SCDOT

LOAD TESTING OF MULTIPLE WIDE LEG CHANNEL BRIDGES

STATEWIDE – SOUTH CAROLINA

8'-6"

LOAD TESTING OF MULTIPLE WIDE LEG CHANNEL BRIDGES

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1 BACKGROUND

1.1 INTRODUCTION

There are about three hundred and seventy prestressed wide-leg channel beam bridges in the SCDOT's bridge inventory. Most of these bridges were constructed in the 1960s through the 1980s with a span length of 30 ft. The channel bridges typically consist of 9 to 15 adjacent precast girders depending on the width of bridge (Figure 1).

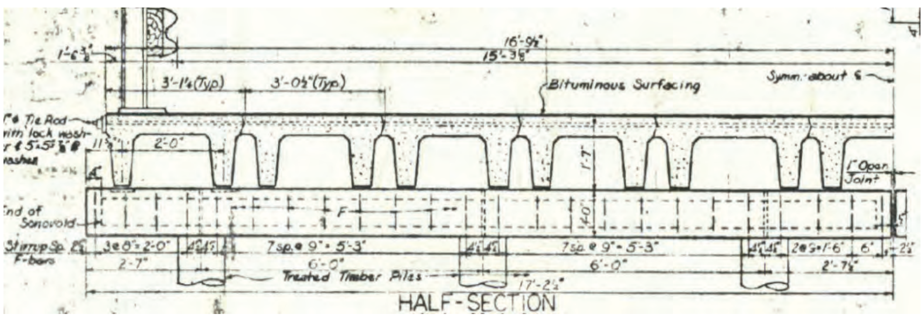


Figure 1: Typical Half Cross Section of Wide-Leg Channel Bridges (SCDOT)

As stated in the skinny-leg channel bridge diagnostic load test report performed by WSP in 2020, the amount of reflective cracking is the key indicator of potential load sharing between the channels. The live load distribution factors (LLDFs) for the skinny-leg channels were categorized into three groups depending on the reflective cracking conditions:

1) For no reflective cracking, the tested LLDF was in line with the AASHTO specified values;
2) for moderate reflective cracking, the LLDF was 0.35 lane;
3) and for severe reflective cracking, the LLDF was 0.5 lane.

Like the skinny-leg channel bridges, for wide-leg channel bridges with heavy reflective cracking the precast channel beams are expected to behave independently under live load with an approximate LLDF of 0.5 lane. This could lead to load postings on the in-service wide-leg channel bridges. In order to avoid or improve the posting of wide-leg channel bridges with severe reflective cracking, the load rating factors can be modified by a K-factor resulting from the comparison of the theoretical and measured test behavior in accordance with AASHTO MBE, 3rd edition, Section 8.8.2.3.

In this report, Chapter 1 to Chapter 3 describe the planning and execution of the diagnostic load testing of the selected wide-leg channel bridges. Then, the results of the diagnostic testing, including the LLDFs and K-factors, are summarized and discussed in Chapter 4. Lastly, load rating recommendations are provided in Chapter 5 of this report.

1.2 OBJECTIVES AND GOALS

The diagnostic load testing performed on the wide-leg channel bridges have two main objectives:

1) Confirm that the visual guide utilized to determine the live load distribution factors for skinny-leg channel bridges are applicable to the wide-leg channel bridges;
2) Determine an appropriate K-factor to avoid/improve the load posting of wide-leg channel bridges.

2 EXPERIMENTAL PLAN

2.1 TESTING THEORY

2.1.1 LIVE LOAD DISTRIBUTION FACTORS

To capture the live load distribution among channel beams, strain gauges were installed on each channel leg of tested bridges. A standard two-axle bucket truck with known weight and axle configuration was driven over the tested bridges at a crawl speed. Details of conducting the diagnostic tests are described in Section 3.3. The live load distribution factor of a channel beam was calculated using the following equation:

$$\mathrm{DFM_i} = \frac{\varepsilon_\mathrm{i}}{\sum_{\mathrm{j}=1}^{\mathrm{n}} \varepsilon_\mathrm{j}}$$

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

2.1.2 K-FACTORS

In accordance with the AASHTO MBE, 3rd edition, Section 8.8.2.3, to obtain an enhanced rating factor based on the diagnostic load test results, the existing load rating factor can be adjusted with a K-factor. The following equation is used to modify the calculated load rating using the load test results:

$$\mathrm{RF_T} = \mathrm{RF_C} \times \mathrm{K}$$

RF_T is the load rating factor after applying the K-factor and RF_C is the rating factor without incorporating the K-factor. The K-factor can be determined through a comparison of measured test behavior with the analytical results using the following equation:

$$\mathrm{K} = 1 + \mathrm{K_a K_b}$$

In this equation K_a accounts for the benefit of the load test and $\mathrm{K_b}$ accounts for the understanding of the load test results when compared with those predicted by theory. $\mathrm{K_a}$ is calculated as follows:

$$\mathrm{K_a} = \frac{\varepsilon_\mathrm{C}}{\varepsilon_\mathrm{T}} - 1$$

The maximum measured response in a member is represented as ε_T, and ε_C is the calculated maximum analytical response at the same location. $\mathrm{K_b}$ can be determined from AASHTO MBE, 3rd edition, Table 8.8.2.3.1-1. See Appendix B for sample K-factor calculations.

2.2 BRIDGE SELECTION

There are about three hundred and seventy-three (373) wide-leg channel bridges in SCDOT's bridge inventory, which were categorized into two groups in this project based on the year of construction due to a change in the standard details: 1) Pre-1976, and 2) Post-1976. One hundred and sixty-one (161) wide-leg channel bridges were constructed at or before 1976.

Two hundred and twelve (212) wide-leg channel beam bridges were constructed after 1976. Figure 2 and Figure 3 show the typical cross-sections of the wide-leg channel beams in each category. These two sections have identical geometry and design concrete strength. The major differences between these two sections are strand diameters and the concrete cover of bottom strands. It should be noted that construction year doesn't always indicate the type of wide-leg channel section. Some bridges constructed after 1976 may have been designed with the pre-1976 plans. One way to determine the bridge type in the field is by identifying the location of the lower strands and measuring the concrete cover through the use of GPR or similar device.

Preliminary load ratings performed on both types of wide-leg channel bridges indicated that recommended posting limits differed between the two designs due to the differences in the size and location of prestressing strands.

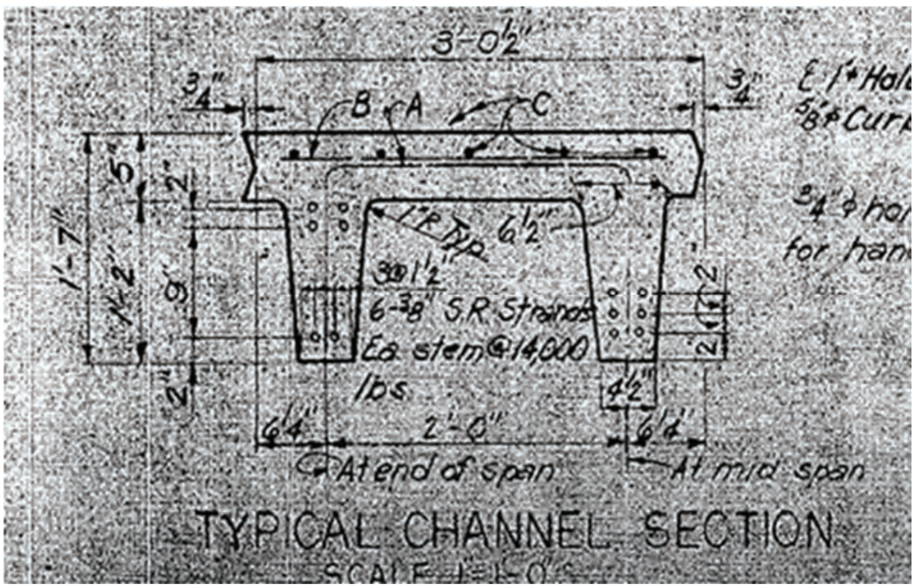


Figure 2: Wide-Leg Channel Beam Typical Section (pre-1976)

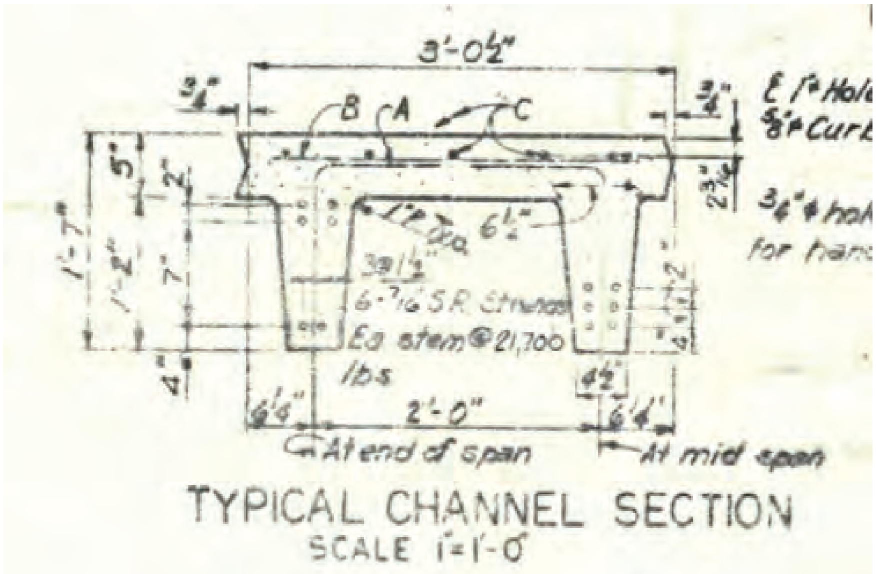


Figure 3: Wide-Leg Channel Beam Typical Section (post-1976)

Table 1 shows the breakdown of the wide-leg channel beam bridges and the numbers that were recommended for testing. The bridges are classified according to their reflective cracking condition. For some bridges, the overlay type or reflective cracking conditions were not available, and these bridges are grouped as 'other'. Based on the inventory size of wide-leg channel beam bridges in South Carolina, twenty (20) wide-leg channel bridges were selected to be load tested for K-factors. This amount of tested bridges provides a significant level of confidence in the applicability of our findings to the broader pool of the wide-leg channel bridges.

Additionally, nine (9) wide-leg channel beam bridges with no, moderate, and heavy reflective cracking were tested to confirm that the skinny-leg channel bridge Visual Guide of live load distribution was also applicable to the wide-leg channel bridges. Since the wide-leg channel bridges with heavy reflective cracking will result in the lowest rating factor, the K-factor load tests were performed on this category of bridges.

One notable difficulty affecting the selection of bridges was the similar geometries of the two wide-leg channel beam design standards. Since the differences between the two standard designs could not be determined based on visual inspection, GPR was used to measure the clear cover of each tested bridge and confirm the correct standard plan. Once the standard plans were confirmed, the bridges were separated to the respective testing groups based on the standard plans and degree of reflective cracking.

Since most of the wide-leg channel bridges are in the upstate region of South Carolina mainly in districts 2, 3 and 4, most of the tested bridges in this project are in the upstate region. Figure 4 shows all the wide -leg-channel bridges in South Carolina and Figure 5 shows the location of the selected bridges for this load testing program.

Table 1: Wide-Leg Channel Beam Bridges Recommended for Load Testing

YEAR OF CONSTRUCTION	REFLECTIVE CRACKING	NO. IN INVENTORY	NO. RECOMMENDED FOR TESTING	DF TESTING	"K" FACTOR TESTING
Pre-1977 (161 Bridges)	None	25	1	1	0
	Moderate	18	1	1	0
	Heavy	107	9	1	9
	No Asphalt	6	1	0	1
	Other	5	0	0	0
Post-1977 (212 Bridges)	None	27	2	2	0
	Moderate	27	2	2	0
	Heavy	91	7	2	7
	No Asphalt	57	3	0	3
	Other	10	0	0	0

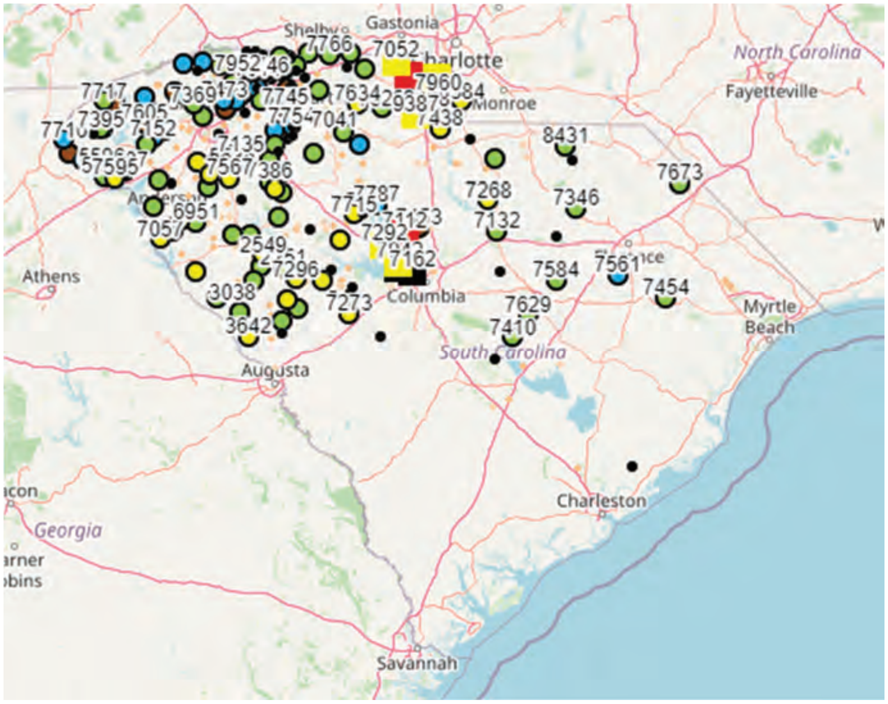


Figure 4: Distribution of Wide-Leg Channel Candidates

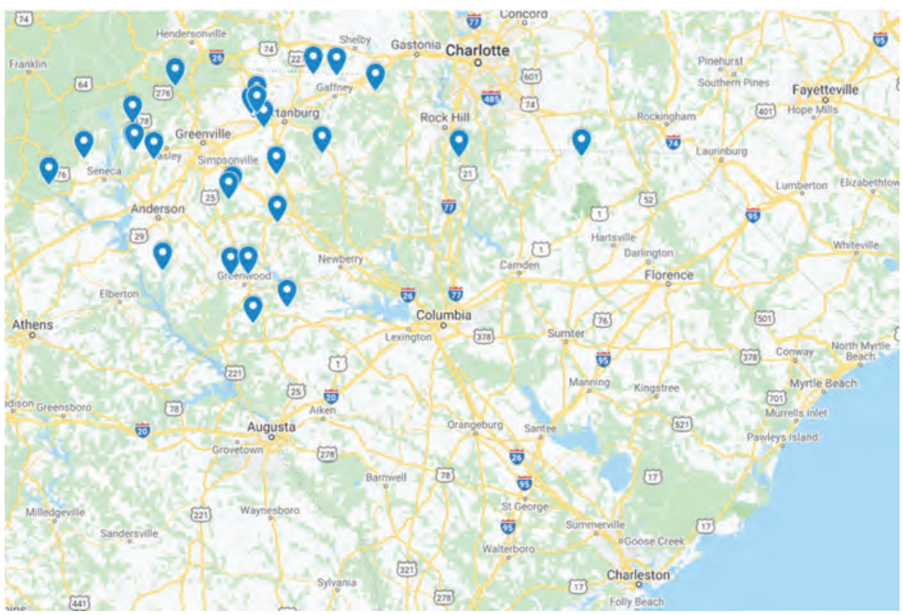


Figure 5: Distribution of Locations of Tested Wide-Leg Channel Bridges

2.3 TEST LOADING

The first phase of the load testing was performed to determine the appropriate live load distribution factors. During this phase, a standard two axle bucket truck loaded to approximately nine (9) tons was used. Details of the bucket truck are included in Section 3.1.3. The weight of the bucket truck was determined based on following two criteria: 1) the test truck weight is sufficient to produce reliably measurable strains on channel beams; and 2) the bridge exhibits linear elastic behavior during the diagnostic load test.

The second phase of load testing was performed on the wide-leg channel bridges to determine the K-factors. During this phase, a standard tri-axle dump truck was used. Details for the dump truck are included in Section 3.1.4. Since the primary intent of the second phase is to calculate the adjustment factor "K", a higher load level is required. As presented in Section 2.1 of this report, the K- factor is dependent on the values of K_a and K_b. The K_b factor is dependent on the he relationship between the unfactored test vehicle effect T and the unfactored gross rating load effect W. AASHTO MBE, 3rd edition, Table 8.8.2.3.1-1 (see Table 2 below), provides guidance on how to determine K_b based on this ratio as well as the member behavior can be extrapolated to 1.33W, where "W" is the unfactored gross rating load effect.

Table 2: AASHTO MBE Table 8.8.2.3.1-1

Table 8.8.2.3.1-1—Values for K_b

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

The intent of "Can member behavior be extrapolated to 1.33W?" in Table 2 is to provide some assurance that the structure has adequate reserve capacity beyond its rated load level W. The existing ratings of the wide-leg channel bridges were controlled by the emergency vehicle EV3 (GW=43 tons). Since the 1.33 W (GW=57.19 tons) will exceed the live load capacity of the wide-leg channel beams based on the existing rating model, the value of K_b will fall in the bottom three rows in Table 8.8.2.3.1-1 and depend on the ratio of T and W. In this case, the only non-zero value of K_b will require a T/W ratio > 0.7. Thus, we could determine the weight of test truck based on the live load capacity of the channel beams and the desired T/W ratio.

The controlling flexural rating vehicle for these wide-leg channel bridges is the EV3 truck, with an unfactored maximum moment of 380.6 k-ft. Since the target T/W ratio is 0.7, a minimum unfactored moment of 266.5 k-ft will be required to be induced by the test truck. Therefore, a tri-axle truck loaded to approximate 62 kips (31 Tons) was used as the Test Truck, which will produce a maximum moment of approximately 269 k-ft.

Because the load induced by the testing will exceed the theoretical cracking moment of the channel beams, the strain gauges may not provide reliable measurements due to the localized flexural cracking of concrete. Therefore, displacement sensors were used to measure the deflections of the channel beams under the test truck loads for the K factor testing.

3 SUMMARY OF LOAD TESTING

3.1 EQUIPMENT

3.1.1 DATA COLLECTION

The data collection from the sensors used the STS4 system manufactured by Bridge Diagnostic Inc (BDI) (See Figure 6). All strain/displacement sensors are connected to the STS nodes through electrical cables. The STS4 nodes communicate with the STS4 base station via Wi-Fi connection. A laptop with STS4 data collection software (STS-LIVE) communicates with the base station via Wi-Fi during the load test (See Figure 7).

Figure 6: STS4 System used in the Load Test (BDI)

Figure 7: Data Collection Schematics

3.1.2 STRAIN TRANSDUCER & CABLE POTENTIOMETER (CPOT)

The diagnostic load testing for DF of the wide-leg channel bridges was conducted with ST350 strain transducers manufactured by Bridge Diagnostic Inc (BDI) (See Figure 8). The ST350 is a resistance-based full Wheatstone Bridge strain gauge designed with four 350 Ω foil gages. The range of strain that can be measured is ± 2000 micro-strain. The nominal gauge length of ST350 strain transducers is 3 inches. The gauge length can be extended to 24 inches by adding an extension arm. A gauge length of 12-inchs was used for the wide-leg channel diagnostic load testing for DF.

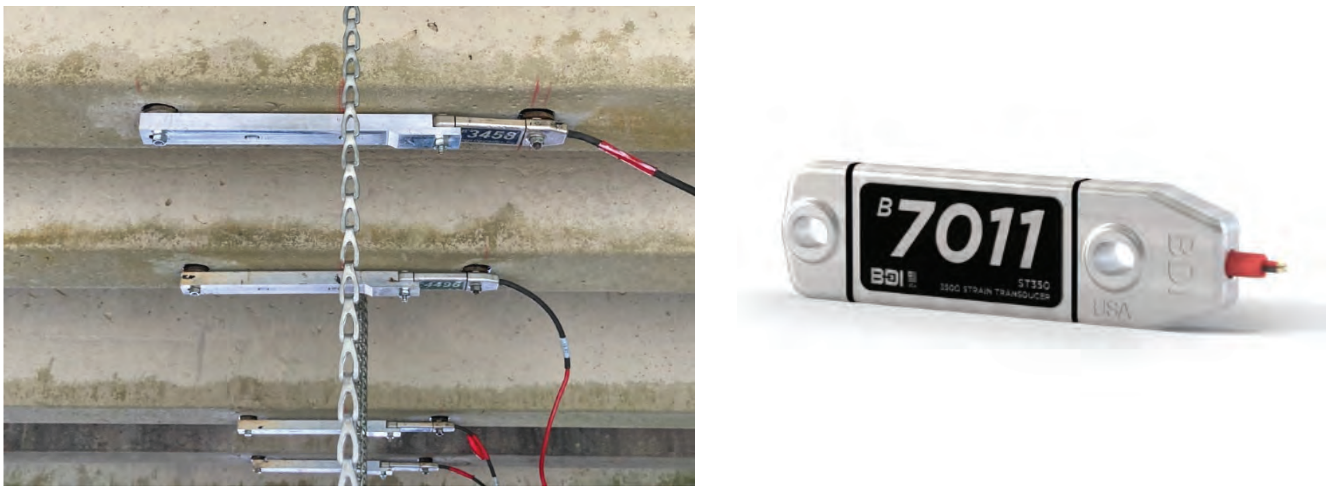


Figure 8: Strain Transducer (BDI) Mounted on bottom of girder (12-in. gauge length)

Considering the channel beams will exceed the cracking moment under the load level needed for the K factor testing, strain gauges may not provide reliable measurements due to localized flexural cracking. The phase 2 load testing of the wide-leg channel bridges was conducted with Cable Potentiometers (CPOTs) manufactured by Bridge Diagnostic Inc (BDI) (See Figure 9). The installed CPOT can measure the displacement with a range of 4 in.

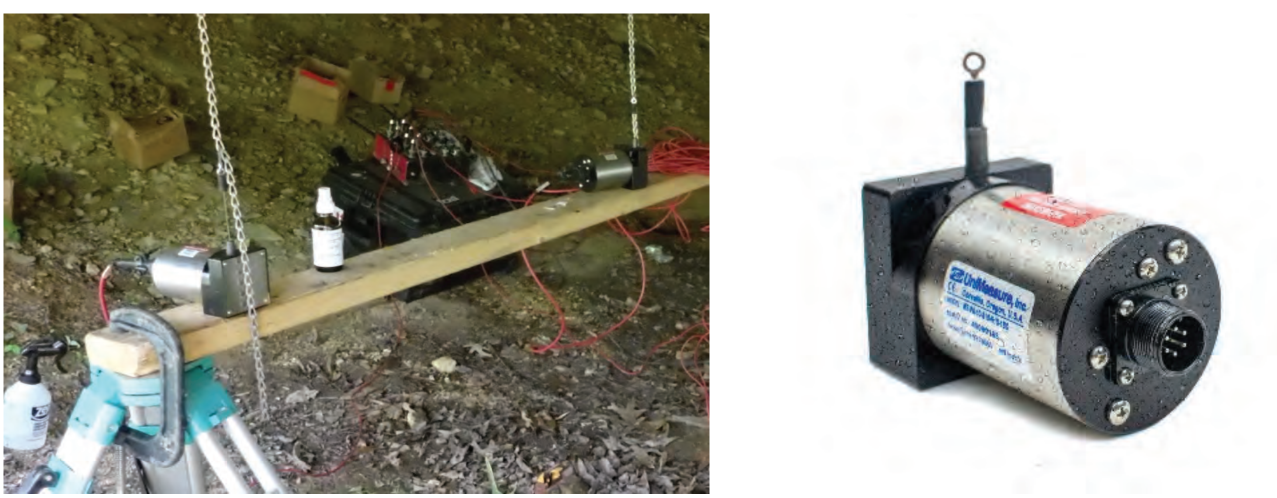


Figure 9: Cable Potentiometer (BDI) Installed on bottom of girder (4-in. range)

3.1.3 LOAD TEST BUCKET TRUCK

A bucket truck, as shown in Figure 10, loaded to approximately nine (9) tons was used for the 1st phase of the load testing to determine the live load distribution factors. The bucket truck was measured before the start of the tests. Figure 11 and Table 3 show the measured dimensions and weights of the bucket truck.

Figure 10: Standard Bucket Truck

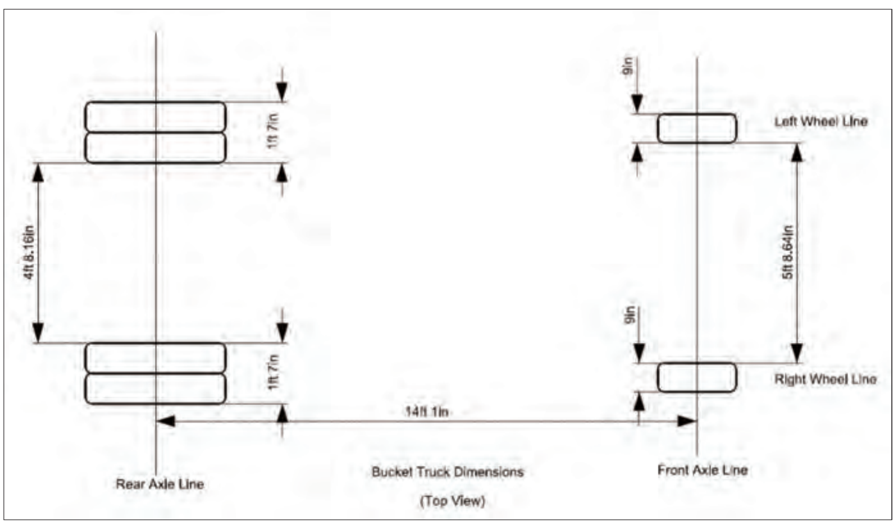


Figure 11: Bucket Truck Dimensions

Table 3: Measured Axle Loads of the Load Test Bucket Truck

WHEEL LINE	FRONT AXLE, LBS	REAR AXLE, LBS
Right	2,840	5,660
Left	3,040	6,780

3.1.4 LOAD TEST DUMP TRUCK

A tri-axle dump truck, as shown in Figure 12, loaded to approximately thirty-one (31) tons was used for the second phase of the load testing to determine the K-factors. The dump truck was measured and weighed prior to each test. Figure 13 and Table 4 show the measured dimensions and weights respectively.

Figure 12: Standard Dump Truck

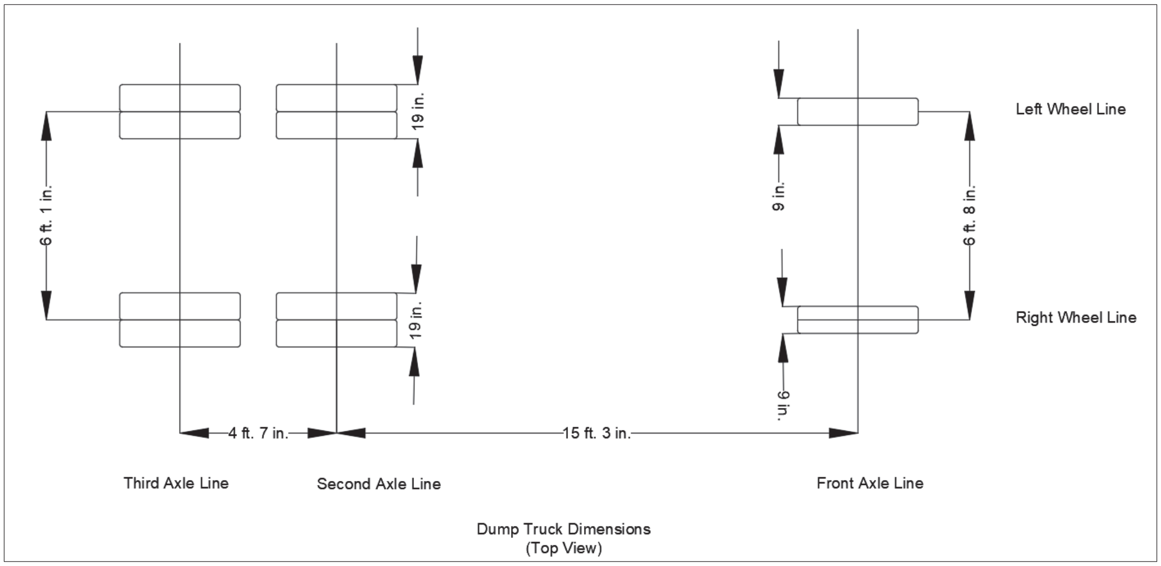


Figure 13: Dump Truck Dimensions

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

BRIDGE ID	WHEEL LINE	FRONT AXLE, LBS	SECOND AXLE, LBS	THIRD AXLE, LBS
3365, 6028, 7119, 7596	Right	7,800	12,560	11,860
	Left	7,940	10,860	10,960
5335, 6637, 6792	Right	7,920	11,840	10,860
	Left	8,800	12,020	11,860
7294, 7155, 7605, 7606, 7766, 8142, 7153	Right	8,200	12,120	11,720
	Left	8,060	11,640	11,360
7842, 7853, 7374, 7741, 6984, 7036	Right	8,340	11,900	11,680
	Left	8,660	12,680	12,900

3.2 INSTRUMENTATION

3.2.1 PHASE 1: LIVE LOAD DISTRIBUTION

The instrumentation used in phase 1 was used to measure the strain of each girder and use that information to calculate the live load distribution factors. To get the structure's response to live load, the mid-span of each girder was instrumented with a strain gauge under each channel leg, as shown in Figure 14 and Figure 15.

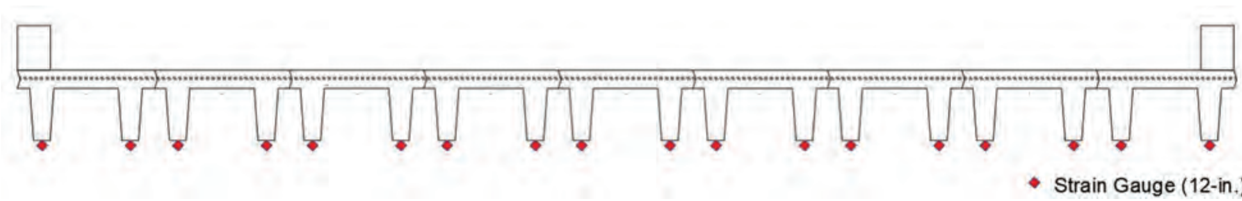


Figure 14: Instrumentation Layout for Wide-leg Channel Bridge Distribution Factor Testing (Typ.)

Figure 15: Wide-leg Channel Bridge Installed with Strain Gauge (Typ.)

3.2.2 PHASE 2: K-FACTOR

The instrumentation used in phase 2 is designed to measure the displacement of girders which is then used to calculate the k-factors. To get the structure's response to live load, the mid-span of each bridges was instrumented with a Cable Potentiometer under all interior beams, as shown in Figure 16 and Figure 17.

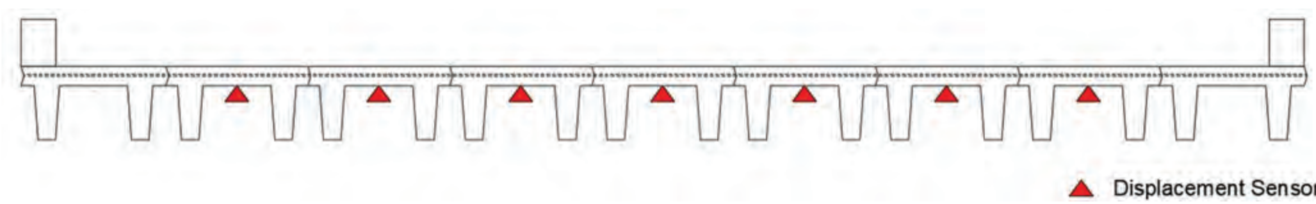


Figure 16: Instrumentation Layout for Wide-leg Channel Bridge "K" Factor Testing (Typ.)

Figure 17: Wide-leg Channel Bridge Installed with Cable Potentiometer (Typ.)

3.3 LOAD TEST PROCEDURE

3.3.1 CONDUCTING THE LOAD TEST

Once all the sensors were installed, prior to testing, the trucks were positioned completely off the bridge. Three pre-test steps 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.

After the pre-test steps were completed, the bucket truck was driven across the instrumented span following the wheel path as discussed in the next section (Section 3.3.2 Wheel Path During Load Test) at crawl speed (approximately 3-5 mph). Longitudinal lines were marked across the middle of the channels on the bridge deck using temporary paint to facilitate the wheel line alignment during load test as shown in Figure 18.

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 (see Figure 19). During phase 1, each of the test cases were repeated twice to ensure the repeatability of results. Based on the width of the bridge and number of channels, 5 to 7 tests were run with 2 passes per test for a total of 10 to 14 tests per bridge. During phase 2, each test was repeated three times to ensure the repeatability of the results. Based again on the bridge width and number of channels, 4 to 6 tests were run with 3 passes per test for a total of 12 to 18 tests per bridge. The wheel path for each test is summarized in the next section.

Figure 18: Temporary Marking Lines on the Bridge Deck

Figure 19: Load Testing in Process

3.3.2 WHEEL PATH DURING LOAD TEST

Phase 1: Live Load Distribution Factors

A sample wheel path configuration for phase 1 is shown in Figure 20. The load paths for each structure are summarized in Table 5.

Table 5: Wheel Path for Each Test of Phase 1 Load Test

Test	Phase 1 Test Bridge ID				
	3669, 7614	**7395**	**7438**	**7810**	**7847**
1	LW 2 ft. from L curb	LW on G2	LW 2 ft. from L curb	LW on G2	LW 2 ft. from L curb
2	LW on G2	LW on G3	LW on G2	LW on G3	LW on G2
3	LW on G3	LW on G6	LW on G3	LW on G6	LW on G3
4	LW on G6	LW on G7	LW on G6	LW on G7	LW on G5
5	RW on G9	LW on G9	LW on G7	RW on G10	RW on G6
6	RW on G10	LW on G10	RW on G10	-	RW 2 ft. from R curb
7	RW 2 ft. from R curb	-	RW 2 ft. from R curb	-	-

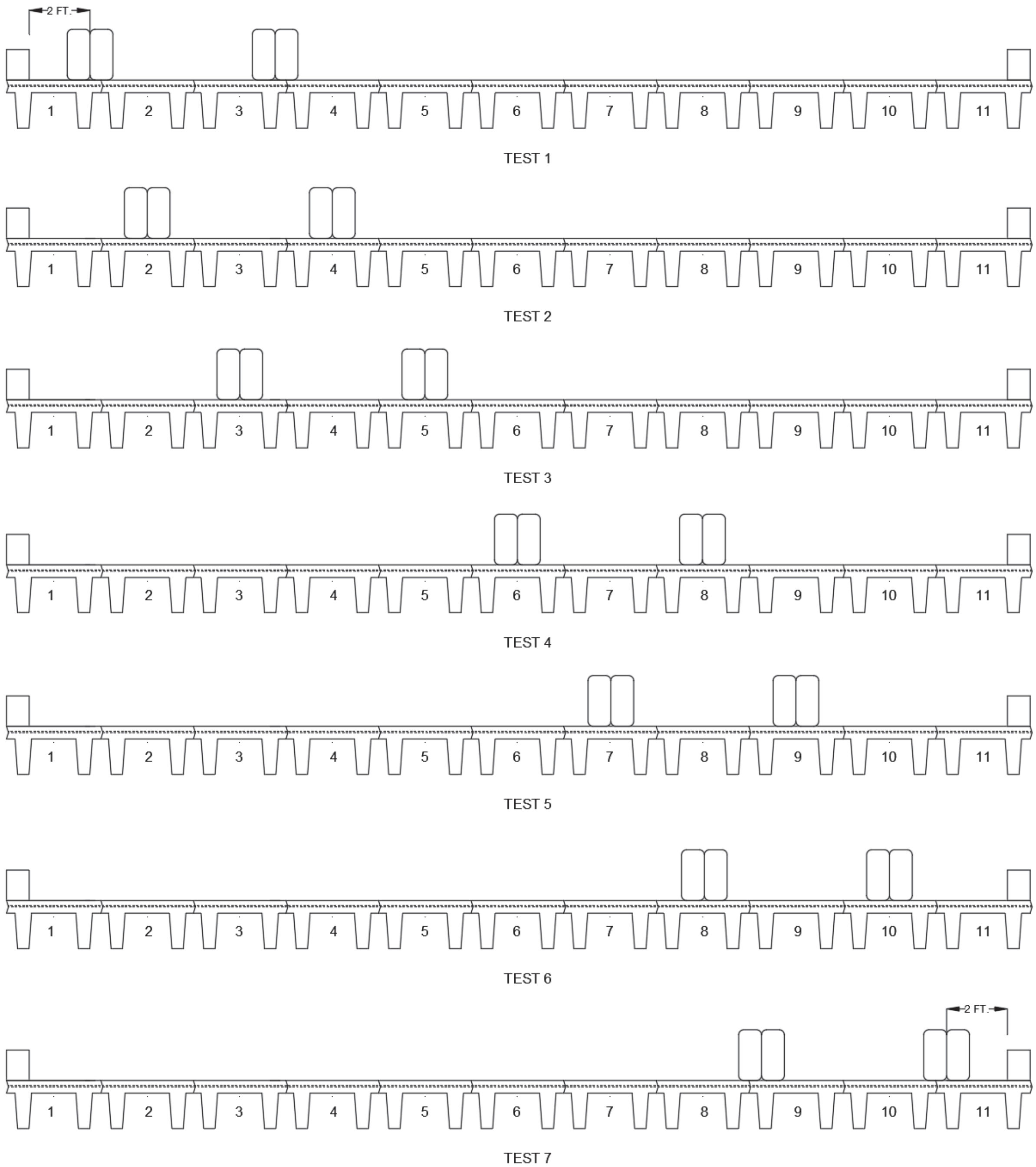


Figure 20: Sample Wheel Path Configuration of Phase 1 Load Test

Phase 2: K-Factor

A sample wheel path configuration for phase 2 testing is shown in Figure 21. The load paths for each structure are summarized in Table 6 and Table 7.

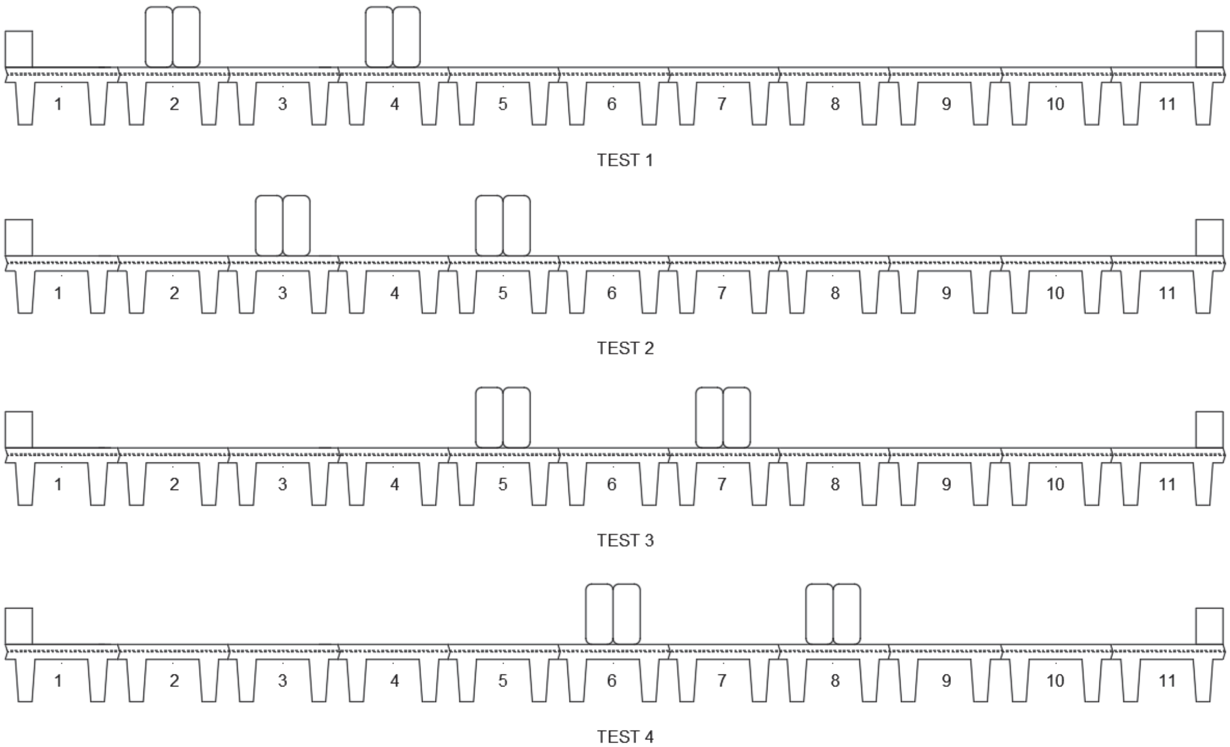


Figure 21: Sample Wheel Path Configuration of Phase 2 Load Test

Table 6: Wheel Path for Each Test of Phase 2 Load Test (Pre-1976)

Test	Phase 2 Test Bridge ID (pre-1976)						
	3365, 5335, 6984, 7036	6028	6637	6792	7153	7596	7119
1	LW on G2	LW on G2	RW on G5	LW on G2	RW on G2	RW on G3	RW on G2
2	LW on G3	LW on G3	RW on G6	LW on G3	RW on G3	RW on G4	RW on G3
3	LW on G5	LW on G5	RW on G8	LW on G6	RW on G6	RW on G6	RW on G5
4	LW on G6	LW on G6	RW on G9	LW on G7	RW on G7	RW on G7	RW on G6
5	-	RW on G9	-	RW on G9	LW on G10	-	-
6	-	RW on G10	-	-	-	-	-

Table 7: Wheel Path for Each Test of Phase 2 Load Test (post-1976)

Test	Phase 2 Test Bridge ID (post-1976)					
	7155, 8142	7294	7374, 7741, 7853	7605, 7766	7606	7842
1	LW on G2	LW on G2	LW on G2	LW on G2	LW on G3	LW on G2
2	LW on G3	LW on G3	LW on G3	LW on G3	LW on G4	LW on G3
3	LW on G6	RW on G7	LW on G5	LW on G6	LW on G7	LW on G6
4	LW on G7	RW on G8	LW on G6	RW on G7	RW on G10	RW on G7
5	RW on G10	-	-	RW on G10	RW on G11	-
6	-	-	-	-	-	-

4 RESULTS OF TESTING

4.1 RESULTS OF DISTRIBUTION FACTOR TESTING

4.1.1 DISTRIBUTION FACTOR TEST RESULTS

Distribution factors (DF) were calculated for each wide-leg channel girder for the 9 bridges tested. This test sample includes bridges from both the Pre-1976 and Post-1976 standard plans, and covers bridges with reflective cracking categorized as none, moderate and heavy. The maximum DF of each bridge is summarized in
Table 8. In Figure 22, the DFs are grouped together based on the reflective cracking conditions. The AASHTO LRFD calculated DF is also provided for reference. It can be noted that the load test calculated DFs for bridges with no reflective cracking are in line with the AASHTO specified DF. Bridges with moderate to heavy reflective cracking have higher DFs than the AASHTO specified value. Based on the load test results presented in Figure 22 and
Table 8, it can be concluded that the wide-leg channel bridges show similar relationships between reflective cracking and load test DFs to those of the skinny-leg channel bridges as anticipated.

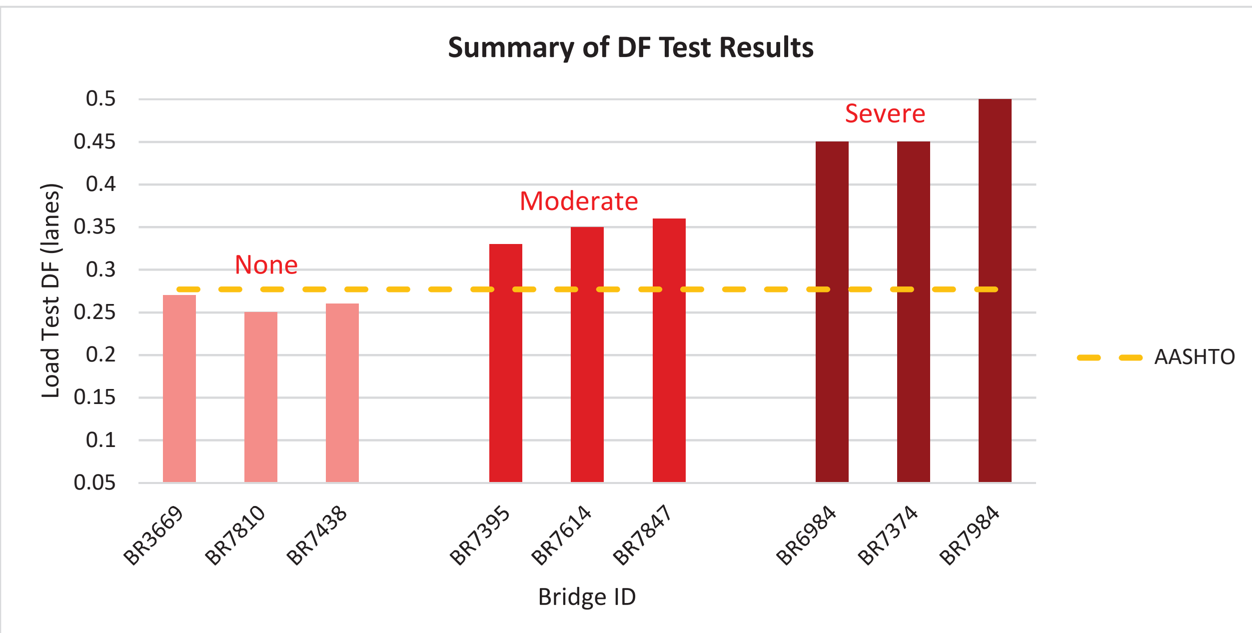


Figure 22: DF Test Summary

A sample distribution factor calculation is provided in Appendix A. Individual bridge information, measured strains and calculated DF for all wide-leg channel bridges are also included in Appendix A.

Table 8: DF Test Summary

Asset ID	Year built	Plan Type (Pre-1976 or Post-1976)	Reflective Cracking Condition	Load Test DF (lanes)
BR3669	1973	Pre	None	0.27
BR7810	1984	Post	None	0.25
BR7438	1981	Post	None	0.26
BR7395	1980	Post	Moderate	0.33
BR7614	1982	Post	Moderate	0.35
BR7847	1984	Post	Moderate	0.36
BR6984	1977	Pre	Severe	0.45
BR7374	1980	Post	Severe	0.45
BR7984	1986	Post	Severe	0.50

4.2 K-FACTOR RESULTS

The k-factor was obtained through comparison of field measured deflection and theoretical deflection caused by load testing in accordance with the AASHTO MBE, 3rd edition, Section 8.8.2. For pre-1976 bridges, the theoretical live load moment causing flexural cracking at mid-span is approximately 104 kip-ft. The moment causing flexural cracking is about 130 kip-ft for post-1976 bridges. Since the applied moment at mid-span of the channel beams due to the load test truck with a DF of 0.5 lane (about 145 kip-ft) is greater than the theoretical cracking moment for both sets of bridges, the effective moment of inertia was conservatively used to calculate the theoretical deflections.

The bridges were sorted by design standard used, with Group 1 representing pre-1976 design standards and group 2 representing post-1976 design standards. The controlling k-factor of each tested wide-leg channel bridge is summarized below in Figure 23 and Figure 24. The controlling K-factor for pre-1976 plan bridges ranges from 1.30 to 1.78 as shown in Figure 23 and Table 9. The controlling K-factor for post-1976 plan bridges ranges from 1.26 to 1.87 as shown in Figure 24 and Table 10.

The measured deflections and K-factors for each tested wide leg channel bridge are provided in Appendix B. A sample calculation for K-factor and theoretical deflection are also provided in the Appendix B.

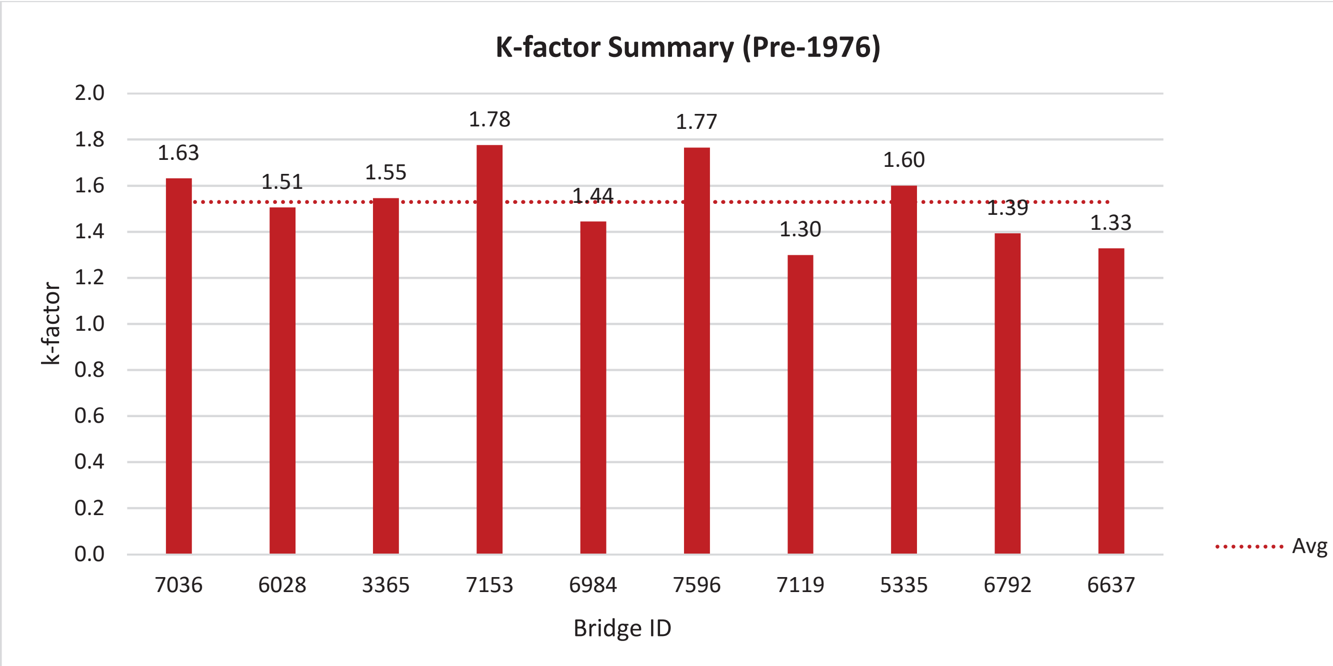


Figure 23: Controlling K-Factor of Each Wide-Leg Channel Bridges in Group 1 (pre-1976)

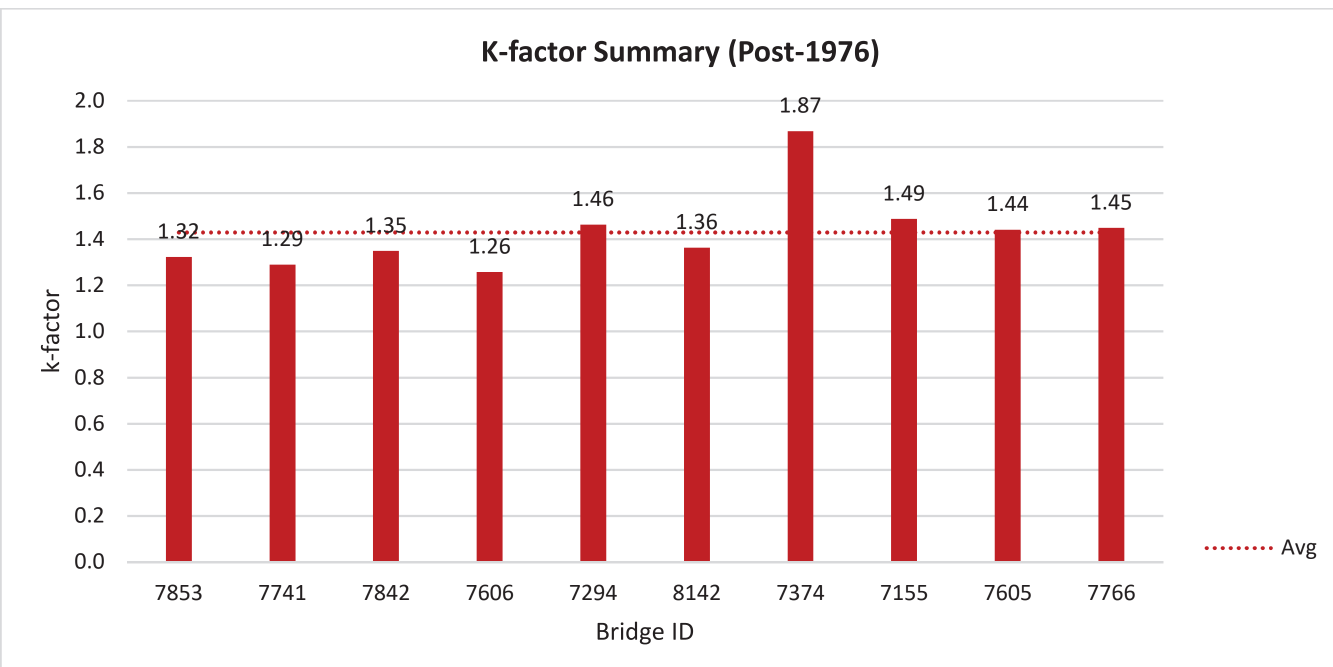


Figure 24: Controlling K-Factor of Each Wide-Leg Channel Bridges in Group 2 (post-1976)

Table 9: Group 1 Controlling K-Factor Summary (pre-1976)

Bridge ID	Year Built	Total Number of Units	Asphalt Thickness (in.)	Reflective Cracking	Controlling Girder	Controlling k-factor
7036	1977	9	0	No Asphalt	G2	1.632
6028	1972	11	1.5	Heavy	G10	1.505
3365	1973	11	3	Heavy	G8	1.545
7153	1978	11	2	Heavy	G4	1.776
6984	1977	9	1	Heavy	G4	1.444
7596	1982	11	2.5	Heavy	G6	1.765
7119	1978	9	1	Heavy	G7	1.298
5335	1968	9	N/A	Heavy	G3	1.599
6792	1975	11	N/A	Heavy	G6	1.393
6637	1975	11	N/A	Heavy	G4	1.328

Table 10: Group 2 Controlling K-Factor Summary (post-1976)

Bridge ID	Year Built	Total Number of Units	Asphalt Thickness (in.)	Reflective Cracking	Controlling Girder	Controlling k-factor
7853	1984	9	0	No Asphalt	G5	1.322
7741	1983	9	0	No Asphalt	G2	1.289
7842	1984	9	0	No Asphalt	G5	1.349
7606	1982	13	2.5	Heavy	G6	1.257
7294	1979	11	1	Heavy	G6	1.463
8142	1988	11	1.5	Heavy	G6	1.363
7374	1980	14	4.25	Heavy	G5	1.868
7155	1978	11	1.5	Heavy	G9	1.488
7605	1982	13	2	Heavy	G4	1.441
7766	1984	11	1.5	Heavy	G6	1.449

4.2.1 STATISTICAL ANALYSIS OF K-FACTOR RESULTS

The K-factors of both pre- and post- 1976 design standards were checked for statistical distribution. Figure 25 shows the K-factor frequency distribution for pre-1976 wide-leg channel beams. K-factors were calculated for a total of 75 wide-leg channel beams from 10 tested bridges. The idealized normal distribution curve was also plotted based on 75 data points with a mean of 1.941 and standard deviation of 0.310. Figure 25 indicates a very slight skew in the distribution to the right. A normalcy test was performed to ensure that a normal distribution would be appropriate to use for this data set. Figure 26 shows the normalcy test for the data, where the expected vs actual K-factor values are compared against the Z values with a mean of zero and variance of one. Figure 26 indicates that the expected values with the normal distribution curve are very close to the actual K-factor values for the Pre –1976 structures.

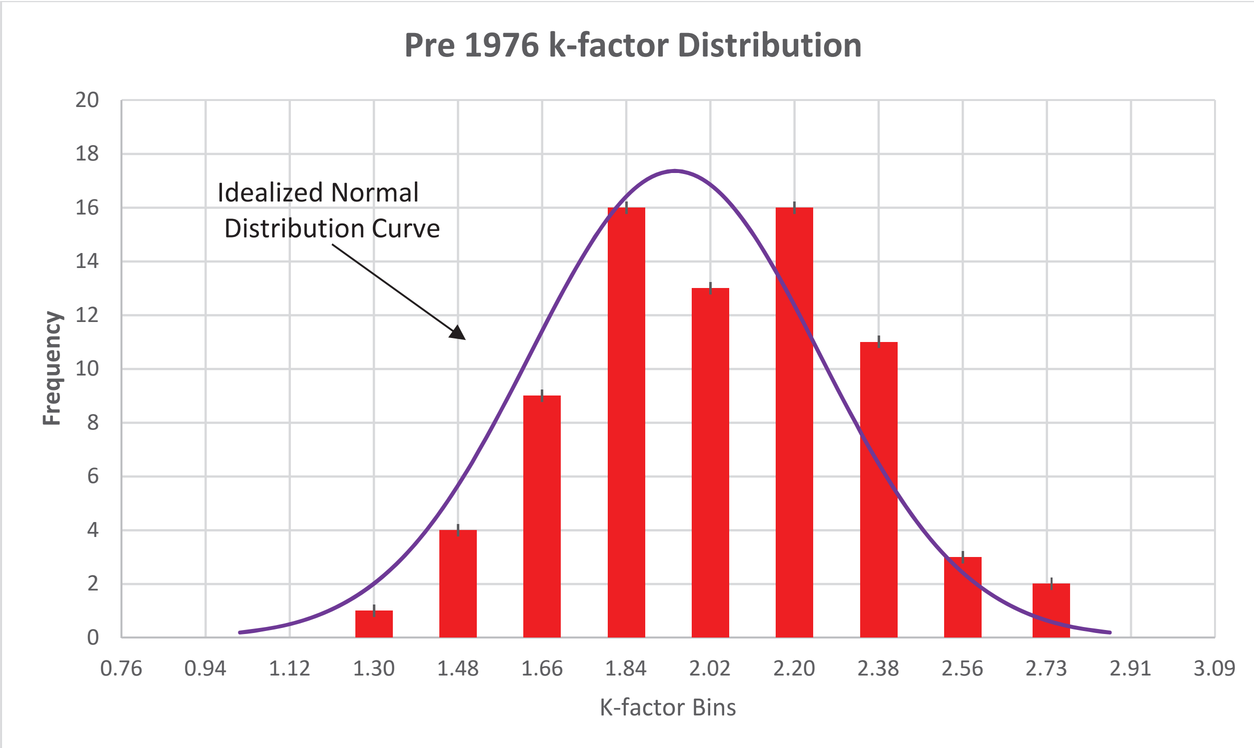


Figure 25: Pre-1976 K-factor Frequency Distribution and Idealized Normal Distribution

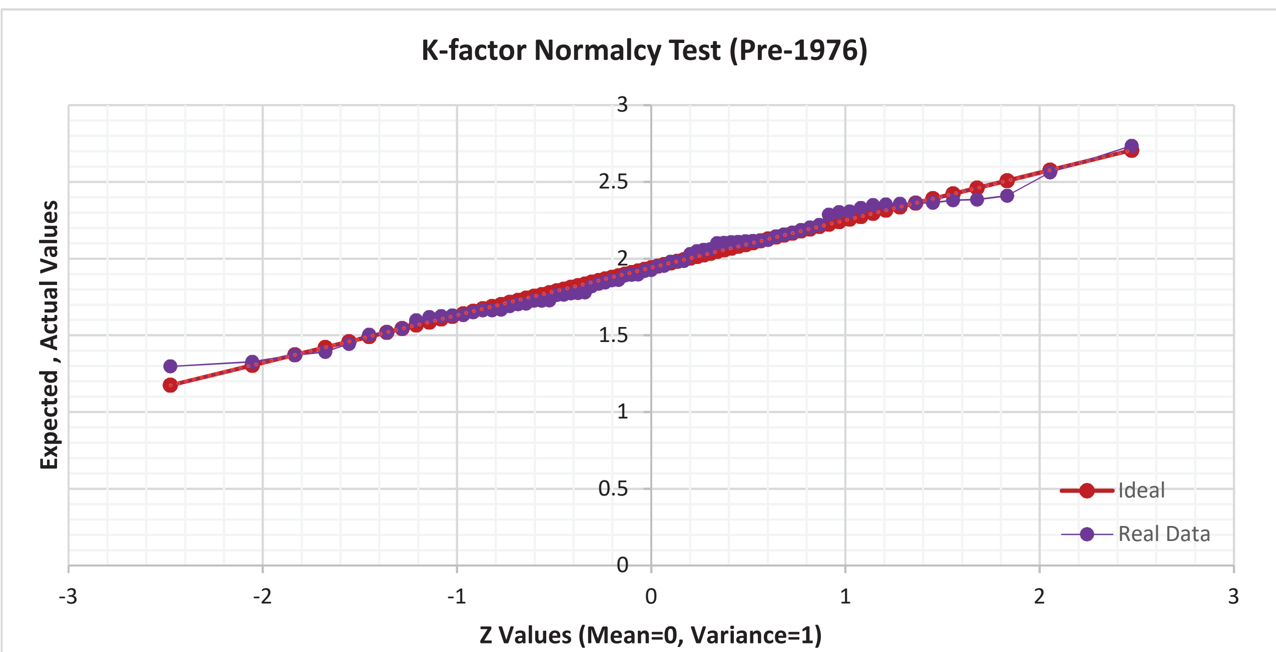


Figure 26: Normalcy Test for Pre-1976 K-Factor Values.

Table 11 below lists the K-factor values for pre-1976 standard details with different confidence intervals.

Table 11: Pre-1976 K-factor Values for Different Confidence Level

Confidence Level	k-factor
95%	1.54
96.5%	1.48
97.5%	*1.43*

Figure 27 shows the frequency distribution of post 1976 type wide leg channel K-factors. There were 80 wide leg channels in the 10 post 1976 bridges that were tested for K-factors. Figure 27 shows that the K-factors are heavily skewed to the right and right which is an indication of a log normal probability distribution. The 'e' based logarithm of the K-factor values therefore tested to determine if they were normally distributed.

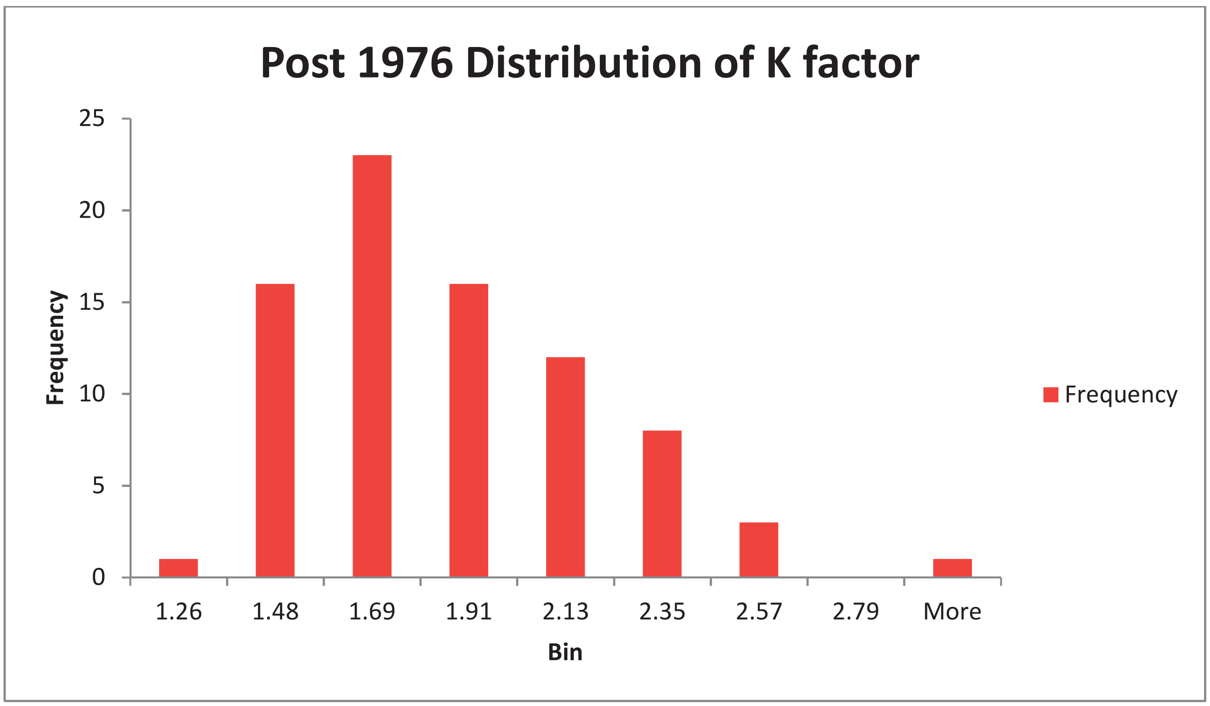


Figure 27: Post-1976 K-factor Frequency Distribution

Figure 28 shows the frequency distribution of the logarithm of post 1976 k-factor values along with the idealized normal distribution curve with average k factor of $\log_e(1.761)$ and standard deviation of $\log_e(0.334)$. A normalcy test was performed to ensure that a normal distribution would be appropriate to use for this data set. Figure 29 shows the normalcy test of the post 1976 k factor data where the logarithm of actual and expected k-factor values are plotted against the z values that is based on an average of zero and variance of one. Figure 29 shows the expected values with the log-normal distribution is a good model of the actual K-factor values.

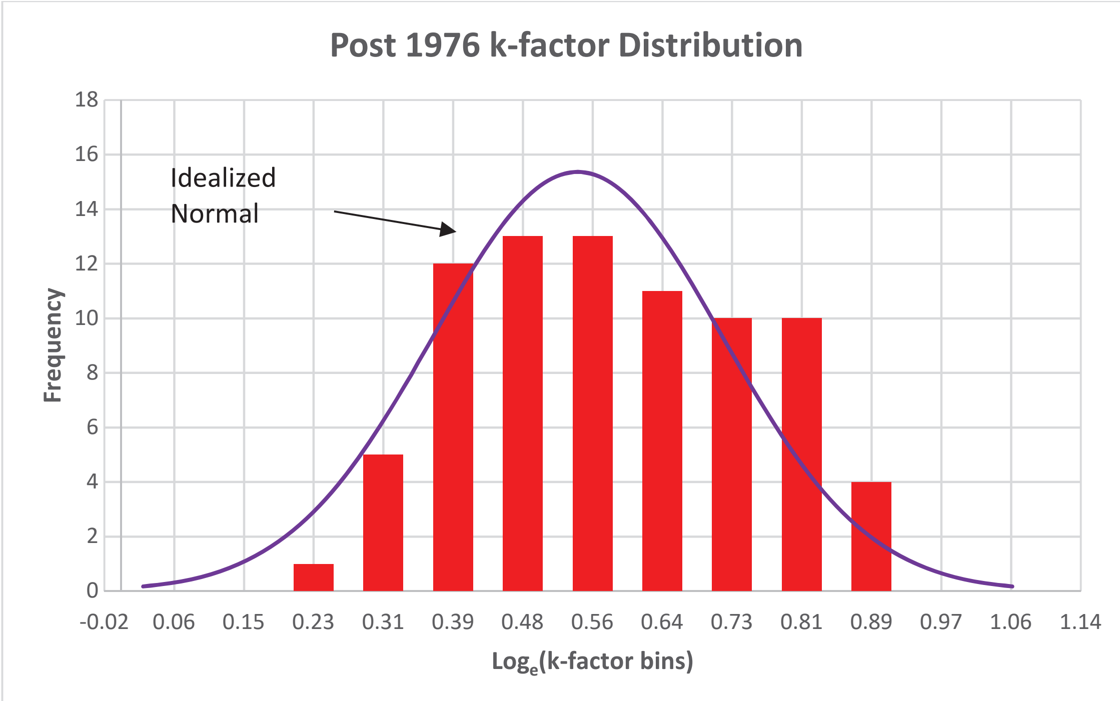


Figure 28: Post-1976 K-factor Frequency Distribution and Idealized Normal Distribution

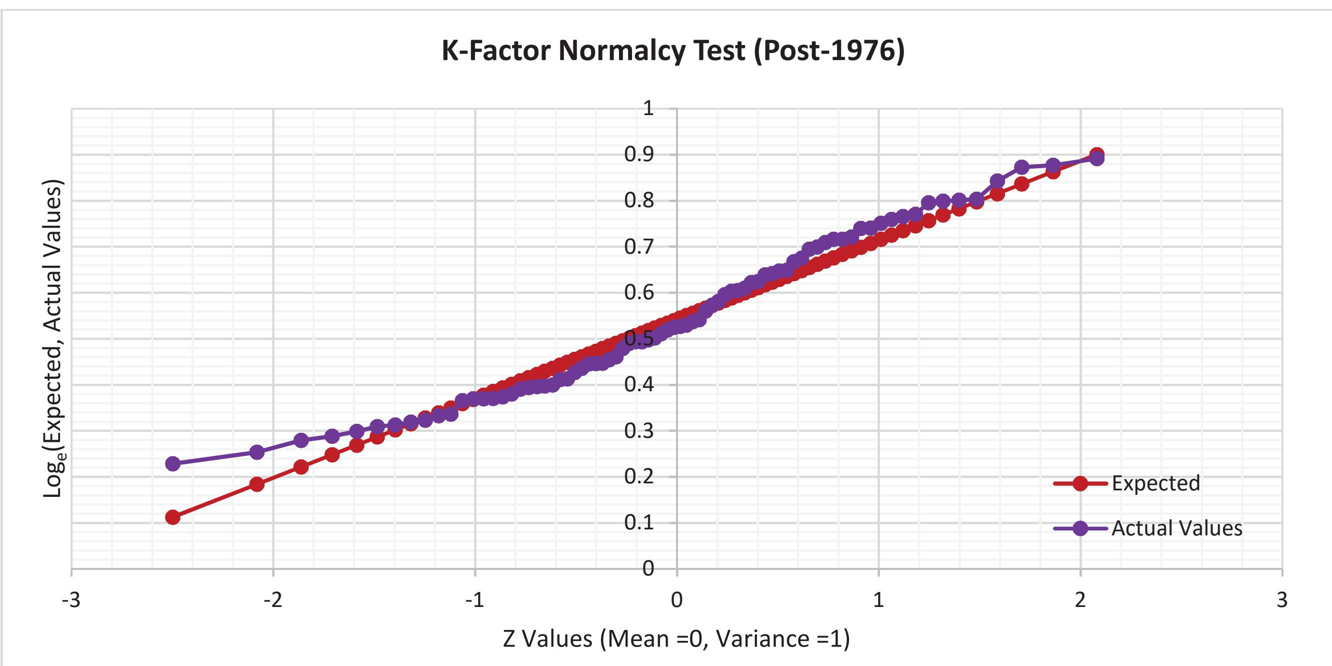


Figure 29: Normalcy Test for Post-1976 K-Factor Values.

Table 12 shows the various K-factor values for post-1976 plan wide leg channel bridges with corresponding confidence level assuming log-normal distribution.

Table 12: Post 1976 k-factor Values for Different Confidence Level

Confidence Level	K-factor
95%	1.38
96.5%	1.33
97.5%	*1.30*

4.2.2 EVALUATION OF APPROPRIATE CONFIDENCE INTERVAL

Reliability is the probability that a system performs correctly during a specific time duration. The AASHTO LRFD Design Specifications has been calibrated for a target reliability index of 3.5 with a corresponding probability of exceedance of 2.0E-04 during the 75-year design life of the bridge. Since bridges contain multiple components connected as a complex system, the effective reliability of the system depends on the confidence we have on both the capacity and demand side of the fundamental design equation.

In choosing an appropriate confidence level fort the k-factors that will be used consideration should be given to the reliability of the other parameters. For instance, the characteristic strength of concrete is typically determined using a 90 percent confidence level. Similarly, on the demand side the characteristic load is derived based on a 5 percent probability of a greater load being applied (i.e. 95 percent confidence level). In order to be consistent with the confidence levels that are applied to the other inputs to the fundamental design equation a confidence level of 97.5 percent would be appropriate and slightly conservative.

5 CONCLUSIONS & RECOMMENDATIONS

Diagnostic load tests were performed on the wide-leg channel bridges to determine the live load distribution factors and appropriate K-factors to use in the load ratings. In the first phase of the testing program, 9 wide-leg channel bridges were tested, and the controlling distribution factors were reported for each tested bridge. In the second phase, 20 wide-leg channel bridges with severe reflective cracking were load tested for K-factors. Out of these 20 bridges, 10 bridges are pre-1976 and the others are post-1976. K-factors were calculated for all tested channels and the minimum K-factor value for each bridge were reported. The distributions of the K-factors were fitted to normal and log-normal distributions and a K-factor was recommended for the entire population of the pre- and post- 1976 wide-leg channel bridges. Conclusion and recommendations based on the load testing results are presented in the following subsection.

5.1 CONCLUSIONS

The following conclusion can be drawn from the results of distribution factor testing:

- The degree of reflective cracking between the channel sections is the best indicator of the distribution factor that should be utilized in the load rating
- For bridges with no reflective cracking, tested DFs are consistent with the AASHTO specified values.
- For bridges with moderate reflective cracking, tested DFs are approximately 0.35 lane.
- For bridges with heavy/severe reflective cracking, tested DFs are close to 0.5 lane.
- Extent of correlation between reflective cracking and DF for both pre- and post-1976 wide-leg channel bridges are in line with those defined in the skinny-leg channel load test report.

The following conclusions can be drawn from the results of K-factor testing:

- K-factors for pre-1976 plan wide-leg channel bridges follow normal distribution with a recommend K= 1.43 with a 97.5% confidence level
- Pre-1976 plan wide-leg channels showed K-factors ranged from 1.298 to 2.735. The average and standard deviation were 1.941 and 0.310 respectively.
- K-factors for post-1976 plan wide leg channel bridges follow a log-normal distribution with a recommended K=1.30 with a 97.5% confidence level
- Post 1976 plan wide-leg channels showed K-factors ranging from 1.257 to 3.003. The average and standard deviations were 1.761 and 0.334.

5.2 RECOMMENDATIONS

The following recommendations are made based on the findings of this study.

- For pre and post 1976 plan wide-leg channel bridges, the Skinny Leg Channel Visual Guide for live load distribution factors should be applied.
- For any pre-1976 plan wide-leg channel bridges, the calculated load rating factors should be modified with a K-factor of 1.43
- For any post-1976 plan wide-leg channel bridges, the calculated load rating factors should be modified with a K-factor of 1.30

APPENDIX

A *DF* RESULTS

SAMPLE DF CALCULATION
Example: BR 07438

Ignoring stiffness contribution from curbs for end channels, the distribution factor of a girder or channel was calculated using the following equation:

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

Where ε_i is the maximum measured strain at a channel during a load test and $\sum_{j=1}^{n} \varepsilon_j$ is the summation of strains of all the girders/channels at the same point in time. Strain and resulting moment in the load bearing girders and slabs for Test-1 Run-1 is shown in the following Table:

Wheel		C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2	C6L1	C6L2	-	C11L1	C11L2	Sum
Left on Ch2	Strain (με)	139.8	193.7	217.4	156.0	149.0	167.0	149.6	100.3	66.5	41.1	27.3	17.0	-	-1.1	-1.0	1447.2
	DF	0.097	0.134	0.150	0.108	0.103	0.115	0.103	0.069	0.046	0.028	0.019	0.012	-	-0.001	-0.001	1.000
	Sum DF	0.230		0.258		0.218		0.173		0.074		0.031		-	0.000		1.000
	Corrected DF	0.211		0.236		0.238		0.188		0.081		0.033		-	0.000		1.000

For example, Channel 2 distribution factor would be the sum of the DF from its two legs. The DF for each leg is calculating using the formula above which is the ratio between the recorded strain on that leg and the sum of the strains recorded on all other legs.

C2L1 DF:

$$DF_{C2L1} = \frac{\varepsilon_{C2L1}}{\sum_{j=1}^{n} \varepsilon_j} = \frac{217.4}{1447.2} = 0.150$$

C2L2 DF:

$$DF_{C2L2} = \frac{\varepsilon_{C2L2}}{\sum_{j=1}^{n} \varepsilon_j} = \frac{156}{1447.2} = 0.108$$

Channel 2 DF:

$$DF_{C2} = 0.150 + 0.108 = 0.258$$

Furthermore, in the case where the truck weight is not evenly distributed between the left and right side, a truck wheel correction factor needs to be applied. It was determined that the left side of the truck was 9% heavier than the right side.

Channel 2 DF (With truck correction applied):

$$DF_{C2} = 0.258 * 0.91 = 0.236$$

Table 13: 03669 Bridge Summary

Asset ID	03669
Test Date	05/21/2021
Asphalt Thickness	2in
Year of Built	1973
ADT	360
County	Greenwood
Facility Carried	S-284
Feature Intersected	John's Creek
Number of Units	11
Latitude	34.23130639
Longitude	-82.23865528
Reflective Cracking	None
Maximum DF	0.27
Controlling Girder	G2

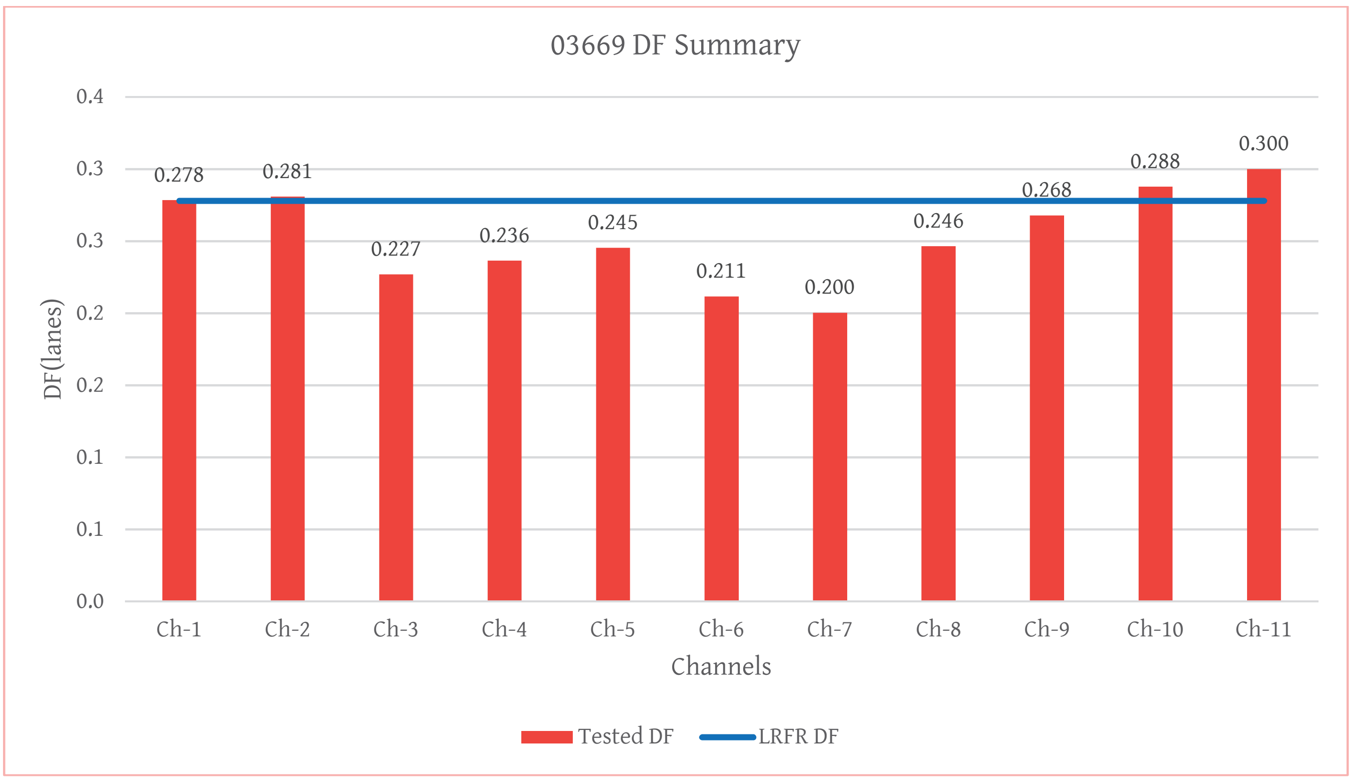


Figure 30: 03669 DF Summary

Table 14: 03669 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW at 2ft from curb	1	229	308	229	185	165	197	173	99	78	44
	2	221	296	231	187	163	197	177	101	80	46
LW on Ch2	3	76	78	235	337	189	170	199	210	163	91
	4	78	81	234	323	187	166	194	196	155	88
LW on Ch4	5	26	25	89	133	183	233	215	191	212	201
	6	25	23	87	130	181	236	217	191	212	204
LW on Ch6	7	4	4	7	8	13	19	33	54	80	121
	8	3	2	6	8	13	20	34	56	82	123
RW on Ch9	9	1	1	2	2	5	7	13	21	33	53
	10	0	0	1	2	5	6	12	20	31	49
RW on Ch10	11	0	1	0	0	1	2	4	7	11	17
	12	2	2	2	2	4	4	7	10	15	21
RW at 2ft from curb	13	0	0	0	0	1	2	4	5	8	12
	14	2	1	1	0	1	1	2	4	6	10

Table 14 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2
LW at 2ft from curb	1	29	17	9	4	2	2	0	0	0	0	0	0
	2	30	18	10	4	3	1	0	0	0	0	0	1
LW on Ch2	3	62	39	23	10	8	5	2	2	0	1	0	1
	4	60	39	23	9	8	5	3	2	1	2	1	1
LW on Ch4	5	138	91	59	34	22	14	8	6	3	3	1	1
	6	140	91	59	33	22	14	8	5	3	3	1	0
LW on Ch6	7	160	250	182	164	191	209	131	75	45	37	16	19
	8	162	247	181	165	193	207	129	74	44	37	15	20
RW on Ch9	9	71	136	175	204	177	165	224	195	118	89	40	40
	10	66	129	161	207	181	164	213	205	122	92	41	41
RW on Ch10	11	24	46	67	112	175	233	187	171	196	250	110	104
	12	28	51	71	119	179	237	196	177	200	256	122	111
RW at 2ft from curb	13	17	29	42	41	98	182	243	172	161	248	274	199
	14	14	26	37	39	93	176	238	168	158	250	265	197

Table 15: 07810 Bridge Summary

Asset ID	07810
Test Date	03/21/2021
Asphalt Thickness	3in
Year of Built	1984
ADT	950
County	Laurens
Facility Carried	S-30-43
Feature Intersected	BR to Little River
Number of Units	11
Latitude	34.47405917
Longitude	-81.97644972
Reflective Cracking	None
Maximum DF	0.25
Controlling Girder	G8

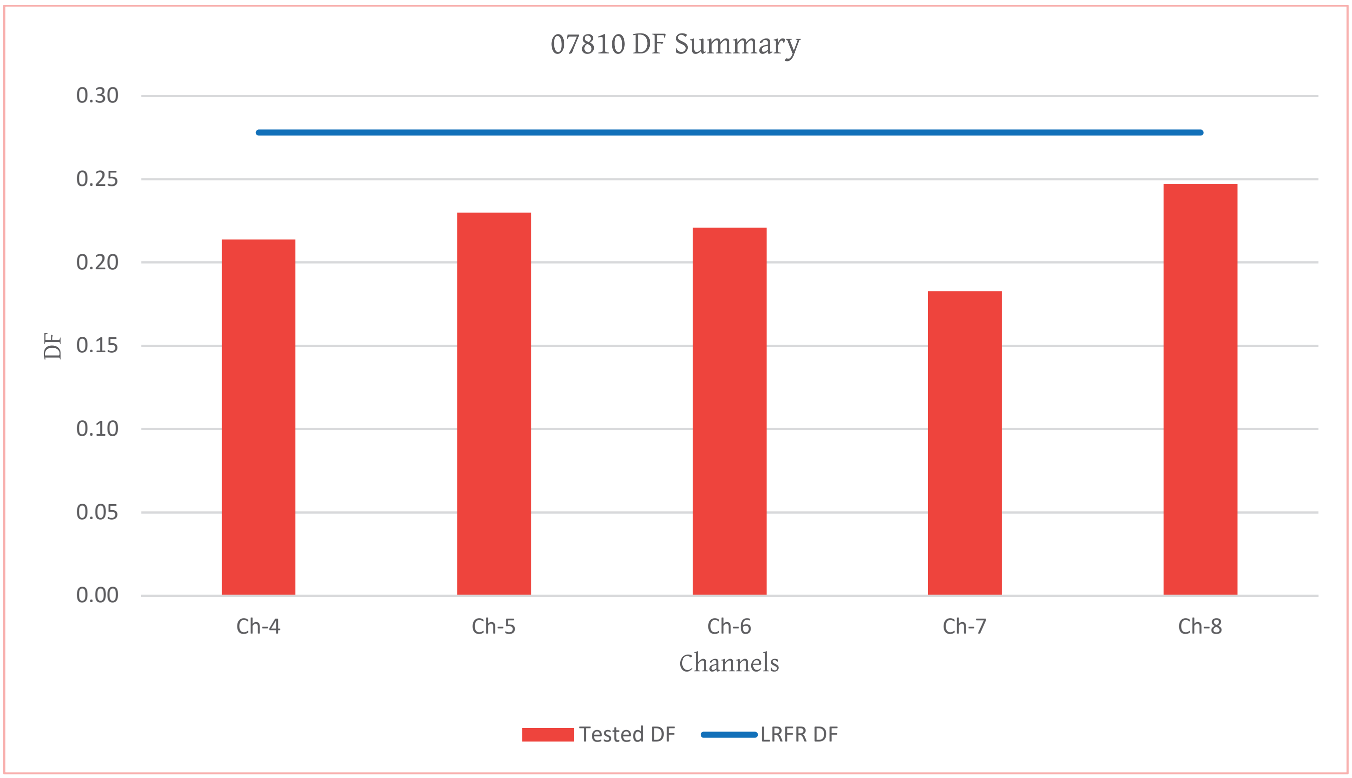


Figure 31: 07810 DF Summary

Table 16: 07810 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW on Ch2	1	97	84	226	346	111	108	142	143	86	16
	2	98	85	212	309	109	109	143	132	80	15
LW on Ch3	3	8	7	18	25	270	262	146	155	146	169
	4	8	10	26	37	273	264	153	162	152	172
LW on Ch6	5	1	2	3	4	17	20	33	50	65	104
	6	0	0	2	2	16	17	31	46	60	91
LW on Ch7	7	0	0	1	2	6	8	16	24	32	48
	8	1	1	1	1	7	9	17	26	34	51
RW on Ch10	9	2	1	1	2	3	5	7	11	15	23
	10	2	1	2	3	4	5	7	12	16	24

Table 16 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2
LW on Ch2	1	35	30	17	14	9	6	4	3	2	1	1	1
	2	32	27	15	12	8	5	3	1	1	0	0	0
LW on Ch3	3	88	72	41	34	22	16	9	7	2	1	1	0
	4	94	77	44	37	24	18	10	8	3	1	1	0
LW on Ch6	5	165	188	124	148	159	172	108	103	22	12	1	0
	6	155	184	119	140	151	169	111	105	23	14	3	1
LW on Ch7	7	62	95	116	163	147	161	233	282	40	24	5	3
	8	68	101	123	165	151	167	225	271	42	26	5	3
RW on Ch10	9	30	48	57	94	133	169	119	119	363	129	33	20
	10	31	50	58	96	134	173	129	131	341	145	38	23

Table 17: 07438 Bridge Summary

Asset ID	07438
Test Date	04/08/2021
Asphalt Thickness	3.5in
Year of Built	1981
ADT	400
County	Chester
Facility Carried	S-13-701
Feature Intersected	Tinkers Creek
Number of Units	11
Latitude	34.77608194
Longitude	-80.9489933
Reflective Cracking	None
Maximum DF	0.26
Controlling Girder	G8

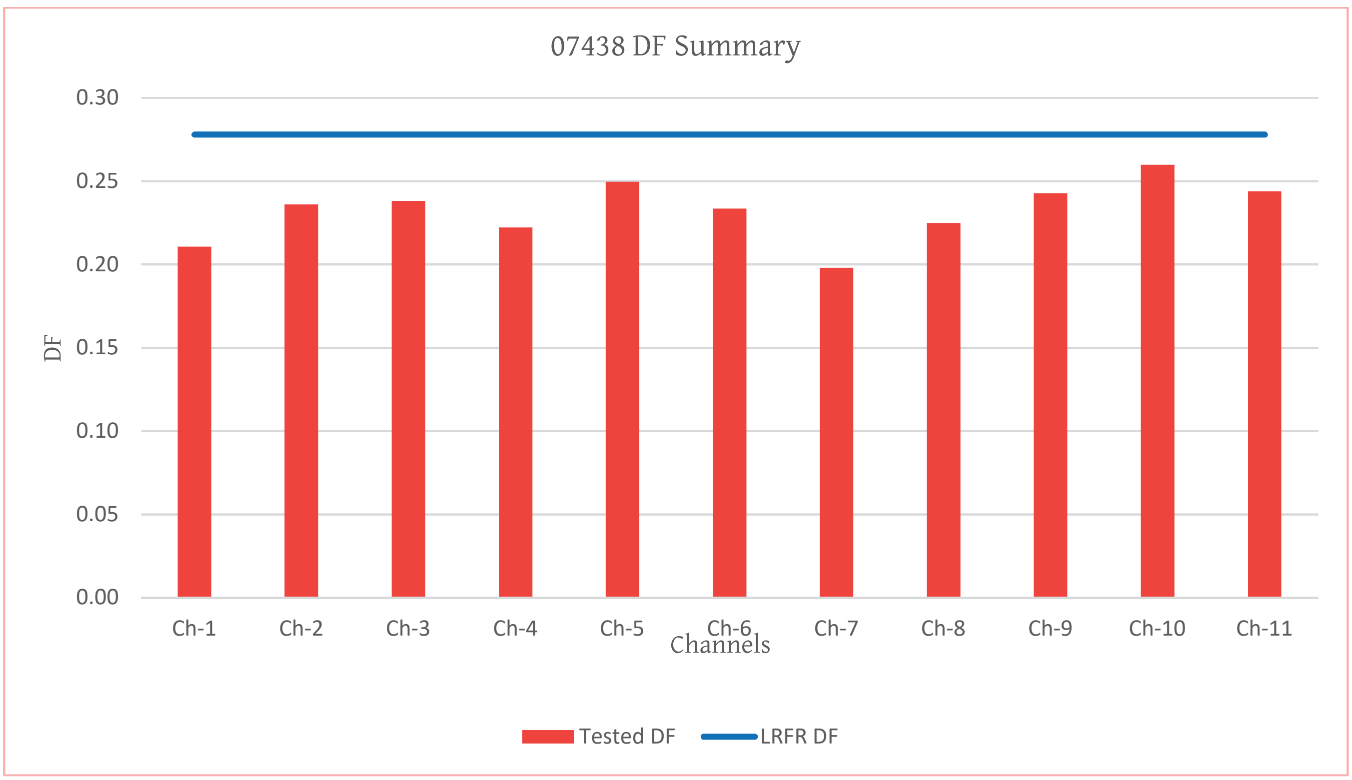


Figure 32: 07438 DF Summary

Table 18: 07438 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW at 2ft from Curb	1	147	196	218	159	152	169	150	100	68	44
	2	142	191	217	159	152	168	152	104	73	47
LW on Ch2	3	103	129	183	181	156	142	155	140	106	63
	4	100	125	180	187	156	142	153	143	114	67
LW on Ch3	5	44	54	82	110	150	165	139	149	163	171
	6	46	56	83	112	153	169	140	150	165	171
LW on Ch6	7	2	3	6	9	13	18	25	43	56	78
	8	3	3	6	8	12	17	24	39	52	73
LW on Ch7	9	2	2	3	4	6	8	12	20	29	41
	10	1	1	3	4	6	8	12	19	29	41
RW on Ch10	11	0	0	0	1	2	3	5	9	14	21
	12	0	0	0	1	2	3	5	8	13	20
RW at 2ft from Curb	13	0	0	0	1	1	2	3	5	8	13
	14	0	0	0	1	1	3	4	7	10	15

Table 18 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2
LW at 2ft from Curb	1	29	18	11	8	5	3	1	1	0	0	0	0
	2	32	20	12	9	5	4	2	1	0	0	0	0
LW on Ch2	3	45	29	18	13	8	5	2	1	0	0	0	0
	4	47	30	18	14	9	6	3	1	0	1	0	0
LW on Ch3	5	85	58	39	30	19	13	7	5	2	1	1	1
	6	89	59	41	31	20	14	7	4	2	0	0	0
LW on Ch6	7	200	182	148	154	155	155	101	69	48	34	25	19
	8	190	182	142	147	147	154	101	67	48	34	26	19
LW on Ch7	9	75	99	139	166	136	138	164	155	98	67	55	42
	10	72	96	135	170	139	138	160	157	100	69	57	43
RW on Ch10	11	33	43	65	110	142	178	145	147	172	168	131	91
	12	32	42	63	107	137	172	141	143	165	161	127	87
RW at 2ft from Curb	13	23	29	45	70	98	169	186	145	144	204	202	133
	14	24	30	46	73	100	170	184	144	143	198	192	124

Table 19: 07395 Bridge Summary

Asset ID	07395
Test Date	04/08/2021
Asphalt Thickness	2in
Year of Built	1980
ADT	900
County	Oconee
Facility Carried	S-13-148
Feature Intersected	Cane Creek
Number of Units	13
Latitude	34.77438306
Longitude	-83.06646389
Reflective Cracking	Moderate
Maximum DF	0.33
Controlling Girder	G5

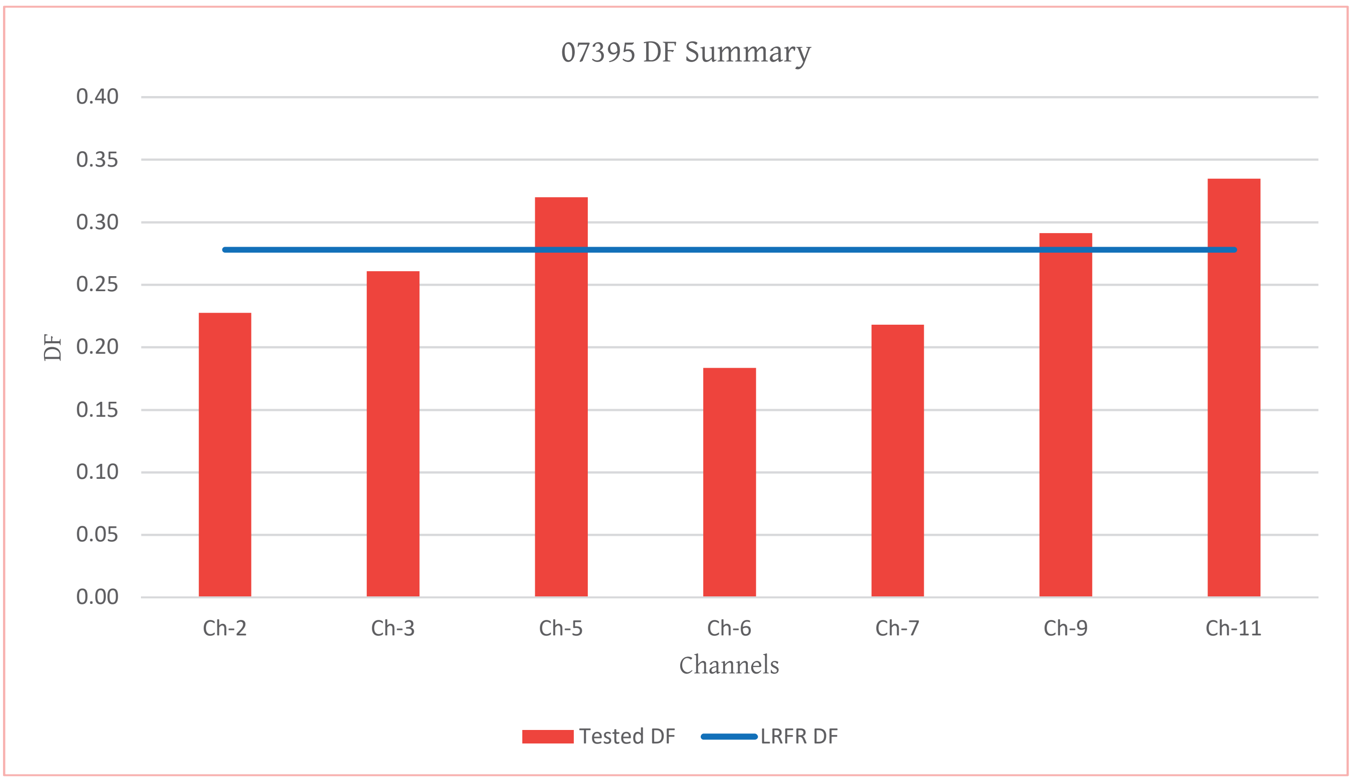


Figure 33: 07395 DF Summary

Table 20: 07395 Maximum Strain Per Case

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2	C6L1	C6L2	C7L1	C7L2
LW on Ch2	1	71	73	209	174	144	126	297	300	80	53	25	19	13	10
	2	70	73	221	165	139	123	309	290	77	50	25	19	15	11
LW on Ch3	3	27	26	65	76	230	209	127	127	219	238	75	57	45	37
	4	32	29	69	82	243	209	129	131	230	236	75	57	45	37
LW on Ch6	5	4	4	4	6	7	8	28	28	81	95	194	173	155	130
	6	2	3	3	4	6	7	26	26	81	97	175	168	153	127
LW on Ch7	7	0	0	0	1	1	2	9	10	46	55	87	131	194	186
	8	2	2	2	2	2	3	10	9	44	52	83	125	190	179
LW on Ch9	9	6	6	8	18	7	7	7	8	7	10	13	19	24	33
	10	0	0	0	3	0	0	0	0	4	7	11	17	23	33
RW on Ch10	11	7	6	4	4	5	6	9	6	3	2	2	5	6	8
	12	3	1	1	0	0	0	0	1	3	4	6	8	11	14

Table 20 Cont'd

TEST	Run	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2	C12L1	C12L2	C13L1	C13L2
LW on Ch2	1	1	0	2	2	1	1	1	0	0	0	0	1
	2	4	4	5	3	4	2	3	0	1	0	1	0
LW on Ch3	3	9	9	5	4	3	2	2	1	2	1	2	1
	4	9	9	5	4	2	2	2	1	2	1	1	1
LW on Ch6	5	302	313	92	63	31	32	7	7	4	3	2	1
	6	287	314	95	63	32	33	7	8	5	5	4	4
LW on Ch7	7	130	138	221	204	93	82	16	16	9	8	4	1
	8	125	135	213	195	89	79	13	13	6	5	0	1
LW on Ch9	9	271	265	125	126	234	291	47	48	23	26	11	11
	10	262	251	120	119	227	278	39	40	21	22	7	7
RW on Ch10	11	95	97	249	183	127	135	240	222	52	54	27	29
	12	99	107	252	186	133	141	246	224	55	55	27	27

Table 21: 07614 Bridge Summary

Asset ID	07614
Test Date	04/07/2021
Asphalt Thickness	1.5in
Year of Built	1982
ADT	2300
County	Spartanburg
Facility Carried	C-42-40
Feature Intersected	Lawsons Fork Creek
Number of Units	11
Latitude	35.03179861
Longitude	-82.08887611
Reflective Cracking	Moderate
Maximum DF	0.35
Controlling Girder	G2

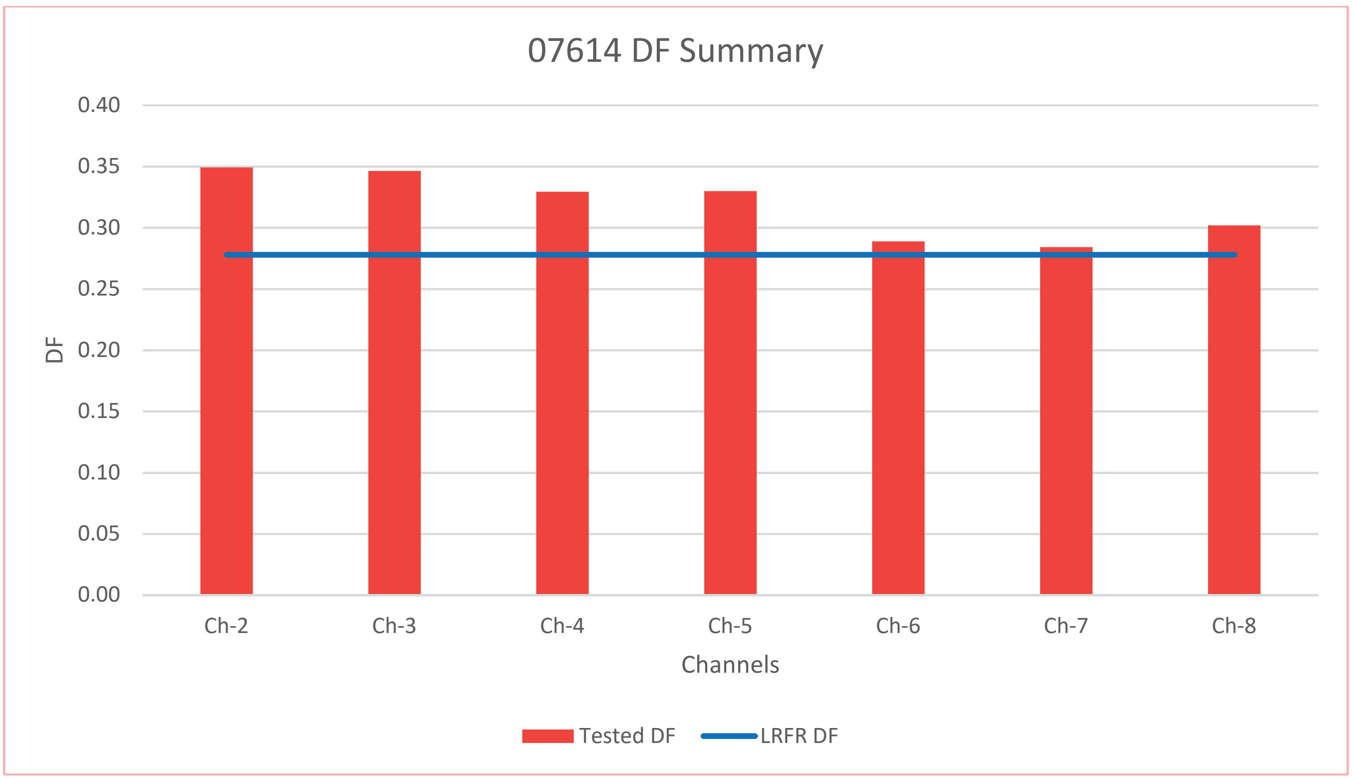


Figure 34: 07614 DF Summary

APPENDIX A

Table 22: 07614 Maximum Strain per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW at 2ft from curb	1	76	121	353	247	158	201	208	166	65	50
	2	90	151	319	229	161	225	182	146	60	47
LW on Ch2	3	38	34	316	308	132	121	236	265	90	70
	4	40	39	320	307	128	118	233	259	87	68
LW on Ch3	5	12	14	52	47	320	316	106	127	261	253
	6	11	15	55	52	307	308	115	139	245	245
LW on Ch6	7	2	3	3	4	9	7	30	32	71	72
	8	2	3	3	4	10	9	31	34	73	76
RW on Ch9	9	1	1	1	2	4	5	9	10	23	23
	10	0	0	0	0	2	3	9	10	22	21
RW on Ch10	11	2	2	2	3	3	4	4	6	10	12
	12	1	1	1	1	2	3	4	6	10	12
RW at 2ft from curb	13	1	1	1	1	1	2	2	3	5	6
	14	0	0	0	0	0	1	1	2	3	5

Table 22 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2
LW at 2ft from curb	1	22	20	8	8	5	4	1	0	0	0	0	0
	2	22	18	11	9	6	5	3	3	2	1	1	0
LW on Ch2	3	29	26	10	8	5	4	3	2	3	2	2	1
	4	27	24	10	10	6	5	3	3	3	2	2	2
LW on Ch3	5	72	63	19	19	9	9	3	5	3	3	2	3
	6	71	61	21	21	12	12	5	5	3	3	1	2
LW on Ch6	7	281	272	134	174	225	255	66	74	21	24	7	10
	8	276	273	138	179	226	262	73	78	23	25	9	10
RW on Ch9	9	55	45	250	294	160	153	293	345	42	53	14	18
	10	56	46	242	291	159	159	289	347	41	50	15	19
RW on Ch10	11	27	25	91	135	229	283	100	117	295	323	56	58
	12	27	25	91	137	216	255	104	120	283	303	49	53
RW at 2ft from curb	13	12	11	35	42	63	83	315	286	97	114	320	249
	14	11	11	34	42	70	106	289	259	105	138	296	214

Table 23: 07847 Bridge Summary

Asset ID	07847
Test Date	04/06/2021
Asphalt Thickness	1.5in
Year of Built	1984
ADT	126
County	Spartanburg
Facility Carried	C-42-431
Feature Intersected	Two Mile Creek
Number of Units	9
Latitude	34.700922
Longitude	-81.979701
Reflective Cracking	Moderate
Maximum DF	0.36
Controlling Girder	G7

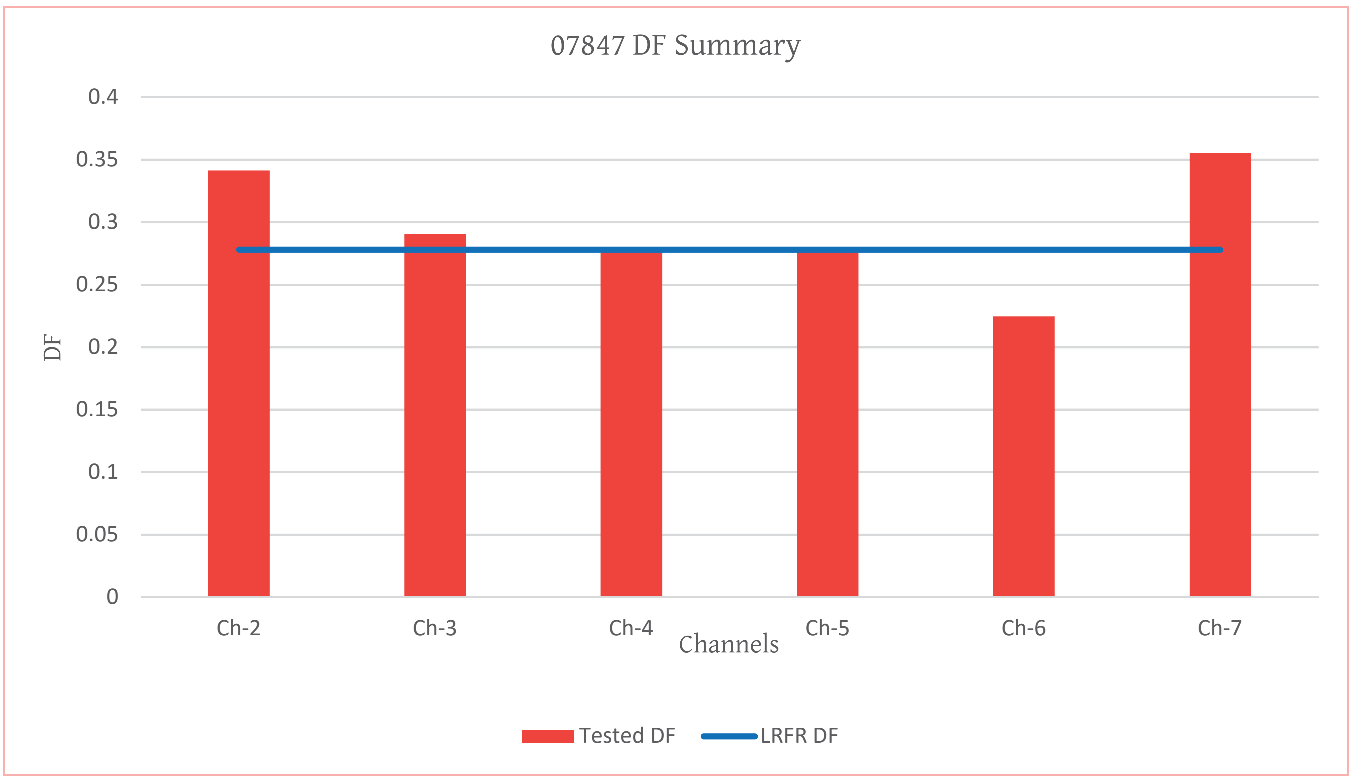


Figure 35: 07847 DF Summary

Table 24: 07847 Maximum Strain per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW 2ft from curb	1	292	346	95	90	181	200	104	70	31	24
	2	295	338	101	92	186	203	104	69	30	24
LW on Ch2	3	67	62	282	302	114	110	201	198	82	56
	4	78	77	300	300	122	119	212	196	84	57
LW on Ch3	5	15	14	64	61	251	245	151	148	214	204
	6	10	13	57	59	237	239	148	145	201	196
LW on Ch5	7	0	1	8	8	31	32	70	84	196	230
	8	0	0	4	6	29	31	65	81	185	227
RW on Ch6	9	5	5	6	7	16	19	32	37	77	107
	10	1	0	0	2	13	15	29	33	76	111
RW 2ft from curb	11	1	3	4	4	8	13	18	24	45	69
	12	3	3	2	3	7	11	15	22	46	69

Table 24 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2
LW 2ft from curb	1	18	13	7	8	0	0	0	4
	2	19	14	9	9	0	3	3	4
LW on Ch2	3	42	30	21	20	2	4	0	2
	4	45	31	23	20	4	4	1	1
LW on Ch3	5	109	72	55	49	11	11	2	2
	6	110	74	55	50	11	10	0	3
LW on Ch5	7	180	193	238	288	50	46	15	14
	8	177	186	230	295	52	46	14	13
RW on Ch6	9	152	196	164	139	297	308	86	87
	10	155	185	153	132	299	291	69	73
RW 2ft from curb	11	88	156	226	199	68	62	364	288
	12	88	156	230	203	82	73	353	281

Table 25: 06984 Bridge Summary

Asset ID	06984
Test Date	04/15/2021
Asphalt Thickness	1in
Year of Built	1977
ADT	126
County	Greenwood
Facility Carried	C-29-904
Feature Intersected	Reddy Creek
Number of Units	9
Latitude	34.00481
Longitude	-82.111892
Reflective Cracking	Heavy/Severe
Maximum DF	0.45
Controlling Girder	4

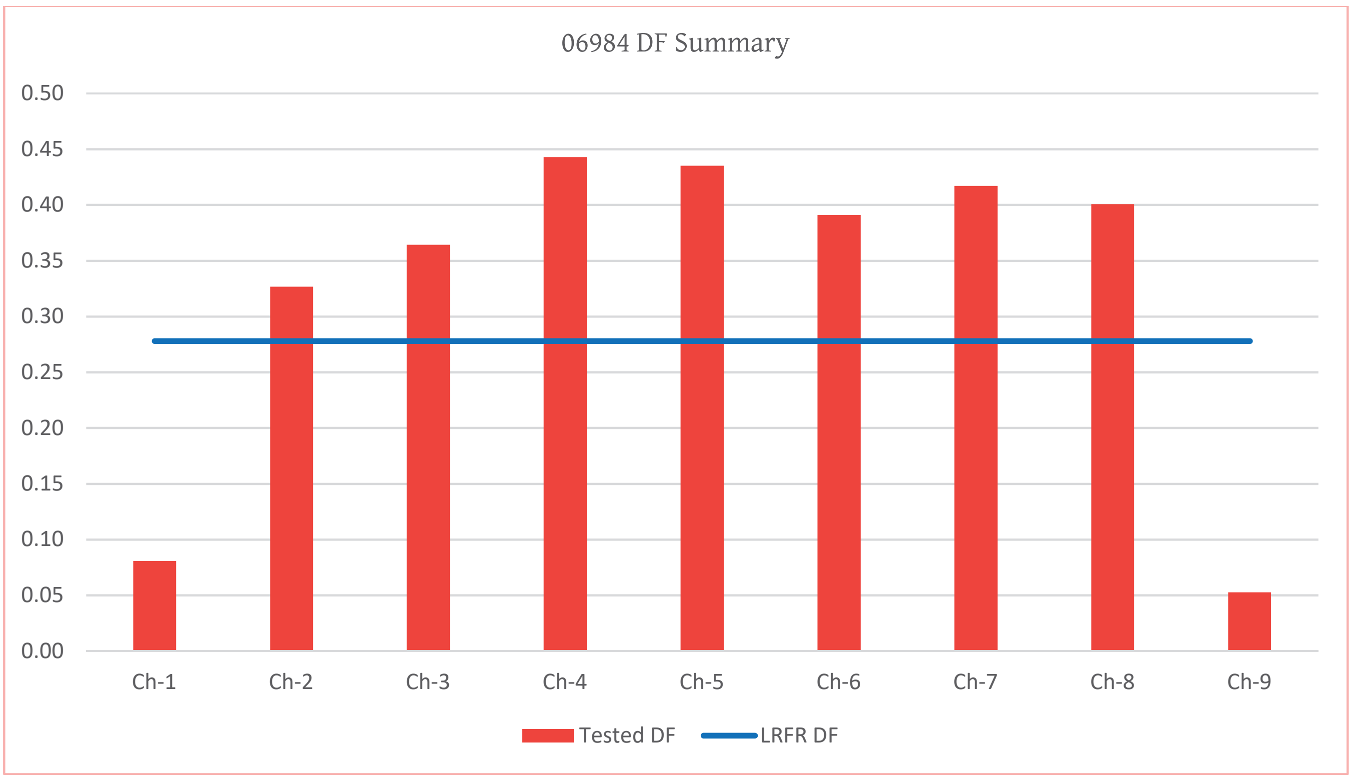


Figure 36: 06984 DF Summary

Table 26: 06984 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW on Ch2	83	96	298	343	91	102	355	372	48	55	83
	78	90	297	358	83	94	353	387	44	50	78
LW on Ch3	13	13	48	45	351	386	75	80	338	376	13
	14	14	49	47	320	372	94	105	321	373	14
LW on Ch5	1	1	3	4	12	13	54	62	343	409	1
	2	3	4	4	14	16	53	61	336	385	2
LW on Ch6	3	3	3	2	5	4	17	22	38	38	3
	2	2	2	1	5	5	16	20	43	48	2

Table 26 Cont'd

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW on Ch2	18	19	7	6	5	4	3	4	18	19	7
	16	17	11	8	2	1	0	1	16	17	11
LW on Ch3	49	53	18	16	7	5	2	1	49	53	18
	46	49	15	14	6	4	2	3	46	49	15
LW on Ch5	91	101	337	359	49	49	5	7	91	101	337
	100	112	320	335	52	50	5	8	100	112	320
LW on Ch6	379	406	100	100	361	318	46	52	379	406	100
	357	408	115	116	335	318	47	53	357	408	115

Table 27: 07374 Bridge Summary

Asset ID	07374
Test Date	04/14/2021
Asphalt Thickness	4.5in
Year of Built	1980
ADT	2500
County	Greenwood
Facility Carried	S-24-39
Feature Intersected	Rocky Creek
Number of Units	14
Latitude	34.2363425
Longitude	-82.13709194
Reflective Cracking	Heavy/Severe
Maximum DF	0.45
Controlling Girder	5

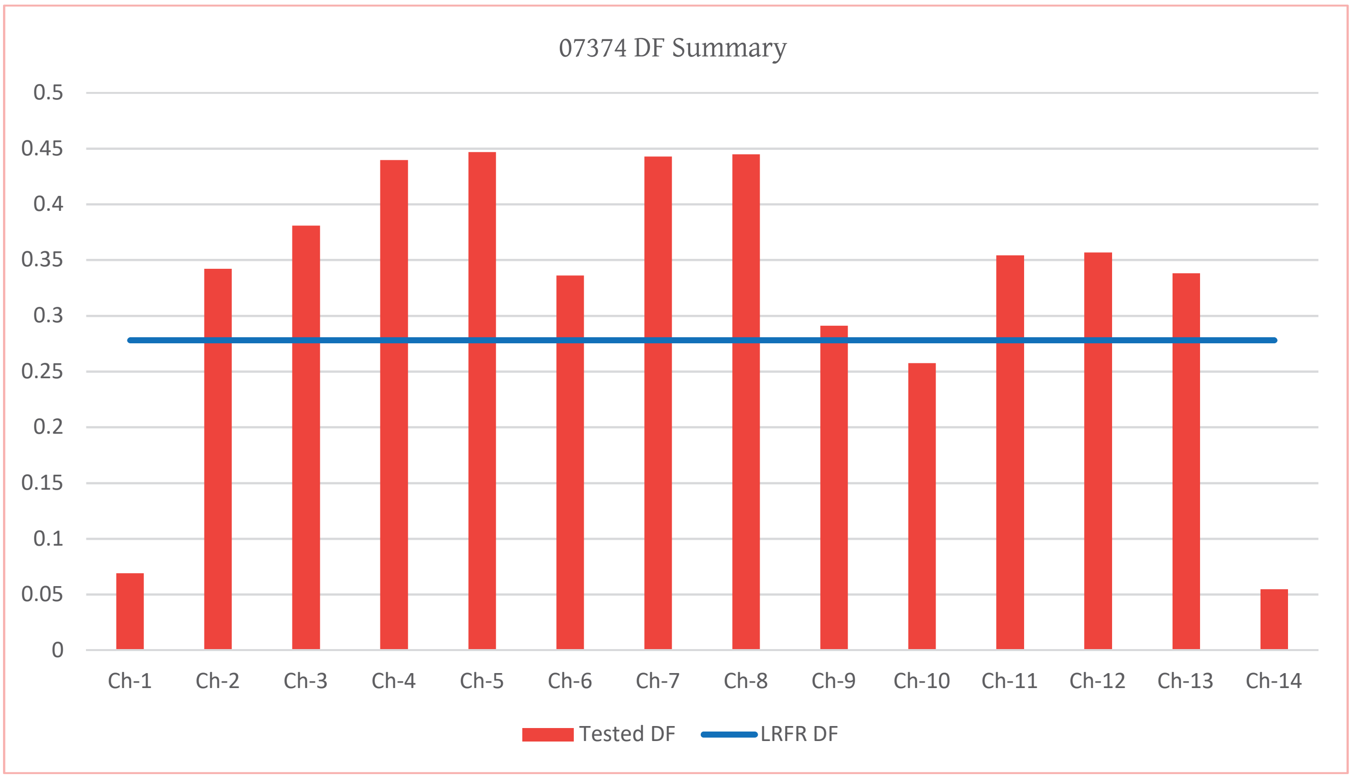


Figure 37: 07374 DF Summary

Table 28: 07374 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2	C6L1	C6L2	C7L1	C7L2
LW at 2ft from curb	72	70	304	291	70	73	338	300	36	35	19	14	6	4	72
	63	63	299	277	50	59	336	291	34	33	17	13	7	3	63
LW on Ch2	10	8	26	23	337	285	84	78	320	288	22	20	3	4	10
	10	11	32	29	329	300	93	89	327	318	36	31	12	12	10
LW on Ch3	1	1	1	1	2	2	18	15	69	68	263	265	63	64	1
	0	0	0	0	3	1	14	10	61	58	245	258	50	52	0
LW on Ch5	1	0	0	0	0	1	3	2	11	12	30	33	308	240	1
	3	4	4	5	4	5	5	7	15	16	32	38	292	259	3
LW on Ch6	3	4	4	3	4	4	5	4	4	4	4	4	5	3	3
	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
RW at 2ft from curb	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0
	1	1	1	1	1	1	1	1	1	1	1	2	3	2	1

Table 28 Cont'd

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2	C6L1	C6L2	C7L1	C7L2
LW at 2ft from curb	3	3	3	3	4	4	4	3	2	2	2	1	2	0	3
	1	1	1	1	1	1	1	1	1	2	2	2	2	0	1
LW on Ch2	1	0	2	1	1	1	1	0	1	1	1	1	1	0	1
	5	3	3	3	3	2	2	3	3	3	2	3	2	0	5
LW on Ch3	317	276	35	29	23	17	6	4	2	1	1	0	0	0	317
	289	266	29	24	18	13	2	1	0	0	0	0	0	0	289
LW on Ch5	98	93	213	150	110	76	16	15	3	3	2	2	2	1	98
	102	97	206	163	125	85	23	22	8	8	6	5	2	1	102
LW on Ch6	27	24	83	115	177	213	86	85	227	226	62	60	10	15	27
	28	30	89	122	180	216	92	88	233	224	75	65	9	15	28
RW at 2ft from curb	8	7	25	26	45	54	294	275	126	125	222	242	36	50	8
	9	8	25	27	43	53	283	263	126	130	213	222	30	46	9

Table 29: 07984 Bridge Summary

Asset ID	07984
Test Date	05/05/2021
Asphalt Thickness	3in
Year of Built	1986
ADT	350
County	Cherokee
Facility Carried	S-286
Feature Intersected	Kings Creek
Number of Units	11
Latitude	35.087162
Longitude	-81.420776
Reflective Cracking	Heavy/Severe
Maximum DF	0.50
Controlling Girder	9

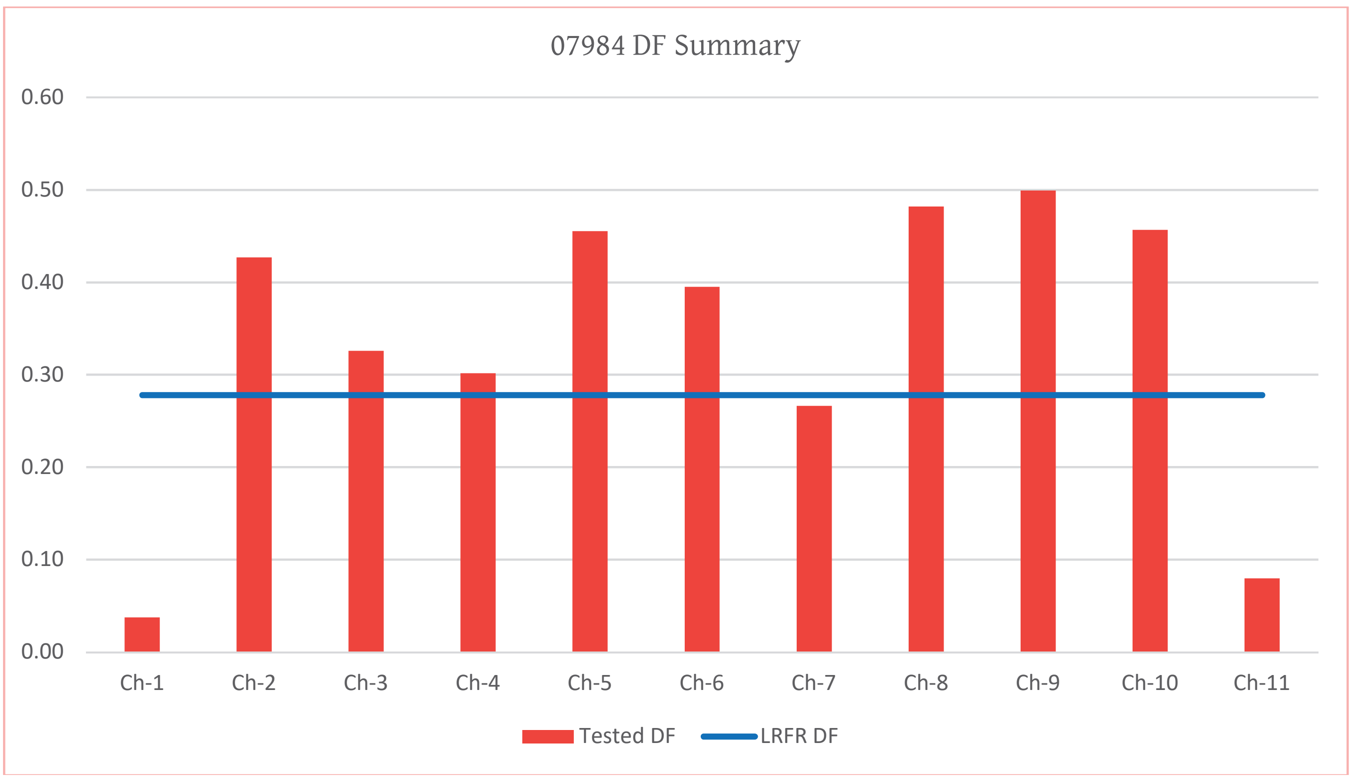


Figure 38: 07984 DF Summary

Table 30: 07984 Maximum Strain Per Test

TEST	Run	C1L1	C1L2	C2L1	C2L2	C3L1	C3L2	C4L1	C4L2	C5L1	C5L2
LW on Ch2	44	30	377	331	88	93	185	240	84	80	44
	47	27	373	340	85	91	179	244	84	80	47
LW on Ch3	16	12	49	42	313	259	98	138	337	338	16
	14	9	51	46	316	232	96	137	334	303	14
LW on Ch6	6	5	4	4	3	3	5	5	19	19	6
	0	3	2	4	3	4	5	8	24	23	0
LW on Ch7	0	1	1	2	2	2	2	4	7	7	0
	0	0	0	0	0	0	1	2	6	5	0
LW on Ch8	2	1	0	0	0	0	0	0	0	0	2
	10	7	3	3	1	2	0	0	0	0	10

Table 30 Cont'd

TEST	Run	C6L1	C6L2	C7L1	C7L2	C8L1	C8L2	C9L1	C9L2	C10L1	C10L2	C11L1	C11L2
LW on Ch2	1	6	12	0	0	0	0	0	0	0	0	0	0
	2	8	17	2	2	1	1	3	1	2	1	0	3
LW on Ch3	3	23	31	3	4	1	0	0	0	1	2	1	2
	4	22	28	3	2	1	0	1	0	0	0	0	1
LW on Ch6	5	379	364	59	59	381	370	29	28	13	9	2	5
	6	388	343	61	60	395	348	31	29	13	9	1	0
LW on Ch7	7	95	113	257	226	87	86	395	372	40	34	3	5
	8	90	107	250	222	100	96	371	365	38	35	4	6
LW on Ch8	9	1	4	25	26	336	318	89	93	347	333	53	71
	10	3	5	31	32	329	295	105	116	340	303	51	68

APPENDIX

B K FACTOR RESULTS

SAMPLE: K- FACTOR CALCULATION

Example: 07153

Maximum distribution factor = 0.5 Lane (heavy reflective cracks)

Elastic modulus of concrete = 4074.28 ksi (5 ksi concrete)

Maximum mid-span moment induced by the load test truck = 258.900 kip-ft (Quick Bridge)

Maximum theoretical deflection at mid-span from test truck = 0.812 in.

Ka factor according to AASHTO MBE 3rd 8.8.2.3.1-2:

$k_a = \delta_c/\delta_t - 1$

Where δ_c is the theoretical deflection and δ_t is the measured deflection.

K_b factor is obtained per AASHTO MBE 3rd 8.8.2.3.1-1. Assuming a EV3 truck, maximum unfactoed live load moment = 380.6 kip-ft. We cannot predict that member behavior can be extrapolated to 1.33 W for all rating trucks. The unfactored moment ratio between EV3 and the test truck is 280.946/380.6 = 0.74 > 0.7, so we use k_b = 0.5. Then, k factor can be obtained based on AASHTO MBE 3rd 8.8.2.3.1-1

$k = 1 + k_a k_b$

The displacement sensor for 07153 were installed at all interior beams (G2-G10). Then, the calculated k_a, k_b and k values for each beam are as followings:

	G2	G3	G4	G5	G6	G7	G8	G9	G10
Measured Max Deflection (in)	0.279	0.227	0.318	0.222	0.299	0.286	0.218	0.247	0.250
ka	1.908	2.575	1.552	2.663	1.719	1.841	2.723	2.291	2.243
kb	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
k-factor	1.954	2.288	1.776	2.332	1.859	1.920	2.361	2.146	2.122

Table 8.8.2.3.1-1—Values for K_b

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

SAMPLE: THEORETICAL DEFLECTION CALCULATION

Example: 07153

Theoretical Deflection at Mid-Span Due to Loaded Dump Truck (BR 07153)

Bridge Span:

$L := 29.021ft$ CL-CL of bearing

Section Properties:

$b := 3ft + 0.5in = 3.042 \cdot ft$ $h_f := 5in$ $w_G := 0.355 \frac{kip}{ft}$

$b_w := 6.5in + 4.5in = 11 \cdot in$ *(Width of two legs. Average of the tapered leg width was used)*

$y_b := 12.3346in$

$I_g := 9793.722in^4$ $S_b := \frac{I_g}{y_b} = 794.004 \cdot in^3$

Concrete Material Properties:

$f'_c := 5ksi$

$E_c := 4074.28ksi$ $E_{ci} := 3644.15ksi$

Prestressing Strand Properties: **12 3/8"-dia. SR Strands**

$E_{ps} := 28500ksi$

$N := 12$ Number of strands

$A_{ps} := 0.08in^2$ Area of one 3/8" SR strands

$F_i := 14kip$ Jacking force

$f_{loss\_total} := 41.0183ksi$ Estimated total prestress loss (BrR output, LFD)

$f_{pe} := \frac{F_i}{A_{ps}} - f_{loss\_total} = 133.982 \cdot ksi$ Effective stress of each strand after all losses

$F_{pe} := f_{pe} \cdot A_{ps} = 10.719 \cdot kip$ Effective force of each strand after all losses

Prestressing Strand Locations:

Prestressing Strand Layer	Center of Gravity	Eccentricity	Distance from support to the draping point
Layer 1 (straight)	$CG_{1\_mid} := 2in$ $CG_{1\_end} := 2in$	$e_{1\_mid} := y_b - CG_{1\_mid} = 10.335 \cdot in$ $e_{1\_end} := y_b - CG_{1\_end} = 10.335 \cdot in$	$a_1 := 0ft$
Layer 2 (draped)	$CG_{2\_mid} := 4in$ $CG_{2\_end} := 11in$	$e_{2\_mid} := y_b - CG_{2\_mid} = 8.335 \cdot in$ $e_{2\_end} := y_b - CG_{2\_end} = 1.335 \cdot in$	$a_2 := \frac{L}{2} = 14.511 \cdot ft$
Layer 3 (draped)	$CG_{3\_mid} := 6in$ $CG_{3\_end} := 13in$	$e_{3\_mid} := y_b - CG_{3\_mid} = 6.335 \cdot in$ $e_{3\_end} := y_b - CG_{3\_end} = -0.665 \cdot in$	$a_3 := a_2 = 14.511 \cdot ft$

Input:

$t_{asphalt} := 2in$

$LLDFs := 0.5$ (Lane, Per Bucket Truck Load Test)

$M_{cr\_BrR\_LFD} := 97.96 kip \cdot ft$ (Per BrR LFD Output,due to applied live load only)

$d_p := 19in - 4in = 15 \cdot in$

Moment Calculations (at mid-span):

1) Moment due to beam selfweight

$$M_G := \frac{1}{8} \cdot w_G \cdot L^2 = 37.373 \cdot kip \cdot ft$$

2) Moment due to wearing surface

$$w_{asphalt} := 0.14 \frac{kip}{ft^3} \cdot t_{asphalt} \cdot (3ft + 0.5in) = 0.071 \cdot \frac{kip}{ft}$$

$$M_{asphalt} := \frac{1}{8} \cdot w_{asphalt} \cdot L^2 = 7.472 \cdot kip \cdot ft$$

3) Maximum Moment due to loaded dump truck:

Axle Load and Spacings:

$P_1 := 16.26kip$ $P_2 := 23.76kip$ $P_3 := 23.08kip$

$S_1 := 15.25ft$ $S_2 := 4.67ft$

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

P.2 is 1.184 ft from mid-span (Left), P.3 is 3.486 ft from mid-span (Right)

$$x_1 := \frac{1}{2} \cdot \frac{P_2 \cdot S_2}{P_2 + P_3} = 1.184 \cdot ft$$

$$x_2 := S_2 - x_1 = 3.486 \cdot ft$$

$$M_{truck} := \frac{P_2}{2} \cdot \left(\frac{L}{2} - x_1\right) + \frac{P_3}{2} \cdot \left(\frac{L}{2} - x_2\right) = 285.541 \cdot kip \cdot ft$$

Check by Quick Bridge:

$M_{truck} := 285.9 kip \cdot ft$ Maximum moment at mid span (Quick Bridge)

Maximum Moment at Mid-Span with corresponding LLDFs:

$M_{truck.channel} := M_{truck} \cdot LLDFs = 142.95 \cdot kip \cdot ft$

Total External Applied Moment at Mid-Span:

$M_a := M_G + M_{asphalt} + M_{truck.channel} = 187.795 \cdot kip \cdot ft$

Effective Moment of Inertia (at mid-span):

1) Cracking moment of inertia:

$$\rho_p := \frac{A_{ps} \cdot N}{b \cdot d_p} = 1.753 \times 10^{-3} \qquad n_p := \frac{E_{ps}}{E_c} = 6.995$$

$$I_{cr} := n_p \cdot A_{ps} \cdot N \cdot d_p^{\,2} \cdot \left(1 - 1.6 \cdot \sqrt{n_p \cdot \rho_p}\right) = 1.243 \times 10^3 \cdot in^4$$

Approximate Method Per PCI Eq. 5-78

2) Cracking moment:

$$M_{cr} := M_{cr\_BrR\_LFD} + M_G + M_{asphalt} = 142.805 \cdot kip \cdot ft$$

3) Effective Moment of Inertia:

$$I_e := \left(\frac{M_{cr}}{M_a}\right)^3 \cdot I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \cdot I_{cr} = 5003.059 \cdot in^4 \qquad \frac{I_e}{I_g} = 0.511$$

Instantaneous Deflection Due to The Effect of Full External Load at Mid-span:

1) Deflection due to effective prestress force after all losses (I.e, F.pe, E.c):

$$\Delta_{i\_F1} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_e} \cdot \left[e_{1\_mid} + \left(e_{1\_end} - e_{1\_mid}\right) \cdot \frac{4 \cdot a_1^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F1} = -0.33 \cdot in$$

$$\Delta_{i\_F2} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_e} \cdot \left[e_{2\_mid} + \left(e_{2\_end} - e_{2\_mid}\right) \cdot \frac{4 \cdot a_2^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F2} = -0.191 \cdot in$$

$$\Delta_{i\_F3} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_e} \cdot \left[e_{3\_mid} + \left(e_{3\_end} - e_{3\_mid}\right) \cdot \frac{4 \cdot a_3^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F3} = -0.128 \cdot in$$

$$\Delta_{i\_F} := \Delta_{i\_F1} + \Delta_{i\_F2} + \Delta_{i\_F3} = -0.648 \cdot in$$

(camber)

2) Deflection due to beam selfweight (I.e, E.c):

$$\Delta_{i\_G} := \frac{5 \cdot w_G \cdot L^4}{384 \cdot E_c \cdot I_e} = 0.278 \cdot in$$

(downward)

3) Deflection due to wearing surface t (I.e, E.c):

$$\Delta_{i\_AWS} := \frac{5 \cdot w_{asphalt} \cdot L^4}{384 \cdot E_c \cdot I_e} = 0.056 \cdot in$$

(downward)

4) Deflection due to live load (I.e, E.c):

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

Deflection at Mid-Span Due to P2:

$$a_{p2} := \frac{L}{2} - x_1 = 13.326 \cdot ft \qquad b_{p2} := L - a_{p2} = 15.695 \cdot ft$$

$$\Delta_{LL}\left(\frac{L}{2}, P_2 \cdot LLDFs, a_{p2}, b_{p2}\right) = 0.508 \cdot in$$

Deflection at Mid-Span Due to P3:

$$a_{p3} := \frac{L}{2} + x_2 = 17.996 \cdot ft \qquad b_{p3} := L - a_{p3} = 11.025 \cdot ft$$

$$\Delta_{LL}\left(\frac{L}{2}, P_3 \cdot LLDFs, a_{p3}, b_{p3}\right) = 0.458 \cdot in$$

$$\Delta_{i\_LL} := \Delta_{LL}\left(\frac{L}{2}, P_2 \cdot LLDFs, a_{p2}, b_{p2}\right) + \Delta_{LL}\left(\frac{L}{2}, P_3 \cdot LLDFs, a_{p3}, b_{p3}\right) = 0.966 \cdot in$$

$$\Delta_{i\_1} := \Delta_{i\_F} + \Delta_{i\_G} + \Delta_{i\_AWS} + \Delta_{i\_LL} = 0.651 \cdot in$$

Deflection (fictitious) at Mid-Span under Sustained Load and Prestress:

1) Deflection due to effective prestress force after all losses (I.g, F.pe, E.c):

$$\Delta_{i\_F1} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_g} \cdot \left[e_{1\_mid} + \left(e_{1\_end} - e_{1\_mid}\right) \cdot \frac{4 \cdot a_1^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F1} = -0.168 \cdot in$$

$$\Delta_{i\_F2} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_g} \cdot \left[e_{2\_mid} + \left(e_{2\_end} - e_{2\_mid}\right) \cdot \frac{4 \cdot a_2^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F2} = -0.098 \cdot in$$

$$\Delta_{i\_F3} := -\frac{F_{pe} \cdot 4 \cdot L^2}{8 \cdot E_c \cdot I_g} \cdot \left[e_{3\_mid} + \left(e_{3\_end} - e_{3\_mid}\right) \cdot \frac{4 \cdot a_3^{\,2}}{3 \cdot L^2}\right] \qquad \Delta_{i\_F3} = -0.065 \cdot in$$

$$\Delta_{i\_F} := \Delta_{i\_F1} + \Delta_{i\_F2} + \Delta_{i\_F3} = -0.331 \cdot in$$

(camber)

2) Deflection due to beam selfweight (Ig, Ec):

$$\Delta_{i\_G} := \frac{5 \cdot w_G \cdot L^4}{384 \cdot E_c \cdot I_g} = 0.142 \cdot in$$

(downward)

3) Deflection due to wearing surface t (Ig, Ec):

$$\Delta_{i\_AWS} := \frac{5 \cdot w_{asphalt} \cdot L^4}{384 \cdot E_c \cdot I_g} = 0.028 \cdot in$$

(downward)

$$\Delta_{i\_2} := \Delta_{i\_F} + \Delta_{i\_G} + \Delta_{i\_AWS} = -0.161 \cdot in$$

Instantaneous Live Load Deflection due to Load Test Truck with DF=0.5 at Mid-Span:

$$\boxed{\Delta_{i\_truck} := \Delta_{i\_1} - \Delta_{i\_2} = 0.812 \cdot in}$$

The followings show the measured displacement and calculated k-factor for each tested wide-leg channel bridges.

Table 31: 03365 Load Test Summary

Asset ID	03365
Test Date	4/28/2021
Year of Built	1973
ADT	120
County	Laurens
Facility Carried	S-30-308
Number of Units	11
Minimum k-factor	1.545
Controlling Girder	G8

Table 32: 03365 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.264	-0.060	-0.298	-0.090	-0.015	-0.001	0.000
	2	-0.259	-0.056	-0.302	-0.091	-0.015	-0.001	-0.001
	3	-0.275	-0.054	-0.308	-0.094	-0.015	-0.001	-0.001
LW on Ch-3	1	-0.026	-0.309	-0.070	-0.346	-0.068	-0.005	-0.002
	2	-0.022	-0.306	-0.072	-0.343	-0.069	-0.006	-0.002
	3	-0.022	-0.309	-0.074	-0.349	-0.072	-0.005	-0.003
LW on Ch-5	1	-0.001	-0.002	-0.021	-0.325	-0.132	-0.232	-0.072
	2	0.000	-0.002	-0.021	-0.327	-0.133	-0.250	-0.068
	3	-0.001	-0.002	-0.024	-0.335	-0.128	-0.250	-0.071
LW on Ch-6	1	0.000	-0.001	-0.004	-0.016	-0.338	-0.107	-0.380
	2	0.000	-0.001	-0.003	-0.018	-0.343	-0.108	-0.396
	3	0.000	0.000	-0.003	-0.019	-0.346	-0.109	-0.407

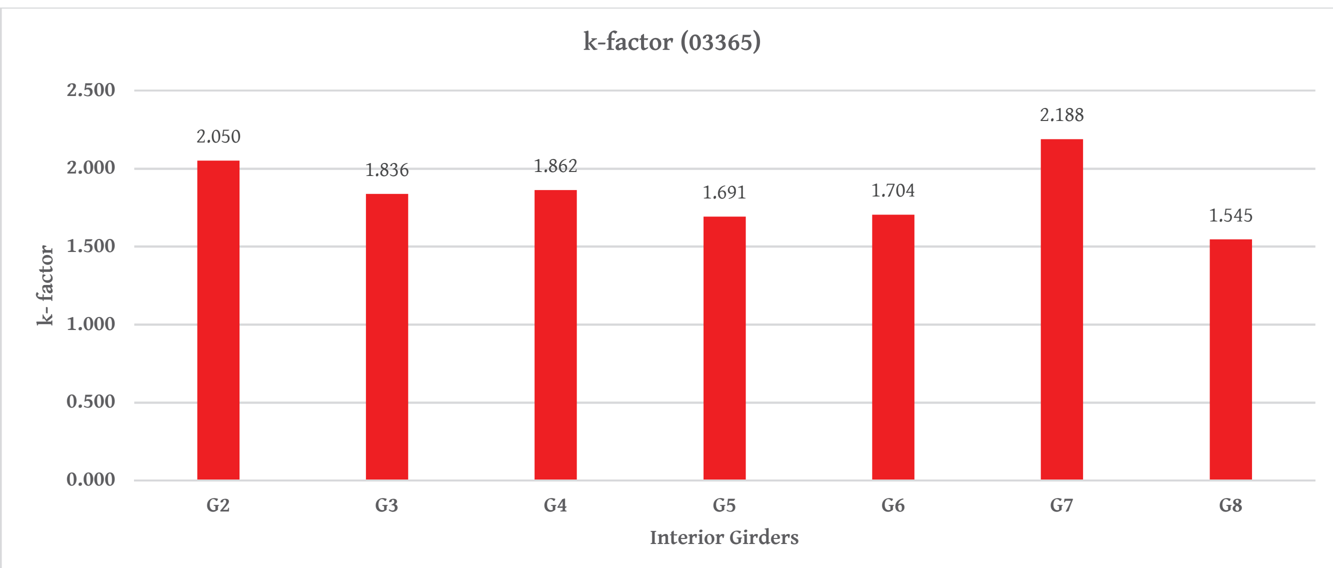


Figure 39: k-factor of 03365

Table 33: 05335 Load Test Summary

Asset ID	05335
Test Date	6/2/2021
Year of Built	1968
ADT	2400
County	Laurens
Facility Carried	S-30-359
Number of Units	9
Minimum k-factor	1.599
Controlling Girder	G3

Table 34: 05335 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.297	-0.161	-0.257	-0.055	-0.020	-0.014	-0.001
	2	-0.306	-0.164	-0.245	-0.055	-0.016	-0.013	0.000
	3	-0.290	-0.163	-0.254	-0.064	-0.020	-0.014	-0.001
LW on Ch-3	1	-0.065	-0.331	-0.114	-0.253	-0.065	-0.039	-0.009
	2	-0.068	-0.331	-0.115	-0.257	-0.066	-0.039	-0.008
	3	-0.069	-0.336	-0.113	-0.262	-0.064	-0.038	-0.008
LW on Ch-5	1	-0.010	-0.021	-0.063	-0.295	-0.150	-0.197	-0.081
	2	-0.007	-0.026	-0.073	-0.280	-0.152	-0.193	-0.089
	3	-0.007	-0.026	-0.075	-0.285	-0.153	-0.194	-0.089
LW on Ch-6	1	0.000	-0.005	-0.027	-0.049	-0.236	-0.208	-0.240
	2	0.000	-0.006	-0.024	-0.055	-0.232	-0.211	-0.231
	3	0.000	-0.006	-0.024	-0.055	-0.236	-0.212	-0.231

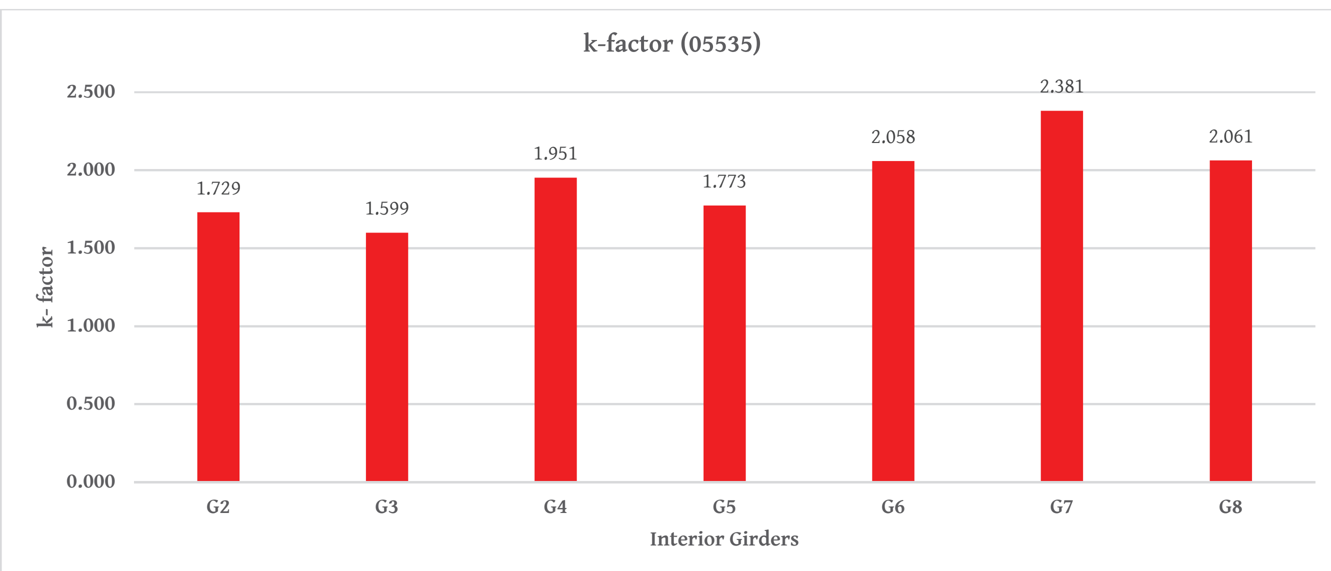


Figure 40: k-factor of 05335

Table 35: 06028 Load Test Summary

Asset ID	06028
Test Date	4/28/2021
Year of Built	1972
ADT	290
County	Abbeville
Facility Carried	S-1-72
Number of Units	11
Minimum k-factor	1.505
Controlling Girder	G10

Table 36: 06028 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.240	-0.153	-0.199	-0.147	-0.062	-0.017	-0.002	-	-
	2	-0.251	-0.154	-0.202	-0.140	-0.057	-0.011	-0.001	-	-
	3	-0.260	-0.150	-0.202	-0.142	-0.057	-0.011	-0.001	-	-
LW on Ch-3	1	-0.065	-0.232	-0.217	-0.236	-0.153	-0.023	-0.005	-	-
	2	-0.064	-0.224	-0.217	-0.237	-0.162	-0.025	-0.005	-	-
	3	-0.065	-0.238	-0.216	-0.243	-0.149	-0.024	-0.006	-	-
LW on Ch-5	1	-0.015	-0.033	-0.125	-0.203	-0.198	-0.223	-0.102	-	-
	2	-0.015	-0.033	-0.125	-0.205	-0.198	-0.220	-0.096	-	-
	3	-0.013	-0.031	-0.119	-0.204	-0.205	-0.233	-0.107	-	-
LW on Ch-6	1	-0.006	-0.013	-0.048	-0.110	-0.233	-0.168	-0.242	-	-
	2	-0.005	-0.011	-0.044	-0.101	-0.241	-0.178	-0.235	-	-
	3	-0.005	-0.011	-0.046	-0.106	-0.245	-0.185	-0.241	-	-
RW on Ch-9	1	-	-	-	-	-	-	-	-0.296	-0.083
	2	-	-	-	-	-	-	-	-0.303	-0.079
	3	-	-	-	-	-	-	-	-0.312	-0.077
RW on Ch-10	1	-	-	-	-	-	-	-	-0.137	-0.375
	2	-	-	-	-	-	-	-	-0.139	-0.376
	3	-	-	-	-	-	-	-	-0.141	-0.393

Figure 41: k-factor of 06028

Table 37: 06637 Load Test Summary

Asset ID	06637
Test Date	6/3/2021
Year of Built	1975
ADT	200
County	Chesterfield
Facility Carried	S-13-30
Number of Units	11
Minimum k-factor	1.328
Controlling Girder	G4

Table 38: 06637 Maximum Displacement Per Test

Test	Run	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9
RW on Ch-5	1	-0.211	-0.078	-0.322	-0.026	-0.001	0.000	-0.001
	2	-0.214	-0.085	-0.328	-0.029	-0.002	0.000	-0.001
	3	-0.212	-0.079	-0.324	-0.025	-0.001	0.000	-0.001
RW on Ch-6	1	-0.016	-0.444	-0.040	-0.403	-0.032	-0.005	-0.001
	2	-0.024	-0.439	-0.041	-0.392	-0.030	-0.004	-0.001
	3	-0.025	-0.440	-0.042	-0.400	-0.033	-0.005	-0.002
RW on Ch-8	1	-0.001	-0.016	-0.009	-0.416	-0.063	-0.277	-0.023
	2	-0.001	-0.016	-0.024	-0.413	-0.072	-0.286	-0.024
	3	-0.001	-0.018	-0.025	-0.429	-0.074	-0.294	-0.024
RW on Ch-9	1	-0.001	-0.001	-0.001	-0.020	-0.321	-0.060	-0.191
	2	-0.001	-0.001	0.000	-0.022	-0.299	-0.068	-0.193
	3	0.000	-0.001	-0.001	-0.027	-0.318	-0.068	-0.204

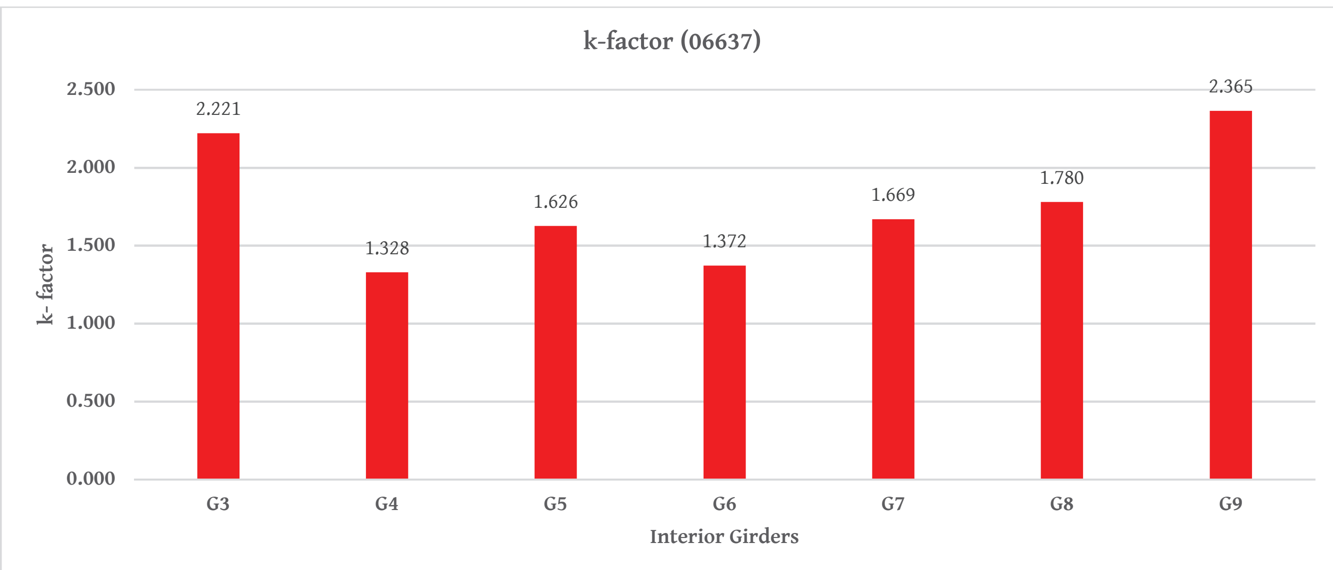


Figure 42: k-factor of 06637

Table 39: 06792 Load Test Summary

Asset ID	06792
Test Date	6/2/2021
Year of Built	1975
ADT	275
County	Union
Facility Carried	S-44-19
Number of Units	11
Minimum k-factor	1.393
Controlling Girder	G6

Table 40: 06792 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.287	-0.142	-0.163	-0.097	-0.016	0.000	-0.001	-	-
	2	-0.306	-0.144	-0.166	-0.086	-0.016	0.000	-0.001	-	-
	3	-0.315	-0.146	-0.162	-0.080	-0.014	0.000	-0.001	-	-
LW on Ch-3	1	-0.095	-0.199	-0.168	-0.200	-0.076	-0.008	-0.004	-	-
	2	-0.092	-0.195	-0.170	-0.203	-0.078	-0.008	-0.005	-	-
	3	-0.095	-0.196	-0.170	-0.204	-0.077	-0.008	-0.005	-	-
LW on Ch-6	1	-0.015	-0.033	-0.016	-0.047	-0.408	-0.131	-0.253	-0.048	-0.009
	2	-0.015	-0.033	-0.015	-0.045	-0.407	-0.133	-0.255	-0.049	-0.010
	3	-0.013	-0.031	-0.016	-0.047	-0.413	-0.133	-0.256	-0.048	-0.010
LW on Ch-7	1	-0.006	-0.013	-0.003	-0.010	-0.052	-0.300	-0.156	-0.198	-0.052
	2	-0.005	-0.011	-0.003	-0.011	-0.050	-0.295	-0.161	-0.191	-0.050
	3	-0.005	-0.011	-0.003	-0.011	-0.050	-0.300	-0.163	-0.192	-0.051
RW on Ch-9	1	-	-	-0.001	-0.001	-0.004	-0.091	-0.267	-0.087	-0.226
	2	-	-	-0.001	-0.002	-0.008	-0.091	-0.258	-0.087	-0.218
	3	-	-	-0.001	-0.002	-0.009	-0.095	-0.263	-0.086	-0.235

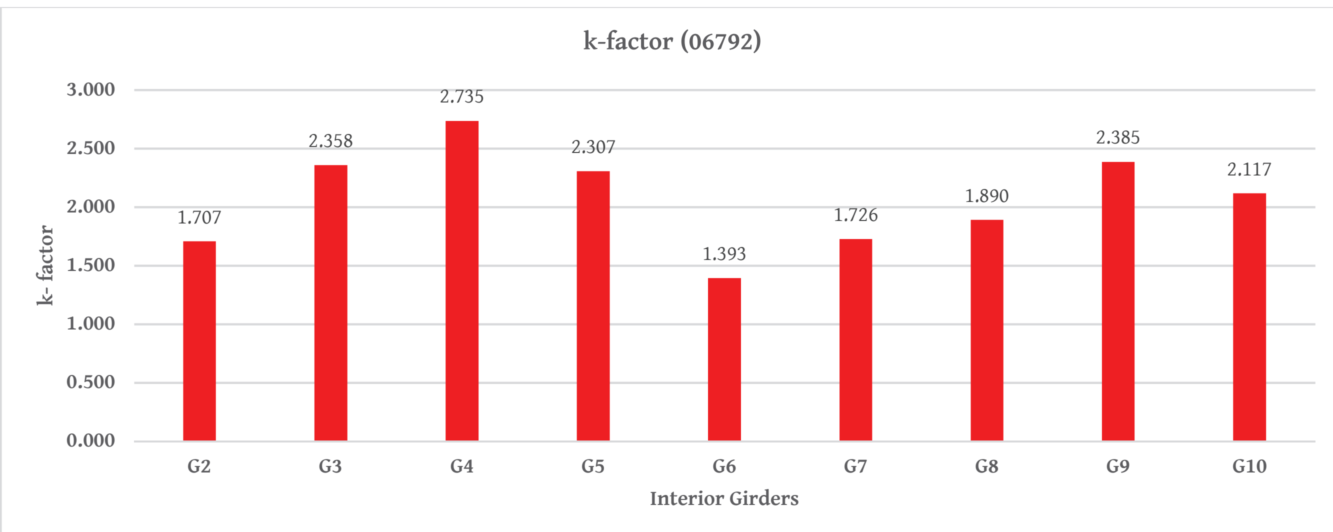


Figure 43: k-factor of 06792

Table 41: 06984 Load Test Summary

Asset ID	06984
Test Date	4/15/2021
Year of Built	1977
ADT	200
County	Greenwood
Facility Carried	C-24-904
Number of Units	9
Minimum k-factor	1.444
Controlling Girder	G6

Table 42: 06984 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.209	-0.110	-0.418	-0.069	-0.021	-0.002	0.000
	2	-0.212	-0.103	-0.418	-0.069	-0.018	-0.002	0.000
	3	-0.215	-0.113	-0.451	-0.073	-0.022	-0.002	0.000
LW on Ch-3	1	-0.052	-0.265	-0.094	-0.351	-0.076	-0.022	-0.002
	2	-0.064	-0.271	-0.084	-0.370	-0.072	-0.019	-0.003
	3	-0.054	-0.277	-0.090	-0.386	-0.086	-0.025	-0.004
LW on Ch-5	1	-0.005	-0.020	-0.028	-0.406	-0.117	-0.330	-0.040
	2	-0.001	-0.019	-0.047	-0.382	-0.140	-0.300	-0.038
	3	-0.002	-0.019	-0.049	-0.383	-0.141	-0.310	-0.040
LW on Ch-6	1	0.000	-0.007	-0.022	-0.027	-0.476	-0.108	-0.288
	2	0.000	-0.005	-0.020	-0.047	-0.454	-0.104	-0.295
	3	0.000	-0.005	-0.018	-0.046	-0.458	-0.102	-0.302

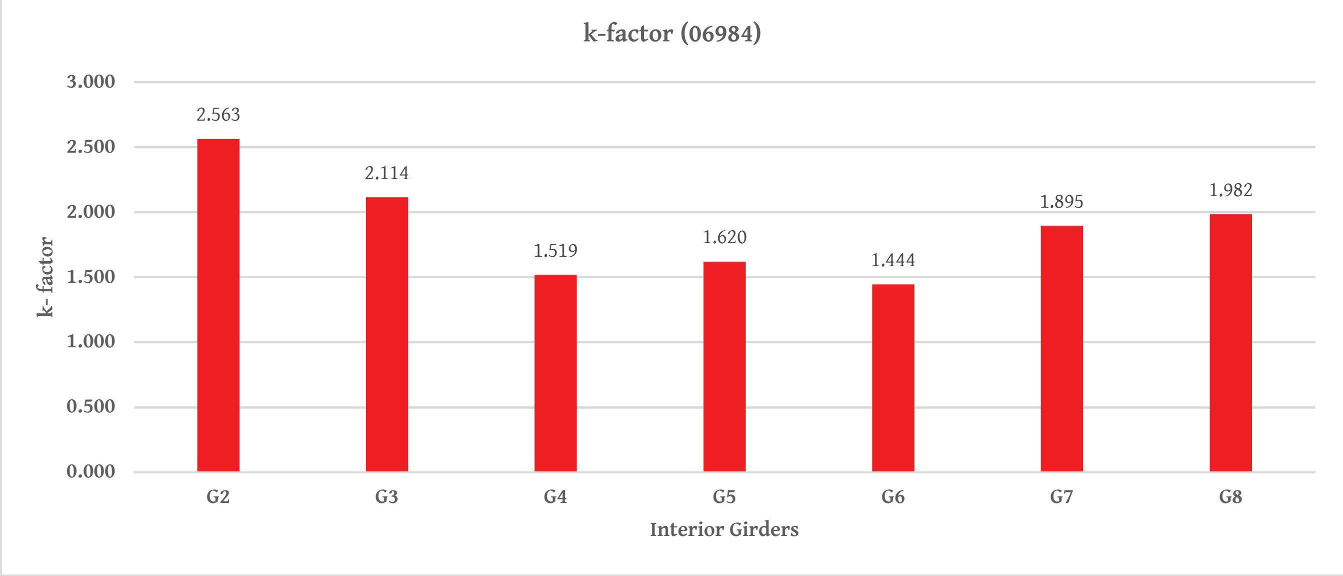


Figure 44: k-factor of 06984

Table 43: 07036 Load Test Summary

Asset ID	07036
Test Date	4/13/2021
Year of Built	1977
ADT	200
County	Spartanburg
Facility Carried	C-42-793
Number of Units	9
Minimum k-factor	1.632
Controlling Girder	G2

Table 44: 07036 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.360	-0.087	-0.351	-0.031	-0.020	-0.007	-0.001
	2	-0.369	-0.094	-0.333	-0.035	-0.014	-0.003	0.000
	3	-0.376	-0.092	-0.336	-0.038	-0.014	-0.004	-0.001
LW on Ch-3	1	-0.056	-0.356	-0.056	-0.544	-0.028	-0.020	-0.006
	2	-0.069	-0.360	-0.073	-0.567	-0.039	-0.015	-0.004
	3	-0.062	-0.360	-0.073	-0.558	-0.043	-0.017	-0.005
LW on Ch-5	1	-0.003	-0.023	-0.038	-0.649	-0.053	-0.285	-0.072
	2	-0.003	-0.021	-0.051	-0.643	-0.068	-0.283	-0.074
	3	-0.002	-0.019	-0.045	-0.627	-0.067	-0.276	-0.070
LW on Ch-6	1	0.000	-0.005	-0.021	-0.050	-0.371	-0.062	-0.260
	2	0.000	-0.005	-0.019	-0.060	-0.360	-0.074	-0.257
	3	0.000	-0.003	-0.017	-0.056	-0.355	-0.073	-0.257

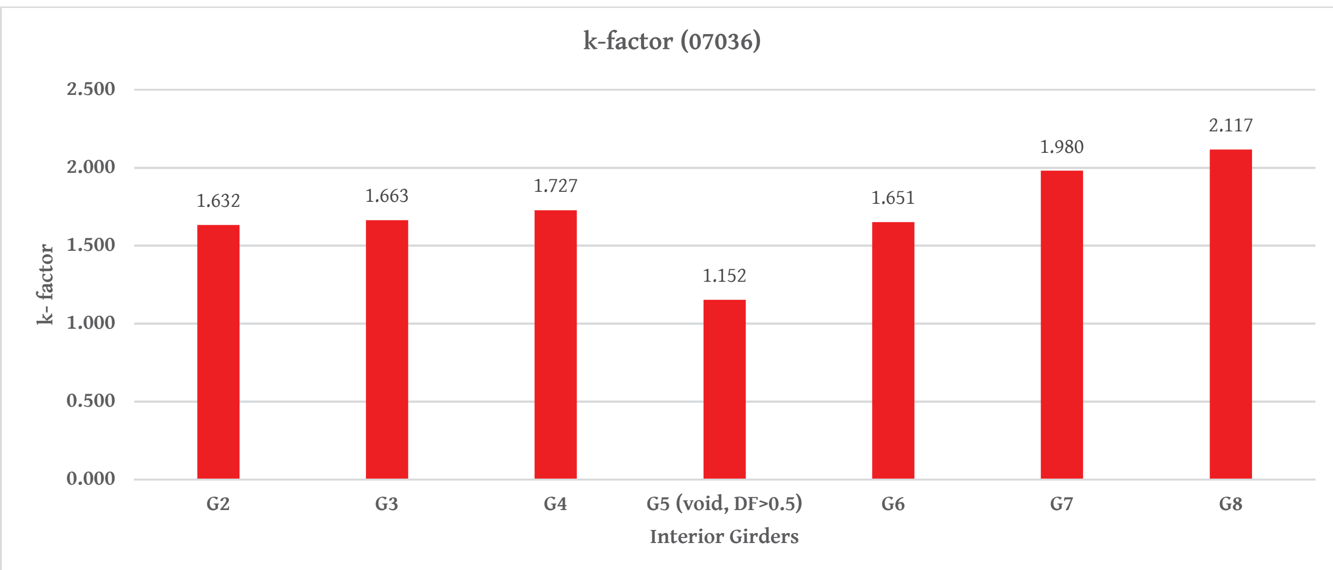


Figure 45: k-factor of 07036

Table 45: 07119 Load Test Summary

Asset ID	07119
Test Date	4/29/2021
Year of Built	1978
ADT	800
County	Greenville
Facility Carried	S-23-97
Number of Units	9
Minimum k-factor	1.298
Controlling Girder	G7

Table 46: 07119 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
RW on Ch-2	1	-0.292	-0.118	-0.227	-0.090	-0.025	-0.006	-0.013
	2	-0.277	-0.109	-0.215	-0.093	-0.024	-0.006	-0.001
	3	-0.283	-0.113	-0.231	-0.094	-0.024	-0.006	0.000
RW on Ch-3	1	-0.074	-0.335	-0.130	-0.253	-0.074	-0.025	-0.005
	2	-0.086	-0.328	-0.135	-0.247	-0.075	-0.025	-0.004
	3	-0.079	-0.330	-0.134	-0.250	-0.081	-0.026	-0.005
RW on Ch-5	1	-0.008	-0.029	-0.107	-0.278	-0.126	-0.482	-0.045
	2	-0.008	-0.030	-0.107	-0.278	-0.138	-0.462	-0.057
	3	-0.008	-0.029	-0.105	-0.279	-0.139	-0.464	-0.061
RW on Ch-6	1	-0.001	-0.008	-0.037	-0.090	-0.314	-0.147	-0.262
	2	-0.001	-0.007	-0.035	-0.090	-0.323	-0.161	-0.263
	3	-0.001	-0.007	-0.036	-0.091	-0.329	-0.161	-0.262

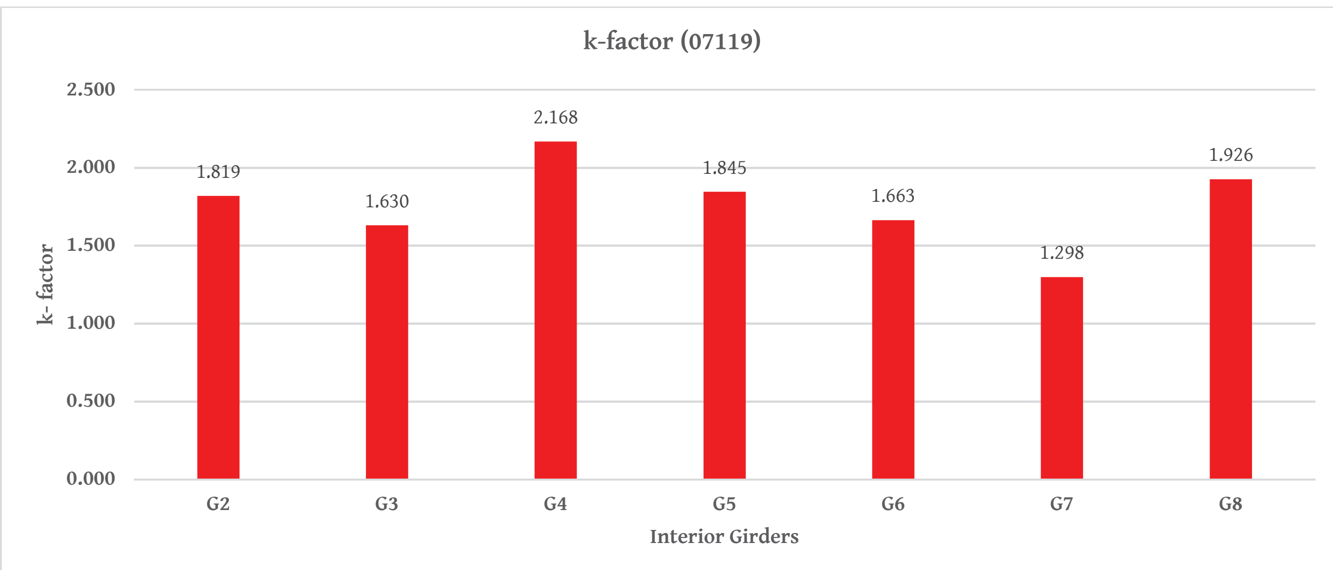


Figure 46: k-factor of 07119

Table 47: 07153 Load Test Summary

Asset ID	07153
Test Date	4/21/2021
Year of Built	1978
ADT	650
County	Pickens
Facility Carried	S-39-64
Number of Units	11
Minimum k-factor	1.776
Controlling Girder	G4

Table 48: 07153 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
RW on Ch-2	1	-0.285	-0.163	-0.320	-0.057	-0.011	0.000	-0.001	-	-
	2	-0.276	-0.164	-0.312	-0.064	-0.010	0.000	0.000	-	-
	3	-0.278	-0.167	-0.322	-0.063	-0.010	0.000	-0.001	-	-
RW on Ch-3	1	-0.100	-0.228	-0.169	-0.216	-0.055	-0.008	-0.002	-	-
	2	-0.094	-0.227	-0.169	-0.223	-0.056	-0.008	-0.002	-	-
	3	-0.095	-0.226	-0.174	-0.226	-0.051	-0.008	-0.001	-	-
RW on Ch-6	1	-	-	-0.014	-0.068	-0.298	-0.159	-0.205	-0.045	-0.018
	2	-	-	-0.019	-0.066	-0.298	-0.156	-0.201	-0.038	-0.015
	3	-	-	-0.019	-0.066	-0.301	-0.156	-0.201	-0.039	-0.015
RW on Ch-7	1	-	-	-0.002	-0.015	-0.078	-0.283	-0.141	-0.245	-0.055
	2	-	-	-0.002	-0.015	-0.080	-0.287	-0.139	-0.238	-0.051
	3	-	-	-0.002	-0.014	-0.068	-0.287	-0.142	-0.257	-0.061
LW on Ch-10	1	-	-	-0.002	-0.006	-0.024	-0.118	-0.219	-0.139	-0.262
	2	-	-	0.000	-0.005	-0.021	-0.113	-0.217	-0.143	-0.243
	3	-	-	0.000	-0.005	-0.021	-0.110	-0.219	-0.143	-0.246

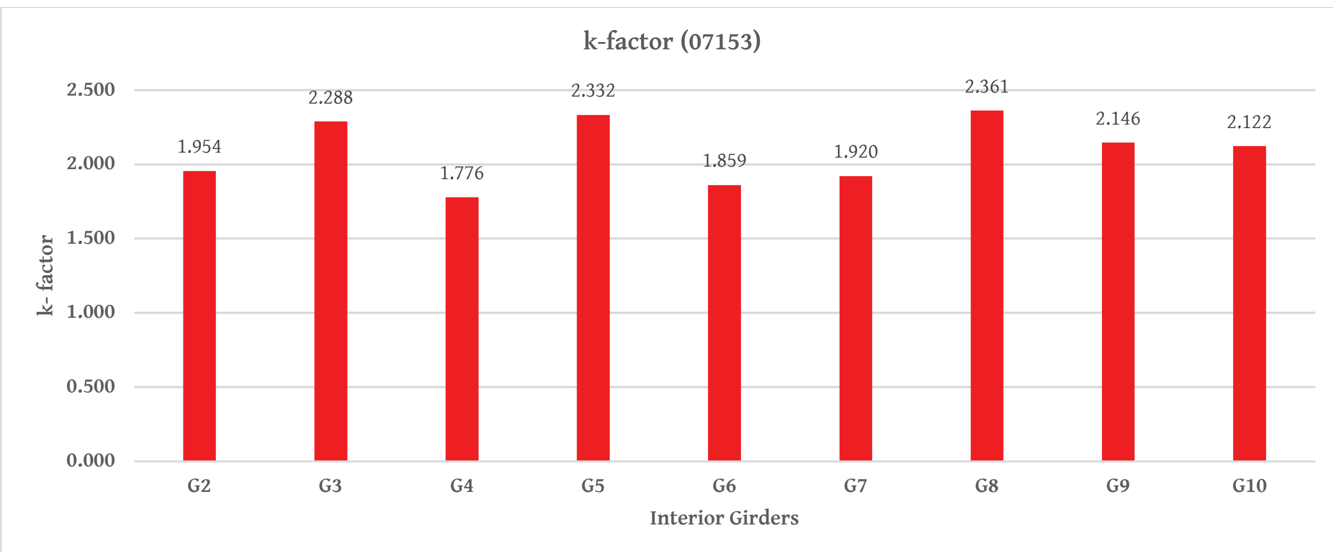


Figure 47: k-factor of 07153

Table 49: 07596 Load Test Summary

Asset ID	07596
Test Date	4/27/2021
Year of Built	1982
ADT	225
County	Oconee
Facility Carried	S-37-160
Number of Units	11
Minimum k-factor	1.765
Controlling Girder	G6

Table 50: 07596 Maximum Displacement Per Test

Test	Run	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9
RW on Ch-3	1	-0.215	-0.142	-0.213	-0.081	-0.015	0.000	-0.001
	2	-0.216	-0.139	-0.218	-0.081	-0.015	0.000	-0.001
	3	-0.219	-0.139	-0.221	-0.082	-0.015	0.000	-0.001
RW on Ch-4	1	-0.046	-0.288	-0.097	-0.281	-0.050	-0.005	-0.001
	2	-0.050	-0.286	-0.094	-0.282	-0.045	-0.003	-0.001
	3	-0.049	-0.290	-0.093	-0.284	-0.044	-0.004	-0.001
RW on Ch-6	1	-0.002	-0.007	-0.043	-0.321	-0.087	-0.261	-0.051
	2	-0.002	-0.008	-0.043	-0.317	-0.095	-0.247	-0.058
	3	-0.001	-0.007	-0.040	-0.315	-0.094	-0.243	-0.057
RW on Ch-7	1	-0.001	0.000	-0.010	-0.065	-0.269	-0.116	-0.222
	2	-0.001	0.000	-0.008	-0.064	-0.272	-0.116	-0.220
	3	-0.001	0.000	-0.009	-0.068	-0.272	-0.118	-0.227

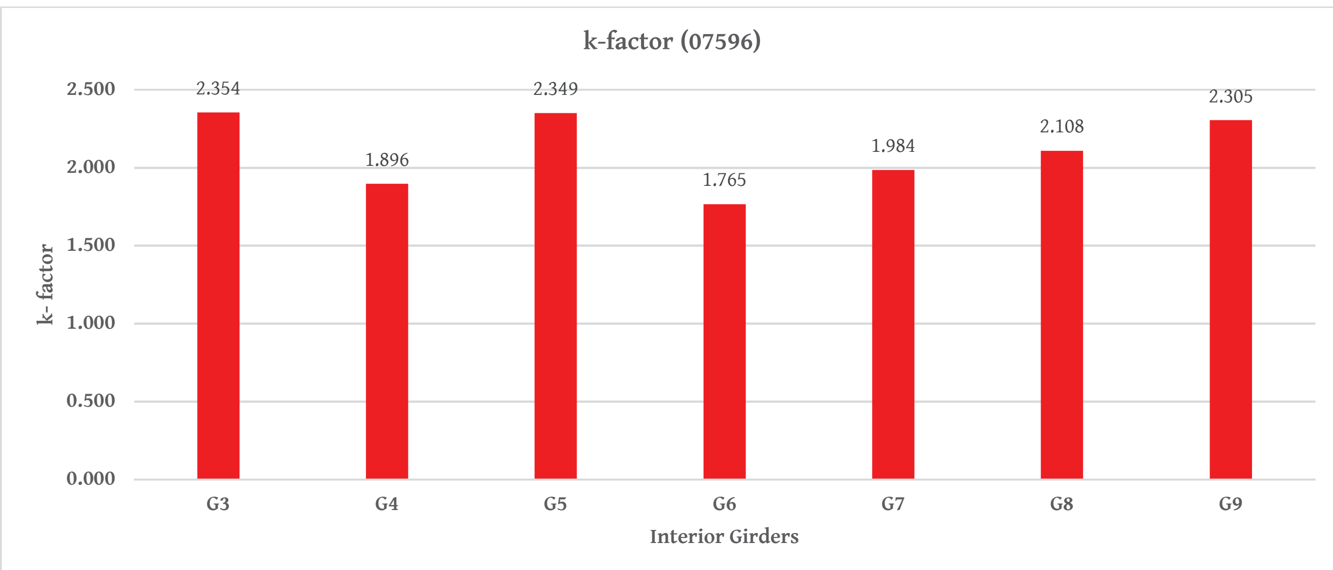


Figure 48: k-factor of 07596

Table 51: 07155 Load Test Summary

Asset ID	07155
Test Date	4/21/2021
Year of Built	1978
ADT	500
County	Pickens
Facility Carried	S-39-142
Number of Units	11
Minimum k-factor	1.488
Controlling Girder	G9

Table 52: 07155 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.147	-0.095	-0.167	-0.122	-0.042	-0.005	-0.003	-	-
	2	-0.152	-0.086	-0.168	-0.121	-0.042	-0.003	-0.003	-	-
	3	-0.153	-0.082	-0.164	-0.130	-0.044	-0.005	-0.004	-	-
LW on Ch-3	1	-0.024	-0.219	-0.111	-0.202	-0.103	-0.020	-0.008	-	-
	2	-0.025	-0.221	-0.110	-0.201	-0.094	-0.017	-0.007	-	-
	3	-0.026	-0.218	-0.108	-0.202	-0.103	-0.020	-0.007	-	-
LW on Ch-6	1	-	-	-0.022	-0.078	-0.196	-0.148	-0.173	-0.105	-0.025
	2	-	-	-0.012	-0.045	-0.195	-0.147	-0.179	-0.109	-0.027
	3	-	-	-0.010	-0.038	-0.193	-0.150	-0.180	-0.118	-0.030
LW on Ch-7	1	-	-	-0.001	-0.009	-0.041	-0.172	-0.128	-0.272	-0.067
	2	-	-	-0.001	-0.008	-0.038	-0.169	-0.130	-0.278	-0.073
	3	-	-	-0.001	-0.008	-0.041	-0.175	-0.127	-0.283	-0.067
RW on Ch-10	1	-	-	-0.001	-0.003	-0.011	-0.052	-0.150	-0.136	-0.238
	2	-	-	-0.001	-0.001	-0.008	-0.053	-0.135	-0.141	-0.233
	3	-	-	0.000	-0.003	-0.014	-0.067	-0.147	-0.145	-0.219

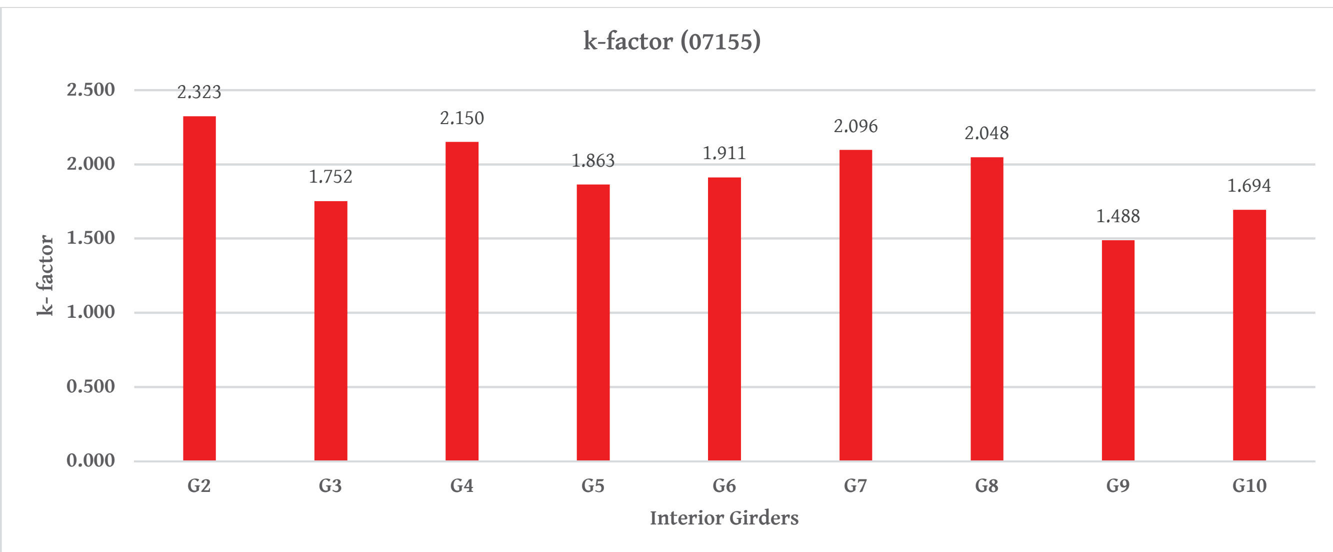


Figure 49: k-factor of 07155

Table 53: 07294 Load Test Summary

Asset ID	07294
Test Date	4/20/2021
Year of Built	1979
ADT	77
County	Saluda
Facility Carried	S-41-227
Number of Units	11
Minimum k-factor	1.463
Controlling Girder	G6

Table 54: 07294 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.204	-0.100	-0.273	-0.058	-0.009	-0.001	-0.001
	2	-0.198	-0.103	-0.271	-0.057	-0.007	0.000	-0.001
	3	-0.203	-0.093	-0.275	-0.052	-0.004	0.000	-0.001
LW on Ch-3	1	-0.033	-0.262	-0.088	-0.250	-0.072	-0.015	-0.006
	2	-0.048	-0.245	-0.087	-0.249	-0.065	-0.012	-0.005
	3	-0.049	-0.248	-0.083	-0.250	-0.065	-0.011	-0.005
RW on Ch-7	1	-0.001	0.000	-0.043	-0.253	-0.110	-0.162	-0.102
	2	-0.001	-0.003	-0.046	-0.245	-0.111	-0.161	-0.094
	3	-0.001	-0.001	-0.043	-0.247	-0.110	-0.165	-0.098
RW on Ch-8	1	0.000	0.000	-0.011	-0.030	-0.288	-0.149	-0.156
	2	-0.001	0.000	-0.010	-0.029	-0.276	-0.137	-0.155
	3	0.000	0.000	-0.010	-0.032	-0.279	-0.139	-0.158

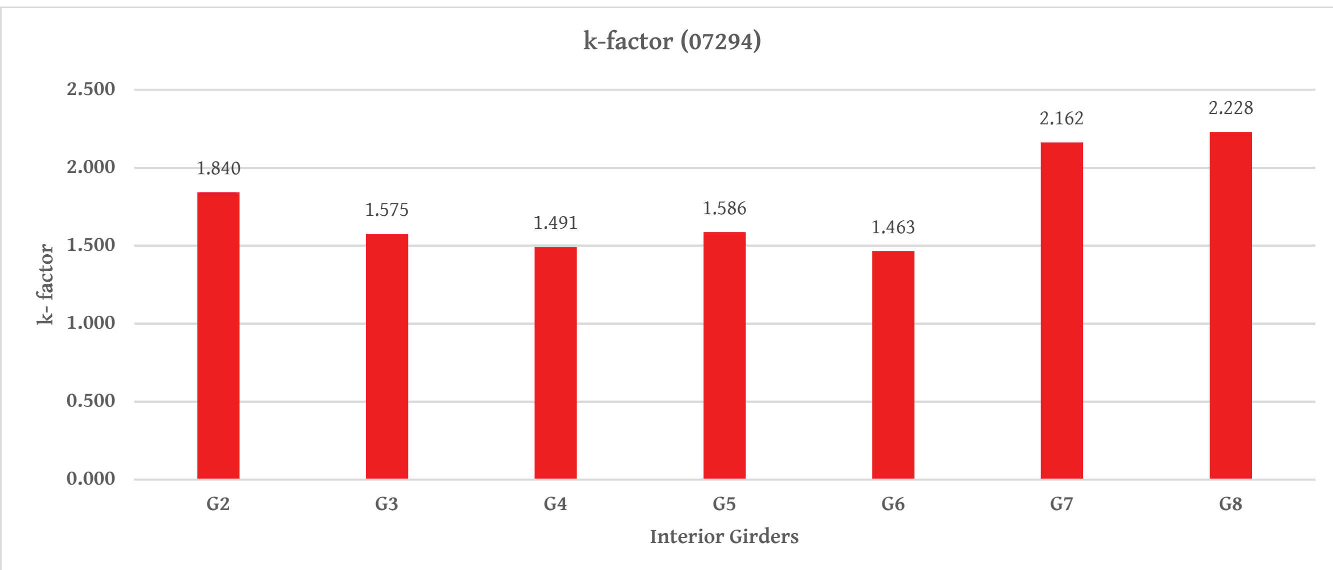


Figure 50: k-factor of 07294

Table 55: 07374 Load Test Summary

Asset ID	07374
Test Date	4/14/2021
Year of Built	19
ADT	2500
County	Greenwood
Facility Carried	S-24-39
Number of Units	14
Minimum k-factor	1.868
Controlling Girder	G5

Table 56: 07374 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.233	-0.080	-0.257	-0.044	-0.011	-0.001	0.000
	2	-0.210	-0.073	-0.233	-0.043	-0.012	-0.001	0.000
	3	-0.208	-0.074	-0.227	-0.047	-0.013	-0.001	0.000
LW on Ch-3	1	-0.069	-0.222	-0.099	-0.227	-0.043	-0.010	-0.001
	2	-0.056	-0.221	-0.098	-0.225	-0.048	-0.011	-0.001
	3	-0.055	-0.226	-0.093	-0.224	-0.048	-0.010	0.000
LW on Ch-5	1	-0.003	-0.010	-0.037	-0.245	-0.082	-0.196	-0.050
	2	-0.003	-0.011	-0.046	-0.247	-0.087	-0.198	-0.048
	3	-0.002	-0.009	-0.041	-0.249	-0.091	-0.197	-0.049
LW on Ch-6	1	-0.001	-0.003	-0.014	-0.068	-0.192	-0.065	-0.239
	2	0.000	-0.002	-0.015	-0.061	-0.196	-0.069	-0.228
	3	0.000	-0.002	-0.014	-0.056	-0.200	-0.069	-0.224

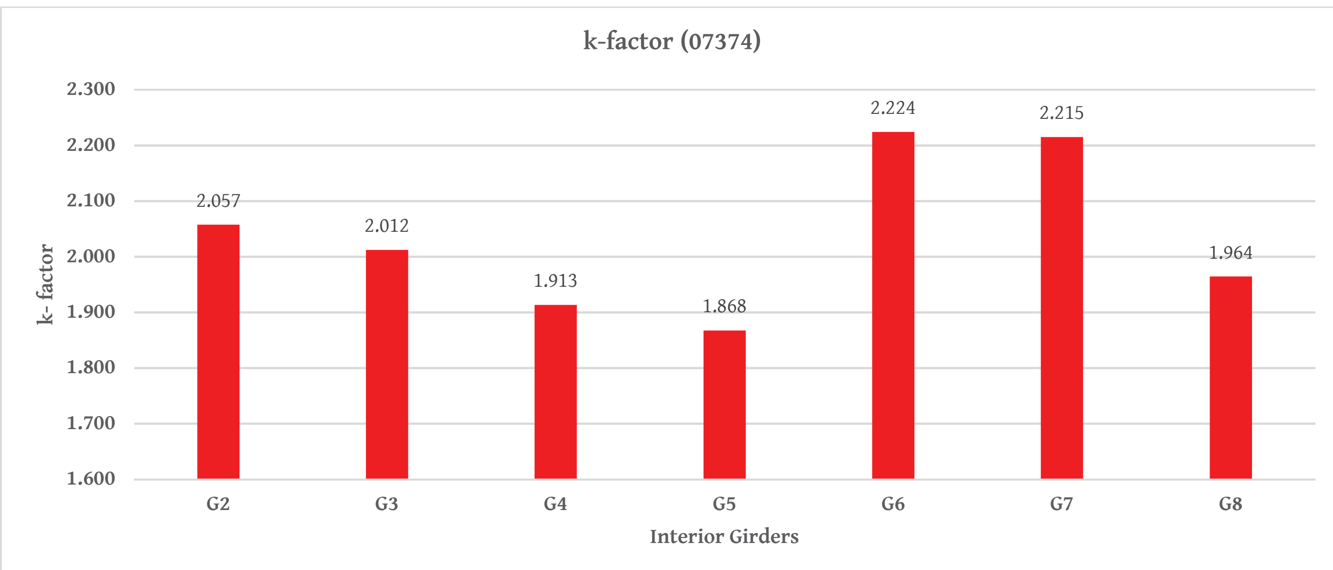


Figure 51: k-factor of 07374

Table 57: 07605 Load Test Summary

Asset ID	07605
Test Date	4/22/2021
Year of Built	19
ADT	1850
County	Pickens
Facility Carried	S-39-137
Number of Units	13
Minimum k-factor	1.441
Controlling Girder	G4

Table 58: 07605 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.113	-0.075	-0.295	-0.044	-0.018	-0.004	0.000	-	-
	2	-0.110	-0.072	-0.298	-0.048	-0.021	-0.006	-0.001	-	-
	3	-0.112	-0.070	-0.297	-0.047	-0.020	-0.005	-0.001	-	-
LW on Ch-3	1	-0.040	-0.140	-0.087	-0.229	-0.060	-0.013	-0.005	-	-
	2	-0.039	-0.143	-0.085	-0.232	-0.057	-0.013	-0.005	-	-
	3	-0.041	-0.148	-0.085	-0.237	-0.056	-0.013	-0.006	-	-
LW on Ch-6	1	-	-	-0.012	-0.020	-0.201	-0.059	-0.248	-0.047	-0.015
	2	-	-	-0.013	-0.021	-0.200	-0.059	-0.254	-0.045	-0.013
	3	-	-	-0.012	-0.018	-0.199	-0.060	-0.249	-0.046	-0.014
RW on Ch-7	1	-	-	-0.007	-0.009	-0.031	-0.173	-0.109	-0.151	-0.093
	2	-	-	-0.006	-0.008	-0.031	-0.173	-0.105	-0.144	-0.092
	3	-	-	-0.006	-0.008	-0.030	-0.171	-0.106	-0.145	-0.092
RW on Ch-10	1	-	-	-0.003	-0.003	-0.009	-0.022	-0.209	-0.092	-0.197
	2	-	-	-0.002	-0.003	-0.008	-0.025	-0.208	-0.088	-0.191
	3	-	-	-0.003	-0.003	-0.009	-0.024	-0.208	-0.085	-0.190

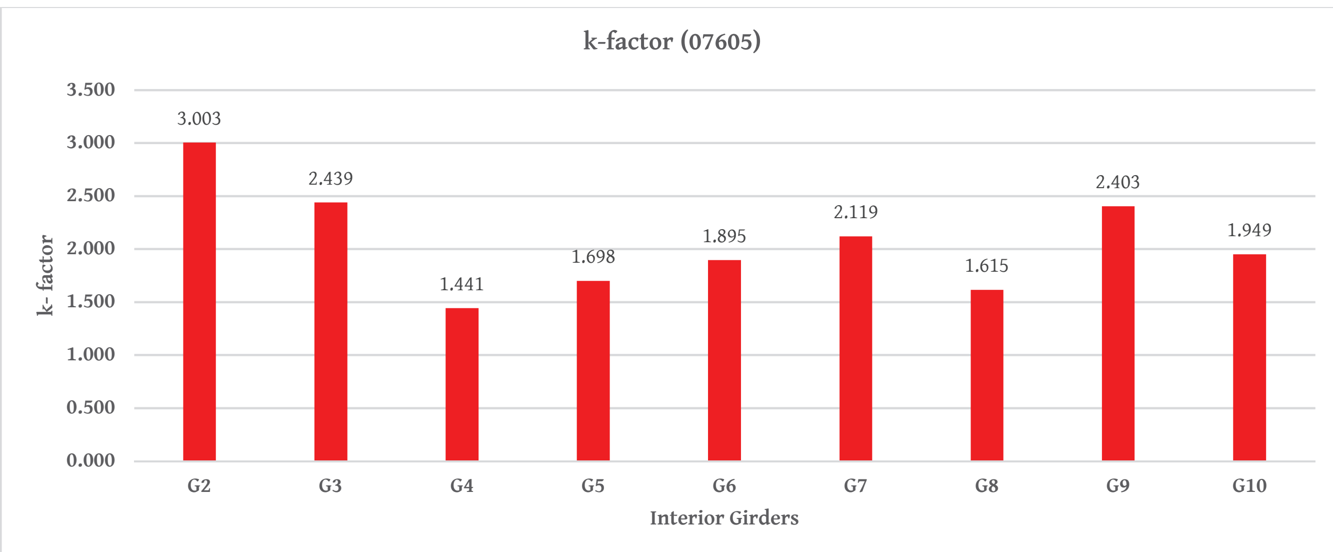


Figure 52: k-factor of 07605

Table 59: 07606 Load Test Summary

Asset ID	07606
Test Date	4/22/2021
Year of Built	1982
ADT	1850
County	Pickens
Facility Carried	S-39-137
Number of Units	13
Minimum k-factor	1.257
Controlling Girder	G6

Table 60: 07606 Maximum Displacement Per Test

Test	Run	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10	Channel-11
LW on Ch-3	1	-0.169	-0.086	-0.217	-0.064	-0.008	0.000	-0.001	-	-
	2	-0.161	-0.091	-0.212	-0.059	-0.013	-0.001	0.000	-	-
	3	-0.162	-0.093	-0.212	-0.059	-0.012	-0.001	-0.001	-	-
LW on Ch-4	1	-0.023	-0.250	-0.068	-0.376	-0.032	-0.005	-0.004	-	-
	2	-0.024	-0.250	-0.069	-0.377	-0.033	-0.005	-0.004	-	-
	3	-0.023	-0.252	-0.071	-0.372	-0.034	-0.006	-0.005	-	-
LW on Ch-7	1	-	-	-0.016	-0.049	-0.187	-0.113	-0.173	-0.071	-0.022
	2	-	-	-0.017	-0.049	-0.175	-0.121	-0.172	-0.071	-0.021
	3	-	-	-0.016	-0.054	-0.189	-0.125	-0.188	-0.058	-0.021
RW on Ch-10	1	-	-	-0.008	-0.016	-0.066	-0.175	-0.113	-0.188	-0.047
	2	-	-	-0.008	-0.015	-0.064	-0.174	-0.111	-0.188	-0.049
	3	-	-	-0.006	-0.013	-0.061	-0.172	-0.108	-0.191	-0.051
RW on Ch-11	1	-	-	-0.001	-0.003	-0.022	-0.063	-0.155	-0.116	-0.148
	2	-	-	0.000	-0.002	-0.021	-0.060	-0.154	-0.117	-0.149
	3	-	-	-0.002	-0.004	-0.023	-0.065	-0.159	-0.123	-0.153

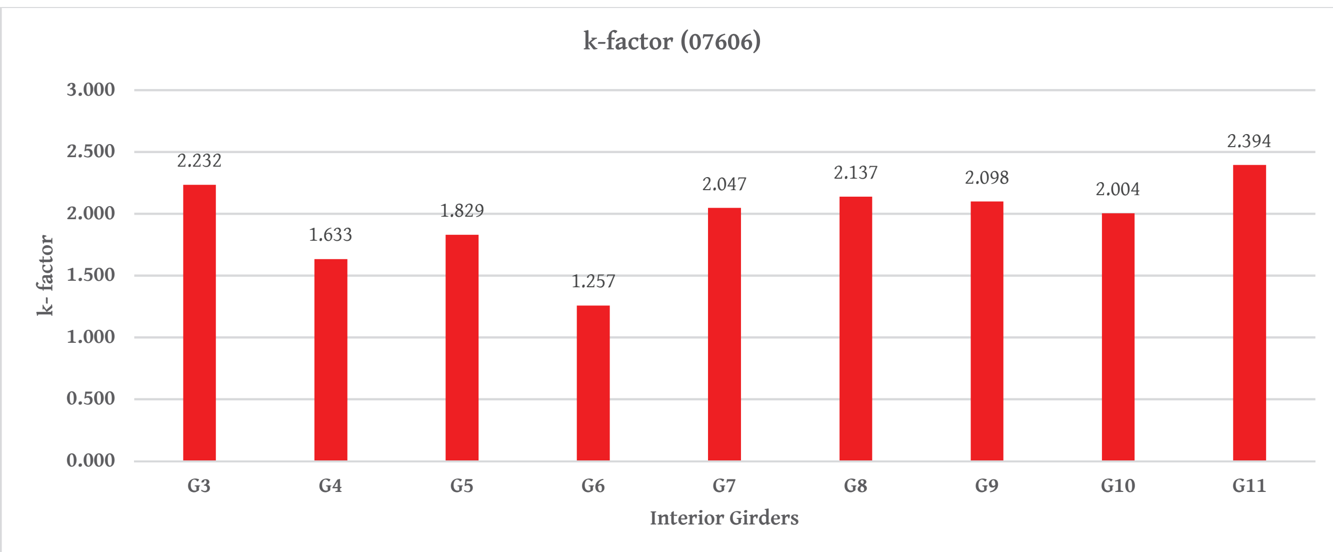


Figure 53: k-factor of 07606

Table 61: 07741 Load Test Summary

Asset ID	07741
Test Date	4/13/2021
Year of Built	1983
ADT	2100
County	Spartanburg
Facility Carried	C-42-134
Number of Units	9
Minimum k-factor	1.289
Controlling Girder	G2

Table 62: 07741 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.357	-0.047	-0.330	-0.025	-0.010	-0.003	-0.001
	2	-0.373	-0.056	-0.329	-0.027	-0.009	-0.002	-0.001
	3	-0.374	-0.052	-0.336	-0.024	-0.008	-0.002	-0.001
LW on Ch-3	1	-0.037	-0.337	-0.052	-0.313	-0.035	-0.014	-0.001
	2	-0.037	-0.334	-0.057	-0.314	-0.036	-0.015	-0.001
	3	-0.036	-0.334	-0.052	-0.308	-0.033	-0.013	-0.001
LW on Ch-5	1	-0.003	-0.011	-0.016	-0.352	-0.050	-0.326	-0.039
	2	-0.002	-0.009	-0.032	-0.344	-0.055	-0.317	-0.039
	3	-0.002	-0.009	-0.031	-0.349	-0.056	-0.326	-0.038
LW on Ch-6	1	0.000	0.000	-0.006	-0.029	-0.331	-0.057	-0.326
	2	0.000	0.000	-0.008	-0.035	-0.329	-0.070	-0.325
	3	0.000	0.000	-0.008	-0.034	-0.328	-0.069	-0.323

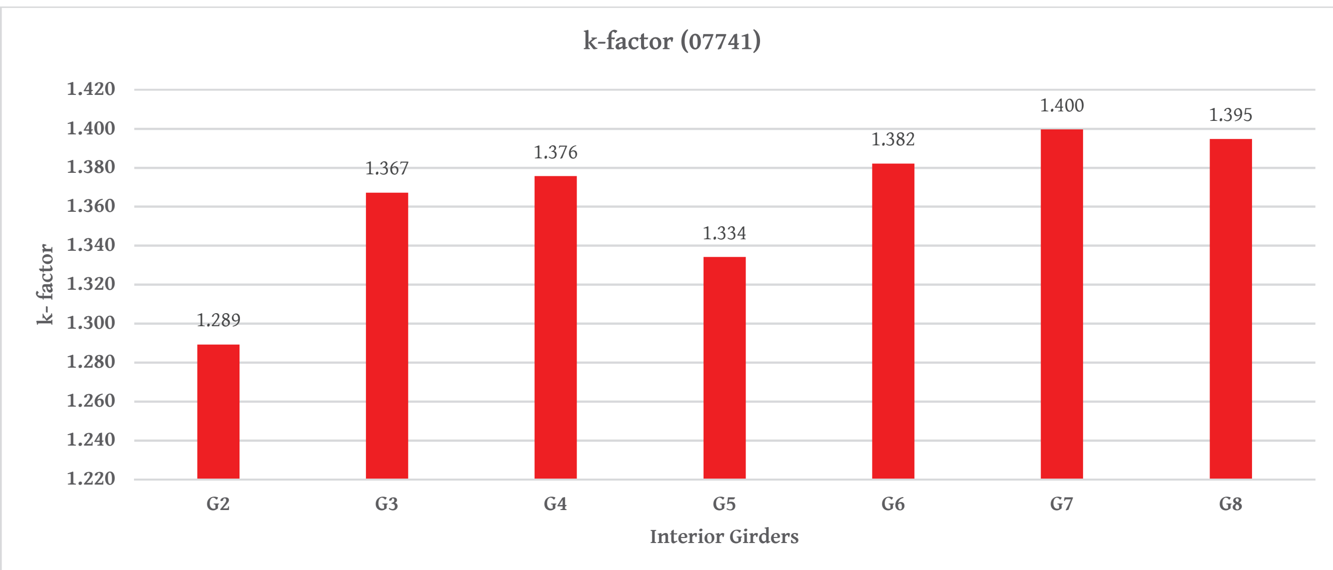


Figure 54: k-factor of 07741

Table 63: 07766 Load Test Summary

Asset ID	07766
Test Date	4/23/2021
Year of Built	1984
ADT	850
County	Cherokee
Facility Carried	S-11-63
Number of Units	11
Minimum k-factor	1.449
Controlling Girder	G6

Table 64: 07766 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.253	-0.109	-0.269	-0.064	-0.004	0.000	0.000	-	-
	2	-0.257	-0.101	-0.251	-0.075	-0.005	-0.001	0.000	-	-
	3	-0.265	-0.095	-0.254	-0.077	-0.006	0.000	0.000	-	-
LW on Ch-3	1	-0.059	-0.212	-0.135	-0.232	-0.051	-0.007	0.000	-	-
	2	-0.063	-0.214	-0.134	-0.245	-0.046	-0.006	0.000	-	-
	3	-0.064	-0.213	-0.131	-0.247	-0.044	-0.006	0.000	-	-
LW on Ch-6	1	-	-	-0.004	-0.038	-0.293	-0.090	-0.285	-0.041	-0.017
	2	-	-	-0.011	-0.043	-0.287	-0.094	-0.273	-0.039	-0.016
	3	-	-	-0.011	-0.039	-0.288	-0.092	-0.277	-0.040	-0.017
RW on Ch-7	1	-	-	-0.002	-0.008	-0.061	-0.239	-0.114	-0.264	-0.063
	2	-	-	-0.001	-0.005	-0.052	-0.227	-0.128	-0.259	-0.068
	3	-	-	-0.001	-0.004	-0.052	-0.231	-0.128	-0.264	-0.068
RW on Ch-10	1	-	-	0.000	0.000	-0.007	-0.049	-0.278	-0.114	-0.260
	2	-	-	0.000	0.000	-0.009	-0.063	-0.261	-0.131	-0.256
	3	-	-	0.000	0.000	-0.009	-0.061	-0.259	-0.124	-0.259

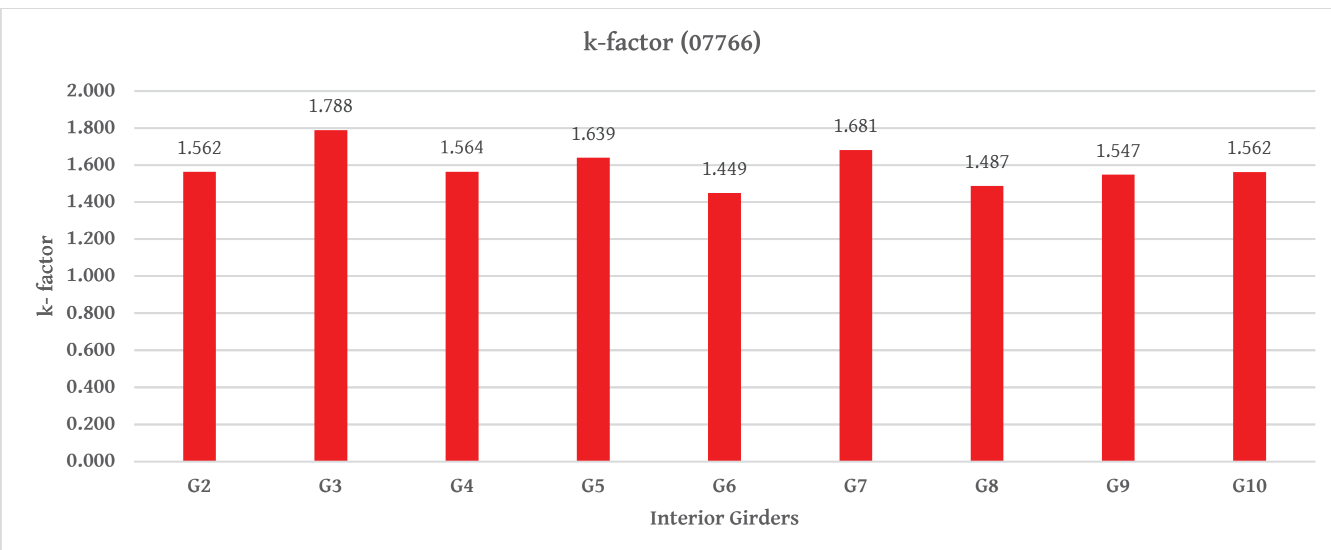


Figure 55: k-factor of 07766

Table 65: 07842 Load Test Summary

Asset ID	07842
Test Date	4/12/2021
Year of Built	1984
ADT	200
County	Spartanburg
Facility Carried	C-42-662
Number of Units	9
Minimum k-factor	1.349
Controlling Girder	G5

Table 66: 07842 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.247	-0.061	-0.280	-0.047	-0.022	0.000	-0.001
	2	-0.258	-0.054	-0.282	-0.037	-0.022	0.000	-0.001
	3	-0.031	-0.268	-0.054	-0.347	-0.049	-0.010	-0.006
LW on Ch-3	1	-0.053	-0.243	-0.059	-0.338	-0.041	-0.008	-0.004
	2	-0.001	0.000	-0.019	-0.037	-0.303	-0.052	-0.309
	3	-0.001	0.000	-0.020	-0.052	-0.287	-0.056	-0.304
LW on Ch-6	1	-0.001	-0.003	-0.031	-0.327	-0.047	-0.226	-0.096
	2	-0.001	-0.005	-0.035	-0.316	-0.060	-0.216	-0.101
	3	-0.247	-0.061	-0.280	-0.047	-0.022	0.000	-0.001
RW on Ch-7	1	-0.258	-0.054	-0.282	-0.037	-0.022	0.000	-0.001
	2	-0.031	-0.268	-0.054	-0.347	-0.049	-0.010	-0.006
	3	-0.053	-0.243	-0.059	-0.338	-0.041	-0.008	-0.004

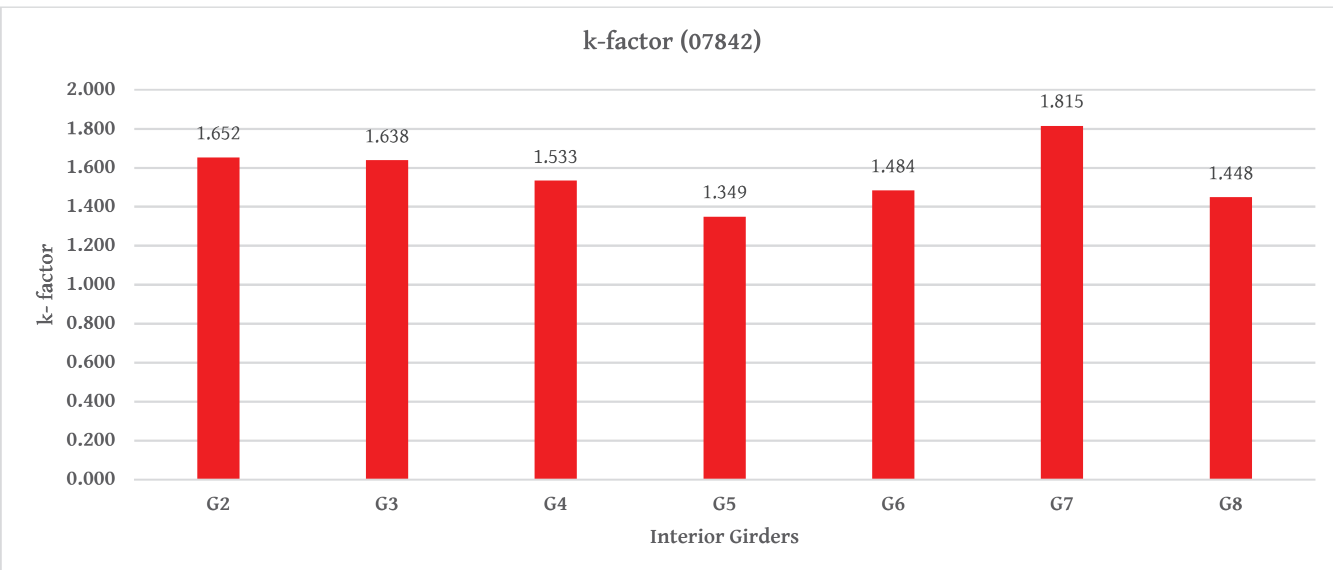


Figure 56: k-factor of 07842

Table 67: 07853 Load Test Summary

Asset ID	07853
Test Date	4/12/2021
Year of Built	1984
ADT	200
County	Spartanburg
Facility Carried	C-42-806
Number of Units	9
Minimum k-factor	1.322
Controlling Girder	G5

Table 68: 07853 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8
LW on Ch-2	1	-0.283	-0.050	-0.303	-0.022	-0.016	-0.004	-0.001
	2	-0.283	-0.062	-0.283	-0.036	-0.015	-0.003	-0.001
	3	-0.298	-0.039	-0.305	-0.018	-0.019	-0.005	-0.001
LW on Ch-3	1	-0.066	-0.246	-0.093	-0.320	-0.033	-0.014	-0.002
	2	-0.064	-0.249	-0.097	-0.285	-0.035	-0.015	-0.001
	3	-0.068	-0.238	-0.090	-0.320	-0.031	-0.014	-0.001
LW on Ch-5	1	-0.003	-0.015	-0.025	-0.367	-0.032	-0.313	-0.025
	2	-0.001	-0.010	-0.041	-0.345	-0.048	-0.299	-0.037
	3	0.000	-0.009	-0.040	-0.348	-0.048	-0.302	-0.037
LW on Ch-6	1	0.000	0.000	-0.013	-0.043	-0.306	-0.077	-0.227
	2	-0.001	-0.001	-0.013	-0.044	-0.306	-0.077	-0.227
	3	-0.001	-0.001	-0.013	-0.043	-0.307	-0.077	-0.230

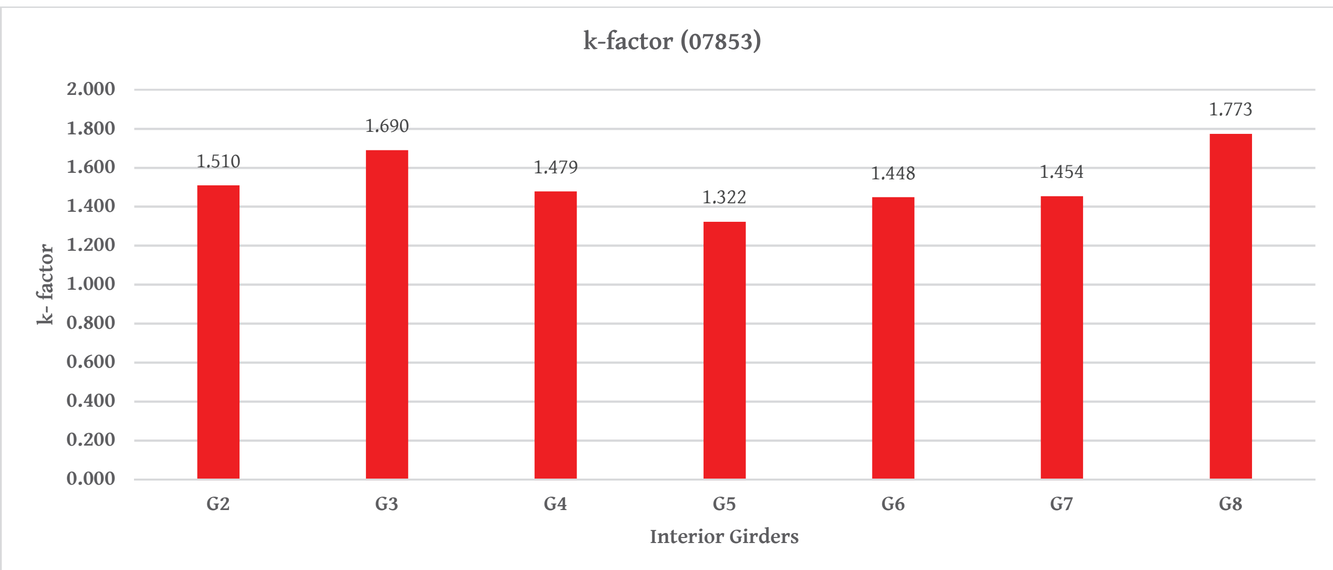


Figure 57: k-factor of 07853

Table 69: 08142 Load Test Summary

Asset ID	08142
Test Date	4/23/2021
Year of Built	1988
ADT	289
County	Cherokee
Facility Carried	S-11-60
Number of Units	11
Minimum k-factor	1.363
Controlling Girder	G6

Table 70: 08142 Maximum Displacement Per Test

Test	Run	Channel-2	Channel-3	Channel-4	Channel-5	Channel-6	Channel-7	Channel-8	Channel-9	Channel-10
LW on Ch-2	1	-0.195	-0.157	-0.234	-0.076	-0.015	-0.006	0.000	-	-
	2	-0.192	-0.149	-0.229	-0.071	-0.012	-0.005	0.000	-	-
	3	-0.201	-0.153	-0.243	-0.075	-0.012	-0.005	0.000	-	-
LW on Ch-3	1	-0.076	-0.179	-0.136	-0.224	-0.083	-0.013	-0.003	-	-
	2	-0.072	-0.177	-0.133	-0.227	-0.085	-0.013	-0.003	-	-
	3	-0.072	-0.180	-0.134	-0.229	-0.083	-0.012	-0.002	-	-
LW on Ch-6	1	-	-	-0.012	-0.042	-0.326	-0.135	-0.211	-0.051	-0.020
	2	-	-	-0.012	-0.037	-0.313	-0.140	-0.205	-0.065	-0.020
	3	-	-	-0.014	-0.046	-0.316	-0.142	-0.202	-0.059	-0.019
LW on Ch-7	1	-	-	-0.002	-0.006	-0.053	-0.241	-0.152	-0.222	-0.067
	2	-	-	-0.001	-0.007	-0.050	-0.240	-0.151	-0.225	-0.069
	3	-	-	-0.001	-0.007	-0.045	-0.239	-0.150	-0.228	-0.071
RW on Ch-10	1	-	-	-0.001	-0.001	-0.012	-0.098	-0.215	-0.113	-0.270
	2	-	-	-0.001	-0.001	-0.013	-0.102	-0.194	-0.114	-0.268
	3	-	-	-0.001	-0.002	-0.014	-0.110	-0.193	-0.104	-0.275

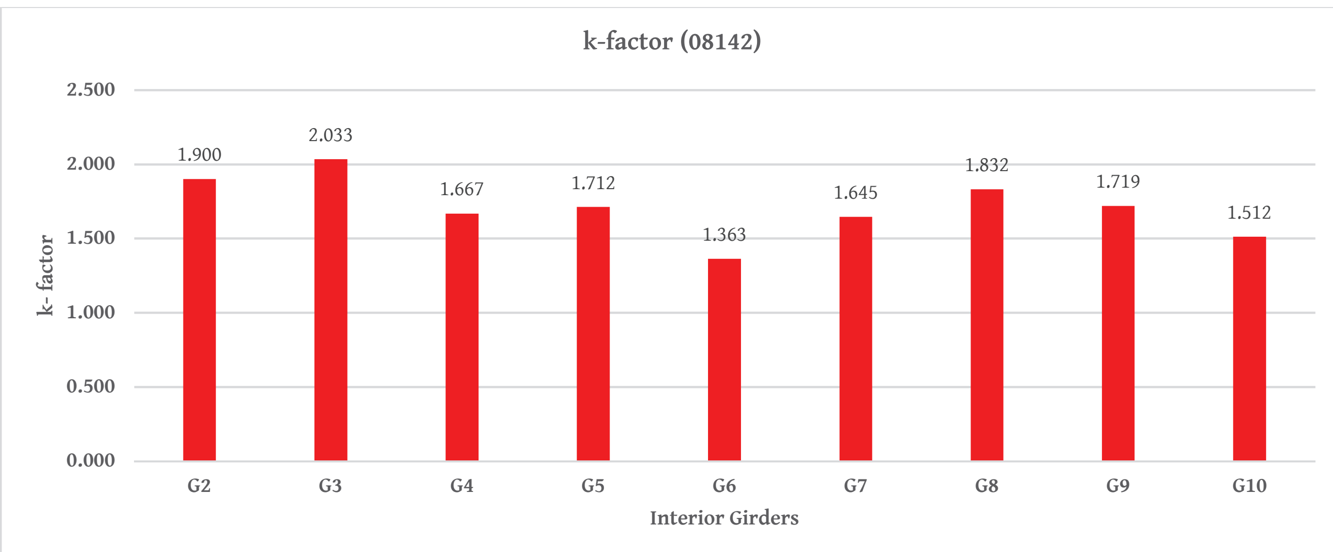


Figure 58: k-factor of 08142

APPENDIX

C SKINNY LEG *DF* FINDINGS

10 Visual Guide for Distribution Factor Selection

Based on the results and observations of all tested bridges, the following Visual Guide was developed to assist in determine the distribution factor of 2.5" leg channel bridges in South Carolina. Our findings concluded that there was strong correlation between the degree of reflective cracking in the deck and the corresponding Distribution Factors. In all cases the distribution factors were found to be at or above the AASHTO LRFD and LFD codes. We recommend engineering judgment be utilized in conjunction with this visual guide in determining distribution factors for load ratings of this bridge type.

If post-tensioned rods are sounded and not identified as being loose or broken, the distribution factor for the 2 ½" leg channels can be based on the degree of reflective cracking. Following photos provide visual guide for the inspector's reference. If post-tensioned rods are loose when inspected, use 0.5 distribution factor for the LRFR load rating regardless of degree of reflective cracking.

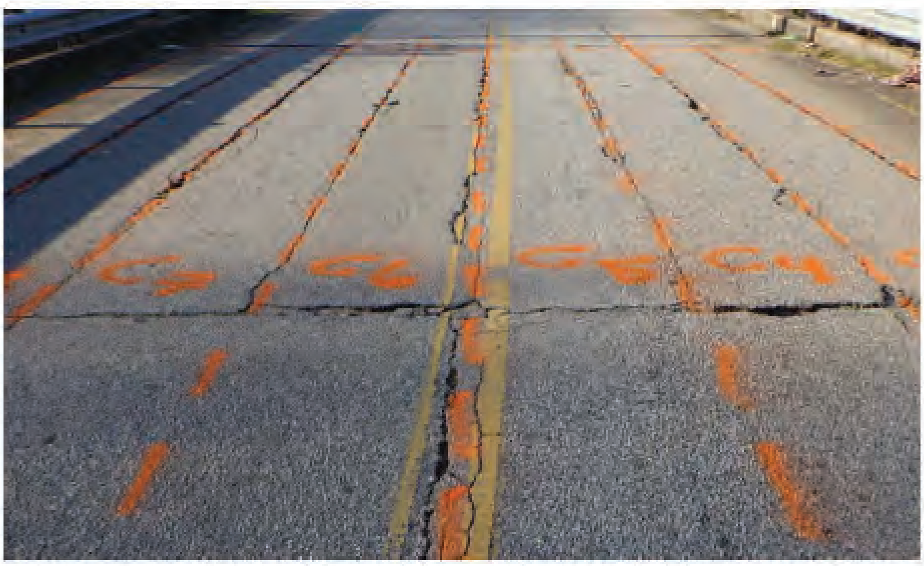
Reflective Crack Condition	Tie Rod Condition	Recommended DF (Lane)
None/Minor	Good	Follow AASHTO
Moderate	Good	0.35
Severe	Good	0.5
Any Condition	Poor/Loose	0.5

Reflective Cracking: None/Minor – DF = Follow AASHTO

Reflective Cracking: Moderate – DF = 0.35 Lane

Reflective Cracking: Moderate – DF = 0.35 Lane

Reflective Cracking: Severe – DF = 0.5 Lane



Reflective Cracking: Severe – DF = 0.5 Lane