Assessment of Tri-Dyne Precast Concrete Panels

By

Kevin Gaspard, P.E.
Senior Pavement Research Engineer

LTRC Project Number 08-2TA

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center
4101 Gourrier Ave
Baton Rouge, LA 70808
www.ltrc.lsu.edu

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Introduction and Literature review

Tri-Dyne Industries has developed precast concrete paving slabs (PCPS) that connect using tongue and groove joints and overlap joints, as shown in Figure 1 (1). This proprietary system is referred to as the "Pro-Active Paving System™" and consists of a series of steel reinforced PCPS containing valves that allow fill material to be injected through the PCPS into voids created by subsidence or base course/subgrade deterioration. Advantages cited by Tri-Dyne for this system are: 1) Subsidence in the pavement structure can be quickly abated by injecting leveling material into the subgrade, 2) multiple subsidence events can be addressed, as the injection ports are reusable with the appropriate injection material, 3) PCPS can be removed and replaced with cranes, allowing for repairs to the subgrade or underlying utilities, should they be present, and 4) PCPS are stronger than conventional pavements and, as a result are longer lasting.

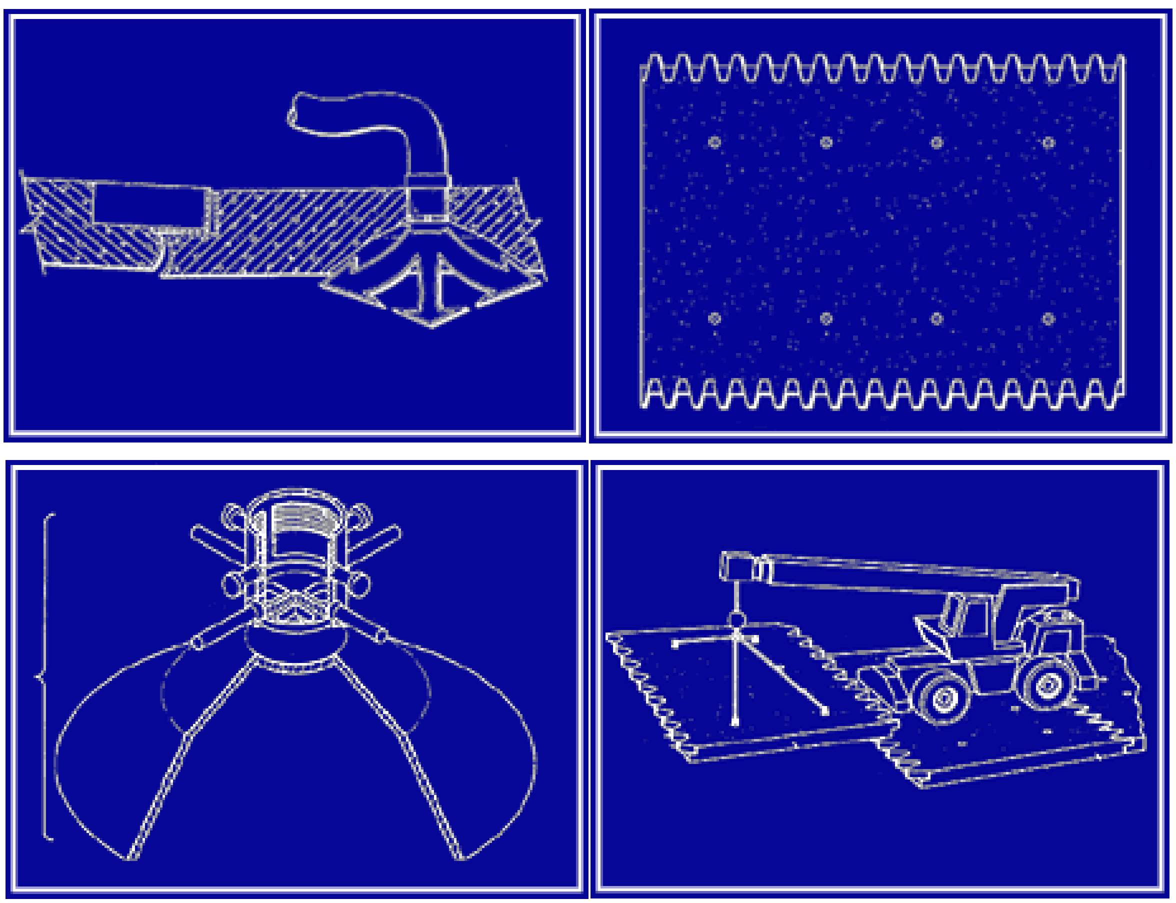


Figure 1 Tri-Dyne PCPS (source Tri-Dyne)

Tri-Dyne PCPS is in the developmental phase and has never been constructed on roadways. Tri-Dyne Industries requested in July of 2005 that the Louisiana Department of Transportation and Development (DOTD) assess the prototype Tri-Dyne PCPS to determine wheteher they would be suitable for use on roadways. The project did not begin until January of 2007, due to circumstances caused by Hurricane Katrina.

Tri-Dyne constructed a test section consisting of six PCPS panels with cast in place approach slabs at the Hanson concrete plant entrance in New Orleans, LA. This report summarizes the assessment of that test section conducted by the Louisiana Transportation Research Center (LTRC).

Literature review:

Precast concrete panels have been successfully used in bridges and buildings. AASHTO has created a (PCPS) technology implementation group to establish a data matrix for PCPS systems constructed on roadways. Five systems, KWIK slab, precast/prestressed concrete pavement (PPCP), Super-Slab system, precast full depth replacement/dowel bar retrofit method (PFDR/DBR), and Uretek USA stitch-in-time, have been identified and discussed in its recent publication (2). Technically speaking, Uretek's stitch-in-time refers to a joint mechanism and is not a PCPS.

PCPS systems can be divided into three major groups: 1) PCPS systems that utilize prestressed concrete panels that are post tensioned once they are installed (PPCP); 2) PCPS systems that utilize grouted joints once installation is complete (KWIK, Super-Slab, PFDR/DBR,); and 3) PCPS systems that utilize nondoweled overlap or tongue and groove joints (Tri-Dyne). All systems make provisions for leveling the PCPS slabs and for void filling once slabs are in place, since, it is practically impossible to build a PCPS to project specified grade or without a void. In some instances, grinding is utilized in an effort to meet the smoothness specifications of the contracting agency or enterprise.

There is a limited amount of published research work on PCPS systems constructed on roadways, and the research publications can be placed in two categories: patching with repair and full lane construction. This report focuses on full lane construction, as the Tri-Dyne tongue and groove joint PCPS are primarily marketed for that purpose. PCPS systems are promoted to have the advantages of quick placement, minimal slab curling, improved material properties, decreased user cost, and durability (3)(4)(5)(6)(7) (8)(9)(10). The major disadvantages cited by the previous sources of PCPS are high initial cost, joint load transfer efficiency, rough profiles, joint faulting, subgrade preparation, base course preparation, grinding, and leveling with a grouting material such as cement slurry or polyurethane.

In the 1960s, South Dakota constructed a 1000 ft. test section of prestressed PCPS that was overlaid with asphaltic concrete (AC) prior to being opened to traffic (3) (4). As with jointed concrete pavement, the major problem with this system was reflective cracks through the AC from the PCPS joints. Japan experimented with several types of joint configurations in prestressed PCPS (5)(6)(7). The systems produced no faulting and were performing well at 9 to 13 years of service.

Merrit, McCullough, and Burns report that, without factoring in user costs, the cost of the prestressed PCPS and typical continuously reinforced concrete pavement (CRCP) in Texas was $170 and $40 per square yard, respectively, for a 2,300 ft section (9). This means that the PCPS pavement cost approximately 465 percent more than CRCP. They also reported that the prestressed PCPS achieved IRI values ranging from 147 to 165. Texas elected not to grind the pavement to meet its ride specifications.

In an interim report recently published by the Minnesota DOT, the fact that, without contract incentives for quick PCPS placement, cast in place slabs with a three day cure time could have been installed within the same time frame as the PCPS system (Super-Slab) was noted (10). Furthermore, based on the construction of a 12 ft. wide, 216 ft. long section, the costs of the PCPS (Super-Slab) and standard concrete panel rehabilitation (MNDOT Type D-1 repair) were $165,805 and $21, 656, respectively.   This means that the PCPS system (Super-Slab) cost more than 765 percent of standard rehabilitation on Minnesota roadways. The IRI in the right wheel path at the time of construction was 150 and was decreased to 84 with diamond grinding.

Methodology

LTRC and Tri-Dyne work plans

A detailed test plan was co developed by Tri-Dyne and LTRC. The agreement was that LTRC would conduct testing with the FWD, Dynaflect, Walking Profiler, pressure sensors, video, and monitor placement of the tongue and groove joint PCPS and would write a report detailing the findings.

Tri-Dyne was to provide and construct the tongue and groove joint PCPS and base course materials, and to furnish and place the pressure sensors. Tri-Dyne placed the slabs in such a way that the non doweled over lap joints were in the transverse direction and the non doweled tongue and groove joints were in the longitudinal direction. They used a light sandy clay (AASHTO A-7) with a liquid limit and plasticity index of 45.3 and 25.5, respectively, to level the slabs. This material does not conform to LADOTD's flowable fill specifications. The subgrade was a sandy loam (AASHTO A-2-6) with a liquid limit and plasticity index of 28.7 and 12, respectively.

Tri Dyne PCPS

Figures 2 through 6 present details for 14 ft. by 16 ft. PCPS and integral curb. The PCPS used on this project were similar, except that the dimensions were 13.46 ft. by 15.5 ft. The center portion of the slab was 10.25 in. thick with steel reinforcement in the top, bottom, transverse, and longitudinal directions, as shown in Figure 2.

Two joint mechanisms were used. In the transverse direction, non doweled overlap joints, as shown in Figure 3, were used. According to Tri-Dyne, these joints were used in an attempt to allow quick placement of the slabs and do not employ dowel bars as load transfer mechanisms, as with typical cast in place pavements used in highway construction. Because of this, load transfer between slabs is possible from only one direction, as shown in Figure 7a. In the longitudinal direction, finger type tongue and groove joints were used, as shown in Figures 4 and 5. These joints connect through overlapping composite-steel-concrete elements and do not employ dowel bar load transfer mechanisms or tie bars to hold the slabs together. Unlike the overlap joint, this joint mechanism allows load transfer across the joint from both directions, as shown in Figure 7b. Further details are presented in the discussion of results section of this report.

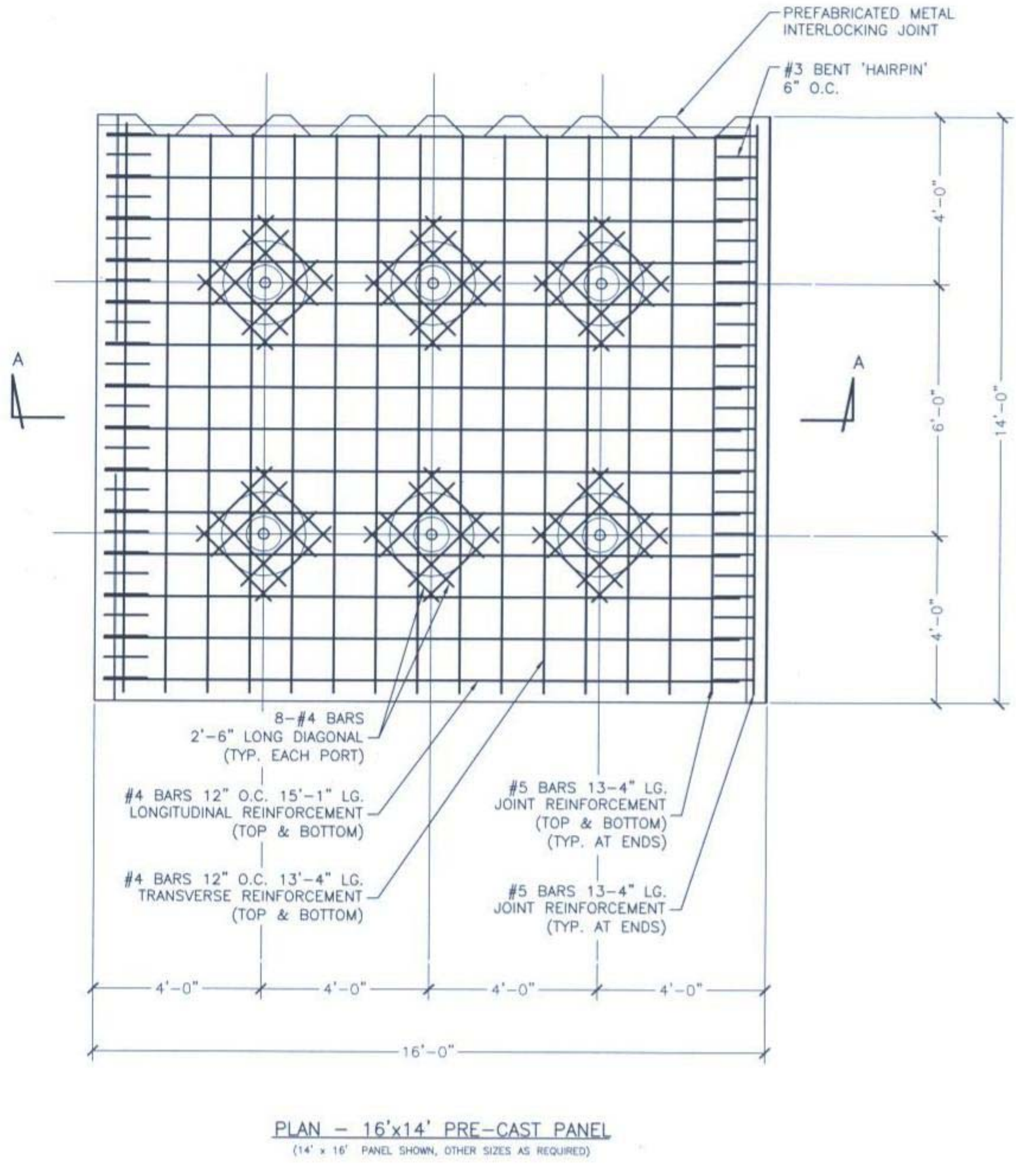


Figure 2: (source Tri-Dyne Industries)

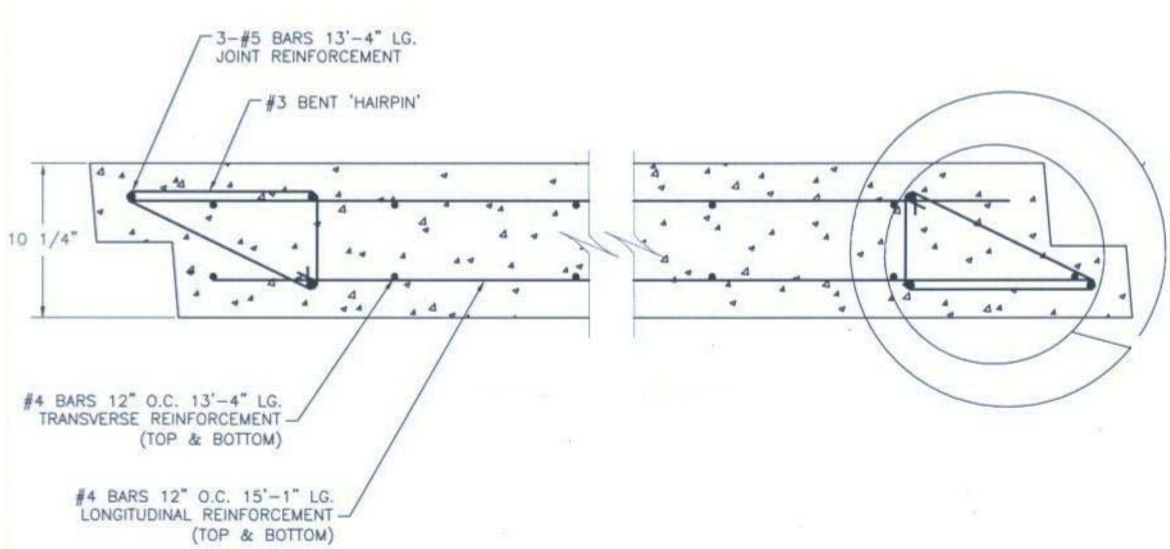


Figure 3: Overlap joint detail (source Tri-dyne)

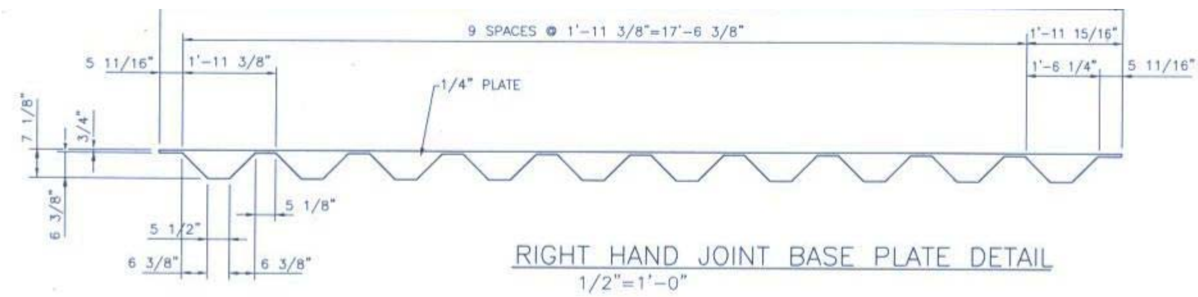


Figure 4: Tongue and groove "finger" joint bottom section (source Tri-Dyne)

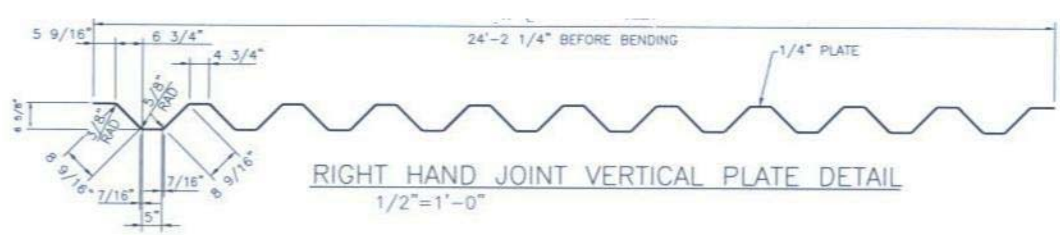


Figure 5: Tongue and groove joint top section (source Tri-Dyne)

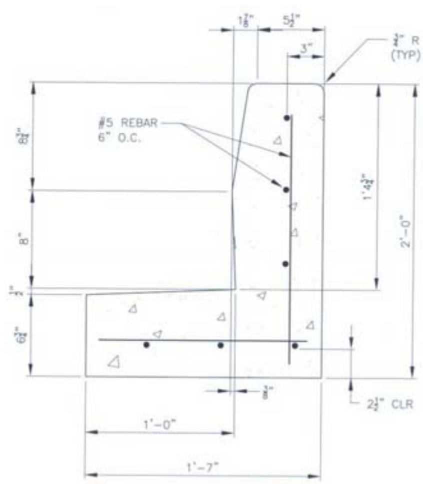


Figure 6: Curb detail (source Tri-Dyne)

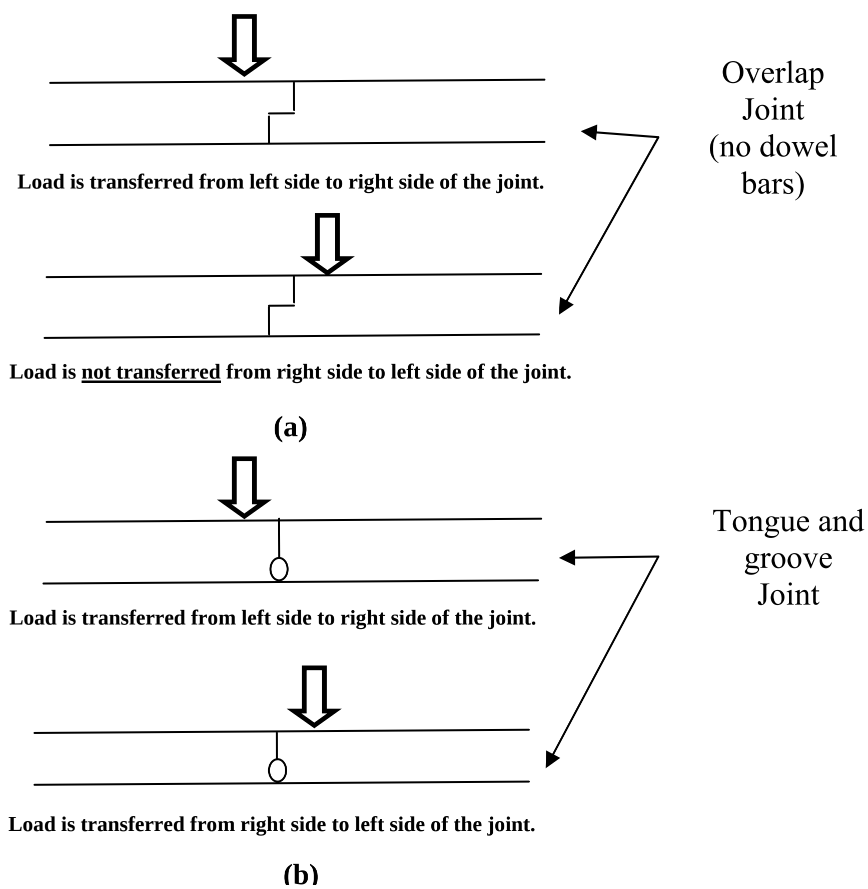


Figure 7: Load transfer detail for joints

Assessment of Precast concrete panels

Pavement performance on roadways is typically measured in two ways: functional and structural. Functional refers to indices such as rideability (IRI, joint faulting, joint width, etc.) and distresses (transverse cracks, joint spalling, longitudinal cracks, etc.). Structural refers to pavement strength and stiffness factors such as resilient and elastic modulus, tensile strength, unconfined compressive strength, and load transfer efficiency at the joints. The devices used to assess the functionality of the precast slabs were the walking profiler and manual fault measurements. Structural assessments were performed with FWD, Dynaflect, and pressure cells.

Functional assessment

Walking profiler: Four baselines, profiled in March and June of 2007, were established, as shown in Figure 8. In March of 2007, the slabs were placed directly on the base course without leveling or stabilizing the slabs, and in June of 2007, the slabs were leveled by Tri-Dyne. Data from the walking profiler was used to plot the profile and determine the IRI.

Though there currently are no DOTD ride specifications for any type of PCPS, as they have not been used on DOTD roadways, it would be reasonable to place them in the category of non-continuous paving, which has the least stringent ride specifications. The current DOTD specifications (601.11) for noncontinuous concrete paving require the IRI to be less than 115 for full payment. If the IRI is greater than 130, the slabs must be corrected or removed and replaced.

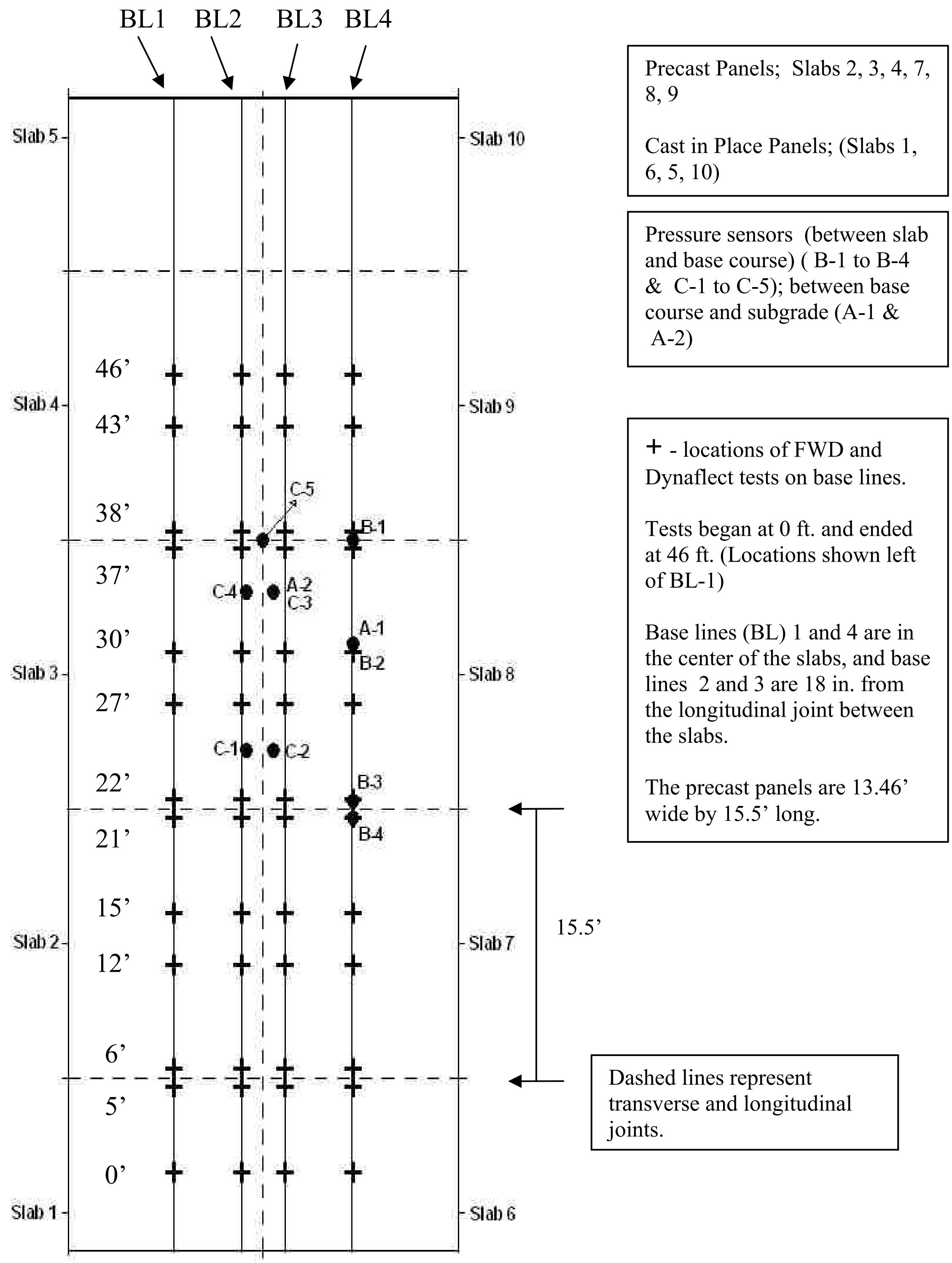


Figure 8: layout

Fault measurements: Faulting at the transverse and longitudinal joints was measured using a straight edge and tape.

According to Technical Bulletin 008.0 CPR, published by the Concrete Paving technology , faults in excess of 0.25 in. require immediate attention as shown in Table 1 (11). The concrete Paving technology is now known as the American Concrete Paving Association (ACPA). One method of providing immediate attention would be diamond grinding. In order for there to be no roughness caused by faulting, the fault should be less than 1/32 in. Setting an acceptance limit for faulting on newly placed PCPS of <1/32 in. based on TB-008.0 would be reasonable.

Table 1 Faulting index

Average fault (in)	Faulting index	Comments
1/32	5	No roughness
1/16	10	Minor faulting
3/32	15	Trigger grinding needed
1/8	20	Expedite project
5/32	25	
3/16	30	Discomfort begins
7/32	35	
1/4	40	Immediate attention required
Table from Technical bulletin (TB-008.0 CPR ,1990), Concrete Paving Technology		

Joint widths: When jointed concrete pavement is constructed, saw cutting is used to establish contraction joints. LADOTD specifies that these joints have widths less than 3/8 in.

Structural assessment

FWD: A structural evaluation was conducted with the FWD at the locations shown in Figure 8. Loads of 9,000, 12,000, and 16,000 lbs were used at each test point with three drops at each load. The data from the FWD were used to determine the layer moduli of the pavement structure, load transfer efficiency of the transverse joints, and voids. The first sensor deflection was used to assess the overall stiffness of the pavement structure and joint assessments. Because of the slab layouts and available space on the job site, assessments of the longitudinal joints was not conducted with the FWD. Load transfer efficiency of the longitudinal joints were inferred from the pressure sensors.

According to FHWA and ACPA sources, load transfer efficiency can be categorized into 3 groups, as shown in Table 2 (13)(14)(15)(16).

Table 2 Load transfer efficiency

Load transfer efficiency	Condition
> 70 %	Good
< 70% and > 50 %	Fair (needs mitigation)
< 50 %	Poor (needs replacement)

Layer moduli values were compared to typical values for concrete pavement (5,000 ksi) and stone base course (45 ksi), as published in TRB 1377 (12).

Void potential or loss of subgrade support is determined by plotting the deflection of the first sensor versus load and determining the Y intercept, as shown in Figure 9 (17)(18)(19). According to the AASHTO 1993 design guide, a Y intercept value greater than 0.002 in. (2 mils) represents either a void or a loss of support (17).

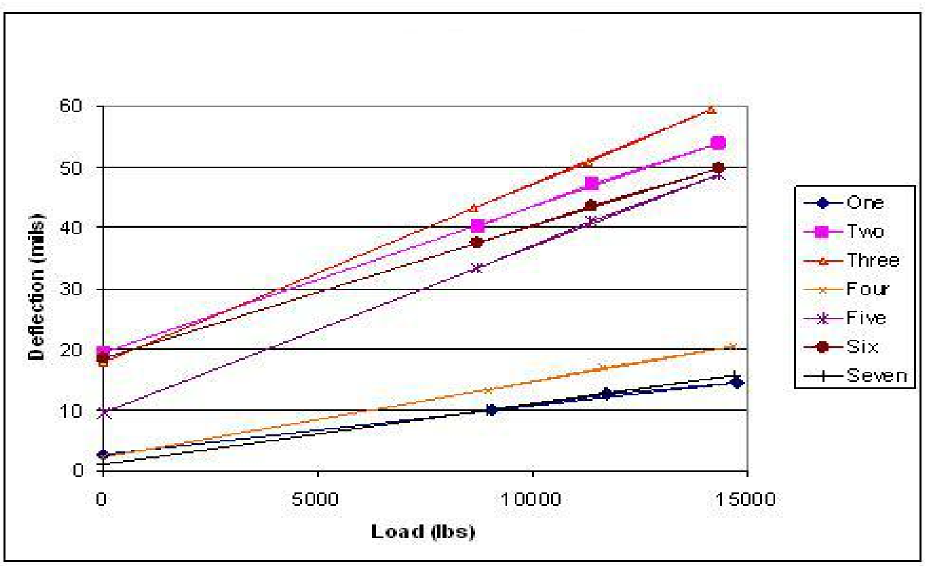


Figure 9: Y intercept diagram for void

Dynaflect: Testing was conducted with the Dynaflect at the same locations tested using the FWD. The data was used to determine the structural number of the locations tested.

Pressure cells: Geokon model 3500 dynamic stress pressure cells were used to determine load distributions throughout the slab by monitoring pressure changes at strategic locations between the slab-stone base course and stone base course subgrade interfacesm, as shown in Figure 8. Prior to placing any additional load to the slabs, the pressure cell values were recorded in an effort to establish an initial reading. Dead loads were placed on the slab by positioning the large cone truck at strategic locations. The axle distribution and loads per axle of the large cone truck are shown in Figure 10. Ten readings were taken at each location prior to advancement to the next location. The difference between the initial reading and additional readings indicated how the induced load was being distributed through the slabs and thereby provided insights as to the load transfer efficiency of the joints and strength of the slabs.

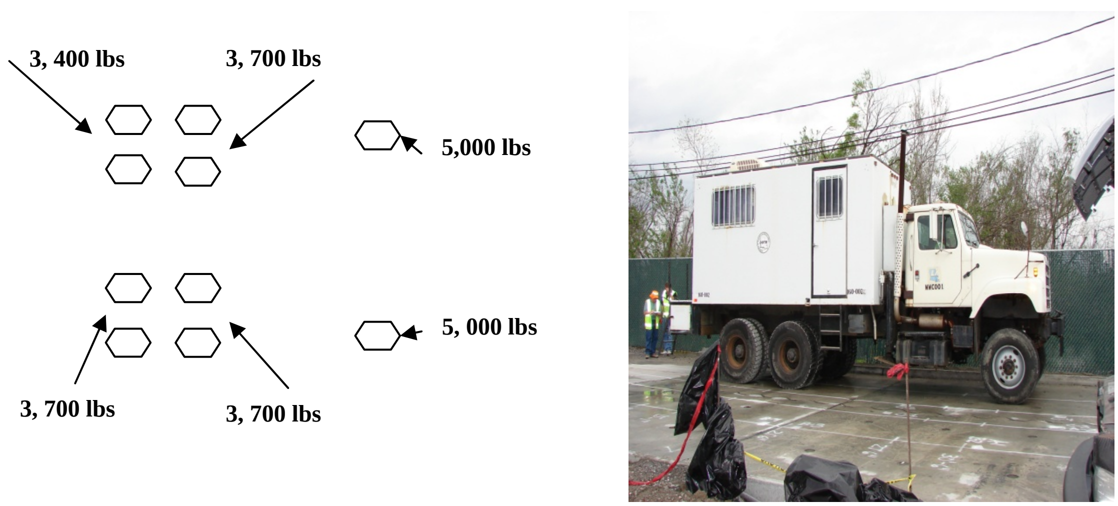


Figure 10: Large cone truck axle distribution and loads

Discussion of results

Construction Report

On January 19, 2007, Tri-Dyne Industries installed the PCPS. Work was begun with the removal of approximately 18 inches of the existing stone driveway followed by the placing and compacting of 8 inches of stone base course, as shown in Figure 11. The integral curbs were placed and leveled as well.

Figure 11: Stone base course

Once the base course and curbs were installed to the satisfaction of Tri-Dyne, the base course was marked so that the precast concrete panels could be precisely placed, as shown in Figure 12. This is an important step, as the PCPS system is modular and requires precise placement.

Figure 12: Slab layout on base course

A thin layer of sand was placed over the entire base course surface, and the pressure sensors were installed at specific locations, as shown in Figure 13. This sand layer was not compacted with a mechanical device.

Figure 13: Sand layer placement and sensors

Once all of the sensors were in place, the first PCPS was installed, as shown in Figure 14. Caution was taken by the construction crew to avoid damage to the pressure sensors during installation of the PCPS.

Figure 14: Precast panel installation

The second panel in the adjacent lane was placed next to the installed panel. Both panels were then lifted with attachments near the inside edges and lowered in a fashion that allowed the metal joints to stitch together as the slabs were slowly lowered, as shown in Figure 15.

Figure 15: Longitudinal joint stitching

The remaining slabs were placed and stitched together in a similar fashion, as shown in Figure 16.

Figure 16: Panel installation system

Injection of leveling material did not occur until June of 2007, meaning an assessment of the finished product was not possible in January. In March of 2007, the slabs were assessed, as outlined in the structural and functional sections of this report, with one exception. Joint faulting and width measurements were not performed, as leveling material had not been injected at that time. In June of 2007, a full assessment of the slabs was conducted, except that joint widths were not measured until October 29, 2007. The following comments about the constructability of the PCPS are based on observations from the aforementioned assessment dates.

It appears that the PCPS can be installed by skilled operators with two lifting devices. The transverse and longitudinal joints stitch together well, but joint faulting and widths were excessive. The maximum observed transverse and longitudinal faults were 0.55 and 0.2 in., respectively, with maximum joint widths above 1 inch in both transverse and longitudinal directions. This will be elaborated upon further in the functional assessment section of this report.

In the March assessment, the fact that the slabs were "rocking" whenever heavily loaded trucks drove across them was noticed. This was occurring probably because the slabs had not been injected with leveling material and voids were present. Excessive movement in the slabs can cause spurious readings in the FWD and especially the Dynaflect since movement induces a dynamic load into the slab. In June, after injecting the slabs with leveling material, they appeared to be more stable (showing less rocking than in March) when heavy trucks drove across them.

Assessment of Precast concrete panels

Functional assessment

Prior to beginning profiling work in March, the slabs were swept clean of all loose material, and the four base lines were painted on the slabs, as shown in Figure 17. Each test point was measured and painted onto the slab base lines so tests could be performed at the same location with each test device. This process was repeated during testing in June as well.

Figure 17: Preparing the panels for testing

Walking profiler: Base lines 1 through 4 were each tested with the walking profiler in March and June of 2007. Profile plots for BL 4 are shown in Figure 18, and the average IRI values are shown in Table 3. Profile plots for BL 1, 2, and 3 are in Appendix 1. The profile data indicates that the slabs were rough, with IRI values ranging from 296 to 382, in the March 2007 assessment. The leveling material operation in June of 2007 did not improve the roughness, as the IRI values ranged from 294 to 413. The precast panels installed on this project did not meet DOTD ride specifications and had IRI values (150 – 165) greater than those reported by others for PCPS (9)(10).

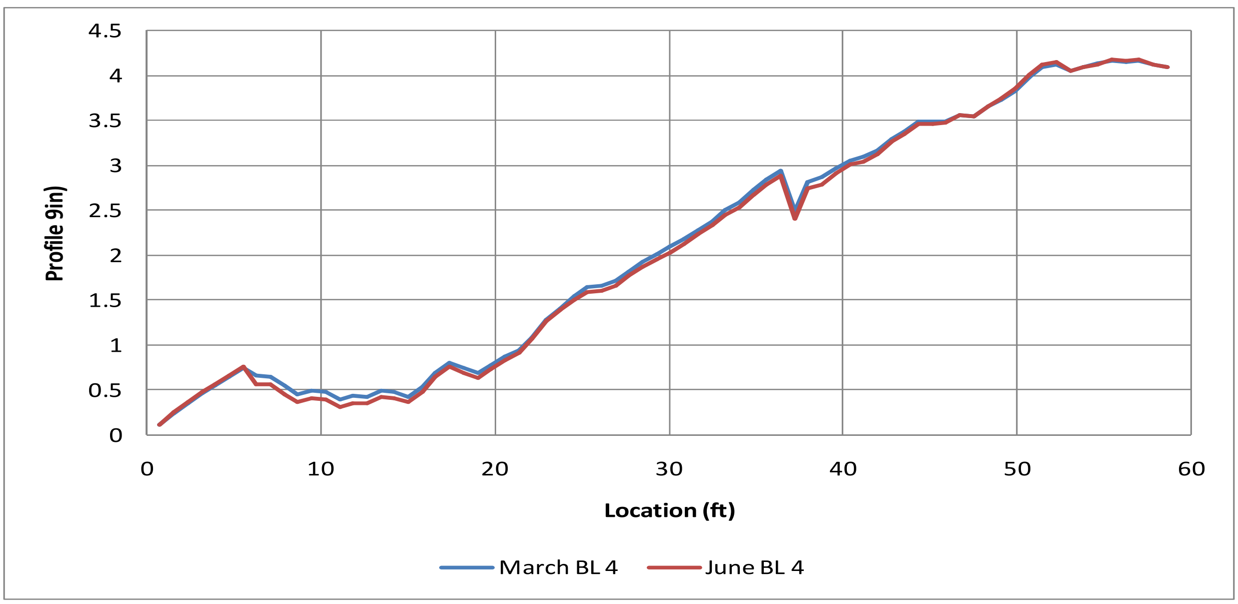


Figure 18: Profiles for BL 4

Table 3 IRI values

Baseline	IRI March 07	IRI June 07
1	296	294
2	353	398
3	382	413
4	362	378

Judging from the profile data, slab 7 appears to have been bending downward in relation to slab 8, as shown in Figure 19. This scenario was observed in slab 2 and 3 profiles also; see Appendix 1. Further discussion of the ramifications of such will be presented in the pressure sensor section.

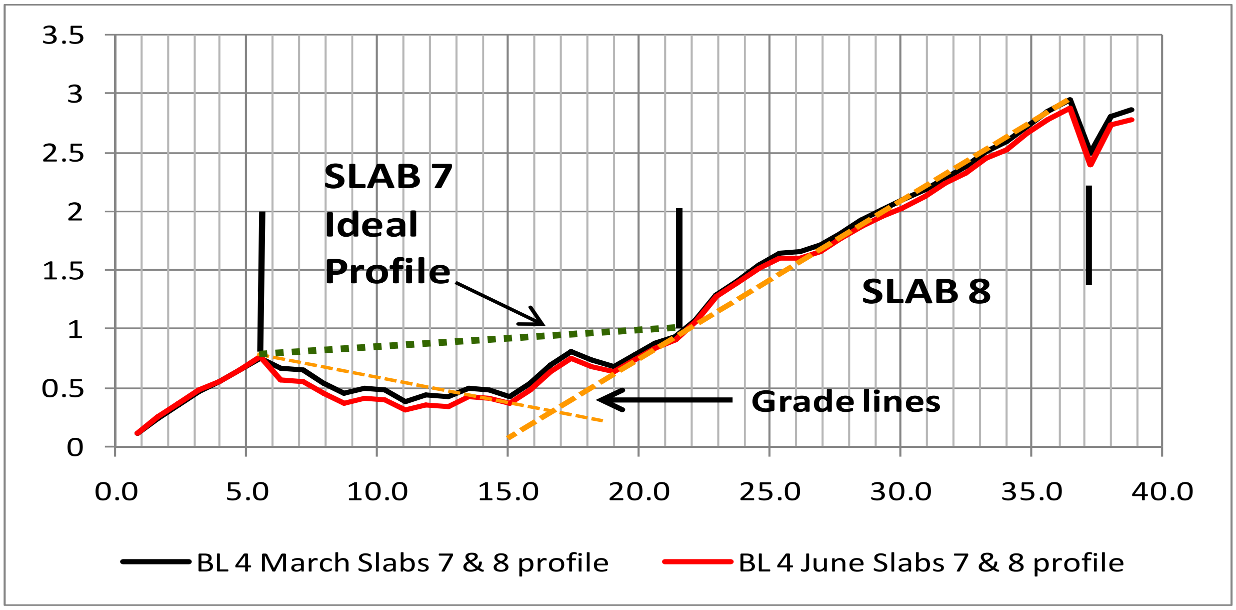


Figure 19: Slab 7 and 8 profile

Fault measurements: Tables 4 and 5 present fault measurements from the June of 2007 assessment of the transverse and longitudinal joints, respectively. Faulting was not measured in the March of 2007 assessment, as no attempt to level the slabs had occurred at that time. Twenty four fault measurements were taken at the transverse joints, and only two of those areas did not have faulting. Eleven fault measurements were taken on the longitudinal joints, with three areas showing no faulting. The maximum faulting observed was 0.55 in. and 0.2 in., respectively, for the transverse and longitudinal joints. Therefore, the precast panels did not meet faulting requirements.

Table 4: Transverse joint faulting

Slab	Transverse Faulting (in.)			Slab	Transverse Faulting (in.)		
	Left edge	BL 1	BL 2		Rt edge	BL3	BL 4
1-2	0.15	0.20	0.45	6-7	0.05	0.40	0.25
2-3	-0.10	0.05	0.00	7-8	-0.20	0.05	0.05
3-4	0.40	0.10	0.30	8-9	0.55	0.40	0.35
4-5	0.20	0.05	0.00	9-10	0.20	0.35	0.20

Table 5: Longitudinal joint faulting

Slab	Distance	Longitudinal Faulting (in)
7-2	0	0.00
7-2	12	0.15
7-2	15	0.15
7-2	21	0.20
8-3	22	0.15
8-3	27	0.00
8-3	30	0.05
8-3	37	0.20
9-2	38	0.20
9-2	43	0.00
9-2	46	0.10

Joint widths: Tables 6 and 7 present joint width measurements from the October 29, 2007 assessment for the transverse and longitudinal joints, respectively. Twenty four joint width measurements were taken at the transverse joint locations, and 21 had widths greater than 3/8 in. Eleven joint width measurements were taken on the longitudinal joints, with 10 having widths greater than 3/8 in. The maximum joint width for the transverse and longitudinal joints was 1.34 in.and 1.08 in., respectively. The PCPS did not meet LA DOTD joint width requirements.

Table 6: Transverse joint width

Slab	Transverse Joint width (in.)			Slab	TransverseJoint width (in.)		
	Left edge	BL 1	BL 2		Rt edge	BL3	BL 4
1-2	1.16	1.22	1.08	6-7	1.19	1.17	1.15
2-3	0.51	0.33	0.56	7-8	0.15	0.62	0.51
3-4	0.49	0.62	0.62	8-9	0.52	0.26	0.65
4-5	1.22	1.15	1.24	9-10	1.34	1.25	1.30

Table 7: Longitudinal joint width

Slab	Distance	Longitudinal Joint width (in)
7-2	0	0.18
7-2	12	0.93
7-2	15	1.07
7-2	21	1.06
8-3	22	1.08
8-3	27	0.92
8-3	30	0.88
8-3	37	1.00
9-2	38	0.72
9-2	43	0.71
9-2	46	0.73

Structural assessment

Void detection from FWD: Figure 20 presents the Y intercept values for testing conducted on the four base lines in March and June. When the Y intercept value is greater than 2, there is potentially a void or loss of support present at that location. On base line 1, voids were present at locations 6, 21, and 27 for the March data set, and no voids were present for the June data set. Voids were present at locations 5 and 21 on base line 2 for the March data set, and no voids were present for the June data set. Base line 3 had voids present at locations 12, 15, 21, 37, and 38 for the March data set and voids present at location 22 for the June data set. Base line 4 had voids present at locations 6, 21, 30, 37, and 38 for the March data set and voids present at locations 37 and 38 for the June data set.

Judging from the void detection tests, one may infer that there is a higher incidence of voids present beneath the slabs when they are placed directly on the subgrade. Injecting leveling material beneath the slabs did reduce the number of voids but did not eliminate them.

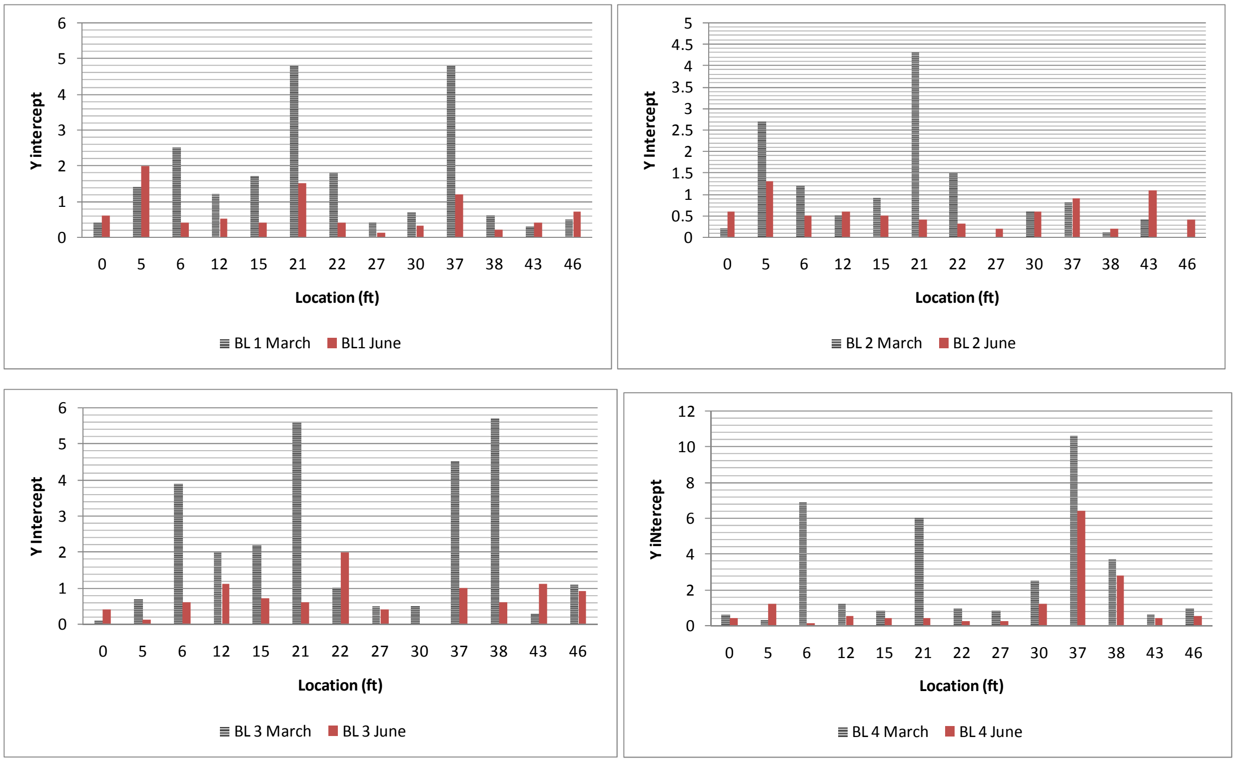


Figure 20: Y intercept values

Load transfer efficiency as determined by the FWD: In both the March and June data sets, all transverse joints on all base lines had load transfer efficiencies below 43 percent, as shown in Figure 21. Therefore, the transverse joint load transfer mechanism evaluated on the PCPS produced poor load transfer.

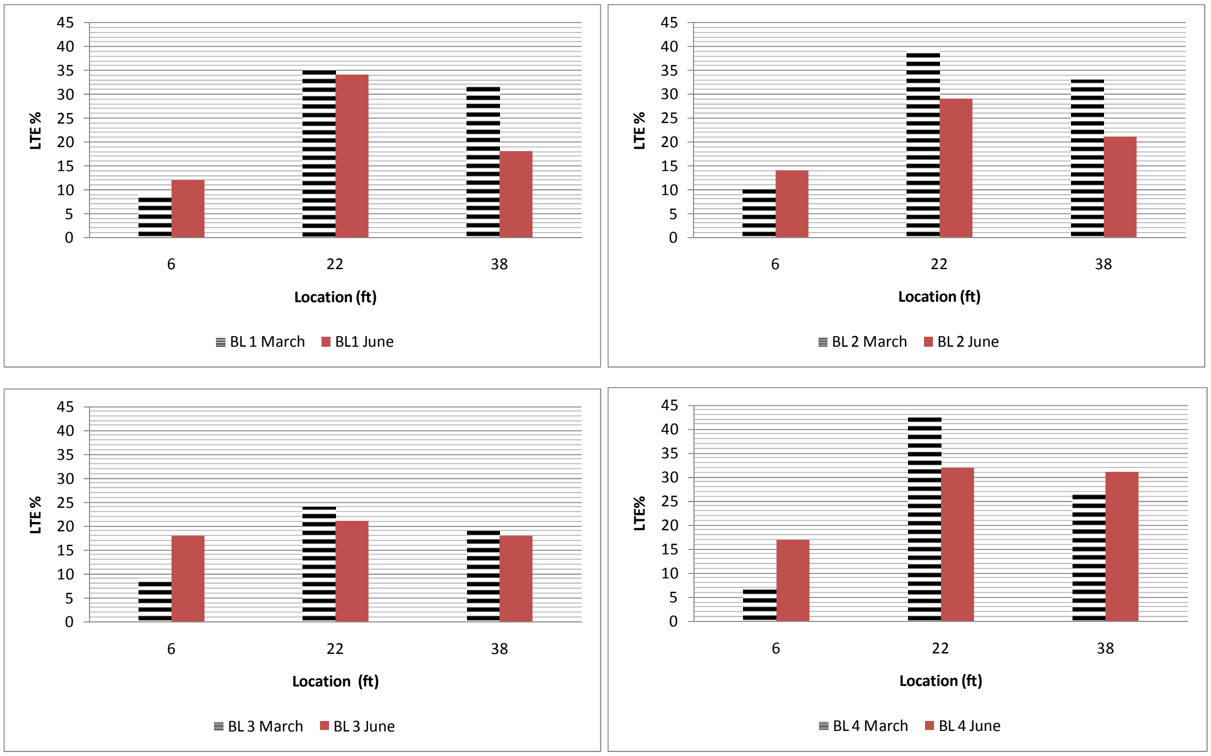


Figure 21: Load transfer efficiency

Layer Moduli values from FWD tests: The layer moduli were determined for the concrete pavement, stone base course, and subgrade for the March and June assessments and are shown in Figure 22.

BL 1: The average modulus values for the cast in place slab, precast concrete panels, stone base course, and subgrade were 2,484, 3,380, 71, and 2.5 ksi, respectively for the March data set; and 1,876, 5,823, 71, and 3.2 ksi, respectively for the June data set.

BL 2: The average modulus values for the cast in place slab, precast concrete panels, stone base course, and subgrade were 2,029, 4,198, 75, and 2.6 ksi, respectively for the March data set and 1,682, 5,124, 81, and 2.7 ksi, respectively for the June data set.

BL 3: The average modulus values for the cast in place slab, precast concrete panels, stone base course, and subgrade were 1,694, 2,600, 80, and 1.8 ksi, respectively, for the March data set and 1,394, 3,357, 68, and 2.6 ksi, respectively, for the June data set.

BL 4: The average modulus values for the cast in place slab, precast concrete panels, stone base course, and subgrade were 1,022, 3,583, 6,7, and 1.4 ksi, respectively, for the March data set and 1,181, 5,032, 67, and 2.2 ksi, respectively, for the June data set.

Modulus values in the 4,000 to 5,000 ksi range are generally found in good concrete pavements typically used on DOTD highways. The modulus values for the cast in place slab were generally less than 2,000 ksi, below the acceptable range. Modulus values for the precast panels in the March data set ranged from 2,600 to 4,198, below the acceptable range. However, these values could be inaccurate due to the instability of the slabs during testing.

The June data set had higher modulus values than the March data set and ranged from 3357 to 5822 ksi. These values were generally acceptable and were higher than the March data set, probably because the slabs were more stable due to the injection of leveling material.

Values for the stone base course modulus were generally higher than 45 ksi, indicating that the base course was acceptable. Subgrade soil modulus was on average 2.4 ksi and was very weak.

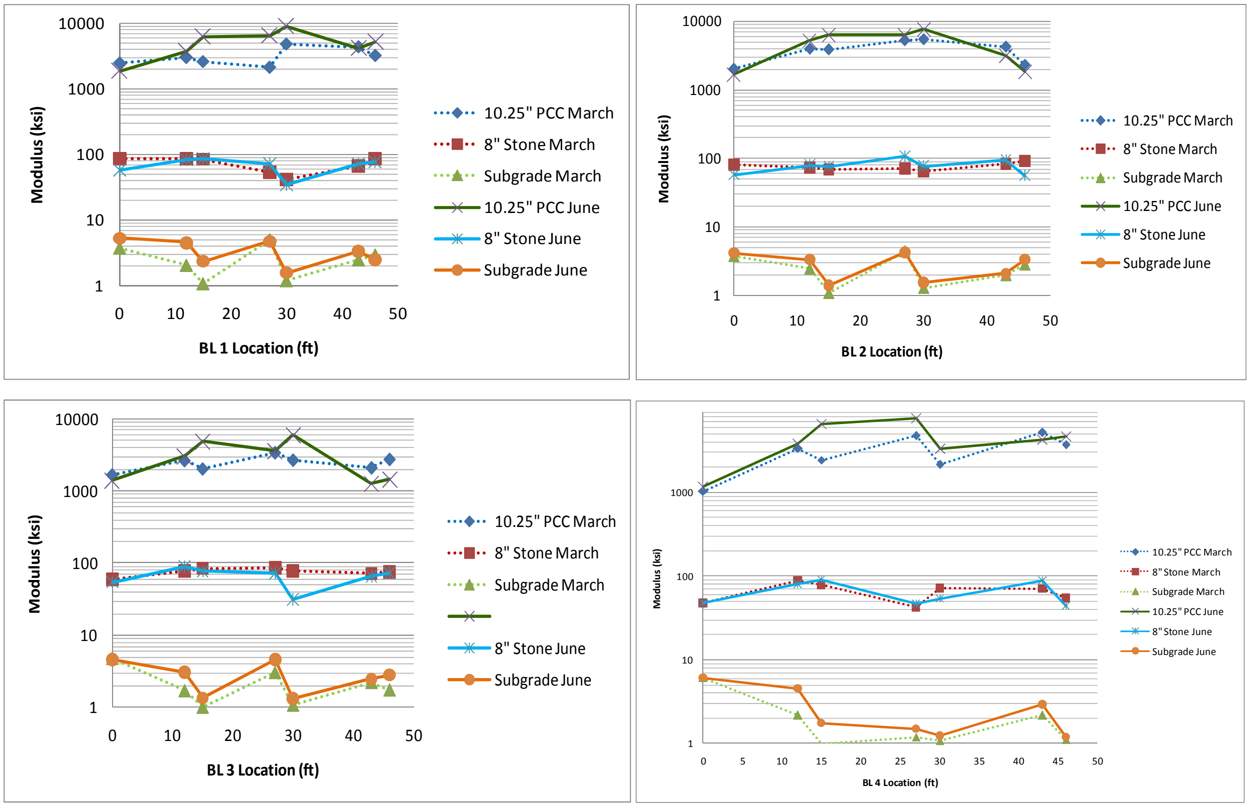


Figure 22: Layer Moduli values

First sensor deflection analysis from FWD: Figure 23 presents the first sensor deflections for the March and June assessments. Trends toward lower deflections were generally observed for the slabs once they were injected with leveling material (June data set). Deflections at the joint locations (2, 21, 22, 37, and 38) were higher than deflections at locations near the center of the slab (12, 15, 37, 30, 43, and 46), indicating a decrease in stiffness at the joints, which is typical. Deflections on the approach side of the joints (21, 37) were higher than deflections on the leave side of the joints (22, 38). This will be discussed further in the section on pressure sensor readings.

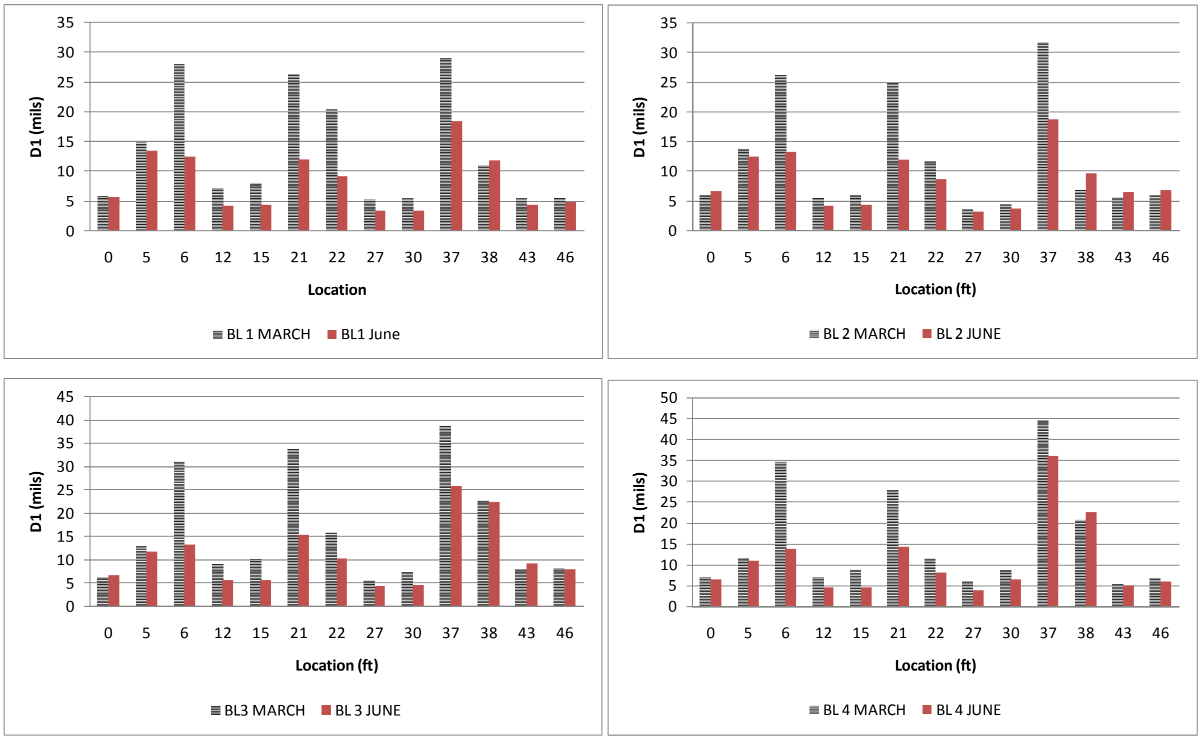


Figure 23: FWD first sensor deflections

Structural number readings from Dynaflect: As shown in Figure 24, many of the test locations were not plotted, due to questionable data. This phenomenon has been previously observed on concrete slabs that were either unstable or had large cracks. Pavement structures of this thickness generally produce SN values greater than 4.5. Due to the problematic data, Figure 24 is presented for informational purposes only.

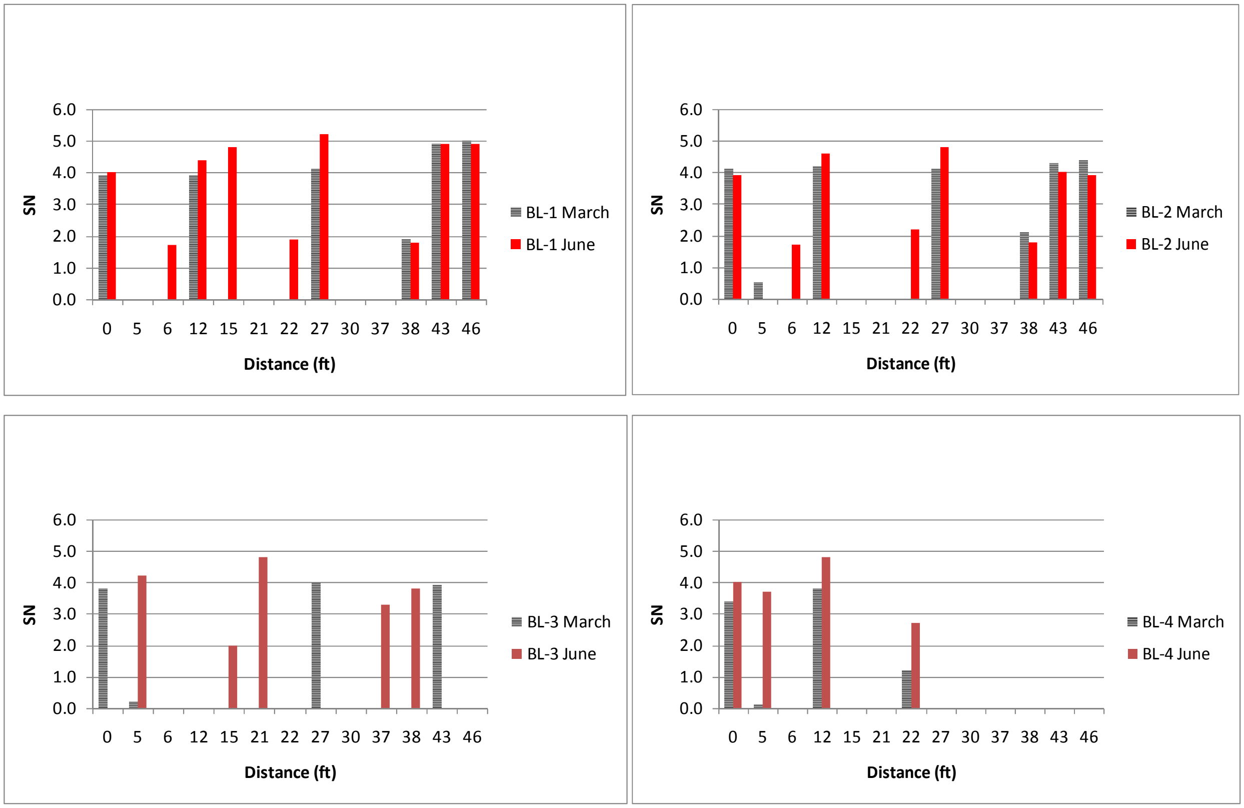


Figure 24: Dynaflect readings

Pressure sensor readings

Loading at locations 21 and 22: Figure 25 presents the pressure cell readings (B-4 and B-3), measured when dead loading from the front wheels of the large cone truck was applied at the approach side (21) and leave side (22) for the March and June evaluations, as shown in Figure 26.

When the load was applied to location 21 (March 12:24:21), the pressure cell readings for B-4 and B-3 were 0.8 psi and 10.1 psi, respectively. If the slabs were equally and fully supported by the base course and properly aligned vertically and horizontally, the pressure readings from pressure cells B-3 and B-4 should be similar when loaded at location 21 and different when loaded at location 22 due to the load transfer mechanism, as shown in Figure 25. However, there was a pressure differential of 9.3 psi across the joint when loaded at location 21. Due to the void at location 21; profile differential between slabs 7 and 8, as shown in Figure 19; and the mechanism of load transfer, as shown in Figure 25; the load applied at location 21 was carried predominately by the base at location 22 (slab 8).

With one exception, a similar scenario was observed for the June data set (June 13:17:07) when the load was placed at location 21. FWD data did not indicate a void at location 21, but the deflections differed significantly from location 21 (14.13 mils) to location 22 (7.97), as with the March 12:24:21 data set. Injecting leveling material beneath the slabs did not fix the problems caused by profile differentials, voids, and the load transfer mechanism.

Appling the dead load to location 22 (slab 8) barely influenced sensor B-4 (slab 7 - location 21) due to the design of the joint.

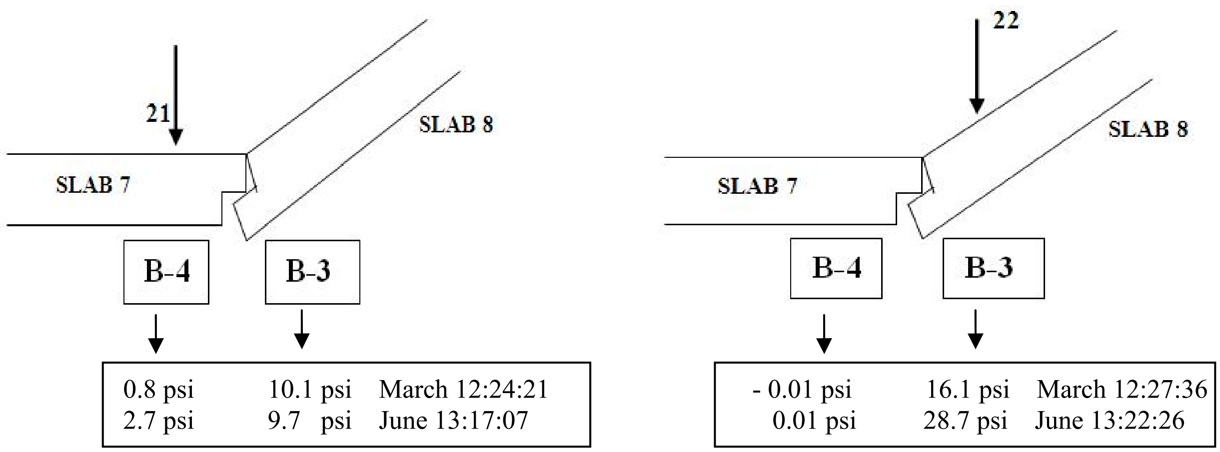


Figure 25: Data summary for slabs 7 and 8

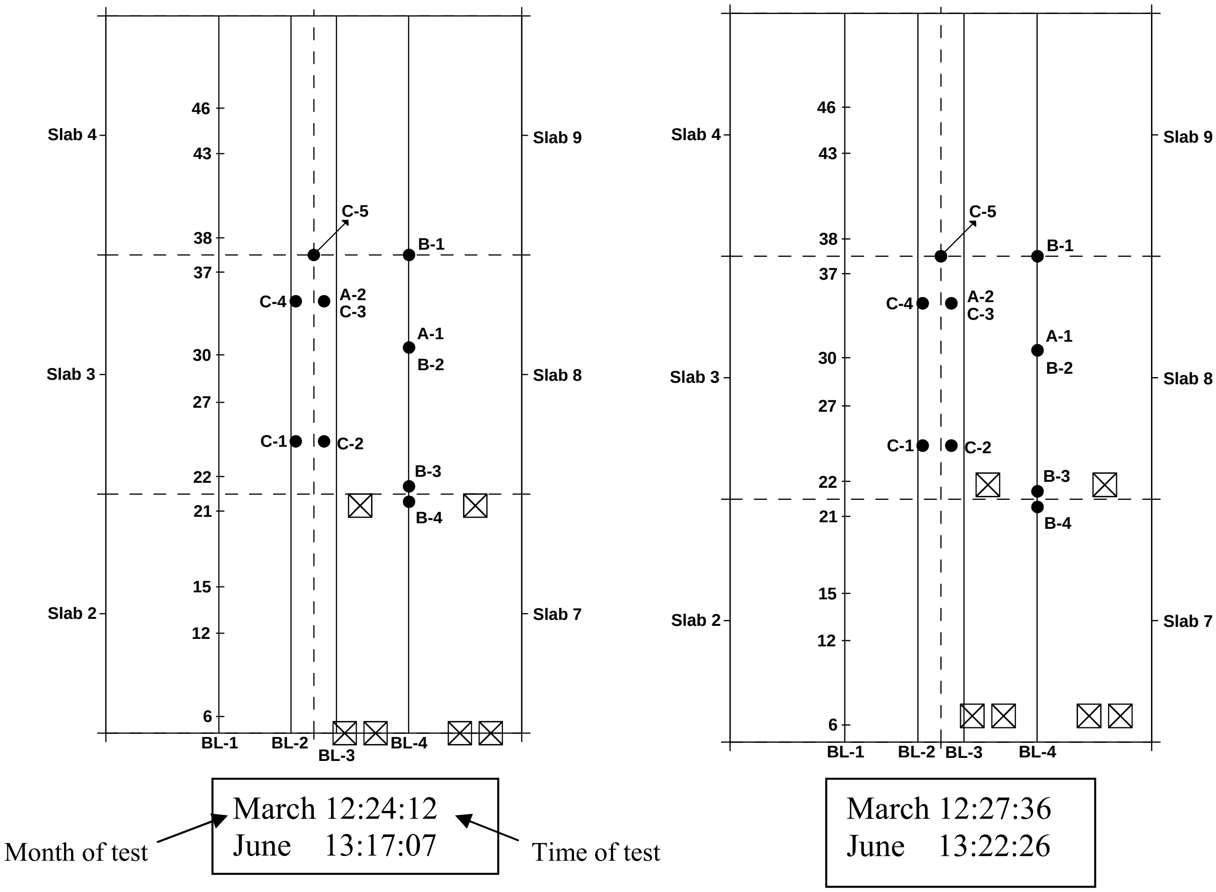


Figure 26: Wheel loading for approach side (21) and leave side (22) of the joint

Simultaneous loading at locations 19.5 and 23.5: The pressures for B-4 and B-3 were 2.5 psi and 30 psi, respectively, for the March 12:41:36 data set when the rear wheels of the large cone truck were located equidistant (24 in.) from the joint at slabs 7 and 8, as shown in Figures 27 and 28. Because the lap joint system was a poor load transfer mechanism, slab 8 bears the full load of the tires (location 23.5) over it and the majority of the load applied (location 19.5) to slab 7. This trend was also evident in the June 13:42:21 data, as the pressures for B-4 and B-3 were 12.2 psi and 40 psi, respectively.

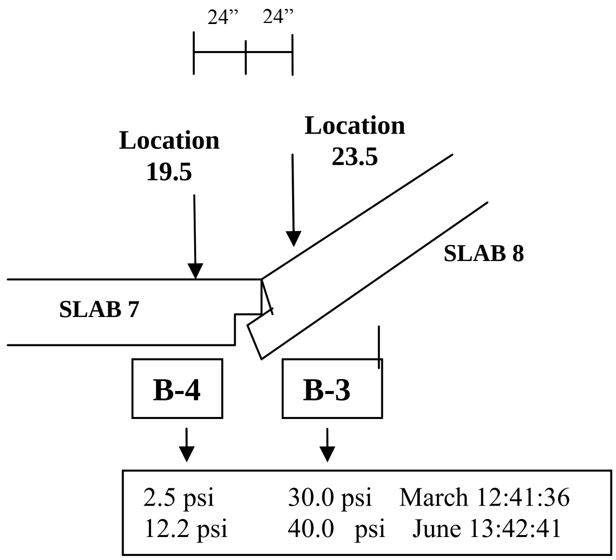


Figure 27: Data summary for slabs 7 and 8

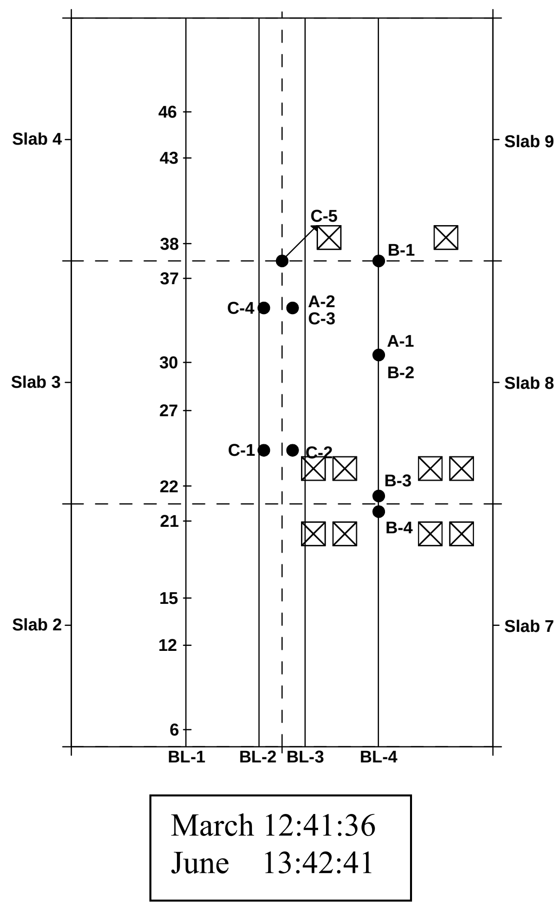


Figure 28: Loading location

Loading near sensor C-2 with front wheels: Figure 29 presents the wheel loading locations as they were when the front wheel of the truck was located near sensor C-2 on slab 8.

As shown in Figure 30, the pressures for C-1 and C-2 were 4.1psi and 6.4 psi, respectively, for the March 14:17:09 data set and 6.9 psi and 4.6 psi, respectively, for the June 15:09:28 data set. Similar pressures across the longitudinal joint at slabs 3 and 8 were observed for both the March and June data sets.

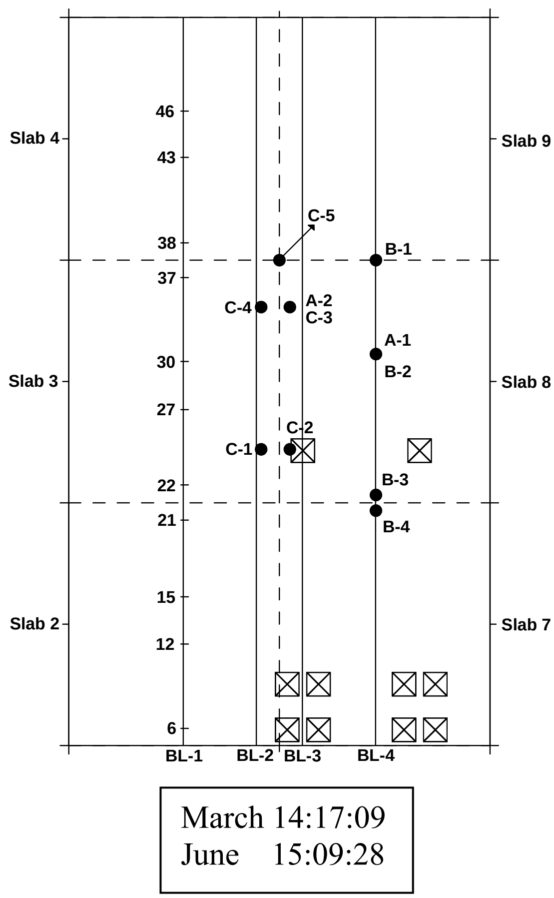


Figure 29: Front wheel load near sensor C-2

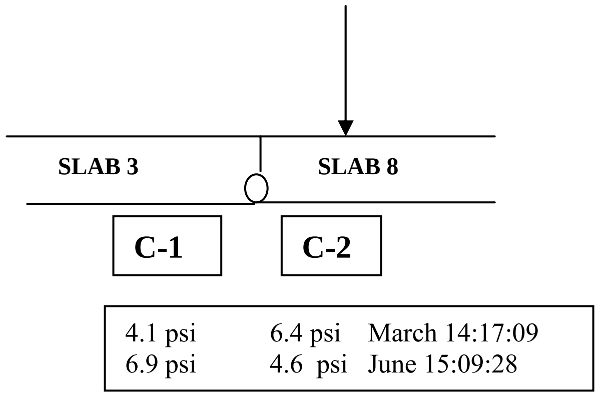
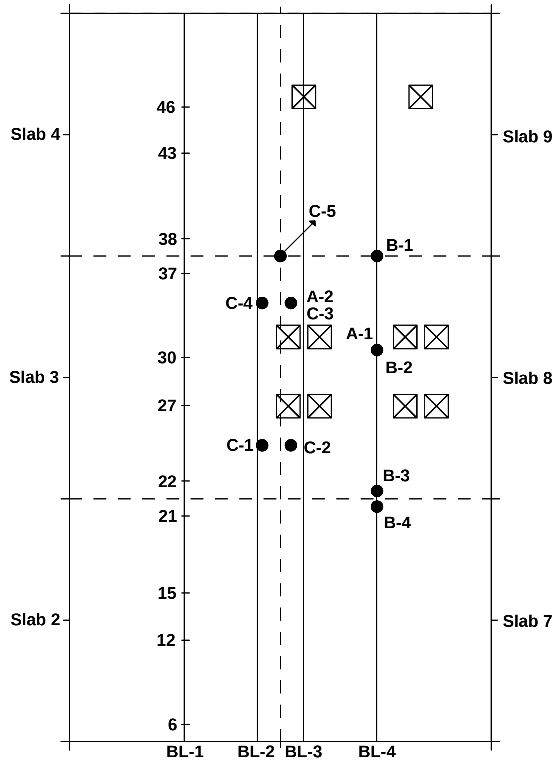


Figure 30: Pressure sensor readings for loading on slab 8

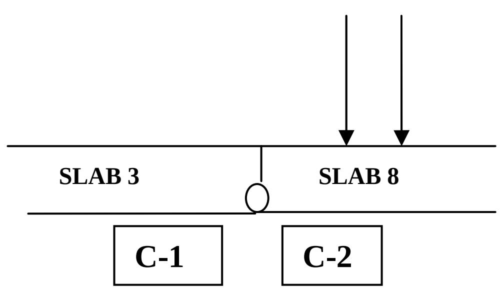
Loading between sensor C-2 and C-3 with rear wheels: Figure 31 presents the loading location as it was when the rear wheels of the truck were located between sensor C-2 and C-3 on slab 8. Comments about the sensor C-4 and C-3 relationship will not be presented because sensor C-3 was nonfunctional during the June evaluation.

As shown in Figure 32, the pressures for C-1 and C-2 were 2.8 psi and 8.2 psi, respectively, for the March 14:27:17 data set and 4.3 psi and 4.4 psi, respectively, for the June 15:25:12 data set. The reason for the differing pressures between the March and June data set could be that the better contact between the slab and base course was created by the injection of leveling material in June.



March 14:27:17
June 15:25:12

Figure 31: Front wheel load near sensor C-2



2.8 psi	8.2 psi	March 14:27:17
4.3 psi	4.4 psi	June 15:25:12

Figure 32: Front wheel load near sensor C-2

Conclusions

LTRC conducted a construction, functional, and structural assessment of the Tri-Dyne PCPS.

The main issues with the Tri-Dyne PCPS are the joints and injection ports. The Tri-Dyne system is designed for quick placement/removal and does not use dowel or tie bars to hold the joints together or provide load transfer. The overlap transverse joint design used on this project allows for load transfer in one direction only. With the joint placement used in this assessment (traffic moving from slab 7 to slab 8), the slab on the leave side of the joint will be subjected to higher loadings than the slab on the approach side of the joint for the life of the slabs. Over time, this can contribute to the formation of a void on the leave side and increased faulting due to densification or loss of material at that location. The tongue and groove joint used at the longitudinal joints appeared to be more robust than the overlap joints based on pressure cell readings but could not be evaluated with the FWD because of site conditions. The injection/lifting ports were placed in the wheel path, which will increase roughness (IRI) and may be spaced too far apart to allow for injection material to be distributed fully beneath the slabs.

Construction of the slabs revealed several issues. First, a depression in the first PCPS panels (7─8 and 2─3) was measured with the walking profiler. It may have been caused by a depression in the base course and the PCPS bent to match the base course profile. This depression could have contributed to a vertical misalignment (fault) at the joints between slabs 7─8 and 2─3 as well as voids at that location. Second, voids beneath the slabs were measured both before and after injection with leveling materials, which means that the injection system method and material used on this project was inadequate. Third, the slabs were constructed with joint widths and faults that did not conform to DOTD or Highway Industry standards. The fact that, according to others, these are problems consistent with all PCPS should be noted (3)(4)(5)(6)(7) (8)(9)(10).

The functional and structural assessment of the Tri-Dyne overlap PCPS indicated that they are not appropriate for full lane placement, based upon highway industry standards. Functional assessment of the slabs indicated that the slabs were constructed with faults on the transverse joints as high as 0.55 in., joint widths in excess of 1 in., and IRI values as high as 413. These conditions do not meet acceptance criteria for new pavements. The structural assessment indicated that, with the exception of the overlap joints, the pavement should withstand traffic loading. The problem with the overlap joints is that, if the slabs are not constructed at the proper grading profile or settle over time to a grade similar to the one constructed on this project, overstress to the slab connection could occur. The finger type tongue and groove joints appear to be robust, but fatigue testing should be conducted to determine whether they could withstand typical traffic loading.

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Appendix 1

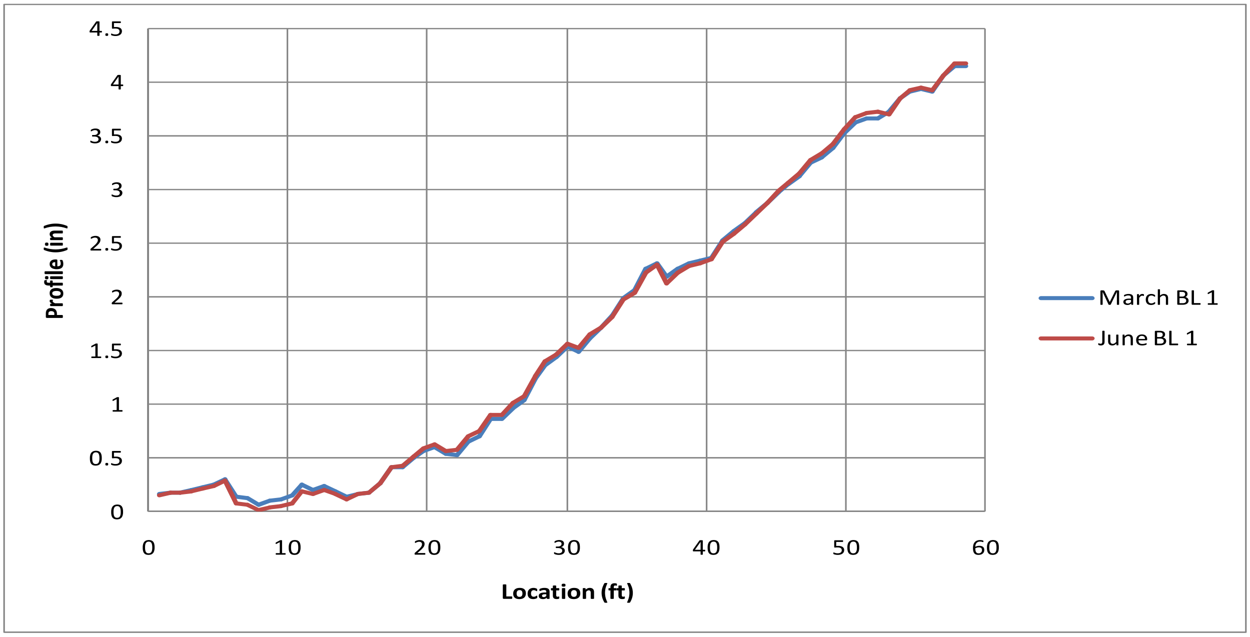

Figure 33: BL 1 profile

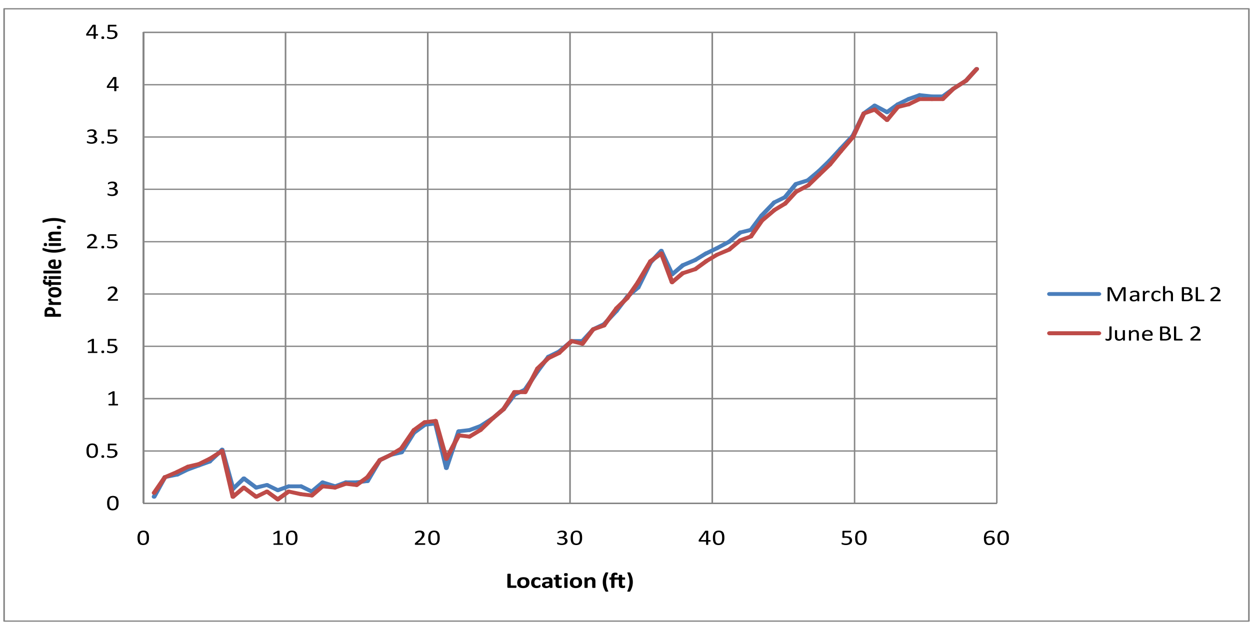

Figure 34: BL 2 profile

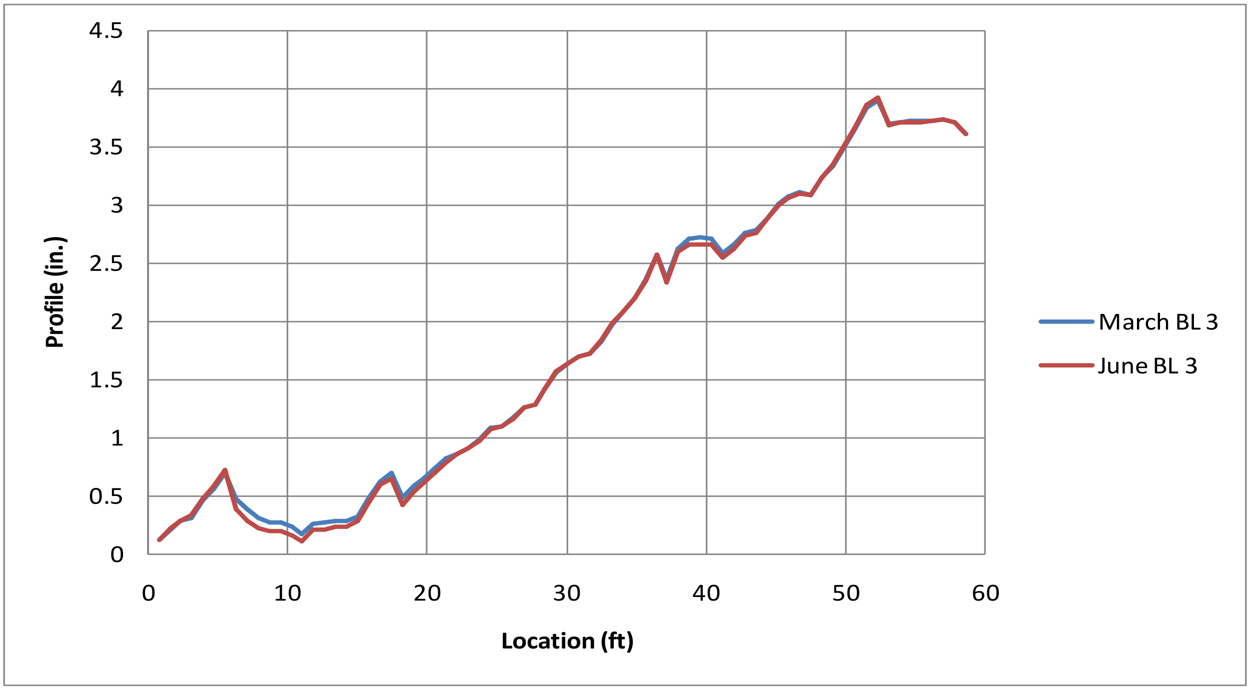

Figure 35: BL 3 profile