LOAD TEST REPORT OF

SLAB BRIDGES ACROSS SOUTH CAROLINA

SUBMITTED TO

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SUBMITTED BY

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Executive Summary

From the load test results of the slab bridges, it is recommended to use distribution factor of 0.5 (lane) for precast concrete slab bridges in South Carolina.

Table 1 Summary of Tested Bridge Condition and Distribution Factor

Asset ID	**Year**	**Design Live Load**	**Span Length**	**Slab Panel Width**	**Reflective Cracking**	**Tie Rod Condition**	**Asphalt Thickness**	**Tested DF**	**Recommended DF**
		(H10/H15)	(ft.)	(ft.)	(None/Minor /Mod-rate/Severe)	(Good/Poor)	(in.)	(Lane)	(Lane)
02006	1954	H10	14	5	Severe	Good	N/A[1]	0.50	0.50
02357	1956	H15	14	5	None	Good	3.50	0.52	0.50
02736	1958	H10	15	5.5	None	Good	0.50	0.43	0.50
05957	1971	H10	14	5	None	Good	2.00	0.54	0.50
06866	1976	H15	15	5.5	None	Good	N/A[1]	0.47	0.50
07337	1980	H15	15	5.5	No Asphalt	Good	No Asphalt	0.62	0.50

1. N/A field data not available

1 Scope

In this report, the live load distribution of six precast concrete slab bridges in South Carolina are summarized. The goal of this report is to identify the visual variables or indicators on a slab bridge that may have direct correlation to measured distribution factors recorded on the instrumented bridges. Distribution factors have been compared against the readily visually inspectable potential indicators including asphalt thickness, year of construction, design load, width of slabs, gaps between slabs, condition of post tensioned rods and degree of reflective cracking, to identify potential patterns. In addition, NDT was performed on each instrumented structure to verify that subject structures match existing plans. Based on observed results, as well as engineering judgment, we produced recommendations to allow engineers to determine appropriate distribution factors for precast concrete slab bridges in South Carolina. In this report, six slab bridges with different superstructure conditions have been selected and diagnostic load testing was performed on these bridges. Results of these diagnostic load tests in conjunction with observed conditions of superstructure components were analyzed and presented.

2 Objectives

The main objectives of this project are:

- Perform diagnostic load tests on set of slab bridges
- Inspect the condition of superstructure components
- Verify rebar matches existing plans
- Test concrete compressive strength
- Calculate LLDF based on diagnostic load tests results
- Correlate superstructure element conditions to LLDF to better understand the factors that affect load sharing
- Provide recommendation to determine LLDF for slab bridges.

3 Bridge Description

The load tested precast concrete slab bridges are designed for H10 or H15 truck loads with a span length of 14 ft. or 15 ft. Figure 1 shows a typical cross section of a slab bridge. The bridge cross section typically consists of 5 ft. or 5.5 ft. wide precast deck panels. The number of deck panels for tested bridges are either 4 or 5 depending on the clear roadway of the bridge. Precast panels are tied together by transverse tie rods at both ends. Thickness of the slab panels are 8.25 in., 9.25 in., or 9.5 in., depending on the design live load and span length. Standard plans for tested bridges are attached in Appendix I.

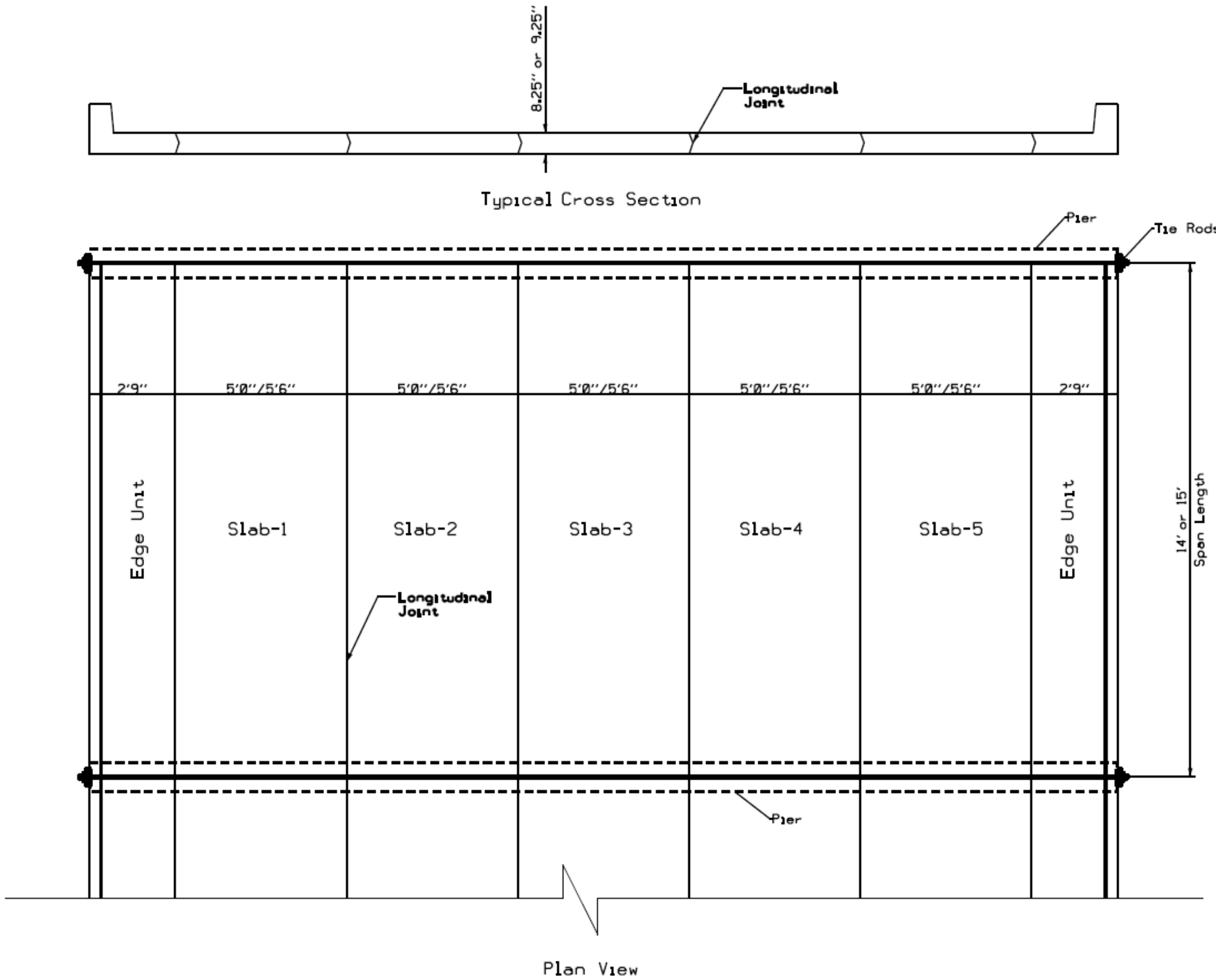


Figure 1: Typical cross section and plan view of a slab bridge.

4 Conditions of Superstructure Components and NDT Tests

4.1 Reflective Cracking and Tie Rod Condition

Reflective cracking caused by the independent movement of one slab relative to the adjacent slabs was identified as potential inspectable indicator after the completion of our Phase 1 testing.

During the visual inspection, all tie rods were sounded with a hammer. This crude inspection process gave indications of a broken or poorly tighten tie rod. However, the process may have provided false positive results because it cannot determine if the tie rods are providing adequate post tensioning to allow the shear keys between slabs to lock as designed. In the attached tables, if the tested span had a tie rod that was determined to be loose or broken, the "Tie Rod Condition" for that span was listed as "poor". If the post tensioning was sounded and had no indications of broken or loose elements, this indicator was identified as "good".

4.2 Nondestructive Testing (NDT)

4.2.1 Rebar number, size and spacing

Nondestructive Test (NDT) was performed at bottom surface of the slab to confirm the size and number of longitudinal rebar as shown in the standard plans. Two different types of NDT were performed: Ground Penetrating Radar and Profoscope. Figure 2 shows a standard slab bridge cross section in the longitudinal direction.

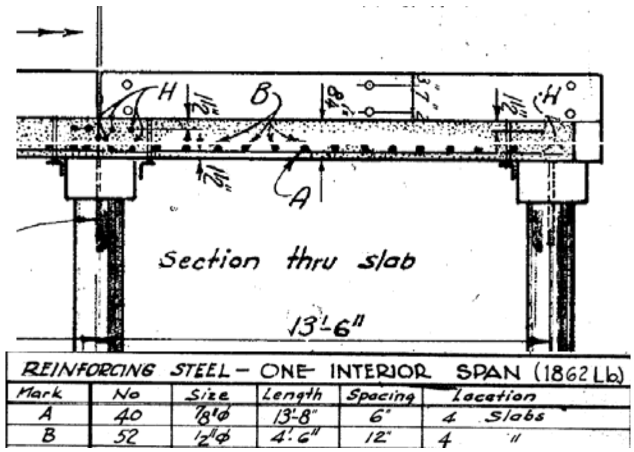


Figure 2. Reinforcing Rebar Details in a Standard Plan of a Slab Bridge.

The Profoscope can determine bar size within +/- 1 bar. For all but one of our tests the Profoscope showed a diameter of 0.75 in. (#6 bar) within 1 bars size of the #7 bar listed on the plans. GPR was used to accurately detect the location and spacing of longitudinal bars. All six bridges matched the design number and spacing of bars . Figure 3 shows a sample GPR B-Scan result of a tested bridge (BR02006). Table 2 lists the NDT results of load test slab bridges and the design reinforcement details corresponding to standard plans.

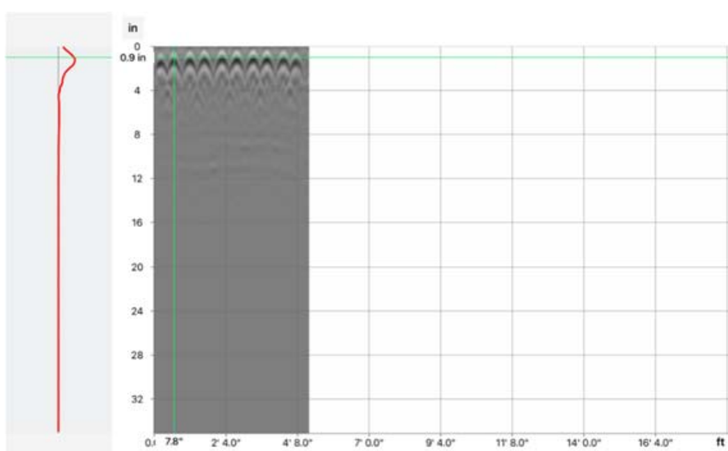


Figure 3. GPR B-Scan showing Rebar as Hyperbola Signatures (BR02006)

Table 2 Slab Bridge NDT Results and Corresponding Standard Plans

Bridge ID	Number of Rebar		Size		Spacing (in)		Standard Plan
	GPR	Design	Profoscope	Design	GPR	Design	
02006	10	10	#6	#7	5.8	6	H10 14' span 24' rdwy (1949)
02357	12	12	#6	#7	5.1	5.25	H15 14' span 24' rdwy (1953)
02736	11	11	#6	#7	6.1	6	H10 15' span 26' rdwy (1957)
05957	10	10	#5	#7	5.9	6	H10 14' span 24' rdwy (1949)
06866	13	13	#6	#7	5.1	5	H15 15' span 31'-6" rdwy (1957)
07337	13	13	#6	#7	5	5	H15 15' span 31'-6" rdwy (1957)

4.2.2 Concrete compressive strength

Concrete compressive strength of the six slab bridges were tested using saw-cut cores. Table 3 compares the design and tested concrete compressive strength. Core 2357, 2736, 6866 were tested using 4-in-diameter unbonded caps. The remaining three cores were tested with sulfur mortar. Per ASTM C1231/C1231M-15, the retainer internal diameter shall between 1.02 to 1.07 of tested core diameter, which limit the cap diameter to approximate 3 in. based on the prepared core size. The 4-in-diameter cap used in the tests is about 1 inch bigger than the requirement, which may lead to an unreliable testing result, such as core 2357. The higher tested strength of core 2006 and 5957 appear to be different mix types when comparing the aggregate and paste with lower cores. Detailed core test reports provided by Boyle Consulting are attached in Appendix H.

Table 3 Comparison between Design and Tested Concrete Compressive Strength

Bridge ID	Core Test Date	Core Prep	Design strength	Tested strength	Tested/Design
			(ksi)	(ksi)	
02006	3/23/20	Bonded Caps	3.75	7.47	1.99
02357	3/20/20	Unbonded Caps	4	2.74	0.69
02736	4/6/20	Unbonded Caps	4	4.9	1.23
05957	3/23/20	Bonded Caps	3.75	6.43	1.71
06866	4/6/20	Unbonded Caps	4	5.01	1.25
07337	3/23/20	Bonded Caps	4	4.47	1.12

5 Instrument Description

Load tests were conducted with ST350 strain transducers manufactured by Bridge Diagnostic Inc (BDI). ST350 is a resistance-based full Wheatstone Bridge strain gauge designed with four fully

350 Ω foil gages. The range of strain that can be measured is ± 2000 micro-strain. Nominal gauge length of ST350 strain transducers is 3 inches. Gauge length can be extended to 24 inches by adding an extension arm. Figure 4 shows a ST350 strain transducer installed at the bottom of slab section. Details of SE350 strain transducer were attached in Appendix E.

Figure 4: ST350 Strain Transducer with 24-in. extension arm.

The data collection from the sensors are done using the STS4 system from BDI. All strain sensors are connected to STS4 nodes through electrical cables. The STS4 node communicates with the STS4 base station via Wi-Fi connection. A laptop with a STS4 data collection software communicates with the base station via Wi-Fi during the load test. Figure 5 shows schematics of the STS4 data collection system.

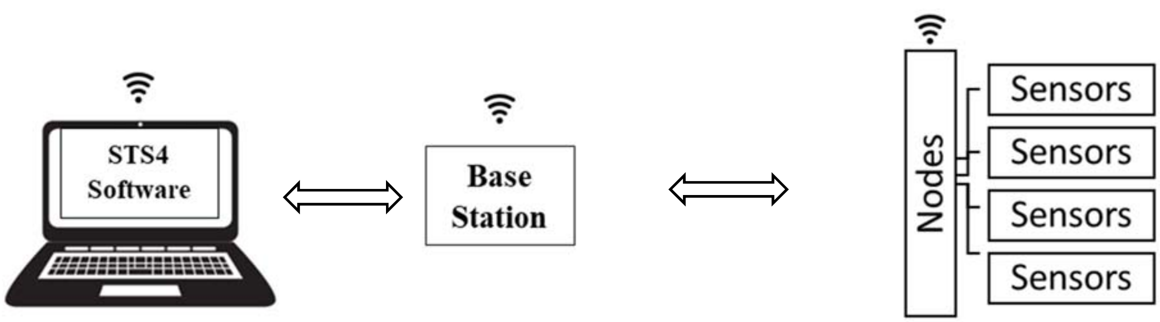


Figure 5 BDI STS4 data collection schematics.

6 Load Test

6.1 Test Phase

To provide for an economical, effective load testing effort, the entire load testing process for slab bridges were conducted in 2 phases as described below:

- Phase 1: Initial load test consisted of two detailed instrumented slab bridges, BR02006 and BR07337, which are representative of the slab bridges. Initial tests were intended to: 1) provide an understanding of the necessary sensors to capture the response of structures; 2) determine truck weight that could be utilized to effectively load the slab bridges for load test purposes; and 3) decide effective load path for load test purposes.

- Phase 2: Included load testing target bridges with refined instrumentation plans. Load tests were conducted in conjunction with gathering visually observable conditions linking bridge conditions and the load distribution.

- Phase 3: (Not completed) Expanded load testing to reach statistically significant conclusions regarding bridge behavior. This phase was not completed due to the results of the distribution factors determined being equivalent to maximum distribution possible per slab.

6.2 Instrumentation Plan

At phase 1, the slab bridges were installed with strain sensors at bottom of the slab at mid-span and in the potential negative moment regions at end of spans. String potentiometers were also installed at mid-span to capture deflection. After phase 1 of the investigation was completed, it was concluded as expected that little negative moments were observed at ends of spans. Based on these findings it was determined fewer stain gauges would be required to allow our team to accurately determine distribution factors for phase 2. Phase two was completed with strain sensors on all slabs at mid-span. The sensor layout for phase 2 is presented in Figure 6.

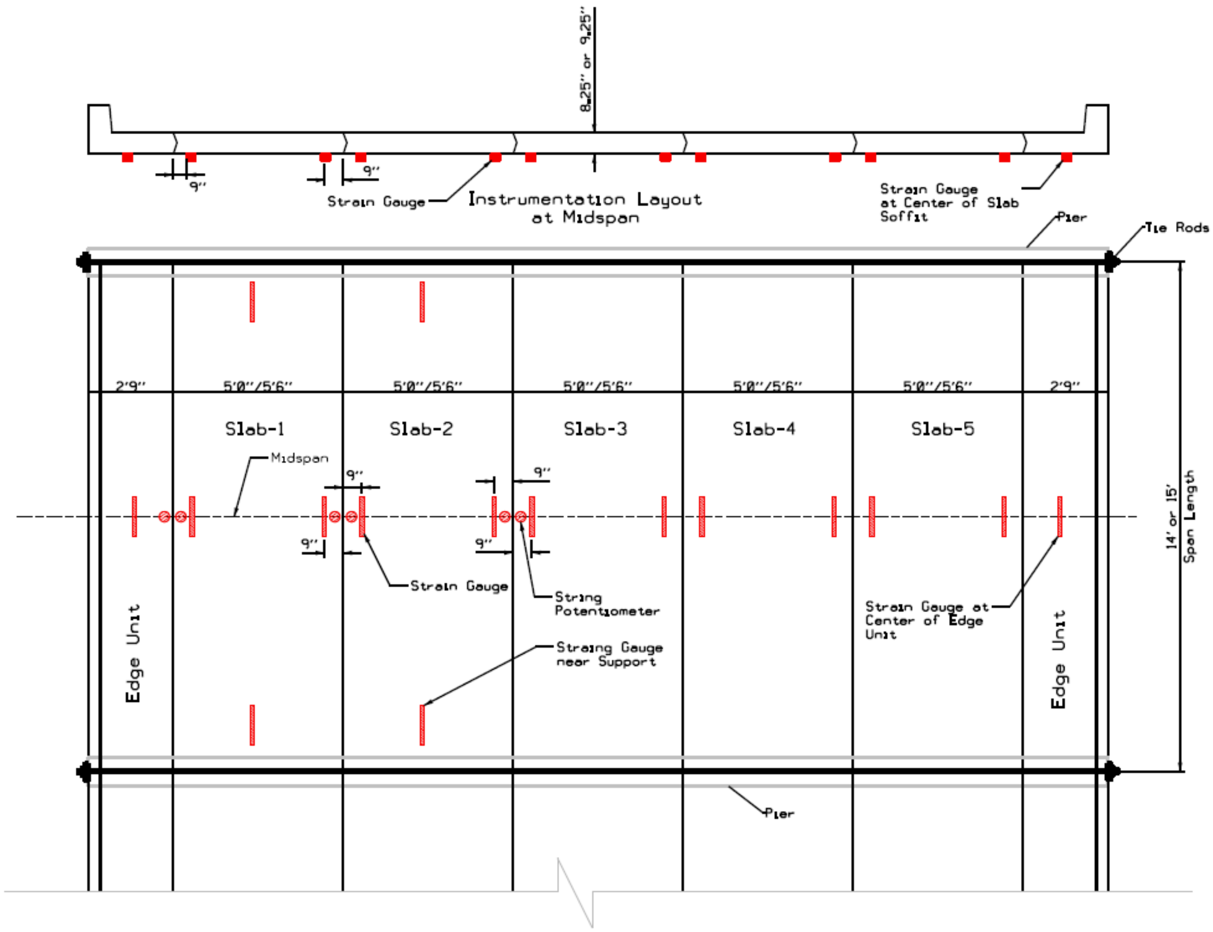


Figure 6: Sensor Layout; Plan View and Cross Section at Midspan.

The gauge length of the sensors was 18" in both phase 1 and phase 2. It was determined that, the manufactured 3 in. gauge length was not adequate to account for the variation of elastic modulus of reinforced concrete slab sections, and effects of potential crack openings. The measured strain presented here must be divided by 6 to account for this increased gauge length and convert the measured strain to actual strain.

Sensors were mounted on the bridges following manufacturer's recommendations. First, grinded the concrete surface to remove localized uneven surfaces and then wiped with acetone to get rid of any dust or grease. The sensors were then mounted on concrete surface using Loctite epoxy adhesive and accelerator for quick setting. Figure 7 shows the installation of strain sensors on one of tested bridges.

Figure 7: Strain Gauge Installation

6.3 Load Test Vehicle

The first two bridges (BR02006 and BR07337) were tested with dump trucks (Figure 8) in phase one during the last week of November 2019. The axle configuration of the dump truck is shown in Figure 11 and Figure 12. During phase 1, single and double dump trucks were utilized loaded near capacity of the bridge, as determined with 2-D BrR model of the bridge. The first two load tests were problematic in that some strain sensor became unglued due to the degree of deflection of the slab members. From the lesson learned from phase 1, it was determined that a truck with a lower GVW will be appropriate for testing these slab bridges. For phase 2, we utilized a lower weight bucket truck as shown in Figure 9 . The axle weight and wheel configurations of the bucket truck are presented in Figure 10 and Table 4. Details of the truck information are attached in Appendix F.

Figure 8 Phase 1 Load Test Truck

Figure 9: Phase 2 Load Test Bucket Truck.

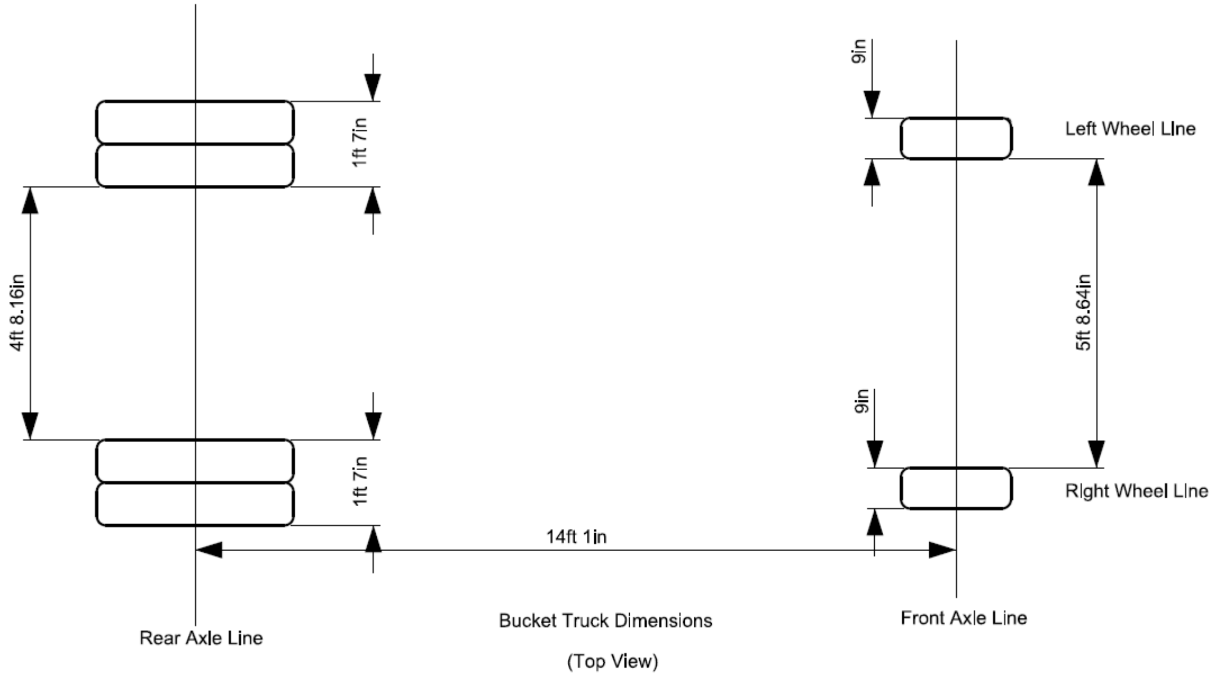


Figure 10: Wheel Configuration of the Load Test Bucket Truck

Table 4: Measured Axle Loads of the Load Test Bucket Truck

	Measured Wheel Load (lbs.)	
	Front Axle	Rear Axle
Right Wheel Line	2580	5980
Left Wheel Line	3040	5960

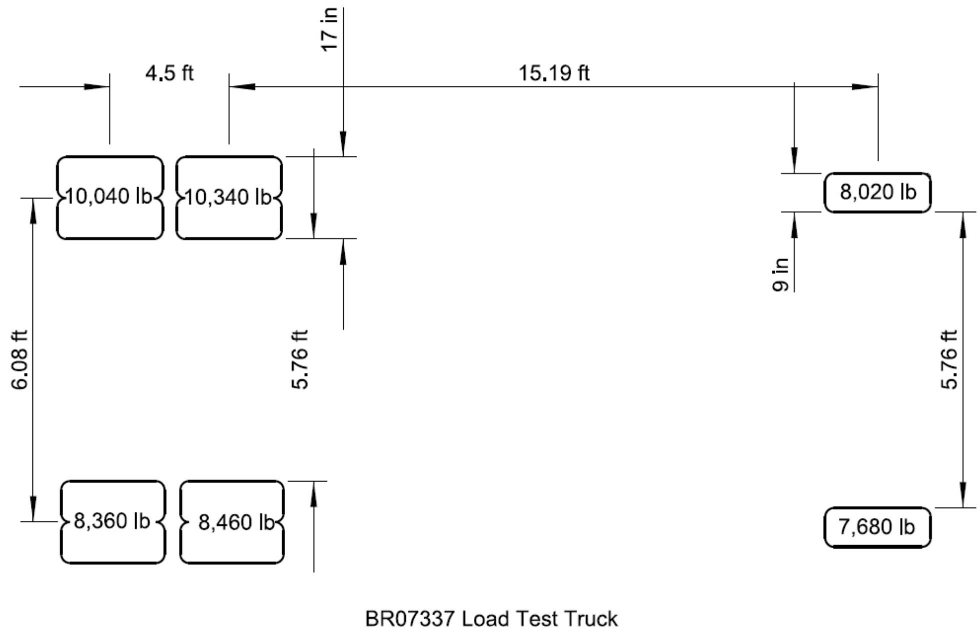


Figure 11: Bridge BR07337 Load Test Vehicle

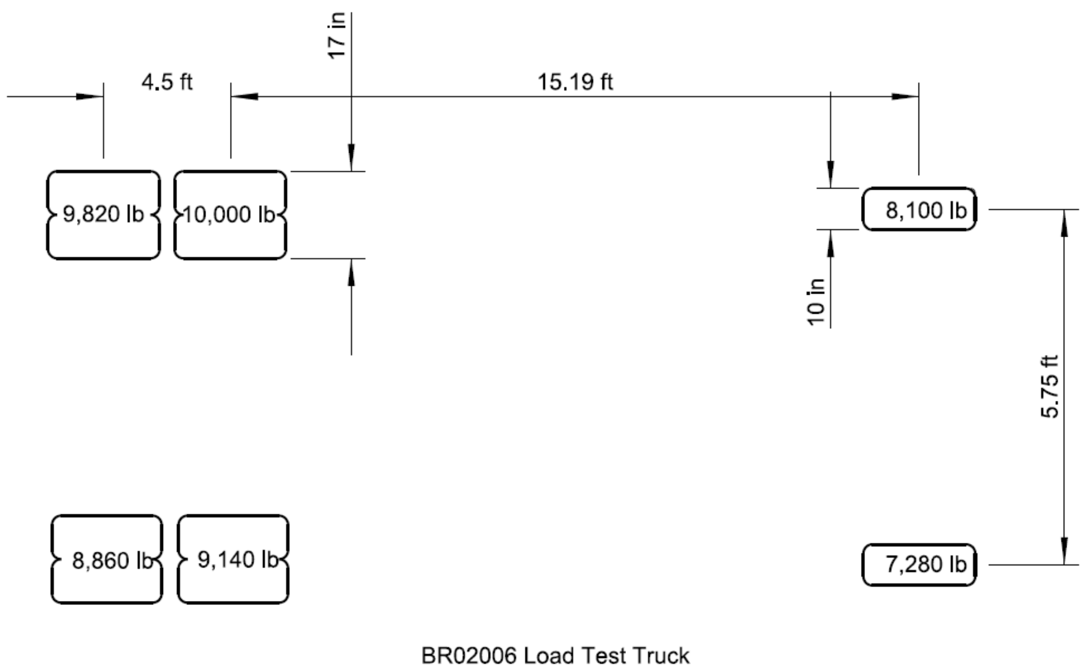


Figure 12: Bridge BR2006 Load Test Vehicle

6.4 Wheel Path During Load Test

One of the main objectives of this diagnostic load test effort is to evaluate the governing transverse distribution factor of deck panels. To ensure the maximum distribution factor was captured at each bridge, 2 to 9 runs where completed on each slab the truck could physically drive across. Scenario-1 in Figure 13 shows the load test wheel paths used in this investigation.

The rear axle positions of the load test truck are shown in 8 different test cases. Rest of the wheel scenarios are shown in Appendix B.

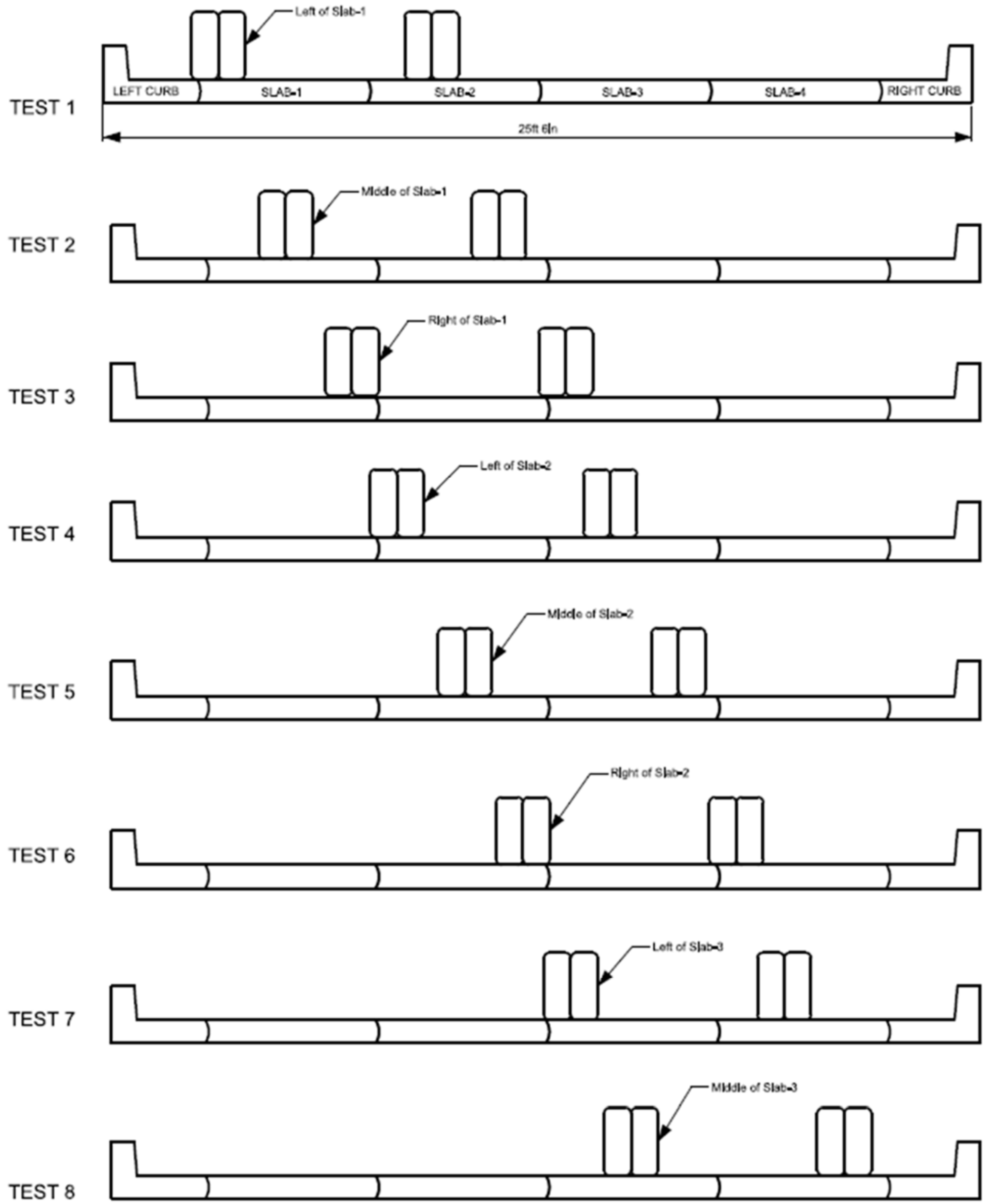


Figure 13: Wheel Locations of the Test Truck on Slab Panels

6.5 Conducting the Load Test

The load test truck was driven at crawl speed (3-5mph) following the wheel path as discussed in the previous section. Longitudinal lines on the joints between two slab panels were marked on the bridge deck using temporary paint to facilitate the wheel line alignment during load test as shown in Figure 14.

In Phase 2, each of the test case was repeated at least twice to ensure the repeatability of tested data. 16 truck passes were conducted to complete 8 sets of test cases. For the first 2 bridges in phase 1 (02006, 07337), 3 passes per test were performed.

Figure 14: Temporary Marking Lines on the Bridge Deck for Proper Truck Alignment During the Load Test

The sampling rate during data collection was set as 10 Hz. On-site data validation was ensured by careful observation of data. Tested data was checked for linearity, reproducibility and symmetry. Any sensor behavior that was inconsistent and unexpected was immediately addressed.

For Phase 1, prior to testing, the trucks were positioned completely off the bridge. Three pre-tests were conducted:

1. Zero out sensors, record data and watch for excessive drift. This allowed cables to "warm up" and any initial fluctuations to stabilize.
2. Re-zero sensors and repeat the above test.
3. Run a single truck across the bridge. Following the test, verify that values are recorded on each sensor, and that their relative magnitudes and signs generally align with expectations. If any apparently erroneous readings are observed, check sensor responsiveness manually, and repeat this pre-test.

For Phase 2, prior to testing, the trucks were positioned completely off the bridge. And the following pre-tests were conducted:

1. Zero out sensors, recorded data and watched for excessive drift. This allowed cables to "warm up" and any initial fluctuations to stabilize.
2. Run a single truck across the bridge. Following the test, verify that values are recorded on each sensor, and that their relative magnitudes and signs generally align with expectations. If any apparently erroneous readings are observed, check sensor responsiveness manually, and repeat this pre-test.

During testing, one person monitored the laptop and recorded data. A second person was responsible for guiding trucks onto assigned marks and coordinating with all team members. The third person drove the truck and two additional people were provided for traffic control.

6.6 Distribution Factor Calculation

The distribution factor of a slab panel was calculated using the following equation:

$$DFM_i = \frac{\varepsilon_i}{\sum_{j=1}^{n} \varepsilon_j}$$

Where ε_i is the maximum measured strain at a slab during a load test and $\sum_{j=1}^{n} \varepsilon_j$ is the summation of strains of all the slabs at the same point in time. See Appendix G for sample calculations.

7 Load Test Results

In this section, load test results for each of tested bridge are presented. Results reported herein include general bridge information, test date, wheel location scenario, tie rod condition, asphalt overlay reflective cracking condition and the calculated Distribution Factor (DF) from load tests. DFs were calculated for all slabs for each test case following the procedure mentioned in Section 6.6 of this report. Maximum DFs for each of the slabs are presented in a bar chart. Maximum strains for all sensors for each test case are also presented in Appendix A. The strain time history for the controlling test case for all tested bridges is presented below. Strain gauges are named in accordance with their locations at a slab. For example, Slab-1L indicates the strain gage installed at Slab-1 near left edge. Similar Slab-2R indicates the strain gage installed near the right edge of Slab-1.

7.1 BR 02006

Table 5: BR02006 Load Test Summary

Asset ID	02006
Date:	11/22/2019
Wheel Scenario.	Wheels on Slab 3 and Slab 4
Reflective Crack	Severe
Tie Rod Condition	Good
Panel Width	5 ft.
No. of Interior Units	4
Span Length	14 ft.
Asphalt Thickness	2 in
Year Built	1954
ADT	210
County	Cherokee
Features Intersected	Unnamed Stream on S 11-226
Latitude	34.93629722
Longitude	-81.73484861
Load Test DF (lane)	**0.50**

Figure 15 BR02006 Top of Deck

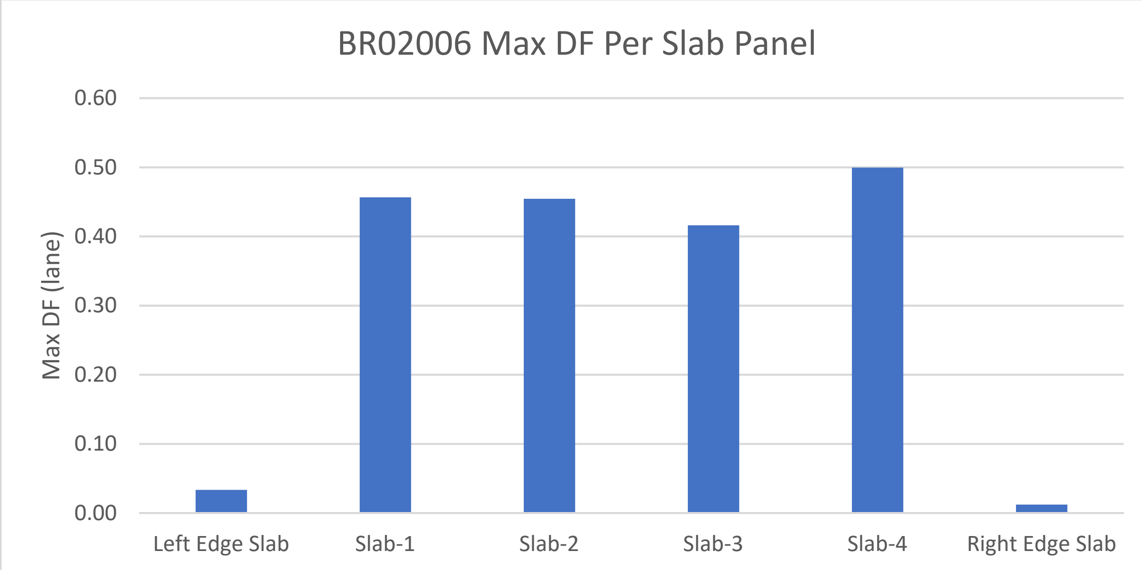


Figure 16 Maximum DF of BR02006 Slabs for all Test Cases

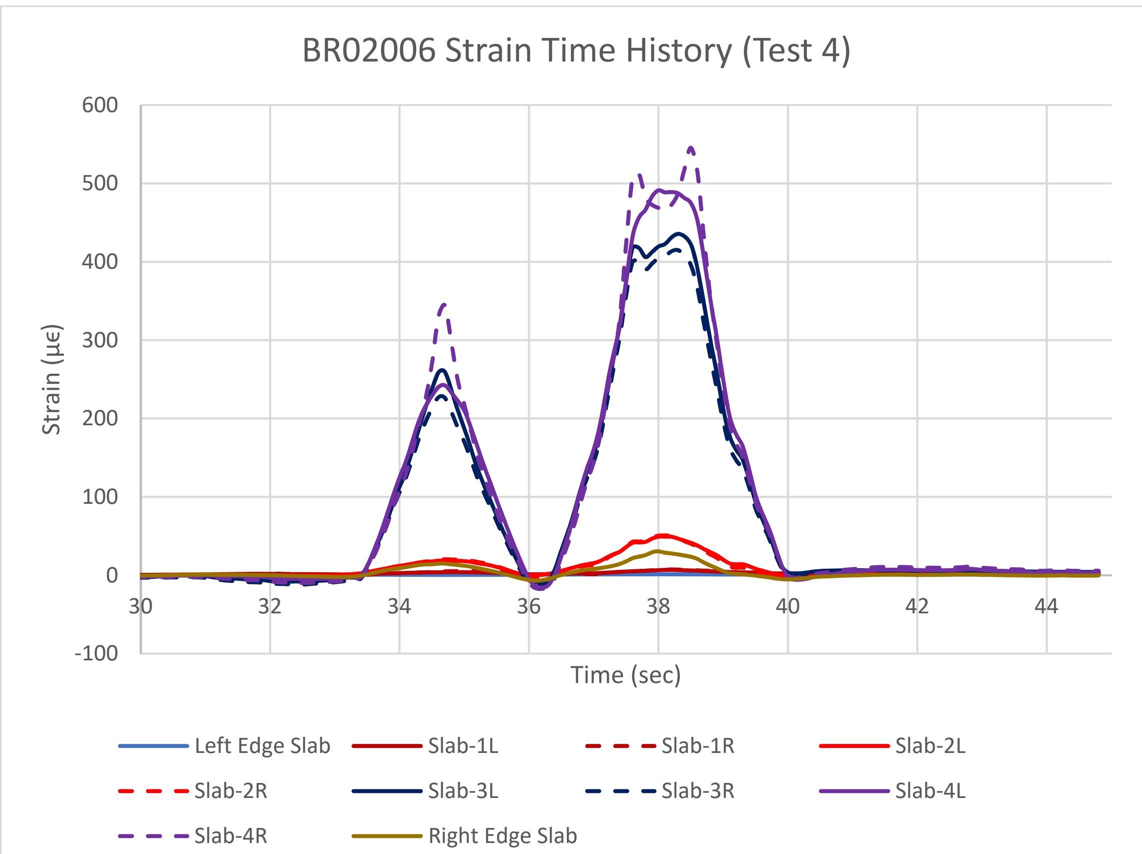


Figure 17 Strain Time History of Controlling Test Case for all Strain Gauges of BR02006 (18-in gauge length)

7.2 BR 02357

Table 6 BR02357 Load Test Summary

Asset ID	BR02357
Date:	02/04/2020
Wheel Scenario.	Wheel on Slab-3 and Slab-4
Reflective Crack	None
Tie Rod Condition	Good
Panel Width	5 ft.
No. of Interior Units	4
Span Length	14 ft.
Asphalt Thickness	3.5 in
Year Built	1956
ADT	2000
County	Greenville
Features Intersected	Grove Creek
Latitude	34.74003611
Longitude	-82.41710194
Load Test DF (lane)	**0.52**

Figure 18 BR02357 Top of Deck

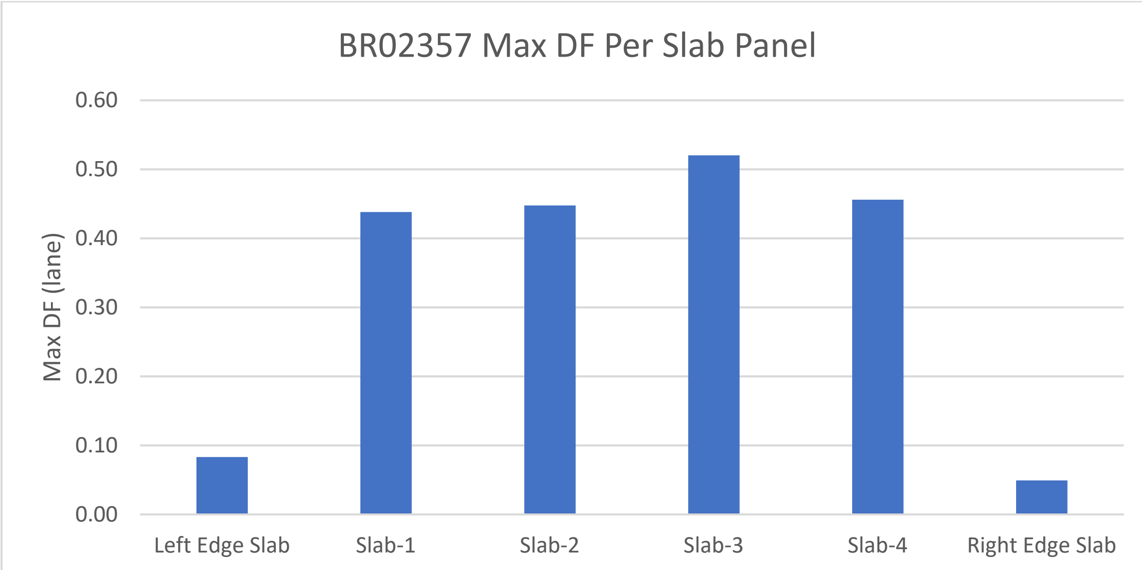


Figure 19 Maximum DF of BR02357 Slabs for all Test Cases

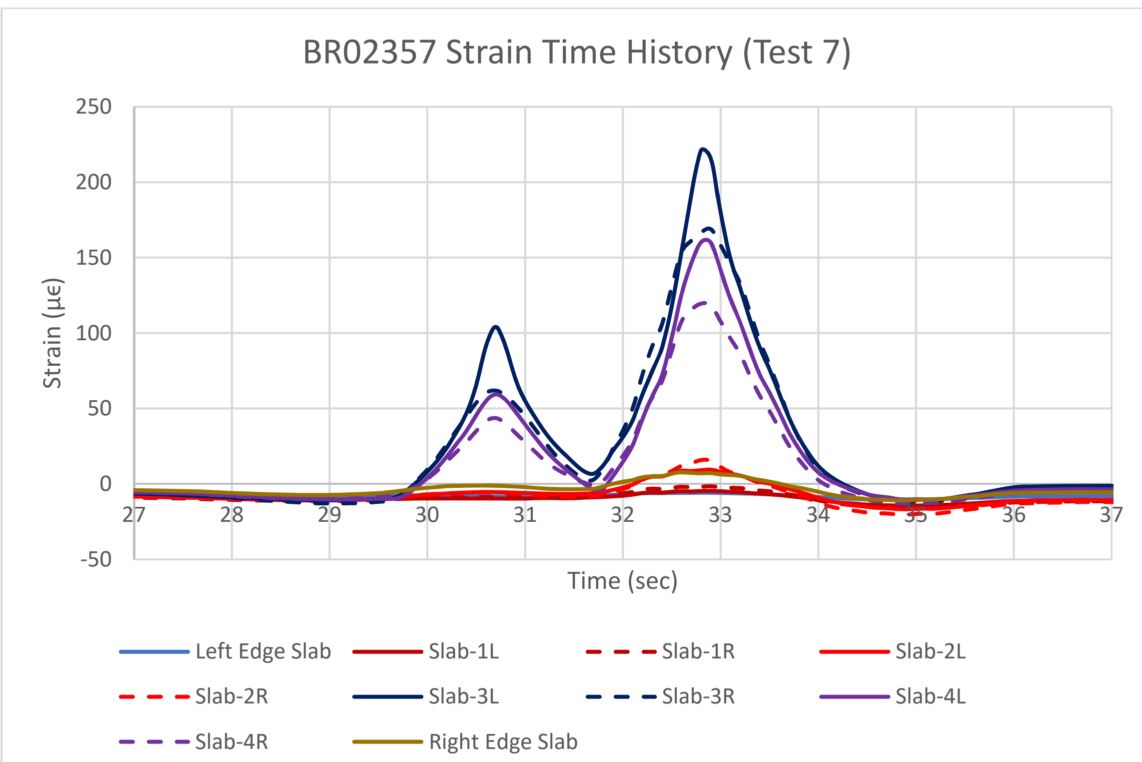


Figure 20 Strain Time History of Controlling Test Case for all Strain Gauges of BR02357 (18-in gauge length)

7.3 BR 02736

Table 7 BR02736 Load Test Summary

Asset ID	BR02736
Date:	02/20/2020
Wheel Scenario.	Wheel on Slab-1 and Slab-2
Reflective Crack	None
Tie Rod Condition	Good
Panel Width	5.5 ft.
No. of Interior Units	4
Span Length	15 ft.
Asphalt Thickness	0.50 in
Year Built	1958
ADT	6500
County	Greenville
Features Intersected	Tributary to Reedy River
Latitude	34.91489167
Longitude	-82.43359
Load Test DF (lane)	**0.43**

Figure 21 BR02736 Top of Deck

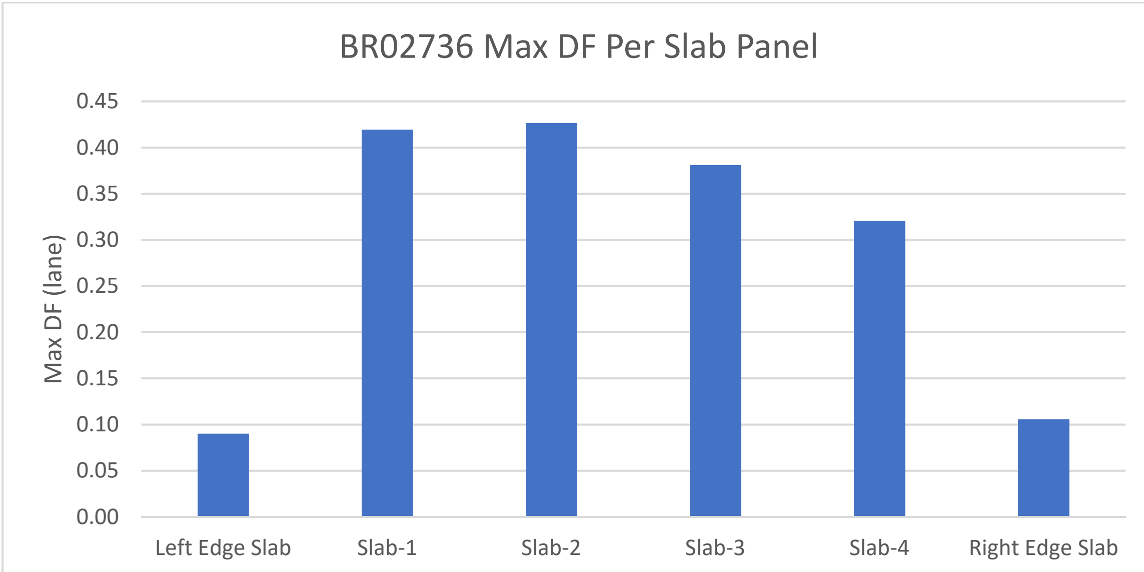


Figure 22 Maximum DF of BR02736 Slabs for all Test Cases

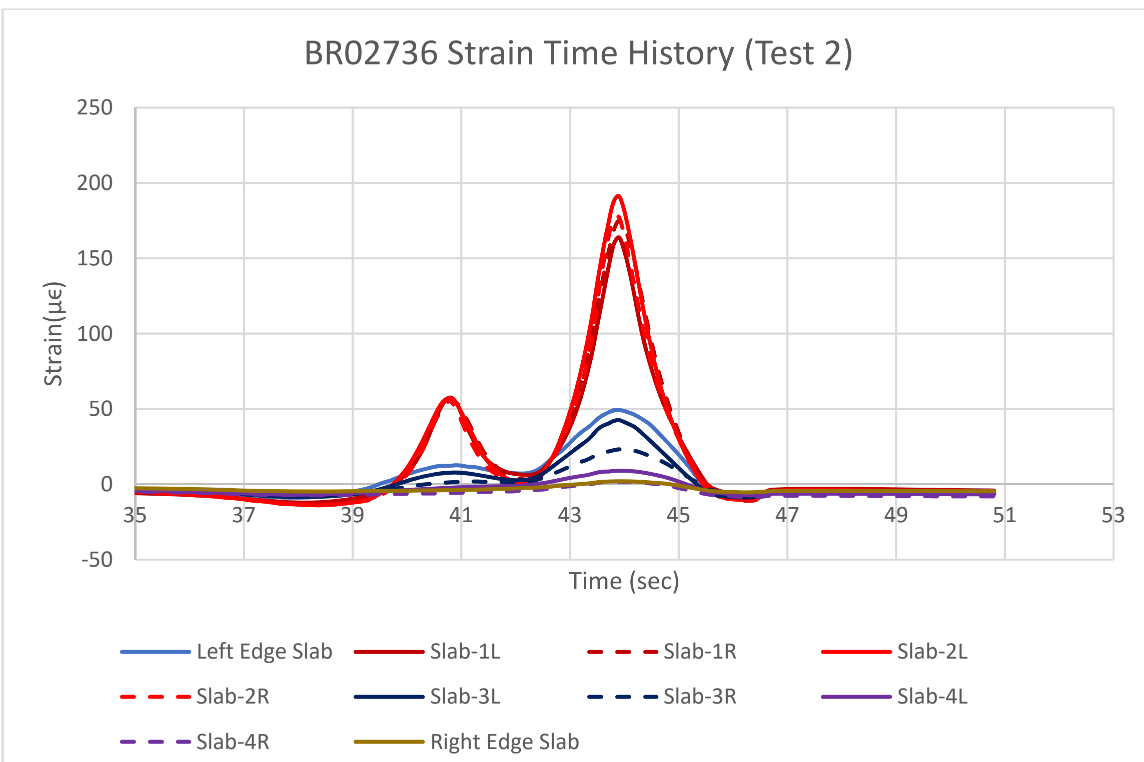


Figure 23 Strain Time History of Controlling Test Case for all Strain Gauges of BR02736 (18-in gauge length)

7.4 BR 05957

Table 8 BR05957 Load Test Summary

Asset ID	BR05957
Date:	02/19/2020
Wheel Scenario.	Wheel on Slab-3 and Slab-4
Reflective Crack	None
Tie Rod Condition	Good
Panel Width	5 ft.
No. of Interior Units	4
Span Length	14 ft.
Asphalt Thickness	2 in.
Year Built	1971
ADT	390
County	Greenwood
Features Intersected	Dunns Creek
Latitude	34.37811389
Longitude	-82.23434028
Load Test DF(lane)	**0.54**

Figure 24 BR05957 Top of Deck

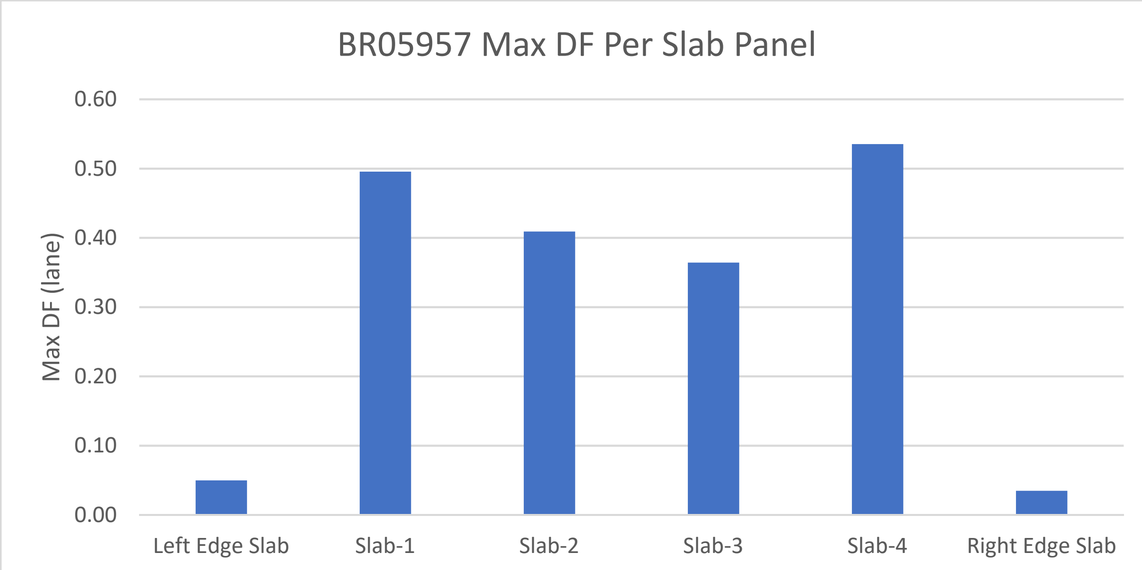


Figure 25 Maximum DF of BR05957 Slabs for all Test Cases

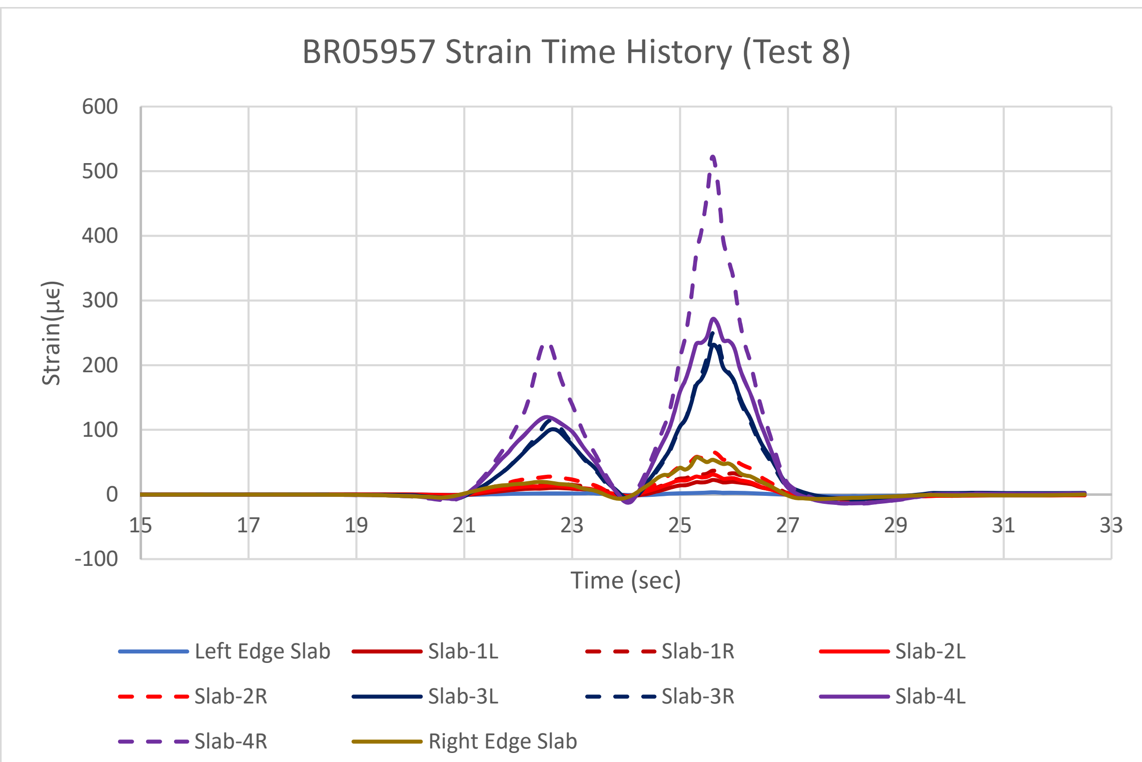


Figure 26 Strain Time History of Controlling Test Case for all Strain Gauges of BR05957 (18-in gauge length)

7.5 BR 06866

Table 9 BR06866 Load Test Summary

Asset ID	BR06866
Date:	02/18/2020
Wheel Scenario.	Wheel on Slab 3 and Slab 4
Reflective Crack	None
Tie Rod Condition	Good
Panel Width	5.5 ft.
No. of Interior Units	5
Span Length	15 ft.
Asphalt Thickness	N/A
Year Built	1976
ADT	2500
County	Greenville
Features Intersected	Grove Creek
Latitude	34.75164167
Longitude	-82.40107194
Load Test DF (lane)	**0.47**

Figure 27 BR06866 Top of Deck

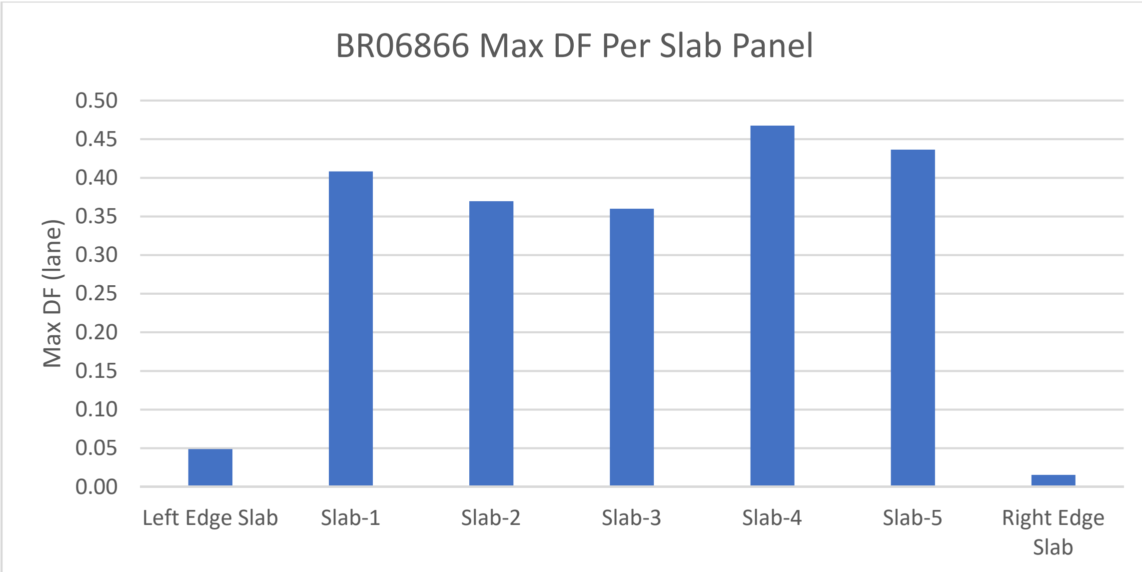


Figure 28 Maximum DF of BR06866 Slabs for all Test Cases

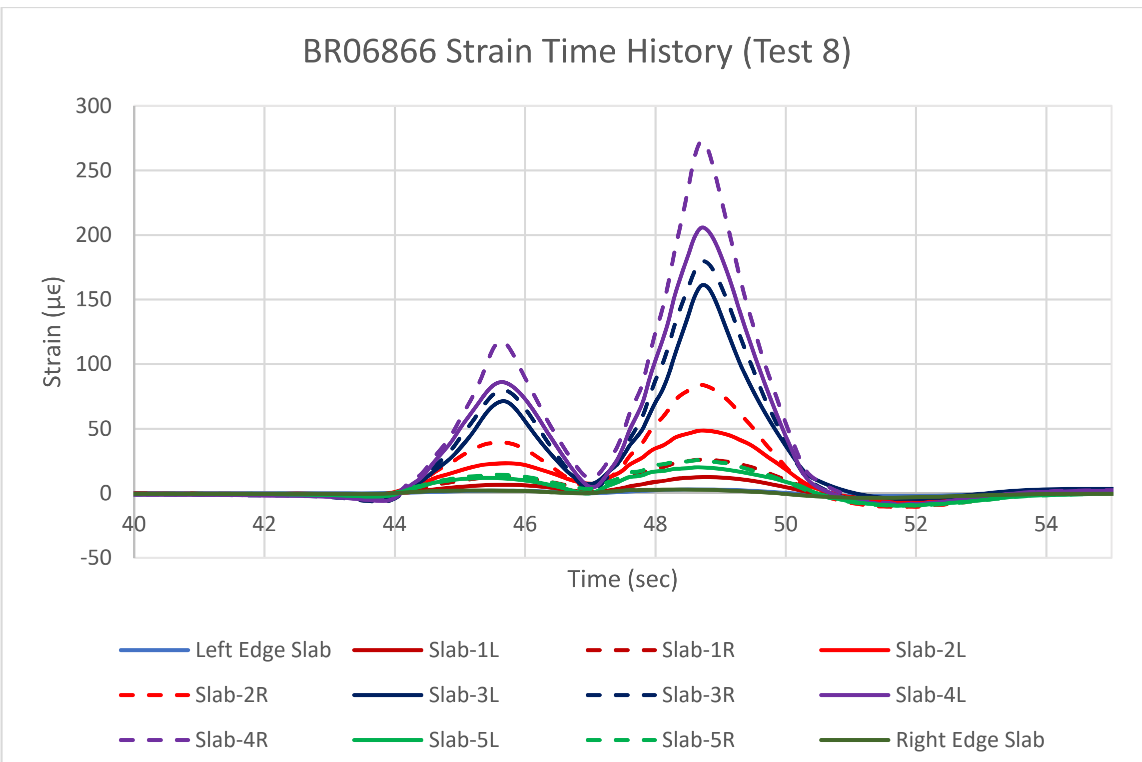


Figure 29 Strain Time History of Controlling Test Case for all Strain Gauges of BR06866 (18-in gauge length)

7.6 BR 07337

Table 10 BR07337 Load Test Summary

Asset ID	07337
Date:	11/22/20
Wheel Scenario.	Wheels on Slab 1 and 2
Reflective Crack	No Asphalt
Tie Rod Condition	Good
Panel Width	5.5 ft.
No. of Interior Units	5
Span Length	15 ft.
Asphalt Thickness	No Asphalt
Year Built	1980
ADT	120
County	Cherokee
Features Intersected	Branch of Thicketty Creek
Latitude	35.00268611
Longitude	-81.68387083
Load Test DF	**0.62**

Figure 30 BR07337 Top of Deck

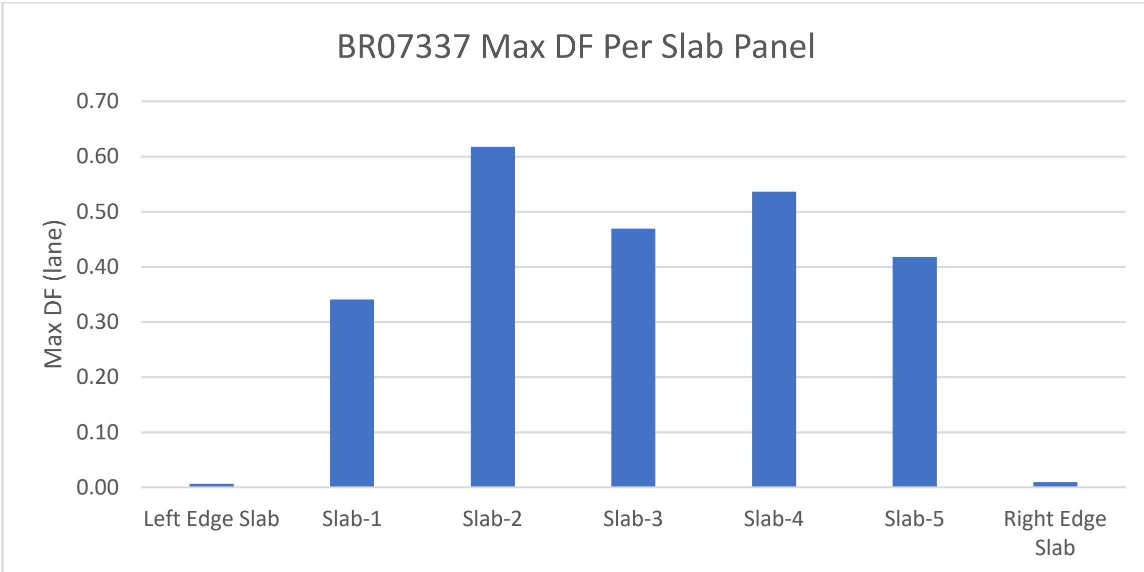


Figure 31 Maximum DF of BR07337 Slabs for all Test Cases

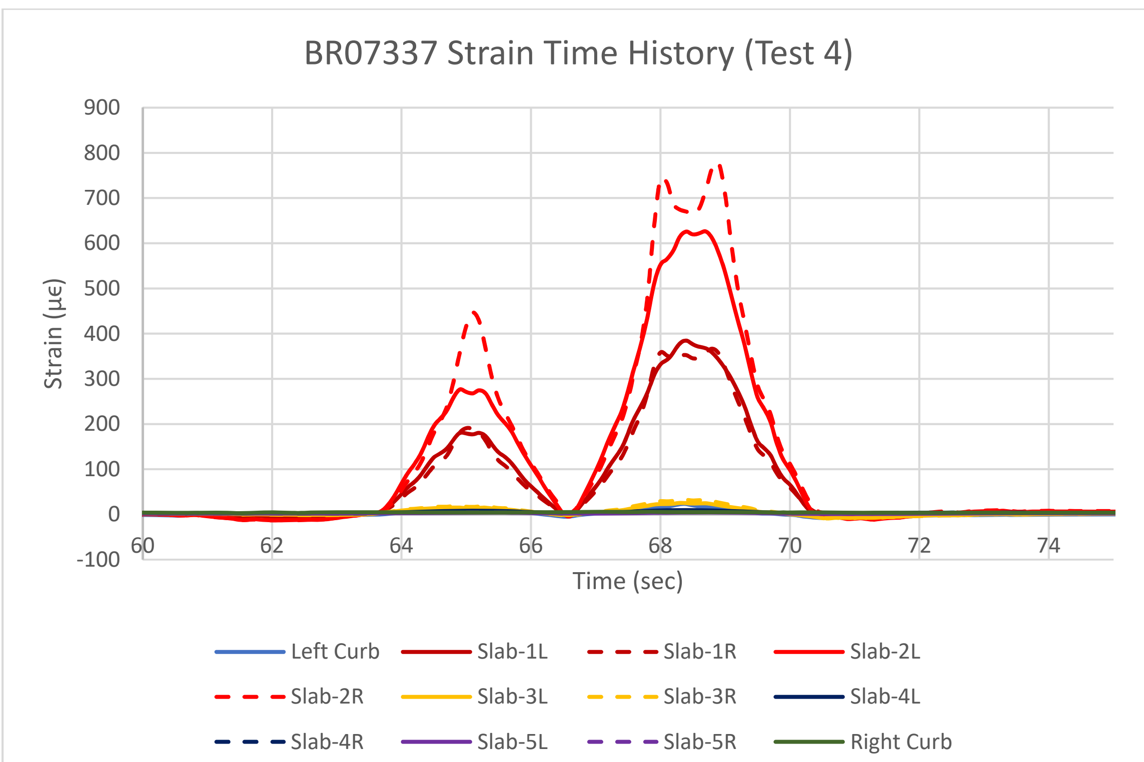


Figure 32 Strain Time History of Controlling Test Case for all Strain Gauges of BR07337 (18-in gauge length)

7.7 Data Interpretation

The distribution factors obtained from the load test were compared against the other inspected parameters. Table 11 lists the potential inspectable parameters along with the load test distribution factors for all the tested bridges. The parameters are Year Built, Design Live Load, Slab Width, Reflective Cracking, Superstructure Condition, Tie Rod Condition, Asphalt Thickness and Gap between slabs.

It was observed that there was no direct correlation between any of the inspectable parameters and the distribution factor from the load test. From the wheel locations of the load test truck, it was observed that for all test cases, one wheel line was on one slab and the other wheel line was on the adjacent slab. If there is no load sharing among the slabs, one slab should entirely take the load coming from one wheel line and the distribution factor for a slab panel should be 0.5. Reflective cracking was assumed to be a controlling factor in lateral load distribution among the slabs. But in the load test results, we have observed that all tested bridges gave the same distribution factor around 0.50 regardless of the reflective cracking conditions. The longitudinal match cast interface between slabs are working as hinge connections and almost no moment is transferred through the hinges in the transverse direction. The load test results showed that irrespective of the condition of the inspection parameters, the distribution factor for the interior precast slab panels are approximately 0.5 lane for all bridges.

Table 11 Parameters of Load Test Distribution Factor of Slab Bridges

Asset ID	**Year Built**	**Design Live Load**	**Slab Width**	**Reflective Cracking**	**Superstructure Condition**	**Tie Rod Condition**	**Asphalt Thickness**	**Controlling Slab Units**	**Avg. Gaps of Controlling Panel**	**Tested DF**
		(H10/H15)	(ft.)	(None/Minor/Moderate/Severe)	1-9	(Good/Poor)	(in.)	Slab-1 to Slab-4/5	(in.)	(Lane)
2006	1954	H10	5	Severe	5	Good	N/A	Slab-4	N/A	0.50
2357	1956	H15	5	None	5	Good	3.50	Slab-3	0.07	0.52
2736	1958	H10	5.5	None	7	Good	0.50	Slab-2	0.075	0.43
5957	1971	H10	5	None	6	Good	2.00	Slab-4	0.065	0.54
6866	1976	H15	5.5	None	7	Good	N/A	Slab-4	0.11	0.47
7337	1980	H15	5.5	No Asphalt	6	Good	No Asphalt	Slab-2	N/A	0.62

8 Comparison of Load Test Results

8.1 Comparison of AASHTO and Load Tested Distribution Factors

Table 12 shows the comparison between AASHTO and tested distribution factors. The multiple presence factor of 1.2 is incorporated within the LRFD single-lane distribution factor equations. To make the results are comparable, the AASHTO LRFR distribution factors shown in Table 12 are values without considering the multiple presence factor. Details of the calculation are attached in Appendix C. Since two axles of tested vehicles have different weight during the load testing, the tested distribution factors may have a value slightly higher than 0.5 lane. LFR distribution factors shown in Table 12 are adjusted with a limitation of 1 wheel maximum.

Table 12 Comparison of AASHTO and Tested Distribution Factor

Asset ID	Year of Built	Standard Plan	AASHTO		Tested	
			LFR (Wheel)	LRFR (Lane)	LFR (Wheel)	LRFR (Lane)
02006	1954	H10 14' span 24' rdwy (1949)	1.043	0.491	1	0.50
02357	1956	H15 14' span 24' rdwy (1953)	1.043	0.491	1	0.52
02736	1958	H10 15' span 26' rdwy (1957)	1.133	0.505	0.86	0.43
05957	1971	H10 14' span 24' rdwy (1949)	1.043	0.491	1	0.54
06866	1976	H15 15' span 31'-6" rdwy (1957)	1.133	0.485	0.94	0.47
07337	1980	H15 15' span 31'-6" rdwy (1957)	1.133	0.485	1	0.62

8.2 Comparison of Theoretical and Tested Concrete Strain

Actual maximum strain measured during load testing and corresponding calculated strain due to test vehicle were compared, as shown in Table 13.

Actual strain (ε_T) was calculated using the measured strain divide by an amplification factor equal to the ratio between gage length (L_{gage}) and standard gage length (3 in.), then times 1.1 to include the extension effect, as suggested by the BDI user manual v2.2.

Theoretical strain (ε_c) was calculated using following equation per Manual of Bridge Evaluation 3rd Edition 8.8.2.3.1-3:

$$\varepsilon_c = \frac{L_T}{S * E}$$

Where:

L_T = calculated theoretical load effect in member corresponding to measured strain

S = section modulus of transformed section

E = member modulus of elasticity

Since slab behaviors are within elastic range under tested trucks, uncracked transformed section modulus was used for the calculation of theoretical strain. Detailed calculations are shown in Appendix D. It was found that the theoretical strains were greater than the measured strains during the load tests. The reasons for this difference are likely due to a higher modulus of elasticity of the concrete, some partial rotational stiffness at the supports and the partial stiffness contribution from the asphalt overlay. The maximum deflections of Bridge 07337 and 02006 are compared with the theoretical deflections, as shown in Table 14. Detailed calculations are attached in Appendix D.

Table 13 Comparison between Theoretical and Tested Strain

Asset ID	Year of Built	Design Load	Tested Concrete Strength (ksi)	Load Distribution Factor (Wheel)	BDI Max Strain[1] ($x10^{-6}$)	Actual Max Strain ($x10^{-6}$)	Theoretical Strain ($x10^{-6}$)	Actual/Theoretical Strain
02006	1954	H10	7.47	1	493.95	90.56	147.16	0.62
02357	1956	H15	2.74[2]	1	188.99	34.65	56.48	0.61
02736	1958	H10	4.9	0.86	188.26	34.51	60.80	0.57
05957	1971	H10	6.43	1	393.48	72.14	66.74	1.08
06866	1976	H15	5.01	0.94	244.54	44.83	51.29	0.87
07337	1980	H15	4.47	1	669.37	122.72	135.03	0.91

1. BDI max average strain of controlling slab under controlling test
2. Design concrete strength (4ksi) was used to calculate the theoretical strain for 02357
3. Strain gauge length is 18 in

Table 14 Comparison between Theoretical and Tested Deflection

Asset ID	Test DF (Wheel)	Test Deflection (in.)	Theoretical Deflection (in.)	Test/Theoretical
BR 02006	1	0.0522	0.096	0.544
BR 07337	1	0.044	0.091	0.484

9 Load Rating Modification Factor K from Load Testing

The calculated load rating results can be modified based on load testing results through an adjustment factor "K" per MBE 3rd 8.8.2.3.

$$K = 1 + K_a K_b$$

Where $K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1$, to account for the benefit derived from load test. K_a is positive for all tested slab bridges in this report with a value between 0 to 0.90 approximately.

Where K_b is to account for uncertainties of the enhancements accounted for in the K_a calculation, as determined in Table 15. A "K" factor above 1, represents a benefit of field load test on the load rating factor. T and W in this table are unfactored test vehicle effect and unfactored gross rating load effect, respectively.

As a sample calculation, we selected Type 3-3 shown in Table 16. In Table 16, "T" is the unfactored load effect from the load test truck and "W" is the unfactored load effect of Type 3-3 truck. Theoretical cracking moment (M_{cr}) calculated using core test concrete compressive strength of tested slab bridges are about 37 to 48 kip-ft, depending on the cross-sectional properties and concrete compressive strength of slab units. If W is smaller than the theoretical cracking moment (M_{cr}), member is within the elastic range and has a linear behavior under rating load level. The factor 1.33 is to provide some assurance to make sure structure has adequate reserve capacity beyond the rating load level. So, in the process of determining the "K_b" factor, if 1.33W is smaller than the theoretical cracking strain, then member behavior can be extrapolated to 1.33W.

Table 15 Values for K_b (MBE 3rd Table 8.8.2.3.1-1)

Can member behavior be extrapolated to 1.33W?		Magnitude of Test Load			K_b
Yes	No	$\frac{T}{W} < 0.4$	$0.4 < \frac{T}{W} \leq 0.7$	$\frac{T}{W} > 0.7$	
√		√			0
√			√		0.8
√				√	1.0
	√	√			0
	√		√		0
	√			√	0.5

Table 16 Calculations of K_a, K_b and K for Load Rating Vehicle Type 3-3 (Example)

Asset ID	S_{trans} (in^3)	M_{cr} (kip-ft.)	Test DF (Wheel)	T (kip-ft.)	W (kip-ft.)	1.33W < M_{cr}	T/W	K_a	K_b	K
02006	754.16	41.22	1	45.31	31.65	No	1.43	0.63	0.5	1.31
02357	1048.25	41.93	1	19.67	31.65	No	0.62	0.63	0	1.00
02736	843.75	37.35	0.86	18.22	30.16	No	0.60	0.76	0	1.00
05957	758.55	38.47	1	19.67	31.65	No	0.62	-0.07	0	1.00
06866	1079.40	48.32	0.94	19.81	32.79	Yes	0.60	0.14	0.8	1.12
07337	1085.47	45.90	1	50.51	35.07	No	1.44	0.10	0.5	1.05

10 Conclusion

Based on the results and observations of all tested bridges, we recommend using 0.5 (lane) as distribution factor for precast panel slab bridges that are similar to the slab bridges presented in this report. The inspectable parameters has negligible effect on the lateral load distribution among the panels of the slab bridges.

Appendix A: Maximum Strains of All Sensors

In Appendix A, maximum strains recorded during the load test of all bridges are presented. The strain gauges are named accordingly to the slab and side of the slab they are located in. For example, Slab-1L indicates the strain gage installed at the first interior slab near the left edge. Similar Slab-1R indicates the strain gage installed near the right edge of the first interior slab. If the strain presented here is to be used to verify a model or any other types of calculation, the value must be adjusted for the gauge length of the sensor.

Table 17 BR02006 Maximum Strains of all Strain Gauges for all Test Cases

Test	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab-2R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Right Edge Slab
1	1	44.94	412.44	382.81	357.74	336.64	60.90	64.04	14.01	11.68	4.12
	2	65.77	398.43	360.17	333.90	331.17	55.81	59.06	11.21	9.81	1.85
	3	64.89	387.21	362.13	332.90	345.82	56.59	59.61	11.08	10.73	5.21
2	4	51.93	297.91	383.95	333.92	449.28	81.80	89.12	15.75	13.20	2.93
	5	52.89	305.24	374.61	324.81	436.93	74.92	80.91	14.74	12.24	2.09
	6	52.80	304.83	368.43	322.72	440.09	74.22	79.91	15.49	13.76	4.82
3	11	4.40	87.89	134.20	229.24	333.32	355.78	363.11	239.64	173.58	8.59
	12	7.86	94.26	139.39	234.45	331.42	362.42	356.80	238.09	170.59	10.88
4	15	2.23	12.90	12.43	81.35	83.74	511.58	374.71	572.88	441.08	23.73
	17	1.33	6.81	7.36	49.33	50.68	435.61	414.94	491.15	545.48	30.44
5[1]	19	33.19	343.48	373.62	350.46	474.24	516.86	482.19	524.36	540.06	31.53
	20	50.77	329.23	356.00	356.89	473.78	510.06	479.44	520.82	538.94	31.64
	21	50.70	306.47	347.05	358.87	488.80	507.81	491.23	511.82	568.59	36.47

[1]Two trucks were used side by side

Table 18 BR02357 Maximum Strains of all Strain Gauges for all Test Cases

Test	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab-2R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Right Edge Slab
1	1	46.14	180.34	126.60	140.88	153.69	13.10	15.39	5.17	1.61	1.30
	2	63.26	159.32	117.10	142.06	141.37	14.01	14.01	5.48	-0.15	-0.20
2	3	19.29	145.48	150.87	120.23	206.62	35.68	31.12	11.00	1.83	-0.06
	4	22.34	149.95	155.00	124.45	214.58	39.15	32.66	14.37	7.23	2.30
3	5	15.95	109.37	157.59	97.12	144.21	142.90	82.97	29.52	11.01	-0.14
	6	16.49	111.04	156.31	99.55	162.51	127.41	75.84	29.33	13.36	2.28
4	7	5.93	46.83	66.74	127.83	111.22	160.62	147.28	45.86	26.82	3.80
	8	5.44	49.05	71.01	121.42	105.29	164.92	144.85	43.71	24.43	2.30
5	9	6.78	33.45	44.03	120.50	144.67	128.66	183.89	54.11	36.72	2.67
	10	6.64	31.53	41.33	109.93	139.60	121.39	165.87	53.27	36.95	3.12
6	11	9.09	27.60	39.65	95.96	189.72	92.53	137.95	112.49	67.38	7.13
	12	8.32	27.43	40.19	96.18	192.43	90.78	132.10	113.75	69.19	9.05
7	13	0.03	-0.09	-0.18	9.47	15.79	221.12	168.85	160.23	119.53	7.78
	14	3.17	6.31	12.12	27.37	53.27	210.63	165.70	166.69	120.54	15.34
8	15	2.47	4.50	7.69	11.75	11.33	155.86	189.45	161.65	194.16	32.98
	16	1.67	3.94	7.65	11.40	10.91	144.78	187.10	148.70	182.86	41.47

Table 19 BR02736 Maximum Strains of all Strain Gauges for all Test Cases

Test	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab-2R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Right Edge Slab
1	1	81.11	205.91	172.71	228.11	119.20	27.02	21.23	11.17	9.29	5.73
	2	85.64	214.60	183.47	237.42	130.56	40.60	29.78	18.44	16.93	13.11
2	3	62.56	183.64	184.65	207.65	176.86	50.95	32.13	18.96	8.20	4.84
	4	49.43	163.88	174.98	191.27	177.27	42.60	23.16	9.06	1.79	2.03
3	5	33.77	98.04	194.55	162.22	216.19	117.46	49.90	25.85	21.18	16.62
	6	42.47	109.57	210.62	178.45	227.27	128.33	58.55	35.03	27.76	21.76
4	7	20.63	54.28	109.27	216.35	123.20	156.29	81.96	34.15	22.27	18.00
	8	23.39	57.41	113.83	214.89	123.46	162.68	81.65	35.13	21.13	15.55
5	9	15.37	38.83	78.15	163.69	154.33	123.10	130.22	46.38	30.86	22.36
	10	13.49	33.76	67.36	141.31	164.30	111.42	136.80	49.78	31.38	24.13
6	11	10.07	26.82	56.82	121.02	201.47	108.72	143.65	69.05	39.96	28.29
	12	18.58	35.72	65.48	132.31	206.94	117.68	153.25	71.89	40.61	29.20
7	13	6.48	14.05	26.00	54.27	97.65	169.87	108.32	129.41	67.62	50.61
	14	11.73	20.20	32.91	58.53	94.76	182.25	120.85	138.71	81.51	60.86
8	15	9.73	17.16	25.47	43.77	53.27	142.44	121.28	114.34	115.58	76.06
	16	6.97	14.02	23.24	40.77	50.77	138.08	118.17	108.44	111.44	72.22

Table 20 BR05957 Maximum Strains of all Strain Gauges for all Test Cases

Test	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab-2R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Right Edge Slab
1	1	64.13	423.33	213.68	210.80	214.23	61.74	46.50	31.48	22.93	4.36
	2	64.91	427.74	221.12	218.98	204.22	62.39	45.08	30.70	21.82	4.58
2	3	45.50	276.80	210.67	196.83	314.15	77.50	60.23	38.44	31.82	6.04
	4	47.97	270.07	218.97	202.36	343.96	84.56	67.35	44.56	35.62	10.38
3	5	34.27	196.78	208.93	183.57	244.40	184.95	102.98	54.36	46.09	6.39
	6	31.42	180.40	207.83	178.61	210.74	217.61	116.72	60.05	51.69	8.38
4	7	21.25	109.83	165.51	189.92	167.20	254.22	190.34	90.99	88.24	16.01
	8	21.53	110.51	163.30	182.94	158.50	249.43	179.05	85.82	81.59	12.34
5	9	20.55	93.74	139.71	152.27	193.05	208.39	239.73	100.06	105.15	15.57
	10	19.10	90.78	136.33	152.72	191.71	202.67	233.81	97.09	102.52	15.73
6	11	12.25	66.80	102.81	107.78	279.31	156.35	215.88	221.49	179.74	23.24
	12	13.12	69.95	106.55	110.89	272.52	154.26	207.81	212.92	180.59	25.16
7	13	5.79	31.92	48.70	45.16	83.04	334.35	197.91	314.17	374.24	35.71
	14	4.75	30.09	46.41	44.20	80.77	331.45	195.28	310.43	359.71	33.26
8	15	3.23	21.50	36.60	31.02	63.67	235.35	233.80	268.76	511.98	45.16
	16	3.35	22.39	36.67	30.84	65.44	230.41	248.99	270.92	522.29	57.22

Table 21 BR07337 Maximum Strains of all Strain Gauges for all Accepted Test Cases

Test	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab2-R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Slab-5L	Slab-5R	Right Edge Slab
1	1	5.23	4.62	0.00	6.36	7.55	20.88	28.52	492.52	504.95	401.45	374.31	19.54
	6	2.18	0.21	0.00	3.36	3.16	19.96	30.52	589.98	444.31	476.92	318.50	18.31
2	8	0.72	3.04	2.91	23.05	22.14	367.13	707.20	548.03	622.24	30.50	19.84	2.78
	12	0.26	0.44	0.31	22.40	21.77	427.95	602.22	618.24	512.78	20.72	13.15	0.98
3	13	4.74	21.28	19.42	582.06	621.22	398.06	865.10	41.96	31.76	7.03	4.04	3.34
	16	8.12	33.62	29.99	709.65	565.66	468.67	736.42	29.21	23.52	10.55	6.61	5.94
4	20	23.10	384.54	366.71	626.21	777.32	25.32	32.23	9.62	8.08	4.37	5.03	6.03
	23	24.31	438.54	327.81	714.94	619.24	18.97	24.56	6.37	6.91	4.32	2.74	4.55
5[1]	26	1.04	17.21	15.08	558.08	562.37	452.01	905.33	553.23	565.23	424.09	390.05	22.32
6[1]	30	14.63	376.28	351.11	669.18	839.99	398.66	793.81	563.41	670.60	87.37	37.65	7.31

[1]Two trucks were used side by side

Table 22 BR06866 Maximum Strains of all Strain Gauges for all Test Cases

TEST	Run	Left Edge Slab	Slab-1L	Slab-1R	Slab-2L	Slab-2R	Slab-3L	Slab-3R	Slab-4L	Slab-4R	Slab-5L	Slab-5R	Right Edge Slab
1	1	44.87	219.52	139.33	189.63	102.90	60.87	33.96	23.34	11.80	4.47	2.26	1.34
	2	41.37	222.78	141.23	189.78	104.72	62.69	35.13	35.77	46.33	32.50	20.68	3.48
2	3	17.89	187.19	143.06	170.53	137.02	81.04	47.93	30.18	17.43	5.02	2.22	0.57
	4	17.43	168.70	140.46	163.91	146.29	84.70	50.88	30.67	18.35	5.85	3.15	0.79
3	5	13.99	108.62	158.43	157.75	185.77	150.89	81.26	44.87	43.61	49.69	41.10	5.18
	6	13.49	100.40	154.96	159.00	178.72	156.34	86.23	47.61	37.25	47.31	31.51	4.07
4	7	11.37	80.20	119.94	205.82	154.83	210.82	118.76	64.90	46.77	10.44	6.57	1.05
	8	10.57	69.61	93.82	201.78	147.67	185.31	118.85	68.17	50.55	26.95	19.36	3.51
5	9	8.75	47.77	70.73	139.51	154.32	157.09	180.75	97.71	78.91	14.98	11.32	1.87
	10	8.67	54.38	78.83	160.67	152.31	168.27	152.04	81.04	62.17	12.83	8.79	1.25
6	11	6.38	32.60	50.44	99.17	175.71	154.97	205.50	162.06	120.12	17.38	14.83	2.18
	12	6.21	31.68	50.56	100.20	173.46	163.06	202.87	176.17	127.73	17.60	15.49	2.18
7	13	4.33	18.77	33.12	66.52	108.83	216.31	169.21	212.96	188.22	22.90	24.09	3.50
	14	4.21	18.79	33.45	67.63	110.64	222.18	172.73	217.29	200.03	23.25	26.40	3.39
8	15	2.97	12.56	26.04	48.56	83.89	160.68	179.20	205.54	272.49	20.10	25.53	2.88
	16	3.92	14.71	27.66	46.01	78.90	147.90	180.37	204.19	295.92	20.95	27.93	3.47
9	17	2.13	9.15	20.31	35.68	63.66	117.27	208.47	185.04	290.33	99.16	65.30	6.81
	18	2.36	8.43	19.72	33.73	60.23	113.56	205.27	177.01	274.19	125.29	74.39	7.12
10	19	0.85	7.46	14.84	22.16	37.72	71.94	120.48	209.21	166.92	243.33	172.64	12.98
	20	1.66	6.43	12.76	19.75	36.30	69.00	117.57	202.99	165.41	238.56	165.46	13.39
11	21	1.39	3.12	9.67	14.07	27.82	49.95	89.96	160.81	196.65	208.08	238.60	15.84
	22	0.58	2.33	9.10	13.92	28.19	51.36	92.28	165.17	193.49	207.64	229.26	15.36

Appendix B: Alternative Truck Load Paths

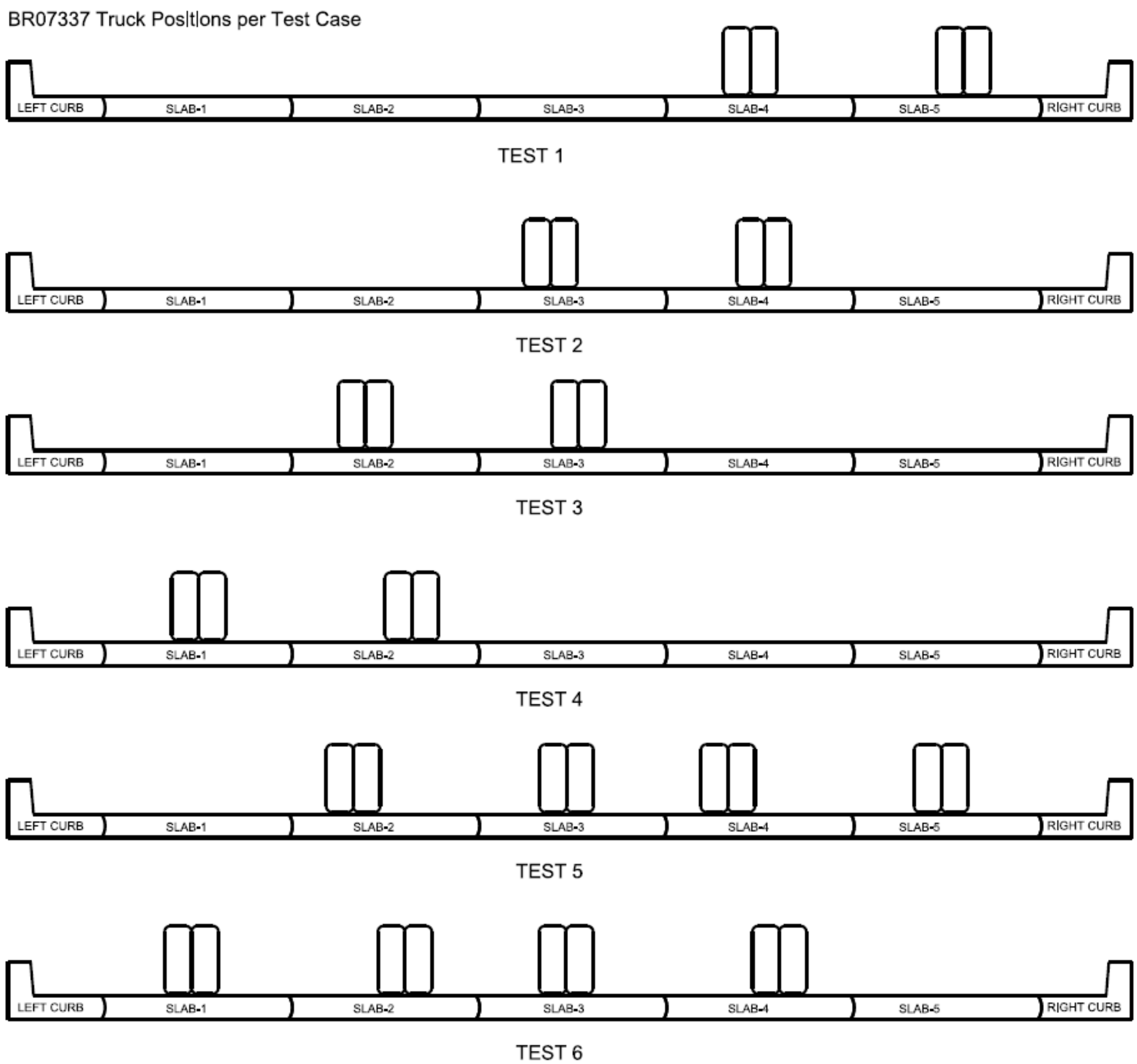


Figure 33: Load Test Wheel Path for BR07337 in Phase-1

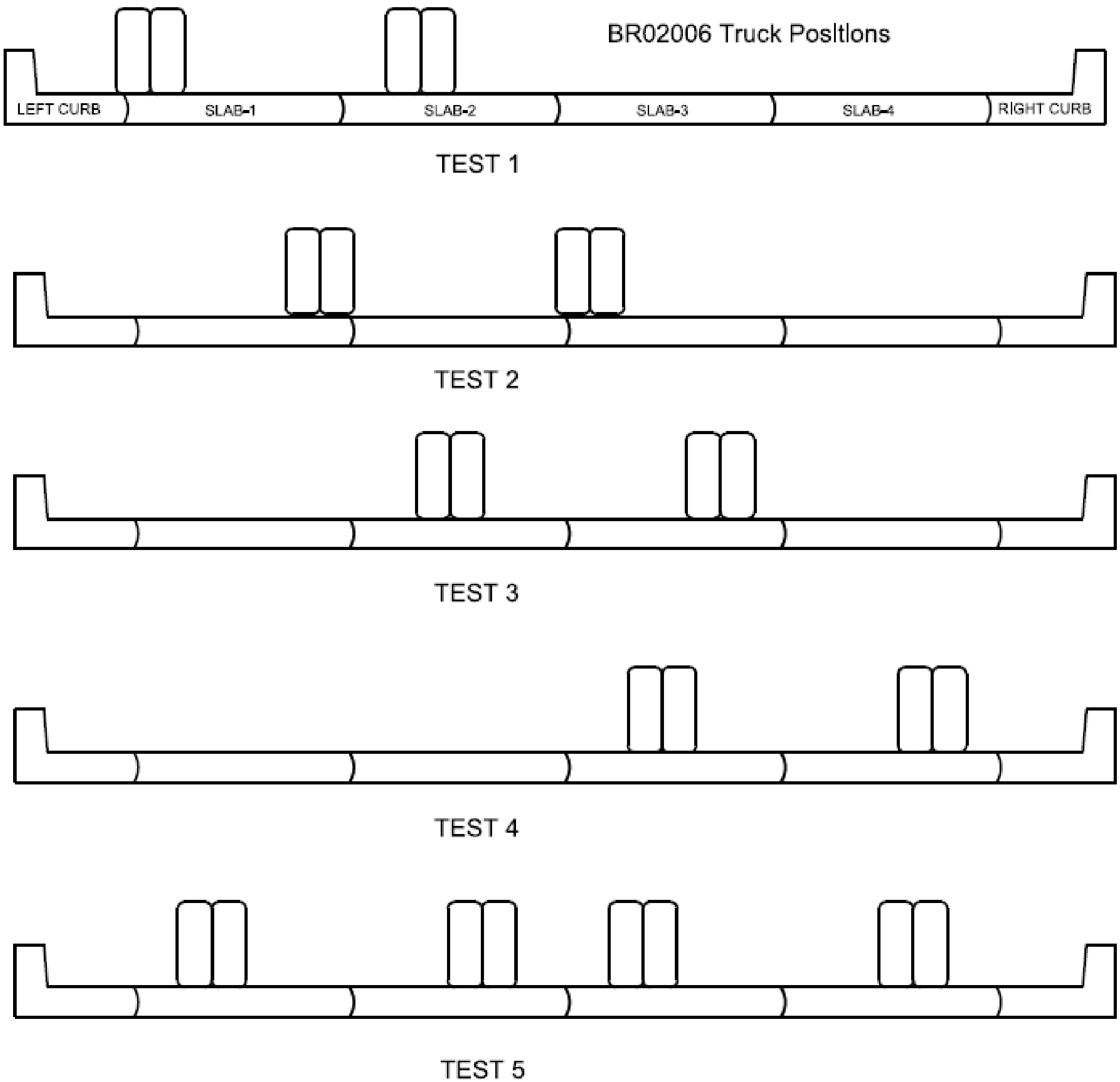


Figure 34: Load Test Wheel Path for BR02006

Appendix C: AASHTO Distribution Factor Calculations and Postings

Distribution factors calculated based on AASHTO Equations:

Asset ID	Year of Built	Standard Plan	LFR (Wheel)	LRFR[1] (Lane)
02006	1954	H10 14' span 24' rdwy (1949)	1.043	0.589
02357	1956	H15 14' span 24' rdwy (1953)	1.043	0.589
02736	1958	H10 15' span 26' rdwy (1957)	1.133	0.606
05957	1971	H10 14' span 24' rdwy (1949)	1.043	0.589
06866	1976	H15 15' span 31'-6" rdwy (1957)	1.133	0.582
07337	1980	H15 15' span 31'-6" rdwy (1957)	1.133	0.582

1. Multiple presence factors are incorporated in the single-lane AASHTO equations

Example: H10 14FT SPAN 24FT RDWY (1949) (LRFR)

```
=========================================
  Moment and Shear Distribution Factors
=========================================


--------------------------------------
            Region 1
--------------------------------------
Start Distance:        0.00(ft)
End Distance:         13.25(ft)
Properties at:         6.63(ft)
Theta:                 0.00(Deg)


-------------------------------------------------
Equivalent Strip Widths for Slab-Type Bridges
(Article 4.6.2.3)
-----------------------------------------------


Compute Shear and Moment Distribution Factors
---------------------------------------------
Input:
L1              =        13.25(ft)
Single Lane W1 =        25.50(ft)
Multi Lane W1  =        25.50(ft)
NL              =         2

One Design Lane Loaded:
E = 10.0 + 5.0 * SQRT(L1*W1) =  101.907 (in)
DF = (1 lane/E)*Strip Width =    0.589 Lanes

Two or More Design Lanes Loaded:
E = 84.0 + 1.44 * SQRT(L1*W1) <= 12.0*W/NL =  110.469 <=  153.000 =  110.469 (in)
DF = (1 lane/E)*Strip Width =    0.543 Lanes
```

Example: H10 14FT SPAN 24FT RDWY (1949) (LFR)

```
AASHTO Standard Specifications for Highway Bridges, Seventeenth  Edition - 2002

---------------------------------------
Concrete Deck on Interior Beams
(Article 3.24.4)
---------------------------------------


============================
Moment Distribution Factors
============================

Input:

S = Avg span length =  13.25  (ft)

E = 4.0 + 0.06S <= 7.0'

W = Strip Width =   5.00  (ft)


One Design Lane Loaded:

DF =  W/E =    1.04
```

The following are anticipated H10/15-44 slab bridge LFR load rating factors for legal loads at operating level based on the suggested distribution factor (1 Wheel) with different thickness of wearing surface.

H10-44 14 FT SPAN (1949)

Vehicle Type	Axle Configuration	Posting Vehicle	GVW (tons)	0"	1/2"	1"	1-1/2"	2"	2-1/2"	3"	3-1/2"	4"	4-1/2"	5"
Single Unit	2 Axles	SC-SU2	20	1.551	1.538	1.525	1.513	1.500	1.487	1.474	1.462	1.449	1.436	1.424
		SC Representative School Bus	17.525	1.483	1.471	1.459	1.447	1.435	1.423	1.410	1.398	1.386	1.374	1.362
	3 Axles	SC-SHV1A	32.5	0.956	0.948	0.941	0.933	0.926	0.918	0.911	0.903	0.896	0.888	0.881
		SC-SHV1B	35	0.956	0.948	0.941	0.933	0.926	0.918	0.911	0.903	0.896	0.888	0.881
		SC-Type 3 (AASHTO Modified)	25	1.357	1.347	1.336	1.325	1.315	1.304	1.294	1.283	1.272	1.262	1.251
	4 or More Axles	SC-SHV2A	33	1.019	1.011	1.003	0.994	0.986	0.977	0.969	0.961	0.952	0.944	0.936
		SC-SHV2B	40	0.952	0.944	0.936	0.928	0.920	0.913	0.905	0.897	0.889	0.881	0.874
		SU4	27	1.268	1.258	1.247	1.237	1.226	1.216	1.206	1.195	1.185	1.174	1.164
		SU5	31	1.268	1.258	1.247	1.237	1.226	1.216	1.206	1.195	1.185	1.174	1.164
		SU6	34.75	1.268	1.258	1.247	1.237	1.226	1.216	1.206	1.195	1.185	1.174	1.164
		SU7	38.75	1.268	1.258	1.247	1.237	1.226	1.216	1.206	1.195	1.185	1.174	1.164
Combinatio n Unit	5 or More Axles	SC-SHV3A	42.5	1.257	1.247	1.238	1.228	1.218	1.208	1.198	1.188	1.179	1.169	1.159
		SC-SHV3B	45	1.194	1.185	1.176	1.166	1.157	1.148	1.138	1.129	1.120	1.110	1.101
		SC - Type 3S2 (AASHTO Modified)	36.6	1.357	1.347	1.336	1.325	1.315	1.304	1.294	1.283	1.272	1.262	1.251
		Type 3-3	40	1.706	1.693	1.680	1.666	1.653	1.640	1.626	1.613	1.600	1.586	1.573
		Lane Type Legal Load (Neg. Momen	40	-	-	-	-	-	-	-	-	-	-	-
		Lane Type Legal Load (Span > 200')	40	-	-	-	-	-	-	-	-	-	-	-
Emergency Vehicles	2 Axles	EV2	28.75	1.018	1.010	1.002	0.993	0.985	0.977	0.968	0.960	0.952	0.943	0.935
	3 Axles	EV3	43	0.771	0.765	0.759	0.753	0.746	0.740	0.734	0.728	0.722	0.716	0.710

H10-44 15 FT SPAN (1957)

Vehicle Type	Axle Configuration	Posting Vehicle	GVW (tons)	0"	1/2"	1"	1-1/2"	2"	2-1/2"	3"	3-1/2"	4"	4-1/2"	5"
Single Unit	2 Axles	SC-SU2	20	1.559	1.544	1.529	1.514	1.499	1.484	1.469	1.454	1.438	1.423	1.408
		SC Representative School Bus	17.525	1.492	1.477	1.463	1.448	1.434	1.419	1.405	1.390	1.376	1.361	1.347
	3 Axles	SC-SHV1A	32.5	0.940	0.931	0.922	0.914	0.905	0.896	0.888	0.879	0.870	0.862	0.853
		SC-SHV1B	35	0.940	0.931	0.922	0.914	0.905	0.896	0.888	0.879	0.870	0.862	0.853
		SC-Type 3 (AASHTO Modified)	25	1.335	1.322	1.310	1.298	1.285	1.273	1.261	1.248	1.236	1.224	1.212
	4 or More Axles	SC-SHV2A	33	0.983	0.973	0.964	0.954	0.945	0.935	0.925	0.916	0.906	0.897	0.887
		SC-SHV2B	40	0.914	0.905	0.896	0.887	0.878	0.869	0.861	0.852	0.843	0.834	0.825
		SU4	27	1.227	1.215	1.203	1.191	1.179	1.167	1.155	1.144	1.132	1.120	1.108
		SU5	31	1.227	1.215	1.203	1.191	1.179	1.167	1.155	1.144	1.132	1.120	1.108
		SU6	34.75	1.227	1.215	1.203	1.191	1.179	1.167	1.155	1.144	1.132	1.120	1.108
		SU7	38.75	1.227	1.215	1.203	1.191	1.179	1.167	1.155	1.144	1.132	1.120	1.108
Combination Unit	5 or More Axles	SC-SHV3A	42.5	1.236	1.225	1.214	1.202	1.191	1.179	1.168	1.157	1.145	1.134	1.122
		SC-SHV3B	45	1.175	1.164	1.153	1.142	1.131	1.120	1.110	1.099	1.088	1.077	1.066
		SC - Type 3S2 (AASHTO Modified)	36.6	1.335	1.322	1.310	1.298	1.285	1.273	1.261	1.248	1.236	1.224	1.212
		Type 3-3	40	1.678	1.663	1.647	1.632	1.616	1.601	1.585	1.570	1.554	1.539	1.523
		Lane Type Legal Load (Neg. Momen	40	-	-	-	-	-	-	-	-	-	-	-
		Lane Type Legal Load (Span > 200')	40	-	-	-	-	-	-	-	-	-	-	-
Emergency Vehicles	2 Axles	EV2	28.75	1.024	1.014	1.004	0.994	0.984	0.974	0.965	0.955	0.945	0.935	0.925
	3 Axles	EV3	43	0.758	0.751	0.744	0.737	0.730	0.723	0.716	0.709	0.702	0.695	0.688

H15-44 14 FT SPAN (1953)

Vehicle Type	Axle Configuration	Posting Vehicle	GVW (tons)	0"	1/2"	1"	1-1/2"	2"	2-1/2"	3"	3-1/2"	4"	4-1/2"	5"
Single Unit	2 Axles	SC-SU2	20	2.267	2.254	2.242	2.229	2.216	2.204	2.191	2.178	2.165	2.153	2.140
		SC Representative School Bus	17.525	2.169	2.156	2.144	2.132	2.120	2.108	2.096	2.083	2.071	2.059	2.047
	3 Axles	SC-SHV1A	32.5	1.395	1.388	1.380	1.373	1.365	1.358	1.350	1.343	1.335	1.328	1.320
		SC-SHV1B	35	1.395	1.388	1.380	1.373	1.365	1.358	1.350	1.343	1.335	1.328	1.320
		SC-Type 3 (AASHTO Modified)	25	1.982	1.971	1.960	1.950	1.939	1.929	1.918	1.907	1.897	1.886	1.876
	4 or More Axles	SC-SHV2A	33	1.490	1.482	1.473	1.465	1.457	1.448	1.440	1.432	1.423	1.415	1.407
		SC-SHV2B	40	1.391	1.384	1.376	1.368	1.360	1.352	1.345	1.337	1.329	1.321	1.313
		SU4	27	1.854	1.843	1.833	1.823	1.812	1.802	1.791	1.781	1.771	1.760	1.750
		SU5	31	1.854	1.843	1.833	1.823	1.812	1.802	1.791	1.781	1.771	1.760	1.750
		SU6	34.75	1.854	1.843	1.833	1.823	1.812	1.802	1.791	1.781	1.771	1.760	1.750
		SU7	38.75	1.854	1.843	1.833	1.823	1.812	1.802	1.791	1.781	1.771	1.760	1.750
Combination Unit	5 or More Axles	SC-SHV3A	42.5	1.836	1.826	1.816	1.806	1.796	1.787	1.777	1.767	1.757	1.747	1.737
		SC-SHV3B	45	1.744	1.735	1.725	1.716	1.707	1.697	1.688	1.679	1.669	1.660	1.650
		SC - Type 3S2 (AASHTO Modified)	36.6	1.982	1.971	1.960	1.950	1.939	1.929	1.918	1.907	1.897	1.886	1.876
		Type 3-3	40	2.491	2.478	2.465	2.451	2.438	2.425	2.411	2.398	2.385	2.371	2.358
		Lane Type Legal Load (Neg. Momen	40	-	-	-	-	-	-	-	-	-	-	-
		Lane Type Legal Load (Span > 200')	40	-	-	-	-	-	-	-	-	-	-	-
Emergency Vehicles	2 Axles	EV2	28.75	1.489	1.481	1.472	1.464	1.455	1.447	1.439	1.430	1.422	1.414	1.405
	3 Axles	EV3	43	1.125	1.119	1.113	1.107	1.101	1.095	1.089	1.083	1.077	1.071	1.065

H15-44 15 FT SPAN (1957)

Vehicle Type	Axle Configuration	Posting Vehicle	GVW (tons)	0"	1/2"	1"	1-1/2"	2"	2-1/2"	3"	3-1/2"	4"	4-1/2"	5"
Single Unit	2 Axles	SC-SU2	20	2.176	2.161	2.146	2.130	2.115	2.100	2.085	2.069	2.054	2.039	2.024
		SC Representative School Bus	17.525	2.081	2.067	2.052	2.038	2.023	2.009	1.994	1.979	1.965	1.950	1.936
	3 Axles	SC-SHV1A	32.5	1.310	1.301	1.292	1.283	1.274	1.266	1.257	1.248	1.239	1.231	1.222
		SC-SHV1B	35	1.310	1.301	1.292	1.283	1.274	1.266	1.257	1.248	1.239	1.231	1.222
		SC-Type 3 (AASHTO Modified)	25	1.860	1.848	1.835	1.823	1.810	1.798	1.785	1.773	1.761	1.748	1.736
	4 or More Axles	SC-SHV2A	33	1.371	1.362	1.352	1.342	1.333	1.323	1.314	1.304	1.294	1.285	1.275
		SC-SHV2B	40	1.275	1.266	1.257	1.248	1.239	1.230	1.222	1.213	1.204	1.195	1.186
		SU4	27	1.712	1.700	1.688	1.676	1.664	1.652	1.640	1.628	1.616	1.604	1.592
		SU5	31	1.712	1.700	1.688	1.676	1.664	1.652	1.640	1.628	1.616	1.604	1.592
		SU6	34.75	1.712	1.700	1.688	1.676	1.664	1.652	1.640	1.628	1.616	1.604	1.592
		SU7	38.75	1.712	1.700	1.688	1.676	1.664	1.652	1.640	1.628	1.616	1.604	1.592
Combination Unit	5 or More Axles	SC-SHV3A	42.5	1.723	1.712	1.700	1.688	1.677	1.665	1.654	1.642	1.631	1.619	1.608
		SC-SHV3B	45	1.637	1.626	1.615	1.604	1.593	1.582	1.571	1.560	1.549	1.538	1.527
		SC - Type 3S2 (AASHTO Modified)	36.6	1.860	1.848	1.835	1.823	1.810	1.798	1.785	1.773	1.761	1.748	1.736
		Type 3-3	40	2.338	2.323	2.307	2.291	2.276	2.260	2.245	2.229	2.213	2.198	2.182
		Lane Type Legal Load (Neg. Momen	40	-	-	-	-	-	-	-	-	-	-	-
		Lane Type Legal Load (Span > 200')	40	-	-	-	-	-	-	-	-	-	-	-
Emergency Vehicles	2 Axles	EV2	28.75	1.429	1.419	1.409	1.399	1.389	1.379	1.369	1.359	1.349	1.339	1.329
	3 Axles	EV3	43	1.056	1.049	1.042	1.035	1.028	1.021	1.014	1.007	1.000	0.992	0.985

The following are corresponding anticipated H10-44 slab bridge load posting (LFR) based on the suggested distribution factor (0.6 Lane/1 Wheel) with different thickness of wearing surface.

H10-44 14 FT SPAN (1949)

	Asphalt=0"	Asphalt=0.5"-1"	Asphalt=1.5"-2"	Asphalt=2.5"
Posting Signs *R-12-6-48*				
BRIDGE WEIGHT LIMIT - TONS				
SINGLE VEHICLE				
2 OR 3 AXLES	31 T	30 T	30 T	29 T
4 OR MORE AXLES	38 T	37 T	32 T	32 T
COMBINATIONS	45 T	45 T	45 T	45 T
EMERGENCY VEHICLE WEIGHT LIMITS				
SINGLE AXLE	16 T	16 T	16 T	16 T
TANDEM	23 T	23 T	23 T	22 T
GROSS	33 T	32 T	28 T	28 T

	Asphalt=3"-3.5"	Asphalt=4"	Asphalt=4.5"	Asphalt=5"
Posting Signs *R-12-6-48*				
BRIDGE WEIGHT LIMIT - TONS				
SINGLE VEHICLE				
2 OR 3 AXLES	29 T	29 T	28 T	28 T
4 OR MORE AXLES	31 T	31 T	31 T	30 T
COMBINATIONS	45 T	45 T	45 T	45 T
EMERGENCY VEHICLE WEIGHT LIMITS				
SINGLE AXLE	16 T	15 T	15 T	15 T
TANDEM	22 T	22 T	22 T	22 T
GROSS	27 T	27 T	27 T	26 T

H10-44 15 FT SPAN (1957)

Asphalt=0”-0.5”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	30	T
4 OR MORE AXLES	32	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	23	T
GROSS	32	T

Asphalt=1”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	29	T
4 OR MORE AXLES	31	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	23	T
GROSS	31	T

Asphalt=1.5”-2”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	29	T
4 OR MORE AXLES	31	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	22	T
GROSS	28	T

Asphalt=2.5”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	29	T
4 OR MORE AXLES	30	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	22	T
GROSS	28	T

Asphalt=3”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	28	T
4 OR MORE AXLES	30	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	22	T
GROSS	27	T

Asphalt=3.5”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	28	T
4 OR MORE AXLES	30	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	15	T
TANDEM	21	T
GROSS	27	T

Asphalt=4”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	28	T
4 OR MORE AXLES	29	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	15	T
TANDEM	21	T
GROSS	27	T

Asphalt=4.5”

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES	28	T
4 OR MORE AXLES	29	T
COMBINATIONS	45	T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	15	T
TANDEM	21	T
GROSS	26	T

H15-44 slab 14FT span bridges (1953) typically are not posted. H15-44 slab 15FT span bridges (1957) need posting only when the thickness of wearing surface is larger than 4 in. Below are corresponding anticipated H15-44 15FT slab bridge load posting (LFR) based on the suggested distribution factor (0.6 Lane/1 Wheel) with 4.5 in and 5 in thickness of wearing surface.

H15-44 15 FT SPAN (1957)

(Asphalt = 4.5”-5”)

Posting Signs

R-12-6-48

BRIDGE WEIGHT LIMIT - TONS		
SINGLE VEHICLE		
2 OR 3 AXLES		T
4 OR MORE AXLES		T
COMBINATIONS		T

EMERGENCY VEHICLE WEIGHT LIMITS		
SINGLE AXLE	16	T
TANDEM	30	T
GROSS	42	T

Appendix D: Theoretical Maximum Strain Calculation at Mid-span

Material Properties:

Concrete Compressive Strength (f_c') (ksi.)	Density (kcf.)	Elastic Modulus (E_c, AASHTO LRFD 5.4.2.4-1) (ksi.)	Modulus of Rupture (f_r, AASHTO LRFD 5.4.2.6) (ksi.)
3.75 (f_c = 1500 psi)	0.145	3902.54	0.46
4 (f_c =1600 psi)	0.145	3986.55	0.48
7.47	0.145	4899.06	0.66
4.9	0.145	4262.67	0.53
6.43	0.145	4662.59	0.61
5.01	0.145	4294.02	0.54
4.47	0.145	4135.41	0.51

Transformed Section Properties:

Bridge ID	Slab Unit Width (ft)	Slab Thickness (in)	Gross Moment of Inertia (in^4)	Tested Concrete Compressive Strength (ksi)	Reinforcing Detail	Transformed Moment of Inertia (in^4)	Transformed Section Modulus (in^3)
02006	5	8.25	2807.58	7.47	10 #7 @ 6"	2999.52	754.16
02357	5	9.5	4286.88	2.74[1]	12 #7 @ 5.25"	4729.01	1048.25
02736	5.5	8.25	3088.34	4.9	11 #7 @ 6"	3334.91	843.75
05957	5	8.25	2807.58	6.43	10 #7 @ 6"	3010.54	758.55
06866	5.5	9.25	4352.99	5.01	13 #7 @ 5"	4761.24	1079.40
07337	5.5	9.25	4352.99	4.47	13 #7 @ 5"	4778.31	1085.47

1. Design concrete strength (4ksi) was used to calculate the theoretical strain for 02357 due to the unreliable core test result

Load Testing Vehicle Wheel Load (Quick Bridge Input, div=100):

BR 02006, 07337

Axle No	X	W
1	0.0	8020.0
2	15.20	10340.0
3	19.62	10040.0

BR 02357, 02736, 05957, 06866

Axle No	X	W
1	0.0	2810.0
2	14.1	5970.0

Theoretical Strain Value:

Asset ID	Test DF (Wheel)	Max. moment at mid-span due to tested truck (kip-ft.)	Max. moment at mid-span for controlling slab (kip-ft.)	Theoretical Strain ($x10^{-6}$)
02006	1	45.31	45.31	147.16
02357	1	19.67	19.67	56.48
02736	0.86	21.19	18.22	60.80
05957	1	19.67	19.67	66.74
06866	0.94	21.19	19.81	51.29
07337	1	50.51	50.51	135.03

BR02006 Deflection Calculation:

Theoretical Deflection at Mid-Span due to Tested Truck (BR02006)

$SPAN := 14ft$ $\quad$ $bearingoffset := 4.5in$

$L := SPAN - bearingoffset \cdot 2 = 13.25 \cdot ft$ $\quad$ CL-CL of bearing

Material Properties

$f'_c := 7.47ksi$

$\gamma_c := 0.145\ kcf$

$$E_c := 120000 \cdot \gamma_c^{\,2} \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4.899 \times 10^3 \cdot ksi$$ *AASHTO LRFD 8th 5.4.2.4-1*

$E_s := 29000ksi$

Transformed Section Properties

$t := 8.25in \quad b := 5ft$

$$n := \frac{E_s}{E_c} = 5.919$$

$A_s := 10 \cdot 0.6in^2 = 6 \cdot in^2$ $\quad$ 10 #7 @ 6"

$d := t - 1.5in = 6.75 \cdot in$

$$y_t := \frac{\frac{b \cdot t^2}{2} + A_s \cdot d \cdot (n-1)}{b \cdot t + A_s \cdot (n-1)} = 4.273 \cdot in$$

$y_b := t - y_t = 3.977 \cdot in$

$$I_{trans} := \frac{1}{12} \cdot b \cdot t^3 + b \cdot t \cdot \left(\frac{t}{2} - y_t\right)^2 + (n-1) \cdot A_s \cdot \left(d - y_t\right)^2 = 3 \times 10^3 \cdot in^4$$

$$S_{b.trans} := \frac{I_{trans}}{y_b} = 754.165 \cdot in^3$$

Wheel Load and Axle Spacings (left side):

$P_1 := 8.02kip$ $\quad$ $P_2 := 10.34kip$ $\quad$ $P_3 := 10.04kip$

$S_1 := 15.2ft$ $\quad$ $S_2 := 4.42ft$

Maximum Moment at Mid-Span and Corresponding Location of Wheels:

P.2 is 1.121 ft from mid-span (Left), P.3 is 3.299 ft from mid-span (Right)

$$x_1 := \frac{1}{2} \frac{P_2 \cdot S_2}{P_2 + P_3} = 1.121 \cdot ft$$

$x_2 := S_2 - x_1 = 3.299 \cdot ft$

$$M_{truck} := \frac{P_2}{2} \cdot \left(\frac{L}{2} - x_1\right) + \frac{P_3}{2} \cdot \left(\frac{L}{2} - x_2\right) = 45.152 \cdot kip \cdot ft$$

Deflection at Mid-span:

Note: Since no crack was observed during load testing, and residual strain was negligible, gross transformed section properties were used to calculate deflection (same assumption used to calculate theoretical strain)

$$\Delta_{mid}(x,P,a,b) := \begin{cases} \dfrac{P \cdot a \cdot (L-x)}{6 \cdot E_c \cdot I_{trans} \cdot L} \cdot \left(2 \cdot L \cdot x - x^2 - a^2\right) & \text{if } x > a \\ \dfrac{P \cdot b \cdot x}{6 \cdot E_c \cdot I_{trans} \cdot L} \cdot \left(L^2 - b^2 - x^2\right) & \text{if } x < a \end{cases}$$

Deflection at Mid-Span Due to P2:

$a_{p2} := \dfrac{L}{2} - x_1 = 5.504 \cdot ft$ $\qquad$ $b_{p2} := L - a_{p2} = 7.746 \cdot ft$

$$\Delta_{mid}\left(\frac{L}{2}, P_2, a_{p2}, b_{p2}\right) = 0.057 \cdot in$$

Deflection at Mid-Span Due to P3:

$a_{p3} := \dfrac{L}{2} + x_2 = 9.924 \cdot ft$ $\qquad$ $b_{p3} := L - a_{p3} = 3.326 \cdot ft$

$$\Delta_{mid}\left(\frac{L}{2}, P_3, a_{p3}, b_{p3}\right) = 0.039 \cdot in$$

$$\Delta_{mid} := \Delta_{mid}\left(\frac{L}{2}, P_2, a_{p2}, b_{p2}\right) + \Delta_{mid}\left(\frac{L}{2}, P_3, a_{p3}, b_{p3}\right) = 0.096 \cdot in$$

BR07337 Deflection Calculation

Theoretical Deflection at Mid-Span Due to Tested Truck (BR 07337)

$SPAN := 15ft \qquad bearingoffset := 4.5in$

$L := SPAN - bearingoffset \cdot 2 = 14.25 \cdot ft$ CL-CL of bearing

Material Properties

$f'_c := 4.47ksi$

$\gamma_c := 0.145 \;\; kcf$

$E_c := 120000 \cdot \gamma_c^{\,2} \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4.135 \times 10^3 \cdot ksi$ *AASHTO LRFD 8th 5.4.2.4-1*

$E_s := 29000ksi$

Transformed Section Properties

$t := 9.25in \quad b := 5.5ft$

$$n := \frac{E_s}{E_c} = 7.013$$

$A_s := 13 \cdot 0.6in^2 = 7.8 \cdot in^2$ 13 #7 @ 5"

$d := t - 1.5in = 7.75 \cdot in$

$$y_t := \frac{\frac{b \cdot t^2}{2} + A_s \cdot d \cdot (n-1)}{b \cdot t + A_s \cdot (n-1)} = 4.848 \cdot in$$

$y_b := t - y_t = 4.402 \cdot in$

$$I_{trans} := \frac{1}{12} \cdot b \cdot t^3 + b \cdot t \cdot \left(\frac{t}{2} - y_t\right)^2 + (n-1) \cdot A_s \cdot \left(d - y_t\right)^2 = 4.778 \times 10^3 \cdot in^4$$

$$S_{b.trans} := \frac{I_{trans}}{y_b} = 1.085 \times 10^3 \cdot in^3$$

Wheel Load and Axle Spacings (left side):

$P_1 := 8.02kip \qquad P_2 := 10.34kip \qquad P_3 := 10.04kip$

$S_1 := 15.2ft \qquad S_2 := 4.42ft$

Maximum Moment at Mid-Span and Corresponding Location of Wheels:

P.2 is 1.121 ft from mid-span (Left), P.3 is 3.299 ft from mid-span (Right)

$$x_1 := \frac{1}{2} \frac{P_2 \cdot S_2}{P_2 + P_3} = 1.121 \cdot ft$$

$x_2 := S_2 - x_1 = 3.299 \cdot ft$

$$M_{truck} := \frac{P_2}{2} \cdot \left(\frac{L}{2} - x_1\right) + \frac{P_3}{2} \cdot \left(\frac{L}{2} - x_2\right) = 50.247 \cdot kip \cdot ft$$

Deflection at Mid-span:

Note: Since no crack was observed during load testing, and residual strain was negligible, gross transformed section properties were used to calculate deflection (same assumption used to calculate theoretical strain)

$$\Delta_{mid}(x,P,a,b) := \begin{cases} \dfrac{P \cdot a \cdot (L-x)}{6 \cdot E_c \cdot I_{trans} \cdot L} \cdot \left(2 \cdot L \cdot x - x^2 - a^2\right) & \text{if } x > a \\ \dfrac{P \cdot b \cdot x}{6 \cdot E_c \cdot I_{trans} \cdot L} \cdot \left(L^2 - b^2 - x^2\right) & \text{if } x < a \end{cases}$$

Deflection at Mid-Span Due to P2:

$a_{p2} := \dfrac{L}{2} - x_1 = 6.004 \cdot ft \qquad b_{p2} := L - a_{p2} = 8.246 \cdot ft$

$$\Delta_{mid}\left(\frac{L}{2}, P_2, a_{p2}, b_{p2}\right) = 0.053 \cdot in$$

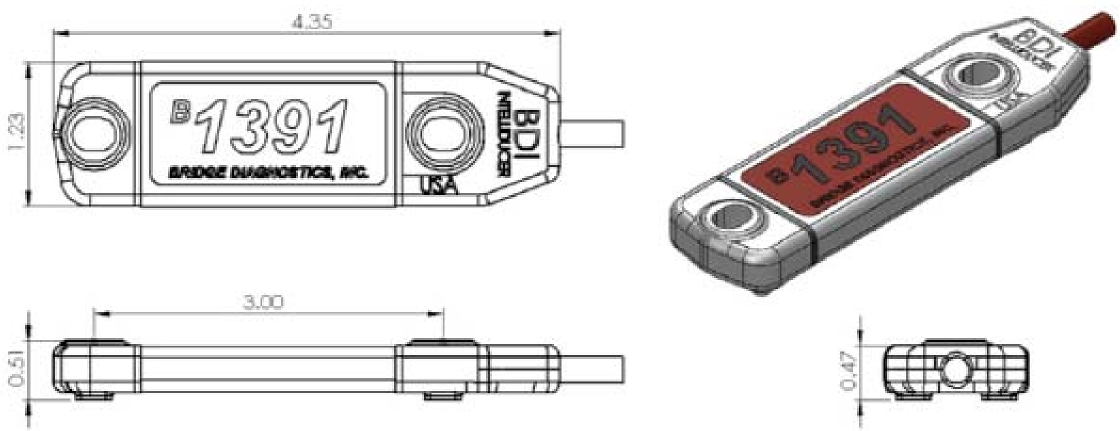
Deflection at Mid-Span Due to P3:

$a_{p3} := \dfrac{L}{2} + x_2 = 10.424 \cdot ft \qquad b_{p3} := L - a_{p3} = 3.826 \cdot ft$

$$\Delta_{mid}\left(\frac{L}{2}, P_3, a_{p3}, b_{p3}\right) = 0.039 \cdot in$$

$$\Delta_{mid} := \Delta_{mid}\left(\frac{L}{2}, P_2, a_{p2}, b_{p2}\right) + \Delta_{mid}\left(\frac{L}{2}, P_3, a_{p3}, b_{p3}\right) = 0.091 \cdot in$$

Appendix E: General Specifications of ST350 Strain Transducers (BDI)



Model	ST350
Range (Resistance)	350 Ω
Excitation Voltage	+1.0 to +10.0 Vdc (output is ratiometric)
Power Rating **Max:** **Typical:** **Intelliducer:**	 300 mW 72 mW @ +5.0 Vdc 13 mW @ +5.0 Vdc*
Circuit	Full Wheatstone bridge with four active 350Ω foil gages
Strain Range	±4000με (Calibrated to ±2000με)
Force required for 1000με	~17lbs. (~76N)
Typical Sensitivity	~500 με/mV/V (individually calibrated to N.I.S.T. standards)
Accuracy	< ±1%
Effective Gage Length	3.0 in (76.2 mm) [Extensions available for use with R/C structures]
Cable Length	IC-02-187 (0.187 in diameter, 22awg, 2 pair, shielded with drain wire, red PVC jacket) or IC-02-250 (0.250 in diameter, 22awg, 2 pair, shielded with drain wire, blue PVC jacket)
Housing	6061-Aluminum
Weather Proofing	IP67 Rated (waterproof to 70 meters available)
Operating Temperature	-58°F to +185°F (-50°C to +85°C)
Weight	3 oz. (85 grams)
Mounting	BDI mounting Tab and adhesive, mechanical connection

Appendix F: Bucket Truck Details used in Diagnostic Load Tests

EVERYTHING YOU NEED TO GET THE JOB DONE 1-800-252-0043

NESCO RENTALS

Equipment Rentals, Parts and Service
www.nescorentals.com

A0119728 - 45' 2014 VERSALIFT SST-40; 2014 RAM RAM 5500 4X4
Warehouse: O11 - OR - TEREX - PHOENIX AZ

CHASSIS SPECIFICATIONS

VIN:	3C7WRNBL4EG208926		
Engine Make / HP:	6.7L CUMMINS / 305		
Transmission:	AISIN 6-SPEED		
Brakes:	HYDRAULIC		
Hitch:	COMBO PINTLE/BALL		
Winch Location:	N/A	**Capacity:**	N/A
Drive Type: ALL WHEEL DRIVE	**Fuel Type:** DIESEL	**Rail Gear:**	N
Rail Gear Type:			

EQUIPMENT SPECIFICATIONS

Serial Number:	EH140070		
Working Height:	45'		
Bucket:	24X30X42 1-MAN	**Capacity:**	350 LBS
Certified Kv:	46KV CAT C		
Rotation:	CONTINUOUS		
Outrigger(s):	TORSION BAR		
Mount:	BEHIND CAB		
Tool Circuit:	HYD AT PLATFORM		
Controls:	SINGLE STICK		
Body:	UTILITY LINE		
Winch/Jib:	N/A	**Capacity:**	N/A
Additional:	N/A		
Over Center: N	**Telescoping:** Y	**Upper Controls:**	Y
Rotator: Y	**Material Handler:** N	**Insulated Boom:**	Y

DIMENSIONS / MILES / HOURS / WEIGHTS

Height: 10'2	**Length:** 23'5	**Width:**	7'9
GVWR: 19,500	**Unladen:** 14,840LBS	**Tow Capacity:**	
Axle Capacity:	7,000LBS front / 12,500LBS rear		
Wheelbase:	WB / CA	**Tire Size:**	225/70R19.5G
Tread Front/Rear:	100%	**Tread Updated:**	6/2014
PTO Hours:	973		
Engine Hours:	5,483	**Hours Updated:**	06/03/2019
Miles:	79,919	**Miles Updated:**	06/03/2019

Additional Specs:

45' 14 VERS SST40-EIH, 14 RAM 5500 4X4 A0119728 EQUIP DESC: EMERGENCY STOP CONTROL, FULL PRESSURE TURRET MOUNTED LOWER CONTROLS, OPEN CENTER HYDRAULIC SYSTEM, FIBERGLASS COMPENSATION, 12 INSULATION GAP, FIBERGLASS INNER BOOM, ELECTROGARD, 15-GAL HYD OIL RESERVOIR, BUCKET LINER AND COVER, HYD LEVELING W/UPPER CONTROL, NON-LUBE BEARINGS AT ALL PIVOT POINTS, MULTI-LINK HOSE CARRIER SYSTEM, BOOM CRADLE AND RATCHET STRAP, ENGINE START/STOP, TWO SPEED ENGINE THROTTLE CONTROL FOR HYD TOOL POWER, EMERGENCY 12V DC HYD SYSTEM, INVERTOR, LADDER RACK ADDED TRUCK DESC: HEAVY DUTY FRONT SUSPENSION GROUP, 4.44 LIMITED SLIP AXLE, AC, COLD WEATHER GROUP INCLUDING ENGINE BLOCK HEATER, RADIO: UCONNECT 3.0 AM/FM, HD VINYL 40/20/40 SLIT BENCH SEAT, PARKVIEW REAR BACKUP CAMERA, AMBULANCE PREP PKG W/220 AMP ALTERNATOR, TRANSFER CASE SKID PLATE SHIELD

Appendix G: Sample Distribution Factor Calculation from Load Test Data

			G1	G2		G3		G4		G5		G6	
			Sensor 1	Sensor 1	Sensor 2	Sensor 1	Sensor 2	Sensor 1	Sensor 2	Sensor 1	Sensor 2	Sensor 1	
		Time (s)	B3459_18A	B3460_18A	B4501_18A	B3313_18A	B4492_18A	B4488_18A	B4499_18A	B3457_18A	B4496_18A	B4498_18A	Sum of Strains per Run
Test 1	Run 1	48.7	40.1790543	324.428284	382.8127441	331.444489	332.513733	60.7360535	64.0401306	14.00516701	11.58530045	2.953277588	1564.698233
	Run 1	49	33.5657043	412.436554	370.9671021	357.383148	336.644165	54.9301147	58.5456238	13.24247646	10.83970451	2.627067566	1651.181661
	Run 2	28.6	65.7444153	304.729584	360.1697083	305.784424	329.243896	55.8086853	59.0551605	11.20848083	9.812324524	0.766563416	1502.323242
	Run 2	29	55.7485809	398.432495	337.5239258	333.900757	329.972565	48.9893799	52.7062836	10.41202259	8.931638718	0.444084167	1577.061732
	Run 3	50.5	62.8281288	307.896271	362.1279602	309.580017	338.615479	56.5863037	59.6143036	11.08187008	9.426105499	0.104255676	1517.860694
	Run 3	50.8	57.0821419	387.209778	349.8163757	332.901001	345.821289	52.9072266	56.2743683	10.58000755	8.897434235	-0.108650208	1601.380972
	AVG		**52.5246709**	**355.855494**	**360.569636**	**328.498973**	**335.468521**	**54.9929606**	**58.3726451**	**11.75500409**	**9.915417989**	**1.131099701**	**1569.084422**

Test 1	Run 1	DF	0.0257	0.2073	0.2447	0.2118	0.2125	0.0388	0.0409	0.0090	0.0074	0.0019
	Run 1	DF	0.0203	0.2498	0.2247	0.2164	0.2039	0.0333	0.0355	0.0080	0.0066	0.0016
	Run 2	DF	0.0438	0.2028	0.2397	0.2035	0.2192	0.0371	0.0393	0.0075	0.0065	0.0005
	Run 2	DF	0.0353	0.2526	0.2140	0.2117	0.2092	0.0311	0.0334	0.0066	0.0057	0.0003
	Run 3	DF	0.0414	0.2028	0.2386	0.2040	0.2231	0.0373	0.0393	0.0073	0.0062	0.0001
	Run 3	DF	0.0356	0.2418	0.2184	0.2079	0.2160	0.0330	0.0351	0.0066	0.0056	-0.0001
	AVG		**0.0335**	**0.2268**	**0.2298**	**0.2094**	**0.2138**	**0.0350**	**0.0372**	**0.0075**	**0.0063**	**0.0007**

Test 1	Run 1	DF	0.03	0.45	0.42	0.08	0.02	0.00
	Run 1	DF	0.02	0.47	0.42	0.07	0.01	0.00
	Run 2	DF	0.04	0.44	0.42	0.08	0.01	0.00
	Run 2	DF	0.04	0.47	0.42	0.06	0.01	0.00
	Run 3	DF	0.04	0.44	0.43	0.08	0.01	0.00
	Run 3	DF	0.04	0.46	0.42	0.07	0.01	0.00
	AVG		**0.03**	**0.46**	**0.42**	**0.07**	**0.01**	**0.00**

Appendix H: Concrete Core Test Report

BOYLE CONSULTING ENGINEERS

Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

Test Report - Compressive Strength of Drilled Concrete Cores

Project Name:	SCDOT	**Location Sampled:**	Core #2006
BOYLE Project #:	20-016	**Owner:**	
Client:	WSP	**Gen. Contractor:**	
Conc. Contractor:		**Orig. Conc. Date:**	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

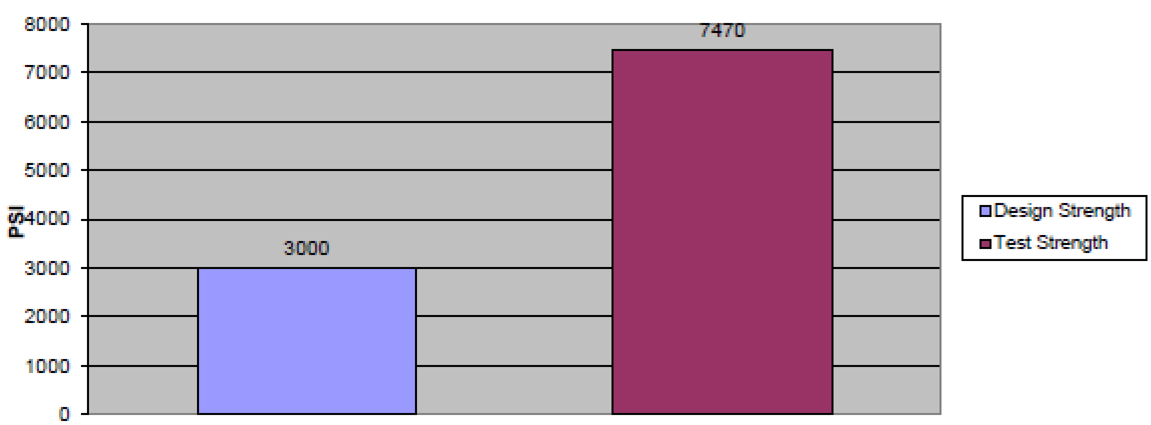
Performed By:	WSP	**Coring Date:**	3/20/20
Nominal Bit Size (in):	2.75	**Core Prep:**	Sawcut End/Bonded Caps
Nom. Max Agg. Size (in):	-	**Min Allwd Diam (in):**	
Moisture Condition:		**Min Pref. Diam (in):**	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	8.23	8.20	8.22	2.79	2.79	2.79	2.94	
Prepared Core Meas. (in.)	5.66	5.67	5.66	2.79	2.79	2.79	2.03	1.000

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	**Load Test Date:**	3/23/20
		Concrete Age - days:	43913
		Core Age - days:	3

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
2006	3/23/20	2.79	6.11	3000	45,664	7470	7470	3	Pass



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

Boyle Consulting Engineers, PLLC

BOYLE CONSULTING ENGINEERS

Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

3/

Project Name:	SCDOT	**Location Sampled:**	Core #2357
BOYLE Project #:	20-016	**Owner:**	
Client:	WSP	**Gen. Contractor:**	
Conc. Contractor:		**Orig. Conc. Date:**	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

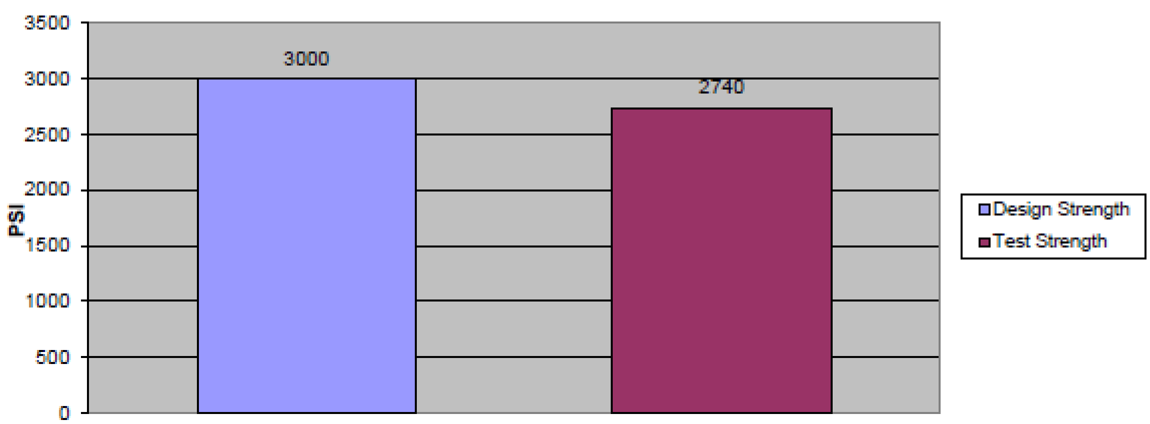
Performed By:	WSP	**Coring Date:**	3/20/20
Nominal Bit Size (in):	2.75	**Core Prep:**	Sawcut End/Unbonded Caps
Nom. Max Agg. Size (in):	-	**Min Allwd Diam (in):**	
Moisture Condition:		**Min Pref. Diam (in):**	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	9.44	9.46	9.45	2.79	2.79	2.79	3.39	
Prepared Core Meas. (in.)	5.66	5.62	5.64	2.79	2.79	2.79	2.02	1.000

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	**Load Test Date:**	3/20/20
		Concrete Age - days:	43910
		Core Age - days:	0

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
2357	3/20/20	2.79	6.11	3000	16,733	2740	2740	3	Fail



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

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Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

3/

Project Name:	SCDOT	Location Sampled:	Core #2736
BOYLE Project #:	20-016	Owner:	
Client:	WSP	Gen. Contractor:	
Conc. Contractor:		Orig. Conc. Date:	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

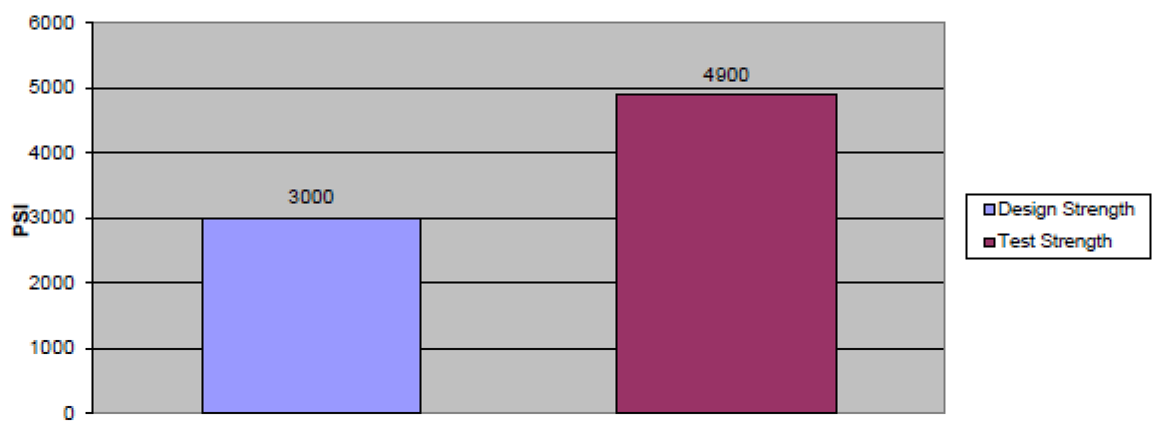
Performed By:	WSP	Coring Date:	4/3/20
Nominal Bit Size (in):	2.75	Core Prep:	Sawcut Ends/Unbonded Caps
Nom. Max Agg. Size (in):	-	Min Allwd Diam (in):	
Moisture Condition:		Min Pref. Diam (in):	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	8.40	8.38	8.39	2.77	2.78	2.77	3.02	
Prepared Core Meas. (in.)	5.69	5.68	5.69	2.77	2.78	2.77	2.05	1.000

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	Load Test Date:	4/6/20
		Concrete Age - days:	
		Core Age - days:	3

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
2736	4/6/20	2.77	6.05	3000	29,627	4900	4900	3	Pass



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

Asphalt Surface: 1 Inch

Boyle Consulting Engineers, PLLC

BOYLE CONSULTING ENGINEERS

Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

Test Report - Compressive Strength of Drilled Concrete Cores

Project Name:	SCDOT	**Location Sampled:**	Core #5957
BOYLE Project #:	20-016	**Owner:**	
Client:	WSP	**Gen. Contractor:**	
Conc. Contractor:		**Orig. Conc. Date:**	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

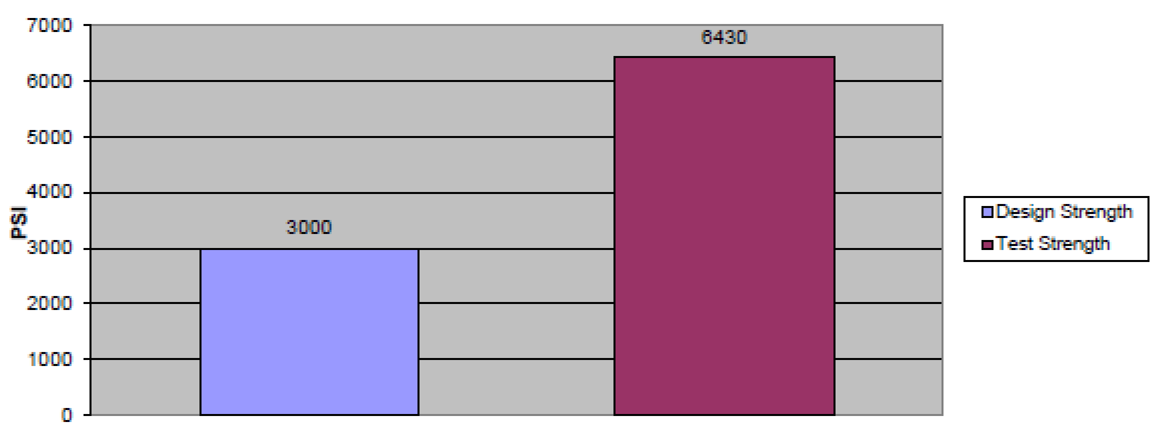
Performed By:	WSP	**Coring Date:**	3/20/20
Nominal Bit Size (in):	2.75	**Core Prep:**	Sawcut End/Bonded Caps
Nom. Max Agg. Size (in):	-	**Min Allwd Diam (in):**	
Moisture Condition:		**Min Pref. Diam (in):**	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	5.79	5.77	5.78	2.80	2.78	2.79	2.07	
Prepared Core Meas. (in.)	5.30	5.30	5.30	2.80	2.78	2.79	1.90	0.992

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	**Load Test Date:**	3/23/20
		Concrete Age - days:	43913
		Core Age - days:	3

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
5957	3/23/20	2.79	6.10	3000	39,528	6480	6430	3	Pass



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

Robert [signature]

Boyle Consulting Engineers, PLLC

BOYLE CONSULTING ENGINEERS

Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

3/

Project Name:	SCDOT	**Location Sampled:**	Core #BR6866
BOYLE Project #:	20-016	**Owner:**	
Client:	WSP	**Gen. Contractor:**	
Conc. Contractor:		**Orig. Conc. Date:**	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

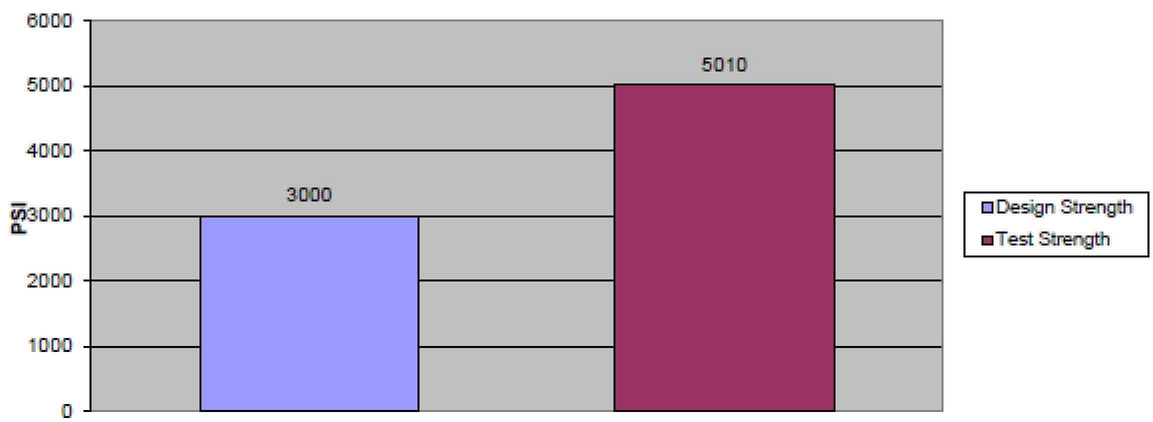
Performed By:	WSP	**Coring Date:**	4/3/20
Nominal Bit Size (in):	2.75	**Core Prep:**	Sawcut Ends/Unbonded Caps
Nom. Max Agg. Size (in):	-	**Min Allwd Diam (in):**	
Moisture Condition:		**Min Pref. Diam (in):**	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	9.74	9.74	9.74	2.77	2.79	2.78	3.51	
Prepared Core Meas. (in.)	5.83	5.85	5.84	2.77	2.79	2.78	2.10	1.000

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	**Load Test Date:**	4/6/20
		Concrete Age - days:	
		Core Age - days:	3

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
BR6866	4/6/20	2.78	6.06	3000	30,362	5010	5010	3	Pass



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

Rebar: Diameter 0.8775" (#7) Measured 8.25 Inches From Top of Core

Robert [signature]

Boyle Consulting Engineers, PLLC

BOYLE CONSULTING ENGINEERS

Branch Laboratory:
4340 Taggart Creek Rd., Ste G
Charlotte, NC 28208
Phone (704) 676-0778

Test Report - Compressive Strength of Drilled Concrete Cores

Project Name:	SCDOT	Location Sampled:	Core #7337
BOYLE Project #:	20-016	Owner:	
Client:	WSP	Gen. Contractor:	
Conc. Contractor:		Orig. Conc. Date:	

CORE SAMPLE COLLECTION & PREPARATION (Re. ASTM - C42)

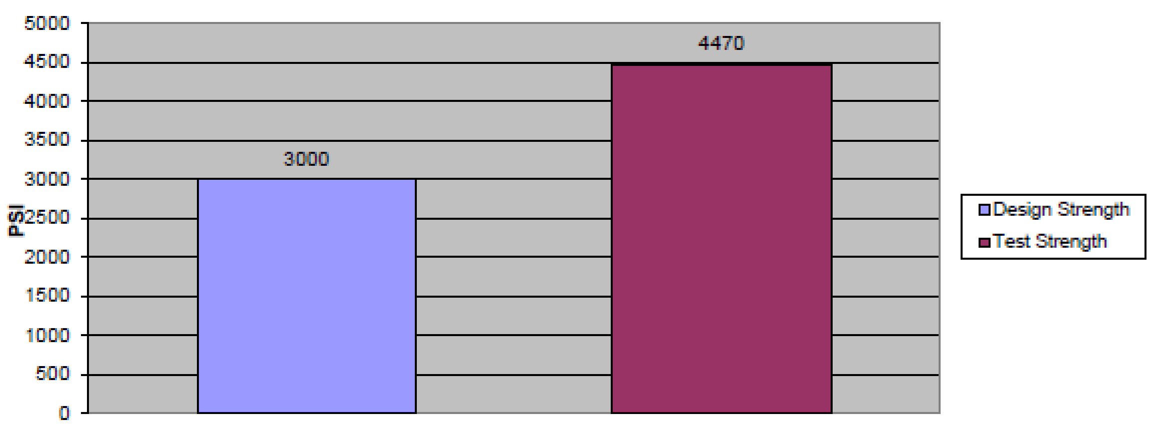
Performed By:	WSP	Coring Date:	3/20/20
Nominal Bit Size (in):	2.75	Core Prep:	Sawcut End/Bonded Caps
Nom. Max Agg. Size (in):	-	Min Allwd Diam (in):	
Moisture Condition:		Min Pref. Diam (in):	

	Lengths Measured to Nearest 0.1"			Diam. Measured to Nearest 0.01"				
	L1	L2	L_avg	D1	D2	D_avg	L/D	Correction Factor - CF
In-Place Core Meas. (in.)	9.87	9.87	9.87	2.79	2.79	2.79	3.54	
Prepared Core Meas. (in.)	5.63	5.64	5.64	2.79	2.79	2.79	2.02	1.000

CORE LOAD TEST (Re. ASTM - C39)

Performed By:	Boyle	Load Test Date:	3/23/20
		Concrete Age - days:	43913
		Core Age - days:	3

Sample No.	Test Date	Diam - in	Area - sq.in.	Design Strength - psi	Max Load - Pounds	Measured Strength - psi	Corrected Strength - psi	Fracture Type	Pass or Fail
7337	3/23/20	2.79	6.11	3000	27,300	4470	4470	3	Pass



Remarks:
Per ASTM C42, Compressive Strength is reported to the nearest 10 psi when the diameter is measured to the nearest 0.01 inches.

Boyle Consulting Engineers, PLLC

Appendix I: Standard Plan (Tested Bridges)

H10-44 14 FT. SPAN 24 FT. ROADWAY (1949): 02006, 05957

H10-44 15 FT. SPAN 26 FT. ROADWAY (1957): 02736

H15-44 14 FT. SPAN 24 FT. ROADWAY (1953): 02357

H15-44 15 FT. SPAN 31.5 FT. ROADWAY (1957): 06866, 07337