The Live Load Response of the Open Web Steel Joist Black Slough Bridge

Piero Caputo
Utah State University

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LIVE LOAD RESPONSE OF THE BLACK SLOUGH RIVER OPEN WEB STEEL JOIST BRIDGE

By

Piero Caputo

A thesis submitted in partial fulfillment

of the requirements for the degree

of

MASTER OF SCIENCE

In

Structural Engineering

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Logan, Utah

2018
ABSTRACT

The Live Load Response of the Open Web Steel Joist Black Slough Bridge

by

Piero Caputo, Master of Science

Utah State University, 2018

Major Professor: Dr. Marc Maguire
Department: Civil and Environmental Engineering

The bridge being studied spans the Black Slough River in the Nucor/Vulcraft facilities near Brigham City, Utah. The bridge is composed of seven heavyweight open web steel joists supporting a concrete deck-slab element and sitting in two concrete cantilever walls serving as bearings. A full-scale live-load test was performed using forklift vehicles at four locations on the deck. During the test, axial strains, deflections, and rotations were measured.

In their provisions, the American Association of State Highway and Transportation Officials (AASHTO) included a simplified number of equations and solutions used to calculate GDFs for standardized decks supported by beam systems. Due to the nonexistence of records for bridges with open web steel joists as deck supporting systems, a number of (AASHTO) GDFs were analyzed and compared to live-load test results to determine which system best mimics this unique bridge live load distribution. Three analysis approaches were also performed to compare their results to the experimental and AASHTO GDFs. In addition, a finite element model (FEM) for this bridge was made by a team of researchers from Kansas University in conjunction with this study. This model was also included in the final methodology comparisons.
The AASHTO calculated GDFs were significantly larger than the measured; on the other hand, the other methods analyzed provided a more accurate prediction of each beam girder distribution factor, following the recorded GDFs trend. The best method was chosen and further theoretical calibration suggestions were made for prediction of the behavior of this bridge using FEA.
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In their provisions, The American Association of State Highway and Transportation Officials (AASHTO) included a simplified number of equations and solutions for the calculations of standardized decks supported by beam system distribution factors. Due to the nonexistence of records for bridges with open web steel joists as deck supporting systems, a number of (AASHTO) distribution factors were analyzed and compared to the live-load test results to determine which system best mimics this unique bridge live load distribution. In addition, a finite element model (FEM) for this bridge was made by a team of researchers from Kansas University in conjunction with this study. This model was also included in the final methodologies comparisons.
The AASHTO calculated GDFs were significantly larger than the measured; on the other hand, the other methods analyzed provided a more accurate prediction of each beam girder distribution factor, following the recorded GDFs trend. The best method was chosen and further theoretical calibration suggestions were made for prediction of the behavior of this bridge using FEA.
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Piero Caputo
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CHAPTER 1 INTRODUCTION

Open Web Steel Joists (OWSJ) are widely used in the pre-engineered metal building (PEMB) industry mainly as roof and floor support systems. Their capacity to sustain loads with long span and lightweight configurations gives this system a great advantage over conventional wide flange steel beams and girders in the industry. As technology advances, many solutions are being proposed to improve the economy, constructability, and performance of bridges. Precast concrete bridges are often used for short span bridges, which prompted the development of the Short Span Steel Bridge Alliance (SSSBA).

It is of great importance to the bridge engineering community that an innovative, effective, and yet more economical configuration be standardized for the simple design of a range of short span bridges. Across the United States of America, great effort has been made to develop innovative deck-supporting steel systems to replace traditional reinforced concrete and hot-rolled steel beams for a range of short span bridges. The SSSBA proposed a number of standardized solutions for a range of spans; their latest solutions include Press-Brake-Formed Tub Girders (PBFTs) and Folded Steel Plate Girder Systems (FSPGs). The (PBFT) girders studied by Michaelson and Barth (2017) were restricted to a maximum single-span of 18.29 m (60 ft.) by the SSSBA and consist of trapezoidal cold-formed galvanized structural steel plates supporting a high-strength precast concrete deck. The main advantages of a PBFT system are its low welding requirement, modular configuration, and simplicity of installation; the last can be observed in Figure 1-1. The second innovative system proposed by the SSSBA is the FSPG. Bridges composed of these types of elements have a span range of 6.10 m (20 ft.) to 18.00 m (59 ft.). Similar to PBFTs, this system uses trapezoidal cold-folded steel plate boxes as deck supporting elements. These systems have been standarized by CDR Bridge Systems. As seen in Figure 1-2, their ease of
errection, installation, and prefabrication makes this system a competitive solution within its span range.

![Figure 1-1](image1.png)

*Figure 1-1. Erection of entire bridge superstructure using PBFT, picture obtained from the (Short Span Steel Bridge Alliance, 2013).*

Surpassing the span range from the previously mentioned systems, the OWSJ system being studied could compete with traditional reinforced concrete or steel beams in a span domain where the previous solutions could be ineffective. In addition, OWSJs may be a safe and economic option, based on their performance in the building industry. Compared to the systems proposed by SSSBA, OWSJs will need more complex manufacturing, and due to their multiple components more difficult to inspect by the regulating agencies but as seen in this case of study, they are capable of

![Figure 1-2](image2.png)

*Figure 1-2. Erection of first FSPG in Pennsylvania, picture obtained from CDR Bridge Systems (2017).*
reaching longer spans. In this study, one of the first investigations into the performance of concrete decks over OWSJ bridge superstructures was performed. This document covers a full scale live load test followed by a live load behavior parametric analysis of an existing OWSJ bridge. The purpose of this document is to begin the investigation of OWSJ bridge behavior.

1.1 Open Web Steel Joists

Open web steel joists are flexural members that use principles of truss behavior to sustain loads over long spans; they are widely used as light-weight flooring and roofing systems in the building industry. These structural systems have been used for more than 90 years as a reliable and economical solution for long span, small load roof and floor structures. The Steel Joist Institute (SJI) is the regulating organization that has been enforcing correct engineering practices since five years after the creation of the first OWSJ (Steel Joist Institute, 2010).

According to SJI, OWSJs are very capable floor and roof supporting elements, reaching spans of 36.59 meters (120 ft.). Using a variety of standard steel shapes like steel pipes, angles, rods, and bars, multiple configurations can be achieved by the combination of different shapes and sections within every component of the system. Their lightweight configuration and large depths, exceeding 1.8 meters (6.0 ft.), combined with the ease of mass producing them makes them competitive in many commercial situations in the United States.
1.2 OWSJ Potential in Bridge Engineering

There is already a great variety of superstructure categories in bridge engineering. The American Association of State Highway and Transportation Officials (AASHTO) lists twelve different slab-on-beam configurations. OWSJ bridges will bring great advantages to the bridge industry over hot rolled W-Shaped bridges. These advantages are listed below:

- Fast production: Based on current OWSJ fabrication times, entire bridges could be delivered within a week.
- Customizable configurations: OWSJ fabrication in the US allows for easy customization.
- Less steel per ft.: Truss shape maximizes efficient use of steel.
• Lighter configuration: Lighter members mean lower construction costs and seismic forces.

• Easier and economical transportation: Their light weight means more members can be transported on fewer trucks.

• Lighter construction equipment required: This opens up construction bidding to more contractors, allowing for more competitive bids.

Considering these advantages and the fact that the OWSJ industry is widely spread across the country, this proposal could generate many savings to cash strapped municipalities in need of economical and safe replacement bridges. If further economic and structural research is done, this structural system could be refined, to an extent, to where the OWSJ system could cover the gap between slab bridges and traditional deck supported bay beam systems.

1.3 Challenges Facing OWSJ Bridges

The above benefits may make the OWSJ system an advantageous solution for short single span bridges. On the other hand, there are several concerns. Probably the most concerning issue is that OWSJ web and chord members are joined by welds in somewhat complex details, a concern for cyclically loaded structures (Lee, 1996). Other challenges stem from their complex internal geometries, making analysis and inspection difficult. Visual inspection is labor intensive, time-consuming, and biased by inspector skill and experience (Dorafshan & Maguire, 2018; Dorafshan, Maguire, Hoffer, & Coopmans, 2017). This issue could be addressed by using visual cameras and automated detection algorithms (Dorafshan, Coopmans, Thomas, & Maguire, 2018; Dorafshan, Thomas, & Maguire, In Press). The nonexistence of bridges composed of OWSJs as deck supporting members; therefore, the absent historical database of physical inspection, full-scale live load testing, and automated crack monitoring makes it hard to mitigate human bias and the
evaluation of redundant or unnecessary variables during design and evaluation of OWSJ bridge structures (Chang, Maguire, & Sun, 2017).

The following is a non-comprehensive list of challenges for OWSJ bridges:

- The limited knowledge of OWSJ behavior under cyclic vehicular loads
- The potential of fatigue induced cracks on welded joints
- No knowledge of long term behavior
- The possible complex inspection procedures, due to the great number of web and chord components. Composite action performance under cyclic loading
- Moment and shear distribution are poorly understood

Clearly, not all of these challenges can be addressed in this document. This document is focused on developing an understanding of OWSJ bridge system behavior, specifically moment distribution and live load behavior.

1.4 Black Slough Bridge

The OWSJ Bridge under investigation is located 3.66 km (2.29 mi) east of Brigham City, Utah, inside Vulcraft Group’s campus, and spans the Black Slough River. This bridge receives an estimated 20 fully loaded and 20 unloaded semi-trailer trucks every working day. The Black Slough Bridge is a single-span, two lane bridge with a cast-in-place concrete deck supported by seven OWSJs, as shown in Figure 1-4. The bridge was designed in 1997 using composite OWSJs, and the 1994 (SJI) Standard Specifications for Long Span and Deep Long Span Steel Joists LH- and DLH-Series and AASHTO Standard (1994) were used for most analyses.
Due to the lack of a design guide or behavioral knowledge for this type of bridge, the engineers intended to over design the bridge to ensure serviceability and long-term performance. During a field inspection prior to testing, the superstructure was shown to be in satisfactory condition, with relatively little maintenance performed on it in the last 21 years.

The OWSJs system is composed of full welded connection at all panel joints, beam spacing of only 1.22 meters (4 ft. 0 in.), cross bracing at 3.26 meters (10 ft. 8 1/2 in.).

1.5 Cost Analysis

The main purpose of using OWSJs in bridges is to reduce costs while maintaining similar performance. A cost comparison between OWSJs and rolled A992 W-shapes was performed considering only the deck supporting members as changing variables and the dead load only as the controlling load case. In addition, knowing that the OWSJ members were overdesigned, using of the Vulcraft Economical Joist Guide(2013), an optimized OWSJ section was obtained to be compared with the other systems. The SSSBA (2013) developed a simple design table, which was
used to obtain a final equivalent bridge cross-section with four W36x210 beams with 2.74 m (9 ft.) spacing. The OWSJ element costs were given by Vulcraft with a total cost of $1,800 per ton. Local manufacturers quoted the prize of the W-shape beams with a total cost of $2,100 per ton (Andalib, Caputo, Dorafshan, & Maguire, 2018).

As seen on Table 1-1 the OWSJs studied cost $68,445.00, representing a $2,295 increase from W-shaped four-beam bridge. The optimized OWSJ represented $45,675 of total savings from the W-shaped beam bridge. These costs do not include protective paint, galvanizing, erection, and maintenance; but, in addition to the speed of fabrication, this cost could represent an economic advantage over traditional steel beam systems for short span bridges (Andalib et al., 2018).

<table>
<thead>
<tr>
<th>Deck Supporting System</th>
<th>Bridge Span (ft.)</th>
<th>Cost/Member (US$/Ton)</th>
<th>Estimated Weight/Member (Ton/ft.)</th>
<th>Number of Members</th>
<th>Total Cost of Deck Supporting Members (US$)</th>
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</thead>
<tbody>
<tr>
<td>Black Slough Bridge OWSJ</td>
<td>75.00</td>
<td>1,800.00</td>
<td>0.072</td>
<td>7</td>
<td>68,445.00</td>
</tr>
<tr>
<td>SSSBA W-Shape</td>
<td></td>
<td>2,100.00</td>
<td>0.105</td>
<td>4</td>
<td>66,150.00</td>
</tr>
<tr>
<td>SJI Optimized OWSJ</td>
<td></td>
<td>1,800.00</td>
<td>0.022</td>
<td>7</td>
<td>20,475.00</td>
</tr>
</tbody>
</table>

**1.6 Report Organization**

This report will be divided by the following outline. First, the literature review and a background on previous research done on bridge live load tests will be presented. Then, a discussion of the live load instrumentation and testing done on the Black Slough Bridge describing the desired data to be collected. Next, an experimental analysis and data processing chapter will be presented. Followed by, a summarized presentation of all analysis results. Finally, a comparison of the all five analysis methods predictions.
CHAPTER 2 LITERATURE REVIEW

2.1 Steel Joist Analysis and Testing

2.1.1 Composite Steel Joist

SJI divides the design stages into non-composite and composite designs. The main difference between the two is that during non-composite design, the top chord must resist construction loads and be designed as a continuous element subjected to a combination of axial and bending stresses (Steel Joist Institute (SJI), 2010). In composite design, the top chord is considered a non-contributing element to bending strength and all bending effects must be sustained by the concrete deck, bottom chord, and web members. In all stages, the bottom chord must be designed as an axially loaded tension member, and the web members should sustain the bending shear effects with a minimum shear force of 25% of the end reactions or the value from equation Eq. 2-1 specified by SJI:

\[ V_{c_{min}} = \frac{(1.6w_L)L}{8} \]  

Eq. 2-1

Where \( w_L \) is the unfactored live load and \( L \) is the joist design length specified in Table 104.2-1 of the specifications as \( L = \text{Span} - 102m \ (33 \text{ ft.}) \). Extra consideration must be given to the interior joists’ vertical web member; since, it must sustain the gravity loads assigned to the member in addition to 2% of the composite bottom chord axial force (SJI, 2010).

2.1.2 Composite Steel Joist Load Tests

Previous literature performed numerous load tests to composite OWSJs. Below is a short summary of previous testing performed in this area. Previous research obtained valuable results from load testing of composite steel joists using load cells. Tide & Galambos (1970) made an early
attempt to study the behavior of composite open-web joists with a concrete slab. Their test was based on analyzing the failure modes of partially-composite elements. They observed two main failure modes: buckling of the top chord and yielding of the bottom chord. Furthermore, Robinson and Azmi (1978) tested four types of steel joists from three manufacturers. The joists spanned 15.25 m (50 ft) and were made of formed metal and a concrete slab. Their results showed that full composite action was achieved using puddle welds as shear connectors, and that failure is correlated to the degree of shear connection. Researchers like Yost, W. Dinehart, P. Gross, Pote, and Gargan (2004); and later Buckley, Dinehart, Green, Gross, and Yost (2007), found an additional failure mode for open web steel joists with crimped-end web members. Their conclusions stated that special attention must be given to a “critical panel zone.” This segment, under the uniform loading condition and an effect at the crimp transition zone, will cause buckling on the web panel near the support before midspan. Figure 2-1 shows the critical web section failure on one of the joist specimens tested (Buckley, Dinehart, Green, Gross, & Yost, 2007).
Further testing and calibration, like that previously mentioned, has been done by Lembeck (1965) following the first recorded composite design. A full-scale load test and the ultimate load analysis and design criteria were studied by Lauder and Esterling (1994) for partially composite steel joists. These and other research converged into the first specification for composite joists and trusses using LRFD (ASCE, 1996).

2.1.3 Effective Moment of Inertia Formulation for Steel Joists

Shear deformation due to flexibility of the web is one of the main reasons why the effective moment of inertia derivation is needed. Band and Murray (1996) derived a relatively simple method to calculate an OWSJ’s effective moment of inertia which was adopted by AISC Design Guide 11 (American Institute of Steel Construction, 2016) and the SJI (2010) manual. Furthermore, the final derivation of the effective inertia of an open web steel joist and joist-girder
was made by Band and Murray (1996), a refinement of Kitterman’s results (1994). This study used ten joists and joist-girders with span-to-depth ratios ranging from 2 to 24; thus, combined with the twenty-five full-scale samples tested, Kitterman (1994) formulated Equation 2-2 which is currently used by today’s standards.

\[
I_e = \frac{1}{\gamma\frac{I_{chords}}{I_{comp}} + \frac{1}{I_{comp}}} \quad \text{Eq. 2-2}
\]

Where \(I_e\) is the effective moment of inertia of an OWSJ accounting for shear deformations, \(I_{chords}\) is the OWSJ non-composite moment of inertia, \(\gamma\) is the web configuration factor, and \(I_{comp}\) is the OWSJ’s composite moment of inertia. The web configuration factor can be obtained using Equation 2-3.

\[
\gamma = \frac{1}{C_r} - 1 \quad \text{Eq. 2-4}
\]

For OWSJs with double angle web and members with a length to depth ratio less than or equal to 6, the formula to determine \(C_r\) is:

\[
C_r = 0.90 \left( 1 - e^{-0.28\frac{L}{D}} \right)^{2.8} \leq 0.9 \quad \text{Eq. 2-5}
\]

For OWSJs composed of continuous rounded rod web members and a length to depth ratio less or equal to 10, Equation 2-6 can be used:

\[
C_r = 0.721 + 0.00725 \left( \frac{L}{D} \right) \leq 0.9 \quad \text{Eq. 2-6}
\]
2.2 Live Load Test

Full scale live load testing allows bridge engineers to understand how these structures perform. Often, tests are carried out to verify the results of a method of analysis (S. Nowak & Tharmabala, 1989). Understanding how each component contributes to the global system helps with the calibration of code provisions.

Research has been done on live load responses of traditional bridge systems. For instance, Eom, Nowak, and Fellow (2001) and Laurendeau et al. (2014) tested the load distribution of steel girder and Pratt truss bridges and came up with reliable live load distribution factors from full scale live load tests. Because the Black Slough bridge was designed with steel joists as girders, which are “open web steel truss-type flexural members” (Yost, W.Dinehart, P.Gross, Pote, & Gargan, 2004) and are not found in the literature, a better understanding would be beneficial to bridge engineers.

2.2.1 Truck Load

Numerous factors are correlated to live load effects: span length, truck weight, axle loads, axle configuration, the position of the vehicle on the bridge, girder spacing, and stiffness of structural members (Nowak, 1994). Live load tests have different categories depending on the desired results. In previous research, S. Nowak and Tharmabala (1989) divided the different testing methods into behavior tests, proof tests, ultimate load tests, stress history tests, and diagnostic testing. Behavior tests are used to validate the results of some theoretical analyses using very low loads in comparison to ultimate loads. Proof tests are performed by applying a static load, which establishes the allowable load for the studied bridge. An ultimate load test helps determine the ultimate load carrying capacity of the bridge. A stress history test defines the distribution of stress
ranges in a bridge’s fatigue-prone areas. Finally, a diagnostic test establishes the causes of failure when analytical methods are unusable.

Most all bridge live load testing can be divided into static and dynamic categories, depending on how the load is applied (S. Nowak & Tharmabala, 1989). Also, some researchers referred to the vehicular load applied at slow speeds as pseudo-static (Laurendeau et al., 2014). Structural reliability, live load behavior, and long term serviciability are obtained from both tests (Collins, 2010; Laurendeau et al., 2014; S. Nowak & Tharmabala, 1989). Long-term deterioration and serviceability of bridges are a great concern throughout the nation. Live load testing of deteriorated bridges has prompted the development of a long-term performance rating useful for design and management (Torres, Zolgadri, Maguire, J. Barr, & W. Halling, 2016).

Truck positioning is essential to analyzing bridge behavior under all possible load scenarios; this is why multiple load paths are applied during live load testing (Liu, Bartlett, & Zhou, 2012). In addition, Barr, Eberhard, and Stanton (2001) found that in order to find reliable experimental girder distribution factors, multiple load paths must be applied to the bridge. These GDFs are further explained in the next section. Although AASHTO specifies the design vehicle load as one HS20 truck with 72 kips total weight or a design tandem with 50 kips total weight, good results can be observed when using vehicles with lighter loads (Barr et al., 2001).

2.3 Girder Distribution Factors

GDFs are a simple bridge characteristic that defines how the live load is transferred from the deck to each deck supporting member. Extensive research done on all traditional types of beam-slab superstructures produced simplified distribution factors given in the AASHTO LRFD Bridge Design Specifications, Section 4.6. These empirically based equations vary by deck and girder type and the number of lanes loaded. For interior steel girders supporting a concrete deck,
AASHTO Table 4.6.2.2.2b-1 uses Equation 2-7 for one design lane loaded, and Equation 2-8 for two or more design lanes loaded.

\[ g = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0L t_s^3} \right)^{0.1} \]  
Eq. 2-7

\[ g = 0.075 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0L t_s^3} \right)^{0.1} \]  
Eq. 2-8

Where \( g \) is the live load moment distribution factor, \( S \) is the spacing of the beam system in feet, \( L \) is the span length of the beam in feet, \( t_s \) is the depth of concrete deck in inches, and \( K_g \) is defined in Equation 2-9 below and given in AASHTO equation 4.6.2.2.1-1:

\[ K_g = n \left( I + Ae_d^2 \right) \]  
Eq. 2-9

Where \( n \) is the modular ratio between the deck and beam, and \( I \) is the non-composite beam moment of inertia in inches to the fourth, and \( A \) is the area given in square inches (American Association of State Highway and Transportation Officials, 2014).

On the other hand, for the exterior beam live load distribution factor calculations, the designer will have to select one of the following methods: the lever rule, a special analysis, or a tabulated approach. The lever rule method is only used when one lane is loaded (see Figure 2-2). The special analysis from C4.6.2.2.2d is used when one or more lanes are loaded. Lastly, the equation from Table 4.6.2.2.2d-1 is used when two or more lanes are loaded.
Figure 2-2. Lever rule model used to calculate "g" for exterior beams.

\[ R = \frac{N_L}{N_b} + \frac{X_{ext} \sum N_L e}{\sum N_b x^2} \]  \hspace{1cm} \text{Eq. 2-10}

\[ g = e g_{interior} \]  \hspace{1cm} \text{Eq. 2-11}

\[ e = 0.77 + \frac{d_e}{9.1} \]  \hspace{1cm} \text{Eq. 2-12}

Where \( R \) is the exterior beam reaction due to vehicle load, \( N_L \) is the total number of lanes loaded, \( X_{ext} \) refers to the distance from the global center of gravity of the beam system to the exterior beam, \( N_b \) is the number of beams in the system, and \( x \) is the distance from the global center of gravity of the beam system to each beam. The variable \( e \) is dually used in the AASHTO code. In Equation 2-4 it is the eccentricity of a design truck, and Equation 2-5 is the correction factor for distribution expressed in Equation 2-6 (American Association of State Highway and Transportation Officials, 2014).

Although widely used in the design of bridge structures, these GDFs are conservative and don’t replicate the real live-load behavior of a bridge superstructure (Barr et al., 2001; Collins,
2010; Torres et al., 2016). Furthermore, a number of factors that increases the strength over time like corrosion on the end bearings, non-structural elements contribution, among others can’t be considered using this simple approach (Akinci, Liu, & Bowman, 2013; Eom, Nowak, & Fellow, 2001; S. Nowak & Tharmabala, 1989).

2.4 Formulation of Experimental Distribution Factors

Historically, GDFs have been used to estimate the percentage of a truck’s load that will go to an interior or exterior girder. These values are the live load ratios applied to each girder when the vehicular load is crossing the bridge’s span (Reiff, Sanayei, & Vogel, 2016). Equation 2-7 shows the experimental approach used to obtain the experimental moment distribution factors (MDF). This derivation has been used for more than 30 years; an early model used a ratio of the static strain of the evaluated member to the summation of all system strains (Ghosn, Moses, & Gobieski, 1986). Later, Kim and Nowak (1997) used Eq. 2-7 derived by Stallings, Member, and Yoo (1993). This development considers the section modulus difference between the exterior and interior beams where it is common to encounter the stiffness contribution by barriers to the exterior beams (Kim & Nowak, 1997).

\[ g_n = \frac{M_n}{\sum N_b M} = \frac{E_n S_n \varepsilon_n}{\sum N_b E S \varepsilon} = \frac{S_n \varepsilon_n}{\sum N_b S \varepsilon} = \frac{\varepsilon_n \omega_n}{\sum N_b \varepsilon \omega} \quad \text{Eq. 2-13} \]

Where \( g_n \) is the MDF of the \( n^{th} \) beam, \( M_n \) is the moment due to bending of the \( n^{th} \) beam, \( M \) is the moment due to bending of the each beam, \( E_n \) is the modulus of elasticity of the \( n^{th} \) beam, \( E \) is the modulus of elasticity of each beam, \( S_n \) is the section modulus of the \( n^{th} \) beam, \( S_m \) is the section modulus of each beam, \( \varepsilon_n \) is the measured strain of the \( n^{th} \) beam, \( \varepsilon_m \) is the measured strains of each beams, and \( \omega_n \) is the section modulus ratio of the \( n^{th} \) beam, and \( \omega_m \) is the section modulus ratio of each beam, the latter also called the weight factor (Kim & Nowak, 1997). A great number
of pseudo-static live load test results have been evaluated with this approach, providing a reliable and simple methodology to compare the AASHTO code equations and methods to the real response of the tested bridges.

Similar to strain measurement, deflection measurements are helpful when observing the GDF. Kocsis and Asce (2004) used bridge deflections caused by linear loads, like sidewalks, curbs or barriers, and compared the experimental girder distribution factors from deflections and the AASHTO specifications. An alternative to Equation 2-7 which uses deflection results is showed in the following equation:

\[ g_n = \frac{M_n}{\sum_{N_b} M_m} = \frac{E I_n \delta_n}{\sum_{N_b} E I_m \delta_m} = \frac{I_n \delta_n}{\sum_{N_b} I_m \delta_m} = \frac{\delta_n \omega_n}{\sum_{N_b} \delta_m \omega_m} \]

Eq. 2-14

Where \( I_n \) and \( I_m \) are the nth and mth beams’ moment of inertia around the x axis, and \( \delta_n \) and \( \delta_m \) are the nth and mth beams’ recorded maximum deflections.

2.5 Spring Analogy Prediction of Distribution Factor

A simplified yet less conservative GDFs analysis technique proposed by Akinci et al. (2013) analyzes the 3-D structure as a 2-D. This method is based on the idealization of the bridge cross section as a system with rotational springs as the beams, these springs are connected by a flexible torsional bar acting like the bridge deck as shown in Figure 2-3. Unitless moments are applied at the location of the wheels; then, absorbed by the beam’s support elements. Akinci et al. (2013) concluded that the FEA results were more accurate than AASHTO results.
2.5.1 Rotational Spring Stiffness

The rotational spring is equivalent to the beam’s stiffness and is relative to the composite moment of inertia of each element. The author suggests using a spring stiffness of 1.0 for interior elements, then a ratio between the exterior and interior beams’ composite moment of inertia will adequately define the exterior beam spring stiffness. All spring stiffness will be relative to one beam element stiffness property; this can be applied to contemplate different spacing, health conditions, and parapet differences for the model.

2.5.2 Torsion Bar Stiffness

The torsion bar, according to the authors, is the most influencing factor for load distribution. Furthermore, in their research, the authors concluded that the most influencing characteristics within the bridges tested are the beam spacing and bridge span. The torsion bar stiffness was calibrated using a history of live load tests and is presented in Equation 2-15.
\[ K_T = \frac{3 \times \sqrt{\frac{L}{45.7}}}{S} \]  

Eq. 2-16

To determine \( K_T \) Akinci et al. (2013) performed a regression analysis and found that \( K_T \) is directly proportional to the square root of the normalized bridge span and inversely proportional to the beam spacing.

2.5.3 Distribution Factor Formulation

This simple analysis needs two final steps to estimate the final local beam bending moment and deflection. First, a single beam model with the entire superstructure moment of inertia has to be created in order to find the bending moment and deflection of this section. Then, after analyzing the torsional bar/rotational spring, the final reactions on the spring will be equivalent to each beam moment distribution factor and the rotations will simulate the beam’s deflection distribution factor.
CHAPTER 3 EXPERIMENTAL PROCEDURE

The measurement of strain, deflection, and end rotation is essential to obtain relevant information of bridge behavior under loading. Strain gauges have multiple applications depending on the bridge superstructure configuration. For a truss bridge, strain gauges measure axial effects on web and chord members. On the other hand, a beam-slab bridge test uses these gauges to measure bending effects on the beams. Thus, the following summary presents the desired data: bottom chord strains at midspan, one-quarter length, and at critical panel zone; web members’ strains at the critical panel zone and midspan; deflections at midspan; and rotations of the end bearings.

During the data processing and analysis, observations of bridge strains and end bearing rotations would be considered valuable data for further research on fatigue and long-term analysis. For the scope of this study, the deflections at midspan were used for the formulation of experimental distribution factors and will be the experimental results evaluated and compared to the analytical method proposed as parametric result sources.

3.1 Bridge Properties

The bridge superstructure consists of a concrete slab over a formed steel deck, with dimensions of 7.67 m (25 ft 2 in.) total width, 30.48 cm (12 in.) total depth, 26.67 cm (10.5 in.) of concrete slab, and 3.81 cm (1.5 in.) of deck-rib. The original design documents specified the concrete deck self weight of 22.78 kN/m³ (145 lb/ft³), and ultimate compression strength of 27.56 MPa (4.00 kip/in.²). The superstructure is composed of seven OWSJs spanning 22.85 m (74 ft 11.5 in.) and spaced at 1.22 m (4 ft.) as shown in Figure 3-1.
The cross bracing providing lateral torsional buckling restraint is spaced at 3.11 m (10 ft. 2.50 in.). Connections were made with 19.05 mm (0.75 in.) diameter steel bolts located on each brace junction at the main OWSJ chords. Full composite action was specified by the designer, and the shear connection between the concrete slab and the OWSJ system is provided by 19.05 mm (0.75 in.) shear studs spaced in the following pattern 24 at 152.4 mm (6 in.), on the first and last thirds of span, and 17 at 914.40 mm (36 in.) on the center third.

According to the Steel Joist Institute (2010), CJ-Series shall be open web, parallel chord, load-carrying steel members. The shear connections between the top chord and overlying concrete slab allow the steel joist and slab to act together as an integral unit.
Figure 3-2. Bridge cross bracing.

Chord members are composed of a double angle sections (nominal U.S. designation 2L8x4x3/4) with the following properties Grade 50 steel, a long legs vertical (AISC, 2011) configuration, 25.40 mm (1 in.) separation; and a single angle properties of 203.20 mm (8 in.) by 101.60 mm (4 in.), with 19.05 mm (0.75 in.) thickness. The designer did not specify the web sections and material; therefore, they were measured in the field. The web members are composed of angle sections with the following characteristics: diagonals with nominal U.S. designation L3.5 x 3.5 x 0.375, vertical members with nominal U.S. designation L3.5 x 3.5 x 0.25, and a panel-length of 914.4 mm (36 in.) from flange to flange (Figure 3-4 a). As shown on Figure 3-4 b, the end panels have a different diagonal section and panel length. The end panel web diagonal are U.S. designation L4 x 4 x 0.5 and a panel-length of 1365.25 mm (53.75 in.)
Figure 3-3. Bridge west section elevation.

Figure 3-4. Bridge sections details a) interior panel b) end panel.
3.2 Sensors

The bridge behavior under applied live load testing data collection was possible with the use of a Bridge Diagnostics Incorporated (BDI) wireless data acquisition system. On a slab-beam bridge, the most commonly measured responses during live load testing are girder deflections, girder and deck strains, ends rotations, and temperature readings (Collins, 2010). For truss bridge response, the following data must be recorded: end rotation; web, chord, stringer, stud, and deck strains; global deflections; and temperature readings (Laurendeau et al., 2014). The bridge testing outlined below used 21 strain gauges, 7 displacement gauges, and 4 tiltmeters, which read the bridge response at a rate of 19.991 Hz.

3.2.1 Strain Transducer

During the live-load test, a total of 21 BDI strain transducers were placed on several desired locations to measure the chords and web-members’ axial behavior during the different load cases. These sensors have been used in previous live load tests like the studies on segmental concrete bridges done by Maguire et al. (2015; 2012). These sensors were placed at member centroids to estimate the axial force in the member, see Figure 3-5. The ST350 model strain gage is a transducer designed to measure axial strain using a Wheatstone configuration containing four 350 Ω foil type gages hooked up to a four-wire configuration.
For the non-destructive load test, a temporary mounting system with two steel tabs specially made by BDI with imperial notation where used. The correct procedure for mounting and installation of these sensor components is described in the BDI ST350 Operation Manual.

After deciding and marking the desired location of each sensor, all sensor were attached to the bridge using two base tabs, see Figure 3-5. Typical BDI Strain Gage Installation. To install the base tabs to the sensor, a TAB JIG was used as a template to ensure a nominal length of 76.2 mm (3 in.) between them, see Figure 3-6. After the bridge member surface was cleaned from debris and dust, Loctite 410 Cyanoacrylate glue was applied to the tabs and the joist-member interface.
Then, a Loctite 7452 Tak Pak accelerator was sprayed onto the glue to optimize adhesion to the steel member (Figure 3-7).

![Loctite 410 Adhesive and Loctite 7452 Accelerant](image)

**Figure 3-7. From Right: Loctite 410 Adhesive and Loctite 7452 Accelerant.**

### 3.2.2 Deflection Gages (Twangers)

The deflections were measured using seven strain-based deflection transducers more colloquially known as twangers. Each sensor was composed of a cantilevered aluminum plate with its support placed between two aluminum plates and a bridge strain gauge attached. The base plates were attached to the member’s bottom chord flange using two 101.6 mm (0 ft.-4 in.) C-clamps, see Figure 3-8.

Prior to testing, an initial downward deflection was applied to the gauge cantilever plate. Using steel wire attached to an eye-bolt and hooked to the tip of the cantilever plate, the wire was then attached to 5 gallon buckets that were filed to the top with concrete and used as weights. As the truck loads moved along the bridge span, the base of the sensor was displaced with the bottom chord while the tip stayed at the same height. As deflection changes the strain in the plate is measured by the built-in strain gauge and related these readings to the joist deflections.
3.2.3 Tiltmeters

Rotations at the bridge ends are a crucial behavioral property that helps researchers determine the boundary conditions. As Eom et al. (2001) observed, corrosion of the bearings causes unexpected restraint to the rotation’s displacements. This is why measurements of the real support behavior of the bridge can give researchers a better understanding of and the tools for a more refined model.

Four single-axis BDI tiltmeters were attached to the bridge top chords as close to the bearings as possible. These sensors use a gravity-induced, enclosed fluid bubble mechanism to measure the level of rotation with respect to the apparatus’s horizontal resting position. All test results were measured in degrees and recorded. Two types of sensor accuracy were used: ±3° and ±0.5°. Three sensors had an accuracy of ±3° while the fourth had an accuracy of ±0.5°. The mounting system was similar to that of the strain gages, which consisted of steel tabs glued to the chord member (see Figure 3-9).
3.2.4 Data Acquisition System

Bridge response data collected by a BDI STS-Wifi Mobile Base Station and Nodes, both shown in Figure 3-10. This apparatus was connected to the computer though a coaxial pin connector. Using the BDI STS-2.0, a sample rate was assigned and all sensors were checked. Then, each test was named, performed, and saved in the computers.
3.3 Instrumentation Plan

The locations of the data recording gauges were chosen to obtain good spatial resolution while obtaining measurements from the most highly loaded regions in order to keep the signal-to-noise ratio high enough to obtain good readings. A total of 40 channels were available for this test. At midspan, 11.43 m (37 ft. 6 in.), seven displacement gauges were clamped to all OWSJ bottom chords, three strain gauges were attached at the vertical and diagonal web members of J1 and J4, see $\varepsilon_2$ label in Figure 3-11, and at members J1 through J5 were instrumented with one strain gauge at the bottom chord. Due to the placement of twangers at the bottom chords, the strain gauges were attached at the bottom chord vertical flange at 11.89 m (39 ft. 0 in.) from the west abutment.

The second most instrumented sections were the critical panel zones of members J1 and J4. Four strain gauges were attached to this panel zone: one at the web diagonal, one at the vertical web member, and two on the bottom chord at 18.26 m (59 ft. 11 in.) and 19.177 m (62 ft. 11 in.) from the west abutment. Members J1 and J4 were instrumented with strain gauges at quarter span, 5.7 m (18 ft. 9 in.) from the west abutment, on the bottom chords. Finally, at the eastern abutments,
tiltmeters were placed below the top flange of J1 and J4, and on the western abutment, J4 and J7 tilmeters were placed below the top flange.

Figure 3-11. Bridge Instrumentation Plan View.

3.4 Live Load Testing

The Black Slough River Bridge test was performed on October 30, 2017. Full-scale load tests are crucial to the process of refining the analysis standards and procedures to accurately predict the behavior of bridge structures. Many researchers have encountered important information about bridge behavior while performing live-load tests. For example, Torres et al. (2016) determined experimental double-tee GDFs and S. Nowak & Tharmabala (1989) evaluated bridge reliability using live load tests. This gives the proper background and backs up live load testing as a valuable tool for predicting the behavior of a new type of bridge.
3.4.1 Loading

Bridge response to live load was measured using four forklift paths. The following labels, LC1 through LC4 (see Figure 3-12), defined each load path. For all paths, the vehicle or vehicles were driven along the bridge length from west to east. The distance from the edge of the bridge deck to the closest front axle vehicle wheel edge was 305 mm (12 in.) for load paths LC1, LC2, and LC4 (Figure 3-12). The remaining path, LC3, consisted of a “single-truck along [the] deck center line.” The distance from the deck edge to the closest front-axle wheel edge was 2.5 m (8 ft. 3 in.).

Figure 3-12. Bridge Load Cases.
Two forklifts were used for the live-load test because they were easily available (Figure 3-13). Their short longitudinal axle spacing of 3.16 m (10 ft 4.5 in.) on center and their heavy weight makes this vehicle a unique and useful load configuration for this relatively short-span bridge. The forklift used for all single-truck paths was a Taylor type heavy duty forklift labeled “RED” for analysis; its total weight was 224.68 kN (50.51 kips). The front axle carried 129.35 kN (29.08 kips), this load was distributed between two dual wheels spaced 1.96 m (6 ft. 5 in.) from the center. The rear axle carried 95.33 kN (21.43 kips) through two single wheels spaced at 2.16 m (7 ft. 1 in.) on center, see Figure 3-14-a. The second forklift used for the LC2 load case, a Hyster 330 forklift labeled “YELLOW,” had similar dimensions to the Taylor forklift and had a total weight of 203.73 kN (45.8 kips). The front axle carried 118.77 kN (26.7 kips.), and the rear tandem carried 84.96 kN (19.1 kips.), see Figure 3-14b.

Figure 3-13. Taylor heavyweight forklift used for Live-Load test.
3.4.2 Live-Load Test Procedure

The load tests consisted of the pseudo-static single and double vehicle cases. Previous research performed by Laurendeau et al. (2014) indicated that pseudo-static load cases provide reliable bridge response data. Furthermore, to achieve better understanding of live-load and temperature effects on bridge life, thermal gradients during live load testing and long term monitoring is found to be an appropriate approach to find causes of concrete element cracking spread (Maguire et al., 2015). The forklifts were driven at an approximately constant walking velocity estimated at 3.0 km/h (1.875 mi/h). At the beginning of the test, the vehicle was positioned at the west end of the bridge. Front tire positions were recorded using measured marks on the deck and a marker was put in the data. This was used to convert the data from the time domain into the position domain for data analysis. Three iterations were performed for each path, providing comparable data which allowed errors to be corrected.
In the field during the installation of the sensor, a team of two measured the real dimensions of the bridge and marked down the load path guidelines for the truck drivers with chalk spray. A segmented line was painted on the deck 0.30 m (1 ft. 0 in.) from each deck edge. Those lines served as guidance for load paths LC1, LC2, and LC4. A segmented line drawn down the longitudinal centerline of the bridge served as a guide for the LC3 path. In addition, five lanes were also painted to highlight the bridge’s start, end, quarter lengths, and midspan.

After all the sensors were placed, the team proceeded to initiate the testing of the bridge on October 30, 2017. One team member guided the vehicles by walking in front of them, controlling their speed and location using the guidelines in Figure 3-15.

At the beginning of each test, the truck was positioned at the west end of the bridge. Then, the person guiding the trucks informed both the vehicle driver and the system controller to start the test. Moreover, when the front axles reached each delineated quarter-span line, the guiding member informed the system controller to perform a “CLICK”; this recorded highlight was used to convert from the time domain, which is the BDI system default, to the front axle position domain. This action is crucial for ease of analysis during data processing. Lastly, as the rear axle touched the east end bearing, the guider informed the controller to finalize the test.

To ensure a pseudo-static load case, the forklifts traveled at an average speed of 2.50 km/h (1.86 mi/h), approximately 30 percent slower than the speed established by Laurendeau et al. (2014). After each test, the vehicle or vehicles returned to the west end, and the process was repeated until the end of each test. The load case organization during the field test was first the three iterations of load case LC1, followed by the LC2 case, then the LC3 path, and lastly load case LC4 (see Figure 3-15).
3.5 Data Processing

Following the infield live-load test, the files generated by the STS2 system were stored on several storage drivers as a text data file format. Excel was the software used to process the data and plot the desired bridge responses. The final text file was exported to Excel. These final files contained a list of data points collected, the readings labeled with each sensor tag, the sampling frequency, and the sensor measuring unit.

Initially, the sensor readings previous to load application had to be adjusted to define the real bridge response to sensor noise. This step, called “ZEROING” by Collins (2010), indicates the first usage of the formerly mentioned CLICKS to mark down the beginning of the truck loading onto the bridge west bearing. Then, the value at the data point highlighted by this click was deducted from all readings up to the last CLICK data point. After the zeroing step, with the location of the highlighted clicks defining when the front axle advanced, the data points were converted to front axle position by assuming a constant speed between each CLICK.
CHAPTER 4 EXPERIMENTAL RESULTS

4.1 Live-Load Test Deflection and Strain Results

4.1.1 Deflection Results

The vertical displacement measured by the BDI twangers on each OWSJ midspan was processed and later used to estimate the experimental Girder Distribution Factor. Table 4-1 presents the maxima of each member’s deflection and the statistical properties of the data collected: average (avg.) and coefficient of variation (COV). As expected, the behavior of each OWSJ while under the four load paths revealed higher deformations at the elements contiguous to the wheel loads. Furthermore, a distribution of load could be estimated through the observation of the results shown in Table 4-1. Some unexpected results are presented. For load case LC2, member J5 had larger deflections than the center member, J4. Likewise, in the case of LC3, J5 presented a greater deflection than the expected maximum loaded element, J4.

<table>
<thead>
<tr>
<th>Joist Label</th>
<th>LC1 Avg. (in.)</th>
<th>LC1 Max. (in.)</th>
<th>LC1 COV</th>
<th>LC2 Avg. (in.)</th>
<th>LC2 Max. (in.)</th>
<th>LC2 COV</th>
<th>LC3 Avg. (in.)</th>
<th>LC3 Max. (in.)</th>
<th>LC3 COV</th>
<th>LC4 Avg. (in.)</th>
<th>LC4 Max. (in.)</th>
<th>LC4 COV</th>
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</thead>
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<tr>
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<td>0.228</td>
<td>0.233</td>
<td>0.0184</td>
<td>0.236</td>
<td>0.240</td>
<td>0.0182</td>
<td>0.116</td>
<td>0.122</td>
<td>0.0690</td>
<td>0.033</td>
<td>0.038</td>
<td>0.1091</td>
</tr>
<tr>
<td>J2</td>
<td>0.203</td>
<td>0.208</td>
<td>0.0192</td>
<td>0.253</td>
<td>0.255</td>
<td>0.0138</td>
<td>0.132</td>
<td>0.138</td>
<td>0.0545</td>
<td>0.071</td>
<td>0.076</td>
<td>0.0620</td>
</tr>
<tr>
<td>J3</td>
<td>0.168</td>
<td>0.170</td>
<td>0.0161</td>
<td>0.259</td>
<td>0.260</td>
<td>0.0027</td>
<td>0.140</td>
<td>0.143</td>
<td>0.0379</td>
<td>0.105</td>
<td>0.110</td>
<td>0.0448</td>
</tr>
<tr>
<td>J4</td>
<td>0.136</td>
<td>0.138</td>
<td>0.0243</td>
<td>0.268</td>
<td>0.270</td>
<td>0.0164</td>
<td>0.148</td>
<td>0.150</td>
<td>0.0230</td>
<td>0.143</td>
<td>0.149</td>
<td>0.0385</td>
</tr>
<tr>
<td>J5</td>
<td>0.105</td>
<td>0.108</td>
<td>0.0409</td>
<td>0.299</td>
<td>0.302</td>
<td>0.0117</td>
<td>0.161</td>
<td>0.163</td>
<td>0.0093</td>
<td>0.200</td>
<td>0.208</td>
<td>0.0355</td>
</tr>
<tr>
<td>J6</td>
<td>0.065</td>
<td>0.068</td>
<td>0.0584</td>
<td>0.280</td>
<td>0.283</td>
<td>0.0107</td>
<td>0.144</td>
<td>0.145</td>
<td>0.0118</td>
<td>0.216</td>
<td>0.222</td>
<td>0.0315</td>
</tr>
<tr>
<td>J7</td>
<td>0.023</td>
<td>0.027</td>
<td>0.1696</td>
<td>0.258</td>
<td>0.263</td>
<td>0.0171</td>
<td>0.125</td>
<td>0.128</td>
<td>0.0256</td>
<td>0.232</td>
<td>0.238</td>
<td>0.0276</td>
</tr>
</tbody>
</table>
Plots were also made to present what are commonly referred to as measured influence lines (i.e., response versus load position) for deflection at mid-span during each load case iteration. Figure 4-1, which displays the influence line for the deflection at midspan when two lanes were loaded, is an example of this. Note that each iteration is presented and many of the lines are near each other, making it difficult to discern individual deflections. However, as seen in Table 4-1, member J5 presents greater deflections than element J4, and the scatter of data is readily apparent. The remainder of the influence line figures can be reviewed in ¡Error! No se encuentra el origen de la referencia.

Similar the maximum deflection results, the location of the front axle was at maximum deflection. When observing Table 4-2 the greatest COV, which correlates to a greater variance in the data, can be seen on the two elements furthest from the load path applied, which is the case of member J1 in the LC4 load case and member J7 in the LC1 load case. This effect does not correlate with the COV behavior of the maximum deflection measurements.
AASHTO defines a deflection limit in Equation 4-1 for vehicular loads covering steel, aluminum, and concrete bridges.

\[ \Delta_{\text{LIMIT}} = \frac{L}{800} \]

Eq. 4-1
Where $\Delta_{LIMIT}$ is the maximum deflection under vehicular load, and $L$ is the working bridge span. By inspection, the maximum deflection occurring on LC1 of 0.60 cm (0.238 in.) in member J1 only reached 21% of the 2.86 cm (1.125 in.) maximum deflection allowed for serviceability and vibrations (American Association of State Highway and Transportation Officials (AASHTO), 2014; S. Nowak & Tharmabala, 1989). The design truck load used for this limit weights 30% more than the forklifts used on the test. However, the forklift’s short axle length could mean higher load concentration; therefore, higher deformations than those impose by a AASHTO design truck for this short span bridge.

4.2 Experimental Girder Distribution Factors

Following evaluation and calibration of the field test results. The experimental single- and multiple-lane GDF were determined for each member. These experimental GDFs will be compared with the ones obtained from the AASHTO LRFD design code, Akinci et al. (2013) torsional spring analogy (TSA), displacement spring analysis (DSA), and two SAP2000 FEA models. The deflection readings from the Twangers and the composite section modulus of both exterior and interior OWSJs were used to calculate the experimental GDFs.

Equation 4-2 used by (Stallings et al., 1993; Kim & Nowak, 1997; and Laurendeau et al., 2014) to obtain the experimental GDFs from live load tests, they used strain results on each beam at the a specific point on the bridge span. The elastic GDF are obtained using a ratio between one girder moment and the sum of all girders moments.

$$DFM = \frac{M_i}{\sum_{j=1}^{n} M_j} = \frac{ES_i \varepsilon_i}{\sum_{j=1}^{n} ES_j \varepsilon_j}$$

Eq. 4-2
Since, deflection instead of strain were being used to obtain the GDFs, an alternative solution was derived, see Equation 4-4. Calculating experimental GDFs required using the American Institute of Steel Construction (2016) effective inertia for the composite open-web steel joist methodology. Using this property for both interior and exterior OWSJs, the bottom section modulus and finally the GDF for each girder were calculated using the following equations:

\[ S_{steel}^{th} = \frac{I_{e}^{th}}{y_{bot}^{th}} \]  \hspace{1cm} \text{Eq. 4-3} \\
\[ GDF_{i} = \frac{S_{steel}^{th} \cdot \Delta_{MAX}^{th}}{\sum_{0}^{n} S_{steel}^{nth} \cdot \Delta_{MAX}^{nth}} \]  \hspace{1cm} \text{Eq. 4-4} \\

Where, \( S_{steel}^{th}, I_{e}^{th}, y_{bot}^{th}, \text{ and } \Delta_{MAX}^{th} \) are the section modulus, effective inertia, centroid from the bottom fiber, and the maximum observed deflection of the \( i^{th} \) joist to be evaluated, respectively; \( S_{steel}^{nth}, \Delta_{MAX}^{nth} \) are the section modulus and maximum observed deflection of the \( n^{th} \) joist, respectively, where \( n \) ranges from one to the number of flexural members on the bridge. The exterior OWSJ properties were determined using an Excel spreadsheet shown in APPENDIX B: AUTOMATED JOIST PROPERTIES CALCULATIONS. The final section properties were for the exterior effective moment of inertia \( I_{e-ext} \) equal to 748,530.20 cm\(^4\) (17,983.51 in\(^4\)), the exterior section modulus \( S_{S-ext} \) equal to 7,933.63 cm\(^3\) (484.14 in\(^3\)); correspondingly, the interior OWSJ properties are effective moment of inertia \( I_{e-int} \) equal to 814,044.61 cm\(^4\) (19,557.50 in\(^4\)) and a section modulus \( S_{S-ext} \) equal to 9,337.19 cm\(^3\) (569.79 in\(^3\)). With these values and Equation 4-4 the final GDFs were calculated, the average of the three tests iterations GDFs shown on Table 4-3. With the information provided in Table 4-3, the comparison of codified and theoretical methods...
was performed; thus, with further research, calibration of a final analysis procedure or can be made and standardized.

The exterior girders have a significantly lower effective flange width 786.4 mm (30.96 in.) compared with the 1219.2 mm (48 in.) of an interior girder; thus, it is expected that the exterior member's composite stiffness will be considerably lower than the interiors girders.

<table>
<thead>
<tr>
<th>Load Case Label</th>
<th>Average Experimental GDF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>J1</td>
</tr>
<tr>
<td>LC1</td>
<td>0.218</td>
</tr>
<tr>
<td>LC2</td>
<td>0.113</td>
</tr>
<tr>
<td>LC3</td>
<td>0.106</td>
</tr>
<tr>
<td>LC4</td>
<td>0.030</td>
</tr>
</tbody>
</table>

CHAPTER 5 GDF COMPARISON

Research was done by Kim and Nowak (1997) and others to compare code versus experimental GDF. Due to the uniqueness of this steel-joist supporting components, all codified GDFs that could simulate the system were used to simulate the live load distribution on the Black Slough Bridge. In addition to the evaluation of the AASHTO equations, the spring analogy from Akinci et al. (2013), a Finite Element Analysis (FEA) with equivalent W-Shape steel beams and a displacement 2D spring analysis were performed. At the end of this chapter, a full comparison of the deflection results will be made.

The main objective of the procedure described in this chapter is to find the method which mimics the real response of the bridge. Many factors may influence bridge behavior and most of them are taken into account in AASHTO formulas and the methods used in this report, but as S.
Nowak & Tharmabala (1989) and others have noted, restraint at end bearings can cause an increase in bridge stiffness. Others factors, like human bias (Chang et al., 2017), could cause similar unpredictable behavior; therefore, representing a challenge to accurately predict the real bridge behavior with any theoretical method.

5.1 AASHTO Distribution Factors

The AASHTO moment GDFs chosen to analyze the interior beams were those specified in the LRFD Design Specifications, table 4.6.2.2b-1. Equations Eq. 5-1 and Eq. 5-2 are the single and multiple lane GDFs for interior beams where $S$ = girder spacing (ft.), $L$ = bridge span (ft.), $K_g$ = longitudinal stiffness parameter (in$^4$), and $t_s$ = concrete deck thickness (in.). These equations are specified for a number of standard bridge superstructure system, the following will be analyzed concrete decks over W-Shape or concrete girders, “structures class a and e”. Additionaly, Eq. 5-3 and Eq. 5-4 corresponding the concrete over wood beams “structure class l” system were also evaluated due to the potential simillarity of behavior with the OWSJs.

\[
DFM_{s,i} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0 \cdot L \cdot t_s} \right)^{0.1} \quad \text{Eq. 5-1}
\]

\[
DFM_{m,i} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12.0 \cdot L \cdot t_s^3} \right)^{0.1} \quad \text{Eq. 5-2}
\]

\[
DFM_{1L} = \frac{S}{12} \quad \text{Eq. 5-3}
\]

\[
DFM_{2L} = \frac{S}{10} \quad \text{Eq. 5-4}
\]

For the exterior girders, the lever rule and special analysis (AASHTO C4.6.2.2d) were used to obtain the GDF for both single and multiple loaded lanes. These results were multiplied
by the respective multiple presence factors: 1.2 for single-lane and 1.0 for two or more lanes loaded (AASHTO, 2014). In Table 5-1 the final experimental and AASHTO code GDFs are shown for each load path.

Table 5-1. Comparison of Experimental and AASHTO GDF.

<table>
<thead>
<tr>
<th>Joist Label</th>
<th>LC1 Exp. AASHTO (a &amp; e) Type</th>
<th>AASHTO (l) Type</th>
<th>Exp. AASHTO (a &amp; e) Type</th>
<th>AASHTO (l) Type</th>
<th>Exp. AASHTO (a &amp; e) Type</th>
<th>AASHTO (l) Type</th>
<th>Exp. AASHTO (a &amp; e) Type</th>
<th>AASHTO (l) Type</th>
<th>Exp. AASHTO (a &amp; e) Type</th>
<th>AASHTO (l) Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>0.188 0.369 0.369</td>
<td>0.098 0.352 0.352</td>
<td>0.092 0.369 0.369</td>
<td>0.026 0.369 0.369</td>
<td>0.148 0.281 0.333</td>
<td>0.114 0.281 0.333</td>
<td>0.180 0.281 0.333</td>
<td>0.217 0.281 0.333</td>
<td>0.177 0.369 0.369</td>
<td>0.234 0.281 0.333</td>
</tr>
<tr>
<td>J2</td>
<td>0.238 0.281 0.333</td>
<td>0.149 0.366 0.400</td>
<td>0.156 0.281 0.333</td>
<td>0.155 0.281 0.333</td>
<td>0.175 0.366 0.400</td>
<td>0.180 0.281 0.333</td>
<td>0.161 0.281 0.333</td>
<td>0.234 0.281 0.333</td>
<td>0.177 0.369 0.369</td>
<td>0.234 0.281 0.333</td>
</tr>
<tr>
<td>J3</td>
<td>0.197 0.281 0.333</td>
<td>0.151 0.366 0.400</td>
<td>0.165 0.281 0.333</td>
<td>0.155 0.281 0.333</td>
<td>0.164 0.366 0.400</td>
<td>0.180 0.281 0.333</td>
<td>0.161 0.281 0.333</td>
<td>0.234 0.281 0.333</td>
<td>0.177 0.369 0.369</td>
<td>0.234 0.281 0.333</td>
</tr>
<tr>
<td>J4</td>
<td>0.159 0.281 0.333</td>
<td>0.157 0.366 0.400</td>
<td>0.180 0.281 0.333</td>
<td>0.217 0.281 0.333</td>
<td>0.107 0.352 0.352</td>
<td>0.098 0.369 0.369</td>
<td>0.177 0.369 0.369</td>
<td>0.284 0.369 0.369</td>
<td>0.284 0.369 0.369</td>
<td>0.284 0.369 0.369</td>
</tr>
<tr>
<td>Avg. Error</td>
<td>357.71% 385.06%</td>
<td>163.47% 178.79%</td>
<td>135.55% 158.58%</td>
<td>284.59% 312.16%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As shown in the last row of Table 5-1, due to the large average error for all load cases, it can be stated that for this specific case of study, the AASHTO method is conservative and will overestimate the final live load contribution for a moment and therefore the final load transfer to the flexural beam members. In a more in-depth examination of each experimental and theoretical result, for both load cases LC1 and LC4 the most heavily loaded beam were J2 and J6 respectively. The error on those cases the range between 18% and 20%, this divergence is high compared with the results (Akinci et al., 2013) maximum 10% error. All calculations are located in APPENDIX E: AASHTO DISTRIBUTION FACTORS CALCULATIONS.
5.2 Spring Analogy

According to Akinci et al. (2013), the spring analogy (RSA) method will estimate accurate moment GDFs with a relatively simple analysis by using of a system of torsional bars and springs as decks and girders, respectively; nevertheless, the range of bridges studied in this research only covered the traditional superstructure systems.

The first step is to determine the torsional spring stiffness using the beam’s moment of inertia, establishing the interior supports’ stiffness as unit, and calculating the exterior supports’ using a ratio of interior versus exterior beams moment of inertia. The guidelines suggested by Akinci et al. (2013) were followed. Using the effective inertia method from AISC Design Guide 11, the interior and exterior OWSJs moments of inertia were obtained for the analysis, the final outputs were 814,044.61 cm$^4$ (19,557.50 in$^4$) and 748,531.45 cm$^4$ (17,983.51 in$^4$) respectively.

Next, the 2-D model generated in SAP2000 had the following inputs. Torsional spring rotational stiffness of 1.00 lb-in./rad and 0.76 lb-in./rad for the interior and exterior respectively, and 3.18 lb-in./rad as the torsional stiffness of the torsion bars. Extra calculations of section properties were made to convert the stiffness into a 2.76 MPa (4000 psi) concrete cylinder diameter; this was due to the input requirements of the software. Solving for diameter on Eq. 5-5 used to find the torsional stiffness of circular cross sections (Klejin, 2008).

$$d = \sqrt[4]{\frac{32 \cdot L \cdot K_T}{\pi \cdot G}}$$  \hspace{1cm} \text{Eq. 5-5}

Where $L$ is the beam spacing, $K_T$ is the torsional stiffness calculated using the spring analogy method; and $G$ is the material shear modulus, which, for a 4 ksi concrete, is 10.36 kN/mm²
(1,502,081.9 lb/in²). Figure 5-1 shows the final model generated in SAP2000 with the spring restrained on the X-axis.

![Figure 5-1. Spring Analogy 2-D model in SAP2000.](image)

The resultant GDFs obtained from the RSA analysis for load case LC1 is shown in Figure 5-2, and full analysis properties formulations are shown in APPENDIX C: SPRING ANALOGY PROPERTIES CALCULATIONS. These results are compared with the full-scale Live-Load test outcomes. As seen in this picture, this analogy correctly simulates the load distribution in both LC1 and LC4 load cases. The reason for this variation in the results could be that the deck thickness was not considered; thus, because the deck of the bridge in this study is considerably large, extra and more complex analysis was made with the intention to incorporate this feature and generate a more fitted simulation of this structure’s live load deflection behavior. The GDF results of the Spring Analogy compared to the Experimental results are shown on Figure 5-2.
5.3 Displacement Spring Analysis

The displacement spring analysis (DSA) reduces a 3-D bridge superstructure to a simple 2-D bar and spring supports. The difference from the RSA is the behavior of the springs and loading. Instead of torsional forces, gravity point loads on the position of each path were applied to a beam member with an equivalent inertia to the deck.

To determine the equivalent beam’s stiffness parameters, a deck effective width was established. This theory is based on the assumption that the vehicular loads are distributed through the deck’s length up until a certain length (effective width), see Figure 5-3.

During analysis, the influence width was increased until the error between theoretical and experimental GDFs tended to zero. The analysis starts assuming an effective width equal to the...
vehicle length, and the GDF were calculated with the deck’s Inertia using this width. Then, the effective width was gradually increased until the best result could be obtained.

The final model is shown in Figure 5-4. It shows the model loaded under LC2. The simulation was generated in SAP2000. The stiffness of the five interior and exterior joint springs were calculated using Equation 5-6 and the values were 89.39 kN/cm (51,041.81 lb/in.) and 67.94 kN/cm (38,792.94 lb/in.), for interior and exterior joints, respectively. To define the rectangular frame member used in SAP2000, the width and depth were specified. A fixed depth of 50.8 cm (20 in.) was chosen to model this element and the width of this frame varied depending on the deck width being analyzed. The moment of inertia for the first iteration, using the forklift length as the deck width, was estimated as 499,908.93 cm$^4$ (12,010.36 in.$^4$), equivalent to a section of 26.67 cm (10.5 in.) depth and 316.2 cm (10 ft. 4.5 in.) length. Following, a number of iterations made progressively increasing the deck width until the deflections obtained from the model were as close
as possible to experimental values. It was determined that the optimal length of 7.62 m (25 ft.) yielded the lowest error. Hereafter, using the Equation 5-6 the stiffness calculation of the spring support was done using the OWSJs properties.

\[
K = \frac{48 \times I \times E}{L^3}
\]  
Eq. 5-6

Where \( I \) is the joist effective inertia from AISC (2011), \( E \) is the elastic modulus of steel (29,000 ksi), and \( L \) is the bridge span in inches. The full structure property calculations are shown in APPENDIX D: DISPLACEMENT SPRING PROPERTIES CALCULATIONS.

![Figure 5-4. Structural Displacement Spring Model used in SAP2000.](image)

Figure 5-5 shows the GDF for the experimental (EXP) spring displacement analysis using the forklift length as the effective width (DPi) and with the optimized deck length (DPf). For LC1 and LC4, the DPi and DPf show a similar pattern to the experimental data.
5.4 Equivalent Inertia Beam

A more detailed analysis was performed using a 3-D Finite Element Model. Considering the relatively low stiffness that an OWSJ has compared to a hot-rolled W-shape beam due to the lack of web-material continuity through the member’s span, an equivalent beam was used to model the truss-like elements to estimate the Black Slough Bridge girder GDF.

This model used seven W-shape members at an equal spacing with full composite action between the deck and beam system as specified by the designer, with 0.75 in. diameter shear studs throughout each member’s span. To model the end bearings, ideal pinned-roller supports were assigned to each beam. As seen in Figure 5-6, these end bearings are connected to the top flange of the beam due to the assignment of an insertion point specified by the user at the top center to
mimic where the joists were supported. Automated edge constraints were assigned to all meshed area elements in SAP2000.

![Figure 5-6. 3-D Equivalent Beam SAP2000 Model North-East Face.](image)

A W36x170 Grade 50 steel beam section was selected which had a moment of inertia on the strong axis of 437,043 cm$^4$ (10,500 in.$^4$), equal to the OWSJ non-composite inertia. This frame element was divided into twelve continuous sections, each with a 65.3 cm (2 ft. 1.7 in.) length. Shell elements were used for the 26.67 cm (10.5 in.) deck component over the beams to simulate the real conditions. This area element assigned as a thick shell was divided into finite elements having a parallel-to-the-beams length of 32.64 cm (1 ft. 0.85 in.) and a width of 30 cm (1 ft.). To model the composite action, spring elements were assigned to connect the deck to the top of the beams by generating a link/support property on the software and defining these joints as fixed all directional properties. In order to model the cross bracing, an equivalent W-Shape element with similar stiffness properties was assigned. To estimate the cross bracing stiffness properties, Equation 5-7 developed by Yura and Helwig (1995) was used:

$$K_{Bracing} = \frac{A_b E}{L_b \cos^2 \theta} \quad \text{Eq. 5-8}$$
Where $K_{Bracing}$ is the bracing stiffness, $A_b$ is the brace elements cross sectional area, $E$ is the modulus of elasticity of brace elements, $L_b$ is the length of the brace elements, and $\theta$ is the vertical angle of the brace elements. The final section assigned in the model was a W12x58.

Following 3-D bridge modelling, the forklift loads were assigned to the deck. These were applied at the area element edge-points as point loads, using front and back axle loads and their expected locations. Only static loads were applied; the front axle was placed at the bridge midspan, as seen in Figure 5-7, and a strip of smaller area elements was made to precisely place the loads where the wheel center would theoretically be located.

![Figure 5-7. Load assignment in SAP2000 for LC2 load case.](image)

GDF cross section plots were made in order to visualize the accuracy of the assumptions made in this 3-D model. As shown in Figure 5-8, a very well-defined correlation was achieved using this method.
5.4.1 Calibrated Finite Element Model

A calibrated finite element model (CaliFEM) prepared by Andalib et al. (n.d.) was developed as a global FEM analyzing the Black Slough Bride with all its components, including the OWSJ section as measured in the field, side rails, and cross bracing. Many deck load distributions were considered, like membrane (no out of plane moment transfer) and shell members with out of plane moment transference. The results are presented in Figure 5-9.
5.5 Analysis Methods Comparison

Presented in this section a comparison of all models with the experimental GDFs, at the end of this section, an analysis method will be chosen as the best. The selection criteria based on the error. As seen later in this section, a relation between all methods is found in the two load cases that positioned the truck on one deck side-face. In this section, the most accurate analysis was selected as the best fit approach.

5.5.1 Load Case 1 Error Analysis

The calculated GDFs, including the experimental resultant, for Load Case 1 are shown in Figure 5-10. As a result, the calibrated FEA analysis clearly overestimates the distribution for the exterior OWSJ J1. On the other hand, the results from the optimized displacement spring method
show the overall lowest margin of error among all predictions, although the TSA method presents
the second greatest average error, it overestimated both the maximum exterior and interior GDFs.

The theoretical and experimental GDFs are displayed in Figure 5-10 and Table 5-2. Comparing plots, the DSA approach kept the lowest error (ABS) to 17% (see Table 5-2). Nevertheless, the DSA underestimated the maximum experimental GDF of 0.2178 on J2 by 2.4%. Both the TSA and FEM_{eqv} overestimated both interior and exterior maximum GDF (see Table 5-2).

![Figure 5-10. Distribution of deflection due to LC1.](image)

As shown in Table 5-2, all analyses had a low margin of error estimating the largest loaded
member J2. The equivalent steel beam FEA is the method that overestimated the deflection GDF of this member the most. The other analysis predicted the behavior of this member within a 5% margin of error. On other hand, for the members located outside the load application boundaries on the bridge deck, the variance increased. This may occur due to a neglected partial composite action or bridge deck rigidity; as Akinci et al. (2013) specified, as the deck becomes more flexible the load will be transferred through the members closest to the vehicle loads. In summary, the best
fit method in this scenario was the spring analogy estimating an interior beam factor of 0.25 of the applied load and an exterior factor of 0.21, compared with 0.2378 and 0.1885, respectively, from the experimental results.

5.5.2 Load Case 2 Error Analysis

In this case, the average error within all methods was considerably reduced from LC1. The analysis that showed the lower average percentage of error was the displacement spring method, where all the analysis approaches except FEM_{eqv} were able to overestimate the exterior GDF (see Figure 5-11). Nevertheless, all approaches failed to meet or overestimate the largest of the measured GDFs, which occurred on J5. Numerous causes could be generating this discrepancy: locations of the trucks during the test and the actual overestimation of the deck stiffness. Nevertheless, with a margin of error of -2% less between EXP GDF on J5 of 0.1647 and the FEM_{eqv}’s estimation of 0.1618 (see Table 5-3), this approach, although conservative in this case,
best predicts this bridge’s GDF. As Table 5-3 shows, although the overall predictions were close to predicting the experimental GDFs, all methods failed to overestimate interior maximum GDF.

![Distribution of deflection due to LC2.](image)

**Figure 5-11. Distribution of deflection due to LC2.**

**Table 5-3. Calculated and experimental distribution factor error summary for LC2.**

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>ABS Error</th>
<th>J1</th>
<th>J2</th>
<th>J3</th>
<th>J4</th>
<th>J5</th>
<th>J6</th>
<th>J7</th>
<th>MAX Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Experiment</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TSA</td>
<td>9%</td>
<td>0.1255</td>
<td>0.1355</td>
<td>0.1445</td>
<td>0.1275</td>
<td>0.1505</td>
<td>0.143</td>
<td>0.133</td>
<td>0.1505</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11%</td>
<td>5%</td>
<td>0%</td>
<td>19%</td>
<td>9%</td>
<td>8%</td>
<td>12%</td>
<td>-9%</td>
</tr>
<tr>
<td>DSAi</td>
<td>5%</td>
<td>0.1202</td>
<td>0.1457</td>
<td>0.1487</td>
<td>0.1505</td>
<td>0.1526</td>
<td>0.1531</td>
<td>0.1293</td>
<td>0.1531</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6%</td>
<td>2%</td>
<td>3%</td>
<td>4%</td>
<td>7%</td>
<td>1%</td>
<td>9%</td>
<td>-7%</td>
</tr>
<tr>
<td>DSAf</td>
<td>5%</td>
<td>0.1211</td>
<td>0.1457</td>
<td>0.1481</td>
<td>0.1499</td>
<td>0.1519</td>
<td>0.1530</td>
<td>0.1303</td>
<td>0.1530</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7%</td>
<td>2%</td>
<td>2%</td>
<td>4%</td>
<td>8%</td>
<td>1%</td>
<td>10%</td>
<td>-7%</td>
</tr>
<tr>
<td>EqvFEM</td>
<td>4%</td>
<td>0.1057</td>
<td>0.1506</td>
<td>0.1524</td>
<td>0.1541</td>
<td>0.1580</td>
<td>0.1618</td>
<td>0.1174</td>
<td>0.1618</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6%</td>
<td>6%</td>
<td>5%</td>
<td>2%</td>
<td>4%</td>
<td>4%</td>
<td>1%</td>
<td>-2%</td>
</tr>
<tr>
<td>CaliFEM</td>
<td>6%</td>
<td>0.1234</td>
<td>0.1444</td>
<td>0.1445</td>
<td>0.1457</td>
<td>0.1500</td>
<td>0.1552</td>
<td>0.1369</td>
<td>0.1552</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9%</td>
<td>1%</td>
<td>0%</td>
<td>7%</td>
<td>9%</td>
<td>0%</td>
<td>16%</td>
<td>-6%</td>
</tr>
</tbody>
</table>
5.5.3 Load Case 3 Error Analysis

Load Case 3 correspond to one forklift path in the bridge span centerline path. As seen in Figure 5-12, no method could over predict maximum exterior and interior GDF. The spring analogy overestimates the experimental maximum deflection factor of 0.1851 happening in J5. This method predicted a maximum deflection distribution in the middle OWSJ J4 equal to 0.20 (see Table 5-4). In the case of exterior supporting member deflection distribution, the spring analogy method underestimated this value with -18% average error; however, both the optimized displacement spring and equivalent steel beam methods overestimated this value with 5% and 1% error, respectively. In this case, if the spring analogy considers the contribution of the concrete deck stiffness to the theoretical stiffness of the torsional bar, more deflection would be distributed to the members located furthest from the wheel loads.

![Figure 5-12. Distribution of deflection due to LC3.](image)

Comparing the GDF paths, the \( \text{FEM}_{\text{cal}} \), \( \text{FEM}_{\text{eqv}} \), and DSA methods present similar trends with an overall ABS error margin of 5.33% (see Table 5-4). Nevertheless, these results underestimate the maximum experimental GDF of 0.1782 in J5. The TSA approach predicts the
interior maximum GDF with a margin of +8% but underestimates the exterior beams with an average -20.5% below EXP (see Table 5-4). The FEM_{cal} and DSA both over predict the exterior GDF by 15% and 3%, respectively. Again, the inability to accurately predict the GDF path using TSA was caused by the excluding of the deck section properties contribution to stiffness and therefore distribution of load through the OWSJs. However, the TSA was the only approach that overestimated the maximum interior GDF by +8%.

Table 5-4. Calculated and experimental distribution factor error summary for LC3.

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>ABS Error</th>
<th>J1</th>
<th>J2</th>
<th>J3</th>
<th>J4</th>
<th>J5</th>
<th>J6</th>
<th>J7</th>
<th>MAX Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Experiment</td>
<td></td>
<td>0.1091</td>
<td>0.1460</td>
<td>0.1546</td>
<td>0.1629</td>
<td>0.1782</td>
<td>0.1587</td>
<td>0.1169</td>
<td>0.1782</td>
</tr>
<tr>
<td>TSA</td>
<td>18%</td>
<td>0.0895</td>
<td>0.124</td>
<td>0.191</td>
<td>0.192</td>
<td>0.191</td>
<td>0.124</td>
<td>0.0895</td>
<td>0.192</td>
</tr>
<tr>
<td>DSAi</td>
<td>8%</td>
<td>0.0999</td>
<td>0.1453</td>
<td>0.1672</td>
<td>0.1752</td>
<td>0.1672</td>
<td>0.1453</td>
<td>0.0999</td>
<td>0.1752</td>
</tr>
<tr>
<td>DSAf</td>
<td>4%</td>
<td>0.1123</td>
<td>0.1472</td>
<td>0.1589</td>
<td>0.1632</td>
<td>0.1589</td>
<td>0.1472</td>
<td>0.1123</td>
<td>0.1632</td>
</tr>
<tr>
<td>EqvFEM</td>
<td>7%</td>
<td>0.1015</td>
<td>0.1534</td>
<td>0.1626</td>
<td>0.1650</td>
<td>0.1626</td>
<td>0.1534</td>
<td>0.1015</td>
<td>0.1650</td>
</tr>
<tr>
<td>CaliFEM</td>
<td>5%</td>
<td>0.1162</td>
<td>0.1501</td>
<td>0.1612</td>
<td>0.1638</td>
<td>0.1574</td>
<td>0.1432</td>
<td>0.1081</td>
<td>0.1638</td>
</tr>
</tbody>
</table>

5.5.4 Load Case 4 Error Analysis

As a mirror of Load Case 1, Load Case 4 placed the forklift on the north face side of the bridge and was expected to show similar outcomes. Nevertheless, all methods were fully applied for comparison purposes. As seen in Figure 5-13, the calibrated FEA and the spring analogy method both closely mimic the maximum deflection distribution, but in the exterior member, the calibrated FEA throws an unacceptable result while both the spring analogy and the displacement
spring methods show very realistic results with 14% and 0% error respectively. Finally, as shown on Table 5-5, even though the average error on both displacement springs analysis was the lowest, the closest to the real live load distribution was the spring analogy method with errors of 3% and 13% for the maximum interior and exterior members, respectively.

![Figure 5-13. Distribution of deflection due to LC4.](image)

<table>
<thead>
<tr>
<th>Analysis Methods</th>
<th>ABS Error J1</th>
<th>J2</th>
<th>J3</th>
<th>J4</th>
<th>J5</th>
<th>J6</th>
<th>J7</th>
<th>MAX Value</th>
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<tbody>
<tr>
<td>TSA</td>
<td>0.0369</td>
<td>0.0509</td>
<td>0.0787</td>
<td>0.131</td>
<td>0.226</td>
<td>0.235</td>
<td>0.241</td>
<td>0.241</td>
</tr>
<tr>
<td></td>
<td>20% 33% 30% 15% 5% 1% 14% 4%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DSAi</td>
<td>0.0446</td>
<td>0.0852</td>
<td>0.1181</td>
<td>0.1521</td>
<td>0.1853</td>
<td>0.2132</td>
<td>0.2016</td>
<td>0.2132</td>
</tr>
<tr>
<td></td>
<td>46% 11% 5% 1% 14% 8% 5% -8%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DSAf</td>
<td>0.0456</td>
<td>0.0859</td>
<td>0.1181</td>
<td>0.1508</td>
<td>0.1830</td>
<td>0.2125</td>
<td>0.2041</td>
<td>0.2125</td>
</tr>
<tr>
<td></td>
<td>49% 12% 5% 2% 15% 8% 4% -8%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EqvFEM</td>
<td>0.0084</td>
<td>0.0407</td>
<td>0.0948</td>
<td>0.1516</td>
<td>0.2103</td>
<td>0.2666</td>
<td>0.2277</td>
<td>0.2666</td>
</tr>
<tr>
<td></td>
<td>73% 47% 16% 2% 2% 15% 7% 15%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CaliFEM</td>
<td>0.0000</td>
<td>0.0391</td>
<td>0.0901</td>
<td>0.1447</td>
<td>0.2020</td>
<td>0.2579</td>
<td>0.2661</td>
<td>0.2661</td>
</tr>
<tr>
<td></td>
<td>100% 49% 20% 6% 6% 11% 25% 15%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table 5-5. Calculated and experimental distribution factor error summary for LC4.*
CHAPTER 6 CONCLUSIONS

A possible new short span bridge type using OWSJ was discussed in this report and a live load test was performed. This test was performed to learn more about the full-scale behavior and load distribution effects of OWSJ bridges. Several analysis techniques were compared to predict the GDFs for the OWSJ bridge. The best fit simplified method was the 2-D displacement spring model showing, only in one load case, an average error greater than 10%. A more refined FEA performed by Andalib et al. (n.d.) shows a calibrated parametric representation of the bridge. Further research must be done in order to develop a fully comprehensive reliability, fatigue, and long-term performance of this system. Future research should examine the data and develop a better understanding of short and long-term properties of the bridge system, including a complete fatigue analysis of the welded joints. The final conclusions and recommendations are listed below:

- During loading on the south “LC1” and north “LC2” bounds, deflection measurements and GDF revealed that the interior OWSJ members, J2 and J5, located in between the forklifts’ wheel load paths sustained the largest percentage of the vehicular load.
- The AASHTO GDF overestimated the experimental GDF for all OWSJ beams in all load cases.
- An overall increase of error in both LC1 and LC4 was observed among all theoretical approaches.
- The spring analogy proposed by Akinci et al. (2013) to calculate deflection GDFs accurately mimicked the deflection distribution throughout the bridge OWSJ. This method correctly estimated the maximum interior and exterior beams’ GDFs with an average error between tests of 6.5% and 12.8%, and in all cases except for LC2, provided a conservative prediction.
• The displacement spring analysis best mimicked the load distribution among the bridge cross section. Nevertheless, it tended to underestimate the maximum GDFs for the most loaded members with an average of -4%.

• A finite element model analysis showed that the correlation between the OWSJ non-composite inertia and a standard W-Shape with the same inertia is achievable; though, a more refined analysis will convey more precise results. This method will likely not be feasible for design, especially with respect to shear GDFs, which are not covered in this report.
REFERENCES


APPENDIX A: LIVE LOAD DEFLECTIONS FOR EACH LOAD CASE

Figure A-1. Bottom Flange Mid-Span Deflection Influence Line LC1 Iteration 1.

Figure A-2. Bottom Flange Mid-Span Deflection Influence Line LC1 Iteration 2.
Figure A-3. Bottom Flange Mid-Span Deflection Influence Line LC1 Iteration 3.

Figure A-4. Bottom Flange Mid-Span Deflection Influence Line LC2 Iteration 2.
Figure A-5. Bottom Flange Mid-Span Deflection Influence Line LC2 Iteration 3.

Figure A-6. Bottom Flange Mid-Span Deflection Influence Line LC3 Iteration 1.
Figure A- 7. Bottom Flange Mid-Span Deflection Influence Line LC3 Iteration 2.

Figure A- 8. Bottom Flange Mid-Span Deflection Influence Line LC3 Iteration 3.
Figure A-9. Bottom Flange Mid-Span Deflection Influence Line LC4 Iteration 1.

Figure A-10. Bottom Flange Mid-Span Deflection Influence Line LC4 Iteration 2.
Figure A-11. Bottom Flange Mid-Span Deflection Influence Line LC4 Iteration 3.
### APPENDIX B: AUTOMATED JOIST PROPERTIES CALCULATIONS

<table>
<thead>
<tr>
<th>Input Data and Section Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Spacing (ft.)</td>
<td>4.00</td>
</tr>
<tr>
<td># Beams</td>
<td>7.00</td>
</tr>
<tr>
<td>Effective Span Length (ft.)</td>
<td>74.63</td>
</tr>
<tr>
<td>Deck Width (in.)</td>
<td>302.00</td>
</tr>
<tr>
<td>Deck Overhang (in.)</td>
<td>7.00</td>
</tr>
<tr>
<td>Select Top Chord Section</td>
<td>2L8X4X3/4LLBB</td>
</tr>
<tr>
<td>Select Bottom Chord Section</td>
<td>2L8X4X3/4LLBB</td>
</tr>
<tr>
<td>Vertical Web Member Section</td>
<td>2L3-1/2X3-1/2X3/8</td>
</tr>
<tr>
<td>Diagonal Web Member Section</td>
<td>2L3X3X1/4</td>
</tr>
<tr>
<td>Top Chord Area (in²)</td>
<td>17.00</td>
</tr>
<tr>
<td>Bottom Chord Area (in²)</td>
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</tr>
<tr>
<td>Inertia of Top Chord (in4)</td>
<td>110.00</td>
</tr>
<tr>
<td>Inertia of Bottom Chord (in4)</td>
<td>110.00</td>
</tr>
<tr>
<td>Beam Depth (in.)</td>
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</tr>
<tr>
<td>Web Member Type</td>
<td>Angles</td>
</tr>
<tr>
<td>Web Panel Width (in.)</td>
<td>36</td>
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<tr>
<td>Non-Composite OWSJ Inertia (in4)</td>
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<tr>
<td>$f_c$ (psi)</td>
<td>4,000.00</td>
</tr>
<tr>
<td>$w_c$ (pcf)</td>
<td>145.00</td>
</tr>
<tr>
<td>Deck Depth (in)</td>
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</tr>
<tr>
<td>Concrete Depth Above Deck (in)</td>
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</tr>
<tr>
<td>$E_c$ (psi)</td>
<td>3,492,062.43</td>
</tr>
<tr>
<td>$E_y$ (psi)</td>
<td>29,000,000.00</td>
</tr>
<tr>
<td>$n$</td>
<td>0.12</td>
</tr>
<tr>
<td>Composite Interior OWSJ (in4)</td>
<td>26,266.39</td>
</tr>
<tr>
<td>Composite Exterior OWSJ (in4)</td>
<td>23,503.59</td>
</tr>
<tr>
<td>$C_r$</td>
<td>0.88</td>
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<tr>
<td>$\gamma$</td>
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</tr>
<tr>
<td>$I_e$ Interior OWSJ (in4)</td>
<td>19,557.50</td>
</tr>
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<td>17,983.51</td>
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<tr>
<td>$S_{top}$</td>
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<tr>
<td>$S_{bot}$</td>
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<td>$S_{topE}$</td>
<td>1,210.60</td>
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<td>$S_{botE}$</td>
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### APPENDIX C: SPRING ANALOGY PROPERTIES CALCULATIONS

#### Spring Analogy Analysis Properties

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<thead>
<tr>
<th>Step #1 (Moving Load Analysis)</th>
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<tbody>
<tr>
<td>Total Bridge Stiffness (in4)</td>
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</tr>
<tr>
<td>Wheel Spacing (ft)</td>
<td>10.375</td>
</tr>
<tr>
<td>Truck Width (ft)</td>
<td>7.083</td>
</tr>
<tr>
<td>Front Wheel Load Location &quot;from West to East&quot; (ft)</td>
<td>40</td>
</tr>
<tr>
<td>Rear Wheel Load Location &quot;from West to East&quot; (ft)</td>
<td>29.625</td>
</tr>
<tr>
<td>Front Axle Weight (#)</td>
<td>29,080.00</td>
</tr>
<tr>
<td>Rear Axle Weight (#)</td>
<td>21,440.00</td>
</tr>
<tr>
<td>Max Deflection (in) @ MidSpan</td>
<td>0.1883</td>
</tr>
<tr>
<td>Moment @ Maximum Deflection Loc. (kip-ft)</td>
<td>814.66083</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Step #2 (Spring Analogy Inputs)</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Spacing $S$ (cm)</td>
<td>121.92</td>
</tr>
<tr>
<td>SPAN Length $L$ (m)</td>
<td>22.75</td>
</tr>
<tr>
<td>$I_2 = I_3 = I_4 = I_5 = I_6$</td>
<td>1.00</td>
</tr>
<tr>
<td>$I_1 = I_7$</td>
<td>0.92</td>
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</table>

<table>
<thead>
<tr>
<th>Step #3 (Torsion Bar Stiffness)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_T$</td>
<td>3.18</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step #4 Rotational Spring Constant</th>
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</thead>
<tbody>
<tr>
<td>$K_i$</td>
<td>1.00</td>
</tr>
<tr>
<td>$K_e$</td>
<td>0.92</td>
</tr>
<tr>
<td>$G$ (Shear Modulus) (#/in²)</td>
<td>1502081.9</td>
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<tr>
<td><strong>TORISIION BAR DIAMETER</strong> (in²)</td>
<td>0.179327985</td>
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</table>
## APPENDIX D: DISPLACEMENT SPRING PROPERTIES CALCULATIONS

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<thead>
<tr>
<th>Displacement Spring &amp; Contribute Length Analysis</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Li1 &quot;Truck Length Width&quot; (FT)</td>
<td>10.375</td>
</tr>
<tr>
<td>Li2 &quot;Final Effective Width&quot; (FT)</td>
<td>25</td>
</tr>
<tr>
<td>I deck1 (in $^4$)</td>
<td>12010.35938</td>
</tr>
<tr>
<td>I deck2 (in $^4$)</td>
<td>28940.625</td>
</tr>
<tr>
<td>K$_{\text{interior}}$ (lbs-in)</td>
<td>37910.18332</td>
</tr>
<tr>
<td>K$_{\text{exterior}}$ (lbs-in)</td>
<td>34859.16418</td>
</tr>
<tr>
<td>K$_{\text{non-composite}}$ (lbs-in)</td>
<td>19607.80881</td>
</tr>
<tr>
<td>Weight of the deck 1 (lbs/ft)</td>
<td>1410.351563</td>
</tr>
<tr>
<td>Weight of the deck 2 (lbs/ft)</td>
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<tr>
<td>H &quot;Height of equivalent beam&quot; (in)</td>
<td>12</td>
</tr>
<tr>
<td>B1 &quot;width of equivalent beam&quot; (in)</td>
<td>83.41</td>
</tr>
<tr>
<td>B2 &quot;width of equivalent beam&quot; (in)</td>
<td>200.98</td>
</tr>
</tbody>
</table>
APPENDIX E: AASHTO DISTRIBUTION FACTORS CALCULATIONS

AASHTO Distribution Factor Calculations

Calculate the longitudinal stiffness parameters, \( K_g \), AASHTO 2010

\[ n := \frac{E_{st}}{E_c} = 8.305 \]

\[ I := I_{st} = (1.012 \cdot 10^4) \text{ in}^4 \]

\[ A := A_{st} = 34 \text{ in}^2 \]

\[ e_g := \left( \frac{D}{2} + \frac{t_s}{2} + h_{deck} \right) = 26.75 \text{ in} \]

\[ K_g := n \cdot \left( I + (A \cdot e_g^2) \right) \cdot \frac{1}{\text{in}^4} = 2.86 \cdot 10^5 \]

\[ w = 24 \text{ ft} \quad \text{Clear Bridge width} \]

\[ N_e := \frac{w}{12 \text{ ft}} = 2 \]

\[ S := \frac{S}{\text{ft}} = 4 \quad L := \frac{L}{\text{ft}} = 75 \quad t_s := \frac{t_s}{\text{in}} = 10.5 \quad \text{Parameters with no units for equations} \]

Interior Beams Moment Distribution Factors

Considering a, e, k type of Cross-Sections

One Lane Loaded Interior Beams Concrete Deck on Steel Girder, Cast in Place Concrete Beams or Precast I Girder:

\[ g_{int1L} := 0.06 + \left( \frac{S}{14} \right)^{0.4} \cdot \left( \frac{S}{L} \right)^{0.3} \cdot \left( \frac{K_g}{12.0 \cdot L \cdot t_s^3} \right)^{0.1} = 0.281 \]

Two or More Lanes Loaded Interior Beams Concrete Deck on Steel Girder, Cast in Place Concrete Beams or Precast I Girder:

\[ g_{int2L} := 0.075 + \left( \frac{S}{9.5} \right)^{0.5} \cdot \left( \frac{S}{L} \right)^{0.2} \cdot \left( \frac{K_g}{12.0 \cdot L \cdot t_s^3} \right)^{0.1} = 0.366 \]
Considering a 1 type of Cross-Section

One Lane Loaded Interior Beams Concrete Deck on Wood Beams:

\[ g_{intW1L} = \frac{S}{12} = 0.333 \]

Two or More Lanes Loaded Interior Beams Concrete Deck on Wood Beams:

\[ g_{intW2L} = \frac{S}{10} = 0.4 \]

Exterior Beams Moment Distribution Factors

For one lane Loaded:

\[ N_{L1} = 1 \quad X_{ext} = 12 \text{ ft} \quad \Sigma x2 = 2 \cdot \left( (4 \text{ ft})^2 + (8 \text{ ft})^2 + (12 \text{ ft})^2 \right) = 448 \text{ ft}^2 \]

Special Analysis Method for Load case #1 (One Lane Loaded)

\[ e = 7.25 \text{ ft} \quad \Sigma e = e = 7.25 \text{ ft} \quad m_1 = 1.2 \]

\[ R_1 = \frac{N_{L1}}{N_0} + \frac{X_{ext} \cdot \Sigma e}{\Sigma x^2} = 0.337 \]

\[ g_{ext1Sp} = m_1 \cdot R_1 = 0.404 \]

See fig # on next page.

Fig. # One lane Loaded Special Analysis Method Diagram
Lever Rule Method for Load Case #1 (One Lane Loaded)

Sum of moment on J2 Girder using an unit load as each wheel load

\[ P := 1 \quad \text{Unit Load.} \]

\[ d := 2.125 \text{ ft} \quad \text{Distance from Deck border to tires center line.} \]

\[ d_f := \frac{d - s}{ft} = 1.542 \quad \text{Distance from Girder centroid to tires center line.} \]

\[ R_{J1} := \frac{P}{2} \left( \frac{S - d_f}{S} \right) = 0.307 \quad \text{Reaction of exterior Girder in function of unit truck forces.} \]

Apply multiple precense factor:

\[ g_{ext1} := m_1 \cdot R_{J1} = 0.369 \]

Table 4.6.2.1-1 Method for Load Case #2 (Two Lanes Loaded):

\[ d_e := 1.75 \text{ in} \]

\[ e := 0.77 + \frac{d_e}{9.1 \text{ in}} = 0.962 \]

\[ g_{ext2} := e \cdot g_{int2} = 0.352 \]

Special Analysis Method for Load case #2 (Two Lanes Loaded)

\[ N_{1/2} := 2 \quad X_{ext} := 12 \text{ ft} \quad \Sigma x2 := 2 \cdot \left( (4 \text{ ft})^2 + (8 \text{ ft})^2 + (12 \text{ ft})^2 \right) = 448 \text{ ft}^2 \]

\[ w_{red} := 29080 \text{ lbf} \quad \text{Force applied by front axle of red forklift.} \]

\[ w_{yellow} := 26700 \text{ lbf} \quad \text{Force applied by front axle of yellow forklift.} \]

\[ \mu := \frac{w_{yellow}}{w_{red}} = 0.918 \]

\[ e_1 := 7.25 \text{ ft} \quad e_2 := \mu \cdot 7.25 \text{ ft} \quad m_2 := 1.0 \]

\[ \Sigma e2 := e_1 - e_2 = 0.593 \text{ ft} \]
\[ R_2 := \frac{N_{L2}}{N_b} + \frac{X_{ext} \cdot \Sigma e2}{\Sigma e2} = 0.302 \]

\[ g_{ext2sp} = m_2 \cdot R_2 = 0.302 \]

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Fig # Two lanes Loaded Special Analysis Method Diagram

Using Findings by Maguire and Torres Find the girder Distribution Factor

**GDF Exterior Two Lane Loaded Case**

\[ MGDF_{ext2L} := -\frac{7 \cdot L}{5000} - \frac{9 \cdot t_y}{1000} + \frac{S}{10} + \frac{4}{25} = 0.361 \]

**GDF Exterior One Lane Loaded Case**

\[ MGDF_{ext1L} := -\frac{L}{500} - \frac{2 \cdot t_y}{125} + \frac{3 \cdot S}{50} + \frac{21}{50} = 0.342 \]

**GDF Interior Two Lane Loaded Case**

\[ MGDF_{int2L} := -\frac{9 \cdot L}{5000} - \frac{t_y}{50} + \frac{29 \cdot S}{500} + \frac{14}{25} = 0.447 \]

**GDF Interior One Lane Loaded Case**

\[ MGDF_{int1L} := -\frac{L}{625} - \frac{9 \cdot t_y}{500} + \frac{S}{25} + \frac{2}{5} = 0.251 \]