SINGLE LANE LIVE LOAD DISTRIBUTION FACTOR FOR DECKED
PRECAST / PRESTRESSED CONCRETE GIRDER BRIDGES

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PRECAST / PRESTRESSED CONCRETE GIRDER BRIDGES

A
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Abstract

The Alaska Department of Transportation (AKDOT) uses the decked precast, prestressed concrete bulb-tee girder for most of its bridge construction. The live load distribution factor (DF) equations provided by the American Association of State Highway and Transportation Officials (AASHTO) for the decked bulb-tee girder system do not differentiate between a single or multilane loaded condition. This practice results in a single lane load rating penalty for decked bulb-tee girder bridges. The research objective of this thesis is to determine DF simplified equations that accurately predict the distribution factor of the decked bulb-tee girder system when it is only subjected to single lane loading.

Eight decked bulb-tee bridges were instrumented. Each bridge was loaded with a single load vehicle to simulate the single lane loaded condition. The experimental data were used to calibrate grillage models of the decked bulb-tee girder system. The calibrated grillage models were used to conduct a parametric study of the bulb-tee girder system subjected to single lane loaded condition. Eight new simplified equations that describe the single lane loaded distribution factor for both shear and moment forces of these bridges are developed in this thesis.

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1 Introduction

1.1 Background and Problem Statement

The State of Alaska is unique in that its construction season is much shorter than other states. Due to this shortened construction season, Alaska Department of Transportation and Public Facilities (AKDOT& PF) uses the decked bulb-tee bridge girder system to construct most of the new highway bridges in the state. The decked bulb-tee girder is a precast, prestressed concrete girder with a top flange wide and deep enough to act as the bridge deck. The construction of these bridge systems only requires placing the girders then connecting the girders with a shear key and grout. There is no requirement for form work or cast in place concrete. This type of system allows for very rapid bridge construction and is ideal for the short construction season in Alaska. While this bridge system is very prevalent in Alaska it is not used much in other states and therefore the system has not been thoroughly researched.

The AKDOT uses the AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications for design and evaluation of Alaska's highway bridges (4). For most bridge systems, the LRFD Specifications has two distinct groups of simplified equations to determine the live load distribution factor (DF) of a bridge girder. One set of equations provides the DF under general or multilane loaded conditions. The other set of equations provides the DF when the bridge is subjected to a single lane loaded condition. Typically, the DF equations for the single lane loaded condition provide a lower

distribution factor than the equations for the multi lane loaded condition. However, the simplified equations for the decked bulb-tee bridge girder system, where the girders are only connected enough to prevent relative vertical translation, do not distinguish between the single lane and multi lane loaded condition. Typically designers use the DF equations for the single lane loaded condition to rate bridges for permit loads or overload conditions where the bridge will be subjected to only one truck. Because the LRFD design does not distinguish between the multilane loaded and single lane loaded condition, there is a load rating penalty for the decked bulb-tee bridge girder system. Due to this load rating penalty, the AKDOT has funded research aimed at determining an accurate set of DF equations that describe the behavior of the decked bulb-tee bridge system under a single lane-loaded condition.

1.2 Current Distribution Factor Equations and Related Studies

By looking at the development of the AASHTO DF equations, we can see that the reason there is no simplified equation for the single lane loaded condition is because of a lack of research on the decked bulb-tee bridge system. The following portion of this chapter is a summary of an extensive literary review conducted by Dr. John Ma who is part of the research team working on the development of single lane loaded distribution factor equation funded by AKDOT (8). The simplified method of bridge design using a distribution factor to uncouple transverse behavior from longitudinal behavior has been a part of the AASHTO design specifications since its first publication in 1931. The first DF equation based off of Newmark's research was a function of only the girder spacing (S) and the type of bridge system (10). It had the following form:

$$g = \frac{S}{D} \quad (1.1)$$

The variable g represents the wheel load distribution factor, S is the girder spacing, and D is a coefficient dependant only on the type of bridge system. This was used for general bridge design and did not have provisions for a single lane loaded condition. The multi-beam precast concrete bridge system was not even addressed in the design specifications until 1965. This category of bridges includes all precast concrete bridges that are made up of precast sections that span the whole length of the bridge. These sections are then placed side by side with a longitudinal joint between each precast section. The precast sections are then connected to one another across this longitudinal joint with shear keys and grout. The cross-section of these multi-beam precast concrete bridges encompasses a wide variety of shapes ranging from solid slab sections, box girder sections, channel sections, double Tee section, to decked bulb-tee sections, as shown in figure 1.1.

Figure 1.1 Cross-sections of Multi-Beam / Precast Systems

The 9th edition of the AASHTO specifications published in 1965 only addressed multi-beam precast concrete bridges as it pertained to the solid slab section. It stated that the

distribution width of a wheel load is equal to $4.0 + 0.06 \times L$ (ft) not to exceed 7.0 ft where L is the span length of the bridge.

In 1970, new DF simplified equation were proposed by Sanders and Elleby in NCHRP Report Number 83 (2). That study extended the spectrum of bridge systems to the multi-beam precast concrete bridges to include the solid slab, circular voided slab, box, and channel cross-sections. That study proposed a revision to the existing S/D equation by making the coefficient D a function of the aspect ratio, relative stiffness of beams and floor, relative diaphragm stiffness, and bridge continuity. The DF equation for multi-beam bridges took the form of Eq. (1.1) while “S” and “D” were defined as follows:

$$S = \frac{12 \cdot N_L + 9}{N_g} \quad (1.2)$$

$$D = 5 + \frac{N_L}{10} + \left(3 - \frac{2 \cdot N_L}{7}\right) \cdot \left(1 - \frac{C}{3}\right)^2 \quad C \leq 3 \quad (1.3)$$

$$D = 5 + \frac{N_L}{10} \quad C > 3 \quad (1.4)$$

$$C = K \cdot \frac{W}{L} \quad (1.5)$$

$$K = \sqrt{\frac{E}{2 \cdot G} \cdot \frac{I_l}{J_l + J_t}} \quad (1.6)$$

The effective girder spacing (S) is a function of the number of lanes (N_L) and the number of girders (N_g). The coefficient D is a function of the number of Lanes (N_L), and “C” a stiffness parameter that depends on the type of bridge, bridge and beam

geometry, and material properties. “W” is the overall width of the bridge in feet; “L” is the span length in feet; “EI” is the flexural stiffness of the transformed beam section per unit width; GJ_t is the torsional stiffness of the transformed beam section per unit width; and GJ_s is the torsional stiffness of a unit width of bridge deck slab.

While equations (1.2) to (1.6) provided more accuracy than the previous S/D equation it did not cover multi-beam double tee, bulb tee, single tee or other such stemmed cross-sections. In 1986, the University of Washington published a new set of DF equations in NCHRP Report 287 (1). That research focused on understanding the distribution of stemmed multi-beam bridge systems to include the decked bulb-tee cross-section. The objective of that study was to modify the existing distribution factor equations so that they could be used for multi-beam bridges with stemmed cross-sections. The Washington study produced the following changes to the design specifications for multi-beam bridge systems.

$$D = (5.75 - 0.5N_L) + (.7 \cdot N_L) \cdot (1 - 0.2C)^2 \quad C \leq 5 \quad (1.7)$$

$$D = 5.75 - 0.5N_L \quad C > 5 \quad (1.8)$$

$$C = K \cdot \frac{W}{L} \quad (1.9)$$

$$K = \sqrt{\frac{E \cdot I}{2 \cdot G \cdot J}} \quad (1.10)$$

The spacing S is defined as the width of the precast member, EI is the flexural stiffness of each girder, GJ is the torsional stiffness of each girder, and all the other variables are the same as in equations (1.2) to (1.6).

The University of Washington Study is the furthest the simplified equations for the bulb-tee bridge system has been developed. Since that study the design community has identified many shortcomings in the S/D formulas. In 1991 in NCHRP Report 12-26, the research team of Zokaie et al. reconstructed the simplified equations governing most other bridge systems not including the bulb-tees (3). The report provided new simplified equations which specify a distinct distribution factor equation for interior and exterior girders, moment and shear forces, and also single and multilane loading condition. The research conducted in the Washington Study only focused on the distribution of moment for design purposes. It only considered multilane loaded cases and did not focus on the distribution of a single lane loaded case for the purpose of bridge rating. Therefore, AASHTO presents only one equation for both single and multilane loaded conditions for bulb-tee bridges, not because research shows the distribution for each of these cases to be the same but because the single lane loaded condition has not been explored.

1.3 Objective of Research

Previous research on the decked bulb-tee bridge system has not studied the shear distribution of the bridge nor the distribution on exterior girders. There has also been no research on the distribution when this bridge system is only loaded under a single lane loaded condition. This thesis focuses on developing single lane live load simplified DF equations for the bulb-tee bridge system. Distribution on both interior and exterior girders as well as the distribution of shear and moment forces is studied. Another

objective of this thesis is to better understand the behavior of the bulb-tee bridge system: how well experimental data compares to grillage models of this bridge, the effects of the longitudinal joint, and how the bridge system differs from a similar bridge system that has transverse flexural continuity.

1.4 Thesis Overview

This thesis discusses how data was collected from eight bridges and the methods used to test these bridges. The results of this testing is presented and compared to existing simplified equations. The data from this testing is used to calibrate grillage models. These grillage models are then used to study the behavior of the bridge system and used to develop a database of the same bridge system with different parameters. The effects of different parameters on the various distribution factors are studied from the grillage model data base. From this data base, simplified equations describing the single lane live load distribution factor are developed which are a function of the parameters which have the most significant effect on the distribution factor.

2 Field Testing

2.1 Summary of Bridges Tested

Table 2.1 shows the tested eight bridges in Anchorage, Alaska. Figure 2.1 shows the underside of a typical tested bridge. In selecting the bridges to instrument, UAF researchers considered the following factors.

- * They are all located in or near Anchorage, Alaska.
- * Traffic can be closed during late night hours for all these bridges.
- * They are all accessible to instrument.
- * They represent different geometry of the bridges in Alaska in terms of skew angles and aspect ratio (length/width).

The research team has also decided to test paired structures to provide verification of the instrumentation and modeling procedures.

Table 2.1 Field Tested Bridges (From May 6 to May 19, 2003)

Name		Bridge Geometry			Girder	
		Span (ft)	Width (ft)	Skew (°)	Spacing (in.)	Depth (in.)
Set 1	W100th NB	116.0	37.0	0	88.4	54.0
	W100th SB					
Set 2*	Dimond Rd	110.0	107.0	0	90.6	54.0
	Dowling Rd					
Set 3	Campbell NB	139.0	37.0	4.3	88.4	65.0
	Campbell SB					
Set 4	Huffman NB	128.0	37.0	27.5	72.0	54.5
	Huffman SB					

* Note: Tee shape girder in Set 2 instead of decked Bulb-Tee shape used in Set 1, 3, and 4.

Figure 2.1 Underside of Campbell Creek Bridge

2.2 Equipment and Personnel

2.2.1 Testing Personnel

The research team consisted of a joint operation between the University of Alaska Fairbanks (UAF) and the Alaska Department of Transportation (AKDOT). From UAF the principal researchers were Dr. John Ma, and Dr. Leroy Hulse with research assistants Sanjay Chaudhury, and Jason Millam. From AKDOT Bridge Section, Gary

Scarborough, and John Orbistondo assisted in the testing. Load vehicle drivers and traffic control personnel also assisted as required.

2.2.2 Equipment

Strain Transducers – Full-bridge reusable strain transducers fabricated by Bridge Diagnostics, Inc. (BDI) Transducers shown in Figure 2.2 were attached to the concrete using Loctite Brand instant adhesive.

Strain Transducer for Structural Testing

Figure 2.2 Strain Transducer

Data Acquisition System – MEGADAC 5414 Series by OPTIM Electronics. This system was connected to a laptop computer utilizing TCS for Windows Version 3.4 Software. Figure 2.3 shows a setup of the mobile lab.

Figure 2.3 Mobile Lab with Data Acquisition Equipment

2.2.3 Load Vehicle Description

The Load vehicle used in the testing was a loaded DOT End Dump Truck. See Figure 2.4 and 2.5 for a picture of the load vehicle approximate vehicle foot print location and wheel loads. Wheel loads were measured on 5/8/03 and have a measured error of $\pm 1\%$. These values changed over the course of the testing period due to rain and different fuel levels.

Figure 2.4 Load Vehicle

Figure 2.5 Load Vehicle Wheel Load Diagram

2.3 Load Positions

2.3.1 Longitudinal Positioning of Test Vehicle

During loading, the load vehicle would travel across the bridge in the same direction for each transverse load position, and for each loading condition. The direction of loading for each bridge is as follows: For Campbell SB and NB, Huffman NB, 100th NB, the direction of loading was the same direction as the direction of normal traffic crossing the bridge (ie. direction of loading was north on Huffman NB); For Huffman SB, and 100th SB the direction of loading was opposite the direction of traffic (ie. direction of loading was north on Huffman SB). The Diamond and Dowling Bridges each had both North and South bound traffic crossing the bridge. Diamond bridge was loaded only on the eastern side of the bridge which typically has North Bound Traffic, however, the direction of loading on this bridge was South, Opposite the direction of traffic. Dowling bridge was loaded only on the western side of the bridge which typically has South Bound Traffic, and the direction of loading on this bridge was South in the same direction as traffic. During the testing period, there were two main methods of loading the bridge: Continuous Loading, and Static Loading.

During Continuous Loading, the load vehicle drove at a constant speed of two miles per hour along a straight longitudinal girder line across the bridge. Data during this loading condition recorded continuously before the load vehicle moved onto the

bridge, while the vehicle moved across the bridge, and as the vehicle moved off the bridge. During this loading condition the transverse position of the vehicle is known, there is no method of relating the measured strain values to the vehicle's longitudinal position on the bridge. Normally, this loading condition was conducted to verify the accuracy of the data and determine whether or not any strain gauges might be malfunctioning.

During the Static Loading, the load vehicle drove to three known longitudinal stop positions along a given girder line. The first set of data was recorded as the vehicle drove onto the bridge to its first stop position. The next set of data was recorded as the vehicle drove from its first stop position to its second stop position. The third set of data was recorded as the vehicle drove from its second stop position to its third stop position located roughly halfway along the length of the bridge. Three data sets were recorded for each girder line or transverse positioning which the vehicle drove along. A minimum of 30 seconds of data was recorded as the vehicle remained stationary in the stop position. Figure 2.7 depicts the three longitudinal stop positions which the load vehicle moved to during the static loading condition. Figure 2.6 shows the load vehicle in the third stop position. The first stop position places the vehicle with its driver's side rear wheel centered at a distance H (representing the girder depth) away from the abutment. The second stop positions locates the vehicle

with its second axle (driver's side wheel) $\frac{1}{4}$ of the span length of the bridge. The third stop position locates the vehicle's second axle (driver's side wheel) at the center line of the bridge span. For non skew bridges, both the driver's side and passenger's side wheels will be located at the same relative longitudinal position. There is an error of $\pm 12''$ in the placement of vehicle on its longitudinal position.

Figure 2.6 Load Vehicle at Third Stop Position at Midspan

Figure 2.7 Longitudinal Stop Positions

Note: The diagram depicts the vehicle at different transverse positions for clarity. During actual loading the vehicle stayed in the same transverse position as it moved to its three longitudinal stop positions.

2.3.2 Transverse Positioning of Load Vehicle

Most of the transverse loading positions are located directly over a girder. These loading positions are defined by the girder number over which they drive. The girder numbering system is as follows: Girder number one is always the furthest girder to the right of the bridge based off the **direction of traffic, not the direction of loading**, girders are then numbered consecutively from right to left. Girder number one on the Diamond Bridge is the easternmost girder. Girder number one on the Dowling Bridge is the westernmost girder. From the perspective of the loading direction, Girder number one is the girder furthest to the **right** for Campbell NB &

SB, Huffman NB, 100th NB, and Dowling. From the perspective of the loading direction, Girder number one is the girder furthest to the **left** for Huffman SB, 100th SB, and Diamond. For each interior girder loaded, the load vehicle is positioned so that its wheels are centered over the centerline of the girder. For each exterior girder loaded, the vehicle is positioned with the centerline of its outside wheel line to be approximately 2 ft from the edge of the bridge. Figure 2.8 shows the wheel loads relative to the girders.

Figure 2.8 Transverse Vehicle Positions (General)

The Huffman St. Bridge had a unique transverse loading positioning because it was an even numbered girder bridge. The exterior girders for the Huffman St. Bridge (G1, G6) were each loaded in the same manner as the exterior girders of the other bridges. Instead of positioning the vehicle directly over each interior girder, the

vehicle was positioned with its driver side wheels on the center line of the bridge (Joint Right), with its wheels centered over the center line of the bridge (Center), and with its passenger side wheels on the center line of the bridge (Joint Left). This can be seen in figure 2.9.

Figure 2.9 Transverse Vehicle Positions (Huffman)

Note: The diagram depicts the girder numbering system as it would appear for Huffman NB. For Huffman SB the girders would be numbered with G1 on the left and G6 on the right because the direction of loading is opposite the direction of traffic.

The Bridges at Diamond and Dowling intersections also had a unique loading positioning referred to as Joint Loading. For each of these bridges, the Joint Loading conditions refers to one wheel line positioned directly on the joint between girder's 3 and 4 while the other wheel line is placed over the top of girder 3. Figure 2.10 shows

the joint loading condition. The remainder of the girder line loading conditions on Diamond and Dowling are similar to the General Position noted in Figure 2.8.

Figure 2.10 Transverse Vehicle Positions (Diamond / Dowling)

Note: The diagram depicts the girder numbering system as it would appear for Dowling. For Diamond the diagram would be a mirror image of this diagram.

2.4 Strain Gauge Positioning

During field testing 24 gauges were placed to measure strain on each bridge except Campbell South Bound and Huffman North Bound. These two bridges only had 22 gauges. There were three main categories of gauge placement: one set of gauges would be used to measure shear response; the second set of gauges would be used to measure flexural stresses due to midspan moment; and the third set of gauges would be used to measure axial stress in the intermediate steel diaphragms. For each girder, there were a potential of six different locations which where shear gauges were placed and two different locations where moment gauges were placed. The shear gauges would always be placed a distance H (H = depth of the girder) away from the

face of the end diaphragm and vertically on the approximate location of the neutral axis (N. A.). The shear gauges could either be oriented 45deg towards (S1) or away (S3) from the end diaphragm or vertically (S2). They could also be positioned either on the right (S1R) or left (S1L) side of the girder. Note that the right and left hand side of a girder is distinguished by the direction in which the girder is loaded. The moment gauges were always positioned at midspan and located either centered on the bottom flange (M1) or on the left hand side of the web (M2). The moment gauges positioned on the web were located vertically at the highest position on the web just below the top flange. The positions of the gauges are shown in Figure 2.11. Figure 2.12 shows shear gauges in place and Figure 2.13 shows the placement of moment gauges. The right or left side of the girder is always based off of the direction the load vehicle moves across the bridge.

Figure 2.11 Gauge Positions (Elevation)

Figure 2.12 Shear Strain Gauges in Place

Figure 2.13 Placement of Moment Strain

The Diaphragm gauges are located at either quarter span or midspan. The gauges were placed halfway between the midpoint of the K brace and the edge of the girder. The gauges are identified by the two girder between which they reside. Figures 2.14 and 2.15 show the general location of the gauges on the steel diaphragms.

Figure 2.14 Gauge Location on Steel Diaphragm

Figure 2.15 Picture of Strain Gauge on Steel Diaphragm

2.5 Data Presentation

Appendix A shows some sample graphs of the data collected. The data shown is for one of the bridges tested at 100th Avenue and Minnesota when the load vehicle was positioned over the first girder line and moved to the three static load positions. Each bridge has between two and four graphs per loading condition. Each graph shows the data of a group of strain gauges (Shear, Moment). The 100th NB Bridge has 4 graphs per loading condition. The title of each of the graphs describes the loading condition. 100th NB has either continuous or static loading over each of the different girders (G1-G5). All of these loading conditions are depicted in Figure 2.8. The graphs of the static loading conditions show all three stop positions for a given girder line in one graph. To distinguish between the different static loading conditions there is a 10 second separation period where the graph is flat and has the same value for 10 seconds between the different static loading conditions.

The first graph shows all of the midspan moment gauges. Each gauge is located at the bottom flange at the M1 position from Figure 2.11. For 100th NB there were no gauges placed in the M2 Position. Each of the different series represents the midspan gauge for a certain girder. For example, the series entitled G1 represents the gauge located at the M1 position on girder number one which is an edge girder located on the right hand side of the 100th NB Bridge.

The second graph shows all of the shear gauges in S1 Position (angled 45deg into the end diaphragm) from Figure 2.11. Since all of the gauges are in the same position, they are only distinguished by the girder number they are on and whether they are on the left or right side of that girder. For example, series G3R represents the shear gauge located on girder number 3 which is the middle girder of the 100th avenue bridge, it is angled into the diaphragm and is on the right side of the girder.

The other two graphs for the Static G1 Loading Condition show the strains from the gauges located in the S2 and S3 positions and show the strains on the intermediate steel diaphragms. This data is used to determine the Distribution Factor of each of the bridges as will be shown in Chapter 3.

3 Field Testing Data Analysis

3.1 Loading Key

Section 3.3 will present data derived from different loading positions. The following labeling system will be used to define the loading position:

100th NB – 3 - G1

The first term indicates the name of the bridge being loaded, the second term represents the longitudinal location of the load, and the final term indicates the transverse position of the load by labeling the girder over which the majority of the load is positioned. The Bridge labels are as following: 100th NB, 100th SB, Huffman NB, Huffman SB, Campbell NB, Campbell SB, Diamond and Dowling. Three labels will define the longitudinal position of the load: 1 represents the shear loading position located a distance of H (height of the girder) away from the abutment; 2 represents the vehicle loading at $\frac{1}{4}$ span; and 3 represents the vehicle loading at midspan. Figure 3.1 shows each of these loading positions.

Figure 3.1 Longitudinal Stop Positions

Note: The diagram depicts the vehicle at different transverse positions for clarity. During actual loading the vehicle stayed in the same transverse position as it moved to its three longitudinal stop positions.

The transverse loading is labeled according to the girder over which most of the load is positioned. For most cases the load is centered over the girder, however for the edge girders the load is positioned as close to the edge of the bridge as was possible which is not necessarily centered over the edge girder. Girders are numbered consecutively with the first girder G1 representing the girder on the right side of the bridge based off the direction of traffic (not necessarily the direction of loading).

3.2 Calculation of Distribution Factors from Strain Data

The data in section 3.3 is presented as a distribution factor which is derived from the strain gauges. These distribution factors are compared to distribution factors derived

from rigorous math models and from the simplified equations presented in the AASHTO LRFD. To determine the moment distribution factors, the following method was used.

$$DF_{\text{moment}} = \frac{\varepsilon_x}{\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4 + \varepsilon_5} \quad (3.1)$$

Where ε_x is the strain measured directly under the loaded girder from the strain gauge at position M1, and ε_{1to5} is the strain measured from all five gauges at position M1 on each respective girder. The single lane distribution factor for shear was calculated in a similar method except each of the girder strains was calculated as an average of the gauges in the S1 position on the left and right side of the girder:

$$\varepsilon_x = \frac{\varepsilon_{S1L} + \varepsilon_{S1R}}{2} \quad (3.2)$$

By determining the shear distribution in this method, the torsional effects on the strain gauges will be averaged out.

Appendix A shows the strain values for loading over Girder 1. The moment distribution factor for the edge girder of 100th ave. bridge can be calculated From Figure A.1. From the series entitled G1 the average strain value when the vehicle is in the 100th NB – 3 - G1 position is -344 micro strains. The distribution factor on this girder is then found using the strains from the other girders as shown in the following example.

$$\frac{-344}{-344 - 259 - 125.5 - 38.4 + 34} = 0.469 \quad (3.3)$$

The DF value reported in the table at the end of this chapter for the moment distribution on exterior girders is .45. This is the average between the value derived in equation 3.3

and the values derived from Girder G5 on the same bridge as well as the values found from girders G1 and G5 on the bridge from 100th South Bound. The shear distribution for this bridge can be found in a similar manner using the data from Figure A.2. The only difference is that the strain values from the left and right side of each girder $(G1L + G1R)/2$ must be averaged.

3.3 Measured Load Distribution from Each of the Tested Bridges

Figures 3.2 to 3.29 show the force distribution on each girder found from the strains and the method described in section 3.2. Since the AASHTO separates the simplified equations for the distribution factor of exterior and interior girders, the following graphs present data from loading conditions which produce the largest distribution factor on either exterior or interior girders.

3.3.1 Data from Twin Bridges Located at 100th Ave.

Figures 3.2 and 3.3 show the moment distribution factor for 100th North Bound, and Figures 3.4 and 3.5 show the moment distribution factor for 100th South Bound. The distribution factor for girder number two in Figures 3.2 and 3.4 is largest when the load vehicle drives closest to the edge of the bridge in the G1 position. The same trend is apparent on the other side of the bridge where girder number four in Figures 3.3 and 3.5 is largest when the load vehicle drives in the G5 position. This shows that for this particular bridge, the worst loading condition for an interior girder is when the vehicle drives closest to the edge of the bridge not when the vehicle is centered over the top of the girder.

Figure 3.2 100th NB – 3 – G1&G2 Figure 3.3 100th NB – 3 – G4&G5

Figure 3.4 100th SB – 3 – G1&G2

Figure 3.5 100th SB – 3 – G4&G5

The G1 and G5 loading on this bridge produce the greatest distribution factors for both the exterior and interior girders. Table 3.1 shows a summary of the moment distribution factors found from 100th NB and SB bridges.

Table 3.1 Distribution Factor for Moment for Bridges at 100th ave.:

	Interior Girders		Exterior Girders	
	G2	G4	G1	G5
100 th NB	.35	.31	.47	.46
100 th SB	.35	.37	.42	.44
Average	.35		.45	

Figures 3.6 and 3.7 show the shear distribution factor for 100th North Bound, and Figures 3.8 and 3.9 show the shear distribution factor for 100th South Bound. The shear distribution factors of the 100th ave bridges were larger than the moment distribution factors. The worst loading condition for shear distribution always occurred when the load vehicle drove directly over the top of the girder regardless whether or not the girder was an interior or exterior girder. Table 3.2 shows a summary of the shear distribution factors found from 100th NB and SB bridges.

Table 3.2 Distribution Factor for Shear for Bridges at 100th ave.:

	Interior Girders		Exterior Girders	
	G2	G4	G1	G5
100 th NB	.46	.43	.73	.69
100 th SB	.43	.41	.69	.53
Average	.43		.66

Figure 3.6 100th NB – 1 – G1&G2

Figure 3.7 100th NB – 1 – G4&G5

Figure 3.8 100th SB – 1 – G1&G2

Figure 3.9 100th SB – 1 – G4&G5

3.3.2 Data from Twin Bridges Located at Campbell Creek

Figures 3.10 and 3.11 show the moment distribution factor for Campbell North Bound, and Figures 3.12 and 3.13 show the moment distribution factor for Campbell South Bound. Both the load on the exterior girder and load directly over the first interior girder appear to make the same distribution factor on the second girder. Figure 3.13 shows the moment distribution on the girders for Campbell SB when the load vehicle is positioned over girder number 4 and 5. The data shown in this graph does not follow the same trend as in the other moment distribution graphs. The moment strain gauge placed on girder number 5 for this bridge had not adhered properly to the concrete during testing. Therefore, the strains found from this gauge are suspect. This suspect strain gauge is probably the cause for the abnormal behavior in the graph of Figure 3.13. Table 3.3 shows a summary of the moment distribution factor values found from the twin bridges at Campbell Creek. The average DF value at the bottom of the table does not incorporate the values found Figure 3.13.

Table 3.3 Distribution Factor for Moment for Bridges at Campbell Creek:

	Interior Girders		Exterior Girders	
	G2	G4	G1	G5
Campbell NB	.38	.30	.53	.61
Campbell SB	.33	.42†	.45	.34†
Average	.34		.53	

† Suspect Data not Included in the Reported Average

Figure 3.10 Campbell NB – 3 – G1&G2

Figure 3.11 Campbell NB – 3 – G4&G5

Figure 3.12 Campbell SB – 3 – G1&G2

Figure 3.13 Campbell SB – 3 – G4&G5

Figures 3.14 and 3.15 show the shear distribution factor for Campbell North Bound, and Figures 3.16 and 3.17 show the shear distribution factor for Campbell South Bound. The worst loading condition for shear distribution always occurred when the load vehicle drove directly over the top of the girder regardless whether or not the girder was an interior or exterior girder. Table 3.4 shows a summary of the shear distribution factors found from Campbell NB and SB bridges.

Table 3.4 Distribution Factor for Shear for Bridges at Campbell Creek:

	Interior Girders		Exterior Girders	
	G2	G4	G1	G5
Campbell NB	.50	.40	.73	.71
Campbell SB	.46	.65	.79	.62
Average	.50		.71

Figure 3.14 Campbell NB - 1 - G1&G2

Figure 3.15 Campbell NB - 1 - G4&G5

Figure 3.16 Campbell SB - 1 - G1&G2

Figure 3.17 Campbell SB – 1 – G4&G5

3.3.3 Data from Twin Bridges Located at Huffman

Figures 3.18 and 3.19 show the moment distribution factor for Huffman North Bound, and Figures 3.20 and 3.21 show the moment distribution factor for Huffman South Bound. The twin bridges at Huffman were not loaded directly over the first interior girder (G2 or G4) during the field testing. The following graphs show the distribution from the exterior girders and from loads placed over either of the center girders (G3 or G4). Huffman NB was not tested with a load placed directly over G4, therefore Figure 3.19 only shows the distribution from the load placed over the exterior girder G6. The reported distribution factor for interior girders is the greater of either the distribution factor of the second girder (G2, G5) when the load was placed on the exterior girder, or

the distribution from one of the interior girders (G3,G4) when the load was placed directly over the top of that girder. This may not be the largest distribution factor for interior girders since the worst loading condition for the interior girder may not have been tested. Table 3.5 shows a summary of the distribution factors for moment of the twin bridges located at Huffman ave.

Table 3.5 Distribution Factor for Moment for Bridges at Huffman:

	Interior Girders		Exterior Girders	
	G2	G5	G1	G6
Campbell NB	.34	.30	.46	.49
Campbell SB	.33†	.28	.51	.49
Average	.31		.49	

† Distribution Factor Found from Girder Number 3 when the load was placed directly over the girder.

Figure 3.18 Huffman NB – 3 – G1&G3

Figure 3.19 Huffman NB - 3 - G6

Figure 3.20 Huffman SB - 3 - G1&G3

Figure 3.21 Huffman SB – 3 – G4&G6

Figures 3.22 and 3.23 show the shear distribution factor for Huffman North Bound, and Figures 3.24 and 3.25 show the shear distribution factor for Huffman South Bound. In the previous sets of bridges tested, the worst loading condition for shear distribution typically occurred when the load vehicle drove directly over the top of the girder regardless whether or not the girder was an interior or exterior girder. Since the second girder of Huffman was not loaded, the reported distribution factor for interior girders for shear is either from girder three or girder four when the load was placed directly over the respective girder.

The twin bridges at Huffman, have a skew angle of 27.5° . The shear distribution of exterior girders on a skew bridge is affected by whether the girder is located on the obtuse or acute corner of the bridge. The girder numbering system is not symmetric for the Huffman north bound and south bound bridges. For Huffman NB, the shear loading position over the G1 girder is in the acute corner, and the shear loading position over the G6 girder is in the obtuse corner. For Huffman SB, the shear loading position over the G1 girder is in the obtuse corner, and the shear loading position over the G6 girder is in the acute corner. Table 3.6 shows a summary of the shear distribution factors found from Huffman NB and SB bridges. The distribution factors for the exterior girders are separated based off the obtuse and acute corner.

Table 3.6 Distribution Factor for Shear for Bridges at Huffman:

	Interior Girders		Exterior Girders	
	G3	G4	Acute	Obtuse
Campbell NB	.48		.64	.61
Campbell SB	.45	.45	.47	.54
Average	.46		.55	.58

Figure 3.22 Huffman NB – 1 – G1&G3

Figure 3.23 Huffman NB – 1 – G6

Figure 3.24 Huffman SB – 1 – G1&G3

Figure 3.25 Huffman SB – 1 – G4&G6

3.3.4 Data from Twin Bridges Located at Diamond and Dowling

Figures 3.26 and 3.27 show the moment distribution factors for Dowling, and Figures 3.28 and 3.29 show the moment distribution factors for Diamond. Both of the bridges at Diamond and Dowling intersections were wide bridges consisting of 14 girders. Only one side of each bridge was tested and moment gauges were only placed on girders 1 to 8, 10, and 11 on Dowling and 1 to 8, 10, and 12 on Diamond. To determine the distribution factor based off the method from section 3.2, the strains on girder number 9 for both bridges and girder 11 for Diamond was found by interpolating between the two surrounding girders. Girders 13 and 14 were assumed to have no strain. The moment gauge placed on G3 for diamond was not located in the center of bottom of girder due to excessive cracking. The gauge was offset from the center by one inch to an area where it could adhere to uncracked concrete. This offset could have caused the strains to be larger than if the gauge were centered on the girder due to the torsional forces in the girder. Both of these bridges are older than the other tested bridges and have been damaged due to over sized vehicles colliding into the exterior girders. While the bridges are both still serviceable, the strains recorded from the testing of these bridges are more sporadic than the other tested bridges. The moment on interior girders for these two bridges is an average of all the distribution factors found on loaded interior girders. Table 3.7 summarizes the moment distribution factors from Diamond and Dowling Bridges.

Table 3.7 Distribution Factor for Moment for Diamond and Dowling:

	Interior Girders						Exterior Girders	
	G2	G3	G4	G6	G7	Ave.	G1	Ave.
Dowling	.19	.34	.46	.21	.37	.32	.49	.46
Diamond	.27	.43	.31	.26	.31		.42

Figure 3.26 Dowling – 3 – G1, G2, G3

Figure 3.27 Dowling – 3 – G4, G6, G7

Figure 3.28 Diamond - 3 - G1, G2, G3

Figure 3.29 Diamond - 3 - G4, G6, G7

During the testing of Diamond and Dowling bridges, the research team did not have enough shear strain gauges to place on both sides of the girder. Without strain readings from both sides of the girder, there is no way to factor out the torsional forces from the shear forces to accurately determine the shear distribution. The shear strain data for these two bridges were used to calibrate a 3D Finite Element model from which we could determine the shear distribution factor from Diamond and Dowling bridges. Section 4.7.4 shows how the 3D Finite Element model compares with the strain data.

3.4 Summary of Findings from all Tested Bridges

The distribution factors calculated from the data obtained by testing the four sets of twin bridges in the Anchorage area are significantly less than the distribution factors found by using the AASHTO LRFD simplified equations (Eqns. 1.7 to 1.10). Table 3.1 and 3.2 show this disparity for the four sets of twin bridges tested. The “% Greater” column shows how much greater the LRFD distribution factor is greater than the distribution factor found from experimental data found by method shown in equation 3.4.

$$\% \text{Greater} = \left(\frac{DF_{\text{LRFD}} - DF_{\text{exp}}}{DF_{\text{exp}}} \right) \quad (3.4)$$

Table 3.8 Distribution Factor for Moment:

	Interior Girders			Exterior Girders		
	Data	LRFD	% Greater	Data	LRFD	% Greater
100 th	.35	.66	89%	.45	.76	69%
Campbell	.34	.66	94%	.53	.76	43%
Huffman	.31	.55	77%	.49	.61	24%
Diamond/Dowling	.32	1.48	363%	.46	.77	67%
	Average %		156%	Average %		51%

Table 3.9 Distribution Factor for Shear:

	Interior Girders			Exterior Girders		
	Data	LRFD	% Greater	Data	LRFD	% Greater
100 th	.43	.60	40%	.66	.76	15%
Campbell	.50	.60	20%	.71	.76	7%
Huffman	.46	.62*	35%	.58	.74*	28%
Diamond/Dowling	.55†	.60	9%	.60†	.77	28%
	Average %		26%	Average %		20%

* Indicates adjusted with skew adjustment factor.

† Indicates value not found directly from data but from a FE model that closely approximates the data.

When comparing the LRFD equations to the experimental data from the eight tested bridges, the distribution factors found from the LRFD equation proved to be an average of 156% greater for moment on interior girders, 51% greater for moment on exterior girders, 26% greater for shear on interior girders, and 20% greater for shear on exterior girders. Appendix B shows the calculations to determine the distribution factor for the four sets of bridges using the LRFD simplified equations.

Equations (1.7 – 1.10) describe the moment distribution on interior girders while the simplified equation governing the rest of the distribution factors is the lever rule. The reason the LRFD equation for moment on interior girders is so much higher than the experimental data is due to comparison of the LRFD equations to the data found from the twin bridges located at Diamond and Dowling. The LRFD equation for these bridges is 363% larger than the experimental values. One of the reasons the LRFD simplified equations are so much larger than the experimental values is that the equations (1.7 through 1.10) are affected by the overall width of the bridge. Diamond and Dowling

bridges are both very wide bridges and the results of the parametric study in chapter 5 show that the distribution factor is not as affected by the overall width of the bridge as the LRFD equations predict. The lever rule which governs the rest of the simplified equation is not based on research of the bulb-tee bridge system, but is a default method for determining the live load distribution of girders. The lever rule is the simple distribution of the vehicle wheel load onto the girders. The lever rule assumes the girders are infinitely stiff and the distribution is only affected by the girder spacing. Since the bridge girders are much stiffer when loaded to produce the maximum shear forces than when loaded for moment, the lever rule predicts the shear distribution better than the moment distribution on exterior girders.

4 Modeling Analysis

4.1 Introduction

In understanding the behavior of the decked bulb-tee bridges, one of the crucial elements is to find a rigorous mathematical model of the bridge that closely matches experimental data. Of the eight different bridges tested, the pair located at the intersection of 100th Ave. and Minnesota are the most standard in that they have no skew, have an average span length and width, and are built with typical decked bulb-tee girders. These two bridges are also the two bridges which when tested, had the most consistent data. The data from these two bridges will be used to evaluate the different models and modeling techniques. These bridges are 114ft long with no skew and have the cross-section shown in Figure 4.1.

Figure 4.1 Cross-section of 100th Ave Bridge

The deck portion of each girder tapers from 10in near the web to 6in near the joint. Each longitudinal joint has been grouted with 4in shear keys placed every four feet. The first modeling method used to evaluate the bridge behavior is the grillage model. The grillage model assumes the bridge is a mesh of frame elements. The mesh consists of longitudinal beams and transverse beams. The longitudinal beams represent the stiffness

of the bridge in the longitudinal direction. The stiffness of the longitudinal beams is governed by the spacing of the beams and the width of the bridge they represent. The transverse beams are perpendicular to the longitudinal beams and rigidly connected to the longitudinal beams. The stiffness of the transverse beams is governed by the width of the bridge deck they represent. By reducing the bridge system from a three dimensional monolithic structure into a mesh of interlocking perpendicular one dimensional frame elements, the bridge behavior can be analyzed using simple frame analysis. The grillage analogy has proven to be an accurate method for describing bridge behavior.

4.2 Longitudinal Joint

The decked bulb-tee bridges are unique in that they have a longitudinal joint between girders. One of the major focuses in this research is on how this longitudinal joint behaves and how it can best be modeled. The transverse beams in the grillage model are represented by a solid rectangular section that varies in depth from 10" at the web to 6" at the joint and is 3.8 ft wide. The behavior of the connections of the transverse rigid beam elements approximate the behavior of the longitudinal joint between bridge girders. All of the grillage models in this thesis show two extreme conditions of the connection between transverse beam elements. The first condition is called the "rigid" condition in which the connection between beam elements is fully fixed and has full transverse continuity. The other condition is called the "hinged" condition in which the joint is flexurally released in transverse direction. The longitudinal joints between bridge girders behave somewhere between these two extreme conditions.

This section explores two other methods which may approximate the behavior of the joint. The first method models the joint as if it were a small rectangular grout section. The model of this grout section consists of connecting the transverse beams with a small one inch long rectangular member with a depth of three inches and width of 3.8ft. The modulus of elasticity of this grout section is approximated at 2500ksi which is roughly half of the modulus of elasticity used to model the girder section. The distribution factor from this reduced section is almost identical to the purely rigid connection. The maximum tensile stress in the small 3 inch grouted section was 2600psi.

Since the forces generated in the grouted section could easily crack the grout, the second method for approximating the joint uses a one inch long beam that has the same properties of a cracked grout section. The moment of inertia of the cracked grout section was determined by using cracked section analysis methods used in reinforced concrete design. The cracked section was assumed to have a width of 3.8ft and a 4" x 1/4" shear plate at the bottom of the cracked section providing the tensile strength. The calculations for determining these cracked section properties are presented in Appendix D. Even the reduced stiffness of the cracked grouted section shows distribution factors very close to the purely rigid connection. Figure 4.2 is a graph which compares all of these models to the experimental data.

Figure 4.2 Variation of Transverse Stiffness in the Longitudinal Joint

4.3 Torsional Constant

The grillage model used to model the twin bridges at 100th ave. has 5 longitudinal beams representing the 5 girders of the bridge. Each beam has the same moment of inertia as the decked bulb-tee girders they represent. Saint-Venant's torsional stiffness constant of the longitudinal beams was approximated using the current method described in the AASHTO LRFD for stocky open sections:

$$J = \frac{A^4}{40 \cdot I_p} \quad (4.1)$$

Where "A" is the area of the girder and I_p is the polar moment of inertia. Other methods of determining "J" such as the standard grillage approximation of adding the horizontal

and vertical moments of inertia together were compared to equation (4.1), yet equation (4.1) produced results that most closely matched the data. Figures 4.3 and 4.4 show a comparison of different torsional rigidity constants for the moment distribution factors for both the hinged and rigid conditions.

Figure 4.3 Evaluation of Torsional Constant in Hinged Grillage Models

Figure 4.4 Evaluation of Torsional Constant in Rigid Grillage Models

4.4 Mesh Density

The transverse stiffness of the bridge deck is approximated by 30 beam lines each separated by 3.8 ft. Different mesh densities were compared, and it was found that increasing the density had little impact on the distribution factor. Following is a graph which compares the moment distribution factor found from three different mesh densities. The first density is 5 longitudinal frame elements and 30 transverse frame elements (5-30). The second density is 15 longitudinal frame elements with 60 transverse frame elements (15-60). The third mesh density is 30 longitudinal frame elements with 60 transverse frame elements (30-60). To increase the mesh in the transverse direction, the frame elements had the same properties as in the coarser mesh except with half the

width. To increase the mesh density in the longitudinal direction, the cross-section of the bridge girders were subdivided into 3 elements for the 15-60 model and into 6 elements for the 30-60. The elements which subdivided the bridge girder had different properties. The middle element had stiffness of the middle portion of the bridge girder and the torsional stiffness found from equation 4.1. The other longitudinal frame elements were modeled as a rectangular section with a height equal to the average height of the tapered flange across the section. The comparison shows that there is little variation in the distribution factor found from the three different models therefore it is sufficient to use the simplest model of 5 longitudinal girders and 30 transverse girders. This density provides slightly more conservative results than the other two finer densities.

Figure 4.5 Evaluation of Mesh Density in Grillage Models

4.5 Effects of the Intermediate Steel Diaphragms (ISD)

Another factor that will affect the behavior of the longitudinal joint is the intermediate steel diaphragms. The exaggerated deflection of the hinged model is shown in Figure 4.6 where each girder rotates slightly at each hinged joint. It appears that any connection between two girders at the bottom flange would help to resist this rotation about the longitudinal joint as shown in Figure 4.7.

Figure 4.6 Exaggerated Deflection of the Hinged Model

Figure 4.7 Intermediate Steel Diaphragms help to resist rotation

In the grillage model, the only restraint the girder has to rotating about the hinge joint is the torsional rigidity of the girder. If the joint behaves as a perfect hinge, the grillage model cannot approximate the effects of intermediate steel diaphragms on preventing the girder from rotating about the hinged joint. The 3D Finite Element (FE) models

developed in Reference 7 for the bridge located at 100th ave. and Minnesota better approximate the affects of the intermediate steel diaphragms. Four different 3D models were developed. One model approximates the longitudinal joint as a hinged joint with perfect transverse flexural release across the longitudinal joint and no elements representing the steel diaphragms in the bridge. The other condition is exactly the same except truss elements were placed between the girders at the same location as the steel diaphragms on the tested bridge. These truss elements have the same stiffness and orientation as the K type diaphragms used on the bridge. Another two models have the same properties as the two models previously described except the longitudinal joint is modeled as having full transverse flexural rigidity. The results from these models show that the hinged FE model without the intermediate steel diaphragms has very close distribution factors as the hinged grillage model. The introduction of the steel diaphragms into the hinged model significantly reduces the distribution factor of the bridge and it has almost the same distribution factors as the FE models with full transverse flexural rigidity and the grillage model that has full transverse flexural rigidity. For all of the bridges, this later set of models better approximates the experimental data both for shear forces and for moment forces.

Figures 4.8 and 4.9 compare the envelope of experimentally found distribution factors for the bridge located at 100th ave to the grillage and FE models. These graphs show that the grillage and FE hinged models do not compare well with the data and have a

distinctly different behavior than do the models that are rigidly connected along the longitudinal joint and/or have intermediate steel diaphragms.

Figure 4.8 Moment DF comparison With/Without ISD DF for all girders.

Figure 4.9 Shear DF comparison With/Without ISD DF for all girders.

Another method used to compare the experimental data to the grillage and 3D FE models is to estimate the midspan moments of the girders from the strains measured at the bridge's midspan and from strains in the 3D finite element model to the moments found from the grillage model. Since the grillage model is comprised of one dimensional beam elements, only member forces can be found from the model. To relate these forces to the experimentally measured strains, and the strains from the 3D finite element model, simple mechanics of materials must be used to estimate the moments the bridge experiences from the measured strains. The following equation is used to make this relationship.

$$M = \frac{E \cdot \epsilon \cdot I}{c} \quad (4.2)$$

Where M is the midspan moment, E is Young's Modulus for the concrete, I is the moment of inertia of the girder, c is the distance from the bottom surface of the girder to the neutral axis, and ϵ is the measured strain. While there are many methods for determining the values of E , I , and c , these are only approximate methods and there is a margin of error for determining each of these values. When calculating E using the method described in the ACI Code, the actual value can vary from 80% to 120% of the calculated value (6). Another source of error in determining these values could come from the wearing surface contributing to the stiffness of the girder and varying the location of the neutral axis. Small variations in these three variables have a significant affect when trying to compare the measured strains to the output of the grillage models and the FE Models. The theoretical values of these three variables based off the simple geometry of the girder and the design strength of the concrete are as follows: $E = 4645$ ksi, $I = 3623345 \text{ in}^4$, and $c = 38.13 \text{ in}$. When solving for the forces in the girders using these values, the sum of the moments in all the girders is 2320 ft-kip. This moment could only be produced if the vehicle weighed 89.3 kips. Since the total weight of our load vehicle was 72.6 kips, it is clear that the true modulus of elasticity and moment of inertia must be less than approximated, or the neutral axis higher than approximated. It is also likely that the weight of the load vehicle was greater than 72.6 kips which is the measured weight of the vehicle at the beginning of the bridge testing. To better evaluate the data so that it would compare with our models, the modulus of elasticity was determined by holding the other variables constant and setting M of equation (4.1) equal

to the total moment of all the girders across the midspan of the bridge (1882 ft-kips). This is the total moment the bridge would experience if it were subjected to a load of 72.6 kips. The new modulus of elasticity was found to be 3769 ksi. This value is 81% of the value obtained using the equation in the ACI code and is within the acceptable range of deviation. While the true modulus of elasticity may not be this exact value, by using this value of E to compute the midspan moments, the experimental data can be compared to the math models.

Figure 4.10 shows the midspan moments of each of the five bridge girders when the load is positioned directly over the middle girder. The experimental moments are derived from the average of four different strain values taken from two different loading runs over two different twin bridges (100th NB & 100th SB). For all of the following graphs, the dotted lines represent the grillage models, the dashed lines represent the 3D FE models, and the

solid lines represent values found from experimental data.

Figure 4.10 3D FE Model With and Without ISD Load over Girder 3

In Figure 4.10, the Grillage Hinged Model and the 3D FE Hinged model without any intermediate steel diaphragms show a much higher midspan moment under girder three than all the other models and the average from the experimental data. This same trend can be seen in Figure 4.11 when the load is placed over the second girder.

Figure 4.11 3D FE Model With and Without ISD Load over Girder 2

Figure 4.12 3D FE Model With and Without ISD Load over Girder 1

When the Load is placed on the exterior girder as shown in Figure 4.12, all the models appear to behave the same and there is no obvious affect of releasing the transverse flexural rigidity. The bridge system also shows little variation between the transverse hinged and rigid conditions when deriving the distribution factor for the exterior girders.

Later in this Chapter, the two methods shown in this section will be used to compare the hinged and rigid models to data from other bridges. The data from the Huffman and Campbell bridges also compare better to a rigid grillage model or a FE hinged model that has intermediate steel diaphragms.

4.6 Boundary Conditions

Due to the concrete diaphragms at the ends of the bridge and the eccentricity of the rollers, the actual support conditions will be somewhere between a fixed and pinned condition. The actual support conditions are impossible to measure so Figure 4.14 compares the distribution factor from models with both pinned and fixed supports to the data. The data is much closer to the pinned conditions than the fixed condition. The total moment across all of the girders was calculated to get an idea of the behavior of the support conditions, yet this showed the total moments to be greater than that of a pinned condition. This is due to other unknown variable such as the modulus of elasticity and depth of the neutral axis which will affect the calculation of moments in the girders from the measured strains. Since the distribution of the data more closely matches the pinned

support condition, the pinned support condition are used in the remainder of the models.

To approximate the concrete end diaphragms, the support conditions are pinned yet torsionally fixed. Figure 4.13 shows the beam layout of the grillage model.

Figure 4.13 Schematic of Diagram of Grillage Model

Figure 4.14 Variation of Model Support Conditions

4.7 Model Comparison to Tested Twin Bridges

4.7.1 100th Ave and Minnesota

This section compares the distribution factors found from experimental data to the DF factors found from the grillage model and the AASHTO LRFD simplified equations. The multiple presence factor has been factored out of the AASHTO LRFD results to compare them with the grillage model and experimental data. The method of calculating the AASHTO LRFD distribution factors for the twin bridges located at 100th Ave and Minnesota is presented in Appendix B. Figures 4.15 and 4.16 show the moment distribution factor for 100th NB and 100th SB. Figure 4.17 shows the average distribution factors of both 100th SB & NB and compares it to the hinged and rigid grillage models.

Figure 4.15 Moment Distribution Factors (100th NB Bridge)

Figure 4.16 Moment Distribution Factors (100th SB Bridge)

Figure 4.17 Moment Distribution Factors (100th NB&SB Bridge)

Figure 4.18 and 4.19 Show the Shear distribution factors for 100th NB & SB Bridges.

Figure 4.20 shows the average shear distribution factors of both 100th SB & NB and compares it to the hinged and rigid grillage models.

Figure 4.18 Shear Distribution Factors (100th NB Bridge)

Figure 4.19 Shear Distribution Factors (100th SB Bridge)

Figure 4.20 Shear Distribution Factors (100th NB&SB Bridge)

4.7.2 Analysis of twin bridges at Campbell Creek

We did not get as consistent of data from the Campbell Creek Bridges as we did from the bridges crossing 100th ave. The Campbell Creek Bridges are long with deep girders.

They have a 4 degree skew angle which is small enough that it hardly affects the bridge behavior. Since these bridges were the first pair of bridges tested of the eight, we had many errors attributed to developing a procedure for setting up the bridges for testing.

For these twin bridges and all subsequent sets of bridges, there is only one 3D FE model

per bridge. The FE model has intermediate steel diaphragms and models the longitudinal joint as a pure hinge.

I will compare the data to the grillage and FE models using the same two methods used to evaluate the data in the 100th Ave. Bridge. The first method compares the derived moments from the strains to the forces found in the beam elements of the grillage model. The second method compares the distribution factors from the data to the models and to LRFD simplified equations. The theoretical values of the mechanical properties of the girders are as follows: $E = 4645 \text{ ksi}$, $I = 633162 \text{ in}^4$, and $c = 45.5 \text{ in}$. When these values are used with the experimental data, the total moment of all girders averages to 1736 ft-kips. This value is less than 1876 ft-kips which is the theoretical moment of placing the load vehicle at midspan on a simply supported beam 139ft long. To standardize the moments found from experimental strains, I found an E value of 5020 ksi which is 108% of the theoretical E value. The Figure 4.21 shows the midspan moments of each of the five bridge girders when the load is positioned directly over the middle girder. The experimental moments are derived from the average of four different strain values taken from two different loading runs over two different twin bridges (Campbell NB & Campbell SB). Figures 4.22, and 4.23 compare the moment distribution of Campbell NB and SB to the distribution factors found from AASHTO LRFD simplified equations. In Figure 4.23 the strain gauge located at Campbell SB - M1G5 did not adhere properly to the concrete and is therefore suspect, causing the distribution factor of Campbell SB – 3- G4&G5 to be off. Figure 4.24 shows the distribution factor average

envelope between the two bridges and compares it to both the grillage and FE models.

The experimental data used in to generate Figure 4.24 does not include the data from the suspect gauge on Campbell SB – M1G5. Figures 4.25 and 4.26 show the shear distribution of Campbell NB and SB and Figure 4.27 shows the distribution factor average envelope between the two bridges comparing it to both the grillage and FE models.

Figure 4.21 Comparison of Flexural Strains to Math Models (Campbell NB&SB)

Figure 4.22 Moment Distribution Factors (Campbell NB Bridge)

Figure 4.23 Moment Distribution Factors (Campbell SB Bridge)

Figure 4.24 Moment Distribution Factors (Campbell NB&SB Bridge)

Figure 4.25 Shear Distribution Factors (Campbell NB Bridge)

Figure 4.26 Shear Distribution Factors (Campbell SB Bridge)

Figure 4.27 Shear Distribution Factors (Campbell NB&SB Bridge)

4.7.3 Analysis of twin bridges at Huffman Intersection

The Huffman twin bridges differ from the Campbell and 100th ave. bridges in that they have a 27.5° skew. I will use the same two methods to compare the math models of this bridge to the experimental data as was done on the Campbell bridges and 100th ave bridges. When comparing the deriving the moments for the Huffman twin bridges, I found the approximate modulus of elasticity to be 3800 ksi. This is 82% of 4645 ksi which is the approximation of the modulus of the concrete using the ACI simplified equation. Figure 4.28 shows the midspan moments of each of the six bridge girders when the load is positioned over the third girder. The experimental moments are derived from the average of four different strain values taken from two different loading runs over two different twin bridges (Huffman NB & Huffman SB).

Figure 4.28 Comparison of Flexural Strains to Math Models (Huffman NB&SB)

Figure 4.28 shows that the experimental data behaves similarly to either the rigid grillage model or the FE hinged model with intermediate steel diaphragms.

Figures 4.29 and 4.30 show the moment distribution of Huffman NB and SB. The load condition noted as Center represents when the load vehicle was evenly placed on both girder's 3 and 4. Figure 4.31 shows the distribution factor average envelope between the two bridges and compares it to both the grillage and FE models. Both the grillage and the FE models show a lower distribution factor for the exterior girders than found from experimentation. Currently, we do not know why this inconsistency exists. The data from the loads placed on the exterior girders of the twin Huffman bridges appears to be very consistent, yet it does not match the values from the math models. However, the distribution on interior girders is consistent. Figures 4.32 and 4.33 show the shear distribution of Huffman NB and SB. The LRFD Values are shown with a skew correction factor from AASHTO LRFD Table 4.6.2.2.3c-1. The calculations for the distribution factor for Huffman are presented in Appendix B.

Figure 4.29 Moment Distribution Factors (Huffman NB Bridge)

Figure 4.30 Moment Distribution Factors (Huffman SB Bridge)

Figure 4.31 Moment Distribution Factors (Huffman NB&SB Bridge)

Figure 4.32 Shear Distribution Factors (Huffman NB Bridge)

Figure 4.33 Shear Distribution Factors (Huffman SB Bridge)

Figure 4.34 shows the distribution factor average envelope between the two bridges and compares it to both the grillage and FE models. For the case of modeling shear distribution on skew bridges, the three dimensional finite element model is a much better math model than the grillage model. When the load vehicle is placed into the acute corner of the bridge, the rear inside wheel load in a grillage model is very close to the supports of the neighboring girder. Because the supports in the grillage model are infinitely stiff, and the shear distribution in the grillage model is found from the reactions of the supports, the grillage model does not distribute the inside rear wheel load realistically. This can be seen in Figure 4.34.

Figure 4.34 Shear Distribution Factors (Huffman NB&SB Bridge)

4.7.4 Analysis of twin bridges at Diamond and Dowling Intersection

Figures 4.35 and 4.36 show the moment distribution of Diamond and Dowling. The LRFD equations give a distribution factor for the interior girder greater than one. While this kind of distribution factor may be possible for a multi lane loaded condition, it is not possible in a single lane loaded condition. The calculations for the LRFD DF are presented in Appendix B. On Diamond Bridge, the gauges located at G3M1 and G7M1 both were not centered at the bottom of the girder. These gauges were shifted left due to excessive cracking in the center of the girder. On Diamond There were no gauges located at positions G9M1 and G11M1. The strains used to determine the distribution

factor for the other data points were found by interpolating between the strains recorded from the gauges located on either side of the position. The strain on gauges 13 and 14 were assumed to be zero.

Figure 4.35 Moment Distribution Factors (Diamond Bridge)

On Dowling Bridge, there was no gauge located at position G9M1. The strain values were interpolated in the same manner as for Diamond Bridge to determine the distribution factor of the remainder of the data points. The strain on gauges 12, 13, and 14 were assumed to be zero.

Figure 4.36 Moment Distribution Factors (Dowling Bridge)

Figure 4.37 only shows the distribution factor of a loaded girder compared with the grillage and FE models. These values have been averaged between Dowling and Diamond.

Figure 4.37 Moment Distribution Factors (Diamond & Dowling Bridges)

Due to the limited number of gauges, the research team during field testing was only able to place shear gauges on one side of girder for the Dowling and Diamond Bridges.

Figures 4.38 through 4.43 show the strain distributions of just one side of each of the girders. Since this distribution does not represent the shear distribution of the girder due to the torsional effects, only the distribution factors for single loading conditions are presented and can only be compared with values derived from the 3D Finite element model (7).

Figure 4.38 Shear Distribution for Load over G1 (Diamond & Dowling Bridges)

Figure 4.39 Shear Distribution for Load over G2 (Diamond & Dowling Bridges)

Figure 4.40 Shear Distribution for Load over G3 (Diamond & Dowling Bridges)

Figure 4.41 Shear Distribution for Load over G4 (Diamond & Dowling Bridges)

Figure 4.42 Shear Distribution for Load over G6 (Diamond & Dowling Bridges)

Figure 4.43 Shear Distribution for Load over G7 (Diamond & Dowling Bridges)

In all of the above cases, the FE model has excellent agreement with the experimental data, therefore we can assume that the shear distribution factor derived from the FE model is comparable to that of the two twin bridges. Figure 4.44 is a graph which shows how the shear distribution from the FE model compares to the LRFD values.

Figure 4.44 Shear Distribution Factors (Diamond and Dowling Models)

5 Parametric Study

5.1 Introduction

Based on findings from modeling the experimentally tested bridges, a grillage model was developed to conduct a parametric study of the decked bulb-tee girder system. The objective of this parametric study was not to understand general bridge behavior but to develop simplified equations that accurately relate to realistic bridges. Two sets of data bases of different grillage models were constructed which vary different bridge parameters. The scope of these data bases were reduced so that they would only include models of spans, girder heights, and girder spacing which are typical of practical decked bulb-tee girder bridges. The current LRFD simplified equations were evaluated using the data base of typical models. In chapter 7 this data base is used to develop and evaluate new simplified DF equations.

5.2 Parametric Study Modeling Assumptions

The programs used to run the grillage models were RISA 3D and SAP 2000 Version 8. The mesh density consisted of one frame element per girder representing the longitudinal stiffness of the bridge and the transverse stiffness was modeled with a frame element positioned every 2.5ft. The end restraints were fixed in all translational degrees of freedom. The bending rotation was released yet the torsion and out of plane rotation were fixed. The transverse beam sections were modeled as a non-prismatic section which

was rectangular with a constant width of 30 inches. The Saint-Venant's torsional stiffness constant of the longitudinal beams was approximated using equation 4.1.

Two different sets of models in the parametric study were used. The first set (Set I) comprised of girder cross-sections similar to the Washington style decked bulb-tee girder except the depth of the girder was increased by one inch to have the same depth as the Alaskan style girder. The flange of each girder varies uniformly from the web to the edge of the girder to simplify the modeling of the non-prismatic transverse beam element. In the first set of models, the girder height (which is related to the stiffness and torsional rigidity), girder spacing, bridge length, and number of girders were all varied. This set modeled three different girder heights: 42", 54", and 66". It varied 4 different girder spacings: 48.5", 68.5", 88.5", and 108.5". It varied the bridge length from 60 to 160 feet at 20 foot increments. Finally It varied the number of girders from 4 to 7. Figure 5.1 shows the different girder cross-sections used in this study. Set I consists of a total of 288 models.

Figure 5.1 Typical Cross-sections of the girders in Set I.

All of the properties for the longitudinal elements for Set I were derived from the cross-sections shown in Figure 5.1.

The second set of models (Set II) is similar to the first with a few refinements. The number of girders were not varied in Set II because the data from Set I shows that the number of girders had little impact on the exterior and interior distribution factors. All the models in set II have only four longitudinal girders. The girder cross-section used in Set II was modeled after the Alaskan Style Decked Bulb-Tee girder cross section. This girder cross-section has a tapered flange which tapers 2 feet from the center of the girder then decreases by one inch to the standard deck thickness for the remaining width of the girder. The transverse beam elements have the same non prismatic rectangular properties in that for a 6" deck the beams height decreases from 11" to 7" over a two foot section then changes to a uniform 6" deep section to the longitudinal joint between girders. Set II consists of models with four different girder heights: 36", 42", 54", and 66" all relative to a 6" deck. Set II has five different girder spacings of 48.5", 60.5", 72.5", 84.5", and 96.5". The deck thickness was varied in set II as well which in turn affected the overall height of the girder. The three different deck thicknesses modeled in set II were 4", 6", and 8". The span length of the models varied from 40 feet to 180 feet at 20 foot increments. Finally, the rigidity of the longitudinal joint was varied in the second set of models. One condition modeled the longitudinal joint as a perfect hinge while the other condition modeled the joint between the girders as a rigid connection. Figure 5.2 shows a

typical cross-section of a girder from which all the beam properties used in the grillage model were derived. Set II consists of a total of 960 models.

Figure 5.2 Typical Cross-section used in Set II

The bridges in the parametric study were all loaded with typical HS20 Truck Loading with axle spacing at 14 feet apart. For moment distribution, the HS20 was positioned on the bridge with the centerline of the bridge halfway between the center of gravity of the truck and the middle axle. For shear distribution, the load vehicle was placed with the rear axle 42 inches from the end of the bridge irrespective of the girder depth in set I, and a distance H equal to the girder depth away from the end of the bridge in set II. In the transverse direction, the exterior girder was loaded with the vehicle's outside wheels placed two feet from the edge of the bridge. The models assume there is no curb on the bridge. The interior girders were loaded with the vehicle centered over the centerline of the girder. Figure 5.3 shows the typical transverse loading of the models in the parametric study.

Figure 5.3 Typical Transverse Loading used in Parametric Study

In some cases, the worst loading condition for the shear interior girder was not always with the vehicle loaded directly centered over the top of the girder, especially for small girder spacing. For these conditions, the load placed two feet away from the edge of the bridge the same as the worst loading condition for the exterior girder was the worst loading condition for the second girder.

5.3 Parametric Study Comparison to LRFD Simplified Equations

This parametric study includes bridges that are unrealistic such as a bridge that has 42" deep girders with a spacing of 108.5" and a length of 160'. Such a bridge could not be designed to carry any kind of truck load. To refine the parametric study so that it consists of only practical bridges the "Standard Decked Bulb Tee Charts" design aids developed by Concrete Technology Corporation in Tacoma Washington were used to reduce the number of bridges in the parametric study data base (11). Table 5.1 lists the span lengths of realistic bridges in set I and Table 5.2 lists the span lengths of realistic bridges in set II.

Table 5.1 Span Lengths (in feet) of Realistic Bridges in Set I
Girder Height (inches)

Girder Spacing (inches)	42	54	66
48.5	40-120	60-140	100-180
68.5	40-120	60-140	80-180
88.5	40-100	40-140	80-160
108.5	40-100	40-120	80-140

Table 5.2 Span Lengths (in feet) of Realistic Bridges in Set II
Girder Height (inches)

Girder Spacing (inches)	36	42	54	66
48.5	40-100	40-120	60-140	100-180
60.5	40-100	40-120	60-140	80-180
72.5	40-80	40-100	40-140	80-160
84.5	40-80	40-100	40-120	80-140
96.5	40-60	40-80	40-120	60-140

Using this refined database of realistic bridges, the results of the two sets of models were compared to the simplified equations presented in the ASHTO LRFD. The models were compared to the results of both the equations provided for bridges which are connected sufficiently to act as a unit, and for bridges which are connected only enough to prevent relative vertical displacement at the girder interface. Figures 5.4 and 5.5 summarize the comparison between the refined math models from Set I and the LRFD Simplified equations. Figure 5.6 compares the error between the simplified LRFD equations set aside for bridges connected only enough to prevent relative vertical displacement and the grillage models of set II with flexural release between girders. Figure 5.7 compares the LRFD equations for bridges sufficiently connected to act as a unit to the grillage models

of set II with full transverse flexural continuity. The plots show the error for each girder position and force. The abbreviations SE, SI, ME, and MI stand for Shear DF on Exterior Girder, Shear DF on Interior Girder, Moment DF on Exterior Girder, and Moment DF on Interior girder respectively.

Figure 5.4 Comparison of Simplified Equations for Bridges Connected Sufficiently to Act as a Unit to Models from Set I

Figure 5.5 Comparison of Simplified Equations for Bridges Connected Only Enough to Prevent Relative Vertical Displacement to Models from Set I

Figure 5.6 Comparison of Simplified Equations for Bridges Connected Sufficiently to Act as a Unit to Rigid Models from Set II

Figure 5.7 Comparison of Simplified Equations for Bridges Connected Only Enough to Prevent Relative Vertical Displacement to Hinged Models from Set II

The LRFD simplified equations provided unconservative values when compared to the results from the grillage models in the case of the DF equation for bridges sufficiently connected to act as a unit for moment on interior girders. The maximum unconservative error for this simplified equation was 10% from the data base in set I and 8% from the rigid models of set II. The bridges where this equation is unconservative relative to the grillage model is when the girder is very stiff (girder height 66in) and the spacing is very close (48.5 & 68.5). The S/D formula for moment on the interior girder of bridges with girders connected only enough to prevent relative vertical translation is an average of 104% conservative when compared to the data from set I and 44% conservative when compared to the data from the hinged models of set II. The reason this simplified

equation relates to data from set II better than the data from set I is not because the grillage models were modeled with flexural release between girders, but instead it is due to the data set only being limited to 4 girders restricting the aspect ratio. It will be clear in the sensitivity study that there is little difference between the transverse flexural released and flexural continuous conditions. Another equation that yields unconservative results is the Lever Rule when compared to the shear DF on interior girders of the hinged models from data set II. These unconservative values occur when the bridge has a very thin deck (4") and a very short span (40 ft).

5.4 Parameter Sensitivity Study

The following graphs summarize the contribution of each of the different parameters to the distribution factors for both moment and shear and for the interior and exterior girders. These graphs show the average and maximum percent change caused by a given parameter when all other parameters are held constant. The average and maximum are over the set of "realistic bridges" described earlier. Following is an example of how the percent change is determined for a given set of bridges.

Table 5.3 Example of Sensitivity Study Calculations

Girder Spacing (in)	Moment of Inertia (in ⁴)	# Girders	Bridge Length (ft)	DF Moment on Interior Girder
48.5	191349.1	4	60	0.275592
48.5	191349.1	5	60	0.259065
48.5	191349.1	6	60	0.253743
48.5	191349.1	7	60	0.251796

Table 5.3 shows all of the bridge parameters held constant except for the number of girders which change from 4 to 7. The Percent Change for this particular bridge type would be found by subtracting the smallest DF value (0.251796) from the largest DF value (0.275592), then dividing by the smallest value. The result would then be 8.63% for this data set. This process is then repeated for each different bridge set in the data base. The average of all of these values from set I is shown in figure 5.8. Figures 5.9 and 5.10 show the impact of different parameters on the distribution factor for the models with transverse flexural continuity and discontinuity respectively of data set II. The parameter entitled Hinged vs Rigid refers to the comparison of the models with transverse flexural continuity and discontinuity. These values are the same for both Figures 5.9 and 5.10 as they compare the two sets of bridges to each other.

Figure 5.8 Sensitivity Study of Models from Set I (Average Change)

Figure 5.9 Sensitivity Study of Hinged Models from Set II (Average Change)

Figure 5.10 Sensitivity Study of Rigid Models from Set II (Average Change)

When evaluating the effects the different parameters have on the distribution, the magnitude of the effects differ between the shear distribution and the moment distribution. For the moment DF both interior and exterior girders, Spacing has the greatest effect followed by span length, and then by girder stiffness. For the Shear distribution spacing had the greatest effect followed by girder stiffness followed by span length. In both cases, the number of girders, the deck thickness, and varied hinged or rigid condition had very little effect on the distribution factor. This conflicts with the LRFD equation for Moment on Interior girders which gives the distribution factor as a function of the aspect ratio. For the single lane loaded condition, this parametric study shows that the aspect ratio has very little effect on the moment distribution factor. Only

when the aspect ratio of the bridge is increased because the girder spacing has increased does the simplified DF equation behave similarly to the models. When the aspect ratio is increased due to increasing the number of girders, the simplified equations behaves incorrectly. Most of the change due to increasing the number of girders occurs when the number of girders is increased from 4 to 5. The addition of subsequent girders has little to no affect on any of the distribution factors.

When comparing the effects of the different parameters between bridges with transverse flexural continuity and discontinuity, the effects are virtually identical in all cases except for the distribution of moment on interior girders. The DF of moment on interior girders for bridges with transverse discontinuity are effected by the stiffness, length, and spacing parameters by an average of 5% more than for transverse continuous bridges. When comparing the effect of a bridge having transverse flexural continuity or discontinuity for distribution of shear and moment on exterior girders, there is little to no affect. For the shear and moment distribution on interior girders, the distribution factor is only effected by an average of 12% by the transverse continuity. The deck thickness has very little effect as well by changing the distribution factor by an average 12% or less depending on the case.

5.5 Summary of Findings

Comparing the two parametric studies to the existing LRFD simplified equations produced the following findings for single lane loaded bulb-tee bridges:

- For shear distributions on exterior and interior girders, the LRFD simplified equations are an average of 32% conservative for both the flexural transverse continuous and discontinuous cases.
- For moment distribution on exterior girders the Lever Rule is an average of 50% conservative for both the flexural transverse continuous and discontinuous cases.
- The LRFD equation for distribution of moment on interior girders for bridges with girders only connected enough to prevent relative vertical translation, is an average of 44% to 96% conservative depending on the aspect ratio of the bridge.
- The single lane distribution factor of bulb-tee bridges is only dependant on the aspect ratio of the bridge when the aspect ratio changes due to change in girder spacing not when the aspect ratio changes due to changing the total number of girders.
- The LRFD equation for distribution of moment on interior girders for bridges with girders sufficiently connected to act as a unit is an average of 21% conservative and could possibly be up to 10% unconservative for bridges with a girder spacing of less than 60" and a span length of greater than 100ft.
- In all DF Cases, girder spacing has the greatest impact on the distribution factor.

6 Theoretical Approach to Understanding Bridge DF

Before developing simplified equations which approximate the single lane loaded distribution factors for bulb-tee bridges, I first worked to better understand the behavior of the bridge by using simple beam theory to model the bridges. To better understand what variables affect the distribution factor of a bridge system, I've developed a simplified model that can be solved algebraically that approximates the bridge system:

Figure 6.1 Simplified DF Model

Figure 6.1 shows the worst loading condition over an interior girder of a five girder bridge model. This model assumes each girder is a spring support system and is torsionally fixed meaning the girders are assumed not to rotate at all about their longitudinal axis. When the girder is assumed torsionally fixed, the nodes at each of the spring reaction locations are assumed not to rotate at all but only translate vertically. The reaction at each of the girders is simply the displacement of the girder times its spring constant. The force of the load is distributed to the other springs by a one dimensional

beam. By resolving unit wheel loads (WL) via the lever rule into respective loads over each neighboring girder, we can solve for the reaction of the girder directly under the loads and determine the distribution factor of that girder. To determine the spring constant of each girder, we assume the displacement of the girder as a function of force is modeled as a point force loaded on a beam supported by two hinge joints shown in Figure 6.2.

Figure 6.2 Simply Supported Span

From the study of mechanics of materials and from Hook's Law, the governing equation which relates deflection of a uniform cross-section beam to the moment induced on the beam is given by the following expression:

$$\frac{d^2}{dx^2}y = \frac{M(x)}{E \cdot I} \quad (6.1)$$

where E is the modulus of elasticity of the material of the beam and I is the area moment of inertia of a cross-section of the girder. From this equation, we can solve for the deflection of the midpoint of the beam as a function of force imposed onto the beam and a function of the unit loads location along the beam.

The vertical displacement at a point located a distance H along the beam with an area moment of inertia I_1 is given by the following:

$$\delta = \text{Load} \cdot \frac{1}{3} \cdot \frac{H^2}{L} \cdot \frac{L^2 - 2 \cdot H \cdot L + H^2}{E \cdot I_1} \quad (6.2)$$

By setting our load to be a unit load, we can find an expression for our longitudinal spring constant K_1 :

$$K_1 = \frac{3}{H^2} \cdot \frac{L \cdot E \cdot I_1}{(L^2 - 2 \cdot H \cdot L + H^2)} \quad (6.3)$$

Next we will solve for a relationship of the distribution of force along the transverse direction of the simplified model. By looking at the relationship between two adjacent girders, we solve for the deflection given a load at one of the girders.

Figure 6.3 Rigid Supported Beam With Induced Deflection at One Support

From our assumption that the girders are torsionally fixed and the transverse beam does not rotate at the girder locations we get the following moment distribution relationship:

$$M(x) = \text{Load} \cdot x - \text{Load} \cdot \frac{S}{2} \quad (6.4)$$

At half the distance between the two girders, when x in the above equation $= S/2$, the moment goes to zero. This shows that if there is a hinge located between the two girders and the girders are torsionally fixed, the transverse stiffness is unaffected by the presence

of a longitudinal joint between the girders. Assuming the transverse moment of inertia of the beam is given by I_2 , the deflection at the end of the beam can be found by:

$$\delta = \frac{-\text{Load} \cdot S^3}{12 \cdot E \cdot I_2} \quad (6.5)$$

For a unit load, the transverse stiffness can be represented by the following relationship:

$$K_t = \frac{E \cdot I_2 \cdot 12}{S^3} \quad (6.6)$$

Now that we have relationships for both the longitudinal and transverse stiffness, we can determine the distribution factor of the bridge system. By examining the vertical force acting on each girder, we can determine the following relationship for the reaction on each girder:

$$\begin{aligned} \text{Load}_1 &= R_1 + (\delta_1 - \delta_2) \cdot K_t \\ \text{Load}_2 &= -(\delta_1 - \delta_2) \cdot K_t + R_2 + (\delta_2 - \delta_3) \cdot K_t \\ \text{Load}_3 &= -(\delta_2 - \delta_3) \cdot K_t + R_3 + (\delta_3 - \delta_4) \cdot K_t \\ 0 &= -(\delta_3 - \delta_4) \cdot K_t + R_4 + (\delta_4 - \delta_5) \cdot K_t \\ 0 &= -(\delta_4 - \delta_5) \cdot K_t + R_5 \end{aligned} \quad (6.7)$$

By using the relationship: $\delta = \frac{R}{K_1}$ we can simplify the above equations into the

following system of linear equations:

$$\begin{pmatrix} R_1 \\ R_2 \\ R_3 \\ R_4 \\ R_5 \end{pmatrix} \cdot \begin{pmatrix} \frac{K_T}{K_L} + 1 & \frac{-K_T}{K_L} & 0 & 0 & 0 \\ \frac{-K_T}{K_L} & 2\frac{K_T}{K_L} + 1 & \frac{-K_T}{K_L} & 0 & 0 \\ 0 & \frac{-K_T}{K_L} & 2\frac{K_T}{K_L} + 1 & \frac{-K_T}{K_L} & 0 \\ 0 & 0 & \frac{-K_T}{K_L} & 2\frac{K_T}{K_L} + 1 & \frac{-K_T}{K_L} \\ 0 & 0 & 0 & \frac{-K_T}{K_L} & \frac{K_T}{K_L} + 1 \end{pmatrix} = \begin{pmatrix} Load_1 \\ Load_2 \\ Load_3 \\ 0 \\ 0 \end{pmatrix} \quad (6.8)$$

We can further simplify this expression by defining an arbitrary variable C such that:

$$C = \frac{K_L}{K_T} + 2 \quad (6.9)$$

By inputting this relationship into the above expression and solving for the reactions, we get.

$$(C-2) \cdot \begin{pmatrix} C-1 & -1 & 0 & 0 & 0 \\ -1 & C & -1 & 0 & 0 \\ 0 & -1 & C & -1 & 0 \\ 0 & 0 & -1 & C & -1 \\ 0 & 0 & 0 & -1 & C-1 \end{pmatrix}^{-1} \cdot \begin{pmatrix} Load_1 \\ Load_2 \\ Load_3 \\ 0 \\ 0 \end{pmatrix} = \begin{pmatrix} R_1 \\ R_2 \\ R_3 \\ R_4 \\ R_5 \end{pmatrix} \quad (6.10)$$

When we solve this system of linear equations, the value of R_2 shows us the percent of $Load_1 - Load_3$ that the second girder in the bridge system resists. This is the distribution factor for that girder. The number of girder in the bridge system governs the size of the matrix, yet the matrix maintains the same basic appearance. The matrix will always be an N x N sized matrix with the N being the number of girders. It is a tridiagonal matrix with the first cell and last cell in the matrix always being C-1. The intermediate diagonal

members will always be C with the cells to the left and right of the diagonal always being -1. When applying this model to a real bridge, all the factors are clear except for the transverse stiffness which depends on I_2 . Because there is not a discrete beam that extends between girders, this value is not apparent. I determined I_2 as a function of deck width in the following equation:

$$I_2 = \frac{1}{12} \cdot w \cdot (8.845\text{in})^3 \quad (6.11)$$

8.845in is the equivalent height of a nonprismatic deck that varies from 10" to 6" halfway between the two girders then changes back to 10" at the girder. This still leaves the term w as an unknown which represents the width of the transverse deck that contributes to the transverse stiffness. To assess the validity of this model, I compared this model to my data base from the first parametric study and solved for w so that it would best fit the results from the grillage models. I found that a deck width of 20" best fits the data for all load cases. I set the distance H , which determines the longitudinal stiffness K_L , as the distance from the bridge support to the center of gravity of the load vehicle. For moment distribution factors, this value was $L/2$ and for shear distribution factors, this value was constant at 12' - 10". Figure 6.4 shows how this matrix equation compares with data from the first parametric study for the different distribution factors. This matrix method shows a good comparison for distribution of shear and moment on both interior and exterior girder. The average percent deviation from the DF values determined in the grillage model was 8% for all loading cases except moment distribution on the exterior

girder which was 16%. I compared this method against the whole data base of bridges not just the ones designated as realistic bridges.

Figure 6.4 Comparison of Matrix Model to Parametric Study I

While this matrix method is too complex to be used affectively in design, we can use the equation to understand how different parameters of the bridge affect the way it distributes loads. The main component of this method is the $\frac{K_L}{K_T}$ term which represent the ratio of the longitudinal stiffness to the transverse stiffness. This term is directly related to the distribution of the bridge. If we look at this term as it applies to moment DF where the value of H is L/2 we have the following relationship.

$$\frac{K_L}{K_T} = \frac{\frac{48}{L^3} \cdot E \cdot I_1}{\frac{E \cdot I_2 \cdot 12}{S^3}} \quad (6.12)$$

This relationship shows the transverse modulus of elasticity divided by the modulus of elasticity in the longitudinal direction. Since the decked bulb Tee bridge girder is made with the same material in the deck and in the girder, and since concrete is an isotropic material, the transverse modulus is equal to the longitudinal modulus and these terms factor out. This helps to explain why the modulus of elasticity does not affect the distribution factor of the bridge.

For the shear distribution, the K_L term (Eq. 6.3) is larger because the center of gravity of the vehicle is much closer to the bridge support and the girder is much stiffer. While the transverse stiffness K_T (Eq. 6.6) remains constant for both the moment distribution and the shear distribution. By comparing the two stiffness ratios, we can see that for a given bridge, the Shear ratio will always be greater than the moment ratio.

$$\text{Shear Ratio: } \frac{\frac{3}{H^2} \cdot \frac{L \cdot E \cdot I_1}{(L^2 - 2 \cdot H \cdot L + H^2)}}{K_T} \quad \text{Where } H \text{ is less than } L / 2 \quad (6.13)$$

$$\text{Moment Ratio: } \frac{\frac{48}{L^3} \cdot E \cdot I_1}{K_T} \quad \text{Where } H = L / 2 \quad (6.14)$$

Since the numerator in equation (6.13) is at its theoretical minimum when H is equal to $L/2$, when H is less than $L/2$ $\frac{K_L}{K_T}$ must be greater than when H is equal to $L/2$. This is why shear distribution factors are higher than moment distribution factors for a given bridge.

Figure 6.5 shows how the distribution factor of the edge girder relates to different values of $\frac{K_L}{K_T}$. This graph shows the distribution of a unit load placed over the edge girder as a function of the number of girders. There is no assumed distribution of the load to other girders via the lever rule it is entirely consolidated on the edge girder. The graph is generated from similar methods as shown in the following five girder system.

$$(C-2) \cdot \begin{pmatrix} C-1 & -1 & 0 & 0 & 0 \\ -1 & C & -1 & 0 & 0 \\ 0 & -1 & C & -1 & 0 \\ 0 & 0 & -1 & C & -1 \\ 0 & 0 & 0 & -1 & C-1 \end{pmatrix}^{-1} \cdot \begin{pmatrix} 1 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} = \begin{pmatrix} DF \\ \vdots \\ \vdots \\ \vdots \\ \vdots \end{pmatrix} \quad (6.15)$$

Figure 6.5 Comparison of Different Longitudinal to Transverse Stiffness Ratios

Figure 6.5 shows the complete spectrum of values as the distribution ranges from perfect

distribution across all girders at $\frac{K_L}{K_T} = .001$ to approximately no distribution at $\frac{K_L}{K_T} =$

100. The range of $\frac{K_L}{K_T}$ values that occurred in the first parametric study for moment

distribution varies from 0.01 to 5.823. Typical values for shear distribution vary from 0.1 to 16.7. The change of the distribution factor on the edge girder as the number of girders

in the bridge increases from 4 to 5 decreases by 18% when $\frac{K_L}{K_T} = 0.01$, 13% when $\frac{K_L}{K_T} =$

.05, and 8% when $\frac{K_L}{K_T} = .1$. This shows why varying the number of girders in the bridge

has little effect on the distribution factor for a single lane loaded condition. The

theoretical maximum that the distribution factor could change when the number of girders on the bridge changes from 4 to 5 is a DF change from 0.25 to 0.2 which is only 20%.

7 Simplified design and Bridge Rating Equations

7.1 Equation Development

While the matrix model provided fairly accurate results and has some academic value in understanding the behavior of bulb-tee bridge systems, the method is too complex to be used effectively in design or bridge rating and may not always yield conservative values required for design equations. In developing simplified equations for the single lane distribution factor of the decked bulb tee bridge system, only three parameters were considered: Spacing, Length, and Girder Stiffness. The parametric study showed that the number of girders, deck thickness, and transverse flexural continuity have an average affect on distribution of 12% or less. The bridge data base from which the following simplified equations were developed is the realistic bridges of set II of the parametric study. For most cases the transverse hinged condition gave a higher distribution factor, however for the distribution of shear on exterior girders, the rigid condition gave a higher distribution factor when the girder spacing was 48.5 inches. The simplified equations were developed from which ever condition (hinged or rigid) gave the larger distribution factor. All of the bridges in the data base only have four girders which for the single lane loaded condition will always have a higher distribution factor than a greater number of girders. By tailoring the simplified equations to this data base we can be sure that the error induced by not including the additional parameters will be conservative. This paper provides two simplified equations for moment and shear distribution on the exterior and interior girders. The first equations are very simple and are only a function of the girder

spacing and conform to the traditional S/D method of determining the DF. The second equation is a function of spacing (S), length (L), and the girder's area moment of inertia (I). None of these equations include the multiple presence factor of 1.2. These Simplified equations are accurate only when the bridge being modeled is within the following parameters:

- The girders are typical decked bulb tee girders with the deck poured together with the girder as a single unit.
- The girder height is between 36 inches and 66 inches.
- The deck thickness is between 4 and 8 inches.
- The number of girders of the bridge is greater than or equal to four.
- The bridge has no skew.
- The span length of the bridge is between 40ft and 180ft.
- The girder spacing is between 4ft and 9ft.
- The bridge is only loaded by a single lane of traffic

The variables used in the following simplified equations are defined as follows.

S = Girder Spacing is the distance between the centerlines of two consecutive girders in units of feet.

L = Span Length of the bridge measured from the centers of each support in units of feet.

I = The area moment of inertia about the horizontal axis of one girder in the bridge system. The moment of inertia used should be calculated from the whole girder

including the whole width of the top flange deck portion. For this calculation, use only the moment of inertia calculated from a girder with a typical 6" deck, do not vary the value if the design calls for a different deck thickness. The units of this term are in (feet)⁴.

Following is a list of recommended equations for the Distribution Factor of the Decked Bulb-Tee bridge girder system when it is subjected to the single lane loaded condition:

Moment over Interior Girder (MI):

$$DF(S) = \frac{S}{13} \quad (7.1)$$

$$DF(S, L, I) = \frac{S}{12.5} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-3}{200} \right) \quad (7.2)$$

Moment over Exterior Girder (ME):

$$DF(S) = \frac{S}{11} \quad (7.3)$$

$$DF(S, L, I) = \frac{S}{10} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-1}{300} \right) \quad (7.4)$$

Shear over Interior Girder (SI):

$$DF(S) = \frac{S}{11} \quad (7.5)$$

$$DF(S, L, I) = \frac{S}{12.5} + \frac{I}{250} - \frac{L}{100} \cdot \left(\frac{S}{100} \right) \quad (7.6)$$

Shear over Exterior Girder (SE):

$$DF(S) = \frac{S}{10} \quad (7.7)$$

$$DF(S, L, I) = \frac{S}{12} + \frac{I}{400} - \frac{L}{100} \cdot \left(\frac{S-3}{100} \right) + 0.07 \quad (7.8)$$

7.2 Evaluation of Proposed Equations

Figures 7.1 through 7.8 compare the simplified equations (Eq 7.1 to 7.8) to data base from parametric study II. The values shown are the ratio of the DF values found from the simplified equation divided by the values found from the parametric study. Values less than one are unconservative. These values are plotted against the span length to show how they vary for different lengths. The data points are also divided into three different series based on deck thickness. Most of the unconservative values occur with a 4" deck thickness and 40' length. The simplified equations tend to give overly conservative values for bridges with an 8" deck.

Figure 7.1 Moment DF for Interior Girders DF(S)

Figure 7.2 Moment DF for Interior Girders DF(S,L,I)

Figure 7.3 Moment DF for Exterior Girders DF(S)

Figure 7.4 Moment DF for Exterior Girders DF(S,L,I)

Figure 7.5 Shear DF for Interior Girders DF(S)

Figure 7.6 Shear DF for Interior Girders DF(S,L,I)

Figure 7.7 Shear DF for Exterior Girders DF(S)

Figure 7.8 Shear DF for Exterior Girders DF(S,L,I)

Figures 7.9 through 7.12 show a summary plot of all of the data for bridges with 6" deck thickness from the second set of models of the parametric study. Each of the following graphs is of the data from the parametric study set II. Only the data points for the bridge models of six inch decks are shown. The graphs do not show the data for the 8" or 4" decks. There is no x axis scale to the graphs; the vertical axis of each of the graphs is the distribution factor. The data is sorted in three nested groups. The top group is of bridges with the same girder spacing. There are five different sets of bridges with the same girder spacing which increase from 48" on the left hand side of the graph to 96" on the right side of the graph. Nested within each set of bridges with equal girder spacing are four sets with the same girder heights. The bridges with girder heights increase from 36" on the left to 66" on the right. Within each set of bridges with equal girder heights, the span lengths of the bridges increase from left to right. The limits of the span lengths for set of bridges with a given girder height and spacing is defined in table 5.1 and 5.2. This table defines the set of "realistic bridges" used to develop the simplified equations shown in the following graphs.

Figure 7.9 Parametric Study Data Base Summary of **Moment** Distribution Factors on **Interior** Girders

Figure 7.10 Parametric Study Data Base Summary of **Moment** Distribution Factors on **Exterior** Girders

Figure 7.11 Parametric Study Data Base Summary of **Shear** Distribution Factors on **Interior** Girders

Figure 7.12 Parametric Study Data Base Summary of **Shear** Distribution Factors on **Exterior** Girders

Tables 7.1 and 7.2 give a summary of the distribution factor values for the four sets of tested bridges. The Calculations for the new simplified equations can be found in appendix C. The LRFD Calculations can be found in Appendix B.

Table 7.1 Distribution Factor for Moment:

Interior Girders					
	Data	LRFD	Grillage	DF(S)	DF(S,L,I)
100 th	.35	.66	.37	.57	.4
Campbell	.34	.66	.37	.57	.39
Huffman	.31	.55	.34	.46	.34
Diamond/Dowling	.32	1.48	.36	.58	.40

Exterior Girders					
	Data	LRFD	Grillage	DF(S)	DF(S,L,I)
100 th	.45	.76	.51	.67	.55
Campbell	.53	.76	.50	.67	.54
Huffman	.49	.61	.43	.55	.44
Diamond/Dowling	.46	.77	.49	.69	.56

Table 7.2 Distribution Factor for Shear:

Interior Girders					
	Data	LRFD	Grillage	DF(S)	DF(S,L,I)
100 th	.43	.60	.57	.67	.58
Campbell	.50	.60	.57	.67	.61
Huffman	.46	.62*	.55	.55	.47
Diamond/Dowling	.55†	.60	.53	.69	.57

Exterior Girders					
	Data	LRFD	Grillage	DF(S)	DF(S,L,I)
100 th	.66	.76	.71	.74	.68
Campbell	.71	.76	.69	.74	.70
Huffman	.58	.74*	.72	.60	.57
Diamond/Dowling	.60†	.77	.66	.76	.68

* Indicates adjusted with skew adjustment factor.

† Indicates value not found directly from data but from a FE model that closely approximates the data.

The new simplified equations give a higher distribution factor for all of the tested bridges except for shear and moment on the exterior girder of the bridge located on Huffman street. It appears the shear distribution on the exterior girder is slightly larger for this skewed bridge. This supports the current practice in the simplified equations for AASHTO LRFD as it provides a shear distribution correction factor for skewed bridges. This thesis does not specifically investigate the affects of skew on the distribution of bulb-tee bridges. The moment distribution on the exterior girder of Huffman bridge is also higher than the simplified equations predict and higher than both the grillage and 3D Finite Element models. The experimental values for this set of twin bridges are very consistent for the distribution of the exterior girders. The research team is still currently working to understand why there is a discrepancy between the experimental values and the rigorous math models.

8 Conclusion

8.1 Summary of Findings

By comparing the existing LRFD simplified equations for distribution factors to the testing results of eight bridges and to the results of rigorous math models from two parametric studies, it appears the existing equation are overly conservative when applied to a single lane loaded condition.

This thesis investigates the effects of the longitudinal joint in the decked bulb-tee system. When matching rigorous math models to the data collected from the eight tested bridges, it appears that the bridges, although they did not have any transverse post-tensioning, behaved as if they had full transverse flexural continuity. The 3D finite element modeling of these bridges shows that this behavior could be caused by the intermediate steel diaphragms. By varying the stiffness of the hinge joint in the grillage model, these models show that this behavior could also be caused by the stiffness of the grouted joint and shear keys. From the data of two identical parametric studies, the affects of transverse flexural continuity and discontinuity is very minimal when compared to other bridge parameters. The results of the parametric study in this thesis suggest that when pertaining to the decked bulb-tee bridge system, there not be a distinction between bridges connected only enough to prevent relative vertical displacement and bridges sufficiently connected to act as a unit.

When conducting the first parametric study of the decked bulb-tee system, the study shows that for a single lane loaded condition, the number of girders and overall width of the bridge has little affect on the bridge's distribution. The current simplified equation for the distribution factor of moment on interior girders includes the aspect ratio as one of its parameters. While the sensitivity study in this thesis shows the span length of the bridge to be an important parameter in the bridge's distribution, the overall width of the bridge does not affect its distribution. The current equation will yield excessively conservative distribution factors for very wide bridges. This was also shown from the experimental data of the wide bridges at Diamond and Dowling Intersections.

There are two sets of new simplified equations presented in this thesis. The first set of equations are only a function of the girder spacing and very simple to use and will provide a conservative approximation of the decked bulb-tee bridge system under a single lane loaded condition. This first set of equation although simple are still more accurate for the single lane loaded condition than the existing LRFD equations provided for this bridge system. The second set of equations are a function of girder spacing, span length, and the girder's moment of inertia. These equation provide an added degree of accuracy but are slightly more complex. In bridge design, when the distribution factor is a function of the girder's moment of inertia, the designer must conduct an iterative process to select an appropriate girder size for the bridge. However, these simplified equations are strictly for the single lane loaded condition which is typically used for bridge load rating rather than design. In bridge rating, the size of the girder is already known and

therefore determining the distribution factor as a function of the girder stiffness is a straight forward process.

8.2 Recommendations for Further Research

Further research should be conducted on the behavior of the longitudinal joint of the decked bulb-tee when subjected to multi lane loaded conditions. It would be interesting to see if the joint has as little affect on the distribution of a multi-lane loaded bridge as it does under single lane loaded condition.

The current LRFD equations governing the type j bridge connected only enough prevent relative vertical displacement should be evaluated under all conditions. This paper shows that the simplified equations for this bridge are not accurate for the single lane loaded condition. There has yet to be a thorough evaluation of these simplified equations for the distribution of on exterior girders and the distribution of shear for this bridge system.

The distribution factor equations presented in this thesis do not account for the affect skew has on bridge distribution. Further research should be conducted to determine a skew correction factor for the simplified equations presented in this thesis.

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A. Sample Data

Figure A.1

Figure A.2

Figure A.3

Figure A.4

B. LRFD Simplified Equation Calculations for Tested Bridges

B.1 Bridges located at 100th and Minnesota

Bridge Parameters.

$$W := 37\text{ft} \quad L := 114\text{ft} \quad \theta := 0\text{deg} \quad \mu := .2 \quad A := 1027.25\text{in}^2 \quad N_L := \frac{36}{12}$$

$$I_x := 358820\text{in}^4 \quad I_y := 360743\text{in}^4 \quad I_p := 727970\text{in}^4 \quad J := \frac{A^4}{40.0 \cdot I_p} \quad S := 7.3678\text{ft}$$

Distribution Factor for moments of Interior Beams:

$$K := \sqrt{\frac{(1 + \mu) \cdot I_x}{J}} \quad K = 3.356 \quad C := K \cdot \left(\frac{W}{L}\right) \quad C = 1.089$$

$$\text{Since } C \text{ is less than } 5 \quad D := \left[11.5 - N_L + 1.4 \cdot N_L \cdot (1 - 0.2 \cdot C)^2 \right] \cdot \text{ft} \quad D = 11.07\text{ft}$$

$$\text{Distribution Factor for Interior moment is:} \quad DF_{IM} := \frac{S}{D} \quad DF_{IM} = 0.666$$

For Shear Distribution Factor for interior Beams the LRFD Spec Says to use the

Lever Rule for determining Shear Distribution Factor.

$$\text{Distance of wheel to middle of outside girder:} \quad d_o := 53.125\text{in}$$

$$\text{Multiple Presence Factor:} \quad \text{mpf} := 1.2$$

$$\frac{d_o}{S} = 0.601$$

$$DF_{S\text{Interior}} := \frac{d_o}{S} \cdot \text{mpf}$$

$$DF_{S\text{Interior}} = 0.721$$

For Both Shear and Moment in Exterior Beams, the LRFD specifications says to use the lever rule to calculate the distribution factor. (See Diagram)

Distance of left wheel to middle of second girder: $d_l := 103.0625\text{in}$

Distance of Right wheel to middle of second girder: $d_r := 31.0625\text{in}$

Multiple Presence Factor: $\text{mpf} := 1.2$

$$\frac{d_l + d_r}{2 \cdot S} = 0.759$$

$$DF_{\text{Exterior}} := \frac{d_l + d_r}{2 \cdot S} \cdot \text{mpf}$$

$$DF_{\text{Exterior}} = 0.91$$

B.2 Bridges located at Campbell Creek

$$W := 37\text{ft} \quad L := 139\text{ft} \quad \theta := 4\text{-deg} \quad \mu := .2 \quad A := 1203\text{-in}^2 \quad N_L := \frac{36}{12}$$

$$I_x := 633162.2402\text{-in}^4 \quad I_y := 409316.8908\text{-in}^4 \quad I_p := I_x + I_y \quad J := \frac{A^4}{40.0 \cdot I_p}$$

Distribution Factor for moments of Interior Beams:

$$S := 7.3678\text{ft}$$

$$K := \sqrt{\frac{(1 + \mu) \cdot I_x}{J}} \quad K = 3.889382 \quad C := K \cdot \left(\frac{W}{L}\right) \quad C = 1.035303$$

Since C is less than 5

$$D := \left[11.5 - N_L + 1.4 \cdot N_L \cdot (1 - 0.2 \cdot C)^2 \right] \cdot \text{ft} \quad D = 11.140762\text{ft}$$

Distribution Factor for Interior moment is:

$$DF_{IM} := \frac{S}{D} \quad DF_{IM} = 0.661337$$

For Shear Distribution Factor for interior Beams the LRFD Spec Says to use the Lever Rule for determining Shear Distribution Factor.

$$\text{Distance of wheel to middle of outside girder: } d_o := S - 3\text{ft}$$

$$\text{Multiple Presence Factor: } mpf := 1.2$$

$$\frac{d_o}{S} = 0.592823$$

$$DF_{S\text{Interior}} := \frac{d_o}{S} \cdot mpf$$

$$DF_{S\text{Interior}} = 0.711387$$

For Both Shear and Moment in Exterior Beams, the LRFD specifications says to use the lever rule to calculate the distribution factor. (See Diagram)

Distance of left wheel to middle of second girder: $d_l := 1.5 \cdot S - 2.5\text{ft}$

Distance of Right wheel to middle of second girder: $d_r := 31.0625\text{in}$

Multiple Presence Factor: $\text{mpf} := 1.2$

$$\frac{d_l + d_r}{2 \cdot S} = 0.756009$$

$$DF_{\text{Exterior}} := \frac{d_l + d_r}{2 \cdot S} \cdot \text{mpf}$$

$$DF_{\text{Exterior}} = 0.90721$$

B.3 Bridges located at Huffman St.

$$W := 37\text{ft} \quad L := 128\text{ft} \quad \theta := 28\cdot\text{deg} \quad \mu := .2 \quad A := 903.25\cdot\text{in}^2 \quad N_L := \frac{36}{12}$$

$$I_x := 329617\cdot\text{in}^4 \quad I_y := 207971\cdot\text{in}^4 \quad I_p := I_x + I_y \quad J := \frac{A^4}{40.0\cdot I_p} \quad S := 6.125\text{ft}$$

Distribution Factor for moments of Interior Beams: $J = 30954.395254\text{in}^4$

$$K := \sqrt{\frac{(1 + \mu) \cdot I_x}{J}} \quad K = 3.574656 \quad C := K \cdot \left(\frac{W}{L}\right) \quad C = 1.033299$$

Since C is less than 5

$$D := \left[11.5 - N_L + 1.4 \cdot N_L \cdot (1 - 0.2 \cdot C)^2 \right] \cdot \text{ft} \quad D = 11.143432\text{ft}$$

Distribution Factor for Interior moment is:

$$DF_{IM} := \frac{S}{D} \quad DF_{IM} = 0.549651$$

For Shear Distribution Factor for interior Beams the LRFD Spec Says to use the Lever Rule for determining Shear Distribution Factor.

Distance of wheel to middle of outside girder: $d_o := S - 3\text{ft}$

Multiple Presence Factor: $\text{mpf} := 1.2$ $\frac{d_o}{S} = 0.510204$

$$DF_{S\text{Interior}} := \frac{d_o}{S} \cdot \text{mpf} \quad DF_{S\text{Interior}} = 0.612245$$

Shear Correction Factor for Skewed Bridges:

$$t_s := 5.5\text{in} \quad K_g := I_x \quad CF := 1 + .2 \cdot \left(\frac{12 \cdot L \cdot t_s^3}{K_g} \right) \cdot \tan(\theta) \quad CF = 1.207635$$

$$DF_{S\text{Interior}} \cdot CF = 0.739368 \quad \frac{d_o}{S} \cdot CF = 0.61614$$

For Both Shear and Moment in Exterior Beams, the LRFD specifications says to use the lever rule to calculate the distribution factor. (See Diagram)

Distance of left wheel to middle of second girder: $d_l := 80.75\text{in}$

Distance of Right wheel to middle of second girder: $d_r := 8.75\text{in}$

Multiple Presence Factor: $\text{mpf} := 1.2$

$$DF_{\text{Exterior}} := \frac{d_l + d_r}{2 \cdot S} \cdot \text{mpf} \quad DF_{\text{Exterior}} = 0.730612$$

$$DF_{\text{Exterior}} \cdot CF = 0.882313$$

$$\frac{d_l + d_r}{2 \cdot S} = 0.608844 \quad \frac{d_l + d_r}{2 \cdot S} \cdot CF = 0.73526$$

B.4 Bridges located at Diamond and Dowling Intersections

$$W := 108\text{ft} \quad L := 110\text{ft} \quad \theta := 0\cdot\text{deg} \quad \mu := .2 \quad A := 1026\cdot\text{in}^2$$

$$I_x := 279223.5\cdot\text{in}^4 \quad I_y := 362787.1875\cdot\text{in}^4 \quad I_p := I_x + I_y$$

$$J := \frac{A^4}{40.0\cdot I_p} \quad S := 7.533333\text{ft} \quad N_L := \frac{108}{12} \quad N_L = 9$$

Distribution Factor for moments of Interior Beams:

$$K := \sqrt{\frac{(1 + \mu) \cdot I_x}{J}} \quad K = 2.78659 \quad C := K \cdot \left(\frac{W}{L}\right) \quad C = 2.735924$$

Since C is less than 5

$$D := \left[11.5 - N_L + 1.4 \cdot N_L \cdot (1 - 0.2 \cdot C)^2 \right] \cdot \text{ft} \quad D = 5.083524\text{ft}$$

Distribution Factor for Interior moment is:

$$DF_{IM} := \frac{S}{D} \quad DF_{IM} = 1.481912$$

For Shear Distribution Factor for interior Beams the LRFD Spec Says to use the Lever Rule for determining Shear Distribution Factor.

$$\text{Distance of wheel to middle of outside girder: } d_o := S - 3\text{ft}$$

$$\text{Multiple Presence Factor: } mpf := 1.2 \quad \frac{d_o}{S} = 0.60177$$

$$DF_{S\text{Interior}} := \frac{d_o}{S} \cdot mpf \quad DF_{S\text{Interior}} = 0.722124$$

For Both Shear and Moment in Exterior Beams, the LRFD specifications says to use the lever rule to calculate the distribution factor.

Distance of left wheel to middle of second girder: $d_l := 1.5 \cdot S - 2.5\text{ft}$

Distance of Right wheel to middle of second girder: $d_r := d_l - 6\text{ft}$

Multiple Presence Factor: $\text{mpf} := 1.2$ $\frac{d_l + d_r}{2 \cdot S} = 0.769911$

$DF_{\text{Exterior}} := \frac{d_l + d_r}{2 \cdot S} \cdot \text{mpf}$ $DF_{\text{Exterior}} = 0.923894$

C. New Simplified Equation Calculations for Tested Bridges

C.1 Bridges Located at 100th and Minnesota.

$$\text{kip} := 1000\text{lb} \quad \text{ksi} := 1000\text{psi}$$

Bridge Input Parameters:

$$S := 88.4 \frac{\text{in}}{\text{ft}} \quad L := 114 \frac{\text{ft}}{\text{ft}} \quad I := 362334.7817 \frac{\text{in}^4}{\text{ft}^4}$$

Moment over Interior Girder (MI):

$$\frac{S}{13} = 0.566667$$

$$\frac{S}{12.5} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-3}{200} \right) = 0.398679$$

Moment over Exterior Girder (ME):

$$\frac{S}{11} = 0.669697$$

$$\frac{S}{10} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-1}{300} \right) = 0.552979$$

Shear over Interior Girder (SI):

$$\frac{S}{11} = 0.669697$$

$$\frac{S}{12.5} + \frac{I}{250} - \frac{L}{100} \cdot \left(\frac{S}{100} \right) = 0.575248$$

Shear over Exterior Girder (SE):

$$\frac{S}{10} = 0.736667$$

$$\frac{S}{12} + \frac{I}{400} - \frac{L}{100} \cdot \left(\frac{S-3}{100} \right) + 0.07 = 0.677793$$

C.2 Bridges Located at Campbell Creek.

$$\text{kip} := 1000\text{lbf} \quad \text{ksi} := 1000\text{psi}$$

Bridge Input Parameters:

$$S := 88.4 \frac{\text{in}}{\text{ft}} \quad L := 139 \quad I := 633162.24 \frac{\text{in}^4}{\text{ft}^4}$$

Moment over Interior Girder (MI):

$$\frac{S}{13} = 0.566667$$

$$\frac{S}{12.5} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-3}{200} \right) = 0.387631$$

Moment over Exterior Girder (ME):

$$\frac{S}{11} = 0.669697$$

$$\frac{S}{10} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-1}{300} \right) = 0.543459$$

Shear over Interior Girder (SI):

$$\frac{S}{11} = 0.669697$$

$$\frac{S}{12.5} + \frac{I}{250} - \frac{L}{100} \cdot \left(\frac{S}{100} \right) = 0.609074$$

Shear over Exterior Girder (SE):

$$\frac{S}{10} = 0.736667$$

$$\frac{S}{12} + \frac{I}{400} - \frac{L}{100} \cdot \left(\frac{S-3}{100} \right) + 0.07 = 0.699528$$

C.3 Bridges Located at Huffman St.

Bridge Input Parameters: kip := 1000lbf ksi := 1000psi

$$S := 72 \frac{\text{in}}{\text{ft}} \quad L := 128 \quad I := 341089.672 \frac{\text{in}^4}{\text{ft}^4}$$

Moment over Interior Girder (MI):

$$\frac{S}{13} = 0.461538$$

$$\frac{S}{12.5} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-3}{200} \right) = 0.342831$$

Moment over Exterior Girder (ME):

$$\frac{S}{11} = 0.545455$$

$$\frac{S}{10} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-1}{300} \right) = 0.441497$$

Shear over Interior Girder (SI):

$$\frac{S}{11} = 0.545455$$

$$\frac{S}{12.5} + \frac{I}{250} - \frac{L}{100} \cdot \left(\frac{S}{100} \right) = 0.468997$$

Shear over Exterior Girder (SE):

$$\frac{S}{10} = 0.6$$

$$\frac{S}{12} + \frac{I}{400} - \frac{L}{100} \cdot \left(\frac{S-3}{100} \right) + 0.07 = 0.572723$$

C.4 Bridges Located at Diamond and Dowling Intersections.

Bridge Input Parameters: kip := 1000lbf ksi := 1000psi

$$S := 90.6 \frac{\text{in}}{\text{ft}} \quad L := 110 \quad I := 279223.5 \frac{\text{in}^4}{\text{ft}^4}$$

Moment over Interior Girder (MI):

$$\frac{S}{13} = 0.580769$$

$$\frac{S}{12.5} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-3}{200} \right) = 0.398635$$

Moment over Exterior Girder (ME):

$$\frac{S}{11} = 0.686364$$

$$\frac{S}{10} + \frac{I}{300} - \frac{L}{10} \cdot \left(\frac{S-1}{300} \right) = 0.559719$$

Shear over Interior Girder (SI):

$$\frac{S}{11} = 0.686364$$

$$\frac{S}{12.5} + \frac{I}{250} - \frac{L}{100} \cdot \left(\frac{S}{100} \right) = 0.574813$$

Shear over Exterior Girder (SE):

$$\frac{S}{10} = 0.755$$

$$\frac{S}{12} + \frac{I}{400} - \frac{L}{100} \cdot \left(\frac{S-3}{100} \right) + 0.07 = 0.682781$$

D. Calculation of Section Properties of the Longitudinal Joint

Assume grout modulus of elasticity

kip := 1000lbf ksi := 1000psi

$$E_c := 57\text{ksi} \cdot \sqrt{2000} \quad E_c = 2549.117494\text{ksi}$$

Assume steel modulus of elasticity

Modular Ratio

$$E_s := 29000\text{ksi}$$

$$n := \frac{E_s}{E_c} \quad n = 11.376486$$

Uncracked
Cross-Section

Worst Case
Cracked
Cross-Section

Assume the width of the cross-section is the same width as the transverse members in the grillage model.

$$w := 3.8\text{ft}$$

To determine the depth of the neutral axis by summing the moments of the areas about the neutral axis to find c :

$$w \cdot c \cdot \frac{c}{2} + 1\text{in}^2 \cdot n \cdot (c - 1.875\text{in}) = 0$$

$$c := .749\text{in}$$

The cracked moment of inertia for a 3.8ft section along the longitudinal joint is:

$$I_{cr} := \left[\frac{1}{12} \cdot 4 \cdot \text{in} \cdot n \cdot \left(\frac{1}{4} \cdot \text{in} \right)^3 \right] + 4 \cdot \text{in} \cdot n \cdot \frac{1}{4} \cdot \text{in} \cdot (1.875 \cdot \text{in} - c)^2 + \left(\frac{1}{12} \cdot w \cdot c^3 \right) + (w \cdot c) \cdot \left(\frac{c}{2} \right)^2$$

$$I_{cr} = 20.870111 \text{ in}^4$$