Live Load Test and Finite Element Analysis of a Box Girder Bridge for the Long Term Bridge Performance Program

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LIVE LOAD TEST AND FINITE ELEMENT MODEL ANALYSIS OF A BOX GIRDER BRIDGE FOR THE LONG TERM BRIDGE PERFORMANCE PROGRAM

by

Dereck J. Hodson

A thesis submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

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2010
ABSTRACT

Live Load Test and Finite Element Model Analysis of a Box Girder Bridge for the Long Term Bridge Performance Program

by

Dereck J. Hodson, Master of Science
Utah State University, 2010

Major Professor: Dr. Paul Barr
Department: Civil and Environmental Engineering

The Long Term Bridge Performance (LTBP) Program is a 20-year program initiated by the Federal Highway Administration to better understand the behavior of highway bridges as they deteriorate due to environmental variables and vehicle loads. Part of this program includes the periodic testing of selected bridges.

The Lambert Road Bridge was subjected to nondestructive testing in the fall of 2009. Part of this testing included a live load test. This test involved driving two heavy trucks across the instrumented bridge on selected load paths. The bridge was instrumented with strain, displacement, and tilt sensors. This collected data was used to calibrate a finite element model. This finite element model was used to determine the theoretical live load distribution factors. Using the controlling distribution factor from the finite element model, the inventory and operating ratings of the bridge were determined. These load ratings were compared to those obtained from using the controlling distribution factor from the AASHTO LRFD Specifications.
This thesis also examined how different parameters such as span length, girder spacing, parapets, skew, continuity, deck overhang, and deck thickness affect the distribution factors of box girder bridges. This was done by creating approximately 40 finite element models and comparing the results to those obtained by using the AASHTO LRFD Specifications.
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INTRODUCTION

The LTBP (Long Term Bridge Performance) Program is a 20-year program in which selected bridges will be subjected to periodic testing, inspection, and monitoring. The goal of this program is to gain a better understanding of how bridges deteriorate due to corrosion, fatigue, weather, and vehicle loads. By gaining a better understanding of how bridges deteriorate, the effectiveness of bridge design and maintenance can be improved. As part of the LTBP Program the Lambert Bridge near Sacramento, CA was selected as a pilot bridge to be monitored for its long-term performance as well as undergoing periodic testing. Part of the periodic testing will include NDT (non-destructive testing) methods such as live load testing. This testing involved installing strain, displacement, and rotation sensors on a bridge as a truck was slowly driven over the bridge on selected load paths. The data collected was analyzed and used to create a working finite element model. From this finite element model, the distribution factors and load ratings were obtained.

The concept of using finite element models to model and determine the overall bridge behavior has been achieved in many studies. In these studies a selected bridge was subjected to a live load test where typically the strain and displacement data was collected. This data was used to create a working finite element model that behaves in a similar manner as the actual bridge. From these finite element models the overall performance, distribution factors, and bridge load ratings were obtained. Some of these studies that involved using live load tests and finite element models to determine a bridges load capacity include Ruth et al. (2005) modeling a steel stringer bridge, Jáuregui and Barr (2004) and Barr et al. (2006) modeling prestressed concrete bridges, and Yost,
Shulz, and Commander (2005) modeling several bridges including a reinforced concrete T-beam bridge, a non-composite steel bridge, a prestressed concrete bridge, and a reinforced concrete pan girder bridge.

In these studies various types of bridges have been studied. However, the testing and modeling of box girder bridges wasn’t as common. There have been numerous studies that have examined the behavior of box girder bridges. Sennah and Kennedy (2002) discussed the various techniques that have been used over the last couple of decades to analyze box-girder bridges. Song and Hida (2003) used a finite element model to determine the live load distribution factors for a theoretical concrete box girder bridge. This study included the effects of different parameters (span length, width, and depth) on distribution factors. Doerrer and Riyadh (2008) used finite element models to determine the shear and moment distribution factors for various theoretical cast-in-place concrete box girder bridges. This study also examined the different variables that affect the distribution factors in box girder bridges such as span length, depth, and number of cells.

In each of the box girder bridge studies, the finite element models were not calibrated using test data or used to model an actual in service bridge. One case did model a spread prestressed concrete box girder bridge and used a finite element model to determine the load-distribution factors (Hughs and Idriss, 2006). However, this bridge was a spread box girder bridge and would behave differently than a typical box girder bridge.

Although there have been multiple studies using a calibrated finite element model based on live load testing, none of these studies have been conducted on cast-in-place, prestressed box girder bridges. Studies have been done on the distribution factors for box
To determine the distribution factors and load ratings of a continuous, cast-in-place, prestressed box girder bridge a finite element model was created. The finite element model was calibrated using data collected during a live load and dynamic tests. The data collected from these tests included strains, girder deflections, rotations near the abutment, modal shapes, and frequencies. The boundary conditions of the finite element model were continually changed until a good correlation between the test data and finite element was obtained. Because of the good correlation between the finite element model and test data, it was concluded that the finite element model was behaving as the actual bridge. This finite element model was used to determine the distribution factors and theoretical load ratings of the bridge. The results indicated that the distribution factors from AASHTO LRFD Specifications were 29% to 46% conservative compared to those obtained from the finite element model for an interior girder. The exterior girder distribution factors were 2% to 9% unconservative for the AASHTO LRFD Specifications compared to those from the finite element model. Using the distribution factors from the finite element models, new load ratings were obtained which significantly improved the inventory rating from 1.61 to 2.56 and the operating 2.69 to 4.27.
REVIEW OF LITERATURE

The use of finite element models in determining the distribution factors for highway bridges is beneficial because the finite element models provide a better approximate value for the distribution factors of a bridge than using the AASHTO LRFD Specifications. These finite element models work best when they are calibrated using data collected during a live load test. Jáuregui and Barr (2004), Hughes and Idriss (2006), Barr et al. (2006), Yost, Shulz, and Commander (2005), and Ruth et al. (2005) developed finite element models based on live load test data. These finite element models were used to determine the theoretical distribution factors and load ratings of the bridges. The results were that the load ratings were larger than those using the distribution factors from the AASHTO LRFD Specifications. The AASHTO LRFD Specifications in each of those cases provided conservative results for the distribution factors which resulted in conservative load ratings.

None of the above studies involved the testing of a standard box girder bridge, although Hughes and Idriss (2006) tested a spread box girder bridge. Various studies have been completed on studying box girder bridges. A comprehensive review of the work done on box girder bridges can be seen in Sennah and Kennedy (2002). Other studies involving the determination of distribution factors for box girder bridges include Doerrr and Riyadh (2008) and Song, Chai, and Hida (2003). Both of these researchers examined the theoretical distribution factors from various box girder bridges by using finite element models. However, these studies were not based on actual highway bridges.
In this study, a prestressed concrete spread box girder bridge was instrumented with strain sensors. By instrumenting the bridge, the distribution factors of the actual bridge were found and compared with those obtained from AASHTO LRFD and Standard Specifications. The bridge was a five span, continuous, prestressed concrete bridge with a skew of 12°. Strain sensors were embedded in the concrete during construction at the fifth span. These sensors were arranged so that the distribution factors could be calculated from the live load test. After the completion of the live load test, the experimentally determined distribution factors for both shear and moment were obtained and compared with those from the finite element model. Once the finite element model was confirmed for accuracy, the distribution factors from the finite element model were obtained for the exterior and interior girders for shear and moment. These factors were than compared to the AASHTO Standard and LRFD specifications. From these comparisons, several conclusions were obtained. The LRFD Specifications were accurate or conservative for all cases in comparison to the finite element model. These specifications were more accurate for interior girders for both shear and moment. The specifications were found to be quite conservative for the exterior girders. The comparisons of the distribution factors for the Standard Specifications were mixed. In the cases of moment for both exterior and interior girders and shear for the exterior girder, the AASHTO LRFD Specifications was conservative. For exterior girder shear, the AASHTO Standard Specifications was found to be highly unconservative by nearly 50%. In all, the study showed that using the AASHTO LRFD Specifications resulted in a
safe and conservative design whereas using the procedures in the Standard Specifications can lead to highly unconservative or conservative designs depending on the specific girder.

Live load distribution factors for cast-in-place concrete box girder bridges (Doerrer and Riyadh, 2008)

This research examined the different variables that affect the distribution factors for box-girder bridges. The authors noted that the design philosophy has changed over the past couple of decades. First there was the design philosophy of ASD (allowable strength design), then Load Factor Design (LFD), and currently LRFD. This latest methodology was used in the study to calculate the distribution factors and compare these factors with those obtained from a finite element model. Although much research has been done on distribution factors, few studies have investigated cast-in-place box girder bridges. The distribution factors that were investigated in this study were for both moment and shear. The researchers examined how different geometric parameters affected the distribution factors. The different parameters included girder depth, deck overhang, span length, girder spacing, and number of cells. To quantify how these variables affect distribution factors, various finite element models were created. These finite element models were compared against other finite element models created by other researchers to ensure accuracy. Once the finite element models were developed and validated, the shear and moment distribution factors were found for each finite element model. Subsequently, these factors were compared to the factors predicted by the AASHTO LRFD Specifications. Based on these comparisons several conclusions were achieved. The use of the procedures in the LRFD Specifications resulted in increasingly
conservative distribution factors as the span length was increased and when the number of cells was increased. The use of the lever rule was inaccurate and produced unusually conservative factors by three times the finite element model results. Interior and exterior girder shear distribution factors were typically conservative compared to the finite element model by 25% to 65%. Other conclusions were that the effects of multiple spans have little effect on distribution factors. Two and three span factors were very similar and the distribution factors for positive and negative moment were usually within 5% of each other.

Nondestructive evaluation of the I-40 bridge over the Rio Grande River (Jáuregui and Barr, 2004)

This study evaluated the experimentally determined inventory and operating ratings of an interstate bridge which had a low calculated load rating. The main objective was to determine a more accurate load rating for the bridge so if possible, fewer permits would be denied and inconveniences for trucks and other motorists would be minimized. The bridge was a precast, prestressed bridge and was part of a series of two three-span bridges and a four-span bridge which was between the three-span bridges. The bridge had calculated inventory and operating ratings of 1.0 and 1.67 as determined using the distribution factors from the 2000 AASHTO Standard Specifications which were known to be quite conservative. To find load ratings that were more applicable to the bridge, a nondestructive live load test was performed. This test included instrumenting eight girders on the second span of the three-span bridge unit with strain transducers. Once the bridge was instrumented, a 238 kN (53.4 k), three axle water truck drove across the bridge on three separate load paths. Each load path was driven on three times to ensure
reproducible results. This was important because not all the lanes could be shut down during the testing period so some vehicles would pass over the bridge during the testing. After the completion of the live load test, three finite element models were created. The first finite element model ignored the stiffness of the pier, in the second finite element model the pier was modeled with frame elements, and the third finite element model had fixed end restraints at the pier. The moments at midspan from the finite element models were compared to the moments based on the live load test. The results showed that the second finite element model behaved similar to the actual bridge and could be used to accurately determine the load ratings. The distribution factors for interior and exterior girders in both positive and negative moment were obtained from the model and compared with those from the AASHTO LRFD Specifications. The results showed that the distribution factors based on the AASHTO LRFD Specifications were more conservative in comparison to those from the finite element model by 3 to 15%. However, this conservatism was not significant enough to increase the load ratings but because there was a decrease in the longitudinal moment in the girders from the piers there was reason to increase the ratings. As a result operating rating was increased to 2.85 and the inventory was increased to 1.7.

Long-term structural health monitoring of the San Ysidro Bridge (Barr et al., 2006)

As a result of increased traffic demands, New Mexico’s Department of Transportation (NMSHTD) expanded a section of a two lane highway running east from Albuquerque to Santa Fe. Part of this expansion project included the widening of a precast, prestressed concrete bridge. NMSHTD decided to, as part of the project, monitor
the long-term effects of the bridge. One of the parameters that was monitored was the load-carrying capacity of the bridge. To determine the load-carrying capacity, a live load test was performed. This test involved instrumenting the bridge girders with strain gauges and driving a three axle, 316 kN (70.9 k) water truck drive over the bridge on predetermined load paths. For this test, three different load paths were chosen. The load paths were selected to maximize the moment on the exterior and first two interior girders. The results from this test were used to validate a finite element model. To validate the model, moments produced from the live load test were compared to the moments calculated from the finite element model. Boundary conditions were adjusted until a good correlation was obtained. The finite element model was subsequently used to calculate the shear distribution factors. Because the actual bridge could not be used to calculate shear loads, a full scale single lane test bridge was built and tested to obtain shear distribution characteristic of typical prestressed girder bridges. Three finite element models were created to compare the shear to the full-scale model. The result was that a frame-shell model was the most accurate which was also the same finite element modeling scheme used for the live load test. Using this finite element model, the distribution factors for both moment and shear were obtained. These factors were compared with those from the AASHTO LRFD Specifications and were found to be less conservative by 1% to 22%. These same factors were used to determine both the operating and inventory ratings of the bridge. In each case considering span, exterior or interior beam, or shear or moment, the finite element model produced results that had higher load carry capacities than those calculated using the distribution factors from the
AASHTO LRFD Specifications by up to 13%. These results were in agreement with other studies that the AASHTO LRFD Specifications are conservative.

Using NDT for finite element model calibration and load rating of bridges (Yost, Schulz, and Commander, 2005)

It was estimated that 14% of the nation’s bridges were structurally deficient and another 14% were functionally obsolete. However, many of these so-called deficient bridges were determined by visual inspection and could actually be operating safely. Rather than spend the estimated $136 billion dollars to fix these bridges the authors proposed that the bridges undergo NDT (non-destructive testing). The NDT involved instrumenting a bridge with strain transducers and inducing a load by having a truck drive over the bridge. The data from this test was then used to create a working finite element model of the bridge. In this particular study, seven different bridges were tested and studied. The results showed that the distribution factors calculated using the AASHTO LRFD Specifications were conservative for many types of bridges including a steel non-composite bridge, a prestressed concrete bridge, a reinforced T-beam bridge, and a reinforced girder bridge. Other observations were that based on 200 highway bridges, 95% of these bridges obtained higher load ratings when a calibrated finite element model was used than when the AASHTO LRFD Specifications were used. Although using a calibrated finite element model improved the load ratings of many bridges the authors noted that it should be taken cautiously. They noted that sometimes the modeling of the bridge can become quite complex, especially with reinforced concrete structures. It was important that the bridge designer be aware of the different
parameters that affect the load rating of the bridge. It was concluded that the current inventory and operating ratings could be safely increased for all seven bridges.

**Live load distribution factors for concrete box-girder bridges (Song, Chai, and Hida, 2003)**

This study examined the distribution factors for box girder bridges using a grillage model. The grillage model was first calibrated by comparing the calculating grillage bending moment to the moment calculated from a typical finite element model for different loading cases. This theoretical bridge was a standard box girder bridge with two spans, four cells, and equal girder spacing. The main purpose of the study was to show how the AASHTO LRFD Specifications impose strict guidelines on the uses of the empirical formulas for calculating girder distribution factors. Many times bridges were constructed outside of the guidelines that the AASHTO LRFD Specifications imposed which inhibited the design of bridges. One of these restraints was the design of non-prismatic sections. Part of the research was dedicated to showing that non-prismatic bridges were conservative based on calculations from the AASHTO LRFD Specifications and non restraints should be used when trying to apply the AASHTO LRFD Specifications. Other parts of the research examined how the width, span length, and depth of girders affected the magnitude of the distribution factors. This portion of the study found that the distribution factors from the AASHTO LRFD Specifications were conservative for both bending and shear distribution factors and were more conservative for exterior girders in comparison to interior girders. The effects of skew on distribution factors were also examined. The results were that the LRFD formulas from AASHTO
did not necessarily produce a less conservative estimate of distribution factors outside the 12° limit.

Field testing and analysis of 40 steel stringer bridges (Ruth et al., 2005)

The University of Cincinnati Infrastructure Institute (UCII) under the Ohio Department of Transportation tested 40 steel girder bridges within the state. These bridges were selected to match the inventory of steel girder bridges in Ohio. This testing included the modal and truckload testing to obtain data such as frequencies, mode shapes, flexibility, stresses, and influence lines of the tested bridge. UCII used this data and bridge plans to develop two finite element models for each tested bridge. The first finite element model created was considered a nominal model. This finite element model took only into consideration the material and geometric properties from the bridge plans. The second model was considered a calibrated model in which the data collected from the field tests were matched to that of the finite element model. The latter model was calibrated by using a software package that UCII created. This software program compared the structural response data to the finite element model and then provided an adjustment to one of six parameters that affect the response of the finite element model. This model was continually refined until an acceptable correlation between the finite element model and actual bridge response was obtained. Once the finite element model was considered acceptable, the lateral distribution factors and load ratings of the bridge were calculated and compared to those calculated based on the AASHTO LRFD Specifications. It was found that in one of the bridges that were tested that the AASHTO LRFD Specifications have been conservative in both the distribution factors and load
ratings. The two models (nominal and calibrated) that were developed produced similar results, the calibrated being slightly less conservative. The AASHTO distribution factors and load ratings were highly conservative which demonstrates the inherent conservatism within the AASHTO LRFD Specifications.

**Literature review in analysis of box-girder bridges (Sennah and Kennedy, 2002)**

This literature review examined the different approaches for the analyses of box-girder bridges that have been used over the last 40 years. These different methods consisted of orthotropic plate theory, grillage-analogy, folded-plate, finite-strip, finite element, and thin-walled curved beam theory.

The orthotropic plate theory was suggested mainly for multi-girder straight and curved bridges. The grillage-analogy typically produced satisfactory results except in its ability to model torsional stiffness. The folded-plate method used plane-stress theory and classical two-way plate bending theory; however, this method was complicated and time consuming. The finite-strip method used the total potential energy theorem. The authors noted that this method had been used quite extensively and in comparison to the finite element model provided savings in time and effort; however, this method was limited to simply supported prismatic structures. The authors concluded that the finite element analysis best modeled the behavior of a bridge, although it was the most time and effort consuming. Thin-walled curved beam theory was found to accurately predict the distribution of bending moments, torque, and shear at any section of a curved beam as long as the axial, torsional, and bending rigidities of the section were known. This
method did not accurately model curved box girder bridges and was best used for straight box girder bridges.

In addition to these analytical solutions there have been experimental studies on scale bridges which were used to compare with the computer models. From these scale models the analytical models were verified. The results from the analyses showed that the finite element model provided the best modeling of the static and dynamic analysis of the structural response of a bridge; however, it was also the most involved and time consuming. Another conclusion made was that the end restraints of the models largely affect the flexibility of a bridge.
LIVE LOAD TEST

Bridge description

The Lambert Road Bridge is located near Elk Grove, California, about 30 miles south of Sacramento and carries approximately 30,000 vehicles a day. The bridge is located on Interstate 5 and crosses over Lambert Road. The bridge was designed as a two span, cast-in-place, prestressed, continuous box-girder bridge. The bridge was built in 1975 using an HS20-44 truck as the design live load. The bridge has an overall span length of 78.7 m (258 ft) comprised of two equal spans of 39.35 m (129 ft) with an 8° skew. Figure 1 shows an elevation view of the Lambert Road Bridge.

The width of the deck (including barrier railings) is 12.8m (42 ft). The barrier railings are 0.3 m (1 ft) wide and 0.8 m (32 in.) high. The bridge carries two southbound lanes. The deck was constructed as a 203 mm (8 in.) thick reinforced concrete slab and the bottom flange of the box girder is 152 mm (6 in.). However, the thickness of the flange varies near the pier. At 3.66 m (1 ft) from the pier, the thickness is 152 mm (6 in.). The thickness increases to 254 mm (10 in.) at the pier. The depth of the bridge, including the deck and box bottom, is 1.7 m (66 in.). The box-girder bridge contains four cells which results in an interior girder spacing of 2.74 m (9 ft). The webs of the exterior girders were constructed with a slope of 2 to 1. Each girder web is 0.3 m (1 ft) thick. Figure 2 shows a cross sectional view of the bridge superstructure.

The concrete used in the deck, bottom flange, and girders had a specified 28-day compressive strength of 24.2 MPa (3,500 psi) and was reinforced with grade 60 steel reinforcement except in the deck in which grade 50 reinforcement was used.
Figure 1 Lambert Road Bridge.

Figure 2 Cross sectional view of Lambert Road Bridge.
Each of the girders was prestressed. The prestressing strands followed a parabolic path throughout each span as shown in Figure 3. The prestressing strands were used to create a camber of 2.0 cm (0.8 in.) at the midspan of each span. At the abutment, the center of gravity of the prestressing strands was 88.9 cm (35 in.) above the bottom of the flange. At 15.72 m (51.6 ft) from the abutment wall the strand center of gravity was 27.94 cm (11 in.) above the bottom of the flange. A point of inflection occurred at 3.94 m (12.9 ft) from the center of the bent. At this location the strand was 50.8 cm (20 in.) from the top of the deck. At the centerline of the bent the strand was 38.1 cm (15 in.) from the top of the deck. The stands used for the post tensioning were low relaxation strands and were not continuous at the pier. This meant that ten girders were post-tensioned rather than five girders. The strands were jacked to a force of 7.52 kN (1.69 kips) which included friction and stress losses.

The diaphragms of the bridge are located at the midspan of each span. These diaphragms are 203 mm (8 in.) thick. Each of these diaphragms has a section cut out for access of the entire cell. An intermediate diaphragm is located at the pier and is 1.83 m (6 ft) thick. The diaphragms follow the 8° skew of the bridge and are comprised of reinforced concrete. The concrete had a specified 28-day compressive strength of 24.2 MPa (3,500 psi) and was reinforced with grade 50 steel reinforcement.
The bridge is supported at the midspan by a bent cap. This cap encompasses the width of the bridge and is 1.83 m (6 ft) thick. The bent column is 1.07 m (3.5 ft) thick and has a varying transverse width. At the ground it is 3.66 m (12 ft) wide and follows a 14 to 1 slope upward to the bottom of the superstructure. This column is supported by a foundation of 5.48 m by 3.66 m (18 ft by 12 ft) and 1.07 m (3.5 ft) thick. This foundation is supported by 24-406.4 mm (16 in.) cast-in-drilled-hole concrete piles having a design loading of 623 kN (70 tons). The reinforced concrete for each of these structural elements is comprised of concrete with a specified 28 day compressive strength of 24.2 MPa (3,500 psi) and grade 60 steel reinforcement. The details of the bridge bent can be seen in Figure 4 and Figure 5.

Figure 4 Elevation view of bent.
The bridge is supported at the ends by abutments which have wing walls attached to the abutments. The abutments, details seen in Figure 6, are 0.46 m (1.5 ft) thick and rest on a neoprene bearing pad. The abutment is supported by a reinforced pile cap which is 12.96 m (42.5 ft) long, 1.22 m (4 ft) wide, and 0.46 (1.5 ft) deep. Each pile cap is supported by 7- 406.4 mm (16 in.) cast-in-drilled-hole concrete piles with a design loading of 623 kN (70 tons). Each wing wall was attached to the abutment after the stressing of the strands in the girders. The wing wall extends outward of the bridge by 5.49 m (18 ft) from the centerline of the abutment and has a thickness of 0.3 m (1 ft). All concrete in these structural elements had a specified 28-day compressive strength of 24.2 MPa (3,500 psi) and was reinforced with grade 60 steel reinforcement.
Instrumentation and load paths

A live load-test of the Lambert Road Bridge was completed in the fall of 2009 in a joint partnership of Utah State University and Bridge Diagnostic Inc (BDI). The main objective of the live load test was to obtain measurements of the response of the bridge while being subjected to a truck load driven at a crawl speed and to use this data to calibrate a finite element model. In general, the live load test consisted of driving a single truck at a low speed (about 8 km/hr or 5 mph) across an instrumented bridge at a selected load path.

The live load test on the Lambert Road Bridge consisted of installing 53 instruments on the bridge. The instrumentation package included 42 strain transducers, 8 twangers (displacement sensors), 2 string pot displacement sensors, and 1 uniaxial rotation sensor.
A twanger consisted of a 45.7 cm (18 in.) long and 3.2 mm (1/8 in.) thick tapered aluminum cantilevered beam. This beam was 11.4 cm (4.5 in.) wide at the fixed end and was tapered down to 2.54 cm (1 in.) on the free end. It had two foil strain gauges attached on the top and bottom sides of the beam near the fixed end which were wired in a full bridge configuration. The twanger was attached to the bottom of a girder and the free end was anchored to the ground with a small chain. When the twanger was anchored to the ground, it was then displaced approximately 3.8 cm (1.5 in.). As the truck was driven along the length of the bridge, the girders deflected which reduced the magnitude of the predeflected cantilevered tip. The change in displacement was obtained by converting the change in voltage from the full bridge arrangement of the strain gauges to a deflection by using the calibration numbers in Table 1. Figure 7 shows a twanger attached to the bottom of a girder on the Lambert Bridge. Figure 8 shows the twangers at Section F which were anchored to the ground by attaching the chain from the twanger to a bucket filled with soil.

Table 1 Twanger calibration numbers

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<td>8</td>
<td>-115.11</td>
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Figure 7 Typical twanger attached to a girder.

Figure 8 Twangers attached to girders and anchored to ground.
The system used to collect the data during the live load test was BDI Wireless Structural Testing System (STS-WiFi). This system operated by having up to four instruments (strain gauge, twanger, string pot, tiltmeter, etc.) attached to a node. This node functioned as a 4 channel data acquisition module in which the node transmitted the data to a base station. This base station received signals from all the nodes and then transmitted the data to a laptop where the data was recorded and saved. Included in this system was a device called an autoclicker. The autoclicker was used to locate the position of the truck as it was driven across the bridge. The autoclicker was attached near the wheel and had a laser attached to it. Modified vice grips with reflective tape was attached to the rim of the wheel. As the tire rotated while being driven across the bridge, the vice grips systematically passed by the laser of the autoclicker which then placed a voltage splice in the data file to record the truck’s position.

Instrumentation plans for the north and south spans can be seen in Figure 9 and Figure 10. In Figure 11 and Figure 12 cross section views of the instrumentation can be seen. The north span was instrumented with 25 strain transducers, two string pot sensors, and three twangers. Section A, located 2.8 m (9.3 ft) south of the abutment wall, was instrumented with strain sensors B1307, B1298, B1310, B1311, and B1300. These sensors were placed on the bottom of the girders so that each girder was instrumented. Section B was located 22.6 m (74 ft) south of the abutment wall, which location was approximately 0.6L (L is the length of the span). Five strain transducers (B1128, B1380, B1394, B1331, and B1312) were placed on the bottom of the centerlines at the girders at this section. Additional strain transducers were placed transversely from the centerline of girder 2. Sensors B1326 and B1243 were placed 0.9 m (3 ft) and 0.3 m (1 ft) east of
girder two on the bottom of the superstructure. Sensors B1334 and B1337 were placed 0.9 m (3 ft) and 0.3 m (1 ft) west of girder 2 on the bottom of the bottom flange. Sensor B1344 was placed at the same location of B1334 but on the top of the bottom flange. By arranging these sensors in this configuration, the bending behavior of the bottom flange could be observed. Sensor B1352 was placed 76.2 mm (3 in.) above the top of the bottom flange on the web of girder 2. Two additional sensors, B1301 and B1390, were placed 1.22 m (4 ft) from the bottom of the bottom flange on the outside of the web of the exterior girders. Section B’ was located 24.1 m (79 ft) south of the abutment wall. At this section, girder two was instrumented to find the location of the neutral axis. A strain transducer (B1319) was placed 76.2 mm (3 in.) from the top of the bottom flange on the side of the web of the girder. The other transducer (B1355) was located on the same web but 0.99 m (39 in.) from the top of the bottom flange. Section D, located 3.6 m (11.7 ft) from the pier, was instrumented with one strain transducer (B1336, B1383, B1129, B1217, and B1126) at the centerline of each girder. Five displacement sensors (SP473, SP471, twanger 6, twanger 7, and twanger 8) were installed on this span at Section C. This section was located 10.9 m (35.8 ft) north of the pier wall. The east exterior girder and middle girder (girders 3 and 5) each had a string pot displacement sensor and a twanger attached to the girders. The other exterior girder (girder 1) had a single twanger attached on it.

The south span was instrumented with sections near the pier, midspan and, abutment of the span. At Section E, 5.3 m (17.3 ft) south from the pier wall, five strain transducers (B1305, B1389, B1342, B1303, and B1044) were attached at each of the centerlines of the girders. One twanger and one strain transducer (twangers 1 to 5,
B1540, B1087, B1329, B1297, and B1046) were installed on each girder at Section F, located 16.0 m (52.6 ft) north of the abutment wall which location was approximately 0.4L. Also at this section, two additional strain transducers (B1061 and B8390) were placed on the outside girder webs, one for each girder. These were placed 1.2 m (4 ft) along the face of the girder from the bottom of the flange. The final section for this span, Section G, was 2.5 m (8.1 ft) north of the abutment wall. This section was instrumented just as Section E was with one strain transducer (B1097, B1795, B1321, B1379, and B1131) attached on each girder. In addition to these gauges, a tiltmeter was attached 0.3 m (1 ft) from the face of the abutment wall on the middle girder.

Sections A and G were instrumented near the abutments to obtain the boundary conditions of the supports. Sections D and E were instrumented near the pier to determine the fixity and stiffness of the pier. Sections B, C, and F were instrumented near the midspan to obtain the larger strains and deflections. Section B' was instrumented so that the neutral axis of the girder could be quantified. Sections B and F were placed at 0.4L and 0.6L and not necessarily right at midspan because velocity transducers for the dynamic testing were placed at these locations so future correlations could be made.

For the live load test, five different load paths were chosen as shown in Figure 13. The load paths were selected to maximize the moment in different girders. The path distances from the centerline of the passenger front axle tire to the edge of the barrier railing (0.3 m out from the edge of the bridge) were 1.7 m (5.6 ft) for Path Y1, 3.7 m (13 ft) for Path Y4, 5.1 (16.7 ft) for Path Y2, 7.5 m (24.7 ft) for Path Y5, and 10.6 m (34.8 ft) for Path Y3.
Figure 9 North span.

Figure 10 South span.
Figure 11 North span instrumented cross sections (looking south).
Figure 12 South span instrumented cross sections (looking south).
Two different trucks were selected to apply the loads for the live load test. The first truck was a tandem rear axle dump truck. This truck had a total weight of 290 kN (65,180 lbs). The front axle had a weight of 60 kN (13,580 lbs) and the back axles each had a weight of 115 kN (25,800 lbs). The axle spacing was 5.8 m (19 ft) for the front axle and 1.3 m (4.3 ft) for the back pair. The width between the front tires was 2.0 m (6.7 ft) and the back tires were 2.2 m (7.2 ft). The truck and axle layout can be seen in Figure 14 and Figure 15.

The second truck was an 18 wheel hauler truck. This truck had a front axle, middle pair axle, and a rear pair axle. The front axle spacing was 5.3 m (17.3 ft), the middle to back pair spacing was 6.6 m (21.7 ft), with each pair axle spacing being 1.3 m (4.3 ft). The width of the front axle was 2.1 m (6.8 ft) and the other axles were 2.2 m (7.2 ft). This truck had a total weight of 325 kN (73,060 lbs). The front axle had a weight of 44 kN (9,840 lbs), the rear axles had a weight of 144 kN (32,360 lbs), and the middle axles had a weight of 137 kN (30,860 lbs). The truck and axle layout can be seen in Figure 16 and Figure 17.
In all, sixteen semi-static live load tests were conducted using the chosen trucks on the load paths. During each test, the trucks were driven approximately 8 km/hr (5 mph) on a selected load path. On some load paths, tests were repeated to ensure that the data collected was valid and that the individual tests produced similar results in each load path. In addition to these static tests, one dynamic loading test was carried out. This was done by having the two trucks drive on load path Y4 at approximately 88 km/hr (55 mph). The two trucks were separated by about 300 m (328 ft) so that the measured response would be decoupled.
After analyzing the strain data, it was determined that high-quality data was recorded. The peak strains from all the tests were just over 20 microstrains while most of the tests yielded maximum strains of 12-18 microstrains. These low peak strains can be attributed to the low weight of the test trucks and or a stiff bridge. Observations from the live load data indicated that the strain readings returned to zero once the truck was off the bridge. Because of this, it can be concluded that the bridge behaved linear-elastically. In some instances, repeat tests were done with the same truck and load path. This was done to ensure that each load path had a good set of data in case one test yielded bad results. From these duplicate tests, it can be clearly seen in Figure 18 that the bridge behaved linear-elastically.

**Strain results**

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Figure 16 18 wheel hauler truck.

Figure 17 18 wheel hauler truck axle layout.
Figure 18 Strain comparison of strain gauge B1297 between two tests on load path Y1.

similarly for similar tests. Figure 18 shows the response of a strain transducer as the 18-wheeler drove across path Y1. Although there is a subtle difference between the two tests, the differences are small and are considered negligible.

It was determined that there were cases of the strain transducers experiencing thermal drifting during the testing. These drifts occurred because the strain transducers had little mass so any small temperature change caused the strain gauges to drift quickly. These drifts varied on each test and sensor. In the first test, thermal drifts were on average about 0.5 to 1.0 microstrain. However, some of the thermal drift magnitude was over half of the actual recorded strain value. These strain values were for transducers that were typically found near the abutments and had low readings of strain (less than 2 microstrain). For larger strains near the midspans, the drift accounted for a small portion of the magnitude and could be considered negligible. In the live load test which was used to calibrate the FEM model, the thermal drifts were quite small in comparison with those
in the first test. In this test, the average thermal drift strain was negligible (less than 0.5 microstrain). Only a handful of transducers experienced any thermal drift that was considered significant. Even though some of these drifts could be considered negligible, they were still accounted and corrected for. To correct for this drift, it was assumed that the thermal drift was linear during the testing period. The difference between the end strain and zero was taken and divided by the length of the testing period. This number was added or subtracted from each time increment to obtain a thermal correction for the strain.

The lateral distribution was also examined to see how the truck loads were distributed laterally across the bridge. In Figure 19, strain gauges from Section B can be seen as the dump truck was positioned at 28.96 m (95 ft) for three separate load paths. The different load paths were Y1, Y2, and Y3. The Y1 and Y3 load paths were chosen because they maximized the strain in the exterior girders. Y2 was chosen because it was
the load path used to calibrate the finite element model and because it maximized the strain in the center girder. Examination of Figure 19 shows that an excellent distribution occurred within the bridge. The exterior load paths had nearly identical distributions just in opposite directions. In looking at load path Y2, it is evident that the distribution was nearly symmetric about the center of the cross section.

**Deflection results**

String pot sensors and twangers were used during the live load testing to determine the bridge deflections. The deflection measurements from these sensors were important because the deflections were indicative of the global response of the bridge from a truck load. From the data recorded, it was observed that all but one of the deflection instruments behaved accordingly. The result from one of the twangers was found to give erroneous results due to a faulty strain gauge. During the calibration process (after the testing had been completed), one of the twangers produced erratic results. Although the data from the live load testing showed that the twanger was properly functioning during this test, an exact calibration number couldn’t be determined so this data was ignored.

In order to allow a comparison of the recorded changes in deflection from the string pot sensors and twangers, two twangers were placed directly by string pot displacements sensors. For the most part, it was concluded that the twangers and string pot sensors behaved similarly. One of the twangers had nearly identical behavior and magnitude as the string pot sensor for multiple tests. The other twanger had the same overall trends as the string pot sensor but had lower peak magnitudes by up to 20% during some tests. Figure 20 and Figure 21 show influence lines for both string pot
sensors and twangers. Figure 20 shows the displacement influence line for the 18 wheeler truck on path Y4. Figure 21 shows the displacement influence line for the dump truck on path Y5. Although there were some differences in this data, the differences were small. From this data it can seen that as the truck was on one span, the other span also experienced some changes in deflection. This indicates that moment was being transferred across the bent resulting in some continuity. Observations from the data for multiple tests showed that the typical maximum displacement was quite small, less than 2 mm (0.08 in.).

Figure 20 Displacement comparison between string pot SP471 and twanger 7.
Rotation results

The live load test had one tilt meter installed on the bridge 0.3 m (1 ft) from the south abutment. The rotations from the live load test for load path Y2 can be seen in Figure 22. From this data it can be seen that the rotation near the abutment was quite small. Also, because there was rotation near the abutment when the truck was on both spans it can be concluded that the pier does not behave fully fixed but partially fixed because there was rotation when the truck was on the opposite span.

Dynamic live load results

As part of the live load test, the 18-wheeler truck and dump truck drove across the bridge at approximately 90 km/hr (55 mph) on load path Y4. This was done to observe the structural dynamic response of the bridge and to observe the effects of the dynamic impact factor. The dynamic response of a bridge to a moving load is important in bridge design since it can increase the design load. The effect is taken into account by increasing
the design moments and shears by up to 33% as outlined in the AASHTO LRFD Specifications (AASHTO, 2007). To observe the dynamic response of the bridge, the strains and deflections of the bridge were compared for the static and dynamic load cases.

In Figure 23 and Figure 24, a comparison of a midspan strain gauge can be seen for both the static and dynamic testing for the 18-wheeler truck and dump truck on load path Y4. It is evident that the effects of the high speed of the truck caused an increase in the peak strain. Even long after the truck leaves the bridge, the bridge was still oscillating. In Table 2, 10 strain transducers that were located at Sections B and F were compared for the dynamic and static loadings. This table lists the maximum strains for a static test and corresponding dynamic test for both the dump truck and the 18-wheel hauler truck. From this table, it can be seen that in most cases the dynamic effects of a moving truck caused a significant increase in peak strain. The average strain increase for these sensors due to the dynamic effect of the trucks was over 20%.
Figure 23 Dynamic and static strains for 18-wheeler loading for sensor B1046.

Figure 24 Dynamic and static strains for dump truck loading for sensor B1046.
Table 2 Dynamic and static strain comparison

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<tr>
<th>Sensor</th>
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The dynamic effects from moving vehicle loads was also examined for deflections. In Figure 25 and Figure 26, twanger 5 can be seen as the two test trucks drove over the bridge on Path Y4 for both the static and dynamic loading. A significant increase in deflection occurred and the bridge continued to oscillate long after the truck(s) was off the bridge. The maximum deflections for each deflection sensor are listed for the loading of the test trucks on Path Y4 for the static and dynamic loadings in Table 3. From this table, it can be seen that a significant increase in deflection occurred when the bridge was loaded with a moving truck. The average increase in deflection was over 35% for the deflections in both truck cases. Another observation that was made was that typically the dump truck had higher amplification effects from the dynamic loading than that of the 18 wheel hauler truck. This was expected since the AASHTO LRFD Specifications (article C3.6.2.1) specify that generally trucks with more axles and higher weights produce lower amplifications than those with fewer axles and less weight. From
the collected data and for this particular bridge it was evident that the dynamic effects of a moving load cannot be ignored in bridge design.

Figure 25 Dynamic and static displacements for 18-wheeler loading for twanger 5.

Figure 26 Dynamic and static displacements for dump truck loading for twanger 5.
Table 3 Dynamic and static displacements comparison

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<th>Sensor</th>
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FINITE ELEMENT ANALYSIS

Description of finite element model

The finite element model for the Lambert Road Bridge was created using solid elements in SAP2000 v.14 (Computers and Structures, 2009). A solid element in the finite element was developed by extruding a shell element to a specific thickness. Most of the solid elements were eight node hexahedrals; however, at the diaphragms and skew sections of the bridge six node triangular solid elements were used. These six node triangular elements were used because the eight node solids became too distorted at these particular locations; however, few triangular elements were used in the finite element model. Each of the nodes of the solid element had three translational degrees of freedom. One of the disadvantages of using solid elements was that the solid elements can be subjected to a condition called shear locking. This condition occurred when the solid element became too stiff under a bending moment and shear deformations occurred instead of bending deformations. To help correct this problem, the incompatible mode was used in SAP2000. This option significantly increased the behavior of the solid element to model bending deformations (Computers and Structures, 2009).

To ensure the highest accuracy of the model, several modeling techniques which were considered good practice were implemented such as low aspect ratio, avoidance of small or large angles in elements, small elements, and avoidance of small to large element transitions. These principles refined the model and prevented inaccuracies from emerging during the analytical process. It was important during the modeling of this bridge to keep the aspect ratio at or below 4 but not to exceed 10. This ratio compared the length of the largest dimension of an element to the shortest dimension of an element.
For the most part, the aspect ratio was kept below 4. Special cases, because of the complex geometry conditions, warranted that the ratio exceed 4 but was still kept below 10. In the cases where the ratio exceeded 4, the ratio was usually only 5 or 6. It was also important that the angles in the solid elements be kept at 90° whenever possible. If this was not possible, it was important that the angles be in the range of 45° to 135°. These guidelines were used because it improved the accuracy of the finite element model by not allowing the elements to become too distorted (Computers and Structures, 2009). Most of the elements in the finite element model were rectangular; however, in the instances where the elements were not rectangular the angles were kept between the 45° to 135° range. To keep the mesh fine, the longitudinal nodes were typically 0.3m (1 ft) on center. Longitudinal increments at diaphragms were sometimes larger, up to 0.76 m (2.5 ft), because of the skewed geometry. In the transverse direction, the nodal locations were typically 38.1 cm (15 in.) or less on center. In the vertical direction, the dimensions of the elements ranged from 15.2 cm to 45.7 cm (6 in. to 18 in.). By using small elements the finite element model was refined and any large to small element transitions were avoided to increase the accuracy of the finite element model. Figure 27 shows a cross sectional view of the finite element model of the Lambert Road Bridge.

Figure 27 Cross-sectional view of FEM showing the solid elements.
The finite element model was divided into several sections with each section having different material properties. These sections included the deck, bottom flange, girders, diaphragms, and parapets. These values as well as the boundary conditions were changed until a strong correlation between the finite element model and the test data was found. The final modulus values for these properties ranged from 6,210 MPa to 49,640 MPa (900 to 7200 ksi).

In addition to using solid elements, tendon elements were used. These tendons were used to model the post tensioning strands in each of the girders. Tendons are elements that are embedded in other elements such as a solid in this case. The tendons can be modeled either as loads or elements. Modeling as an element takes into consideration loses due to elastic shortening and time dependent effects, which were not considered for this finite element model. For this finite element model, there was no concern for any long-term effects so the tendons were modeled as loads. Although a tendon has 6 degrees of freedom, it cannot have more degrees of freedom than the element it is embedded in (Computers and Structures, 2009). So in this case, the tendon was restricted to three translational degrees of freedom. The tendons were attached from the abutment end to the bent. The tendons followed a parabolic path as shown in Figure 3. Ten tendons were used to model the post-tensioning as loads and were discretized in 1.5 m (5 ft) sections along the longitudinal direction.

In all 10 tendons were used, approximately 32,000 solid elements were used, and about 55,000 joints were created for the finite element model. Figure 28 shows a 3D view of the finite element model.
The finite element model was calibrated to replicate the behavior of the actual bridge by using the data collected during the live load and dynamic tests. The live load data included the rotation, strains, and displacements from the testing of the Lambert Road Bridge. The dynamic data included the mode shapes and frequencies of the Lambert Road Bridge. The dynamic-test data was gathered during a force vibration dynamic test. A more detailed description of the test and results can be found in Tim Thurgood’s thesis. (Thurgood, 2010). This testing was completed during the same week as the live load test. The test involved placing six vertical and one horizontal velocity transducers at various locations on the bridge as can be seen in Figure 29. The vertical transducers were placed on the deck near the parapets while the horizontal transducer was placed inside the box girder cell. In addition, an electro-magnetic shaker applied a sinusoidal force to the bridge while the velocity transducers recorded the corresponding response of the bridge. From this data, the first six mode shapes and frequencies of the bridge were obtained.
This data was used to calibrate the finite element model with the dynamic properties of the bridge and played a major role in determining the boundary conditions of the finite element model.

The boundary conditions were obtained by examining the data from both the live load and dynamic tests. From the live load test, the strains at the piers and abutments were examined. These strains showed that a negative moment was developed at the supports. From this behavior it could be concluded that the supports were behaving in a semi-fixed condition. Since the tiltmeter recorded rotations when the truck was on both spans of the bridge it was evident that the supports were partially fixed. To determine the degree of fixity, different restraints were applied to the abutments and pier until the
dynamic response of the finite element model correlated with the mode shapes and
frequencies of the data collected during the dynamic testing. The best results were
obtained by placing linear restraints in the longitudinal and vertical directions at the top
of the deck at the abutments. The restraints that were placed at this location were springs.
The vertical springs had a large stiffness of 175,130 kN/mm (1,000,000 k/in.) while the
longitudinal springs had a stiffness of 175 kN/mm (1,000 k/in.). Additional vertical and
transverse springs were placed at the bottom of the abutment and had stiffness values of
10,510 kN/mm and 88 kN/mm (60,000 k/in. and 500 k/in.). To model the behavior of the
pier, springs were placed at the bottom of the flange at the location of the pier. These
springs were in the transverse and vertical directions. The stiffness of the springs in the
transverse direction was 880 kN/mm (5,000 k/in.) while the vertical springs varied in
stiffness. These stiffness values were 17,510 kN/mm (100,000 k/in.) at the center of the
pier while the other vertical springs at the pier had magnitudes of 1,140 kN/mm (6,500
k/in.). With these boundary conditions, the first three mode shapes and frequencies of the
finite element model were correlated to the actual structural response of the bridge. To
ensure that the mode shapes from the finite element model were the same from the
dynamic test data, a MAC (Modal Assurance Criterion) analysis was done. The MAC
analysis insured that a mode shape from the dynamic testing at a particular frequency was
the same mode shape of that from the finite element model. In some cases, the finite
element model would yield the same frequency as the test data but produced a different
mode shape. The mode shape from the finite element model may have been controlled in
the transverse or longitudinal direction while the actual test data showed that the majority
of the participating mass was actually in the vertical direction. By performing a MAC
analysis, it could be determined that these two mode shapes were not the same even though they yield the same frequency.

The MAC analysis results in a vector comparison between two mode shapes and is represented by a number ranging from 0 to 1. A value of 1 indicates that two mode shapes are the same while a value of 0 showed there is no correlation between two mode shapes. A value of 0.6 indicates an acceptable correlation. The MAC analysis can be shown mathematically as shown in Equation 1.

\[
\text{MAC} \left( \{\Phi_A\}_q, \{\Phi_B\}_r \right) = \frac{||\{\Phi_A\}_q^T \{\Phi_B\}_r||}{(||\{\Phi_A\}_q^T \{\Phi_A\}_q|| ||\{\Phi_B\}_r^T \{\Phi_B\}_r||)}
\]

\( \{\Phi_A\}_q \) = Mode shape vector for mode q of data set A
\( \{\Phi_B\}_r \) = Mode shape vector for mode r of data set B

A MAC analysis was completed for each of the first three mode shapes by comparing the relative displacements from the finite element model to the displacements from the dynamic test. A strong correlation was found between the first 3 modes of the dynamic testing and the finite element model. These three modes are listed in Table 4 with a comparison between the frequencies and the value from the MAC analysis. In Table 5, the MAC matrix can be seen. This table shows a comparison between the first three modes of the finite element and dynamic test data. The values of the diagonal represent the MAC number between the modes. The non-diagonal values show the comparisons between different mode shapes. A value of zero indicated that no relationship existed between the different mode shapes and was the desired value.

The first three modes from the finite element model can be seen in Figure 30, Figure 31, and Figure 32.
The finite element model had an excellent correlation for the first two modes as denoted by its high MAC value. Although Mode 3 did not have an excellent MAC value it still exhibited a strong correlation between the finite element model and test data. From Table 5 it could seen the numbers not on the diagonal were near or at zero which was also indicative of a good relationship between the two sets of data. The frequencies between the finite element model and dynamic test of the Lambert Bridge also indicated a strong correlation.

Table 4 Dynamic and modal testing comparisons

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<th>Model Frequency</th>
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Table 5 MAC matrix

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<th>Mode 2</th>
<th>Mode 3</th>
</tr>
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Figure 30 Finite element model representation of mode 1.
Comparison with strain

Prior to using the finite element model to predict future behavior, it must be calibrated by comparing its strains, deflections, and rotations with the measured field results. The Lambert Road Bridge was instrumented with sensors near the abutments, pier, and midspans. From observations, deflections and strains near the midspans
experienced the largest magnitudes. These values were considered the most critical since they reflect the bridge behavior at locations that would experience the highest stresses. For the Lambert Road Bridge, the critical locations were Sections B and F.

The solid element used to develop the finite element model did not calculate the strain at a location directly but it did calculate the stress. The change in strain from the finite element model predicted was calculated by using the stress from the solid element at a particular location, where that location represented the location where the strain transducer of interest was attached to the bridge. Typically 4 solid elements were connected to a joint or node where a node represented a strain transducer on the Lambert Road Bridge. The stresses of 4 solids at a node were averaged to obtain the stress. The modulus of elasticity was known so the strain was calculated using Hooke’s law which is defined in Equation 2.

\[ \sigma = E \times \varepsilon \]  

(2)

\( \sigma \) = Change in stress due to the applied load  
E = Young's modulus of elasticity of concrete  
\( \varepsilon \) = Change in strain due to the applied load

A comparison of the measured and predicted strains can be seen in Figures 33 to 40. These figures show the live load data graphed with the strain calculated from the finite element model. It should be noted that the live load data was collected at 40 Hz (about 6 cm or 0.2 ft increments) whereas the truckloads were placed on the finite element model typically in 3.05 m (10 ft) increments. As a result, the finite element model had more interpolation in the graphs than the live load data.
The most important sections used for calibrating the finite element model were Sections BB and FF. The sensors that were mainly considered at these sections were those that were at the centerline of the girders and not those used to find the neutral axis or compare the bending of the lower flange. These sections were located near the midspans on each span and had the highest strain recordings. Typical peak strains ranged from about 12 to 20 microstrain. These strain gauges were important because they recorded the maximum strains in the bridge and were the most important to ensure that the finite element model was capturing the large strains of the bridge. Figure 33 through Figure 36 show two sensors from each midspan section. It can be seen from these figures that an excellent correlation exists between the live load data and the finite element model. Although, the data did not match up extremely close when the truck was on the opposite span, it should be mentioned that the finite element model was off by about only 1 microstrain from the live load data.

The strain gauges were examined near the abutments. The abutments were primarily instrumented to determine the boundary conditions that exist at the supports. The gauges near the abutments experienced a compressive strain when the truck was on the opposite span indicating that there was a negative moment near the abutment. This means that the abutments did not act pinned or as a roller but partially fixed. Figures 37 and 38 show the relationship between the finite element model and the live load data for strain transducers located at Section A and G which were located about 2.46 m and 2.85 m (8.2 and 9.3 ft) from the abutment wall. Figure 37 shows the result when the truck was directly over an abutment sensor while Figure 38 was more representative of a typical abutment strain transducer. The small strains in Figure 38 should be noted and realized.
that other strain transducers at the abutments typically recorded even lower strain magnitudes. Upon examination of Figure 37 and Figure 38, it did not appear that the strongest correlation existed between the live load and finite element model at certain truck positions. However, at these particular locations the strain was smaller than 1 microstrain. Modeling these small strains was difficult and was not nearly as important as reproducing the large strains that occurred. Because many of the strain data at the abutments were of such small magnitude, it was not as important for the finite element model data to match up with the live load data. It was more important that the large strains were captured rather than the small strains near the abutments.

Figure 33 Strain comparison between strain gauge B1128 at Section B and FEM.
Figure 34 Strain comparison between strain gauge B1331 at Section B and FEM.

Figure 35 Strain comparison between strain gauge B1297 at Section F and FEM.
Figure 36 Strain comparison between strain gauge B1329 at Section F and FEM.

Figure 37 Strain comparison between strain gauge B1310 at Section A and FEM.
In Figure 39 and Figure 40 the graphs for the sensors that were located near the pier can be seen. These sensors were important during the calibration process because the strain values as compared to other sensor locations were of respectable magnitude and were useful in determining the stiffness and behavior of the pier. As seen in Figure 39 and Figure 40, a good relationship can be seen between the live load and finite element model data. Although the finite element model did tend to overestimate the actual strain value at high strain locations, it was only by about 1 micro strain which was about 10% larger than the live load data. These figures were quite typical for the rest of the sensors at these locations. From these two figures it was evident that some fixity exists at the pier.

To determine the overall relationship between the finite element model and the actual bridge behavior, the strains between these two variables were plotted against each other. The more linear the relationship between the trend line of actual bridge data and
the finite element model was indicative of a more accurate model. For this particular graph, the sensors at the centerlines of the girders at the critical sections (Sections B and F) were graphed against each other as can been seen in Figure 41. The results were that a linear relationship of 0.97 existed between the two sets of data. This means that finite element model was less than 3% from predicting the actual strain. The coefficient of correlation, $R^2$, can also be seen on the graph. A value of 0.97 was found which indicated a very good relationship between the finite element model and actual bridge behavior. This means that almost 98% of the live load data can be explained by the finite element model.

![Figure 39 Strain comparison between strain gauge B1336 at Section D and FEM.](image)
Figure 40 Strain comparison between strain gauge B1044 at Section E and FEM.

Figure 41 FEM vs live load strain relationship.
It was also important to ensure that the finite element model was distributing the applied loads laterally to the adjacent girders. In Figure 42 and Figure 43 the lateral distribution for the live load and the finite element model can be seen. Figure 42 shows a lateral section with the tuck on load path Y2 at truck position 29 m (95 ft). Figure 43 was the lateral distribution for the same cross section but with the truck on load path Y1 and 29 m (95 ft). From these figures it was evident that the finite element model was distributing the loads among the girders as expected.

Comparison of the neutral axis was also examined as part of this analysis. Strain transducers B1355 and B1319 on Section B’ were arranged so that the neutral axis and member stiffness could be determined. By comparing the neutral axis between the finite element model and live load data it could be seen if the cross section of the finite element model was stiffened proportionally. This means that the bottom flange or deck of the finite element model was not behaving too stiff or not stiff enough. The sensors used in determining the location of the neutral axis can be seen Figure 44 in which a good relationship can be seen between the finite element model results and the live-load data. The neutral axis was plotted for the live load data and the finite element model as seen in Figure 45. In this figure, only the neutral axis from the truck position 25 m to 40 m (82 ft to 131.6 ft) was plotted. This was done because after 40 m there were numerous spikes as the truck crossed midspan and before 9 m the neutral axis was still spiking rapidly since both the strain transducers were either negative or positive. By limiting the graph to the truck positions of 25 m to 40 m, the graph of the neutral axis was cleaner which made it easier to recognize the neutral axis. The average neutral axis location was determined using all the test data and was calculated to be 91.44 cm (36 in.) above the bottom of the
concrete box for the live load data, 88.9 cm (35 in.) for the finite element model, and 86 cm (34 in.) for the calculated value based on geometry. These values were very close to each other. Differences between the live load and calculated value were because the concrete in the deck had a higher compressive strength than that in the bottom flange and the effects of the parapets increased the strength in the top portion of the bridge moving the location of the neutral axis upwards. Because the value of the neutral axis was very close to the live load value, the finite element model was stiffened proportionally.

Figure 42 Lateral distribution of dump truck on load path Y2.
Figure 43 Lateral distribution of dump truck on load path Y1.

Figure 44 Strain gauges at Section B' configured to locate neutral axis.
Figure 45 Neutral axis location for FEM and live load data.

Comparison with deflection

Comparing the finite element model deflections with the measured bridge deflections was important because it defined the global behavior of the bridge. Strain correlation was important but the strains could have been subjected to local strain behavior and not necessarily the global response of the bridge. Several figures of the comparison between the finite element model and measured displacements from the live load test can be seen in Figures 46 through 49. Figure 46 and Figure 47 are from Section C while Figure 48 and Figure 49 are from Section F. From these figures it was evident that an excellent correlation existed between the deflection values for the finite element model and the live load data. It can also be seen that even when the truck was on the opposite span of the deflection sensor that a positive deflection occurs. This validated that the pier was partially fixed as gathered from previously mentioned strain and rotation data.
Figure 46 Displacement comparison between twanger 7 and FEM.

Figure 47 Displacement comparison between twanger 6 and FEM.
Figure 48 Displacement comparison between twanger 5 and FEM.

Figure 49 Displacement comparison between twanger 4 and FEM.
To determine the overall correlation between the finite element model and the live load test data, the two sets of deflection data were plotted against each other as seen in Figure 50. The more linear the relationship between the trend line between the live load test data and the finite element model indicated a more accurate model. The results were that a linear relationship of 0.97 existed between the two sets of data. This indicated that a good relationship between the finite element model and the measured deflections existed. The coefficient of correlation, $R^2$, is also shown on the graph. A value of 0.98 was obtained which indicated a very good relationship between the finite element model and actual bridge data and that almost 98% of the live load data can be predicted by the finite element model.

![Figure 50 FEM vs live load displacement relationship.](image-url)
Comparison with rotation

The Lambert Road Bridge had one tilt meter installed on the middle girder 0.3 m (1 ft) from the south abutment wall during live load test. A comparison between the rotations for both the finite element model and live load data are shown in Figure 51 and Figure 52. The magnitudes of the rotations were very small with the peak rotation values of approximately 0.00015 radians (.0086 degrees). In Figure 52, the rotations from the finite element model and data from live load test were graphed against each other for load paths Y1, Y2, and Y3. As seen in the figure, a linear relationship of 0.99 existed between the two sets of data. Also shown on the figure is the coefficient of correlation which had a value of 0.96. This means that the finite element model can accurately predict nearly 96% of the live load data. By having a strong correlation of rotation near the abutment, it was evident that the boundary conditions at the abutment were modeled correctly.

Figure 51 Comparison between tiltmeter and FEM on path Y2.
Figure 52 Comparison between tiltmeter and FEM on path Y1.

Figure 53 FEM vs live load rotation relationship.
Comparison of distribution factors.

The live load distribution factors can be defined as how well a vehicle load is laterally distributed among the girders of a bridge. There are two types of forces that can control using the distribution factors, shear and moment. All bridge girders must be properly designed for both types of loads. For this study only the moment distribution factors are examined. The distribution factors for the shear force were not considered because they could not be measured in the field. The calculated live load distribution factors that were compared were based on recommendations from the 4th edition of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007). This code recommended various equations in determining the distribution factors for exterior and interior girders and different types of bridges. In an ideal situation, a finite element model of every bridge designed would be developed and the distribution factors could be obtained using this model. However, creating an accurate model and obtaining the distribution factors may not be the most effective use of time because it can be tedious and time consuming. In order to simply the process, the AASHTO LRFD Specifications were developed as a set of guidelines for finding these distribution factors. These guidelines were based on an NCHRP study and have been found to lead to conservative results. However, they do provide a reasonable value for a simplistic calculation.

From the AASHTO LRFD Specifications, the Lambert Road Bridge was classified as a type “d” bridge and as such, the distribution factors for an interior girder are defined in Table 4.6.2.2.2b-1 in as Equation 3 for a single lane or as Equation 4 for multiple lanes (AASHTO, 2007):
DF = \left(1.75 + \frac{S}{3.6}\right) \left(\frac{1}{L}\right)^{0.35} \left(\frac{1}{N_C}\right)^{0.45}

(3)

DF = \left(\frac{S}{5.8}\right) \left(\frac{13}{N_C}\right)^{0.3} \left(\frac{1}{L}\right)^{0.25}

(4)

DF = \text{Distribution Factor}

S = \text{Girder Spacing (ft)}

L = \text{Length of beam span (ft)}

N_C = \text{Number of Cells}

These equations were valid only if the bridge met certain criteria. These criteria included that the bridge span be between 18.29 m and 73.17 m (60 and 240 ft), have girder spacing of 2.13 m and 3.96 m (7 and 13 ft), and have more than 3 cells (AASHTO 2007). Because the Lambert Road Bridge met all these requirements, Equations 3 and 4 were used to calculate the distribution factors.

Because the Lambert Road Bridge had 8 degrees of skew, an additional factor had to be applied to the distribution factor. This skew factor was defined in the AASHTO LRFD Specifications in Table 4.6.2.2.2e-1 (AASHTO 2007) as:

\[ SF = 1.05 - 0.25 \times \tan(\theta) \leq 1 \]

(5)

SF = \text{Skew factor}

\( \theta \) = \text{Degree of skew (0°-60°)}

Equations 3 and 4 were for the calculation of the distribution factors for an interior girder. The distribution factor for an exterior girder was calculated similarly except that it was multiplied by an additional factor, g. This factor was provided in Table
4.6.2.2.2d-1 in the AASHTO LRFD Specifications (AASHTO 2007) and is reproduced here as Equation 6.

\[ g = \frac{W_e}{14} \]  

(6)

\( g \) = exterior distribution girder factor

\( W_e \) = the total overhang plus half the girder spacing

Because the web of the exterior girder was sloped and not perpendicular like the interior girders (see Figure 2), the girder spacing varied between the exterior girder and the adjacent girder. To calculate \( W_e \), the girder spacing between the exterior and interior was used at the mid height of the cross section.

The distribution factors calculated from the finite element model were determined by first finding the longitudinal location at which the moment for a simply supported bridge girder would be maximized from the loading of an AASHTO HS20-44 truck. This truck is shown in Figure 54 with its applied loads. The first axle had a loading of 35 kN (8 kips) and the back two axles had loadings of 145 kN (32 kips) each. The axle spacing used for this analysis was 4.27 m (14 ft) for both axial spacings, even though the back axial spacing can vary between 4.27 m and 9.15 m (14 and 30 ft). The transverse wheel spacing for the truck was 1.83 m (6 ft) from center to center of the wheel.

Figure 54 AASHTO HS20-44 truck.
The moment distribution factors for the bridge were obtained for the exterior and interior girders for the 1 lane, 2 lanes, and 3 lanes load cases. In each load case, an HS20-44 truck was placed in the corresponding lane(s). According to the AASHTO LRFD Specifications, the truck must be kept in its respective 3.66 m (12 ft) lane and cannot get closer than 0.61 m (2 ft) to the edge of the lane. The AASHTO LRFD Specifications defined an interior girder as the web and the associated half flanges between a web under consideration and the adjacent web. An exterior girder was defined as an exterior web with the half flanges between the adjacent girder and the beam overhang (AASHTO, 2007). For the case of a single loaded lane, the truck was placed transversely throughout the maximum moment location until the moment in each of the five bridge girders was maximized. The largest moments for the three interior girders and for the two exterior girders were considered to be the governing cases for the interior and exterior single lane case. These moments were then divided by the maximum moment produced from the HS20-44 truck loading on a simple supported beam and then multiplied by a multi-presence factor to determine the distribution factor. The multiple presence factor takes into account the probability that each lane(s) will be simultaneously loaded at any given time. The multiple presence factors were only multiplied to the calculated distribution factors from the finite element model since the equations listed in the AASHTO LRFD Specification already take into consideration the effects of multiple lanes being loaded simultaneously. The factors for single, two, and the three lane loaded case are listed in Table 6.

For the two lane analysis, two trucks were placed in lanes on the finite element model. The process of positioning the trucks transversely was repeated as in the case of a
single truck until the moment in each girder was maximized. The process was similarly repeated for the case of three lanes by using three trucks.

Two different analyses were performed using the finite element model. The first analysis included the parapets as a composite member and the second analysis excluded them from the finite element model. The results are listed in Table 7 where FEM(P) represents the case in which the parapets were included and FEM(NP) represents the case where the parapets were excluded from the analysis. Table 7 also lists the calculated distribution factors from the AASHTO LRFD Specification.

<table>
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<tr>
<th>Girder Case</th>
<th>AASHTO LRFD</th>
<th>FEM(P)</th>
<th>% Difference</th>
<th>FEM(NP)</th>
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*Note-% Difference= [(AASHTO/FEM-1)*100]
In comparing the distribution factor listed in Table 7, the AASHTO LRFD Specifications were always conservative for the interior girder on average by 35%. When the parapets were excluded from the finite element model, the AASHTO LRFD Specifications over estimated the distribution factors by 20-37% for an interior girder for all three load cases.

For the exterior girder, the results were different. When the parapets were included in the finite element model, the model was found to be unconservative. However, the differences in the exterior girder distribution factors from the finite element model and AASHTO LRFD Specifications were quite small. For the single and two lane cases the differences between the finite element and AASHTO LRFD Specifications distributions were 2%. For the case of three lanes the finite element distribution factor was conservative by 9%. Excluding the parapets from the finite element model, the AASHTO LRFD Specifications were once again found to be conservative for an exterior girder. In each of the three lane cases the AASHTO LRFD Specifications were 16-30% conservative. Overall, the AASHTO LRFD Specifications were overly conservative. In each case it was always the interior girder controlling and overall the three lane case was the controlling lane factor.

There were several reasons that the distribution factors from the AASHTO LRFD Specifications were conservative to those from the finite element model. One reason that the AASHTO LRFD Specifications were overly conservative compared to the finite element model was the fixity of the support and pier. From the finite element model, it was observed that large negative moments existed at the support and near the pier. These
partially fixed boundary conditions resulted in a 25% reduction in the moment compared
to the moment produced from a simply supported configuration.

Another reason why the distribution factors from the AASHTO LRFD
Specifications were typically conservative to those obtained from the finite element
model was the procedure in calculating the distribution factors from the finite element
model. When calculating the distribution factors from the finite element model, the
maximum moment produced by the loading of the HS20-44 truck was found. This was
the moment that the individual interior and exterior girders were divided into in
calculating the distribution factor. This moment was based on a simply supported
condition rather than the actual support conditions (partially fixed) of the bridge.
Because of this procedure in determining the distribution factors, a lower distribution
factor was expected from the finite element than that obtained from the AASHTO LRFD
Specifications.

The AASHTO LRFD Specifications also stated that a type “d” cast-in-place box
girder bridges were typically conservative because the way the live load distribution
factors were derived. Rather than use an algorithm to find the peak of an influence
surface in determining the factors, the factors were found by first placing the vehicle
longitudinally and then transversely using an I-Section of the box. After which the
present factor for the interior beams were multiplied by the amount of girders in the
bridge (AASHTO, 2007).

In the AASHTO LRFD Specifications it stated that a bridge having intermediate
diaphragms will have a lower distribution factors than using Equations 3 and 4 because
the diaphragms stiffen the bridge (AASHTO, 2007). To determine the effects of the
diaphragms in the finite element model the diaphragms were removed. The results showed that the diaphragms did reduce the distribution factor by only 1 to 2%. This reduction was considered quite small and it was concluded that the intermediate diaphragms didn’t have a major factor in affecting the live load distribution factors.

Parametric study of distribution factors

To better understand the distribution factors obtained from the finite element model, a study of different variables affecting the distribution factors was completed. From the AASHTO LRFD Specifications (Equations 3 through 6) it can been seen that the distribution factors for type “d” box girder bridges were affected by individual span length, number of cells, spacing between girders, the overhang distance, and skew. To show how these bridge characteristics and others affect the distribution factors, different finite element models were developed to demonstrate the effects of these parameters.

The 7 different parameters investigated for this study include the span length, beam spacing, overhang distance, skew, effect of parapets, deck thickness, and continuity. The effects of the parapets could be examined in each case but was only examined in the span length case for ease. It was expected that similar trends would occur if the parapets were investigated in each individual case study so only one case study was needed. The distribution factors for each case were obtained in the same method as mentioned in the section titled, Comparison of Distribution Factors. Also, for this study only 1 and 2 lane cases were considered.

For each of these parameters, a standard finite element model was developed unless otherwise noted. The standard finite element model consisted of using solid elements in 0.3 m (1 ft) transverse and longitudinal increments. The bridge was a single
span bridge which was simply supported. The bridge had 4 cells with a girder spacing of 2.1 m (7 ft). There was a 0.91 m (3 ft) foot overhang and 0.3 m (1 ft) wide parapets. From the bottom flange to the top of the deck the height of the bridge was 1.7 m (5.5 ft). The deck thickness was 203 mm (8 in.), the bottom flange was 152 mm (6 in.), and the girders were 0.3 m (1 ft) thick. The slope of the exterior girders were 2 to 1. The bridge was 42.7 m (140 ft) in length and the concrete used had a strength of 27.8 MPa (4,000 psi). Unlike the finite element model for the Lambert Bridge, the finite element models for the parametric study did not use any tendons to model post-tensioning strands.

For each parameter, the distribution factors from the AASHTO LRFD Specifications and finite element model for the interior and exterior girders were calculated for both the 1 and 2 lane cases. It should be noted that the results from this study replicated the support conditions that were used to derive the distribution factors for the AASHTO LRFD Specifications. For the finite element models, it was assumed that the boundary conditions were simply supported when in reality there would be some type of fixity at the supports. Also, the finite element models also included the stiffening affects of the parapets. It is important to remember these modeling options when evaluating the distribution factors because of how they affect the magnitude of the distribution factors. It is important to recognize the trends from the factors rather than the actual magnitude because the values reflect a non-calibrated finite element model that was simply supported. Also, with each parameter a distribution factor ratio will be presented and discussed. This ratio was defined as the AASHTO Specification distribution factor divided by the distribution factor obtained from the finite element model. This ratio can be used to recognize how well the AASHTO LRFD Specifications
relates to the finite element model distribution factor. This means that any value over 1 indicates the AASHTO LRFD Specifications was conservative and a value less than one indicates the finite element distribution factor was conservative.

The first variable examined was the length of the span. This length varies depending on if the distribution factors being obtained were at a location of negative or positive moment. Since the finite element model was simply supported only a positive moment was produced. Therefore the span was defined as the length of the span for which moment was being calculated. The different lengths of the single span bridge were 18.3 m, 30.5 m, 42.7 m, 54.9 m, 67.1 m, and 73.2 m (60 ft, 100 ft, 140 ft, 180 ft, 220 ft, and 240 ft). These lengths were chosen because they encompassed the entire span length range as defined in the AASHTO LRFD Specifications (AASHTO, 2007). The AASHTO LRFD Specifications allowed Equations 3 and 4 for type “d” box girder bridges to be valid between 18.3 m through 73.2 m. This study also included the effects of parapets in the analysis. The distribution factors for these two variables are shown in Figure 55 and Figure 56. These figures show the distribution factors for the interior and exterior girders for the AASHTO LRFD Specifications and finite element models.

The distribution factors for the finite element models from both lane loading cases follow a similar trend, the longer the length of span the lower the distribution factor. This means that for larger moments the vehicle load was distributed more laterally among the girders than for smaller moments. By excluding the parapets in the finite element model, the distribution factor for the exterior beams were decreased and the interior distribution factors were increased. For all the finite element cases the length had little affect after 45 m. Most of the change in the distribution factor for the length came at the shorter spans.
of less than 45 m (150 ft). The governing case for the distribution factor is the 2 lane interior beam. Figure 57 and Figure 58 show a comparison of the distribution factor ratios for the 1 and 2 lane cases for the investigated lengths.

![Figure 55 Distribution factors for the variable length/parapet (1 lane).](image1)

![Figure 56 Distribution factors for the variable length/parapet (2 lanes).](image2)
Figure 57 Distribution factor ratios for the variable length/parapet (1 lane).

Figure 58 Distribution factor ratios for the variable length/parapet (2 lanes).
As can be seen in Figure 57 and Figure 58, the AASHTO LRFD Specifications were conservative for interior girders for both the 1-lane and 2-lane loading conditions. It can also be seen that excluding the parapets in the finite element model the exterior girder was more conservative while the interior was less conservative than including the parapets in the finite element model. Another trend that can be seen is that the AASHTO LRFD Specifications increases its conservatism until about 30 m (100 ft). From this value until 74 m (240 ft) the AASHTO LRFD Specifications becomes progressively less conservative.

The next variable considered in this analysis was beam spacing. The AASHTO LRFD Specifications based equations were applicable for a girder spacing between 2.1 m and 3.9 m (7 and 13 ft). In this analysis, the standard model was used except the beam spacing was adjusted to values of 2.1 m, 2.7 m, 3.3 m, and 3.9 m (7, 9, 11, and 13 ft). These values were used because the AASHTO LRFD Specifications dictate the applicable range of girder spacing is between 2.1 m and 3.9 m (7 and 13 ft) to be able to use the Equations 3 and 4 (AASHTO, 2007). Figure 59 shows the changes in distribution factors for the girder spacing parameter.

As expected, increasing the girder spacing increased the distribution factor. This was expected since the girders were further from each other so a girder would not distribute the load to the adjacent girders. It can be seen that the distribution factors from the finite element model increased by about 9% for every 0.6 m (2 ft). For this variable, the interior two-lane case was the controlling distribution factor.
Figure 59 Distribution factors for the variable girder spacing.

Figure 60 Distribution factor ratios for the variable girder spacing.
In Figure 60 the distribution factor ratios for the parameter girder spacing can be seen. In both lane cases, the interior beam was found always to be conservative while the exterior girder had varied results. For the 1-lane case, the finite element model was conservative for exterior girders until the upper limit of the spacing requirements was reached. For the 2-lane case, girder spacing of anything just over 3 m (~10 ft) produced conservative results for the AASHTO LRFD Specifications. In examining the trends of the distribution factors, it can be seen that the AASHTO LRFD Specifications in all cases became increasingly conservative with respect to increased girder spacing in comparison to the distribution factors from the finite element model. This was especially true for the 2-lane case. The 2-lane case had a much larger magnitude of slope than the 1 lane case which means for higher girder spacing the AASHTO LRFD Specifications became even more conservative for 2-lane cases.

The next variable that was considered in the analysis was the thickness of the deck slab. The AASHTO LRFD Specifications does not take this variable into consideration for a box girder bridge. However it could be expected that the deck thickness could contribute to the distribution of vehicle loads since other types of bridges consider this variable when computing distribution factors. The standard model was used with the deck thickness being varied with values of 203 mm, 254 mm, 305 mm, and 356 mm (8 in., 10 in., 12 in., and 14 in.). In Figure 61 and Figure 62 the distribution factors and ratios can be seen for the deck thickness variable.
Figure 61 Distribution factors for the variable deck thickness.

Figure 62 Distribution factor ratios for the variable deck thickness.
It can been seen that a deck thickness of 203 mm to 356 mm (8 to 14 in.) plays a minimal role in the distribution of truck loads. This would explain why the AASHTO LRFD Specifications does not take deck thickness into consideration when calculating the distribution factors for box girder bridges.

The next variable that was examined was the $W_e$ factor. This factor was used to determine the distribution factor for exterior girders. This variable takes into consideration the beam spacing and total overhang. This variable was defined as Equation 6. Because $W_e$ takes into account two variables, spacing and overhang, there could be two ways to model this parameter. However, because beam spacing has all ready been examined only the overhang will be examined. The AASHTO LRFD Specifications states that the value of $W_e$ cannot exceed the beam spacing. In order that a larger range of $W_e$ could be studied, the standard model spacing was changed to 2.7 m (9 ft) from 2.1 m (7 ft). The results for this analysis are presented in Figure 63.

![Figure 63 Distribution factors for the variable overhang.](external-url)
As expected the overhang had little effect on the interior girder for the finite element model. Although the distribution factor decreased slightly in both lane cases for the interior girder of the finite element model, the decrease was quite small. The distribution factors for an exterior girder increased by about 0.01 for each 0.3 m (1 ft) of overhang in both lane cases for the finite element model. The AASHTO LRFD Specifications for an exterior girder was the only variable that seems to be largely affected by the overhang. The interior beam for the 2-lane case was the controlling distribution factor.

The ratios show that the interior girders in both cases were conservative with respect to overhang and were not largely affect by the length of the overhang. The exterior girder ratios were largely affected by the overhang. The AASHTO LRFD Specifications dictate (Equation 6) that designing for a larger $W_e$, results in a higher

![Figure 64 Distribution factor ratios for the variable deck overhang.](image-url)
distribution factor since it is expected that the exterior girder will take on more of the load. However, from the finite element models, it appeared that the exterior girder was not affected by the overhang as much as the AASHTO LRFD Specifications predicted it would. This was evident from the Figure 64 which shows that the ratio increased in both lanes cases with respect to the length of the overhang.

The next variable that was considered in the analysis was skew. The AASHTO LRFD Specifications dictates that Equation 5 should be used to adjust for skewed bridges. The skew values that were considered in the analysis were 0, 15, 30, 45, and 60 degrees since 0 to 60 degrees was the range the AASHTO LRFD Specifications encompassed. The standard model was used for this analysis but with each model having a different skew. Figure 65 shows the distribution factors for the skewed bridges.

The results show that the greater skew the lower the distribution factor. The reason for this was that some of the longitudinal moment was transferred into a torsional

![Figure 65 Distribution factors for the variable skew.](image)
moment because of the skew. The skew affected the exterior girder more than the interior girder for the finite element model since the distribution factor for the exterior girder was lowered more than the interior girder was lowered. For this parameter the 2-lane interior girder is the controlling factor for the distribution factor.

Figure 66 shows the ratios of the distribution factors. From this figure it was difficult to see any definite trends among the distribution factors among the exterior girders. The 1-lane case the ratio is held steady while the 2-lane case increases with larger skews. For the exterior girder a trend can be seen. Even though for the two lanes case the AASHTO LRFD Specifications and finite element model show that the distribution factor decreased with an increase in skew, the ratio did not decrease during the entire skew range. From about 15 degrees the ratio decreased but anything before this range the ratio increased. The finite element model was conservative for both lane cases for the exterior girders. The AASHTO LRFD Specifications were mostly conservative for the interior girder in the 1-lane analysis. For the 2-lane analysis, the AASHTO LRFD Specifications was conservative from about 0 to 30 degrees and unconservative from about 30 to 60 degrees.

The last variable examined was continuity. The standard finite element model was used but had two spans with a continuous midspan support. The spans that were examined were the same as in the length variable which were 18.3 m, 30.5 m, 42.7 m, 54.9 m, 67.1 m, and 73.2 m (60 ft, 100 ft, 140 ft, 180 ft, 220 ft, and 240 ft). The AASHTO LRFD Specifications did not consider this variable in the equations for calculating the distribution factors.
Figure 66 Distribution factor ratios for the variable skew.

Figure 67 Distribution factor for the variable continuity.
Comparing Figure 67 to Figure 51 it can be seen that the having a continuous structure did not significantly affect the distribution factor for an interior girder. The distribution factor for an interior girder for the continuous case was typically within 1% or 2% of that of the length variable for one and two loaded lane cases. The continuity of the structure did affect the exterior girder distribution factor. The exterior girder distribution factor was on average about 15% higher than those from the standard length scenario. The interior girder for two lanes is the controlling distribution factor for this variable. In Figure 68 the distribution ratios for continuity can be seen.

From Figure 68 it can be seen that the interior girder distribution factor obtained from the finite element model were lower than those from the AASHTO LRFD Specifications. It can also be seen for larger spans that the AASHTO LRFD Specifications distribution factors were nearly the same value as those obtained from the finite element model. The exterior girder distribution factor for the AASHTO LRFD Specifications
Specifications was also unconservative. These values were approximately half the value of the distribution factors obtained from the finite element model.

After evaluating the different parameters that affect the lateral distribution of truck loads to girders, a better understanding of the behavior of box girder bridges was gained. From this analysis the 2-lane interior girder was always the controlling case for the distribution factor for each parameter. As mentioned before, the overall trends of the distribution factor was more important than the actual magnitudes of the factors for the finite element model. It should be noted that even though in this study it was mentioned whether the AASHTO LRFD Specifications or finite element model was conservative, that could change if using a calibrated finite element model.

From the above study it was observed that the procedure for calculating exterior girder distribution factors as outlined in the AASHTO LRFD Specifications was unconservative. The AASHTO LRFD Specifications did not predict the values of the exterior distribution factors as can be seen in Figure 69. In this figure the exterior distribution factors from the parametric study (only span length, girder spacing, beam overhang, and skew were plotted) for the AASHTO LRFD Specifications and finite element model were plotted against each other. As can be seen in Figure 69, the linear relationship between the two sets of distribution factors was 0.74 with a coefficient of correlation of 0.53. With these low values it was evident that there was little correlation between the distribution factors from the AASHTO LRFD Specifications and the factors obtained from the finite element model and that the AASHTO LRFD Specifications produced unconservative distribution factors for exterior girders.
It is proposed that a better equation be used to calculate the distribution factor for exterior girders for box girder bridges. The proposed equation was obtained empirically and is shown in Equation 7.

\[
DF_{\text{ext}} = \left[ 0.9 \times DF_{\text{int}} - \frac{2}{W} \right] \times \left( \frac{W}{S} \right) \tag{7}
\]

\(DF_{\text{ext}}\) = Proposed distribution factor for exterior girder

\(DF_{\text{int}}\) = Distribution factor for interior girder based on AASHTO LRFD Specifications

\(W\) = The total deck overhang plus one half of the girder spacing (ft)

\(S\) = Girder spacing (ft)

By using Equation 7 rather than Equation 6, the exterior girder distribution factors from using this equation correlate more to the finite element model as compared to the factors from AASHTO LRFD Specifications. In Figure 70 the distribution factors for the finite element model and proposed exterior girder equation were plotted against each other. From this figure it can be seen that a linear relationship of 1.08 exists between the
two sets of data with a coefficient of correlation of 0.84. Although Equation 7 is more complex than that from the AASHTO LRFD Specifications, it offers a significant improvement in determining the exterior girder distribution factors. It should also be noted that the implementation of Equation 7 resulted in conservative distribution factors whereas the current AASHTO LRFD Specifications resulted in unconservative results.

**Load rating**

There are two different load ratings that are used to determine the load capacity of a bridge. These are the inventory and operating ratings. The inventory rating is a factor that specifies the live load that can safely exist on a structure for an indefinite period of time. The operating rating is a factor that corresponds to the maximum permissible live load that the structure may be subjected to. These factors are multiplied by the bending moment caused by a HS20-44 truck to obtain the maximum vehicle load that the bridge can safely carry. The load rating relationship is defined in Equation 8.
RF = Bridge load rating (operating or inventory)

Rn= Nominal flexure capacity

γ_d= Dead load factor (1.3)

γ_l= Live load factor (1.3 for operating, 2.17 for inventory)

D= Nominal dead load effect (composite and non-composite dead load)

L= Nominal live load effect (caused by a HS20-44 truck)

I= Live load impact factor (15.24/(L+38))

Using Equation 8, the inventory and operating ratings for the Lambert Bridge were obtained for an interior beam and are listed in Table 8. In this table the load ratings are listed in which the distribution factors obtained from the finite element model and AASHTO LRFD Specifications were used. The controlling distribution factor used for the finite element model was 0.51 which was for the interior beam and 3 lanes case. The controlling distribution factor obtained from the AASHTO LRFD Specifications was 0.66 which was for the multi-lane case. It was evident that the live load distribution factors obtained from the finite element model substantially increased the load ratings by 59% compared to those obtained using the AASHTO LRFD Specifications. The AASHTO LRFD Specifications were overly conservative in estimating the load ratings of the bridge.

<table>
<thead>
<tr>
<th>Load ratings</th>
<th>Inventory</th>
<th>Operating</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEM</td>
<td>2.56</td>
<td>4.27</td>
</tr>
<tr>
<td>AASHTO</td>
<td>1.61</td>
<td>2.69</td>
</tr>
</tbody>
</table>
SUMMARY AND CONCLUSIONS

Summary

As part of the Long Term Bridge Performance (LTBP) Program, the Lambert Road Bridge near Sacramento, CA was subjected to live load and dynamic testing. This periodic testing was part of the LTBP program to quantify the behavior of a bridge and how it changes over time with respect to weather, vehicle loads, corrosion, and fatigue.

The testing of the Lambert Road Bridge consisted of live load and dynamic testing. The live load test included driving a 32 ton truck and a 37 ton truck over the bridge which was instrumented with strain, tilt, and displacement sensors placed strategically at various locations on the superstructure. The dynamic testing included instrumenting the bridge with velocity transducers and then using an electromagnetic shaker to induce a sinusoidal force to the bridge so the modal shapes and frequencies of the bridge could be experimentally determined. The data collected from both tests was used to calibrate a finite element model so that its behavior matched the actual bridge behavior as recorded from the tests. Many studies have been conducted in which finite element models were validated through live load tests and then used to determine distribution factors of a bridge. However, little research had been done on box girder bridges in which finite element models are validated by field tests. Using this calibrated finite element model, the analytical moment distribution factors were found for the exterior and interior girders for the all possible loaded lane scenarios. Using the controlling distribution factor from the finite element model, the operating and inventory ratings of the bridge were calculated and compared to those using the approximate method described in the AASHTO LRFD Specifications. It was found that while using
the AASHTO LRFD Specifications can provide a quick estimate of the distribution factors it resulted in highly conservative load ratings

Conclusions

The distribution factors based on the analyses from the finite element model were compared to those obtained using the simplified procedures presented in the AASHTO LRFD Specifications. Several conclusions were made.

1. The AASHTO LRFD Specifications were slightly unconservative for the exterior girders. The exterior girder distribution factors using calculated AASHTO LRFD Specifications were found to be between 2% to 9% unconservative compared to those obtained using the finite element model.

2. The calculated interior girder distribution factors from the AASHTO LRFD Specifications were overly conservative in comparison to the finite element factors conservative by 29% to 46%.

3. It was determined that the controlling distribution factor for both the finite element model and AASHTO LRFD Specifications was based on the three loaded lane interior girder which had values of 0.51 and 0.66, respectively. The finite element based distribution factors were lower because of partially fixed supports and stiffening effects of the parapets. The partially fixed end conditions resulted in a 25% moment reduction in comparison to the moment produced from an HS20-44 truck on a pin and roller bridge.

4. The load ratings from the finite element model were 2.56 and 4.27 as compared to those from using the AASHTO distribution factors which were 1.61 and 2.69. The
finite element model had load ratings which were 59% higher than those from the AASHTO LRFD Specifications.

5. An equation for calculating exterior distribution factors was determined and found to estimate the distribution factors better than the AASHTO LRFD Specifications equation. This equation was found to be unconservative while the current equation is slightly conservative.

**Recommendations for future work**

In this study, the moment distribution factors and load ratings for an in service cast-in-place prestressed box girder bridge as well as the distribution factors for 40 finite element models were calculated. In each of these cases it was found that typically the distribution factors were lower than the AASHTO LRFD Specifications for interior girders and higher for exterior girders.

It is proposed that a better equation be determined to calculate the interior distribution factors for box girder bridges because of the highly conservative interior girder distribution factors obtained using the AASHTO LRFD Specifications. This equation can be determined through the testing and development of finite element models of box girder bridges. The tests on these bridges needs to vary the different variables affecting the distribution factors such as girder spacing, span length, skew, and also the effects of multiple spans. With this testing, the proposed equation for exterior distribution factors (equation 8) can also be validated.

There are also several recommendations for the testing of the bridges. Although the dynamic test focused on determining the fixity of the supports of the finite element model, the test was time consuming and can be cumbersome. It is recommend that rather
than using dynamic test data to determine the boundary conditions that strain transducers and tilt sensors be placed near the abutments to provide data for accurately modeling the supports. It is also recommended that displacement sensors be used during the live load test. The displacement data provided valuable information for the calibration of the finite element model and was easier to use than strain data. Also, the heaviest truck should be used for the live load test since working with small displacement and stain data can be difficult.
REFERENCES


