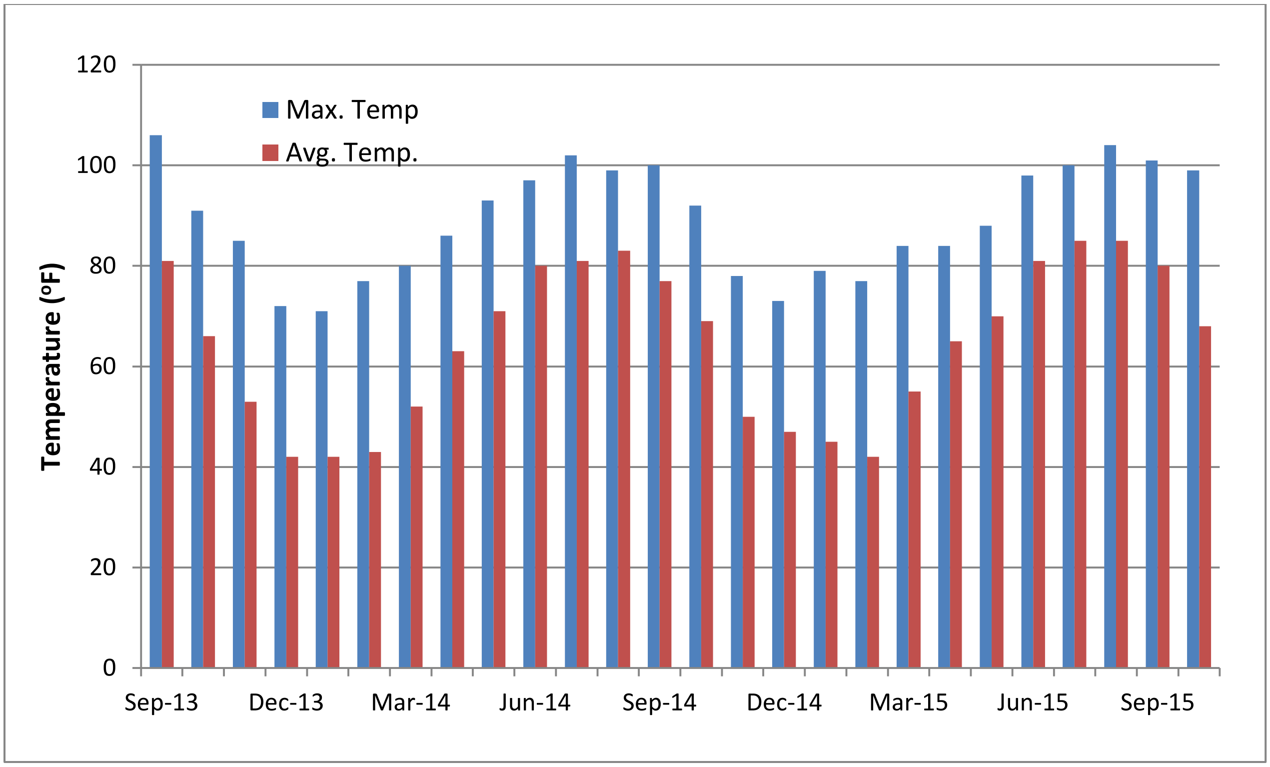


Figure 12: Temperature data at Savoy, Texas (near US 82 Highway)

Due to the climatic variant cycles and potential formation of mineral Ettringite, the control section exhibited large movements which were captured from elevation studies. Some of these movements and associated cracking in soils extending up to 1 ft. deep and 3 in. wide are shown in Figure 13 below.

Figure 13: Cracks observed at test section 3 (control section)

2.2 Task 2: FWD and Profilograph Testing – Stations 1 and 2:

2.2.1 Falling Weight Deflectometer (FWD) Studies:

Falling Weight Deflectometer (FWD) studies are conducted at Test sections 1 and 2. These studies assist in evaluating the subgrade and base modulus and integrity of the pavement. Figure 14 (a, b and c) provide the actual FWD testing process at the US 82 site. Modulus 6.0 software is utilized in the current study to analyze the test data and details of the analysis are provided in Figures 15 and 16. Figure 15 and 16 detail the back calculated modulus at two different time periods i.e. 2014 and 2016. It can be observed that the modulus of the treated subgrade did not vary with time. However, peak modulus of surface asphalt concrete showed a peculiar trend.

(a)

(b) (c)

Figure 14: (a) Falling Weight Deflectometer (b) Drop hammer and (c) Geophones

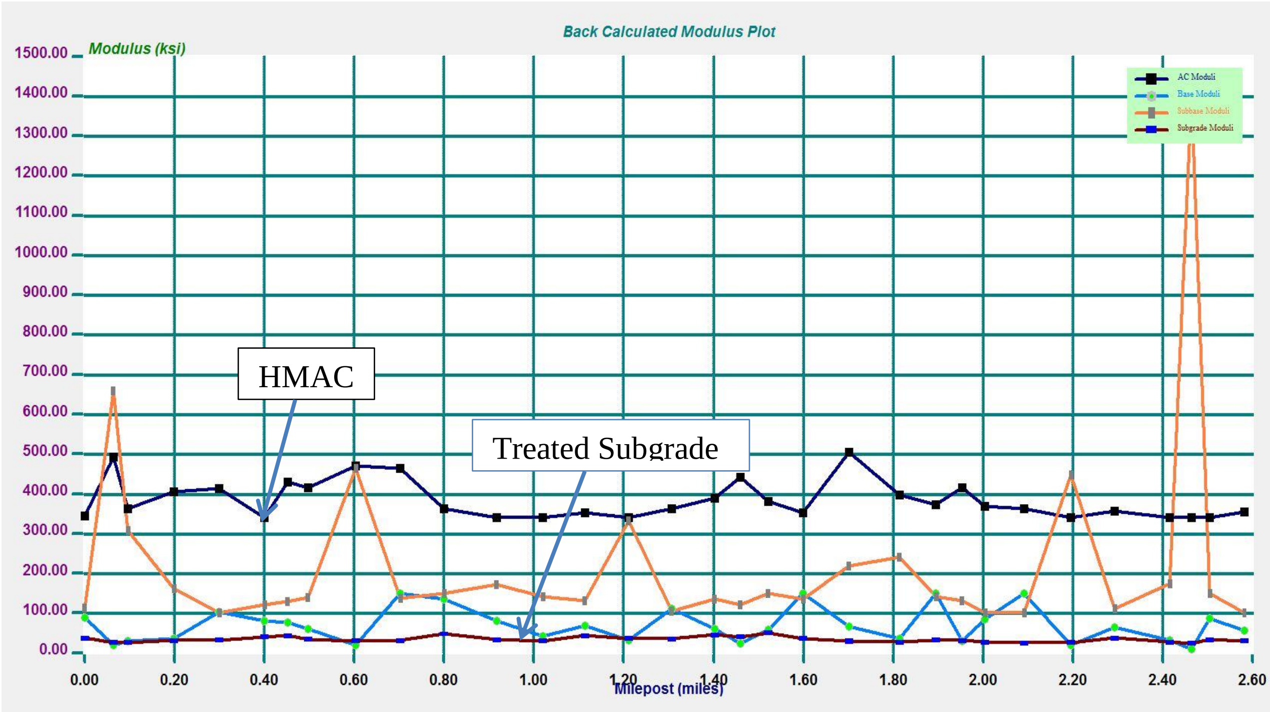


Figure 15: Variation of modulus in test section 1 on 6/15/2014

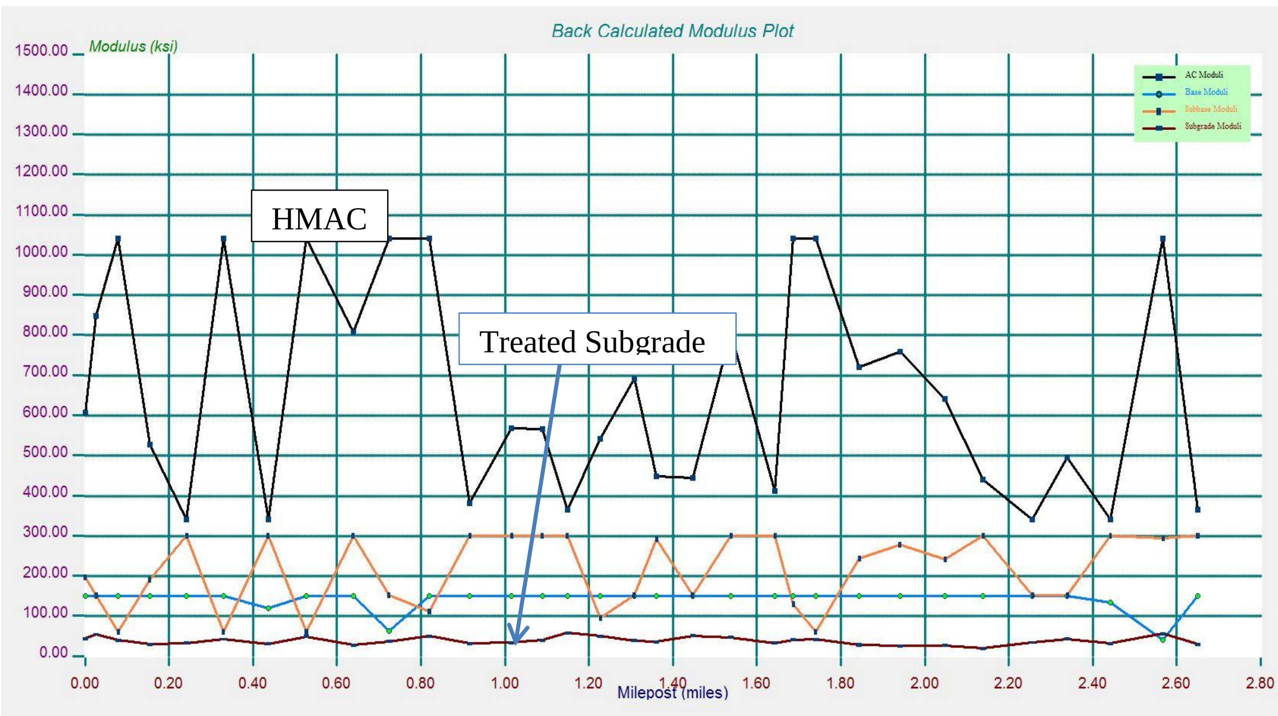


Figure 16: Variation of modulus in test section 1 on 7/15/2016

Table 2: Subgrade Modulus (ksi)

Date	Segmentation Number	From (mile)	To (mile)	Mean (ksi)	Std Deviation (ksi)
6/15/2014	1	0	2.6	38.4	1.2
7/15/2016	1	0	2.6	37.8	9.8

Table 2 provides the mean modulus of the subgrade and standard deviation obtained at test section 1 from the analysis. FWD analysis conducted in June 2014 revealed that the layer strengths and remaining years to be "Very Good". This means that the pavement is sound right after construction. Recent FWD analysis conducted in July 2016 revealed that remaining years columns are designated to be "Very Good" as shown in Figure 17. 10+ years in the final column of Figure 17 represent pavement is structurally sound and no load associated problems are anticipated. However, rut remaining life has been reduced to 2-5 years suggesting an imminent problem.

DISTRICT : 17--Bryan
COUNTY : 21--BRAZOS
HIGHWAY : FM2818
TEMPERATURE("F) start: 85.0 knd: 85.0
AVERAGE RUT DEPTH(in): 0.7
ALLIGATOR CRACKING : 25.0 %

ASPH. THICKNESS : ****
MONTH TESTED : JUL
DESIGN LOAD (lbs): 9000
20 year 18 KIP(m): 4.10
LANES : 2
SENSORS: 0 12 24 36 48 60 72

	** NORMALIZED DEFLECTION (mils) **					ASPH TEMP	DESIGN SCI	LAYER STRENGTHS			REMAINING LIFE (yrs)	
STATION	R1	R2	R3	R4	R7	DEF F	MILS	UPR	LWR	SGR	RUT	CRK
0.000	4.27	2.37	1.77	1.51	0.88	85.0	1.36	VG	VG	VG	2-5	10+
0.027	3.72	1.92	1.53	1.29	0.78	85.0	1.31	VG	VG	VG	2-5	10+
0.079	4.19	2.48	2.03	1.78	1.17	85.0	1.20	VG	VG	GD	2-5	10+
0.154	5.25	3.14	2.51	2.17	1.27	85.0	1.47	VG	VG	GD	2-5	10+
0.242	5.41	2.85	2.33	1.94	1.07	85.0	1.86	VG	VG	GD	2-5	10+
0.329	4.13	2.41	2.03	1.78	1.04	85.0	1.22	VG	VG	GD	2-5	10+
0.436	6.32	3.03	2.51	2.11	1.12	85.0	2.44	VG	VG	GD	2-5	10+
0.527	3.83	2.17	1.79	1.56	0.98	85.0	1.18	VG	VG	VG	2-5	10+
0.639	4.60	3.12	2.54	2.17	1.28	85.0	0.96	VG	VG	GD	2-5	10+
0.724	4.27	3.01	2.57	2.24	1.42	85.0	0.79	VG	VG	MD	2-5	10+
0.820	3.64	2.03	1.59	1.39	0.87	85.0	1.15	VG	VG	VG	2-5	10+
0.916	5.33	2.75	2.34	2.10	1.26	85.0	1.88	VG	VG	GD	2-5	10+
1.016	4.67	2.68	2.13	1.85	1.18	85.0	1.41	VG	VG	GD	2-5	10+
1.089	4.34	2.47	1.90	1.59	0.90	85.0	1.33	VG	VG	VG	2-5	10+
1.149	4.48	1.85	1.51	1.29	0.72	85.0	1.99	VG	VG	VG	2-5	10+
1.227	4.48	2.14	1.78	1.56	1.02	85.0	1.73	VG	VG	GD	2-5	10+
1.308	4.49	2.47	2.03	1.76	0.97	85.0	1.45	VG	VG	VG	2-5	10+
1.361	4.89	2.59	2.11	1.90	1.04	85.0	1.67	VG	VG	GD	2-5	10+
1.447	4.58	2.19	1.67	1.36	0.72	85.0	1.77	VG	VG	VG	2-5	10+
1.538	3.73	2.09	1.65	1.41	0.79	85.0	1.17	VG	VG	VG	2-5	10+
1.643	5.17	2.88	2.29	1.92	1.10	85.0	1.63	VG	VG	GD	2-5	10+
1.687	3.86	2.28	1.82	1.59	1.05	85.0	1.11	VG	VG	GD	2-5	10+
1.740	4.10	2.40	2.04	1.78	1.16	85.0	1.20	VG	VG	GD	2-5	10+
1.844	4.78	3.07	2.53	2.15	1.33	85.0	1.15	VG	VG	GD	2-5	10+
1.941	4.87	3.13	2.61	2.32	1.45	85.0	1.17	VG	VG	MD	2-5	10+
2.048	5.06	3.30	2.68	2.30	1.39	85.0	1.18	VG	VG	GD	2-5	10+
2.139	6.29	4.08	3.46	3.03	1.82	85.0	1.48	VG	VG	PR	2-5	10+
2.256	5.94	2.86	2.19	1.95	1.20	85.0	2.28	VG	VG	GD	2-5	10+
2.339	4.61	2.56	1.91	1.62	0.85	85.0	1.47	VG	VG	VG	2-5	10+
2.442	6.16	2.85	2.29	2.10	1.17	85.0	2.46	VG	VG	GD	2-5	10+
2.567	4.35	2.52	1.90	1.66	0.96	85.0	1.29	VG	VG	VG	2-5	10+
2.650	5.53	3.08	2.44	2.11	1.32	85.0	1.75	VG	VG	GD	2-5	10+
MEAN:	4.73	2.65	2.14	1.85	1.10		1.49	VG	VG	GD		
STD DEV:	0.75	0.48	0.42	0.37	0.24		0.41					
COF VAR:	15.81	17.97	19.62	20.22	21.94		27.49					

* 0~2:Failed 2~5:Problem 5~10:OK for Now 10+:Good

Figure 17: Snap shot of test results output using Modulus 6.0 software

2.2.2 Surface Profiler Studies:

A profiler is used to measure the true profile of the pavement. From this test two important parameters “Present Serviceability Index –PSI” and “International Roughness Index – IRI” are determined to analyze the health of the pavement. Ride Quality SPEC 5880 software from TxDOT was utilized to analyze the surface profile of the pavement. From raw input file, the software determines the cost of each distress in the pavement and calculates the total pay adjustment to the contractor. In the current study the software is utilized to determine the PSI and IRI values as they are representation of the pavement performance. Figure 18 presents the PSI and IRI summary recorded in Oct 1st 2014. These values are a representation of the pavement performance right after its construction.

Profile Length(Miles) 2.1057 Length(Station Units) 0111+18.1ft.

Distance	Station	PSI	IRI(L)	IRI(R)	Avg IRI	Pay*SectLen	Pay
00.1000	5+28.0	4.84	45.82	38.07	42.00	$360*(0.1000/0.10)	$ 360
00.2000	10+56.0	4.82	45.89	41.07	43.00	$340*(0.1000/0.10)	$ 340
00.3000	15+84.0	4.90	38.39	37.95	38.00	$440*(0.1000/0.10)	$ 440
00.4000	21+12.0	4.78	47.79	45.35	47.00	$260*(0.1000/0.10)	$ 260
00.5000	26+40.0	4.94	31.98	38.45	35.00	$500*(0.1000/0.10)	$ 500
00.6000	31+68.0	4.91	37.58	37.07	37.00	$460*(0.1000/0.10)	$ 460
00.7000	36+96.0	4.80	37.48	53.04	45.00	$300*(0.1000/0.10)	$ 300
00.8000	42+24.0	4.87	38.22	42.45	40.00	$400*(0.1000/0.10)	$ 400
00.9000	47+52.0	4.20	62.80	83.61	73.00	-$160*(0.1000/0.10)	-$ 160
01.0000	52+80.0	4.52	59.95	58.95	59.00	$ 20*(0.1000/0.10)	$ 20
01.1000	58+08.0	4.74	47.93	51.94	50.00	$200*(0.1000/0.10)	$ 200
01.2000	63+36.0	4.84	41.56	42.32	42.00	$360*(0.1000/0.10)	$ 360
01.3000	68+64.0	4.92	36.85	36.60	37.00	$460*(0.1000/0.10)	$ 460
01.4000	73+92.0	4.92	34.98	38.49	37.00	$460*(0.1000/0.10)	$ 460
01.5000	79+20.0	4.74	46.21	53.26	50.00	$200*(0.1000/0.10)	$ 200
01.6000	84+48.0	4.87	38.87	40.81	40.00	$400*(0.1000/0.10)	$ 400
01.7000	89+76.0	4.77	44.08	50.31	47.00	$260*(0.1000/0.10)	$ 260
01.8000	95+04.0	4.78	39.80	52.80	46.00	$280*(0.1000/0.10)	$ 280
01.9000	100+32.0	4.79	37.78	53.77	46.00	$280*(0.1000/0.10)	$ 280
02.0000	105+60.0	4.67	46.20	61.77	54.00	$120*(0.1000/0.10)	$ 120
02.1000	110+88.0	4.88	33.94	44.32	39.00	$420*(0.1000/0.10)	$ 420
						Pay Adjustment Subtotal	$ 6360

Ave Left IRI 42.6 Ave Right IRI 47.7 Ave IRI 45.15

Figure 18: Snap shot of the Ride Quality software output

For flexible pavements PSI is determined by the following equation:

$$PSI = 5.03 - 1.91\ \log(1 + SV) - 1.38\, RD^2 - 0.01\sqrt{C + P}$$

Where SV was the mean slope variance, RD was the mean rut depth and C and P were cracking and patching indices.

International Roughness Index can be determined and is represented by inch/miles. From the figure it can be noted that the IRI value is uniform throughout the test section except for an increase in roughness at 0.9 mile mark. These PSI and IRI indices will be monitored throughout the project period and utilized in performance analysis of pavement. The profiler data has been analyzed over a period of 05/05/2014 till 08/01/2016. A PSI value closer to 5 and IRI values less than 40 represent a healthy pavement.

2.2.2.1 Variation of Pavement serviceability Index (PSI) and International Roughness Index (IRI) from stations 0 to 100:

Surface profiler data obtained at different time periods was helpful to analyze the pavement performance before and after reconstruction. Initial profiler data from 05/05/2014 revealed the damaged pavement profile and the corresponding deformations at different stations can be observed from Figures 19 and 23. During the period of 08/01/2014, the pavement has been repaved and the PSI values varied from 4.5 to 5 as shown in Figures 17 and 20; representing a healthy pavement profile. Recent profiler data (09/01/2015) as presented in Figures 18 and 21, shows that the PSI and IRI values are exhibiting healthy trends since pavement construction, except at station 65+00 where the PSI value reduced to 3.9 and IRI value increased to 83. Figures 22 and 26 reveal that, the current condition of the pavement section is deteriorating, which could be attributed to the formation of Ettringite.

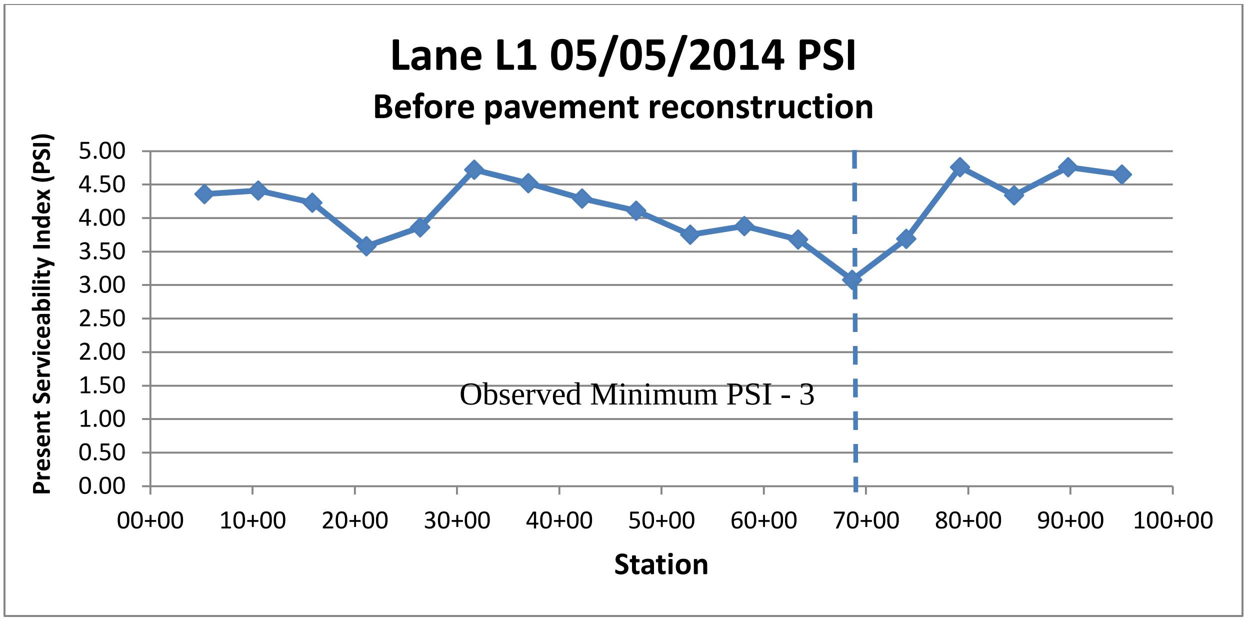


Figure 19: Variation of PSI at Stations 0 to 100 on Lane L1 on 05/05/2014

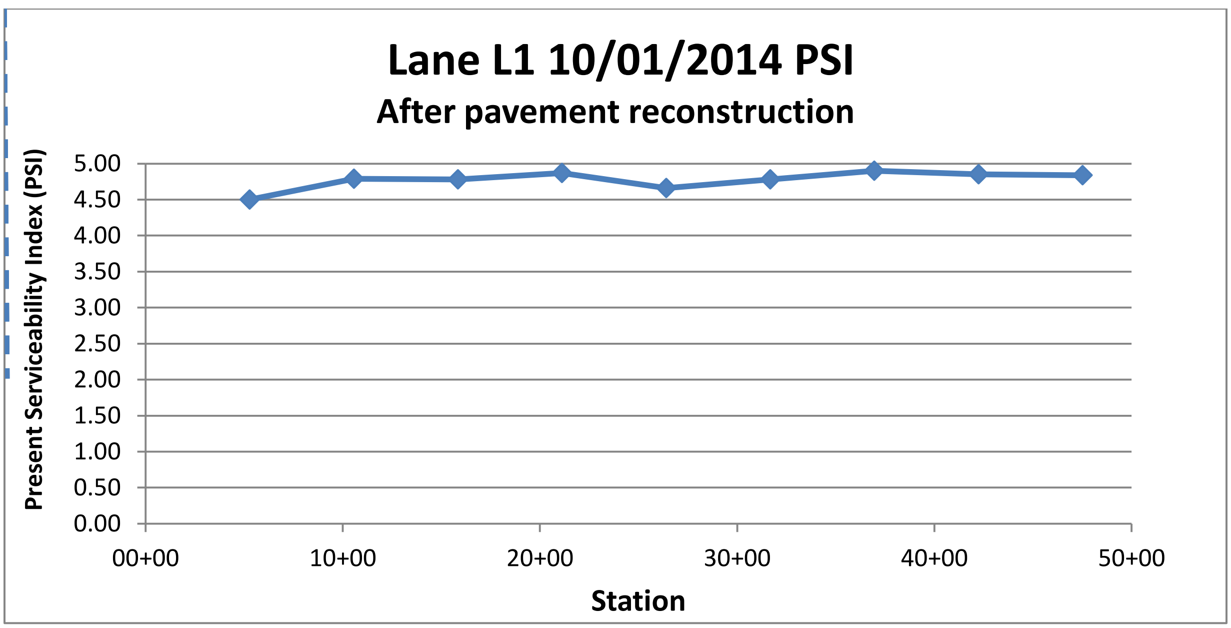


Figure 20: Variation of PSI at Stations 0 to 50 on Lane L1 on 10/01/2014

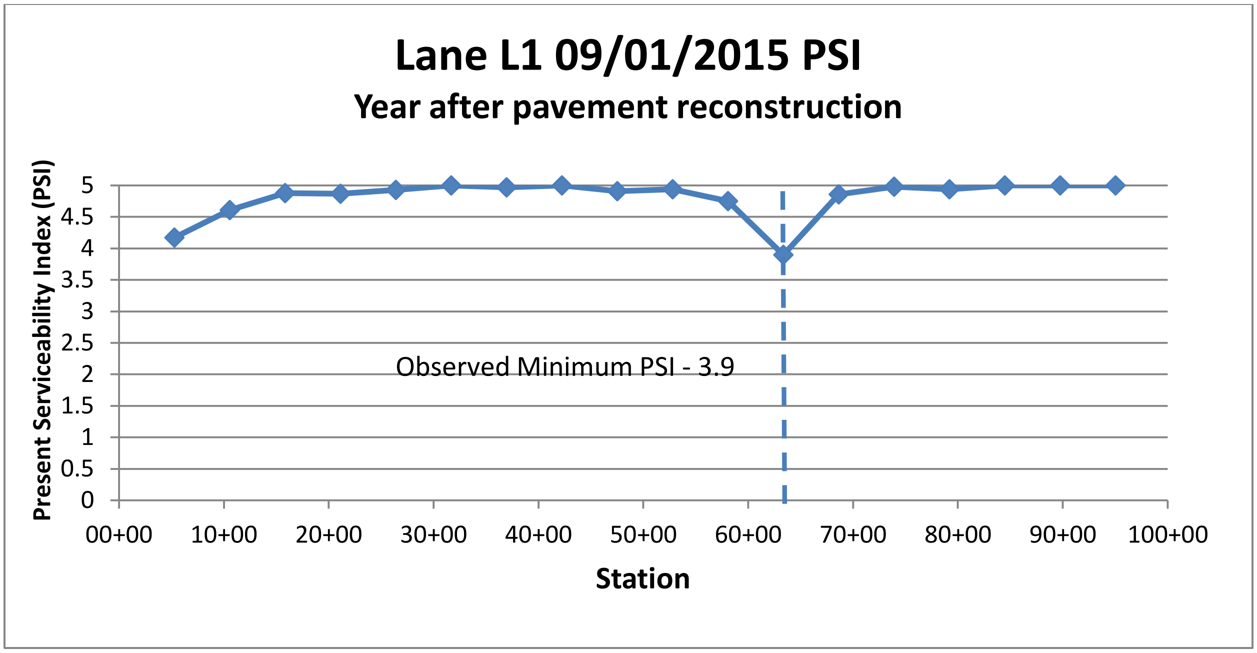


Figure 21: Variation of PSI at Stations 0 to 100 on Lane L1 on 09/01/2015

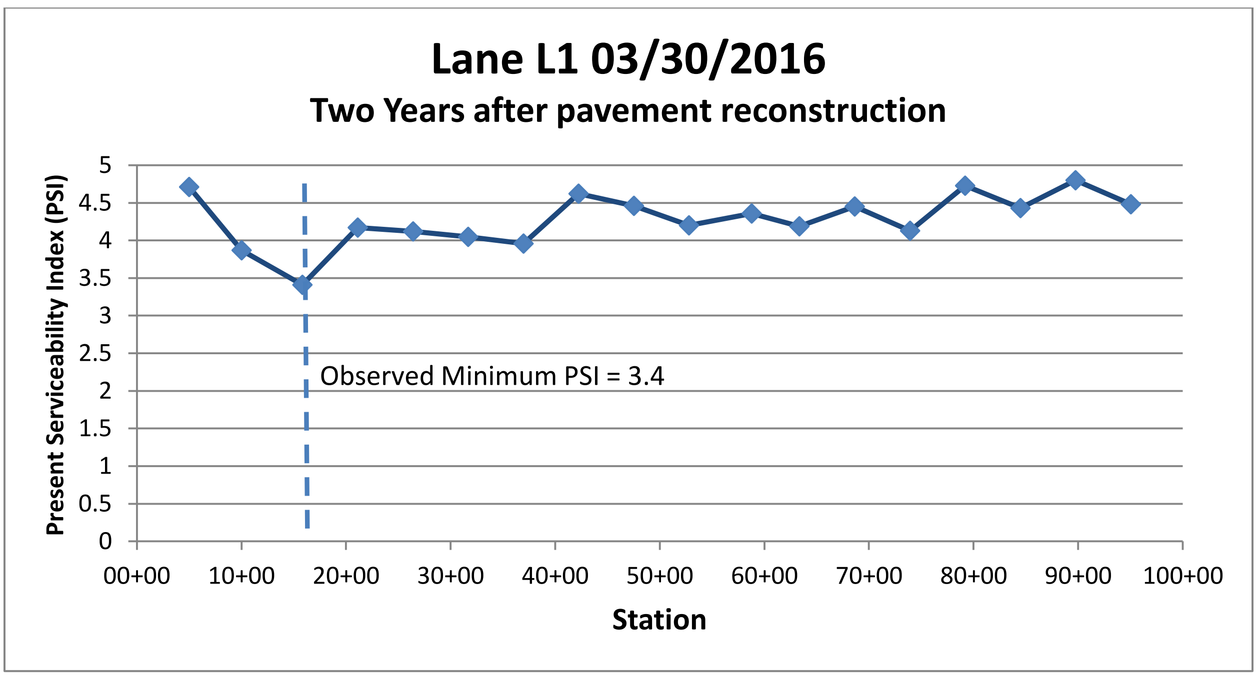


Figure 22: Variation of PSI at Stations 0 to 100 on Lane L1 on 09/01/2015

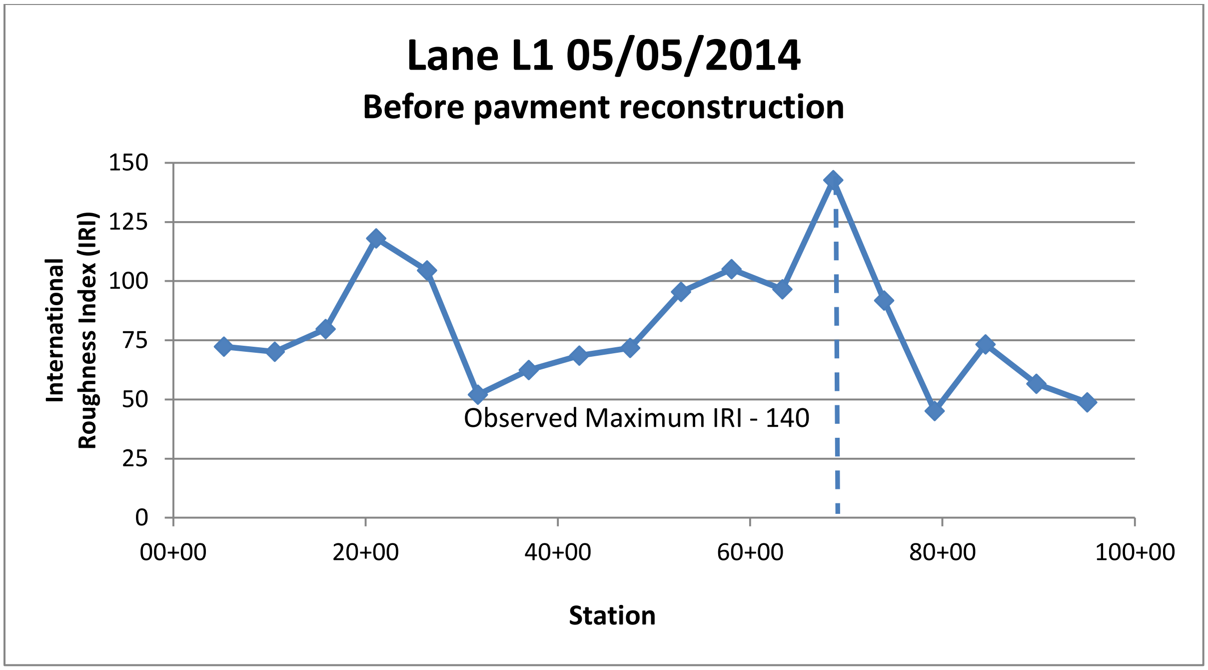


Figure 23: Variation of International Roughness Index (IRI) at Stations 0 to 100 on Lane L1 on 05/05/2014

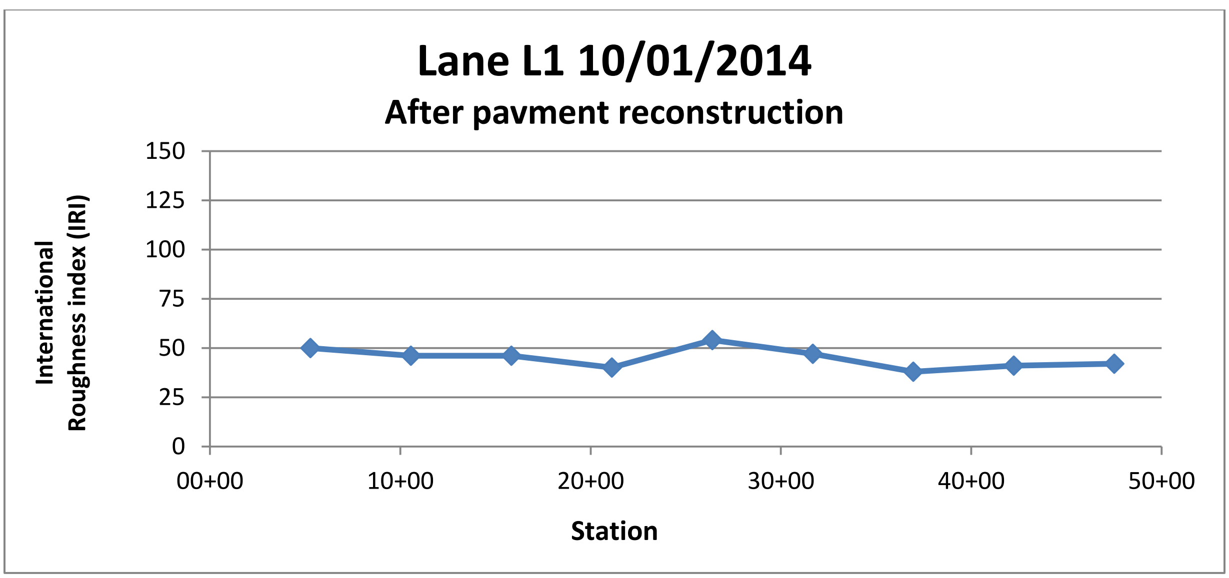


Figure 24: Variation of International Roughness Index (IRI) at Stations 0 to 50 on Lane L1 on 10/01/2014

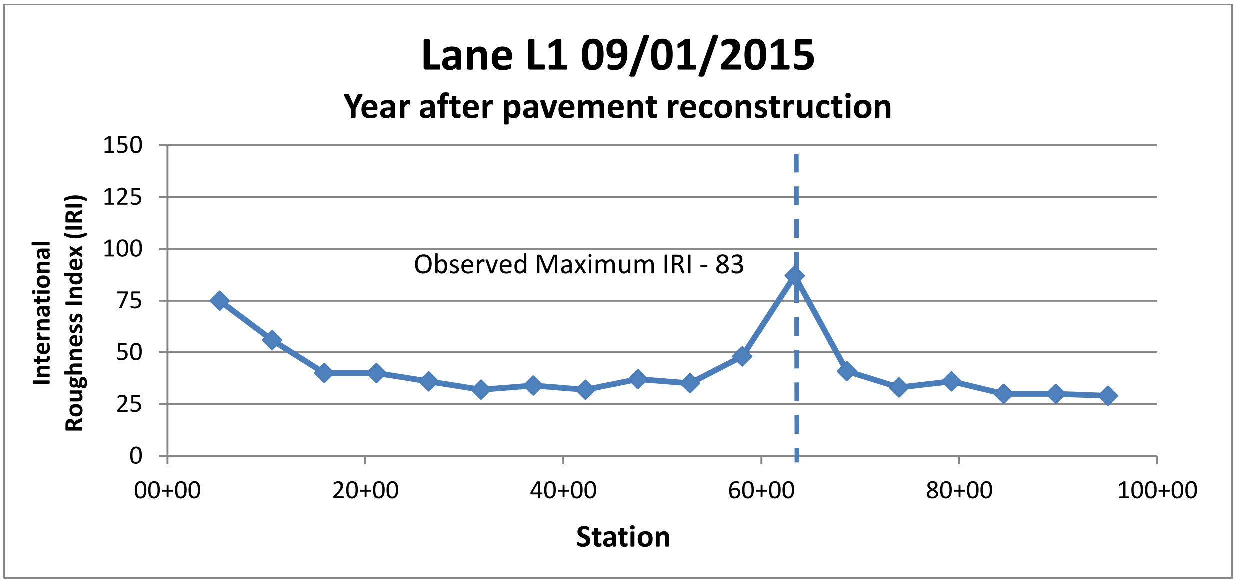


Figure 25: Variation of International Roughness Index (IRI) at Stations 0 to 100 on Lane L1 on 09/01/2015

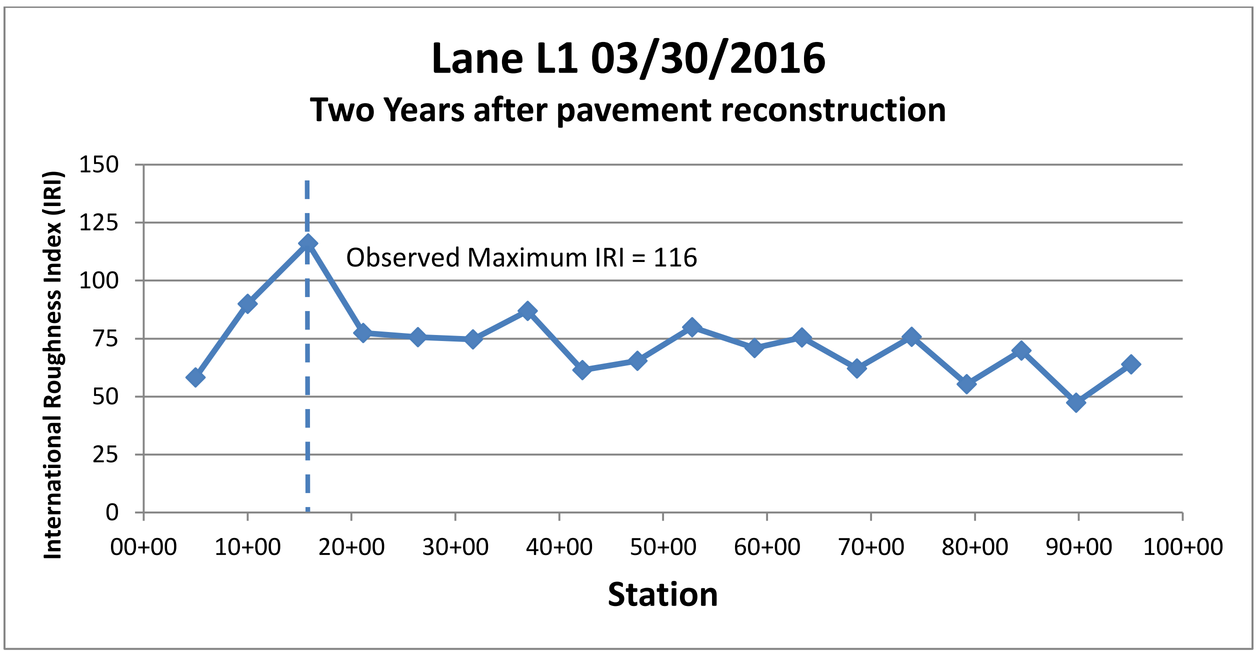


Figure 26: Variation of International Roughness Index (IRI) at Stations 0 to 100 on Lane L1 on 09/01/2015

Table 3 presents the variation of “Present Serviceability Index” recorded at different time periods. For Test section 1 and 2, a minute reduction in PSI values is observed with time.

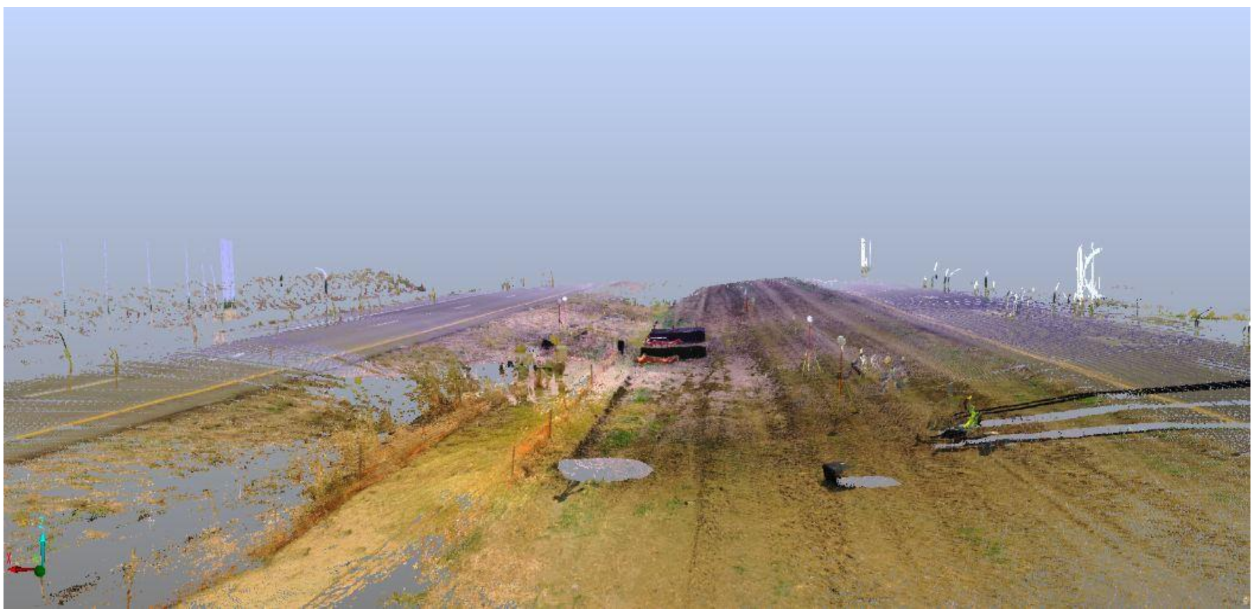
Table 3: Present Serviceability Index (PSI) values recorded during the monitoring period

	Test Section 1 (Lime + fly ash with extended mellowing)			**Test Section 2 (Lime with extended mellowing)**		
Station	89+00 to 170+00			88+00 to 89+00		
Date Tested	10/1/14	9/1/15	3/29/16	10/1/14	9/1/15	3/29/16
PSI recorded	4.8	4.76	4.15	4.77	5	4.63

2.1.2 LIDAR investigations:

LIDAR investigations are implemented at test section 3 to map the elevation and surface changes occurring due to Ettringite formation. FARO FOCUS series X 330 Laser Scanners were utilized to perform LIDAR investigations at the site as shown in Figure 27. X330 has a scanning range of 330 m and is equipped with GPS and remote scanning features. SCENE 5.4 software is used for registering and post processing the image files for construction of 3 D cloud space of the scanned project site as shown in Figure 28 (a) and (b). Constructed cloud space and associated data set are then transferred to builder application for further analysis.

Figure 27: LiDAR investigations at Test section 3



(a)

(b)

Figure 28 (a) (b) 3D reconstructed images of test section 3 using SCENE software

Initial scans showed considerable disturbances from the grass blades present at the site. LiDAR investigations are conducted at Test section 1 and 2 as shown in Figure 29 (a) and (b) below.

(a)

(b)
Figure 29 3D reconstructed images of test section 1 shown in (a) and test section 2 shown in (b) using SCENE software

Current study attempts to investigate the feasibility of LiDAR in evaluating the pavement test sections. However, due to limited time frame and availability a comprehensive pavement evaluation could not be conducted with these initial scans produced from LiDAR.

3: LABORATORY INVESTIGATIONS:

3.1 1-D Swell Strain Test

Swell strains of treated soil specimens collected from the field are determined using the standard procedure as shown in Figure 30 (b). Figure 31 provides the test results obtained from consolidation apparatus in the laboratory.

(a) (b)

Figure 30: Sample collection shown in (a) and laboratory swell test shown in (b)

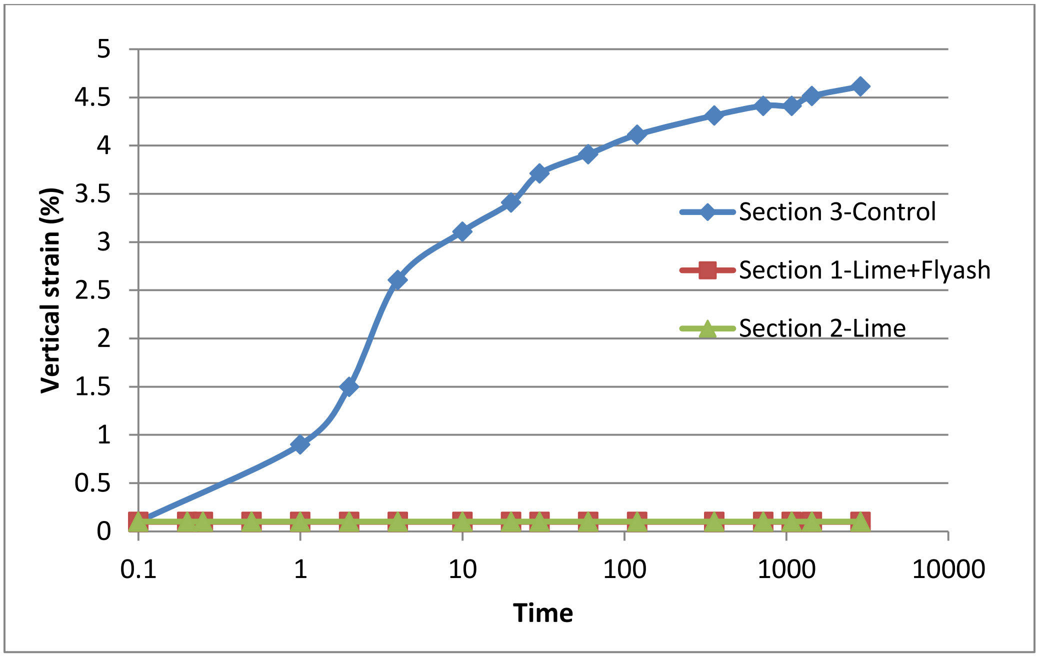


Figure 31: One Dimensional Swell test results during pavement reconstruction

It can be noticed that control soil with lime treatment has shown considerable vertical swell strain of 4.5%, whereas lime + fly ash treated subgrade with extended mellowing did not show any strains.

2.3.1 Three-Dimensional Free Swell Tests (Volumetric Free Swell Tests)

Three-Dimensional (3-D) free swell test measures the potential of the clay to swell in three (3) directions when soaked under water as presented in Figure 32. Volumetric strain underwent by the soil specimen is determined by measuring the vertical and radial swell strains. Two identical specimens compacted at their maximum dry density and optimum moisture content (OMC) are used for each test section and represented as OMC-1 and OMC-2. Figure 33 shows the vertical swell strain vs. elapsed time for the soils from all three test sections. From the results it is evident that soil from test section 1 (Lime + FA with extended mellowing) did not exhibit any swelling strains compared to soils from test section 3 (control section).

Figure 32: Three Dimensional Swell test

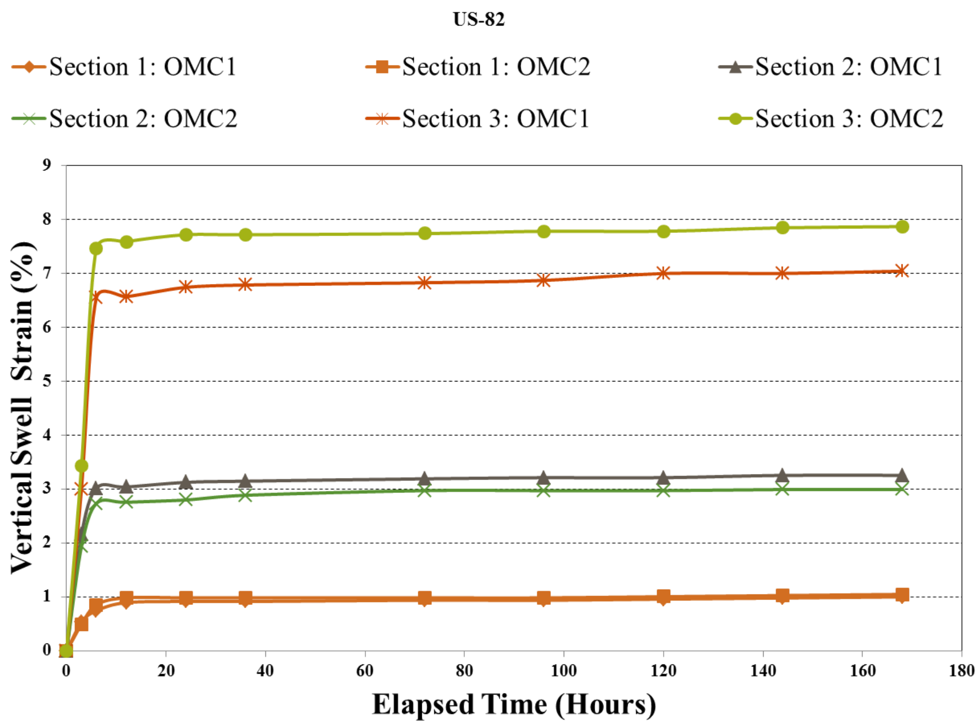


Figure 33: Summary of vertical swell strain results

Table 4 Vertical, Radial and Volumetric Swell Strains of all test section

	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC1	OMC2	OMC1	OMC2	OMC1	OMC2
Section 1	1.15	0.99	0.09	0.19	1.33	1.37
Section 2	3.21	3.05	0.62	0.67	4.45	4.39
Section 3	7.85	7.10	2.99	3.18	13.83	13.46

Table 4 presents the recorded swell strain measurements on samples retrieved from the test site. Three dimensional swell tests conducted at no confinement and complete saturation conditions represent the worst scenario possible to the soil at the field. It is observed that control soil from test section 3 has shown considerable volumetric swell strain of 13.5%, whereas lime + fly ash treated subgrade with extended mellowing and lime with extended mellowing soils exhibited least swell behavior which represents effectiveness in stabilization procedure.

3.2: Reactive Alumina and Silica assessment studies:

Reactive alumina (Al) and silica (Si) are the aluminum and silica present in amorphous or poorly crystalline Al/Si phases, including amorphous alumino silicate, organically complex alumina and hydroxyl-Al polymers present in montmorillonite interlayers. These measurements were important since alumina and silica constitute the compositions of Ettringite and Thaumasite respectively. Reactive alumina and silica measurements were conducted using Inductively Coupled Plasma-Mass Spectroscopy (ICP-MS) as shown in Figure 34. These measurements were conducted by a procedure modified after Foster (1953). To determine the reactive alumina and silica, 15gm of soil was mixed with 150mL of 0.5 N NaOH and boiled. After boiling, the solution was centrifuged at 8000rpm and filtered using a 0.1μm membrane type filter paper. The extract obtained was stored in a plastic bottle, and the ICP analysis was performed on the clear extract. ICP analysis requires a clear solution. If the resultant extract is dark colored, it could be due to organics or iron oxides in the soils. Organics can be removed by treating the solution with hydrogen peroxide (H_2O_2). It was reported that iron oxides (Fe_2O_3) coat the clay surface, which prevent clay from releasing alumina to react with lime to form pozzolanic compounds (Joffe, 1949). Iron oxides can be removed by treating 10 ml of the solution with 1mL of 6N HCl and agitating every hour. The solution is left overnight and filtered using 0.1μm membrane-type filter paper the next morning. Once a clear extract was obtained, ICP analysis was performed on the soil samples at different dilution ratios.

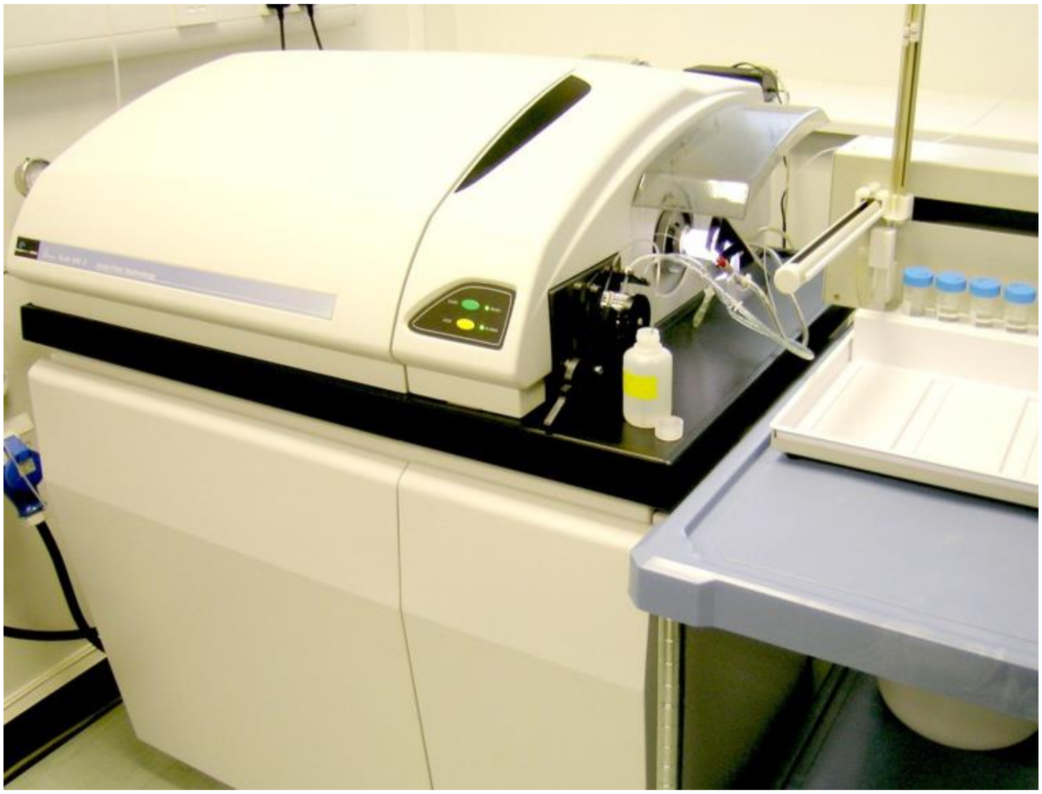


Figure 34: ICP-MS instrument for determination of Silica and Alumina levels

From initial assessment studies prior to construction of US 82, the soils comprise around 323 ppm of reactive alumina (Al) and 187 ppm of reactive silica (Si). During construction/mellowing process the soil is allowed to react with lime and fly ash thereby consuming the available Al and Si in order to accelerate the formation of Ettringite mineral.

From recent investigations conducted on April 2015, five core samples were retrieved from US82 pavement section where cracking and heaving is observed. The cores are numbered sequentially beginning at the East End #1 and towards Westbound #5 as shown in Figure 35. Core 1 is within test Section 1 (where lime + FA treatment is conducted), whereas, cores 2, 3, 4 and 5 are collected from pavement with conventional lime with 3 day mellowing construction. Table 5 presents the reactive alumina and silica measurements obtained from the core samples retrieved.

Figure 35: Core samples retrieved from US 82 pavement site

Table 5: Reactive Alumina and Silica measurements

Location	Reactive Al (ppm)		Reactive Si (ppm)	
	Mean	St. Dev	Mean	St. Dev
C1	174.8	0.6	169.1	5.3
C2	143.8	0.3	126.2	0.8
C3	47.6	0.2	113.4	1.4
C4	118.3	0.3	158.7	1.2
C5	227.3	1.2	68.6	0.3

Table 5 summarized reactive alumina (Al) and silica (Si) levels of the samples retreieved from five coring sites along US 82 pavement section. All locations have lower reactive Al levels compared to initial reading of 323 ppm prior to construction (TxDOT 6618 report). Similarly, silica levels at locations C3, C4 and C5 decreased from their initial reading of 187 ppm. This shows that portion of the available Al and Si might have been consumed towards Ettringite formation prior to construction. However, at locations C1 and C2, reactive Si levels increased to

269 and 226, respectively. This could be attributed to the addition of fly ash during the stabilization process near test section 1.

4.0 SUMMARY AND RECOMMENDATIONS

This final report summarizes the field monitoring studies conducted at the three treated sections on US 82 highway. Control section (Lime with 3 day mellowing - test section 3) showed considerable deformations where cracks that are a feet deep and three inches wide are observed and associated movements are recorded. Investigations identified potential heave areas within the test sections occurring due to Ettringite formation. Reactive alumina and silica levels retrieved from cored samples are further investigated in order to assess Ettringite formation at heave areas. Reduced silica and alumina levels from the core samples revealed the formation of Ettringite.

From monitoring studies, it can be concluded that novel construction techniques lime with fly ash and lime treated sections with extended mellowing performed better compared to traditional construction with 3 day mellowing period. A breif overview of the construction and design guidelines adopted for this implementation has been presented in Table 1 and other sections of this report. The benefits of novel construction technique and associated pavement performance include reduced maintenance costs, better pavement ride and safety. These stabilization solutions could prolong the life of pavements by 25 to 40 % built on high sulfate soils with minimal distress and this significantly reduces maintenance costs for each district. Also, safety will be enhanced as roughness due to heave bumps will be drastically reduced. All these lead to significant improvements to pavement infrastructure assets currently serviced by TxDOT.