SCDOT

LOAD TESTING OF MULTIPLE RC TEE-BEAM BRIDGES

STATEWIDE – SOUTH CAROLINA

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SCDOT

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1 BACKGROUND

1.1 INTRODUCTION

Approximately 300 reinforced concrete (RC) tee-beam bridges in SCDOT's bridge inventory are currently recommended for posting. In general, many of these bridges carry high ADT and posting them will have a considerable impact on the motoring public. Therefore, a targeted testing program, including both material testing and load testing, was developed and performed with the goal of reducing posting requirements for this family of structures. This testing program was performed in April and May of 2021 and is described in the following sections of the document.

1.2 OBJECTIVES AND GOALS

The bridges in this family of structures have varying structural attributes including but not limited to:

- Year of construction
- Span length
- Number and spacing of RC Tee-Beams in cross-section
- Beam Size
- Reinforcement detailing
- Material properties
- Existence of widenings
- Type of widening

Furthermore, preliminary review of existing load ratings for the RC Tee-Beam bridges that are currently recommended for posting indicate that the posting recommendations are not based on a consistent controlling mechanism. The reported load ratings show that the RC Tee-Beam capacity is controlled in some instances by shear and by flexure in other instances.

The overall goal of the program is to reduce the posting requirements that are currently recommended based on the simplified load ratings. To do this efficiently the number of variables that exist in the RC Tee-Beam population were considered. Ultimately, to reduce the posting requirements, the program needed to determine which overly conservative assumptions were used in the original load ratings and come up with more realistic assumptions. Therefore, the intent of this testing program is to test specific features of the bridges that are expected to include conservative assumptions. WSP's load testing experience on other bridges in SCDOT's inventory was leveraged to identify these specific features.

First, to address bridges with low shear ratings, potential gains in shear capacity were identified from higher concrete strengths, as the web concrete is a large contributor to the shear resistance of an RC Tee-Beam. Therefore, it was determined that the best testing approach would be taking cores from low rating bridges to determine the actual concrete strengths. Many of SCDOT's older bridges do not have material strengths specified in the original plans, therefore material properties are assumed based on AASHTO MBE guidance. Other testing programs that WSP has performed have commonly found that concrete strengths estimated in this manner are lower than the actual material strength.

Next, to address bridges with low flexural ratings, several features of the bridges were identified as potential load testing targets. Like bridges with low shear ratings, it was clear that higher concrete strength would increase flexural load ratings. Additionally, it was identified that calculating "K" factors through diagnostic testing presented the potential to prove that the RC Tee-Beam bridges are performing better than traditional theoretical calculations predict. Furthermore, upon review of the current load ratings, having a better understanding of load distribution through construction joints between original cross-sections and widened sections was identified as having additional potential to increase the load ratings.

2 EXPERIMENTAL PLAN

2.1 TESTING THEORY

The RC Tee-Beam bridge testing program that was performed can be broken down into several different portions as described in Section 1.2 of this report. The theory behind each portion of this testing program is explained below.

Initially, concrete cores were taken from all RC Tee-Beam bridges with low shear ratings. The shear resistance of an RC Tee-Beam is provided by the shear reinforcement and concrete in the web of the RC Tee-Beam. If concrete cores determine that the actual strength of the concrete is higher than was assumed in the load rating calculations, a higher shear resistance will be provided by the concrete. This will directly increase the shear capacity of the section and therefore increase the shear load rating of the RC Tee-Beams.

Following this, load testing procedures were developed to address bridges with low flexural ratings. As described in Section 1.2 of this report, these load testing procedures have two goals. The first goal is to calculate “K” factors for each tested RC Tee-Beam, in accordance with Section 8.8.2.3 of AASHTO MBE. This section of AASHTO MBE provides guidance on determining adjustment factors, known as “K” factors, that can be used to directly adjust the load ratings by considering the actual response (as measured in a load test) compared to the theoretical response (as calculated based on code guidance) and is defined as:

$$K = 1 + K_a K_b$$

Where K_a is directly calculated based on the measured response of the bridge versus the theoretical response under the same loading, while K_b is a variable determined based on several aspects of the testing that is performed. A more in-depth discussion about K_b is provided in Section 2.2 of this report. Ultimately, K can be greater than or less than 1.0. If $K > 1.0$, the response of the bridge has indicated that the load capacity may be higher than theoretical calculations and the load ratings can be improved, and if $K < 1.0$ the response of the bridge has indicated that the load capacity may be lower than theoretical calculations and the load ratings may have to be reduced.

The second goal of the load testing was only applicable to bridges that have been widened during their lifetime. Preliminary review of current load ratings for structures with widenings show that the assumptions made when considering the interface between the two sections (original and widened) are inconsistent across different structures with no clear difference in the joint detailing. Some load raters assume the existing and widened structures are sufficiently connected to act as a unit for load rating purposes, while others may consider them as two individual structures. The inconsistent assumptions on structure response lead to different load rating results. As such, it is important to evaluate the performance of the joints, and the level of load sharing between the original RC Tee-Beam sections and widened sections.

It is possible to determine the level of load sharing across a joint by loading the structure directly adjacent to the interface between the two sections and measuring the response of the original and widened structural members nearest the joint. Review of plans of the RC Tee-Beam bridges that are currently recommended for load postings showed that four different widening types have been used over the years to widen RC Tee-Beam structures in South Carolina as listed below:

- Slab Widening
- RC Tee-Beam Widening
- AASHTO Girder Widening
- Steel Girder Widening

Examples of the four different widening types are shown in Figure 1 through Figure 4.

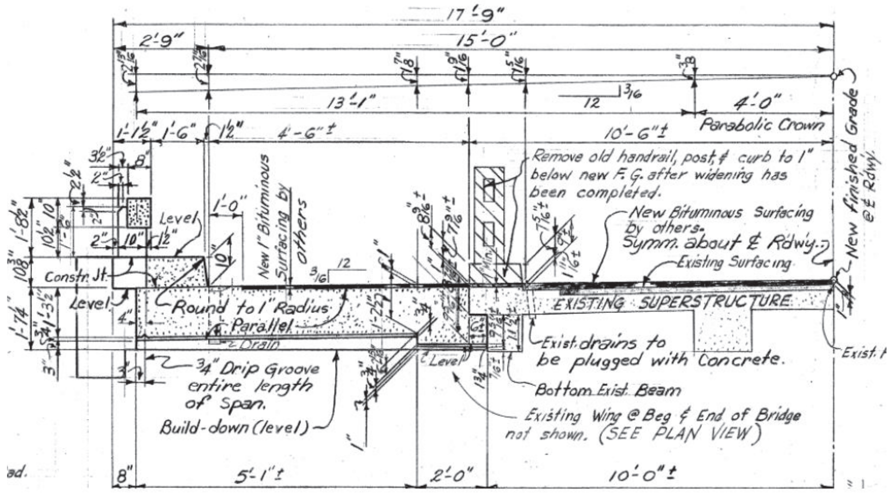


Figure 1: RC Tee-Beam Bridge with Slab Widening

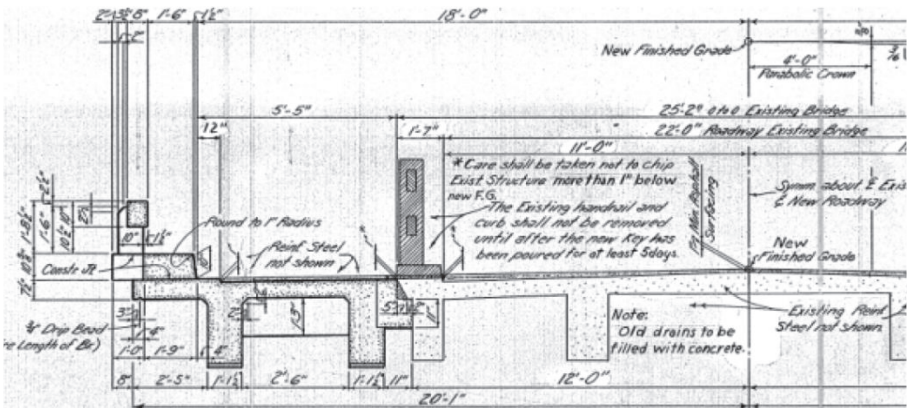


Figure 2: RC Tee-Beam Bridge with RC Tee-Beam Widening

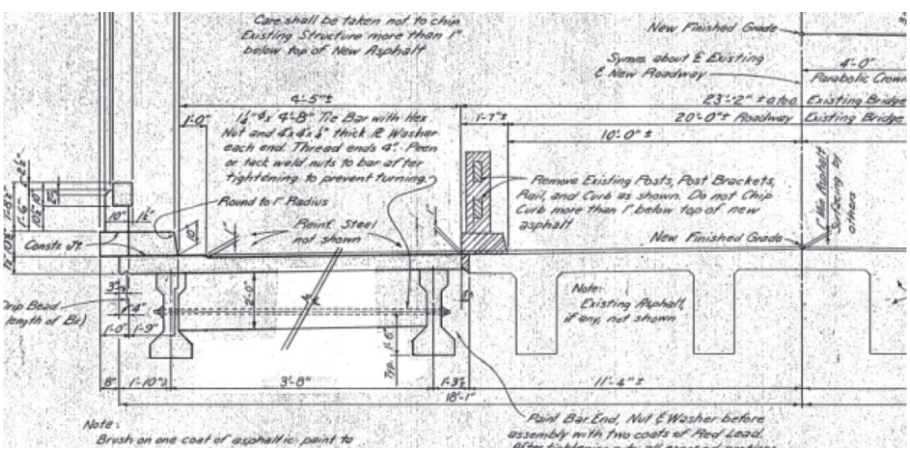


Figure 3: RC Tee-Beam Bridge with AASHTO Girder Widening

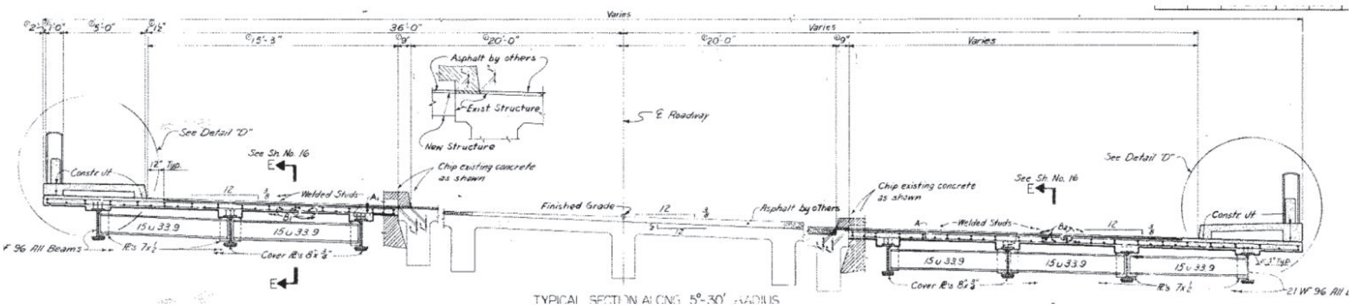


Figure 4: RC Tee-Beam Bridge with Steel Girder Widening

In addition to the various widening types, each type of widening employs different detailing methods at the interface between the original section and widened section. Furthermore, some structures have had multiple widenings constructed over their lifetimes which do not always use similar detailing, and in some cases are completely different types of widenings. Due to the variability found in these structure widenings, extra considerations had to be made for testing these joints. In general, the testing theory is the same for each type of joint. That is, for each type of joint the testing consists of applying a load on one side of the interface and measuring the response on both sides of the interface. However, the instrumentation layout and testing layout need to be adjusted to accommodate the different widening types. The specifics of these adjustments are discussed in Section 3.2 and Section 3.3 of this report.

2.2 TEST LOADING

To calculate the "K" factor, the measured response will be compared directly to the calculated response. Therefore, the test truck should closely match a truck configuration used in the rating process. The testing program used a standard tri-axle dump truck with an axle spacing similar to the Type 3 legal load truck. It is important to note that SCDOT uses a modified Type 3 truck, as compared to the standard AASHTO Type 3 truck. However, the only difference is the axle load proportions so it does not affect the test loading. Figure 5 shows a standard tri-axle dump truck.

Figure 5: Standard Tri-Axle Dump Truck

When calculating "K" factors, the weight of the test truck is used to determine K_b. AASHTO MBE Table 8.8.2.3.1-1, as shown in Figure 6, provides guidance on how to determine K_b, based on two criteria:

- Can member behavior be extrapolated to 1.33W, where "W" is defined as the unfactored gross rating load effect.
- Magnitude of the Test Load (T) relative to the magnitude of the controlling vehicle in the load rating (W).

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

Figure 6: AASHTO MBE Table 8.8.2.3.1-1

T and W as shown in Figure 6 represent the Test Truck Weight and Controlling Rating Effect, respectively. As the bridges that were included in this portion of the load testing, as well as the bridges that the findings may be applied to, have varying structural attributes (span length, material properties, beam size, reinforcement layout, etc.) the linearity of the measured response will vary. The linearity determines whether the test results can be extrapolated to 1.33W. Though this extrapolation may be possible for some of the bridges that were tested, it is conservative to assume it will not be possible for all bridges included in this program. Therefore, the only non-zero value of K_b would require a T/W ratio > 0.7. Thus, the Test Truck Weight (T) was based on the 25 Ton total weight of the Type 3 truck.

To maintain a T/W ratio not less than 0.7, the required Test Truck Weight (T) is 0.7 x 25 = 17.5 Tons. All test trucks were weighed prior to their first use for testing and after their final use for testing. All test truck weights were greater than the minimum required 17.5 Tons, and no significant changes in weights were noted throughout the use of an individual truck. Table 1 shows the Test Truck Weights that were used, as well as the respective T/W ratio and which bridges were tested with each weight.

Table 1: Test Truck Weights

TEST TRUCK WEIGHT, TONS	T/W RATIO	BRIDGES TESTED
18.64	0.75	320, 745, 1052, 1123, 1272, 1758*, 2112, 2827
17.88	0.72	398, 428, 877, 1276
17.99	0.72	403, 404
18.35	0.73	347, 568, 580, 627, 640, 1036, 1758*, 1836, 1856, 2067, 2133, 2610

*BR1758 Was tested twice due to traffic control constraints

2.3 BRIDGE SELECTION

The overall intent of the testing program was to apply the findings of the testing to as many structures as possible in SCDOT's inventory. However, with the level of variance in structural attributes, it was known that the findings may not be applicable to all RC Tee-Beams in SCDOT's inventory. Therefore, bridges selected for testing were all priority bridges for SCDOT, where a priority bridge is defined as any bridge carrying a US Route, Interstate, or SC Route. Selection was performed in this manner to ensure that if the findings are not applicable to all other RC Tee-Beam structures, the findings will still be able to address the priority bridges that were directly tested.

Though certain bridges were prioritized, the bridge selection was still made with the intent of applying the findings to the remaining RC Tee-Beam structures in SCDOT's inventory. Therefore, bridges selected for load testing were geographically distributed throughout the state. Furthermore, to ensure that each widening type will be sufficiently represented in the testing data, a minimum of five bridges were selected to represent each widening type. The only exception to this is for structures widened with steel girders. There are significantly less RC Tee-Beam structures with this type of widening when compared to the other three widening types, and it is not possible to identify five bridges with adequate access to install instrumentation. To the extent of WSP's knowledge, there are only seven RC Tee-Beam bridges with steel girder widenings in South Carolina. Of these bridges, only three bridges were found to be candidates for load testing. Though this is less than the five-bridge minimum used for other widening types, it represents approximately 42% of the population of RC Tee-Beam bridges with steel girder widenings in South Carolina.

Ultimately, twenty-five RC Tee-Beam structures were selected for load testing. Figure 7 shows the location of the bridges that were load tested and Table 2 provides a brief description of each bridge that was load tested.

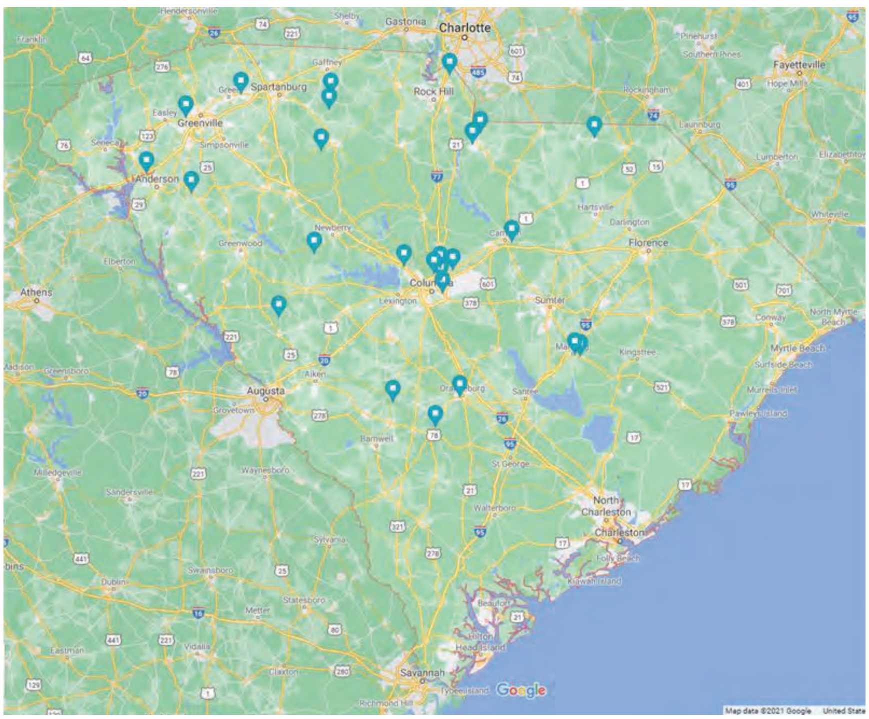


Figure 7: Location of Load Tested RC Tee-Beam Bridges

Table 2: RC Tee-Beam Bridges Selected for Load Testing

BRIDGE ID	COUNTY	WIDENING TYPE
320	Anderson	RC Tee-Beam
347	Lancaster	Steel Girder
398	Chesterfield	AASHTO Girder
403	Clarendon	Slab
404	Clarendon	Slab
428	Kershaw	RC Tee-Beam
568	Newberry	AASHTO Girder
580	Union	AASHTO Girder
627	Cherokee	AASHTO Girder
640	Union	AASHTO Girder
745	Richland	Steel Girder
877	Lancaster	RC Tee-Beam
1036	Edgefield	None
1052	Greenville	RC Tee-Beam
1123	Richland	RC Tee-Beam
1272	Spartanburg	Slab
1276	York	RC Tee-Beam
1758	Richland	Steel Girder
1836	Richland	Slab
1856	Bamberg	RC Tee-Beam
2067	Orangeburg	RC Tee-Beam
2112	Anderson	RC Tee-Beam
2133	Barnwell	None
2610	Richland	Slab
2827	Richland	None

3 SUMMARY OF LOAD TESTING

3.1 EQUIPMENT

A combination of strain sensors and displacement sensors were used to measure the structural response of all RC Tee-Beam bridges tested in this program. All sensing equipment used in load testing is manufactured by Bridge Diagnostics Inc. (BDI).

To measure strain, BDI ST350 Strain Transducers were installed on the RC Tee-Beams as well as widening components (i.e. reinforced concrete slabs, AASHTO Girders and steel beams). These strain transducers are durable, re-usable strain gauges that can be mounted to different materials. For use on concrete surfaces, an extension bracket is used to increase the gauge length of the sensor.

To measure displacement, deflectometers were used. A deflectometer consists of a steel plate on a mounting bracket. The steel plate has a weldable foil strain gauge attached to its underside. The mounting brackets were secured to the bottom of the structural component being instrumented and the steel plate was anchored to a fixed point below the deflectometer using a chain. As the bridge deflects, the strain change in the steel plate is measured and converted to a displacement.

Figure 8 shows a ST350 Strain Transducer (left) with an extension bracket installed next to a deflectometer (right) on an exterior RC Tee-Beam of Bridge 2827.

Figure 8: Installed Instrumentation BR 2827

Data was captured using BDI STS4 data acquisition hardware for all the load testing that was performed. The BDI STS4 data acquisition system is a two-part system that consists of battery powered nodes that are wired to the sensing instruments. These nodes then communicate with a battery powered base station that utilizes a wireless access point to transmit to an onsite computer where data is recorded. Figure 9 and Figure 10 show the wired nodes and the base station, respectively.

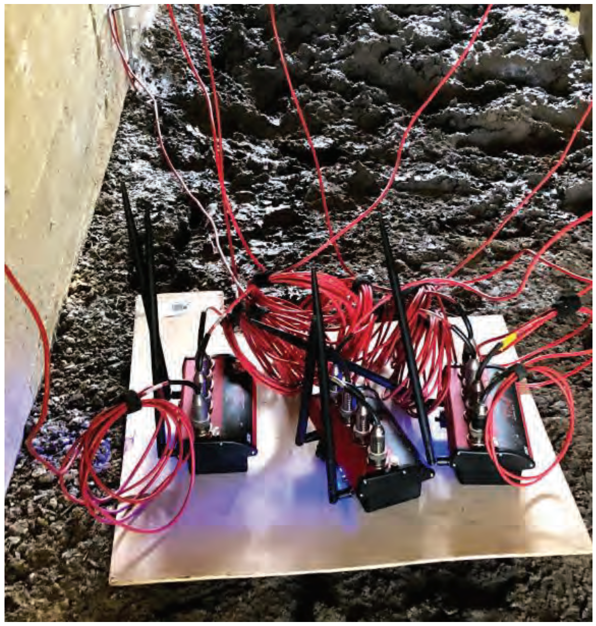


Figure 9: STS Nodes

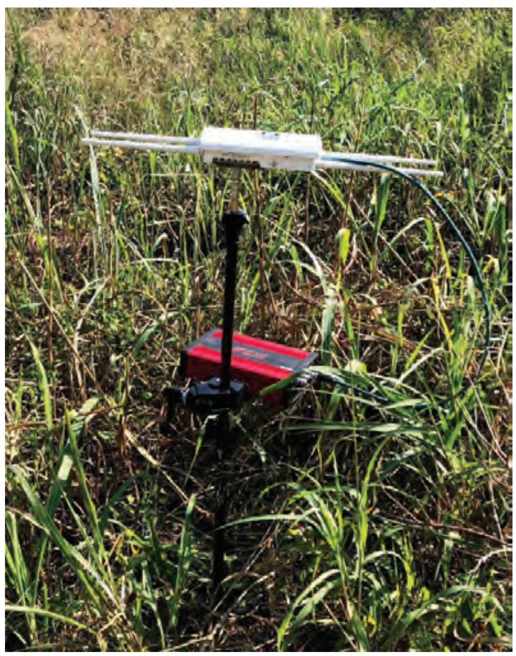


Figure 10: STS Base Station

3.2 INSTRUMENTATION

The number and location of sensors installed on each bridge varied based on each specific bridge's configuration, as well as access constraints. As discussed in Section 1 and Section 2 of this report, the load testing portion of this testing program was primarily aimed at determining "K" factors for RC Tee-Beam bridges controlled by flexure. Therefore, instrumentation was installed at mid-span of each tested RC Tee-Beam to measure the maximum flexural response of the targeted Tee-Beam. Furthermore, this testing program focused on gathering information about the level of load sharing between original sections and widened sections of RC Tee-Beam bridges that have been widened. To capture this load sharing, instrumentation was also installed at mid-span of the nearest primary member of widened sections.

As discussed in Section 3.1 of this report, both displacement and strain sensors were used for this load testing program. Both types of sensors were used to provide redundant information on the measured response of the test bridges. The non-homogeneous properties of concrete may affect the reliability of strain readings. Generally, a larger area of concrete can be included in the measured region by extending the gauge length of a strain sensor, which helps account for the non-homogenous properties of the concrete. For this testing, a 24-inch gauge length was used for all strain gauges installed on concrete surfaces. However, by extending the gauge length, the possibility of spanning a crack is increased, which could artificially increase strain readings in the tension zone. Additionally, with the variability in properties (both geometric and material) of the RC Tee-Beams included in this program, it becomes difficult to predict whether our applied loadings will engage any existing cracks in the RC Tee-Beams or possibly induce new cracking. Due to this, the deflectometers were considered as the primary instrumentation for this program with strain sensors considered as the secondary.

The number of sensors installed on a specific structure varied based on the following factors:

- Access constraints
- Number of beams in the cross-section
- Type of widening

In general, one strain sensor and one deflectometer were installed on each RC Tee-Beam in the cross section. The maximum number of sensors installed on a single bridge was sixteen, which was limited by the number of channels available in the STS system. In the case that a bridge has more than 8 RC Tee-Beams, the install team considered the access constraints and symmetry of construction to determine which RC Tee-Beams would be instrumented. Furthermore, on widened structures with one or more construction joint, instrumentation was typically installed on both sides of all joints as described above. Figure 11 and Figure 12 show representative instrumentation plans for this testing program.

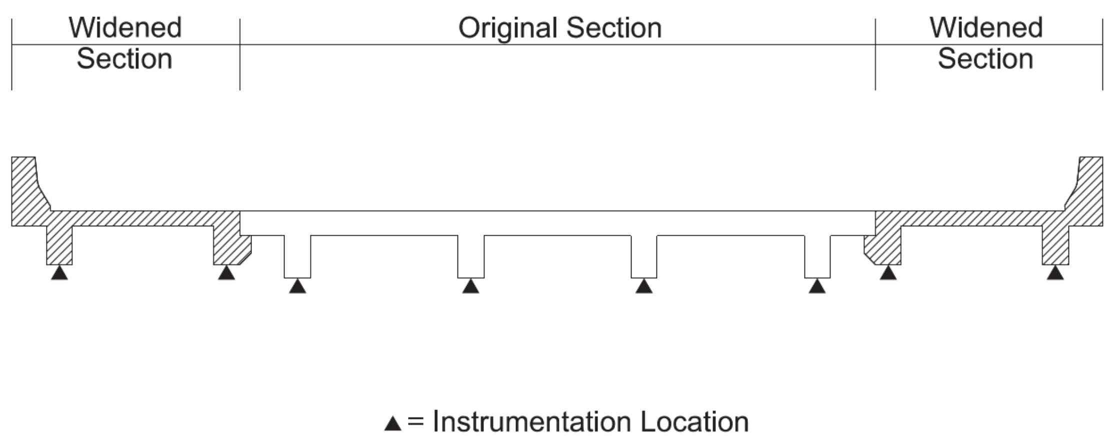


Figure 11: Representative Instrumentation Plan

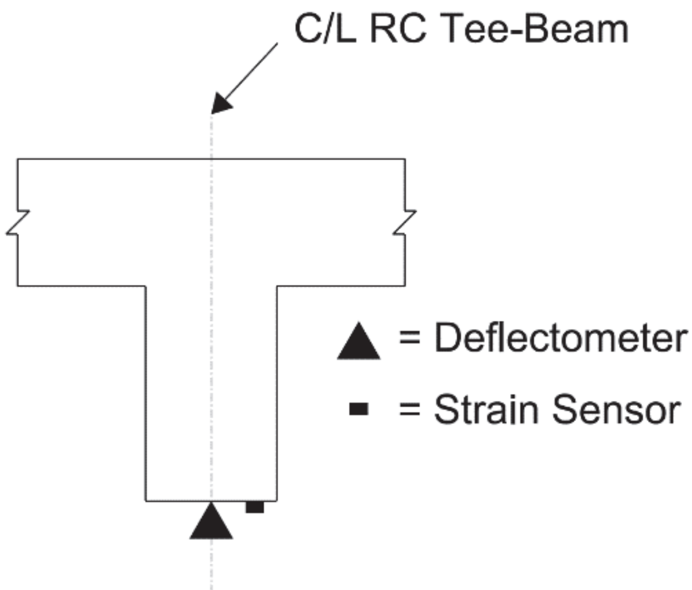


Figure 12: Representative Instrumentation Detail

Table 3 summarizes the instrumentation used on each RC Tee-Beam bridge tested in this program. An asterisk denotes that specific bridge did not specifically follow the representative instrumentation shown in Figure 11 and Figure 12, which was generally due to under-bridge access constraints.

Table 3: Instrumentation Summary

BRIDGE ID	WIDENING TYPE	NO. OF RC TEE- BEAMS	NO OF CONSTRUCTION JOINTS	NUMBER OF DEFLECTOMETERS	NUMBER OF STRAIN SENSORS
320	RC Tee-Beam	6	2	6	6
347	RC Tee-Beam & Steel Girder	6	4	8	8
398	AASHTO Girder	4	2	6	6
403	Slab	4	2	8	8
404	Slab	4	2	8	8
428	RC Tee-Beam	6	2	6	6
568	AASHTO Girder	3	1	4	4
580	AASHTO Girder	4	2	6	6
627	AASHTO Girder	3	2	5	5
640*	AASHTO Girder	3	1	2	2
745	Steel Girder	4	1	5	5
877	RC Tee-Beam	6	2	6	6
1036	None	4	0	4	4
1052	RC Tee-Beam	6	2	6	6
1123*	RC Tee-Beam	22	6	8	8
1272	Slab	4	2	8	8
1276	RC Tee-Beam	8	2	8	8
1758*	Steel Girder	5	2	5	5
1836	RC Tee-Beam and Slab	6	4	8	8
1856	RC Tee-Beam	10	4	8	8
2067	RC Tee-Beam	10	1	8	8
2112	RC Tee-Beam	6	2	6	6
2133	None	4	0	4	4
2610*	Slab	5	2	9	5
2827	None	4	0	4	4

Figure 13 shows the installed instrumentation on Bridge Number 2827, which is a bridge without a widening. This is representative of bridges without widenings and bridges with RC Tee-Beam widenings. Figure 14 shows the installed instrumentation on Bridge Number 568, which is a bridge with an AASHTO Girder widening. Note that instrumentation was installed on all RC Tee-Beams and the AASHTO Girder adjacent to the construction joint. This is representative of the instrumentation for all widened bridges except for RC Tee-Beam widenings.

Figure 13: Installed Instrumentation - BR 2827

Figure 14: Installed Instrumentation - BR 568

3.3 LOAD TEST PROCEDURE

The load test procedure for this load testing program was similar to all other load testing performed under this contract. First, paths were marked on the roadway to guide the test truck over a pre-determined path. Following this, the test truck crossed the instrumented span at a crawl speed (approximately 3-5 mph) on a specific test line. Each test path was tested 3 times, to ensure repeatability of results. All traffic was stopped or shifted for each test performed to ensure the measurements were not influenced by any loading besides the test truck. The load test paths used for this load testing program were determined with two goals:

1. Produce maximum flexural loading on all instrumented RC Tee-Beams
2. Produce maximum loading adjacent to construction joints

Different test truck paths were used to produce maximum flexural loading depending on the location of the targeted RC Tee-Beam. For interior tee-beams, the center of the test truck was in line with the centerline of the RC Tee-Beam. For exterior tee-beams, one wheel line of the test truck was located as close to the centerline of the RC Tee-Beam as possible. Depending on the roadway and curb/barrier geometry, the proximity to the centerline of the exterior RC tee-beam varied. It is important to note that even if the test truck could not be located directly over an exterior RC tee-beam, the loading did represent a critical loading case for that tee-beam as the roadway and curb/barrier geometry will restrict all other traffic just as it restricted the test truck location.

To produce the maximum loading adjacent to a construction joint, the edge of one wheel line of the test truck was located approximately 3" from the centerline of the construction joint. In general, the test truck was located entirely in the original section of the structure. If possible, a similar test would be performed with the test truck located entirely in the widened section of the structure, though this configuration was rare as most widenings were not wide enough to fit the entire test truck. One exception to this loading configuration was for bridges with multiple widenings. If a widening was not wide enough to fit the entire test truck, a decision was made based on roadway constraints and available information on construction joint detailing to determine the exact location of the test truck which will produce the maximum loading on the construction joint under consideration.

Figure 15 and Figure 16 show a representative test truck location to test an interior and exterior RC Tee-Beam, respectively.

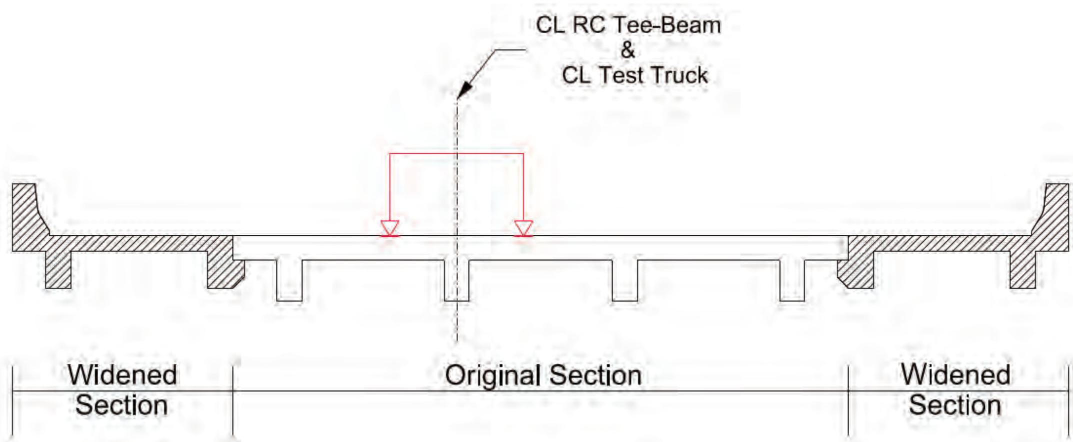


Figure 15: Representative Interior RC Tee-Beam Test Truck Location

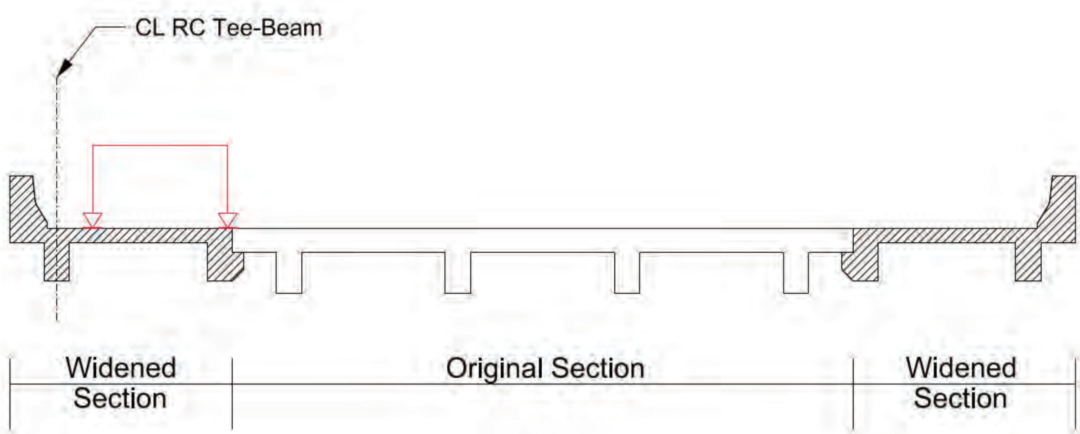


Figure 16: Representative Exterior RC Tee-Beam Test Truck Location

Figure 17 shows a representative test truck location to test a construction joint between the original section and a widened section.

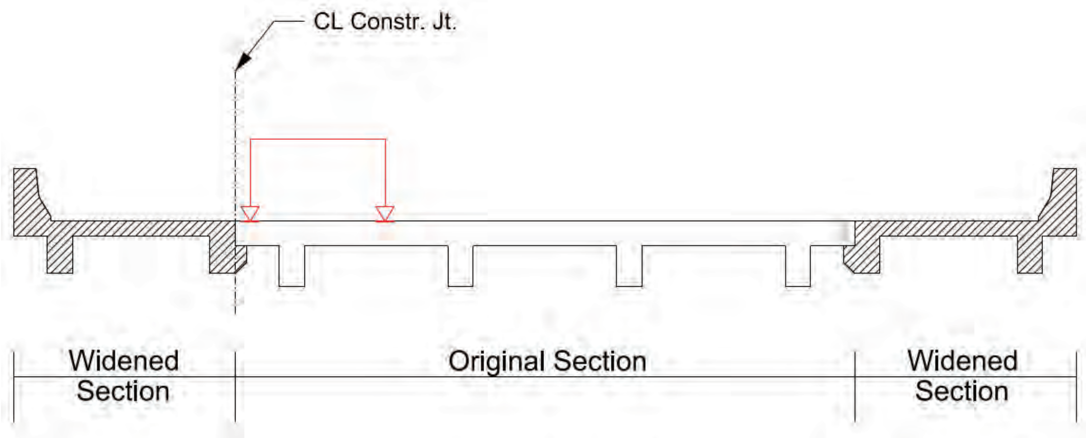


Figure 17: Representative Construction Joint Test Truck Location

The number of tests performed on a bridge varied depending on the number of RC Tee-Beams and the number of construction joints in the cross-section of the bridge. Table 4 summarizes the number of tests performed on each tested bridge.

Table 4: Summary of Tests Performed

NUMBER OF TESTS	BRIDGE ID
2	640
4	568, 1036, 2133, 2827
5	627
6	320, 428, 877, 1758*, 2112
7	745
8	398, 403, 404, 580, 1052, 1272, 2610
10	1123, 1276, 1836, 2067
12	347
14	1856

*Tested in two days

Figure 18 shows a load test being performed on Bridge Number 1123. In this photo, the installed instrumentation can be seen under the bridge and the test truck is in the middle of a test run. A WSP engineer on the right side of the photo is responsible for ensuring that the test truck remains on the pre-determined test path for the duration of the test.

Figure 18: Load Testing of BR 1123

4 RESULTS OF TESTING

4.1 RESULTS OF LOAD TESTING

Time-history data was recorded for both displacement and strain during all tests. In total, 124 individual RC Tee-Beams were tested from the 25 bridges included in this testing program. The collected data makes it possible to compute K-factors for each tested girder and better understand the amount of load sharing through the different widening details. The findings of the testing program are discussed in the following section.

4.1.1 K-FACTOR FINDINGS

As discussed in Section 2 of this report, a K-factor can be calculated in accordance with AASHTO MBE Section 8.8.2.3 based on the measured response from the testing. The equation to compute K is shown below.

$$K = 1 + k_a k_b \text{ where,}$$

$$k_a = {}^{\varepsilon_c}/_{\varepsilon_t} - 1 \text{ where,}$$

ε_c = calculated theoretical response and,

ε_t = response measured in test and,

$$k_b = 0.5 \text{ conservatively, as described in Section 2.2}$$

As can be seen in the above equations, the maximum tested response of each individual RC Tee-Beam is compared directly to the calculated theoretical maximum response to develop the K-factor. As each individual K-factor will be directly applied to that Tee-Beam's respective load rating, it is important to decide the properties used in the theoretical response calculation. Although AASHTO LRFD Design Specifications and MBE guidance can be used as reference for most properties, some load raters may adjust assumptions based on a specific bridge's plans and/or engineering judgement. Furthermore, a preliminary review of existing load ratings indicated that in some instances the input geometry of a specific tee-beam did not match the existing plans. Therefore, a combination of AASHTO guidance, previous rating assumptions, and plan information were used to calculate the theoretical deflections for the RC Tee-Beams tested in this program. For each property, the effect on the original load rating, calculated response, and applicability of findings to the broader RC Tee-Beam population were considered when determining which assumption should be used for the K factor calculation. Table 5 summarizes the assumptions applied in the theoretical response calculations.

Table 5: RC Tee-Beam Property Assumptions

PROPERTY	ASSUMED VALUE CONSISTENT WITH
Concrete Strength, f'_c	Previous Load Rating
Reinforcement Strength, F_y	Previous Load Rating
Tee-Beam Dimensions	Existing Plans
Reinforcement Detailing	Existing Plans
Tee-Beam Effective Width, b_{eff}	Previous Load Rating
Live Load Distribution Factor, LLDF	Previous Load Rating

It is important to note that in general, for interior beams, the tee-beam effective width and LLDF used in the previous load rating was consistent with AASHTO guidance. For exterior beams and beams that were constructed in a widening, the previous load ratings sometimes strayed from the AASHTO guidance to account for nearby construction joints. For this reason, it was determined to use the values assumed in the previous ratings, as the calculated K-factors will account for any conservatism found in the load rater's assumptions. However, if detailing differences were noted, it was determined to remain consistent with plan information. This ensures that theoretical calculations do not artificially increase or decrease the calculated K-factors.

Using the assumed properties shown in Table 5, in conjunction with a theoretical moment calculated for the respective test truck and bridge configuration, allows us to calculate the theoretical strain and deflections for each RC Tee-Beam that was tested. Considering the variability in design and detailing for the RC Tee-Beams included in this testing program, as well as the broader population in SCDOT's inventory, gross-section properties were used for all theoretical calculations. These theoretical values were then compared to the measured response to determine a K_a value and ultimately a K-factor. The use of gross-section properties produces lower theoretical strain and deflection values than the consideration of effective section properties, which ultimately leads to conservative K-factors. A sample K-factor calculation can be found in Appendix C.

Table 6 shows the controlling strain calculated and deflection calculated K-Factor for each bridge tested in this program.

Table 6: Controlling RC Tee-Beam K-Factors

BRIDGE ID	STRAIN CALCULATED K-FACTOR		DEFLECTION CALCULATED K-FACTOR	
	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER
320	4.94	2.24	3.88	1.80
347	1.87	N/A	1.55	N/A
398	1.78	N/A	2.01	N/A
403	1.78	N/A	1.61	N/A
404	1.55	N/A	1.62	N/A
428	2.00	2.32	1.59	1.80
568	1.88	2.32	2.57	2.49
580	1.62	N/A	1.94	N/A
627	1.93	N/A	1.84	N/A
640	N/A	N/A	N/A	N/A
745	2.34	3.04	3.20	3.61
877	2.88	2.88	2.63	2.54
1036	1.84	2.77	1.77	2.05
1052	1.54	4.30	1.60	3.08
1123	1.74	N/A	1.81	N/A
1272	2.06	N/A	2.21	N/A
1276	2.24	3.21	1.96	2.17
1758	2.18	N/A	1.73	N/A
1836	1.70	N/A	1.61	N/A
1856	1.19	N/A	1.12	N/A
2067	1.59	3.40	1.51	2.77
2112	2.20	2.92	1.85	1.73
2133	1.76	1.77	1.72	1.81
2610	2.52	N/A	3.26	N/A
2827	1.59	1.39	1.66	1.21

As can be seen in Table 6, the K-factors calculated using strain data were generally consistent with those calculated using deflection data with an overall trend of the strain-calculated K-factors being slightly higher than the deflection-calculated K-factors. This trend can be attributed to many things, but as discussed in Section 3 of this report, the deflection data is more reliable for this type of concrete structure, and as shown in Table 6, the deflection data typically leads to more conservative K-Factors. As such, all of the following result and discussion will focus on the deflection data. In any instance where abnormal patterns were observed in the deflection data, the strain data was evaluated to better understand and confirm any observations.

Table 7 shows the maximum and minimum controlling K-Factors based on deflections.

Table 7: Maximum & Minimum Controlling K-Factors

RC TEE-BEAM TYPE	MAXIMUM CONTROLLING K-FACTOR	MINIMUM CONTROLLING K-FACTOR
Interior	3.88	1.12
Exterior	3.61	1.21

As many of the bridges included in this program have been widened, some discussion is warranted on the definition of an interior and exterior RC Tee-Beam. For the purposes of this report, an exterior Tee-Beam is the one that is the exterior structural element in the current bridge configuration and an interior Tee-Beam is any other Tee-Beam. Therefore, an RC Tee-Beam that was an exterior beam in the bridge's original configuration will be considered as an interior beam after widening. Per this definition, of the 124 RC Tee-Beams that were tested in this program, 103 were interior beams and 21 were exterior beams. K-factor data was grouped in several different methods to identify any trends.

Figure 19 through Figure 21 show the K-factor histogram plots with a 0.5 range of bins for all RC Tee-Beams, interior RC Tee-Beams, and exterior RC Tee-Beams, respectively.

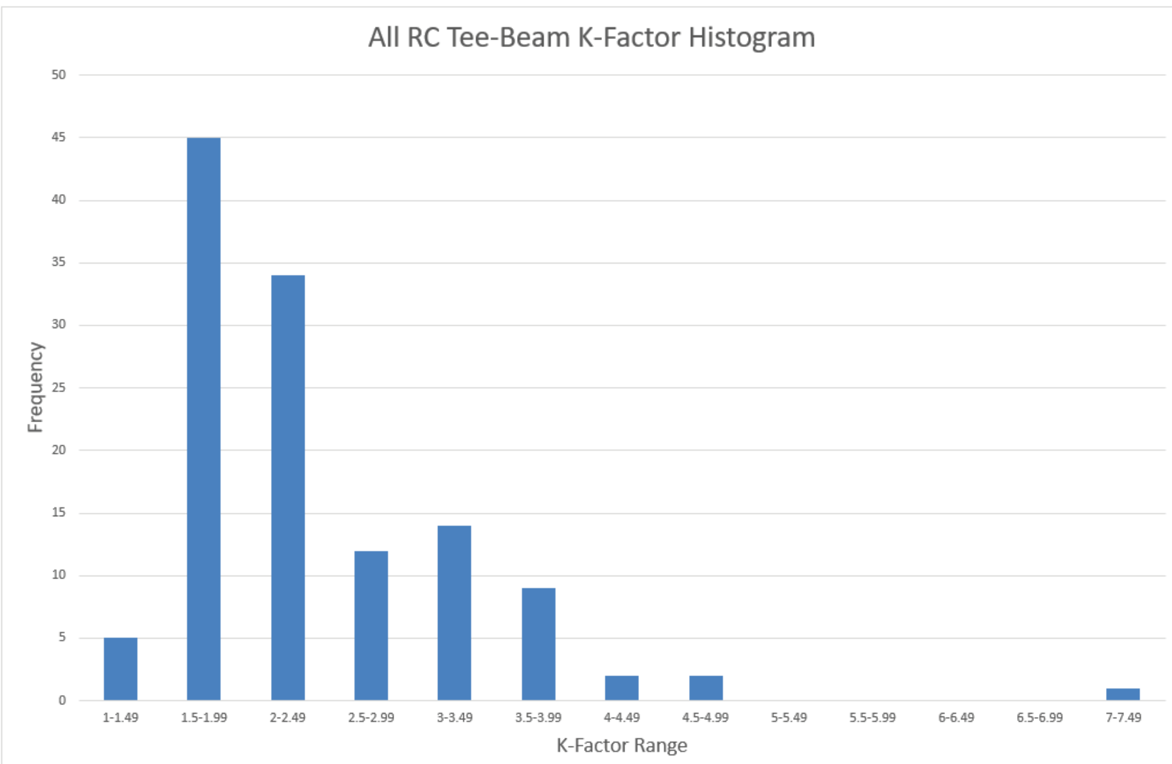


Figure 19: All RC Tee-Beam K-Factor Histogram

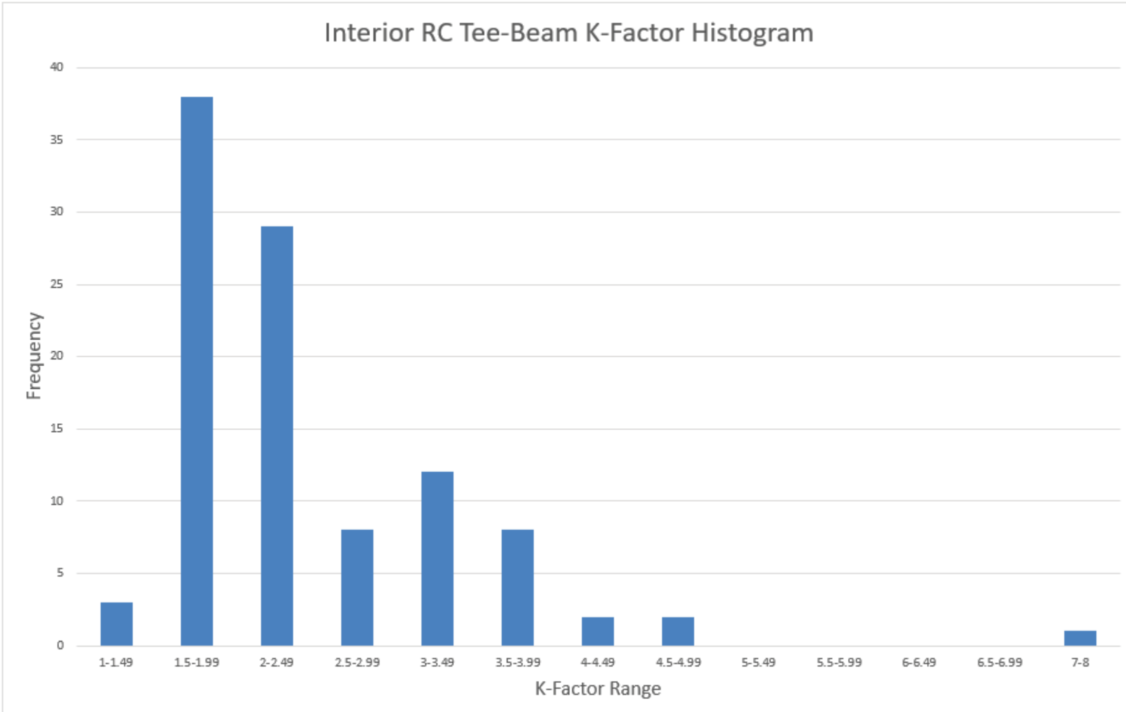


Figure 20: Interior RC Tee-Beam K-Factor Histogram

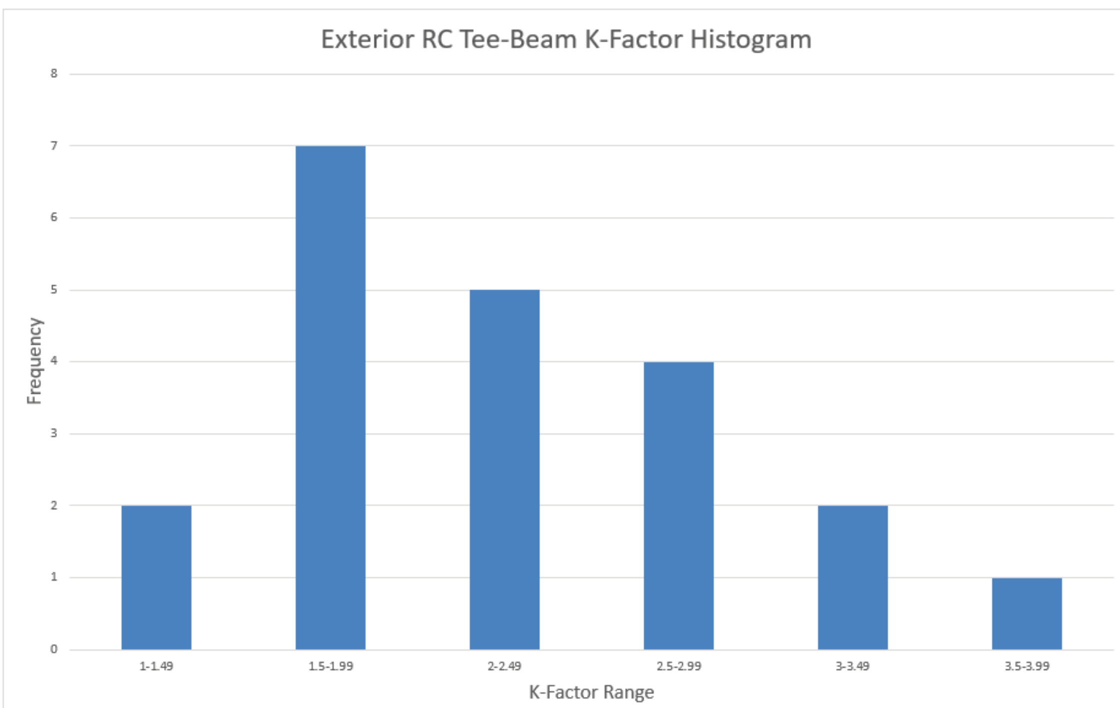


Figure 21: Exterior RC Tee-Beam K-Factor Histogram

As discussed previously, ensuring that the correct load rating assumptions are accounted for in calculating the RC Tee-Beam K-factors is extremely important. This is because the assumptions made in load rating the RC Tee-Beams can have a significant impact on the load rating of individual RC Tee-Beams. Specifically, the assumptions made by load raters in the vicinity of construction joints can affect both the loading on the individual RC Tee-Beams, as well as the calculated capacity. Review of the most recent load ratings for the 25 bridges included in this testing program indicated that there were four general assumptions regarding the live load distribution and the effective width (b_{eff}) of the RC Tee-Beams adjacent to construction joints, as shown in Table 8.

Table 8: Load Rating Assumption Combinations

LIVE LOAD DISTRIBUTION	RC TEE-BEAM EFFECTIVE WIDTH
Continuous Through Joint	Continuous Through Joint
No Sharing Through Joint	Continuous Through Joint
No Sharing Through Joint	No Sharing Through Joint
Continuous Through Joint	No Sharing Through Joint

When considering the potential differences between the assumptions shown in Table 8, the RC Tee-Beam Effective Width assumption cause a negligible difference in the load rating. If the construction joint is assumed to act as a monolithic deck section, AASHTO guidance results in an effective width controlled by ½ of the girder spacing. If the construction joint is assumed to have no sharing, the effective width is extended to the construction joint. Generally, this results in several inches of difference in the effective width and does not impact the load rating significantly. However, the effect of the assumed live load distribution can have significant impacts on the load rating. In this case, if the construction joint is assumed to act as a monolithic deck section, the load is shared to adjacent RC Tee-Beams or structural elements. However, if the joint is assumed to transfer no live load, the girder nearest to the joint is expected to carry a much larger portion of the live load. Considering the potential impacts of these assumptions on the load rating, and on the resulting K-Factors, the K-Factors calculated from this load testing program were also grouped in the following three classes:

- RC Tee-Beams not adjacent to a construction joint
- RC Tee-Beams adjacent to a construction joint, with continuous live load distribution assumed through the joint
- RC Tee-Beams adjacent to a construction joint, with no sharing of live load assumed through the joint

Of the 124 RC Tee-Beams tested in this program, 61 are adjacent to a construction joint. Among them, 42 were assumed to have a continuous live load distribution through the construction joint in the most recent load ratings.

Figure 22 through Figure 24 show the K-factor histogram plots for all RC Tee-Beams included in this testing program based on their proximity to construction joints and live load distribution assumptions used in the load rating. These histogram plots group the RC Tee-Beams into K-factor "bins" with 0.5 ranges.

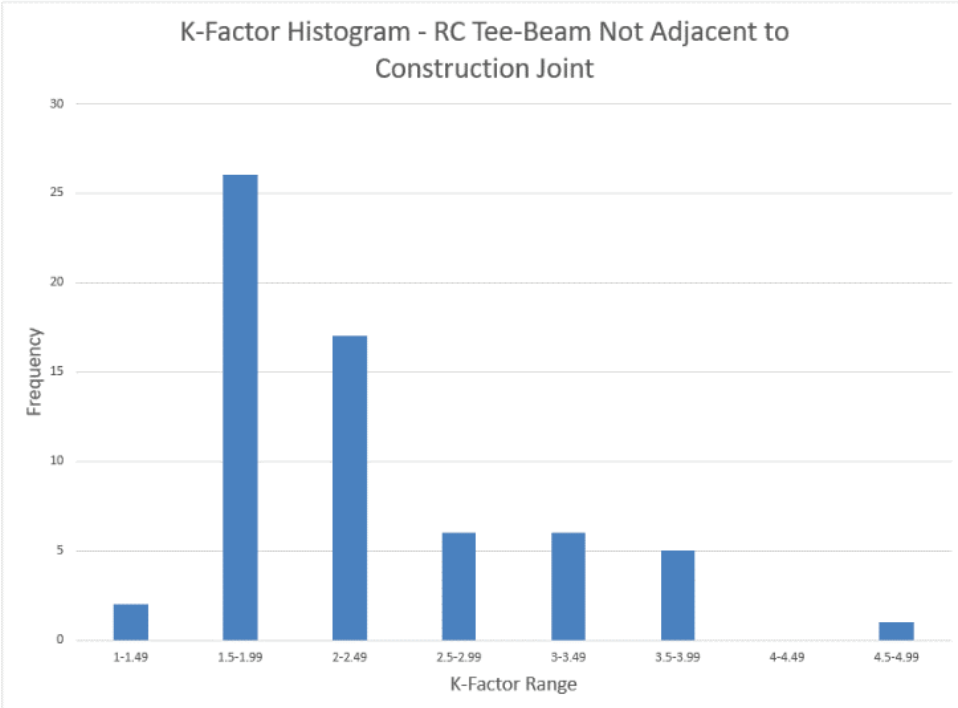


Figure 22: K-Factor Histogram - RC Tee-Beams Not Adjacent to a Construction Joint

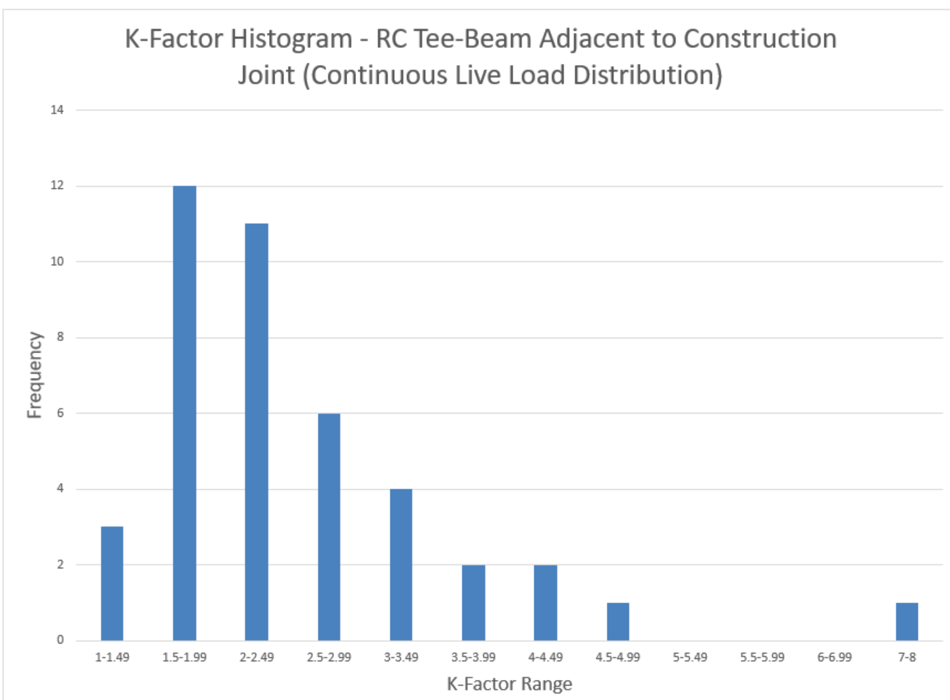


Figure 23: K-Factor Histogram - RC Tee-Beams Adjacent to a Construction Joint (Continuous Live Load Distribution)

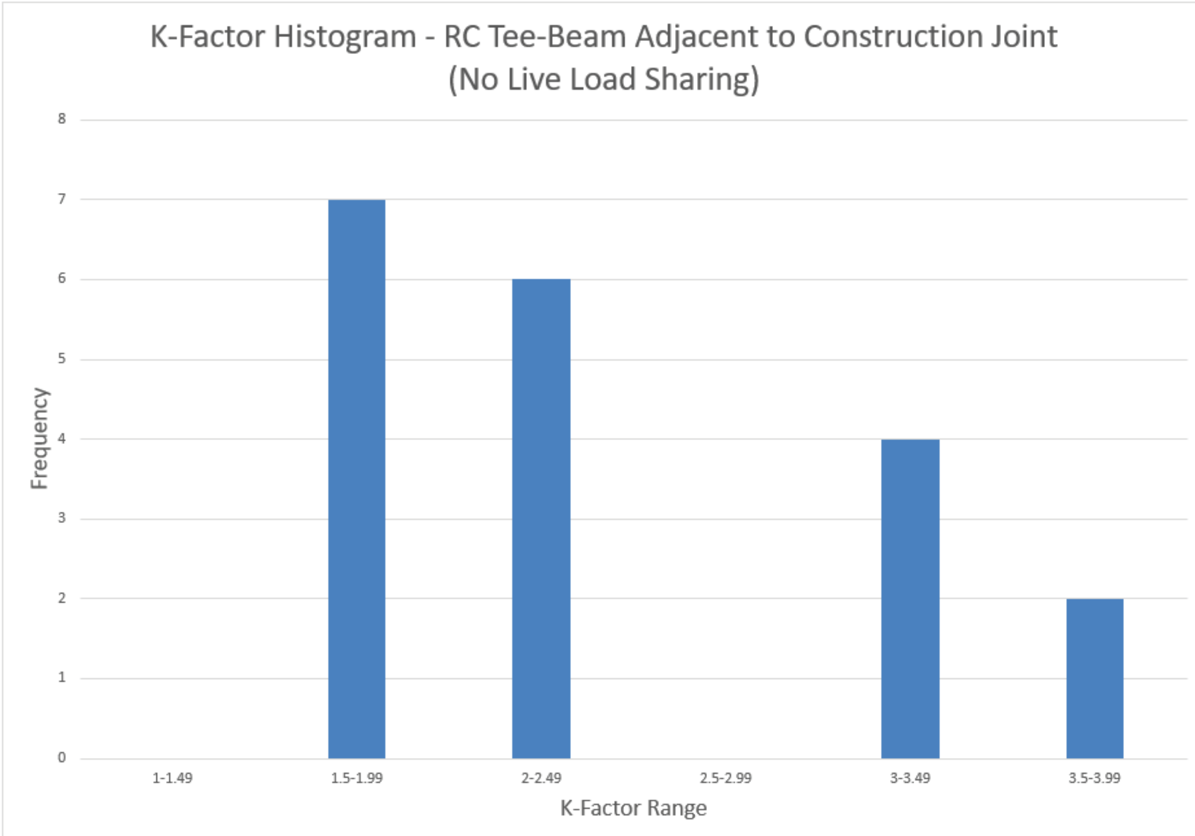


Figure 24: K-Factor Histogram - RC Tee-Beams Adjacent to a Construction Joint (No Live Load Sharing)

4.1.2 LOAD SHARING THROUGH CONSTRUCTION JOINTS

Data collected during this load testing program indicates that the level of load sharing through construction joints between original RC Tee-Beam sections and widened sections is extremely variable. This is to be expected as many attributes of an individual structure will determine the amount of load that will pass through the construction joint, including:

- Construction Joint Detailing (mechanical connection, bearing connection, friction connection, etc...)
- Existence of diaphragms
- Girder Spacing (original and widened)
- Girder Stiffness (original and widened)
- Roadway surface

Table 9 provides general information on all construction joints that were tested in this testing program.

Table 9: Summary of Construction Joints

WIDENING TYPE	BRIDGE ID	ORIGINAL BEARS ON WIDENING?	DIAPHRAGMS?	MECHANICAL CONNECTION?
RC Tee-Beam	320	No	Yes	Tie Rods
	347	No	Yes	Tie Rods
	428	No	Yes	Rebar & Anchor Bolts
	877	Yes	No	Anchor Bolts
	1052	Yes	Yes	Tie Rods
	1123	No	Varies	Varies
	1276	Yes	No	No
	1836	Yes	No	Anchor Bolts
	1856	Yes	No	Varies
	2067	No	No	Rebar
	2112	Yes	No	Anchor Bolts
Slab	403	Yes	No	No
	404	Yes	No	No
	1272	Yes	No	Rebar
	1836	Yes	No	No
	2610	No	No	No
AASHTO Girder	398	No	No	No
	568	No	No	No
	580	No	No	No
	627	No	No	No
	640	No	Yes	Rebar & Tie Rods
Steel Girder	745	No	Yes	Tie Rods
	347	No	No	No
	1758	No	No	No

Considering the amount of variability in the tested construction joints, the most appropriate method to identify the load sharing through the joints is examining simultaneously recorded deflections on either side of the joint during the tests that loaded one side of the joint. Table 10 through Table 13 show the deflections measured on either side of each construction joint tested during this program, grouped by type of widening. In these tables, the simultaneously recorded deflections are reported for each girder adjacent to the specific joint being tested. The deflection ratio represents the amount of deflection measured in each girder as a portion of the total deflection measured in the two girders.

Table 10: Construction Joint Deflection Measurements - RC Tee-Beam Widening Type

BRIDGE ID	JOINT	ORIGINAL GIRDER DEFLECTION, INCHES	WIDENED GIRDER DEFLECTION, INCHES	DEFLECTION RATIO, ORIGINAL TO WIDENED GIRDER
320	Joint 1	-0.0061	-0.0047	56% : 44%
	Joint 2	-0.0127	-0.0076	62% : 38%
347	Joint 2	-0.0124	-0.0072	63% : 37%
	Joint 3	-0.0120	-0.0097	55% : 45%
428	Joint 1	-0.0175	-0.0077	69% : 31%
	Joint 2	-0.0144	-0.0080	64% : 36%
877	Joint 1	-0.0111	-0.0106	51% : 49%
	Joint 2	-0.0123	-0.0098	56% : 44%
1052	Joint 1	-0.0132	-0.0096	58% : 42%
	Joint 2	-0.0118	-0.0095	55% : 45%
1123	Joint 1	-0.0273	-0.0233	54% : 46%
	Joint 2	-0.0172	-0.0156	53% : 47%
1276	Joint 1	-0.0087	-0.0057	60% : 40%
	Joint 2	-0.0091	-0.0078	54% : 46%
1836	Joint 2	-0.0085	-0.0074	54% : 46%
	Joint 3	-0.0112	-0.0099	53% : 47%
1856	Joint 1	-0.0106	-0.0026	80% : 20%
	Joint 2	-0.0135	-0.0093	59% : 41%
	Joint 3	-0.0159	-0.0111	59% : 41%
	Joint 4	-0.0138	-0.0045	76% : 24%
2067	Joint 1	-0.0108	-0.0032	77% : 23%
2112	Joint 1	-0.0147	-0.0089	62% : 38%
	Joint 2	-0.0194	-0.0078	71% : 29%

Table 11: Construction Joint Deflection Measurements - Slab Widening Type

BRIDGE ID	JOINT	ORIGINAL GIRDER DEFLECTION, INCHES	WIDENED GIRDER DEFLECTION, INCHES	DEFLECTION RATIO, ORIGINAL TO WIDENED GIRDER
403	Joint 1	-0.0146	-0.0041	78% : 22%
	Joint 2	-0.0089	-0.0074	55% : 45%
404	Joint 1	-0.0147	-0.0050	75% : 25%
	Joint 2	-0.0108	-0.0068	61% : 39%
1272	Joint 1	-0.0133	-0.0117	53% : 47%
	Joint 2	-0.0141	-0.0082	63% : 37%
1836	Joint 1	-0.0120	-0.0029	80% : 20%
	Joint 4	-0.0169	-0.0039	81% : 19%
2610	Joint 1	-0.0100	-0.0031	76% : 24%
	Joint 2	-0.0108	-0.0039	73% : 27%

Table 12: Construction Joint Deflection Measurements - AASHTO Girder Widening Type

BRIDGE ID	JOINT	ORIGINAL GIRDER DEFLECTION, INCHES	WIDENED GIRDER DEFLECTION, INCHES	DEFLECTION RATIO, ORIGINAL TO WIDENED GIRDER
398	Joint 1	-0.0264	-0.0026	91% : 9%
	Joint 2	-0.0307	-0.0021	94% : 6%
568	Joint 1	-0.0333	-0.0124	73% : 27%
580	Joint 1	-0.0356	-0.0066	84% : 16%
	Joint 2	-0.0300	-0.0069	81% : 19%
627	Joint 1	-0.0440	-0.0058	88% : 12%
	Joint 2	-0.0508	-0.0074	87% : 13%
640	Joint 1	-0.0177	-0.0160	53% : 47%

Table 13: Construction Joint Deflection Measurements - Steel Girder Widening Type

BRIDGE ID	JOINT	ORIGINAL GIRDER DEFLECTION, INCHES	WIDENED GIRDER DEFLECTION, INCHES	DEFLECTION RATIO, ORIGINAL TO WIDENED GIRDER
347	Joint 1	-0.0123	-0.0078	61% : 39%
	Joint 4	-0.0128	-0.0065	66% : 34%
745	Joint 1	-0.0249	-0.0126	66% : 34%
1758	Joint 1	-0.0271	-0.0047	85% : 15%

Some bridges that were included in this testing program may appear in more than one of the above tables. For example, Bridge 347 is in both Table 10 and Table 13. This indicates that Bridge 347 has had two widenings constructed, one with RC Tee-Beams and one with steel girders.

4.2 RESULTS OF CORE TESTING

Concrete cores were taken from a total of 40 RC Tee-Beam bridges for this testing program. Three cores were taken from the original deck section of each of these bridges and sent to Boyle Laboratories LLC. of Charlotte, North Carolina for testing. Each core was broken, and the core strength was calculated in PSI. The average break strength and standard deviation of the three cores from each bridge was then calculated. Recommended concrete strengths were then calculated as the average break strength minus 1.65 standard deviations of the three cores, which represents a 95% one sided confidence interval. Table 14 presents a summary of the recommended concrete strength based on the concrete core results for these 40 bridges.

Table 14: Recommended Concrete Strength based on Core Results

BRIDGE NUMBER	RECOMMENDED CONCRETE STRENGTH, PSI
52	7,152
256	4,304
257	3,742
320*	7,963
347*	5,578
398*	6,831
403*	4,745
404*	4,723
428*	6,436
568*	6,637
577	3,443
627*	5,782
640*	5,772
745*	5,222
819	5,115
877*	1,437
957	5,330
961	3,690
1036*	4,293
1052*	6,741

BRIDGE NUMBER	RECOMMENDED CONCRETE STRENGTH, PSI
1054	4,411
1064	4,705
1123*	3,119
1272*	5,254
1276*	9,500
1624	4,677
1650	5,654
1758*	3,032
1836*	3,948
1856*	5,731
1860	2,634
2067*	5,056
2112*	6,357
2113*	2,457
2133*	5,367
2610*	2,001
2827*	2,061
2957	4,370
3166	3,486
3797	3,853

* Bridge was load tested

5 APPLICATION OF FINDINGS

5.1 K-FACTOR APPLICATION

With 124 individual RC Tee-Beams tested in a wide geographic range, which represent a variety of designs and construction eras, a statistical approach can be taken to apply the findings of this testing program to the broader inventory of RC Tee-Beam bridges in SCDOT's inventory.

Figure 19 through Figure 21 show similar trends in the K-Factors for all tested RC Tee-Beams. In the three plots, there is a clear grouping of RC Tee-Beam K-factors in the 1-3 range, with outliers in the 3 to the maximum computed K-factor of 7.43 range decreasing in frequency. Review of these three figures and the trends they illustrate indicates that the distribution of the K-factors calculated for the tested RC Tee-Beams follows a log-normal distribution. A log-normal distribution is a continuous probability distribution of a random variable whose logarithm is normally distributed. In this case, the random variable would be the RC Tee-Beam K-factor. A log-normal distribution is related to a normal distribution, which is commonly used in statistics, with one major difference. Log-normal distributions eliminate the probability of any negative values. This is consistent with determining the distribution of K-factors, as a negative K-factor is theoretically impossible. Figure 25 and Figure 26 show a normal distribution overlaid on the histogram of all RC Tee-Beam K-Factors and a log-normal distribution overlaid on the same histogram. The normal distribution and log-normal distribution were developed with appropriate defining values, as shown in the figures.

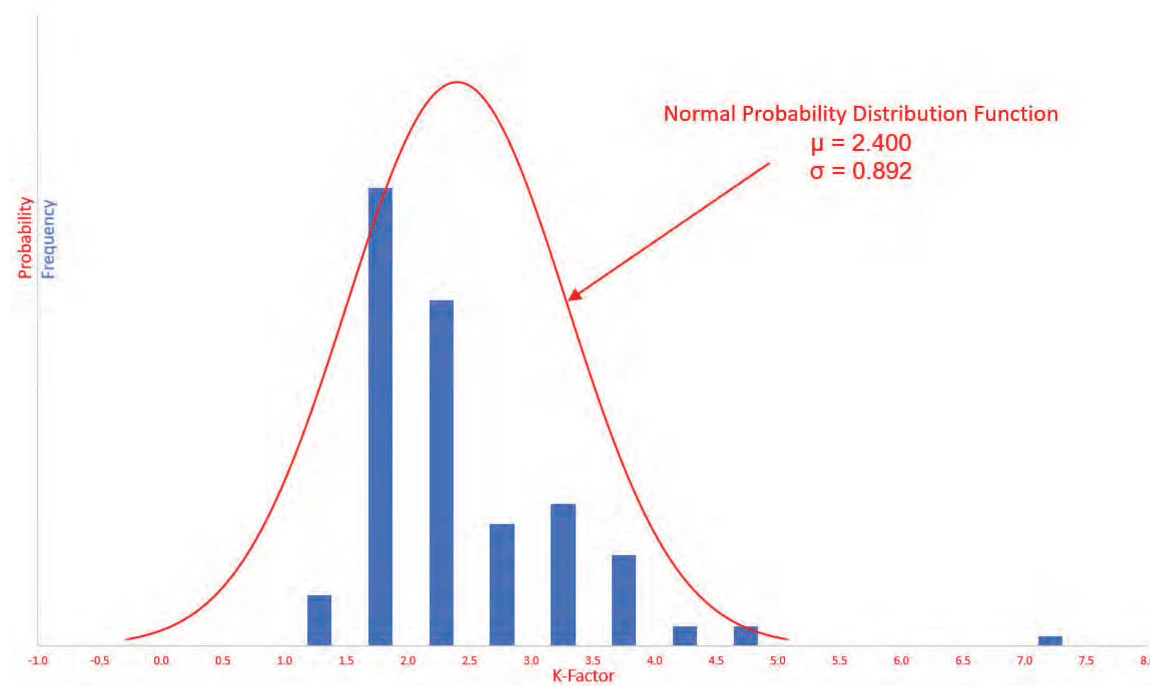


Figure 25: Normal Distribution vs. K-Factor Histogram

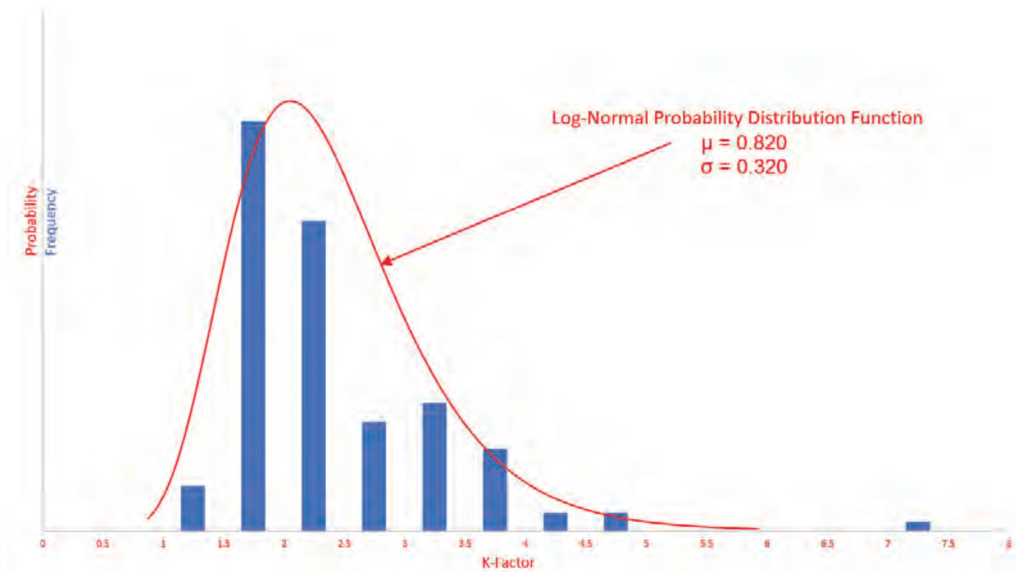


Figure 26: Log-Normal Distribution vs. K-Factor Histogram

It is clear by comparing Figure 25 and Figure 26 that the log-normal distribution is much more representative of the distribution of the calculated K-factors. The log-normal distribution does not include the possibility of negative K-factors and has a very low probability of K-factors that are less than 1.0 which aligns with K-factors that were calculated for the 124 tested RC Tee-Beams, none of which were below 1.0. On the other hand, the normal distribution does produce a real probability of a negative K-factor and a much more significant probability of K-factors less than 1.0. Furthermore, the log-normal distribution does account for the skewed probability of K-factors in the 1.5 to 2.5 range. This also aligns with the K-factors that were calculated for the 124 tested RC Tee-Beams, while the normal distribution does not predict the greater probability of the K-values in the same range.

Though it is clear that the K-factors calculated for the RC Tee-Beams included in this test program display a log-normal distribution, more discussion is warranted on the higher calculated K-factors. Though the K-factors have been calculated in accordance with AASHTO MBE guidance, the possibility of becoming unconservative may become a concern when using higher K-factors. As a precaution, to ensure that conservatism is maintained in the load ratings it may be prudent to "cap" the K-factors at a reasonable level. Table 15 shows the percent of calculated K-factors represented in each histogram "bin" as well as the cumulative number of calculated K-factors at each "bin" level, when considering all RC Tee-Beams tested in this program.

Table 15: Percent of K-Factors in Bin Ranges

BIN RANGE	% OF K-FACTORS IN BIN	CUMULATIVE % OF K-FACTORS
1-1.49	4%	4%
1.5-1.99	36%	40%
2-2.49	27%	68%
2.5-2.99	10%	77%
3-3.49	11%	89%
3.5-3.99	7%	96%
4-4.49	2%	98%
4.5-4.99	2%	99%
5-5.49	0%	99%
5.5-5.99	0%	99%
6-6.49	0%	99%
6.5-6.99	0%	99%
7-7.49	1%	100%

As can be seen in Table 15 approximately 77% of calculated K-factors are less than or equal to 3.0. Considering this, "capping" all calculated K-factors at 3.0 will introduce more conservatism into the application of K-factors to the broader population, while still accurately representing the tested population. To do this, our K-factor equation can be modified as follows:

$$K = 1 + k_a k_b \ \leq 3.0$$

If a calculated K-factor is greater than 3.0, it will be considered an outlier and not included in the log-normal distribution. Removing these outliers leaves a sample size of 96 RC Tee-Beams. The 28 RC Tee-Beams with K-factors greater than 3.0 were investigated for a common trend. These 28 RC Tee-Beams came from 11 different bridges. It was observed that core breaks for all 11 of these bridges indicated the presence of much higher strength concrete than what was assumed in the original load ratings, and thus used to calculate the K-Factors. In fact, at least one core from all eleven of these bridges had a 50% higher compressive strength than what was used in the original load ratings and at least one core from 10 of the 11 bridges had more than 200% higher compressive strength than what was used in the original load ratings. This may not be obvious from reviewing Table 14, as the recommended concrete strengths in that table are based on a statistical confidence level and in some instances (where the standard deviation was high) are below all tested concrete strengths. However, it does present a reasonable justification for the high K-factors calculated in this program. It is important to note that as these K-Factors were calculated using the assumed values from the existing load ratings they will not produce unconservative results when directly applied to those load ratings. However, when considering the application of these K-Factors to the broader population of RC Tee-Beams in South Carolina it is conservative to assume that overly conservative concrete strengths will not always be used in the load ratings. Ultimately, this is why a statistical approach is used when recommending a concrete strength based on testing.

Figure 27 shows a K-factor histogram plot for all RC Tee-Beams with a calculated K ≤ 3.0. Again, this histogram plot groups the RC Tee-Beams into K-factor "bins" with 0.5 ranges.

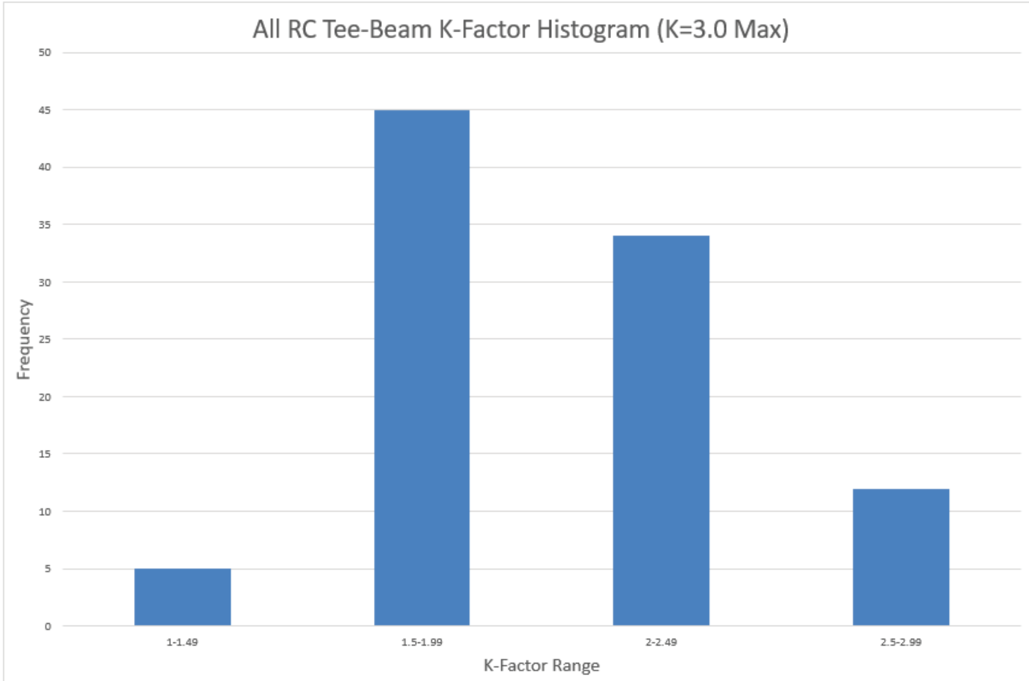


Figure 27: All RC Tee-Beam K-Factor Histogram (K=3.0 Max)

Figure 28 shows the log-normal distribution overlaying the above histogram. The log-normal distribution shown in Figure 28 was developed with statistical parameters developed from the 99 RC Tee-Beams where K ≤ 3.0.

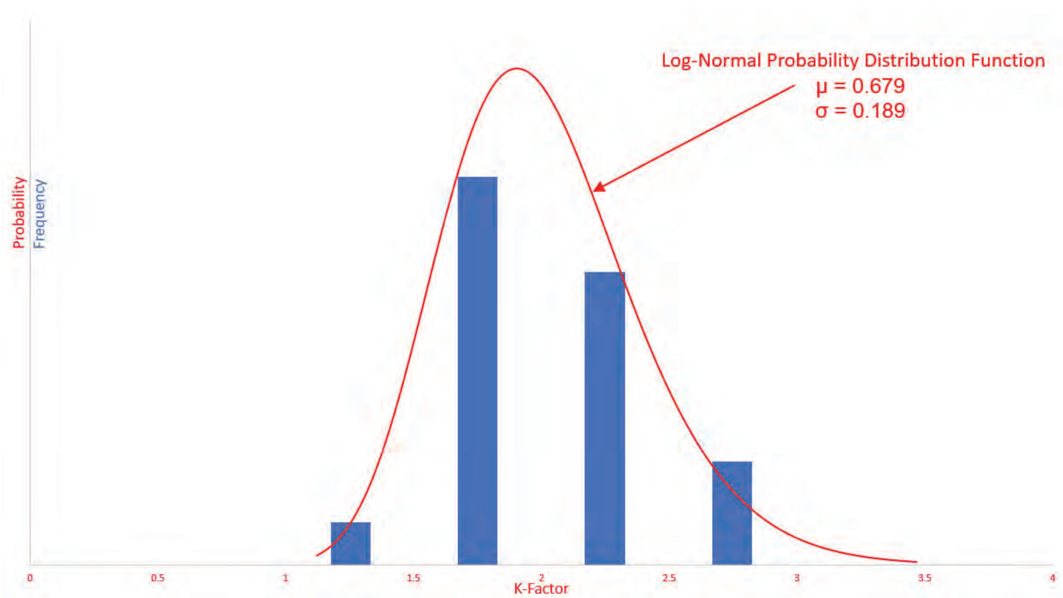


Figure 28: Log-Normal Distribution vs. All RC-Tee Beam Histogram (K=3.0 Max)

Figure 28 shows that the exclusion of K-Factor values greater than 3.0 does not change the fact that the calculated K-Factors follow a log-normal distribution. It does however reduce the probability of a higher K-Factor return, which have been mainly driven by conservative assumptions (low f'c values). The log-normal distribution shown in Figure 28 makes it possible to develop confidence levels for predicted K-Factor ranges that will align with the data collected from this load testing program. Table 16 provides a summary of several standard statistical confidence levels for all RC Tee-Beams tested in this program. The confidence levels shown in Table 16 are one sided in order to identify a lower bound K-Factor value.

Table 16: K-Factor Confidence Levels – All RC Tee-Beams (K = 3.0 Max)

CONFIDENCE LEVEL	MIN K-FACTOR LIMIT
84.10%	1.64
97.75%	1.36

Table 16 is useful in understanding the overall performance of all RC Tee-Beams included in this testing program. However, as the calculated K-factors are heavily influenced by the assumptions that were made in load rating, it is important that those assumptions are considered when applying the findings of this program to the larger population of SCDOT's RC Tee-Beam bridges.

It is important to note that the sample size of RC Tee-Beams adjacent to a construction joint with no live load sharing through the joint assumed was much lower than the other two classifications. However, considering the information collected, a log-normal distribution is still most representative of the findings. When considering the proximity of construction joints and the assumed live load distribution through those joints, all RC Tee-Beam classifications include calculated K-Factors greater than 3.0. Therefore, capping the K-Factors at 3.0 for each class is still appropriate to maintain conservatism. Table 17 shows a summary of RC-Tee Beams that fall into each class.

Table 17: Summary of RC Tee-Beam Classes

RC TEE-BEAM CLASSIFICATION	NUMBER OF CALCULATED K-FACTORS		
	Total	≤ 3.0	Percent ≤ 3.0
Not Adjacent to Construction Joint	63	51	81%
Adjacent to Construction Joint – Continuous Live Load Distribution	42	32	76%
Adjacent to Construction Joint – No Live Load Sharing	19	13	68%

Figure 29 through Figure 31 show the log-normal distributions overlaid on the K-Factor histograms for the RC Tee-Beams included in this testing program when considering the proximity to a construction joint and the assumed live load distribution with a maximum possible K-Factor of 3.0.

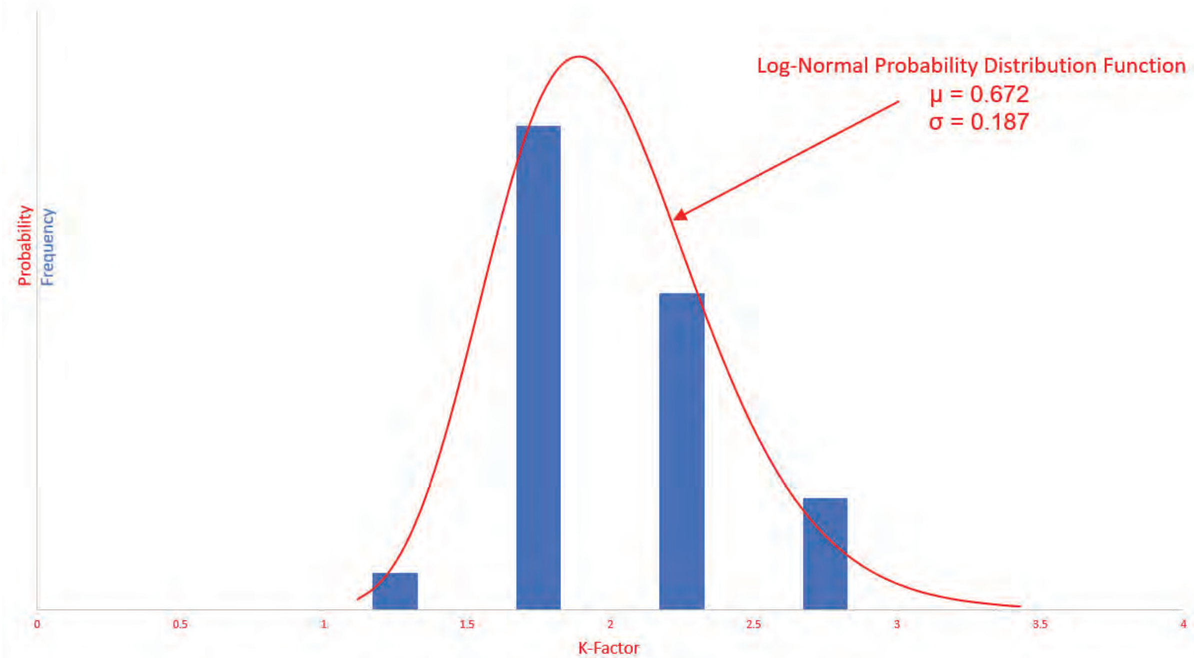


Figure 29: Log-Normal Distribution vs. RC-Tee Beam Histogram (Not Adjacent to Construction Joint; K = 3.0 Max)

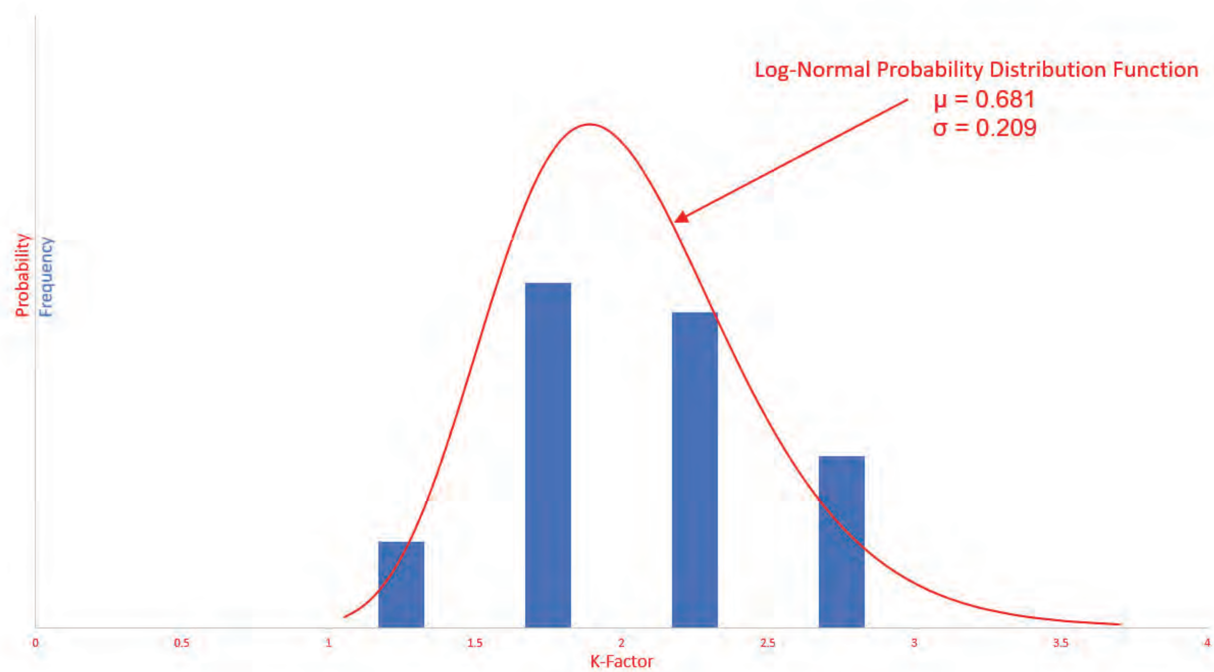


Figure 30: Log-Normal Distribution vs. RC-Tee Beam Histogram (Adjacent to Construction Joint; Continuous Live Load Distribution Assumed; K = 3.0 Max)

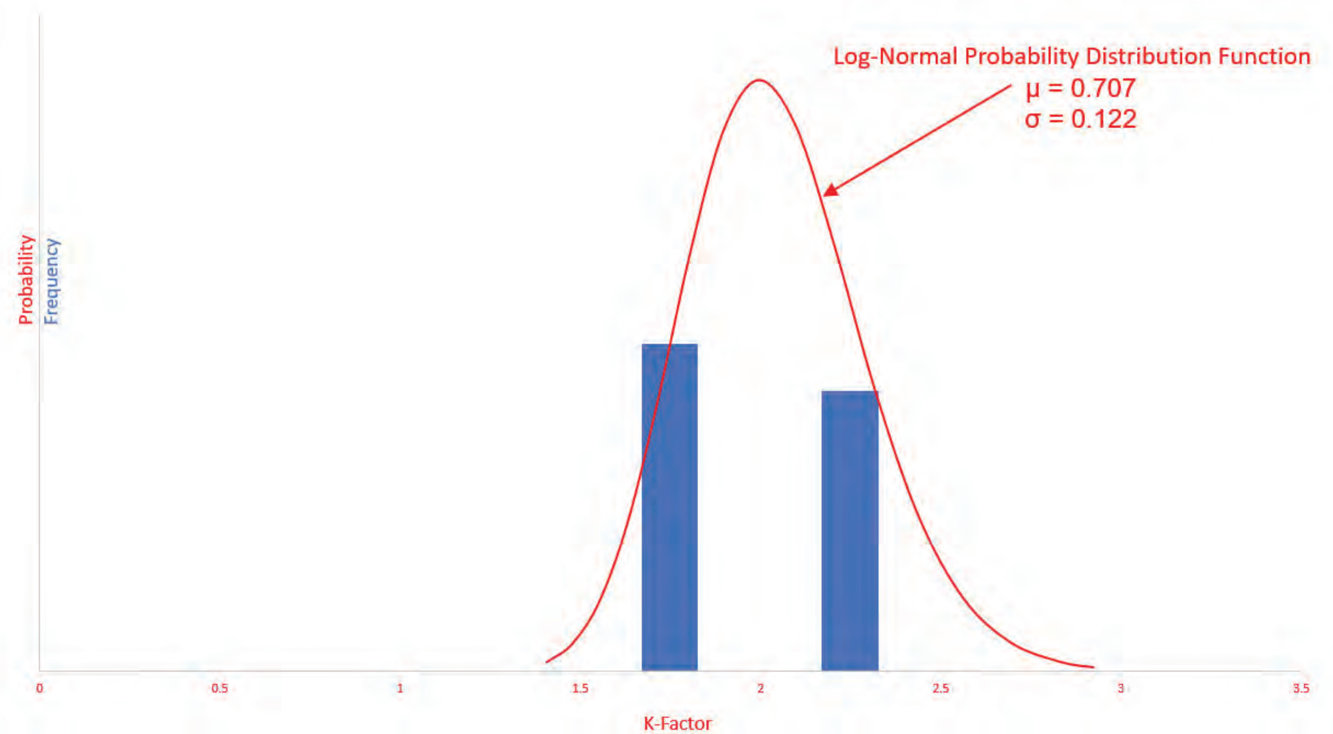


Figure 31: Log-Normal Distribution vs. RC-Tee Beam Histogram (Adjacent to Construction Joint; No Sharing of Live Load Assumed; K = 3.0 Max)

Using the log-normal distributions shown in Figure 29 through Figure 31 we are able to develop K-Factor confidence levels for each RC Tee-Beam class as shown in Table 18. Again, the confidence levels shown in Table 18 are one sided in order to identify a lower bound K-Factor value.

Table 18: K-Factor Confidence Levels per RC Tee-Beam Class

RC TEE-BEAM CLASS	CONFIDENCE LEVEL	MIN K-FACTOR LIMIT
Not Adjacent to Construction Joint	84.10%	1.62
	97.75%	1.35
Adjacent to Construction Joint – Continuous Live Load Distribution Assumed	84.10%	1.60
	97.75%	1.30
Adjacent to Construction Joint – No Live Load Sharing Assumed	84.10%	1.80
	97.75%	1.59

5.2 APPLICATION OF LOAD TRANSFER FINDINGS

The construction joint testing of this load testing program produced variable findings. In general, the RC Tee-Beam type widening showed the best load transfer when considering simultaneous deflection of the tee-beams adjacent to the construction joints. This can be observed by comparing the deflections summarized in Table 10 through Table 13.

It is important to note that though the RC Tee-Beam widenings showed the most consistent load transfer, there were RC Tee-Beam type widenings that showed little load transfer through the construction joint (Joint 1 of Bridge 2067, Joint 2 of Bridge 2112, Joint 1 of Bridge 1856, and Joint 4 of Bridge 1856). Moreover, joints with identical detailing on the same bridge sometimes showed varying levels of load transfer through the joints (Joints 1 & 2 of Bridge 320). This is also true for other types of widenings, such as the slab widening of Bridge 403 where Joint 1 showed little to no load transfer while Joint 2 showed good load transfer through the joint, though the detailing is similar for both joints.

In general, it does appear that the design intent of the joint detailing is reflected in the deflection data shown in Table 10 through Table 13. The more rigid details that are clearly intended to transfer load between the original structure and widened structure did show a higher level of load sharing through the recorded deflections, when the less rigid details that may not have intended to transfer load between the original structure and widened structure typically showed a lower level of load sharing through the recorded deflections. Though the rigidity of the detailing may be subjective, a good indicator of the design intent generally seems to be the existence of continuous reinforcement or mechanical anchorages through the construction joint at the deck level. Figure 32 and Figure 33 show representative examples of the different detailing techniques employed at construction joints.

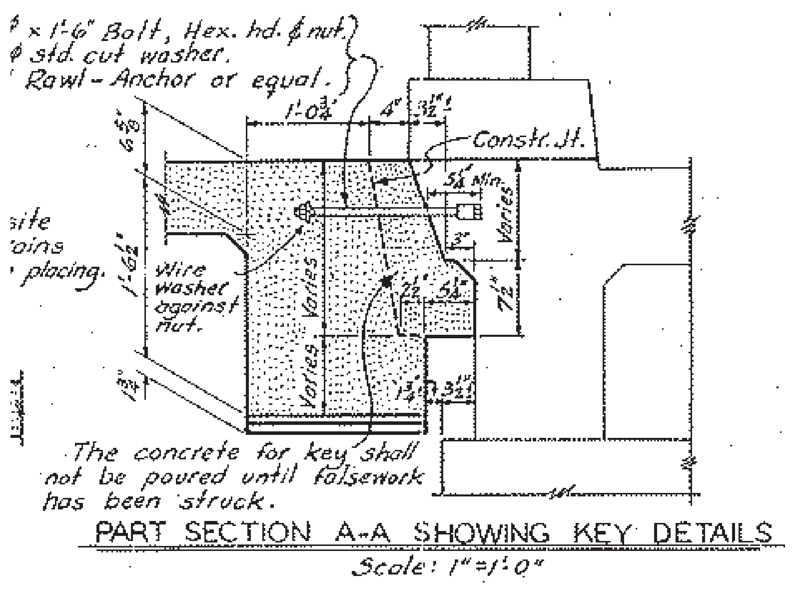


Figure 32: Construction Joint Detailing Example - Mechanical Anchorage Connection

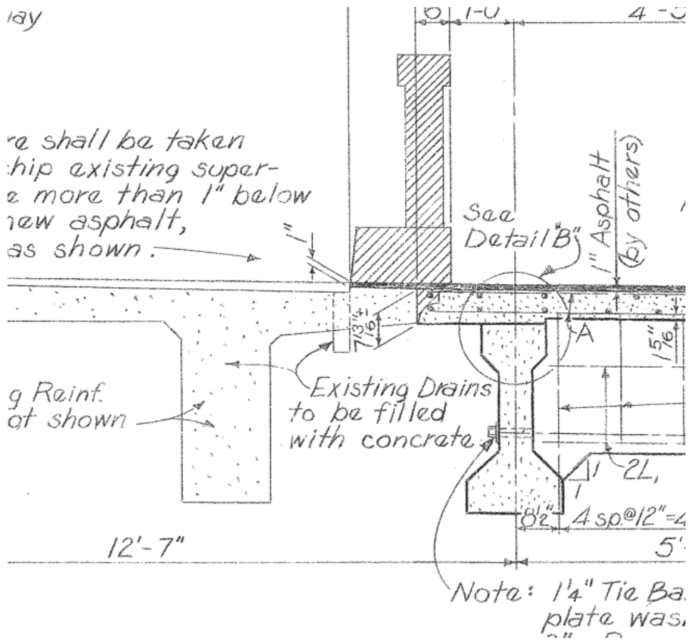


Figure 33: Construction Joint Detailing Example - No Connection

The detail shown in Figure 32 is from the RC Tee-Beam widening constructed on Bridge 1836 in 1951. In this detail, hex headed bolts are used to provide continuity between the original structure and widened structure. In contrast, the detail shown in Figure 33 is from the AASHTO Girder widening constructed on Bridge 627 in 1968. This detail includes no clear load path between the original structure and the widened structure beyond friction at the concrete interface. The recorded deflections from construction joint testing of Bridge 1836 and Bridge 627 (see Table 10 and Table 12) showed minimum deflection ratios of 53% : 47% and 88% : 12%, respectively.

The trend explained above, with respect to the joint detailing, is consistent when comparing the recorded deflections from construction joint testing. However, the conclusions that can be drawn from these observations are binary in nature. Typically, if the design intent of the construction joint was to transfer load, the recorded deflections at either side of the construction joint did show a moderate level of load sharing and if the design intent of the construction joint was not to transfer load, the recorded deflections at either side of the construction joint showed very little (or essentially zero) load sharing. Though there is some load sharing for the latter, this can be attributed to friction between the two concrete surfaces at the construction joint and cannot be relied upon at all levels of loading as the friction may be overcome by higher loads.

Considering the variability of the results of all construction joint testing performed in this testing program, there seems to be no specific guidance that can be provided on the actual level of load sharing of a specific construction joint. Though load sharing can be expected to be present at details with continuous reinforcement or mechanical anchorages passing through the construction joint, the actual level of load sharing cannot be predicted. On the other hand, at details with no continuous reinforcement or mechanical anchorage passing through the construction joint, some load sharing may be present at low load levels, but this cannot be relied upon for all load levels. Ultimately, for the broader population of RC Tee-Beam bridges in SCDOT's inventory, no recommendations can be made for the level of load sharing at a construction joint strictly based on widening type or joint detailing. However, the findings of the construction joint testing can be leveraged to inform assumptions for future load ratings of the specific bridges that were included in this testing program.

5.3 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 for 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 95 percent single sided 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.

6 CONCLUSIONS & RECOMMENDATIONS

6.1 CONCLUSIONS

Several conclusions can be drawn from the findings of the RC Tee-Beam testing program that was performed.

- All RC Tee-Beams included in this testing performed better than theory would predict, producing K-factors greater than 1.0 for all 124 RC Tee-Beams that were directly tested.
- Calculated K-Factors for all RC Tee-Beams included in this testing program exhibited a log-normal distribution, regardless of how the tee-beams were classified or grouped.
- Construction joints with continuous reinforcement or mechanical anchorages passing through the construction joint show higher levels of load transfer through the joint, though the specific level of load sharing is not possible to predict.
- Construction joints with no reinforcement or mechanical anchorages passing through the construction joint show low levels of load transfer through the joint, which can most likely be attributed to friction at the interface. This load transfer mechanism cannot be relied upon for all load levels.

6.2 RECOMMENDATIONS

Based on the conclusions drawn from the RC Tee-Beam testing program, the following recommendations can be made for SCDOT's inventory of RC Tee-Beam Bridges. Please note that all K-Factor recommendations presented herein are only applicable to positive flexure load ratings. These K-Factors cannot be used for negative flexure or shear load ratings.

- K-factors that were calculated for specific RC Tee-Beams, as shown in Appendix B, can be directly applied to the load rating of the respective RC Tee-Beam. If updated concrete strengths are used in future load ratings for any of the RC Tee-Beams included in this testing program, the K-factors shown in Appendix B are no longer valid. Rather the general K-factors presented above should be used based on the proximity of a construction joint and the assumed live load distribution.
- Regardless of the widening type, a K-Factor of 1.35 can be applied to all RC Tee-Beams in SCDOT's inventory that are not adjacent to a construction joint, with a 97.75% confidence level. This includes interior and exterior RC Tee-Beams that are not adjacent to a construction joint.
- Regardless of the widening type, a K-Factor of 1.30 can be applied to all RC Tee-Beams in SCDOT's inventory that are adjacent to a construction joint if the load rating assumes that live load is continuously distributed through the construction joint, with a 97.75% confidence level. This includes interior and exterior RC Tee-beams that are adjacent to a construction joint, regardless of the assumed effective width of the respective Tee-beam.
- Regardless of the widening type, a K-Factor of 1.59 can be applied to all RC Tee-Beams in SCDOT's inventory that are adjacent to a construction joint, if the load rating assumes that live load does not distribute through the construction joint, with a 97.75% confidence level. This includes interior and exterior RC Tee-beams that are adjacent to a construction joint, regardless of the assumed effective width of the respective Tee-beam.
- The level of load sharing shown in Table 10 through Table 13 can be used to inform live load distribution assumptions for future load ratings of the specific structures tested in this testing program. If live load distribution assumptions are changed from the most recent load ratings, the K-factors presented in Appendix B are no longer valid, and rather the general K-factors presented above should be used based on the proximity of a construction joint and the assumed live load distribution.
- Engineering judgement and AASHTO guidance should be used when assuming the level of load sharing through any construction joint not specifically tested in this testing program.

APPENDIX

A EQUIPMENT DATA

Bridge Diagnostics Inc.

We Stand Below Our Work

Partnering with DOTs, design firms & researchers since 1989

STRAIN TRANSDUCER

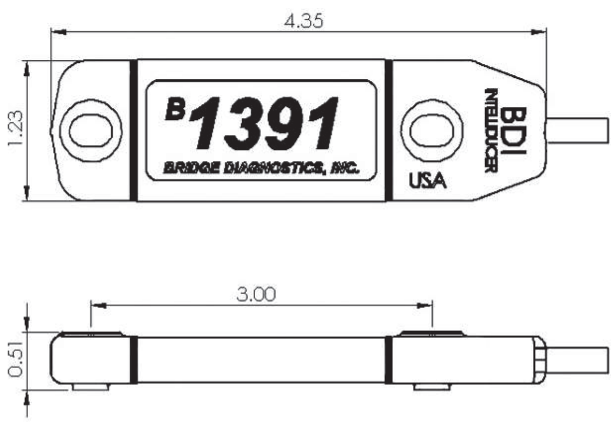
BDI TRANSDUCER

The **BDI Strain Transducer** has been designed for structural testing in tough field conditions. These accurate, rugged, and fully-waterproofed units can be installed very quickly for all types of measurement applications.

APPLICATIONS

- Steel
- Timber
- Fiber reinforced polymer (FRP)
- Fatigue Monitoring
- Tension Rods
- Laboratory Testing
- Pre-stressed and post-tensioned concrete
- Reinforced concrete (with gage extensions)

DIMENSIONS

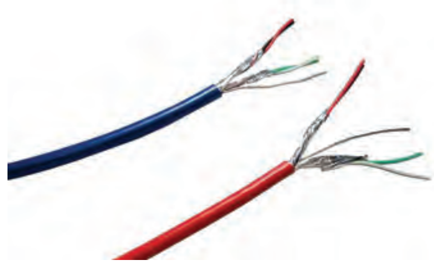


FEATURES

- Cost effective
- Installs in 5 minutes or less
- Completely reusable— lasts for years
- Waterproof to 20ft (~6.1m)
- Field-grade instrumentation cable
- Specify cable type and length
- Compatible with most data acquisition systems
- N.I.S.T. traceable calibration

OPTIONS

Reusable Tabs and Tab Jig



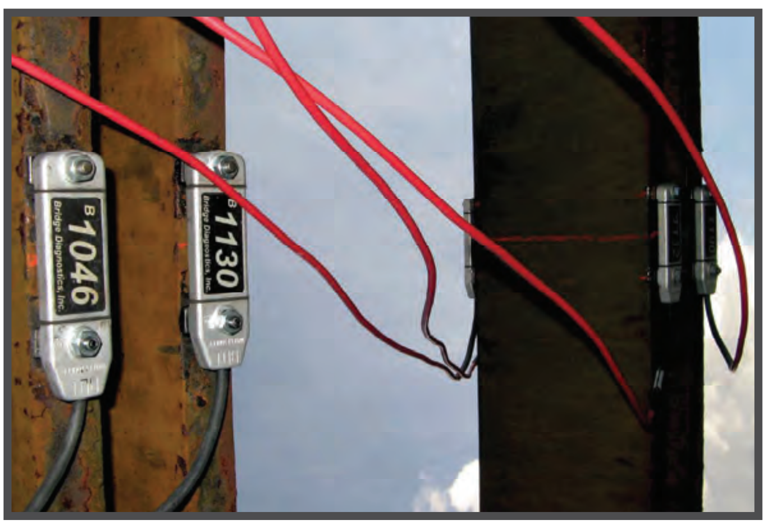
Cable Options

Rugged aluminum covers protect transducers for long-term monitoring

MODEL	ST350
RANGE (RESISTANCE)	350Ω
EXCITATION VOLTAGE	+1.0 to +10.0 Vdc (output is ratiometric)
POWER RATING MAX: TYPICAL: INTELLIDUCER:	 300mW 72mW @ +5.0 Vdc 13mW @ +5.0 Vdc*
CIRCUIT	Full wheatstone bridge with four active 350Ω foil gages
STRAIN RANGE	±4,000 με (Calibrated to ±2,000 με)
FORCE REQUIRED FOR 1000με	~17lbs. (~76N)
TYPICAL SENSITIVITY	~500Ω/mV/V (individually calibrated to N.I.S.T. standards)
ACCURACY	<±1%
EFFECTIVE GAGE LENGTH	3.0in (76.2mm) [Extensions available for use with R/C structures]
CABLE	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

* Intelliducer connectors operate at +5.0 Vdc only.

STEEL



CONCRETE

GAGE LENGTH EXTENSION

Bridge Diagnostics, Inc.
1995 57TH COURT NORTH, SUITE 100 | BOULDER, CO 80301-2810 USA
TEL. +1 (303) 494-3230

STS4 WIRELESS STRUCTURAL TESTING SYSTEM

DESCRIPTION

The new STS4 from BDI is the world's only data acquisition system that has been designed by civil engineers expressly for structural testing. This next-generation wireless system is rugged, highly efficient, and compatible with existing STS-WiFi systems.

BDI has developed our STS systems based on the experience we've gained through testing hundreds of structures all over the world in difficult field conditions. Because we've slogged through the mud, rappelled from ropes, and swayed in bucket trucks—all in bad weather—we know that ease-of-use is a must. Therefore, all of our sensors are very easy to install, the software is simple to operate, and the built-in sensor verification routines ensure you'll collect quality data. The field time saved using the STS4 compared to standard data acquisition systems will more than pay for itself after just a few uses.

STS4 PRIMARY NODE

APPLICATIONS

- Highway and Railroad Bridges: Steel, concrete, timber, FRP
- Lift Bridges: Wirelessly record torques, displacements, and other parameters
- Hydraulic Structures: Radial gates, nav-lock, lift, and miter gates.
- Laboratory Testing: Ideal to help students understand the capabilities of sensor measurements and data acquisition equipment.
- Cable Forces: Use our BDI Accelerometers to help measure in-situ cable forces.

FEATURES

STANDARD STS4 FEATURES

- A complete wireless "turn-key" load testing system
- Intelliducer sensors automatically identify themselves — no tracking channel numbers!
- Standard 802.11b/g/n wireless protocol with wired Ethernet backup
- Backwards compatible with STS-WiFi systems.
- Existing owners can reuse their sensors

CAPABILITIES

NEW STS4 CAPABILITIES

Based on the larger ranges of sensors being used for structural testing, the new STS4 has all of the same features as our highly-successful STS-WiFi testing systems, but are smaller, lighter weight, and more versatile.

- New Extension Node: Communication and power for up to 16 data channels via the expansion port on the STS4 Primary Node.
- Auto Temperature compensation support for sensors with thermistors.
- Increased sensor voltage input range to ±5.0 VDC
- Added internal SD Flash memory (up to 16 GB)
- Programmable excitation voltage (+1 to + 5 VDC)
- New +15 VDC unregulated excitation port
- Increased sample rate of up to 1,000Hz
- Programmable shunt capabilities to verify sensor functionality.
- Power over Ethernet support (POE)
  Power one Primary Node and three Extension Nodes while trickle charging the battery!
- Internal Li-Ion battery with integrated charging circuitry
- More efficient power conservation modes
- Fully IP67 rated
- Compatible with existing WinSTS Software
- New, completely redesign STS-LIVE data acquisition software with graphing and evaluation capabilities. Mac OS X and multi-language support.
- Custom programming with LabView Support
- New STS Base Station, with wireless repeater capabilities (no cables between multiple Base Stations), increased range, and POE support.

SENSORS

STRUCTURAL TESTING SENSORS

Select from our ruggedized range of sensors below. Or, if you already have your own, chances are they can be configured to plug-and-play into the STS4, just send us your specifications. Many of the following sensors can be supplied with internal thermistors to allow for temperature compensation.

- BDI temperature-compensated strain transducers
- Tiltmeters
- LVDT's
- Accelerometers
- BDI AutoClicker Load Position Indicator
- Load cells
- String wire potentiometers
- Foil strain gage completion units
- Pressure transducers
- Wireless torque modules
- Piezometers
- Amperage transducers
- Universal terminal plug allows many other sensors

WE STAND BELOW OUR WORK!

STS 4 EXTENDER NODE

	STS4-4-IW3	STS4-4-ID5 (Extender Node)
Measurement Type	Single-ended or Differential: voltage, millivolts, digital	Single-ended or Differential: voltage, millivolts, digital
Processor	Stellaris® Arm® Cortex™-M3	Stellaris® Arm® Cortex™-M3
Memory **Internal Memory:** **Internal MicroSD Flash:**	8 MB (Operating System) 2Gb Standard (Expandable to 16Gb), Auto measurement data back-up system.	8 MB (Operating System) 2Gb Standard (Expandable to 16Gb), Auto measurement data back-up system.
Maximum Sample Rate	1000 Hz	1000 Hz
Programmable Gain Settings	13 gain settings, ranging from 1mV diff, to 10V single ended	13 gain settings, ranging from 1mV diff, to 10V single ended
Analog to Digital Resolution	24-bit ADC	24-bit ADC
A/D Convertor Type	Sigma delta	Sigma delta
Voltage Reference System	Ratiometric[1]	Ratiometric[1]
A/D Temperature Tolerance	Gain drift 1 ppm/°C	Gain drift 1 ppm/°C
Input Channels	4	4
Tempearature sensor Inputs (Thermistor)	One per input channel	One per input channel
STS4-4-ID5 Support	Up to 3 Extension Nodes	n/a
Excitation Voltages		
Vx (programmable)	+0 to +5 VDC @ 20mA (per channel)	+0 to +5 VDC @ 20mA (per channel)
V_{+15}	+15 VDC @ 200mA (combined)	+15 VDC @ 200mA (combined)
Analog Voltage Accuracy		
Vx (programmable)	16 bit resolution, typ. 5ppm/°C	16 bit resolution, typ. 5ppm/°C
V_{+15}	±5%	±5%
Signal Input Voltage Range	±5.0 VDC	±5.0 VDC
Power Supply		
Li-Ion Battery	+10.8 VDC (Nominal), 6.2Ah, 67Wh	n/a
DC Supply	+24 VDC @ 3.0 Amp (max for charging)	n/a
Power over Ethernet	+48 VDC per - IEEE 802.3af	n/a
Node-to-Node	+9VDC to +24VDC, power source dependent (supply only)	+9VDC to +24VDC, power source dependent (input and supply)
Typical Power Consumption		
Base Consumption	0.7W	0.7W
Typical Acquisition[2]	1.5W	1.5W
Sleep Mode	<10mW	<10mW
Communication		
Wireless	802.11b/g/n (2.412 - 2.484 GHz)	n/a
Ethernet	10T-Base (TCP/IP)	n/a
Node-to-Node	Proprietary high speed Low Voltage Differential Signal communication protocol	Proprietary high speed Low Voltage Differential Signal communication protocol
Sensor Interface		
Connector	10-Pin Mil-Spec circular bayonet snap-lock. IP67 Rated.	10-Pin Mil-Spec circular bayonet snap-lock. IP67 Rated.
Intelliducer Support[3]	Yes	Yes
Physical		
Enclosure	Combination aluminum extrusion and high strength molded parts.	Combination aluminum extrusion and high strength molded parts.
Protection	IP67	IP67
Size	8.0in x 4.5in x 3.25in (203mm x 115mm x 83mm)	8.5in x 4.5in x 2.0in (215mm x 115mm x 51mm)
Weight	2.63 Lbs. (1200 g.)	1.37 Lbs. (625g.)
Operating Temperature **Battery Operation:** **DC Supply Only:**	-10°C to +55°C -30°C to +65°C	n/a -30°C to +65°C
Storage Temperature	-40°C to +85°C	-40°C to +85°C
Compliance		
CE	Coming Soon!	Coming Soon!
FCC	Coming Soon!	Coming Soon!
Wireless Module:	FCC, IC, and CE Certified	n/a
Computer Requirements for BDI Software		
WinSTS	Windows® XP, Vista, 7 (32 or 64-bit OS)	
STS-LIVE	Windows® XP, 7 (32 or 64-bit OS), MAC OS X 10.7 or Higher	
Interference To Third Party Software	Platform independent TCP/IP client/server, LabView[4] support	
Multi-Language Support	STS-LIVE	
Warranty	3 Years	3 Years

[1] Ratiometric: The system reference voltages are all derived from the same high precision ultra stable source. Any residual drift would change excitation and ADC reference effectively canceling drift out.

[2] Typical power drain is calculated with four 350 Ω full bridge strain transducer connected to the system and collecting data at the highest sample rate possible. This does not include battery charging power consumption.

[3] Intelliducer support refers to BDI's intelligent sensor connector interface. The intelligent sensor interface contains the sensor ID, calibration factor, gain setting, etc. within a memory chip inside the sensor connector.

[4] BDI can provide a *.dll file for custom programming capabilities with LabView.

COMPATIBILITY WITH STS-WiFi

New STS4 nodes and Intelliducer sensors have all been designed to be backwards-compatible with existing BDI STS-WiFi systems, you'll just need to upload and install the latest WinSTS from our website. The new STS4 nodes and sensors will appear alongside other nodes in the WinSTS screen. For systems consisting of all STS4 nodes, all previous Intelliducer sensors will work, and a completely new software package called STS-LIVE will be used to operate the system and activate the new capabilities.

WIRELESS LOAD TESTING: THE ONLY WAY TO GO!

Bridge Diagnostics, Inc.
1995 57TH COURT NORTH, SUITE 100 | BOULDER, CO 80301 USA
TEL. +1 (303) 494-3230

APPENDIX

B K-FACTOR SUMMARY

Project #

Project SCDOT RC Tbeam K-Factor Summary Computed by Date

Subject Checked by Date

Girder naming convention matches label diagrams

BR 320

Girder	K-Factor
1-1	2.27
1-2	7.43
1-3	4.82
1-4	3.88
1-5	4.55
1-6	1.80

BR 347

Girder	K-Factor
3-8	1.55
3-9	2.85
3-10	2.61
3-11	2.37
3-12	3.01
3-13	1.69

BR 398

Girder	K-Factor
1-5	2.50
1-6	2.28
1-7	2.01
1-8	2.22

BR 403

Girder	K-Factor
5-1	2.25
5-2	1.61
5-3	1.62
5-4	1.69

BR 404

Girder	K-Factor
7-1	2.11
7-2	1.72
7-3	1.62
7-4	1.68

Project SCDOT RC Tbeam K-Factor Summary Computed by Date
Subject Checked by Date

BR 428

Girder	K-Factor
8-1	1.97
8-2	1.99
8-3	1.65
8-4	1.59
8-5	1.72
8-6	1.80

BR 568

Girder	K-Factor
19-1	2.49
19-2	2.57
19-3	3.20

BR 627

Girder	K-Factor
1-3	2.05
1-4	2.20
1-5	1.84

BR 580

Girder	K-Factor
10-3	2.20
10-4	2.06
10-5	1.94
10-6	1.94

BR 745

Girder	K-Factor
5-9	3.20
5-10	3.33
5-11	3.31
5-12	3.61

BR 877

Girder	K-Factor
1-1	2.54
1-2	4.50
1-3	3.03
1-4	2.63
1-5	4.25
1-6	2.58

BR 1036

Girder	K-Factor
1-1	2.05
1-2	1.87
1-3	1.77
1-4	2.77

BR 1052

Girder	K-Factor
9-1	3.08
9-2	2.02
9-3	1.72
9-4	1.60
9-5	1.86
9-6	3.43

BR 1123

Girder	K-Factor
2-14	2.28
2-15	2.63
2-16	2.57
2-17	2.36
2-18	2.09
2-19	1.81
2-20	2.49

BR 1272

Girder	K-Factor
7-1	3.40
7-2	2.21
7-3	2.25
7-4	3.38

BR 1276

Girder	K-Factor
1-1	2.02
1-2	1.98
1-3	2.22
1-4	1.96
1-5A	2.17
1-6A	2.10
1-7A	2.27
1-8A	2.21

Project #

Project SCDOT RC Tbeam K-Factor Summary Computed by Date

Subject Checked by Date

BR 1758

Girder	K-Factor
3-4	1.73
3-5	3.39
3-6	2.22
3-7	2.26

BR 1836

Girder	K-Factor
7-1	2.17
7-2	3.37
7-3	1.61
7-4	1.70
7-5	2.49
7-6	1.68

BR 1856

Girder	K-Factor
25-2	1.45
25-3	1.32
25-4	3.94
25-5	3.83
25-6	3.22
25-7	3.70
25-8	1.12
25-9	1.77

BR 2067

Girder	K-Factor
8-3	1.92
8-4	2.00
8-5	2.62
8-6	2.31
8-7	1.58
8-8	1.51
8-9	1.66
8-10	2.77

BR 2112

Girder	K-Factor
1-1	2.56
1-2	1.85
1-3	1.99
1-4	2.06
1-5	1.86
1-6	1.73

*Label diagram does not include widening. Original girders named 1-4 to match label diagram.

Project SCDOT RC Tbeam K-Factor Summary Computed by Date

Subject Checked by Date

BR 2133

Girder	K-Factor
5-1	1.81
5-2	1.89
5-3	1.72
5-4	1.93

BR 2610

Girder	K-Factor
2M-1	3.89
2M-2	3.26
2M-3	3.53
2M-4	3.83
2M-5	3.62

BR 2827

Girder	K-Factor
4-1	1.21
4-2	1.66
4-3	1.83
4-4	1.35

APPENDIX

C K-FACTOR CALCULATIONS

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 26-Apr-21
Subject	Bridge 320 Girder 1-1 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

Section Inputs:

L	30	ft	Span Length
S_1	2.417	ft	Spacing to Interior Girder Web
S_2	1.896	ft	Distance to End of Slab
b_w	13.5	in	Web Width
h_f	7.5	in	Flange Depth (Depth of Slab)
h	42.313	in	Height of Total Section
d_b	3.25	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	1.2	ksi	Compressive Strength, Concrete
f_y	40	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	3		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10		that is closest to the tension face.
A_{s1}	3.81	in^2	Area of Reinforcement
d_1	39.063	in	
# Bars Row 2	3		
Bar Size Row 2	#10		
A_{s2}	3.81	in^2	Area of Reinforcement
d_2	35.313	in	
# Bars Row 3	0		
Bar Size Row 3	0		
A_{s3}	0	in^2	Area of Reinforcement
d_3	0	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck
DF	0.3235		Distribution Factor

Calculations:

d_{avg}	37.1875	in	Weighted Average depth of Rebar
$A_{s,total}$	7.62	in^2	Total Reinforcement Area

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 26-Apr-21
Subject	Bridge 320 Girder 1-1 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

b_{eff}	37.25	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1974538	psi	Modulus of Elasticity, Concrete
n	14.69		Modular Ratio
f_r	259.81	psi	Modulus of Rupture, Concrete
y bar	17.02	in	Depth of Neutral Axis, Uncracked Section
y_t	25.29		Depth of Neutral Axis from tension face, Uncracked Section
I_g	127197.5858	in^4	Gross Moment of Inertia
M_a	616.8498	k-in	Applied Moment

Stress in Concrete (psi)	82.53
Average Stress in Steel (psi)	1436.53
Stress in Extreme Steel (psi)	1570.08
Strain in Concrete (in/in $*10^6$)	41.80
Average Strain in Steel (in/in $*10^6$)	49.54
Strain in Extreme Steel (in/in $*10^6$)	54.14
Expected Strain @ Tension Face (in/in $*10^6$)	62.12
Midspan Deflection (in)	0.02653

Say Midspan deflection is Moment x L^2/12EI

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date	27-Apr-21
Subject	Bridge 320 Girder 1-2 Strain and Deflection Analysis	Checked by CRG	Date	6/15/21

Section Inputs:

L	30	ft	Span Length
S_1	2.417	ft	Web Spacing
S_2	7.563	ft	Web Spacing other side
b_w	13.5	in	Web Width
h_f	8.5	in	Flange Depth (Depth of Slab)
h	29.5	in	Height of Total Section
d_b	3	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	0.65	ksi	Compressive Strength, Concrete
f_y	33	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	2		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10	square bars, use #11	that is closest to the tension face.
A_{s1}	3.12	in^2	Area of Reinforcement
d_1	26.5	in	
# Bars Row 2	1		
Bar Size Row 2	#10	square bars, use #11	
A_{s2}	1.56	in^2	Area of Reinforcement
d_2	26.5	in	
# Bars Row 3	0		
Bar Size Row 3	0		
A_{s3}	0	in^2	Area of Reinforcement
d_3	0	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck	??
DF	0.384		Distribution Factor	

Calculations:

d_{avg}	26.5	in	Weighted Average depth of Rebar
$A_{s,total}$	4.68	in^2	Total Reinforcement Area

Project #

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by	JRW	Date	27-Apr-21
Subject	Bridge 320 Girder 1-2 Strain and Deflection Analysis	Checked by	CRG	Date	6/15/21

b_{eff}	59.875	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1453221	psi	Modulus of Elasticity, Concrete
n	19.96		Modular Ratio
f_r	191.21	psi	Modulus of Rupture, Concrete
y bar	9.53	in	Depth of Neutral Axis, Uncracked Section
y_t	19.97		Depth of Neutral Axis from tension face, Uncracked Section
I_g	53095.74367	in^4	Gross Moment of Inertia
M_a	732.2112	k-in	Applied Moment

Stress in Concrete (psi)	131.38
Average Stress in Steel (psi)	4670.94
Stress in Extreme Steel (psi)	4670.94
Strain in Concrete (in/in $*10^6$)	90.41
Average Strain in Steel (in/in $*10^6$)	161.07
Strain in Extreme Steel (in/in $*10^6$)	161.07
Expected Strain @ Tension Face (in/in $*10^6$)	189.54
Midspan Deflection (in)	0.10249

Say Midspan deflection is Moment x L^2/12EI

Section Inputs:

L	30	ft	Span Length
S_1	7.563	ft	Web Spacing
S_2	7.750	ft	Web Spacing other side
b_w	17	in	Web Width
h_f	8.5	in	Flange Depth (Depth of Slab)
h	31.5	in	Height of Total Section
d_b	3	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	0.65	ksi	Compressive Strength, Concrete
f_y	33	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	4		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10	square bars, use #11	that is closest to the tension face.
A_{s1}	6.24	in^2	Area of Reinforcement
d_1	28.5	in	
# Bars Row 2	2		
Bar Size Row 2	#9	square bars, use #10	
A_{s2}	2.54	in^2	Area of Reinforcement
d_2	24.75	in	
# Bars Row 3	2		
Bar Size Row 3	#8	square bars, use #9	
A_{s3}	2	in^2	Area of Reinforcement
d_3	24.75	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck	??
DF	0.6129		Distribution Factor	

Calculations:

d_{avg}	26.92068646	in	Weighted Average depth of Rebar
$A_{s,total}$	10.78	in^2	Total Reinforcement Area

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 27-Apr-21
Subject	Bridge 320 Girder 1-3 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

b_{eff}	88.5	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1453221	psi	Modulus of Elasticity, Concrete
n	19.96		Modular Ratio
f_r	191.21	psi	Modulus of Rupture, Concrete
y bar	9.64	in	Depth of Neutral Axis, Uncracked Section
y_t	21.86		Depth of Neutral Axis from tension face, Uncracked Section
I_g	85586.05795	in^4	Gross Moment of Inertia
M_a	1168.67772	k-in	Applied Moment

Stress in Concrete (psi)	131.59
Average Stress in Steel (psi)	4709.82
Stress in Extreme Steel (psi)	5140.17
Strain in Concrete (in/in $*10^6$)	90.55
Average Strain in Steel (in/in $*10^6$)	162.41
Strain in Extreme Steel (in/in $*10^6$)	177.25
Expected Strain @ Tension Face (in/in $*10^6$)	205.44
Midspan Deflection (in)	0.10148

Say Midspan deflection is Moment x L^2/12EI

Section Inputs:

L	30	ft	Span Length
S_1	7.583	ft	Web Spacing
S_2	7.750	ft	Web Spacing other side
b_w	17	in	Web Width
h_f	8.5	in	Flange Depth (Depth of Slab)
h	31.5	in	Height of Total Section
d_b	3	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	0.65	ksi	Compressive Strength, Concrete
f_y	33	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	4		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10	square bars, use #11	that is closest to the tension face.
A_{s1}	6.24	in^2	Area of Reinforcement
d_1	28.5	in	
# Bars Row 2	2		
Bar Size Row 2	#9	square bars, use #10	
A_{s2}	2.54	in^2	Area of Reinforcement
d_2	24.75	in	
# Bars Row 3	2		
Bar Size Row 3	#8	square bars, use #9	
A_{s3}	2	in^2	Area of Reinforcement
d_3	24.75	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck
DF	0.6129		Distribution Factor

Calculations:

d_{avg}	26.92068646	in	Weighted Average depth of Rebar
$A_{s,total}$	10.78	in^2	Total Reinforcement Area

Project SCDOT RC Tbeam K-Factor Calculations Computed by JRW Date 27-Apr-21
Subject Bridge 320 Girder 1-4 Strain and Deflection Analysis Checked by CRG Date 6/15/21

b_{eff}	88.5	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1453221	psi	Modulus of Elasticity, Concrete
n	19.96		Modular Ratio
f_r	191.21	psi	Modulus of Rupture, Concrete
y bar	9.64	in	Depth of Neutral Axis, Uncracked Section
y_t	21.86		Depth of Neutral Axis from tension face, Uncracked Section
I_g	85586.05795	in^4	Gross Moment of Inertia
M_a	1168.67772	k-in	Applied Moment

Stress in Concrete (psi)	131.59
Average Stress in Steel (psi)	4709.82
Stress in Extreme Steel (psi)	5140.17
Strain in Concrete (in/in *10^6)	90.55
Average Strain in Steel (in/in *10^6)	162.41
Strain in Extreme Steel (in/in *10^6)	177.25
Expected Strain @ Tension Face (in/in *10^6)	205.44
Midspan Deflection (in)	0.10148

Say Midspan deflection is Moment x L^2/12EI

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 27-Apr-21
Subject	Bridge 320 Girder 1-5 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

Section Inputs:

L	30	ft	Span Length
S_1	7.583	ft	Web Spacing
S_2	2.417	ft	Web Spacing other side
b_w	13.5	in	Web Width
h_f	8.5	in	Flange Depth (Depth of Slab)
h	29.5	in	Height of Total Section
d_b	3	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	0.65	ksi	Compressive Strength, Concrete
f_y	33	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	2		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10	square bars, use #11	that is closest to the tension face.
A_{s1}	3.12	in^2	Area of Reinforcement
d_1	26.5	in	
# Bars Row 2	1		
Bar Size Row 2	#10	square bars, use #11	
A_{s2}	1.56	in^2	Area of Reinforcement
d_2	26.5	in	
# Bars Row 3	0		
Bar Size Row 3	0		
A_{s3}	0	in^2	Area of Reinforcement
d_3	0	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck
DF	0.383815		Distribution Factor

Calculations:

d_{avg}	26.5	in	Weighted Average depth of Rebar
$A_{s,total}$	4.68	in^2	Total Reinforcement Area

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 27-Apr-21
Subject	Bridge 320 Girder 1-5 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

b_{eff}	59.875	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1453221	psi	Modulus of Elasticity, Concrete
n	19.96		Modular Ratio
f_r	191.21	psi	Modulus of Rupture, Concrete
y bar	9.53	in	Depth of Neutral Axis, Uncracked Section
y_t	19.97		Depth of Neutral Axis from tension face, Uncracked Section
I_g	53095.74367	in^4	Gross Moment of Inertia
M_a	731.858442	k-in	Applied Moment

Stress in Concrete (psi)	131.32	
Average Stress in Steel (psi)	4668.69	
Stress in Extreme Steel (psi)	4668.69	
Strain in Concrete (in/in $*10^6$)	90.36	
Average Strain in Steel (in/in $*10^6$)	160.99	
Strain in Extreme Steel (in/in $*10^6$)	160.99	
Expected Strain @ Tension Face (in/in $*10^6$)	189.44	
Midspan Deflection (in)	0.10244	Say Midspan deflection is Moment x L^2/12EI

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 26-Apr-21
Subject	Bridge 320 Girder 1-6 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

Section Inputs:

L	30	ft	Span Length
S_1	2.417	ft	Spacing to Interior Girder Web
S_2	1.896	ft	Distance to End of Slab
b_w	13.5	in	Web Width
h_f	7.5	in	Flange Depth (Depth of Slab)
h	42.313	in	Height of Total Section
d_b	3.25	in	Center of bottom row of bars to tension face

Material Inputs:

f'_c	1.2	ksi	Compressive Strength, Concrete
f_y	40	ksi	Yield Strength, Steel
E_s	29000	ksi	Modulus of Elasticity, Steel

Reinforcement Inputs:

# Bars Row 1	3		Steel Layer 1 is the reinforcement
Bar Size Row 1	#10		that is closest to the tension face.
A_{s1}	3.81	in^2	Area of Reinforcement
d_1	39.063	in	
# Bars Row 2	3		
Bar Size Row 2	#10		
A_{s2}	3.81	in^2	Area of Reinforcement
d_2	35.313	in	
# Bars Row 3	0		
Bar Size Row 3	0		
A_{s3}	0	in^2	Area of Reinforcement
d_3	0	in	
# Bars Row 4	0		
Bar Size Row 4	0		
A_{s4}	0	in^2	Area of Reinforcement
d_4	0	in	
# Bars Row 5	0		
Bar Size Row 5	0		
A_{s5}	0	in^2	Area of Reinforcement
d_5	0	in	

Load Inputs:

M_a	158.9	k-ft	Applied Moment from Truck
DF	0.323275		Distribution Factor

Calculations:

d_{avg}	37.1875	in	Weighted Average depth of Rebar
$A_{s,total}$	7.62	in^2	Total Reinforcement Area

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by JRW	Date 26-Apr-21
Subject	Bridge 320 Girder 1-6 Strain and Deflection Analysis	Checked by CRG	Date 6/15/21

b_{eff}	37.25	in	Effective Flange Width
E_s	29000	ksi	Modulus of Elasticity, Steel
E_c	1974538	psi	Modulus of Elasticity, Concrete
n	14.69		Modular Ratio
f_r	259.81	psi	Modulus of Rupture, Concrete
y bar	17.02	in	Depth of Neutral Axis, Uncracked Section
y_t	25.29		Depth of Neutral Axis from tension face, Uncracked Section
I_g	127197.5858	in^4	Gross Moment of Inertia
M_a	616.42077	k-in	Applied Moment

Stress in Concrete (psi)	82.48
Average Stress in Steel (psi)	1435.53
Stress in Extreme Steel (psi)	1568.98
Strain in Concrete (in/in $*10^6$)	41.77
Average Strain in Steel (in/in $*10^6$)	49.50
Strain in Extreme Steel (in/in $*10^6$)	54.10
Expected Strain @ Tension Face (in/in $*10^6$)	62.08
Midspan Deflection (in)	0.02651

Say Midspan deflection is Moment x L^2/12EI

Project #

Project: SCDOT RC Tbeam K-Factor Calculations — Computed by: JRW — Date: 14-Jun-21

Subject: Bridge 320 — Checked by: CRG — Date: 6/15/21

K Factor Inputs

T = 18.64 Tons

W = 25 Tons

Theoretical Response @ Mid-Span

Girder	Strain, με	Deflection, in	
1-1	62.12	0.0265	* From Theoretical Response SS's
1-2	189.44	0.1024	* From Theoretical Response SS's
1-3	205.44	0.1015	* From Theoretical Response SS's
1-4	205.44	0.1015	* From Theoretical Response SS's
1-5	189.44	0.1024	* From Theoretical Response SS's
1-6	62.08	0.0265	* From Theoretical Response SS's

Measured Response @ Mid-Span

Girder	Strain, με	Deflection, in	
1-1	12.21	0.0075	* From Load Test, Test 6 Run 3
1-2	13.81	0.0074	* From Load Test, Test 5 Run 3
1-3	22.40	0.0117	* From Load Test, Test 4 Run 3
1-4	23.14	0.0150	* From Load Test, Test 3 Run 3
1-5	14.15	0.0127	* From Load Test, Test 2 Run 2
1-6	17.81	0.01016	* From Load Test, Test 1 Run 1

Calculate K Factors

K= 1 + Ka*Kb — AASHTO MBE Eq. 8.8.2.3.1-1

Ka = $\varepsilon_c / \varepsilon_t - 1$

Kb =

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	$\frac{T}{W} < 0.4$	$0.4 \le \frac{T}{W} \le 0.7$	$\frac{T}{W} > 0.7$	K_b
√		√			0
√			√		0.8
√				√	1.0
	√	√			0
	√		√		0
	√			√	0.5

T/W = 0.7454

Kb = 0.5 Conservatively

Project #

Project	SCDOT RC Tbeam K-Factor Calculations	Computed by	JRW	Date	14-Jun-21
Subject	Bridge 320	Checked by	CRG	Date	6/15/21

Final K-Factors Each Girder Strain Analysis

Girder	Ka	Kb	K	
1-1	4.089743	0.5	3.045	Exterior
1-2	12.71597	0.5	7.358	Interior
1-3	8.171568	0.5	5.086	Interior
1-4	7.879596	0.5	4.940	Interior
1-5	12.38745	0.5	7.194	Interior
1-6	2.48496	0.5	2.242	Exterior

Controlling K Factor	
Interior	Exterior
4.940	2.242

Final K-Factors Each Girder Deflection Analysis

Girder	Ka	Kb	K	
1-1	2.549948	0.5	2.275	Exterior
1-2	12.85041	0.5	7.425	Interior
1-3	7.648442	0.5	4.824	Interior
1-4	5.765388	0.5	3.883	Interior
1-5	7.097198	0.5	4.549	Interior
1-6	1.607906	0.5	1.804	Exterior

Controlling K Factor	
Interior	Exterior
3.883	1.804