Lateral Load Distribution and Deck Design Recommendations for the Sandwich Plate System (SPS) in Bridge Applications

by

Devin K. Harris

Dissertation research submitted to the faculty of the Virginia Polytechnic Institute and State University in partial fulfillment for the requirements of the degree of Doctor of Philosophy in Civil Engineering

Tommy Cousins, Co-Chair
Thomas M. Murray, Co-Chair
Elisa D. Sotelino
Carin L. Roberts-Wollmann
John J. Lesko
Michael C. Brown

October 2007

Blacksburg, VA

Keywords:
Sandwich Plate System, Lateral Load Distribution, Dynamic Load Allowance, Deck Design, Parametric, Natural Frequency
Lateral Load Distribution and Deck Design Recommendations for the Sandwich Plate System (SPS) in Bridge Applications

Devin K. Harris

Abstract

The deterioration of the nation’s civil infrastructure has prompted the investigation of numerous solutions to offset the problem. Some of these solutions have come in the form of innovative materials for new construction, whereas others have considered rehabilitation techniques for repairing existing infrastructure. A relatively new system that appears capable of encompassing both of these solution methodologies is the Sandwich Plate System (SPS), a composite bridge deck system that can be used in both new construction or for rehabilitation applications. SPS consists of steel face plates bonded to a rigid polyurethane core; a typical bridge application utilizes SPS primarily as a bridge deck acting compositely with conventional support girders. As a result of this technology being relatively new to the bridge market, design methods have yet to be established. This research aims to close this gap by investigating some of the key design issues considered to be limiting factors in implementation of SPS. The key issues that will be studied include lateral load distribution, dynamic load allowance and deck design methodologies.

With SPS being new to the market, there has only been a single bridge application, limiting the investigations of in-service behavior. The Shenley Bridge was tested under live load conditions to determine in-service behavior with an emphasis on lateral load distribution and dynamic load allowance. Both static and dynamic testing were conducted. Results from the testing allowed for the determination of lateral load distribution factors and dynamic load allowance of an in-service SPS bridge. These results also provided a means to validate a finite element modeling approach which would could as the foundation for the remaining investigations on lateral load distribution and dynamic load allowance.

The limited population of SPS bridges required the use of analytical methods of analysis for this study. These analytical models included finite element models and a stiffened plate
model. The models were intended to be simple, but capable of predicting global response such as lateral load distribution and dynamic load allowance. The finite element models are shown to provide accurate predictions of the global response, but the stiffened plate approach was not as accurate. A parametric investigation, using the finite element models, was initiated to determine if the lateral load distribution characteristics and vibration response of SPS varied significantly from conventional systems. Results from this study suggest that the behavior of SPS does differ somewhat from conventional systems, but the response can be accommodated with current AASHTO LRFD bridge design provisions as a result of their conservativeness.

In addition to characterizing global response, a deck design approach was developed. In this approach the SPS deck was represented as a plate structure, which allowed for the consideration of the key design limit states within the AASHTO LRFD specification. Based on the plate analyses, it was concluded that the design of SPS decks is stiffness-controlled as limited by the AASHTO LRFD specification deflection limits for lightweight metal decks. These limits allowed for the development of a method for sizing SPS decks to satisfy stiffness requirements.
Acknowledgements

I would like to thank the following people who assisted me throughout my graduate career:

My dissertation committee, your service is much appreciated. Dr. Tommy Cousins, for serving as the Co-chair on my committee and his continuous support during my time at Virginia Tech. You have helped me tremendously in my research and career pursuits and at the same time allowed me to flourish on my own. Dr. Murray for serving as a Co-chair on my committee. You experience and insight has given me the opportunity to visualize concepts from a different perspective which I feel will be invaluable in my future endeavors. Dr. Elisa Sotelino, for provide guidance in the numerous aspects. I expect that your advice and guidance will continue to prove invaluable to me throughout my career. Dr. Carin Roberts-Wollmann, for your assistance and guidance through both my masters and doctoral work. I am also grateful for your guidance in the classroom where you will continue to serve as my role model for teaching. Dr. Michael Brown, for serving on my committee and assisting during the field investigation of the Shenley Bridge. Dr. Lesko for serving on my committee and giving me the opportunity to become an IGERT Fellow.

I would also like to express my gratitude to some friends and colleagues who have helped me in both my research and on a personal basis including: Josh Boggs, Scott Cirmo, Dr. Andrei Ramniceanu, Dr. Charles Newhouse, Chris Carroll, Justin Marshall, Bernard Kassner.

I am also thankful to my research sponsors, VDOT and Intelligent Engineering for providing the support and interest for this project. I would like to thank Via Department of Civil and Environmental Engineering and the NSF IGERT program for their financial support than enabled me to pursue my PhD.

Finally and most importantly, I would like to thank my wife, Arlene, for her support throughout my life and during my graduate career at Virginia Tech. You were always there to provide just what I needed to succeed. Without your love, encouragement and support, none of this would have been possible.
Table of Contents

Abstract ........................................................................................................................................... ii
Acknowledgements .......................................................................................................................... iv
List of Tables ....................................................................................................................................... xii
List of Figures ...................................................................................................................................... xiv

Chapter 1 – Introduction ...................................................................................................................... 1
1.1 – Sandwich Plate System Overview ............................................................................................ 1
1.1.1 – History of Sandwich Construction ....................................................................................... 2
1.1.2 – Brief History of SPS ............................................................................................................. 4
1.1.2.1 – Current Bridge Inventory ................................................................................................. 6
1.1.2.1.1 – Lennoxville Bridge ....................................................................................................... 6
1.1.2.1.2 – Schönwasserparke Bridge ............................................................................................. 7
1.1.2.2 – Materials .......................................................................................................................... 7
1.1.2.2.1 – Steel Plates .................................................................................................................... 7
1.1.2.2.2 – Sandwich Core .............................................................................................................. 8
1.1.2.2.2.2 – Mechanical Properties of SPS Polyurethane Core ................................................... 11
1.1.2.2.2.3 – Characteristics of SPS Polyurethane Core ............................................................... 13
1.2 – Project Overview and Scope ..................................................................................................... 13

Chapter 2 – Literature Review ........................................................................................................ 14
2.1 – Literature Review Overview .................................................................................................. 14
2.2 – Bridge Superstructure Design Overview .................................................................................. 14
2.2.1 – Live Load Analysis ............................................................................................................. 15
2.2.2 – Girder Design ..................................................................................................................... 16
2.2.2.1 – Beam-Line Method (Distribution Factors) .................................................................. 17
2.2.2.2 – Grillage Method ............................................................................................................. 17
2.2.2.3 – Finite Element Method ................................................................................................. 18
2.2.2.4 – Finite Strip Method ...................................................................................................... 19
2.2.3 – Deck Design ..................................................................................................................... 19
2.3 – Lateral Load Distribution and Load Distribution Factors (DF) ............................................. 21
2.3.1 – Lateral Load Distribution (AASHTO Standard Specification) ......................................... 23
2.3.2 – Lateral Load Distribution (AASHTO LRFD Specification) ................................................... 23
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.3.3 – Lateral Load Distribution (CHBDC Specification)</td>
<td>26</td>
</tr>
<tr>
<td>2.3.4 – Summary of Relevant Lateral Load Distribution Research</td>
<td>27</td>
</tr>
<tr>
<td>2.3.4.1 – Evaluation of distribution factors</td>
<td>27</td>
</tr>
<tr>
<td>2.3.4.2 – Parameter specific effect on lateral load distribution</td>
<td>32</td>
</tr>
<tr>
<td>2.3.4.3 – Development of simplified methods for lateral load distribution</td>
<td>34</td>
</tr>
<tr>
<td>2.3.5 – Load Distribution Factors for Shear</td>
<td>38</td>
</tr>
<tr>
<td>2.3.6 – Summary of Lateral Load Distribution</td>
<td>40</td>
</tr>
<tr>
<td>2.4 – Dynamic Load Allowance (DLA) or Impact Factors (IM)</td>
<td>40</td>
</tr>
<tr>
<td>2.4.1 – Source of amplification</td>
<td>40</td>
</tr>
<tr>
<td>2.4.2 – Dynamic Amplification – Experimental Methods</td>
<td>41</td>
</tr>
<tr>
<td>2.4.3 – Dynamic Amplification – Analytical Methods</td>
<td>43</td>
</tr>
<tr>
<td>2.4.4 – Dynamic amplification in other bridge types</td>
<td>45</td>
</tr>
<tr>
<td>2.4.5 – Dynamic amplification for design</td>
<td>46</td>
</tr>
<tr>
<td>2.4.6 – Summary of dynamic load allowance</td>
<td>47</td>
</tr>
<tr>
<td>2.5 – Bridge Deck Analysis</td>
<td>48</td>
</tr>
<tr>
<td>2.5.1 – SPS Deck Design</td>
<td>48</td>
</tr>
<tr>
<td>2.5.2 – AASHTO Deck Design Considerations</td>
<td>50</td>
</tr>
<tr>
<td>2.5.2.1 – Decks in Slab-Girder Bridges</td>
<td>50</td>
</tr>
<tr>
<td>2.5.2.1.1 – Linear Elastic Method (Equivalent Strip)</td>
<td>51</td>
</tr>
<tr>
<td>2.5.2.1.2 – Yield-Line Method</td>
<td>53</td>
</tr>
<tr>
<td>2.5.2.1.3 – Empirical Method</td>
<td>53</td>
</tr>
<tr>
<td>2.5.2.2 – Bridges with Orthotropic Type Bridge Decks</td>
<td>54</td>
</tr>
<tr>
<td>2.5.2.2.1 – Orthotropic Decks</td>
<td>54</td>
</tr>
<tr>
<td>2.5.2.2.2 – Grid Decks</td>
<td>55</td>
</tr>
<tr>
<td>2.5.2.3 – AASHTO Limit States</td>
<td>55</td>
</tr>
<tr>
<td>2.5.2.3.1 – AASHTO Strength Limit State</td>
<td>56</td>
</tr>
<tr>
<td>2.5.2.3.2 – AASHTO Serviceability Limit State</td>
<td>56</td>
</tr>
<tr>
<td>2.5.2.3.2.1 – Deflection Limits</td>
<td>56</td>
</tr>
<tr>
<td>2.5.2.3.3 – AASHTO Fatigue Limit State</td>
<td>57</td>
</tr>
<tr>
<td>2.5.2.3.3.1 – Fatigue and Fracture Limits</td>
<td>58</td>
</tr>
<tr>
<td>2.5.3 – Considerations for SPS Deck Design</td>
<td>59</td>
</tr>
</tbody>
</table>
Chapter 3 – Analytical Models

3.1 – Finite Element Modeling of Bridge Structures

3.1.1 – Simplified Models vs. Detailed Models

3.1.2 – Slab-Girder Bridge System

3.1.3 – Slab-Girder FEA Models

3.1.4 – Finite Element Software

3.1.5 – Element Selection

3.1.6 – Equivalent Nodal Load

3.1.7 – Boundary Conditions

3.1.8 – Summary of Finite Element Modeling for Slab-Girder Bridges

3.2 – Development of Finite Element Model for SPS

3.2.1 – Element Selection and Validation

3.2.1.1 – Deck Element

3.2.1.2 – Girder Element

3.2.1.3 – Composite Connection Element

3.2.2 – Details of Simplified FE Models

3.2.3 – Comparison of Simplified Models to Measured Results

3.2.3.1 – Virginia Tech Laboratory Specimen

3.2.3.1.1 – Load Case C1 Comparison

3.2.3.1.2 – Load Case C2 Comparison

3.2.3.2 – Shenley Bridge Test (November 2003)

3.2.3.2.1 – Shenley Field Test Comparisons (November 2003)

3.3 – Stiffened Plate Model for Bridge Structures

3.3.1 – Energy Method for Plates Continuous over Elastic Supports

3.3.2 – Development of Plate Solutions for SPS Bridge

3.3.2.1 – Equivalent Plate Properties

3.3.2.2 – Representation of Equivalent Stiffening Elements

3.3.2.3 – Boundary Conditions

3.3.2.4 – Comparison of Stiffened Plate Results to Experimental Results

3.3.3 – Summary of Stiffened Plate Modeling
5.4 – Validation of Analytical Models ................................................................. 166
5.4.1 – Finite Element Model ................................................................................ 166
5.4.1.1 – Shenley Field Test (June 2005) Comparisons (Deflection and Strain) ........ 166
5.4.1.1.1 – Load Case A Comparison ................................................................. 166
5.4.1.1.2 – Load Case B Comparison ................................................................. 168
5.4.1.1.3 – Load Case C Comparison ................................................................. 170
5.4.1.1.4 – Load Case E Comparison ................................................................. 171
5.4.1.2 – Summary of Model Comparison and Selection ........................................ 173
5.4.1.3 – Shenley Field Test (June 2005) Comparisons (Distribution Factors) ....... 173
5.4.2 – Stiffened Plate Model ................................................................................. 175
5.4.2.1 – Shenley Field Test Comparisons (June 2005) ........................................ 175
5.4.2.1.1 – Load Case A ...................................................................................... 175
5.4.2.1.2 – Load Case B ...................................................................................... 176
5.4.2.1.3 – Load Case C ...................................................................................... 178
5.4.2.1.4 – Load Case E ...................................................................................... 179
5.4.2.2 – Summary of Stiffened Plate Comparisons .............................................. 180
5.4.2.3 – Comparison of Stiffened Plate Distribution Factors ................................. 181
5.5 – Parametric Investigation of Lateral Load Distribution of SPS Bridges .......... 183
5.5.1 – Investigated Parameters ............................................................................ 184
5.5.2 – Parametric Model Development ................................................................. 184
5.5.2.1 - Parametric Model Capabilities ............................................................... 185
5.5.2.2 – Parametric Investigation Methodology and Assumptions ....................... 186
5.5.3 – Analysis of Results – Parametric Investigation of Lateral Load Distribution .... 188
5.5.3.1 – Parameter Influence ............................................................................. 189
5.5.3.1.1 – Girder Spacing .................................................................................. 190
5.5.3.1.2 – Span Length ..................................................................................... 191
5.5.3.1.3 – Plate Rigidity and Deck Thickness ..................................................... 193
5.5.3.1.4 – Longitudinal Stiffness Parameter ....................................................... 196
5.5.3.2 – Summary of Parameter Influence on Lateral Load Distribution ............... 197
5.5.3.3 – Comparison to Code Predictions and Upper Limits Solutions ................ 198
5.5.3.4 – Comparison to Equivalent Design Reinforced Concrete Model ............... 202
List of Tables

Table 1 – Comparison of Tension and Compression Mechanical Properties ......................... 12
Table 2 - LRFD Lateral Load Distribution Factors for Concrete Deck on Steel Girders ............. 24
Table 3 – Equivalent Distribution Factors from CHBDC .......................................................... 27
Table 4 – Proposed Calibration Constants for NCHRP Project 12-62 ....................................... 37
Table 5 – Summary of Dynamic Load Allowance (Impact Factors) ......................................... 47
Table 6 – Summary of Equivalent Strip Widths ........................................................................ 52
Table 7 – Summary of AASHTO Deflection Limits .................................................................. 57
Table 8 – AASHTO Standard Specification Allowable Fatigue Stress Ranges .......................... 59
Table 9 – Comparison of ANSYS Shell Elements ................................................................. 72
Table 10 – Comparison of ANSYS Beam Elements ............................................................... 73
Table 11 – Element Summary for Simplified Finite Element Models .................................... 76
Table 12 – Comparison of Vertical Deflections for Load Case C1 under Maximum Load ......... 83
Table 13 - Comparison of Strains for Load Case C1 under Maximum Loading ...................... 85
Table 14– Comparison of Wire Pot Deflections for Load Case C2 under Maximum Load ...... 87
Table 15- Comparison of Strains for Load Case C2 under Maximum Loading ....................... 89
Table 16 – Summary of Equivalent Flexural Rigidity Prediction Methods ............................... 103
Table 17 – Comparison of Equivalent Flexural Rigidities ........................................................ 104
Table 18 – Summary of Boundary Conditions for Full Scale Panel Comparison ...................... 112
Table 19 – Summary of Loading Conditions for Key Limit States ......................................... 125
Table 20 – Plate Properties for Serviceability (Deflection) Limit State ................................... 130
Table 21 – Summary of Relationships Between Plate Deflected Surface and Moment/Stress .. 148
Table 22 – Comparison of FEA Solution with Classical Plate Solution ................................. 149
Table 23 – SPS Deck Stiffness Relative to an 8 in. Reinforced Concrete Deck Stiffness ......... 153
Table 24 – Summary of Lateral Load Distribution Factors for Single Truck Loading .......... 163
Table 25 – Summary of Lateral Load Distribution Factors for Paired Truck Loading .......... 163
Table 26 – Critical Distribution Factors for Shenley Bridge vs. Code Provisions .................. 164
Table 27 – Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case A ............... 167
Table 28 - Bottom Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case A. 168
Table 29 – Top Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case A ...... 168
Table 30 - Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case B ............... 169
# List of Figures

Figure 1 – Sandwich Plate System .................................................................................................. 1  
Figure 2 – Representations of Sandwich Construction Configurations ........................................ 2  
Figure 3 – Comparison of Equivalent Weight Systems – Sandwich vs. Thin Plate ..................... 3  
Figure 4 – Shenley Bridge - Quebec, Canada ............................................................................. 5  
Figure 5 – Shenley Bridge Construction ................................................................................... 5  
Figure 6 – Oxidized Weathering Steel ...................................................................................... 8  
Figure 7 – Microphase Separation ............................................................................................ 9  
Figure 8 – Cross-linked Polymer Chain .................................................................................. 10  
Figure 9 – HS-20 and Tandem Design Trucks ......................................................................... 16  
Figure 10 – Design Lane Load .................................................................................................. 16  
Figure 11 – Slab-Girder to Equivalent Grillage Model ............................................................... 18  
Figure 12 – Generic Finite Element Method Discretization ...................................................... 19  
Figure 13 – Deflected shape of slab-girder cross-section .......................................................... 22  
Figure 14 – Lever Rule Representation of Slab-Girder Bridge .................................................. 25  
Figure 15 – 1988 CAN/CSA DLA vs. Frequency ...................................................................... 47  
Figure 16 – Comparison of Orthotropic and SPS Configurations ............................................ 50  
Figure 17 – Equivalent Strip Representations .......................................................................... 51  
Figure 18 – Generic Yield-Line Failure Mechanism .................................................................. 53  
Figure 19 – Orthotropic Deck System ..................................................................................... 55  
Figure 20 – Representative Models Types for Slab-Girder Bridges ......................................... 65  
Figure 21 – Chung and Sotelino Eccentric Beam Idealizations ................................................. 67  
Figure 22 – Comparison of Beam Boundary Condition Representations .................................. 70  
Figure 23 – Shell Element Performance Comparison ............................................................... 73  
Figure 24 – Shell and Beam Element Deformation with Rigid Link ......................................... 75  
Figure 25 – ANSYS Representation of “Base” Model ................................................................. 76  
Figure 26 – ANSYS Representation of “Expanded” Model ....................................................... 77  
Figure 27 – ANSYS Representation of “Complete” Model ....................................................... 77  
Figure 28 – Virginia Tech Half-Scale Laboratory Specimen ..................................................... 79  
Figure 29 – Cross section View of Virginia Tech Laboratory Specimen .................................... 79
Figure 92 – Peak Tensile Stress Range Comparison ................................................................. 143
Figure 93 – Required Core Thickness vs. Girder Spacing and Flexural Rigidity ..................... 145
Figure 94 – Section Design Spreadsheet (Screenshot) ............................................................ 146
Figure 95 – Classical Plate Model Geometry ........................................................................ 149
Figure 96 – Shenley Bridge, Quebec, Canada ................................................................. 154
Figure 97 – Shenley Bridge Cross-Section ................................................................. 155
Figure 98 – Shenley Bridge Plan View ............................................................................ 155
Figure 99 - Representation of SPS Cross Section with Instrumentation Layout(not to scale)... 156
Figure 100 – Elevation of Shenley Bridge with Instrumentation Locations ....................... 156
Figure 101 - Deflectometer Mounted on Girder ................................................................. 157
Figure 102 - Strain Gauge Mounted on Girder .................................................................. 157
Figure 103 – Campbell Scientific CR9000 .................................................................... 157
Figure 104 – Three-Axle Dump Truck used for Shenley Bridge Live Load Testing .......... 158
Figure 105 – Loading Configurations for Shenley Bridge Live Load Tests ....................... 158
Figure 106 - Typical Displacement Response with Noise and Offset Baseline .................... 160
Figure 107 - Typical Displacement Response for Quasi-Static Load Case C - Interior girder .. 161
Figure 108 - Typical Strain Response for Quasi-Static Load Case C - Interior girder .......... 161
Figure 109 - Lateral Load Distribution Factors for Single Truck Load (Shenley Bridge)....... 164
Figure 110 - Lateral Load Distribution for Paired Truck Load (Shenley Bridge)................. 165
Figure 111 – FEA Distribution Factors vs. Measured and Code (Single Truck) .................... 174
Figure 112 – FEA Distribution Factors vs. Measured and Code (Paired Trucks) ................. 174
Figure 113 – Comparison of Stiffened Plate Model Deflection Predictions for Load Case A... 176
Figure 114 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case B.... 177
Figure 115 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case C... 179
Figure 116 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case E... 180
Figure 117 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case A) ........ 181
Figure 118 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case B)........ 182
Figure 119 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case C)........ 182
Figure 120 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case E)........ 183
Figure 121 – ANSYS Parametric Bridge Model Flow Diagram ........................................ 185
Figure 122 – Single Truck Loading - Parametric Investigation of Lateral Load Distribution ... 186
Figure 123 – Two Truck Loading - Parametric Investigation of Lateral Load Distribution...... 186
Figure 124 – Distribution Factor for All Models and Loadings vs. Girder Spacing .............. 188
Figure 125 – Lateral Load Distribution Factors (Critical) vs. Girder Spacing...................... 190
Figure 126 – Interior Distribution Factors vs. Span Length.............................................. 191
Figure 127 – Exterior Distribution Factors vs. Span Length.............................................. 192
Figure 128 – Interior Distribution Factors vs. Span Length (Variable Spacing – 1 truck)...... 192
Figure 129 – Interior Distribution Factors vs. Span Length (Variable Spacing – 2 trucks).... 193
Figure 130 – Interior Distribution Factors vs. Plate Flexural Rigidity............................... 194
Figure 131 – Exterior Distribution Factors vs. Plate Flexural Rigidity............................... 194
Figure 132 – Interior Distribution Factors vs. Plate Thickness.......................................... 195
Figure 133 – Exterior Distribution Factors vs. Plate Thickness.......................................... 195
Figure 134 – Interior Distribution Factors vs. Longitudinal Stiffness (Kg).......................... 196
Figure 135 – Exterior Distribution Factors vs. Longitudinal Stiffness (Kg).......................... 197
Figure 136 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule....................... 198
Figure 137 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule....................... 199
Figure 138 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule....................... 199
Figure 139 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule....................... 200
Figure 140 – SPS Distribution Factors vs. AASHTO LRFD (all scenarios)......................... 201
Figure 141 – SPS Distribution Factors vs. Lever Rule (all scenarios)................................. 202
Figure 142 – Distribution Factors for SPS vs. Reinforced Concrete (Interior – One Truck).... 203
Figure 143 – Distribution Factors for SPS vs. Reinforced Concrete (Interior – Two Trucks)... 204
Figure 144 – Distribution Factors for SPS vs. Reinforced Concrete (Exterior – One Truck).... 204
Figure 145 – Distribution Factors for SPS vs. Reinforced Concrete (Exterior – Two Trucks).. 205
Figure 146 – Ratio of SPS LDF to R/C LDF (Single Truck).................................................... 205
Figure 147 – Ratio of SPS LDF to R/C LDF (Two Trucks)..................................................... 206
Figure 148 – SPS Distribution Factors to NCHRP Project 12-26........................................ 206
Figure 149 – SPS Distribution Factors vs. NCHRP Project 12-62 (Interior – One Truck)....... 207
Figure 150 – SPS Distribution Factor vs. NCHRP Project 12-62 (Interior – Two Trucks)...... 208
Figure 151 – SPS Distribution Factors vs. NCHRP Project 12-62 (Exterior – One Truck)..... 208
Figure 152 – SPS Distribution Factors vs. NCHRP Project 12-62 (Exterior – Two Trucks)..... 209
Figure 153 – SPS Distribution Factors vs. NCHRP 12-62 Equations (All Scenarios).......... 210
Figure 154 – Typical Displacement Response for Dynamic Field Test of Shenley Bridge ...... 213
Figure 155 – Comparison of Static to Dynamic Displacement Response ............................................. 214
Figure 156 – Comparison of Static to Dynamic Strain Response .......................................................... 214
Figure 157 – Mode Shape for Shenley Bridge Model 5 ........................................................................ 220
Figure 158 – Natural Frequency of SPS Parametric Bridges vs. Span Length .................................. 222
Figure 159 – Span Length vs. Mass Comparison of SPS and R/C Bridge Models .................................. 223
Figure 160 – Natural Frequency vs. Span Length (SPS and R/C Models) .............................................. 224
Figure 161 – Comparison of SPS Natural Frequencies with Literature Results .................................. 225
Figure 162 – Load Case 4 Configuration for Shenley Bridge Field Test – November 2003 .............. 245
Figure 163 - Comparison of Girder A Deflection for Load Case 4 .................................................... 246
Figure 164 - Comparison of Girder C Deflection for Load Case 4 .................................................... 246
Figure 165 – Load Case 5 Configuration for Shenley Bridge Field Test – November 2003 .............. 246
Figure 166 - Comparison of Girder A Deflection for Load Case 5 .................................................... 247
Figure 167 - Comparison of Girder C Deflection for Load Case 5 .................................................... 247
Figure 168 – Load vs. Panel Edge Strain (SG 6-9) Locations – Half-Scale Bridge (Test 1) .......... 248
Figure 169 – Load vs. Panel Edge Strain (SG 6-9) Locations – Half-Scale Bridge (Test 2) .......... 248
Figure 170 – Stress Comparison for Service II Limit State (Simple Support) All Load Cases ... 249
Figure 171 – Stress Comparison for Service II Limit State (Fixed Support) All Load Cases .... 249
Figure 172 – Plate Stress for Service II Limit State (Simple Support) All Load Cases ............. 250
Figure 173 – Plate Stress for Service II Limit State (Fixed Support) All Load Cases ............. 250
Figure 174 – Stress Comparison for Strength I Limit State (Simple Support) All Load Cases .... 251
Figure 175 – Stress Comparison for Strength I Limit State (Fixed Support) All Load Cases .... 251
Figure 176 – Plate Stress for Strength I Limit State (Simple Support) All Load Cases ............. 252
Figure 177 – Plate Stress for Strength I Limit State (Fixed Support) All Load Cases ............. 252
Figure 178 – Transverse Shear Stress Comparison for Strength I Limit State .................... 253
Figure 179 – Transverse Shear Stress Comparison for Strength I Limit State .................... 253
Figure 180 – Static Displacement Response- Girder A (Load Case A) ........................................... 256
Figure 181 – Static Displacement Response- Girder B (Load Case A) ........................................... 256
Figure 182 – Static Displacement Response- Girder C (Load Case A) ........................................... 257
Figure 183 – Static Strain Response- Girder A (Load Case A) .................................................... 257
Figure 184 – Static Strain Response- Girder B (Load Case A) .................................................... 258
Figure 185 – Static Strain Response- Girder C (Load Case A) .................................................. 258
Figure 186 – Static Displacement Response- Girder A (Load Case B) ........................................ 259
Figure 187 – Static Displacement Response- Girder B (Load Case B) ........................................ 259
Figure 188 – Static Displacement Response- Girder C (Load Case B) ........................................ 260
Figure 189 – Static Strain Response- Girder A (Load Case B) .................................................. 260
Figure 190 – Static Strain Response- Girder B (Load Case B) .................................................. 261
Figure 191 – Static Strain Response- Girder C (Load Case B) .................................................. 261
Figure 192 – Static Displacement Response- Girder A (Load Case C) ........................................ 262
Figure 193 – Static Displacement Response- Girder B (Load Case C) ........................................ 262
Figure 194 – Static Displacement Response- Girder C (Load Case C) ........................................ 263
Figure 195 – Static Strain Response- Girder A (Load Case C) .................................................. 263
Figure 196 – Static Strain Response- Girder B (Load Case C) .................................................. 264
Figure 197 – Static Strain Response- Girder C (Load Case C) .................................................. 264
Figure 198 – Static Displacement Response- Girder A (Load Case E) ........................................ 265
Figure 199 – Static Displacement Response- Girder B (Load Case E) ........................................ 265
Figure 200 – Static Displacement Response- Girder C (Load Case E) ........................................ 266
Figure 201 – Static Strain Response- Girder A (Load Case E) .................................................. 266
Figure 202 – Static Strain Response- Girder B (Load Case E) .................................................. 267
Figure 203 – Static Strain Response- Girder C (Load Case E) .................................................. 267
Figure 204 – Dynamic Displacement Response – Girder A (Load Case B) .............................. 268
Figure 205 – Dynamic Displacement Response – Girder B (Load Case B) .............................. 268
Figure 206 – Dynamic Displacement Response – Girder C (Load Case B) .............................. 269
Figure 207 – Dynamic Displacement Response – Girder A (Load Case C) .............................. 269
Figure 208 – Dynamic Displacement Response – Girder B (Load Case C) .............................. 270
Figure 209 – Dynamic Displacement Response – Girder C (Load Case C) .............................. 270
Figure 210 – Dynamic Displacement Response – Girder A (Load Case E) .............................. 271
Figure 211 – Dynamic Displacement Response – Girder B (Load Case E) .............................. 271
Figure 212 – Dynamic Displacement Response – Girder C (Load Case E) .............................. 272
Chapter 1 – Introduction

During a time when the current bridge inventory is aging and traffic demands are on the rise, the need exists for the development of new bridge technologies to offset these changes. These new technologies must improve service life and speed of construction, lower cost, and provide reserve strength for additional traffic demands. These requirements are in line with the initiatives of the United States Federal Highway Administration to reduce the number of deficient bridges in the U.S.. One technology, that appears to meet these initiatives, is a bridge deck system called the Sandwich Plate System.

1.1 – Sandwich Plate System Overview

The Sandwich Plate System (SPS) is an innovative bridge deck system that can be used for both new bridge construction and bridge rehabilitation applications. SPS consists of steel plates bonded to a rigid polyurethane core as shown in Figure 1. The SPS deck is analogous to an I-beam when subjected to flexure with the steel plates acting as the flanges and the elastomer core as the web. The steel plates are designed to resist the loads resulting from flexure while the core resists the transverse shear. A typical bridge application utilizes SPS primarily as a bridge deck acting compositely with conventional support girders, but other applications have also been given consideration. A SPS bridge deck is typically constructed from a series of panel segments, matching the width of the bridge, connected together along the span of the bridge.

![Figure 1 – Sandwich Plate System](image)

SPS is similar to a conventional orthotropic plate solution, but without the required intermediate stiffeners; the polyurethane core serves the same purpose as intermediate stiffeners by providing continuous support to the steel plates. Since the core is continuous the local buckling effect resulting from discretely spaced stiffeners is eliminated. Additionally, the lack of
intermediate stiffeners eliminates the “hard spots” inherent to orthotropic decks resulting in a more continuous system. The versatility of SPS allows for implementation in a wide variety of bridge applications including new construction, deck replacement, and deck rehabilitation. The following sections provide more thorough descriptions of the Sandwich Plate System as it relates to bridge applications.

1.1.1 – History of Sandwich Construction

The principles behind SPS are the same as for conventional sandwich construction, in which rigid face plates bound a softer, less stiff core. In conventional sandwich construction the face plates, outer layers, are primarily intended to resist in-plane and lateral loads while the core material resists transverse shear loads. The configuration of the face plates is fairly standard, but wide variations in the core structure allow sandwich panels to be tailored for specific applications [Figure 2]. Stiffness in sandwich construction is proportional to the core stiffness and as a result variations in core material, core thickness, and geometric configuration allow for section optimization without a significant increase in weight. For design purposes, consideration of the cost/benefit should be employed in the selection of the core material and geometry (Vinson 1999). The behavior of sandwich composites is similar to that of laminated composites in that each layer maintains unique properties, but must function as a unit. For this reason the application of laminate theory has often been employed in the design of sandwich structures.

![Sandwich Plate System Configurations](image)

*Figure 2 – Representations of Sandwich Construction Configurations*
When comparing the use of sandwich construction to an equivalent weight non-sandwich solution, considerable improvements in flexural stiffness and a reduction in flexural stress can be achieved with sandwich construction (Vinson 1999). The sandwich configuration increases the distance between the face plates, increasing the flexural stiffness in turn reducing the normal flexural stresses [Figure 3]. This comparison is analogous to that between a rectangular beam section and an I-section of equal weight; the flanges of the I-section increase the moment of inertia in turn reducing the peak stress. These improvements allow for a sandwich structure to be tailored for a specific application by varying the core material, core thickness, face material, and face thickness.

![Figure 3 – Comparison of Equivalent Weight Systems – Sandwich vs. Thin Plate](image)

While SPS can likely be considered the first application of sandwich construction in a bridge application, the technology of sandwich construction dates back to the World War II era. A British World War II bomber, the Mosquito, is credited with being the first application of sandwich construction in the 1940s (Vinson 1999). The design of this aircraft revolutionized the aircraft industry and was later extended to missile and spacecraft structures including the Apollo Capsule (Davies 2001). Some of the notable benefits of conventional sandwich construction include: weight savings over conventional structures, resistance to local deformation, good rigidity, thermal and acoustical insulation, good fatigue resistance, and ease of mass production (Plantema 1966). These same characteristics are what make SPS an attractive solution for use in the bridge market.
1.1.2 – Brief History of SPS

Intelligent Engineering (IE) developed and patented the SPS technology over a ten year period in collaboration with Elastogran GmbH, an affiliate of BASF. The Sandwich Plate System was initially developed for use in the maritime industry as a deck replacement system for deteriorating ship decks. After a few years IE began to branch out into the Civil Engineering industry, specifically the bridge market. Current and potential SPS applications include ship repair, new-build ship components, maritime overlays, new bridge construction, bridge deck repair/rehabilitation, stadium risers, and building floor systems (Kennedy et al. 2006). While the maritime applications provide significant insight into the behavior of SPS, the focus herein will be related to the application of SPS in the bridge sector.

When compared to conventional bridge solutions such as reinforced concrete and orthotropic systems, SPS offers a number of advantages including: weight savings due to the light weight of the panels, speed of construction, impact resistance, and acoustic damping without sacrificing strength. The design of SPS panels can be tailored to specific applications by varying the plate and core thicknesses. For new construction applications, the panels can be designed with support girders significantly smaller than a conventional solution. In rehabilitation applications, the SPS panels can be designed to work with the existing superstructure; this would typically result in additional capacity due to the reduction in dead load of the deck system.

In 2003 the Shenley Bridge was constructed in Saint-Martin, Québec, Canada utilizing a SPS deck [Figure 4]. The Shenley Bridge has a span of 73.8 ft and a transverse width of 23.3 ft and was designed by IE with detailed finite element analyses, conforming to the Canadian Bridge Design Specification (CHBDC) (CSA 2000). A total of ten prefabricated SPS panels, eight full size (7.9 ft x 23.3 ft) panels and two scaled (5.4 ft x 23.3 ft) end panels, were used in the construction of the bridge [Figure 5]. The panels were made composite with the three longitudinal support girders through slip critical bolts connecting the girder flanges to cold-formed angles welded to the SPS panels. Additionally, the panels were made continuous with slip-critical bolts on the underside between panels and transverse welds along the panel seams on the topside [Figure 5].
Figure 4 - Shenley Bridge - Quebec, Canada

a) Deck panel installation

b) Underside connections

Figure 5 – Shenley Bridge Construction
Construction of the bridge occurred over a seven day period during November 2003. The construction began with the placing of the longitudinal plate girders on the abutments and connecting the lateral bracing diaphragms. With the girders in place, the SPS panels were placed and bolted to the girders and between panel joints sequentially along the span and then welded together at the panel seams. The final steps in the construction were attaching the guardrails and applying the wearing surface.

The composite bridge system was designed for the required ultimate, fatigue, and serviceability limit states required by the CHBDC. As part of the Ministry of Transportation of Québec accreditation process a series of validation tests were required for the SPS bridge system in accordance with CHBDC requirements. The purpose of these tests was to establish the ability of an SPS bridge structure to carry the anticipated truck loads, establish the behavior of SPS bridge, and provide field data to confirm the finite element analyses.

The Shenley Bridge is of primary interest for this research effort because at the time of project inception it was the only SPS bridge application in service. Additionally, it is the only new-build SPS bridge in service to date (IE 2007).

1.1.2.1 – Current Bridge Inventory

Since the construction of the Shenley Bridge two SPS repair and rehabilitation applications have been installed to date including the Schönwasserparke Bridge (July 2005 – Krefeld, Germany) deck overlay and the Lennoxville Bridge (October 2005 – Québec, Canada) deck replacement. These projects are not within the scope of this research, but a brief discussion is presented in the following section to demonstrate the range of applicability of SPS in bridge structures.

1.1.2.1.1 – Lennoxville Bridge

The Lennoxville Bridge (IE 2006a) in Québec, Canada, utilized SPS to rapidly replace the degrading deck of a historic through-truss bridge without affecting the superstructure. An increase in traffic and loads over the bridge’s life required the bridge to be posted and the use of a conventional concrete deck would make the situation worse. The light weight of SPS panels allowed for the existing superstructure to remain and provided additional capacity due to the reduction in weight. After the old deck system was removed, SPS panels were installed and
made composite with the existing girders. The entire installation was completed within a week, during November 2005, with minimal impact on existing traffic.

1.1.2.1.2 – Schönwasserparke Bridge

The Schönwasserparke Bridge (IE 2006b) in Krefeld, Germany, served as the first application of SPS technology as an overlay for strengthening. The original bridge structure, built in 1972, employed an orthotropic deck which was in need of rehabilitation or replacement. Using SPS as an overlay system allowed for incorporation of the existing deck into the SPS configuration. A cavity for the polymer core was created by using the existing deck as the bottom plate and the polymer was injected on-site. This procedure significantly reduced the anticipated construction time of conventional solutions and provided additional fatigue resistance to the deteriorating structure. The overlay was completed in July 2005; the bridge was out of service for only five weeks.

1.1.2.2 – Materials

Of fundamental importance to the understanding of SPS behavior are the component materials and their interaction. Characterization and an understanding of these component materials provides insight into the benefits and limitations of SPS.

1.1.2.2.1 – Steel Plates

The steel used in the SPS deck is of primary interest because the majority of the load resistance is derived from the steel. ASTM A588 Grade 50 steel is used for the steel plates in the SPS panels. A588 steel is classified as corrosion resistant high-strength low-alloy steel with a minimum yield strength of 50 ksi and an ultimate tensile strength of 70 ksi (AISC 2001). The AASHTO equivalent of A588 is M270 Grade 50W. This type of steel is considered a “weathering steel” and is highly resistant to corrosion making it a suitable material for bridge applications. Under atmospheric conditions the steel develops a protective oxide coating which provides the resistance to corrosion [Figure 6]. The development of this protective coating is a function of the alloy content and environmental exposure conditions. Some of the environmental factors that affect the weathering process include: degree of exposure, atmospheric environment, and the degree of atmospheric contamination. The corrosion resistance of weathering steel is
estimated to be 2-4 times that of carbon structural steel (ISG 2004) and for this reason it has been used in many steel bridge applications.

![Figure 6 – Oxidized Weathering Steel](image)

While weathering steel does maintain a high degree of corrosion resistance there are a number of scenarios where it may not be suitable. Some of these applications include: atmospheres containing concentrated and corrosive fumes, locations subjected to saltwater spray or salt-laden fog, applications where the steel is submerged in water or buried in soil, applications where the steel is in direct contact with moisture retaining timber, and bridges located over enclosed highways where salt spray can accumulate on the steel (ISG 2004). These types of applications may hinder the development of the protective oxide coating required for corrosion resistance.

One potential issue with the use of weathering steel as the top plate of the SPS deck is corrosion due to water leaking through the protective coat of the wearing surface. This continued exposure to water, and likely salts, could lead to corrosion of the top steel plate. Corrosion of the top plate would result in a reduction in strength and stiffness of the SPS system and also expose the polymer core to the environment. While this type of corrosion is unlikely, it highlights the importance of a quality wearing surface to the performance of the SPS deck. Since the objective of this research effort focuses on the global behavior of the SPS deck, no further discussion of the steel material characteristics is provided hereafter.

1.1.2.2.2 – Sandwich Core

The primary purpose of the sandwich core in SPS is to provide continuous support to the steel plates and to prevent local buckling resulting from the steel plate slenderness. Additionally,
the core material provides resistance to transverse shear stresses. While the use of steel plate in the bridge industry is not new, the interaction of steel plates with a polymer core is, and this, warrants further discussion. Of primary interest is the behavior of the core under service and fatigue loads as well as the environmental and exposure response.

The core material used in SPS is comprised of rigid foam polyurethane elastomer. Polyurethanes are classified as a family of heterogeneous polymers which are the product of the reaction between isocyanates and hydroxy compounds (CRC 2006). The resultant polymer is a microphase separated block copolymer containing alternating blocks of hard and soft segments [Figure 7]. In microphase separation, the copolymer is divided into distinct segments of constituent monomers connected by chemical bonds. The properties of polyurethane vary significantly depending on the types of isocyanate, type of hydroxy compound, and additives used in the formulation. As a result, polyurethane can be engineered for a large number of applications ranging from clothing materials to construction materials (CRC 2006). Polyurethanes range from very soft elastomers to rigid plastics depending on polymer chain stiffness, interchain attraction, percent crystallinity and degree of cross-linking (Hepburn 2002).

![Figure 7 – Microphase Separation](image)

Polyurethanes formulated with diisocyanate/diol reactions produce a flexible, linear polymer as a result of the diol containing only two hydroxyl functional groups (-OH). This formulation does not allow branching of the polymer chain and as a result no physical cross-links are formed. This type of formulation yields a thermoplastic polyurethane. Polyurethanes of this form rely primarily on entanglements and microphase separation for the development of their mechanical properties. When subjected to temperatures higher than the polymer’s glass transition temperature, \( T_g \), this polyurethane may break down and soften.
In a formulation with diisocyanate/polyol reactions, the hydroxy compound (polyol) contains multiple functional hydroxyl groups allowing the polymer chain to branch and form a rigid cross-linked network of polymer chains [Figure 8]. This type of formulation yields a thermoset polyurethane. The mechanical properties for thermoset polyurethane are derived primarily from the rigid physical cross-links between the polymer chains, but also benefit from microphase separation and entanglements. When this type of polyurethane is subjected to high temperatures, degradation is resisted primarily by the physical cross-links. Control of the degree of cross-linking allows for variation in the mechanical properties. Closely spaced cross-links result in a rigid polymer, while an increase in spacing increases the polymer chain flexibility.

![Monomer Chain](image1)

**Figure 8 – Cross-linked Polymer Chain**

The polyurethane used in SPS is a blend of a thermoplastic and thermoset polyurethane, resulting in a rigid elastomer. In this formulation, the rigidity is provided by the cross-links and the flexibility is derived from the high percentage of linear chain segments. The elastomeric properties provide a high tolerance to mechanical stress allowing for rapid recovery from deformation (IE 2002).

In a bridge application utilizing SPS, the core material is subjected to the same static, dynamic and cyclic loads as the structural steel plates. While the core material is primarily intended to prevent local buckling and transfer shear between the faceplates, it also needs to withstand these loads over the life of the structure. A brief summary of the properties and characteristics of the polyurethane core is provided to demonstrate the core material’s adequacy for bridge applications. The material characterization was initially conducted in accordance with
ASTM or DIN standards for use in maritime applications. Measurements were taken over a wide range of temperatures as polymers tend to demonstrate dramatic changes though their glass transition region (IE 2002).

**Thermal Characteristics of SPS Polyurethane Core**

The mechanical properties of polymeric materials are often characterized by the variation in properties over a range of temperatures. Of particular importance is the temperature at which a polymer transitions from a stiff glassy phase to the soft rubbery phase; this transition temperature is referred to as the glass transition temperature, $T_g$. Typical structural applications utilize polymers over a temperature range that will not exceed their $T_g$, allowing the polymers to maintain their rigid forms and desirable mechanical properties. Structural applications of polymeric materials above the $T_g$ typically include adhesives, sealants, and coatings where the rubbery behavior is necessary.

Miscible polymers maintain a single weighted $T_g$ in between the $T_g$’s of the constituent monomers. The SPS core material is classified as an immiscible block copolymer with two distinct $T_g$’s, one for each of the two microphase separated monomers. Of importance is the relatively high $T_g$, 95°C (203°F), of the hard phase. This $T_g$ allows the core to maintain its stiffness over the design operating temperature range even though the glass transition temperature of the soft phase is only -59°C (-74°F). The soft phase provides flexibility to the core and prevents the material from becoming too brittle.

**1.1.2.2.2 – Mechanical Properties of SPS Polyurethane Core**

While the stress strain-behavior of elastomers maintain a similar appearance to other structural solids, the mechanical properties of elastomers are related to the polymer properties such as structure backbone, molecular weight, thermal properties, and degree of cross-linking through thermodynamic relationships. This difference is significant beyond the elastic limit, but not as evident in the elastic region where Hooke’s Law can still be considered applicable.

As is the case with elastomers, the SPS polyurethane core maintains different stress-strain relationships in tension and compression and as a result, different elastic moduli for each scenario. The elastic modulus is also dependent on temperature, with a significant decrease
observed at higher temperatures. As the temperature approaches the glass transition temperature of a polymer, the specimen transitions from a solid glassy phase to a soft rubber phase.

A summary of the mechanical properties for tension and compression is shown in Table 1 (IE 2002) over a range of temperatures. From these test results it is evident that the limit of the elastic strain for the core is well beyond the ±2,000 με elastic limit for A588 steel assume perfect bond, demonstrating that the core will remain elastic for typical design applications. The linear elasticity of the core material allows Hooke’s constitutive relationships to be preserved.

*Table 1 – Comparison of Tension and Compression Mechanical Properties*

<table>
<thead>
<tr>
<th></th>
<th>Tension</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>-80°C</td>
<td>-60°C</td>
<td>-40°C</td>
<td>-20°C</td>
<td>23°C</td>
<td>60°C</td>
</tr>
<tr>
<td>E (ksi)</td>
<td></td>
<td>559.7</td>
<td>424.1</td>
<td>256.0</td>
<td>168.8</td>
<td>126.8</td>
<td>63.2</td>
</tr>
<tr>
<td>σp (ksi)</td>
<td></td>
<td>5.6</td>
<td>4.3</td>
<td>4.1</td>
<td>3.3</td>
<td>2.3</td>
<td>1.2</td>
</tr>
<tr>
<td>σy (ksi)</td>
<td></td>
<td>13.0</td>
<td>11.0</td>
<td>8.6</td>
<td>6.3</td>
<td>4.9</td>
<td>3.0</td>
</tr>
<tr>
<td>εy (%)</td>
<td></td>
<td>7.2</td>
<td>11.1</td>
<td>13.2</td>
<td>15.1</td>
<td>32.1</td>
<td>43.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Compression</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>-80°C</td>
<td>-60°C</td>
<td>-40°C</td>
<td>-20°C</td>
<td>23°C</td>
<td>60°C</td>
</tr>
<tr>
<td>E (ksi)</td>
<td></td>
<td>562.5</td>
<td>408.0</td>
<td>195.4</td>
<td>169.1</td>
<td>111.0</td>
<td>72.7</td>
</tr>
<tr>
<td>σp (ksi)</td>
<td></td>
<td>7.6</td>
<td>4.9</td>
<td>4.5</td>
<td>3.1</td>
<td>2.6</td>
<td>1.5</td>
</tr>
<tr>
<td>σ10%ε (ksi)</td>
<td></td>
<td>20.8</td>
<td>14.5</td>
<td>10.4</td>
<td>8.0</td>
<td>4.6</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Additionally, tests were conducted by IE (2002) to determine the shear modulus variation with temperature and the Poisson’s ratio. Over the temperature range of -45°C to 100°C (-49°F – 212°F) the shear modulus varied from 88.3 ksi to 12.6 ksi, similar to the tension and compression test behavior. The Poisson’s ratio did not vary with temperature, as expected, and ranged from 0.36-0.38 over three different test methods.
1.1.2.2.2.3 – Characteristics of SPS Polyurethane Core

As with any structural material, the durability of the SPS core is of significant interest because the core is expected to meet or exceed the service life of the design structure. Typical physical characteristics of polymeric materials include good chemical resistance, lightweight, and thermal and electrical insulation.

The SPS polyurethane core also exhibited these physical characteristics. The core material was tested by IE (2002) for saltwater resistance, chemical resistance, tensile impact/toughness, and hardness as part of the Classification Society and Regulatory Agency approval process. A detailed discussion of the physical characteristics of the SPS core are not included as it is beyond the scope of this research.

1.2 – Project Overview and Scope

With SPS being relatively new to the bridge market, there has only been a single new application. As a result, there is no standard in place for the design using SPS, and the design was performed using a detailed finite element model. This research effort investigated the global behavior of SPS bridges with a focus on critical design parameters such as lateral load distribution, dynamic load allowance, and deck design procedures. The results of this investigation are expected to aid in the development of design recommendations for the use of SPS bridges in the United States.
Chapter 2 – Literature Review

2.1 – Literature Review Overview

Limited research has been performed on the evaluation of SPS for bridge structures with the majority of the research conducted at the Virginia Tech Structures and Materials Research Laboratory (Martin and Murray 2005). To gain an understanding of the global behavior of SPS in bridge applications, a study of its behavior under realistic conditions is necessary. This section highlights relevant literature related to global behavior, analysis and design of bridges with a specific emphasis on lateral load distribution, dynamic load allowance under live loads and deck design methodologies.

2.2 – Bridge Superstructure Design Overview

The design of bridges in the United States is governed by the American Association of State and Highway Transportation Officials (AASHTO) with additional requirements imposed by individual state governing bodies. The design process is robust and encompasses a wide variety of topics including the roadway alignment, environmental considerations, superstructure design, and substructure design. While all of these topics are critical in the design of a bridge structure, the following discussion focuses on the design of the bridge superstructure with an emphasis on slab-girder bridges (or girder bridges) as it relates to SPS.

The superstructure as defined by AASHTO includes the structural parts of the bridge that provide the horizontal span (AASHTO 2004). These structural parts include the longitudinal girders, transverse stringers, bracing and diaphragms, deck system, and guardrails. Individually, each of these components has a defined behavior, but a certain degree of complexity is introduced when these components act as a unit, as is the case for the superstructure. The primary cause of the complexity in a bridge superstructure comes from the interaction of the girders and deck system. Typical girders are characterized by the Euler-Bernoulli beam equation (Beer and Johnston 1981) \[ Eq. 1 \], with a unidirectional response. The behavior of decks is considerably more complex as described by classical plate theory (Ugural 1999) with the response coupled in orthogonal directions \[ Eq. 2 \]. The combined action of these two components results in a multi-directional response.
Current bridge design guidelines include provisions for a wide variety of bridge types, but these provisions are currently limited to established materials such as steel, concrete, and timber. While SPS is comprised primarily of steel, the AASHTO design guidelines have not been validated for this composite system and there still remains some uncertainty in the applicability of the guidelines. With SPS, the composite deck is the only significant variant from conventional design, but the influence of the deck is far reaching in the overall design process. Not only must the deck system be designed properly for strength, serviceability and fatigue, but the amount of load sharing must be determined for the design of the supporting members. This research focuses primarily on the deck design considerations, lateral load distribution behavior, and dynamic characteristics of SPS for bridge applications.

2.2.1 – Live Load Analysis

AASHTO LRFD (AASHTO 2004) provides guidelines for the design of bridges subjected to load combinations that include the effects of scenarios such as dead load, live load, earthquake, ice and collision. These load combinations allow for consideration of the limit states for strength, serviceability, extreme loading event, and fatigue. The following discussion considers only the live load scenario.

The live load for a bridge is considered to be a function of the number of design lanes, the bridge configuration and combinations of notional design trucks [Figure 9] and lane loads [Figure 10]. For design purposes, the objective is to configure the loading to produce the greatest force effect on the static system and design for this effect. Due to the multi-dimensional nature of the bridge structure, final configuration of the loading is highly variable and not always simple to determine, often becoming an iterative or trial and error process. A number of approximate and refined methods of analysis exist that allow for simplification of this process. Some of these methods are discussed briefly in the following sections.

Live load in a bridge structure is by nature in motion and of considerable mass. The effect of this large mass in motion produces a dynamic effect on the bridge structures. This
dynamic effect is considered additional to the static live load effect and can be significant. During design, this dynamic amplification of the static load must be considered as part of the design live load. A more detailed discussion of this dynamic effect is presented in 2.4 – Dynamic Load Allowance (DLA) or Impact Factors (IM).

**Figure 9 – HS-20 and Tandem Design Trucks**

![Design Lane Load](image)

**Figure 10 – Design Lane Load**

2.2.2 – Girder Design

In a slab-girder bridge system, the girders serve as the primary load bearing members of the superstructure and represent a significant portion of the overall cost. As with most structures it is in the best interest of the designer to use the lightest and least costly girders available. The AASHTO LRFD design procedures (AASHTO 2004) for girders is very similar to those observed in other design codes based on LRFD principles, requiring the design resistance to be greater than the factored load effect [Eq. 3]. In a slab-girder bridge, the load path passes through the deck before reaching the girders. The amount of load transferred from the deck to the girders is primarily a function of the lateral stiffness of the deck and the relative stiffness of the girders (Barker and Puckett 1997), and is often referred to as lateral load distribution. This load sharing
effect directly influences the girder design with more uniform load distribution resulting in smaller girders. As a result, it is critical to understand and accurately quantify the load sharing phenomenon to design an economical structure.

\[
\frac{\phi R_d}{\text{Design Strength}} \geq \sum \eta_i \gamma_i O_i \quad \text{Eq. 3}
\]

2.2.2.1 – Beam-Line Method (Distribution Factors)

To reduce the level of complexity bridges are often modeled as 2-D systems by using the beam-line method with distribution factors. The beam-line method is an approximate method of analysis that considers each beam separately subjected to a fraction of the original loading; separate consideration is often given to the response of moment and shear. Distribution factors are a representation of the amount of the total load that is transferred to a given girder from the deck. They can be determined from refined numerical or analytical analyses considering the relative stiffness of the bridge components or through simplified equations. The simplifying assumption that a bridge can be reduced to the analysis of a simple beam is a significant stretch from reality, but the use of this methodology has proven to be effective in the design of bridges for many years. More consideration of the beam-line method is given in the following section related to lateral load distribution factors (2.3 – Lateral Load Distribution and Load Distribution Factors (DF)).

2.2.2.2 – Grillage Method

The grillage method is considered to be a more accurate method of analysis than the beam-line method. Grillage assumes that the bridge system can be modeled as a skeletal grid structure with frame members [Figure 11] representing both the longitudinal and transverse behavior; the global response can then be determined by using stiffness or displacement methods of analysis. While this method is considered to be more accurate, it can still only approximate the bridge response because the displacement within the grillage is irregular and the moment is only a function of the curvature along the length of a frame member (Barker and Puckett 1997). Some of the advantages of the grillage method include the ease of use with typical frame analysis software, easily interpreted and verified results, and analysis procedures familiar to most engineers. One disadvantage is that the grillage method requires some experience in the model...
development. In addition, it is not considered to be a rigorous method of analysis because it does not exactly converge to the solution of the mathematical model of the system, but this method will not be considered further herein. Details and examples of the grillage method are available in the work by Hambly (1991).

![Grillage Method Diagram](image)

**Figure 11 - Slab-Girder to Equivalent Grillage Model**

### 2.2.2.3 – Finite Element Method

One of the most common and powerful methods for the analysis of a bridge structure is the finite element method (FEM). FEM is a numerical technique, requiring a computer, which discretizes the system into a number of elements and nodes [Figure 12]. Typical element types for structural analysis include: 1-D line elements, 2-D plane elements, 3-D frame elements, 3-D shell elements, and 3-D brick elements with different shape functions used to describe the behavior of an element. A discretized system is developed to represent the geometric configuration, boundary conditions, and applied loading of the original system. Using the principle of minimum potential energy, nodal displacements are determined for the discretized system; internal forces and stresses are then determined for these nodal displacements. A more in-depth discussion of the finite element method can be found in the work of Cook (2001).
With the improvement in processing capabilities of the computer, finite element modeling has become increasingly popular and more commonplace in structural analysis. FEM allows for an accurate representation of the actual structure under consideration, and yields accurate results with a rapid solution time when used correctly. While FEM is a powerful tool that can often speed the analysis process, it can also lead to an incorrect analysis if used improperly. Some common problems associated with the use of FEM include: a limited understanding of the principles of FEM, element incompatibility, improper boundary conditions, equivalent nodal loading error, input error, and improper solution interpretation. A further discussion of the FEM as it relates to bridge analysis and SPS is in later sections (3.1 – *Finite Element Modeling of Bridge Structures*).

2.2.2.4 – *Finite Strip Method*

A derivative of FEM is the finite strip method, which considers the bridge as a series of finite strips that span between simple supports. The principles behind the finite strip method and FEM are the same, but the former uses a special shape function that considers the boundary conditions to describe the behavior of the strips. Since this method is specific to certain boundary conditions, it will be given no further consideration; FEM is simple enough to handle these boundary conditions.

2.2.3 – *Deck Design*

In slab-girder construction, the deck serves a number of purposes including: transferring the load to the longitudinal support members, resisting transverse loads and serving as the
primary structure of the riding surface. With the girders resisting the majority of the load in the longitudinal direction, the primary structural purpose of the deck is to resist the transverse load in the bridge. This transverse load produces both tensile and compressive stresses in the deck as a result of the deck spanning between girders, similar to a continuous beam.

The majority of slab-girder bridges in the United States were constructed using reinforced concrete for the deck, with wood and steel contributing to a much lesser extent. As a result, many of the methods for deck design focus on concrete decks. The design procedures within the AASHTO LRFD (AASHTO 2004) include both empirical and traditional design methodologies for reinforced concrete decks. The traditional design methods are further divided into approximate and refined methods of analysis. Empirical methods include both the finite element and grillage methods. Approximate methods include the equivalent strip method while the refined methods include isotropic and orthotropic plate theory. A further discussion of the equivalent strip method is presented in 2.5.2.1.1 – Linear Elastic Method (Equivalent Strip), while a discussion of plate theory is included in Chapter 3.

Provisions do exist for the design of orthotropic decks, but their design methodologies incorporate the effect of the transverse/longitudinal ribs. Unlike reinforced concrete decks, orthotropic decks are designed to be integral to the resistance of the superstructure. As a result, their design is coupled with that of the main support members. AASHTO LRFD (AASHTO 2004) includes interaction equations for the deck subjected to global tension [Eq. 4] and global compression [Eq. 5]. Each equation is representative of the interaction between ratios of the applied loading to the nominal resistance in the deck and ribs. Similar to the design of reinforced concrete decks, AASHTO LRFD design procedures include approximate methods and refined methods of analysis such as grillage, finite element, and finite strip.

\[
\frac{P_u}{P_r} + \frac{M_{ur}}{M_{rr}} \leq 1.33 \quad \text{Eq. 4}
\]

\[
\frac{M_{fb}}{M_{rb}} + \frac{M_{fr}}{M_{rr}} \leq 1.0 \quad \text{Eq. 5}
\]

As is evident from the above description of design methodologies for reinforced concrete and orthotropic decks, SPS cannot be directly assigned to either category. Both reinforced concrete and orthotropic decks maintain different mechanical properties in orthogonal directions;
SPS is considered an isotropic material because both the steel plates and the polymer core are independently isotropic. When comparing SPS to reinforced concrete, the mechanical behavior is significantly different and it is inappropriate to directly apply the design provisions to SPS without validation. Similarly, the inclusion of the steel deck in the global resistance does not allow for direct consideration of the orthotropic deck design methodologies. These differences demonstrate that there are currently no provisions within the AASHTO LRFD that can be considered directly applicable to the design of a SPS deck. A further discussion of existing deck design considerations is presented in a later section (2.5 – Bridge Deck Analysis).

2.3 – Lateral Load Distribution and Load Distribution Factors (DF)

Lateral load distribution and load distribution factors (DF) describe the load sharing behavior observed in a plate elastically supported by girders. This load sharing behavior stems from the coupled longitudinal and transverse response of plates as described in Eq. 2. The end result is a composite system response versus an individual component response; the girders and deck act as a unit with the degree of interaction a function of the lateral stiffness of the deck and the relative stiffness of the girders (Barker and Puckett 1997). This phenomenon is illustrated in Figure 13 with the most heavily loaded girder undergoing the largest deflection and the remaining girders deflecting, but to a lesser degree. The largest contributor to the sharing phenomenon in slab-girder bridges has been demonstrated to be girder spacing with lesser contributions from parameters such as span length, deck thickness, and longitudinal stiffness (Zokaie 1992).
The use of distribution factors allows engineers to predict the bridge response by treating the longitudinal and transverse effects as uncoupled. In design, distribution factors are used to distribute the live loading applied to the deck between the supporting members based on their relative resistance. These distributed loads are then factored and used as the design loads for the supporting members allowing for design by conventional methods based on the girder type. This method is often referred to as the beam-line method since it takes a three-dimensional bridge system and reduces it to an equivalent two-dimensional beam. The distribution factor, often referred to as $g$ or $DF$, can be determined in a number of ways, but a general definition for the beam-line method is the ratio of the maximum response in a system ($F_{\text{refined}}$) to the maximum response of a single member ($F_{\text{beam}}$) subjected to the same loading [Eq. 6]. Slightly different versions of the beam-line method are included in both the AASHTO LRFD (AASHTO 2004) and the AASHTO Standard (AASHTO 2002) specifications as approximate methods of analysis. While the methodology is slightly different, the principles are the same allowing designers a simple method of designing support girders without using a refined method of analysis such as finite element or grillage.

$$g = \frac{F_{\text{refined}}}{F_{\text{beam}}} \quad \text{Eq. 6}$$
2.3.1 – *Lateral Load Distribution (AASHTO Standard Specification)*

The AASHTO procedures (AASHTO 1996), prior to the introduction of the LRFD specification in 1994, utilized a simple approach [Eq. 7] for lateral load distribution which was primarily a function of the longitudinal girder spacing and bridge type, where $S$ represents the girder spacing in feet and $D$ is a function of the bridge type. This empirical load distribution factor has survived since 1931 with minimal changes (Zokaie 1992) and is sometimes referred to as the “s-over” equation. The “s-over” equation has provided designers with a very simple method for lateral load distribution, but the range of applicability is limited. The original method of distribution was proposed for relatively short simple spans with small girder spacing. Also of note is that the distribution factor from the “s-over” equation is for a wheel-line or half the axle weight rather than an entire truck.

$$g = \frac{S}{D} \quad \text{Eq. 7}$$

Some of the problems with this method include an oversimplification of the bridge structure, modifications over time that led to inconsistencies in the load distribution criteria, no consideration for skew and inconsistent verification for various bridge types (Zokaie 1992). Other significant problems observed with the “s-over” equation are that it has been shown to be unconservative in some cases (>40%) and overly conservative in others (>50%) (AASHTO 1994).

2.3.2 – *Lateral Load Distribution (AASHTO LRFD Specification)*

The problems observed with the overly simplified methodology in the AASHTO Standard specification prompted a study from the National Cooperative Highway Research Program (NCHRP) to develop better methods for lateral load distribution. The research project consisted of three levels of analysis, with increasing accuracy and complexity between levels. The goal was to develop simple equations to predict lateral load distribution with improved accuracy over a wider range of applicability (Zokaie 1992). In the project, a parametric study of a large population of bridges in the United States was performed to determine the variables with the most influence on the lateral load distribution. The results indicated that the parameters with the most influence on lateral load distribution for slab-girder bridges are girder spacing, span length, longitudinal stiffness, and slab thickness. Girder spacing proved to be the most
significant parameter. These key parameters were then incorporated in the development of simplified empirical equations for the prediction of lateral load distribution and were formulated statistically based on the relative contribution of the key parameters. The final equations yielded different equations for shear, moment, interior girder, and exterior girders and also included corrections for skew. The recommendations from the study for lateral load distribution were incorporated in the AASHTO LRFD specification with slight adjustments. Representative equations for the lateral load distribution equations from the study and the AASHTO LRFD are shown in Table 2 for a steel I-girder bridge supporting a concrete deck. It should be noted that the equations already include multiple presence factors which account for the probability of trucks being present in adjacent lanes of multiple lane bridges.

| Table 2 - LRFD Lateral Load Distribution Factors for Concrete Deck on Steel Girders |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|
| Interior                        | Applicability   | Exterior        | Applicability   |
| One design lane loaded          |                 |                 |                 |
| Moment                          |                 |                 |                 |
| \( g = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12Lt_g} \right)^{0.1} \) | \(|3.5 \leq S \leq 16.0\)
\(|4.5 \leq t_s \leq 12.0\)
\(|20 \leq L \leq 240\)
\(|N_b \geq 4\)
\(|10,000 \leq K_g \leq 7 \times 10^6\) | \( \text{Lever rule}^* \) | *does not include multiple presence factor |
| Two or more design lanes loaded |                 |                 |                 |
| \( g = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12Lt_g} \right)^{0.1} \) | \(|3.5 \leq S \leq 16.0\)
\(|4.5 \leq t_s \leq 12.0\)
\(|20 \leq L \leq 240\)
\(|N_b \geq 4\)
\(|10,000 \leq K_g \leq 7 \times 10^6\) | \( g = e \cdot g_{\text{interior}} \)
\( e = 0.77 + \frac{d_x}{9.9} \) | \(|-1.0 \leq d_x \leq 5.5\) |
| One design lane loaded          |                 |                 |                 |
| Shear                           |                 |                 |                 |
| \( g = 0.36 + \frac{S}{25} \) | \(|3.5 \leq S \leq 16.0\)
\(|4.5 \leq t_s \leq 12.0\)
\(|20 \leq L \leq 240\)
\(|N_b \geq 4\) | \( \text{Lever rule}^* \) | *does not include multiple presence factor |
| Two or more design lanes loaded |                 |                 |                 |
| \( g = 0.2 + \frac{S}{12} - \left( \frac{S}{35} \right)^{2.0} \) | \(|3.5 \leq S \leq 16.0\)
\(|4.5 \leq t_s \leq 12.0\)
\(|20 \leq L \leq 240\)
\(|N_b \geq 4\) | \( g = e \cdot g_{\text{interior}} \)
\( e = 0.6 + \frac{d_x}{10} \) | \(|-1.0 \leq d_x \leq 5.5\) |
As is evident from Table 2, the simplified formula for lateral load distribution has some limits. AASHTO states that these distribution factors are limited to bridges with fairly regular geometry, constant cross section consistent with those specified, four or more beams, parallel beams with approximately the same stiffness, roadway portion of cantilever overhang ≤ 3 ft., and a small plan curvature. Similar to the AASHTO Standard specification these equations are only considered valid for bridges within the ranges of applicability and a more refined analysis may be required for bridges outside these ranges. For bridges outside the range of applicability and exterior girders subjected to a single lane loading, the lever rule can be applied for determination of an upper bound on lateral load distribution. The lever rule is a simple static distribution method that assumes all interior supports are hinged, preventing the load from being transferred across an interior support [Figure 14]. The solutions yields reactions for the supporting beams which represent the fraction of the total load that goes into that support member. This methodology would be equivalent to a very flexible deck with a low degree of rotational restraint, none provided across interior members. It should also be noted that an additional provision exists for the exterior girder in a bridge with diaphragms or cross-bracing because their contribution was neglected in the development of the distribution factor equations (AASHTO 2004). The provision states that the distribution factor for moment and shear in an exterior girder shall not be less than that from assuming the cross-section deflects and rotates as a rigid cross-section.

![Figure 14 – Lever Rule Representation of Slab-Girder Bridge](image-url)
2.3.3 – Lateral Load Distribution (CHBDC Specification)

In the United States, the prevalent bridge design codes are the AASHTO Standard specification (AASHTO 2002) and the AASHTO LRFD specification (AASHTO 2004), while the Canadian Highway Bridge Design Code (CHBDC) (CSA 2000) is the standard in Canada. The CHBDC utilizes a limit state design approach similar to that of both AASHTO specifications and was derived primarily from the Ontario Highway Bridge Design Code (3rd Ed.) in an effort to provide a unifying standard across all of the Canadian provinces. While the CHBDC is not considered applicable in the United States, it is considered relevant to this research effort because it was the standard used for the design of the Shenley Bridge, the first SPS bridge application. Even though future designs in the United States will be based on the AASHTO specifications, the behavior of the Shenley Bridge is expected to resemble that of the CHBDC rather than the AASHTO specifications. A brief summary of the methodology for lateral load distribution from the CHBDC will be presented in this section.

The procedure used in the determination of lateral load distribution factors is somewhat different than both AASHTO specifications; it is based on the principle of equal distribution with a modification factor \( F_m \) or \( F_v \) to account for the transverse variation in intensity compared to average intensity. Rather than calculate distribution factors and then apply them to shears and moments, the moment [Eq. 8] and shear [Eq. 9] for each girder are calculated directly from equations provided in the CHBDC. The maximum moment \( M_t \) and shear \( V_t \) for a simple beam, including dynamic amplification factors, are determined for both the notional CL-W truck (entire truck) and CL-W lane load separately and scaled based on the ratio of the number of design lanes \( n \) and multiple presence factor \( R_L \) to the number of girders \( N \). Details of the calculations for the amplification factors \( F_m \) and \( F_v \) are found within the CHBDC in section 5.7.1.2 (moment) and 5.7.1.4 (shear).

\[
M_g = F_m M_{g \text{ avg}} = F_m \frac{nM_i R_i}{N} \quad \text{Eq. 8}
\]

\[
V_g = F_v V_{g \text{ avg}} = F_v \frac{nV_i R_i}{N} \quad \text{Eq. 9}
\]

The CHBDC methodology is similar to the AASHTO LRFD in that it provides separate consideration for moment and shear. These considerations are included in the amplification factor calculations \( F_m \) and \( F_v \). For comparison to the AASHTO LRFD and AASHTO Standard
specification distribution factors, the CHBDC equations for moment and shear must be converted to equivalent distribution factors as shown in Table 3.

Table 3 – Equivalent Distribution Factors from CHBDC

<table>
<thead>
<tr>
<th></th>
<th>LRFD</th>
<th>Standard Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment</td>
<td>$g_{eq\ CHBDC} = \frac{M_g}{M_r} = \frac{F_w n R_L}{N}$</td>
<td>$g_{eq\ CHBDC} = 2 \cdot \frac{M_g}{M_r} = 2 \cdot \frac{F_w n R_L}{N}$</td>
</tr>
<tr>
<td>Shear</td>
<td>$g_{eq\ CHBDC} = \frac{V_g}{V_r} = \frac{F_j n R_L}{N}$</td>
<td>$g_{eq\ CHBDC} = 2 \cdot \frac{V_g}{V_r} = 2 \cdot \frac{F_j n R_L}{N}$</td>
</tr>
</tbody>
</table>

2.3.4 – Summary of Relevant Lateral Load Distribution Research

Researchers over the years have studied lateral load distribution by considering the effect of a variety of parameters. Some have attempted to check the validity of the AASHTO equations for specific scenarios while others have tried to further simplify the methodology. This section provides a brief summary of the investigations of lateral load distribution in bridges relevant to the implementation of SPS in bridge structures. The discussion of previous research are divided into three categories: 1) the evaluation of distribution factors, 2) the effects of specific variables on lateral load distribution and 3) the development of simplified methods for lateral load distribution. The following discussion provides insight into the critical factors for lateral load distribution in slab-girder bridges with the aim of developing similar approaches for SPS in bridge applications.

2.3.4.1 – Evaluation of distribution factors

The evaluation of distribution factors has been studied since their implementation in the AASHTO design specification. Field measurements, refined methods of analysis, or a combination of both have been the primary methods for evaluating lateral load distribution. A survey of recent studies will be introduced in this section with an emphasis on those comparing predictions from the LRFD specification (AASHTO 2004) and the latest standard specification (AASHTO 2002).
Prior to the inclusion of the results of the NCHRP 12-26 project results in the AASHTO specification, Tarhini and Frederick (1992) studied the effects of lateral load distribution in I-girder bridges with a concrete deck. Their study considered a wide range of variables that included size and spacing of girders, presence of cross bracing, slab thickness, span length, span continuity, and composite action. To produce the maximum loading effect, a series of HS-20 trucks were positioned on the bridge. Finite element software, ICES STRUDL II, was used in this study as the primary means of calculating accurate distribution factors. The calculated distribution factor was determined as the ratio of maximum girder moment in the bridge from finite element analysis (FEA) to the maximum moment in a single girder subjected to the wheel load. Their parametric study demonstrated that lateral load distribution was most influenced by span length and girder spacing; the former was not considered in the AASHTO Standard specification. Of note was the negligible contribution of composite action, cross-bracing, and changes in moment of inertia (either changing slab thickness or girder size). A simple proposed formula based on the FEA results was presented [Eq. 10] that included both the contribution of span length and girder spacing. A comparison to current research at the time indicated that the equation performed well in predicting lateral load distribution and could yield values considerably less than the AASHTO specification predictions. The proposed equation was later validated with comparisons to field measurements of distribution factors in a series of bridges in Ohio (Tarhini and Frederick 1995). Results from the comparisons again yielded predictions lower than the AASHTO Standard specification and within reasonable agreement to the field measurements, but the comparison was for a limited data set with fairly similar configurations.

\[
DF = 0.00013L^2 - 0.021L + 1.25\sqrt{S} - \frac{(S + 7)}{10}
\]

Eq. 10

Kim and Nowak (1997) investigated lateral load distribution factors for I-girder bridges through a series of field tests. Their study included the testing of two bridges with concrete decks supported by steel girders under normal traffic loading conditions and also a controlled loading. The two bridges under study had different girder spacing, span lengths, transverse widths, and diaphragm configurations. Their research demonstrated that accurate lateral load distribution factors can be measured from normal traffic loading without interruption of traffic by using data filtration techniques. Distribution factors calculated from measured strains were compared to predictions from the AASHTO Standard and LRFD specifications, demonstrating
that the code predictions were conservative for the cases studied. Upon comparison of the results between the bridges, it was noted that the bridges with wider girder spacing and diaphragms were able to distribute the load more uniformly to the girders. This trend would indicate the ineffectiveness of closely spaced diaphragms in load distribution.

Research by Shahawy and Huang (2001) suggested that distribution factors specified by the AASHTO Standard and LRFD specifications were overly conservative for concrete slab-girder bridges. This observation was based on field tests performed by the Florida Department of Transportation Structural Research Center. Further evaluation of the inconsistencies was performed with a finite element parametric study of 645 prestressed slab-girder bridges with variations in girder spacing, span length and overhang length. The simply supported bridges had a constant deck thickness and contained no intermediate diaphragms. Results from the analyses were used to determine lateral load distribution factors for the worst case loading scenario; these distribution factors were then compared with AASHTO LRFD predictions with varying degrees of success. The prevalent trend was that as the girder spacing and deck overhang length decreased, significant differences occurred between the model and code predictions. This trend was observed for both exterior girders with two or more design lanes loaded and interior girders with one lane loaded. Based on these trends, Shahawy and Huang proposed a series of modifications for the two cases as shown in Eq. 11 and Eq. 12. In Eq. 11, \( \phi_1 \) is an empirical deck overhang coefficient that represents the effect of overhang length and girder spacing, while \( \phi_2 \) is an empirical span coefficient that is a function of span length and girder spacing. These modification coefficients are determined graphically. The distribution factor, \( g_{LRFD} \), is from the LRFD code equations for an interior girder with 2 or more design lanes loaded. Eq. 12 was also developed empirically, but yielded closed-form solutions as function of deck overhang length and girder spacing. The equations were then validated when compared to finite element models and a series of live-load tests; a significant reduction in the percent error was observed in both validation methods.

\[
\begin{align*}
\text{Interior girder with 2 or more design lanes loaded} & \quad g = g_{LRFD} \left( 1 - \phi_1 \cdot \phi_2 \right) \quad \text{Eq. 11} \\
\text{Exterior girder with 1 design lane loaded} & \quad g = g_{LRFD} \left( 1 - \phi_1 \cdot \phi_2 \right) \quad \text{Eq. 12}
\end{align*}
\]
\[
g = 0.147 + 0.235d_e + 0.163S \text{ (for } S \leq 1.83\text{m[6 ft]})
\]
\[
g = \frac{S - 0.914}{S} (1.044 + 0.469d_e - 0.081S) \text{ (for } S > 1.83\text{m})
\]

Barr et al. (2001) performed a comprehensive investigation of live load distribution in prestressed concrete slab-girder bridges to assess the distribution factors from the AASHTO LRFD specification. The objective of the investigation was to evaluate the effects of lifts (haunches), end diaphragms, intermediate diaphragms, skew, load type, and continuity on lateral load distribution. A detailed finite element model was developed to replicate the live load behavior of a five girder, three span continuous bridge with 40° skew. Comparisons between live load test results and the finite element model were in good agreement, providing validity to the model. The full model along with a model of one of the spans, modeled as simple supports, were then compared with distribution factors from the AASHTO LRFD, AASHTO Standard specification and the Ontario Highway Bridge Design Code (OHBDC) over a range of skew angles (0-60°) with varying success. The AASHTO LRFD predictions proved to be conservative for all comparisons, but appeared overly conservative when compared to the continuous model. The AASHTO Standard specification was at times conservative and other times unconservative. While the OHBDC was consistently unconservative for the simple span model it was in good agreement with the continuous model up to its limiting skew angle of 20°. To study the individual effect of the parameters each was then included in the models of the simple span; distribution factors were determined over a range of skew angles from 0-60°. The results demonstrated that lifts and end diaphragms significantly reduced lateral load distribution while intermediate diaphragms had minimal effect. The effect of continuity was inconsistent, with some cases experiencing an increase in distribution while others experienced a decrease. Skew resulted in a decrease in all cases, but it was demonstrated that the reduction was reasonably represented by the AASHTO LRFD specification equation for skew. Distribution factors calculated for lane loading, versus a standard truck, were shown to be on average 10% lower.

Eom and Nowak (2001) studied lateral load distribution of steel girder bridges in Michigan to assess the accuracy of the AASHTO LRFD and Standard Specification predictions. The study consisted of comparisons between field tests, finite element models and code predictions. The focus of the study was on bridges with spans ranging from 33 to 148 ft (10 to
45 m) and five to twelve girders with spacing ranging from approximately 4 to 9 ft (1.2 to 2.8 m). All of the bridges had simple supports and minimal skew. For the field testing, the bridges were loaded using a 164 kip (730 kN), 11-axle truck and midspan strains were recorded and used to calculate load distribution factors for the bridges. Strain measurements were recorded for both one and two truck crossings with the superposition of two single truck crossings used to validate the linear elasticity of the two truck crossings. When comparing the field test results to code specified distribution factors, it was observed that for most cases, those specified by the code were conservative. Finite element models for each of the tested bridges were developed for comparison to the field test results, with equivalent nodal loads used to represent the applied truck loads. Due to the uncertainty in the actual condition of the supports due to corrosion, three sets of boundary conditions were considered: pin-roller, pin-pin, and partially restrained with springs. The pin-roller and pin-pin configurations served as the upper and lower bounds respectively on the strain response while the partially restrained model was continually adjusted to match actual behavior.

While the majority of the lateral load distribution research has focused on conventional materials such as steel and concrete, the growing popularity of composites in infrastructure has become more commonplace. Stiller et al. (2006) investigated the lateral load distribution of two different glass fiber-reinforced polymer (GFRP) decks supported by steel girders. The study focused on determination of the lateral load distribution and the level of composite behavior for a shear studded connection. Since no provisions exist in the AASHTO Standard specification for a GFRP deck system, the lateral load distribution behavior of the GFRP decks was treated as similar to that of a concrete deck with reduced distribution capabilities in the design of the bridges. Live load testing on the bridge allowed for calculation of lateral load distribution factors from strain measurements. The results demonstrated that the GFRP deck did not distribute the load laterally as well as a concrete deck, but the reduction was not as much as originally anticipated.

Similar research was conducted by Moses et al. (2006), but the scope included the live load testing of three bridges with a similar GFRP deck system supported on steel girders. Their research focused on evaluation of the effective width and lateral load distribution in a GFRP deck. Live load testing was performed and distribution factors were calculated from maximum strain and deflection responses observed during test. These results were then compared to
distribution factors from a static distribution ("lever rule"), the AASHTO LRFD and the AASHTO Standard specifications. The comparison demonstrated that both AASHTO specifications can be unconservative, but the "lever rule" proved adequate in describing lateral load distribution for GFRP decks similar to the ones tested.

**2.3.4.2 – Parameter specific effect on lateral load distribution**

From the AASHTO Standard specification it can be interpreted that the only parameter with a significant influence on lateral load distribution is girder spacing. This interpretation does not seem to agree with conventional wisdom because it seems obvious that other components contribute as well. The results of the NCHRP Project 12-26 confirmed that the other components do contribute, but the dominant factor remained girder spacing. Since the introduction of the AASHTO LRFD in 1994, which basically included the recommendations of the NCHRP 12-26 project, researchers have continued to study the influence of various components on lateral load distribution. This section provides a summary of the research related to the effect of various parameters on lateral load distribution.

Recommendations from the NCHRP 12-26 project for an increase in lateral load distribution values when deck and girder continuity was present were not included in the AASHTO LRFD to prevent a misleading sense of accuracy in an approximate method (Mabsout et al. 1998). Using the finite element method, Mabsout et al. studied the effects of continuity on lateral load distribution in steel girder bridges. Distribution factors were determined from deflections using two modeling techniques, 1) beam centroid centered on slab centroid and 2) beam centroid offset from slab center. The research effort illustrated that an increase in lateral load distribution is observed with the addition of continuity and recommended the inclusion of a correction factor to compensate.

The effect of secondary bridge elements such as parapets and lateral bracing has been the subject of a number of research efforts in recent years. The work performed as part of the NCHRP 12-26 project neglected the contribution from these elements and as a result their influence is not included in the AASHTO LRFD distribution factors. Conner and Huo (2006) studied the effects of parapets and aspect ratio, span/width, on lateral load distribution using results from finite element models. Their study included the evaluation of lateral load distribution for two span continuous bridges with and without skew that included variations in
the bridge aspect ratio and deck overhang length. The effect of the parapet contribution was demonstrated to reduce lateral load distribution, but this contribution decreased as the length of the overhang increased, which is similar to the trends observed by Shahawy and Huang (2001). Comparisons to the AASHTO LRFD and AASHTO Standard specifications demonstrated that their predictions are still typically conservative even though they ignore the increase in stiffness from the parapet contribution. An additional observation from the study demonstrated that the effect of aspect ratio was nonexistent for small ratios and negligible for ratios greater than 2. As a result, the effect of changes in aspect ratio was not considered to significantly affect lateral load distribution.

A study performed by Eamon and Nowak (2002) assessed the edge stiffening effects of barriers and sidewalks as well as diaphragms on bridge resistance and lateral load distribution. Their study considered the effects on a wide variety of bridge structures by varying span length, girder spacing, deck thickness, girder type (steel/prestressed), and various combinations of the barriers, sidewalks, and diaphragms. Finite element models based on previously validated techniques were used to predict bridge response. The first phase of the study consisted of the evaluation of bridge responses in the elastic range. Within the elastic range it was demonstrated that secondary elements result in a reduction in lateral load distribution; this change occurs because of a shift in the position and magnitude of the maximum moment in the bridge structure. Some notable trends observed included: 1) diaphragms tend to be more effective at wider girder spacing and longer spans with no effect observed through an increase in number of diaphragms, 2) combinations of barriers and sidewalks are more effective for closely spaced girders and longer span, and 3) the effect of all elements is more evident with increased bridge flexibility. The second phase of their study included the evaluation of secondary element in the inelastic range, specifically as ultimate capacity was approached. In this region, lateral load distribution factors were shown to decrease as a result of internal redistribution of load at and beyond yielding; this correlates directly to an overall increase in structural capacity. The secondary elements were shown to further improve the capacity since they aided in distributing the load.

The effect of secondary elements on lateral load distribution was also considered in a study by Chung et al. (2006). Typical Indiana bridges with similar girder spacing and span lengths were used in the study; some of the bridges had K-frame bracing while others had diaphragms. Their study compared field test measurements from the bridges with a series of
detailed finite element models to determine the influence of secondary elements on lateral load distribution. The models developed included a base model that neglected the secondary elements altogether and other models that considered the secondary elements separately and in combination. Similar to the work by Eamon and Nowak (2002), it was determined that the influence of secondary elements resulted in significant reductions in lateral load distribution. Chung et al. also considered the influence of longitudinal and transverse cracking on lateral load distribution by comparing base FE models without cracking to FE models that included the effects of cracking. The results indicated that even though transverse cracking resulted in reduced deck stiffness and increased deflection, the lateral load distribution was not significantly affected. Longitudinal cracking was shown to increase lateral load distribution up to 17% in the cases considered, but when considered in conjunction with secondary elements it is still overestimated by code predictions.

2.3.4.3 – Development of simplified methods for lateral load distribution

Since their inclusion in the AASHTO LRFD specification, the lateral load distribution factors proposed by Zokaie (1992) have become widely accepted and shown to be an improvement over those from the AASHTO Standard specification. Even so, researchers have continued to evaluate their performance and consider ways to improve them, sometimes offering new methods. The primary reasons for the attempts to develop new methods relate to minimizing the iterative process required in the initial design phase and expansion of the limits of applicability.

Phuvoravan et al. (2004) considered methods for simplifying the AASHTO LRFD distribution factors because they require an iterative solution in the design phase. The term representative of the longitudinal stiffness, $K_y$, is typically unknown in the initial stages of design and requires an initial guess that is to be checked after the member is selected. Phuvoravan et al. developed a simplified equation [Eq. 13] based on the original equations that eliminated the variables requiring iteration. The longitudinal stiffness parameter was eliminated by curve fitting the relationship with span length using a known data population. The curve fit was conservative for all the data, ensuring the simplified equation would also be equivalent or more conservative than the AASHTO LRFD equation. Slab thickness was also eliminated by assuming it to be 8 in. based on typical Indiana bridge configurations. In addition, corrections
were made to the skew correction factor because it includes the longitudinal stiffness term $K_g$. Comparisons to field test results, finite element models, and code equations indicated that the simplified equation resulted in conservative distribution factors for the bridge population in the NCHRP 12-26 project and a small population of bridges in Indiana.

$$LDF = g = 0.15 + 0.73 \left( \frac{S}{L} \right)^{0.8} e^{\frac{L}{590}} \quad \text{(US Units)}$$

$$LDF = g = 0.15 + 0.042 \left( \frac{S}{L} \right)^{0.8} e^{\frac{L}{180,000}} \quad \text{(SI Units)} \quad \text{Eq. 13}$$

As the Illinois Department of Transportation (IDOT) transitioned from the AASHTO Standard to the AASHTO LRFD specification, a group of IDOT engineers investigated the effect of the changes on typical Illinois bridges (Tobias et al. 2004). Part of the investigation included analyzing the influence on lateral load distribution. A parametric study was first performed to provide an understanding of the influence of variables such as span length, beam spacing, ratio of longitudinal to transverse superstructure stiffness, degree of skew, and beam location (interior/exterior). From this study it was determined that the influence of the ratio of the longitudinal stiffness to transverse stiffness term, $K_g / (L \cdot t_i^3)$, was negligible for typical Illinois bridges and could be eliminated. This determination was similar to that of Phuvoravan et al. (2004), but rather than curve fitting to eliminate the term, a constant value was recommended. The constant value was based on typical results for given support girder types and assumed to be 1.02 for concrete deck on steel girders, 1.10 for prestressed I-girders, and 1.15 for bulb-tees. This simplification eliminated the need for iteration in the initial design phase.

Huo et al. (2004) proposed the consideration of a modified form of Henry’s method for lateral load distribution due to its simplicity. The original form of Henry’s method was developed by the Tennessee Department of Transportation (TDOT) in 1963 as simple method for determining lateral load distribution based on equal distribution principles. The roadway was divided by an equivalent lane width, reduced by a multiple presence factor, and finally distributed based on the number of beam lines. An additional modification was included for steel and prestressed I-beams. An evaluation of 24 in-service bridges was performed by determining the lateral load distribution as predicted by Henry’s method, AASHTO LRFD, AASHTO Standard specification, and finite element modeling. The bridges were categorized by
supporting member type with variations in span length, transverse width, beam spacing, slab, continuity, skew, and overhang length. Henry’s method was shown to perform reasonably well when compared to the other prediction methods, but adjustments to the multiple presence factor based on supporting member type resulted in better agreement. The proposed method was shown to be more conservative than the AASHTO LRFD methods without a significant gain in value.

Puckett et al. (2006) performed an extensive investigation, similar to that of Zokaie (1992), to develop an improved set of equations for determining distribution factors for moment and shear. Their study included a wide variety of bridge types, including those investigated by Zokaie, and also considered a wide range of analysis methods. The objective of the study was to develop new equations that are simpler to apply than the current provisions and have a wider range of applicability while also maintaining an analytical basis. While the current provisions of the AASHTO LRFD are considered an improvement over the overly simple provisions of the AASHTO Standard specification, they are often considered difficult to implement, especially in the initial design stage when little information is known about the planned structure. The approach employed within their investigation focused on developing simple models capable of accurately predicting lateral load distribution. These methods included modified versions of the lever rule [Eq. 14] and Henry’s Method [Eq. 15]. For the case of an interior girder subjected to a single lane loading, the distribution behavior predicted by a parametric formulation [Eq. 16] was in better agreement with the rigorous results. The variation in response for this loading scenario was attributed to the transverse response being much more localized than for the other scenarios. The calibration constants for the proposed method are presented in Table 4. Similar methods were also developed for shear distribution factors, but will not be discuss herein. It should also be noted that these simplified formula are under review for inclusion in the AASHTO LRFD as replacements for the current equations. For this reason, these equations will be considered in the evaluation of SPS lateral load distribution behavior.
One lane loaded for moment (also multiple loaded lanes for shear)

\[ m g_m = m \gamma_s \left[ a_m \left( g_{\text{lever rule}} \right) + b_m \right] \geq m \left[ \frac{N_{\text{lanes}}}{N_g} \right] \]  

Eq. 14

Multiple lanes loaded for moment

\[ m g_m = m \gamma_s \left[ a_m \left( \frac{W_c}{10N_g} \right) + b_m \right] \geq m \left[ \frac{N_I}{N_g} \right] \]  

Eq. 15

Alternative method for interior moment – one lane loaded (Parametric formula)

\[ m g_m = m \gamma_s \left[ a_m g_{PF} + b_m \right] \geq m \left[ \frac{N_{\text{I}}}{N_g} \right] \]  

where

\[ g_{PF} = \left( \frac{S}{D} \right)^{\text{Exp}1} \left( \frac{S}{L} \right)^{\text{Exp}2} \left( \frac{1}{N_g} \right)^{\text{Exp}3} \]  

Eq. 16

Table 4 – Proposed Calibration Constants for NCHRP Project 12-62

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Girder Location</th>
<th># Lanes Loaded</th>
<th>( a_m )</th>
<th>( b_m )</th>
<th>( \gamma_s )</th>
<th>Exp1</th>
<th>Exp2</th>
<th>Exp3</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lever Rule</td>
<td>Interior</td>
<td>1</td>
<td>0.53</td>
<td>0.19</td>
<td>1.04</td>
<td></td>
<td></td>
<td></td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>Exterior</td>
<td>1</td>
<td>0.97</td>
<td>-0.24</td>
<td>1.04</td>
<td></td>
<td></td>
<td></td>
<td>NA</td>
</tr>
<tr>
<td>Uniform</td>
<td>Interior</td>
<td>2 or more</td>
<td>1.17</td>
<td>-0.08</td>
<td>1.04</td>
<td></td>
<td></td>
<td></td>
<td>NA</td>
</tr>
<tr>
<td>Distribution</td>
<td>Exterior</td>
<td>2 or more</td>
<td>1.14</td>
<td>-0.12</td>
<td>1.06</td>
<td></td>
<td></td>
<td></td>
<td>NA</td>
</tr>
<tr>
<td>Parametric</td>
<td>Interior</td>
<td>1</td>
<td>1.53</td>
<td>-0.17</td>
<td>1.04</td>
<td>0.40</td>
<td>0.18</td>
<td>0.00</td>
<td>42.30</td>
</tr>
<tr>
<td>Formula</td>
<td>Exterior</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NA</td>
</tr>
</tbody>
</table>

As shown in the aforementioned research on lateral load distribution, the majority of the investigations have focused on conventional structures such as concrete deck on steel or prestressed concrete stringers. Most researchers have come to agree that the parameter with the most influence on lateral load distribution is girder spacing with some influence from other parameters such as span length, longitudinal stiffness, and secondary elements. There has been no specific indication that the deck type significantly influences lateral load distribution. This leaves room for the possibility that lateral load distribution for a SPS deck system can be determined with the existing provisions or a modification thereof.
2.3.5 – Load Distribution Factors for Shear

The majority of the research on lateral load distribution has been focused on distribution factors for moment with very little devoted to distribution factors for shear. The AASHTO Standard specification (AASHTO 2002) does not include specific distribution factors for shear in the design process and were assumed equal for both shear and moment. Provisions to account for shear were included in the AASHTO LRFD (AASHTO 2004) based on the recommendations of the NCHRP 12-26 project. Prior to the NCHRP 12-26 project, very little research had been performed on the evaluation of shear distribution factors. One of the biggest issues with the evaluation of the lateral distribution of shear through live load testing is with recording support reactions. Typical bridges are not instrumented with load cells at the support; therefore most analysis is typically conducted through scaled lab measurements or using finite element models (Barr and Amin 2006).

For typical rolled steel I-beams, shear failure is not often a controlling limit state and customary design procedures are to design for flexure and check for shear (Segui 2003). This is not the case with plate girders where the shear strength is a function of the slenderness parameter \((h/t_w)\) and intermediate stiffener spacing. The required shear strength is essential for member design in the case of a plate girder and as a result, so is the lateral load distribution for shear. The SPS bridge deck can be used in conjunction with either roller I-beams or plate girders (Shenley Bridge) and as a result the consideration of lateral load distribution for shear warrants further discussion. The following is a discussion of the relevant research related to lateral load distribution for shear; the research is limited, but is expected to provide some insight in the issues related to shear.

A study by Barr and Amin (2006) on distribution factors for shear in I-girder bridges. Their study included a comparison of distribution factors calculated from reactions of a full scale laboratory specimen, finite element models, and AASHTO LRFD specifications. The laboratory specimen was a three girder, simply supported single lane bridge with a 40 ft span, 11 ft width and a 6 in. reinforced concrete deck. The AASHTO LRFD accurately predicted the distribution with variations in girder spacing and span length, but were unconservative for the exterior girder for certain overhang lengths; this was similar to the findings of Shahawy and Huang (2001). Also of note is that the AASHTO LRFD skew correction factors performed reasonably well for the exterior girder, but were overly conservative for the interior girder.
Similar to previous work for flexure (Huo et al. 2004), Huo et al. (2005) examined the use of Henry’s method for determining distribution factors for shear. Their study concluded that the basic form of Henry’s method produced unconservative results when compared to validated finite element results and required modifications to improve the accuracy. These modifications were developed empirically based on the type of bridge and also included a correction for skew. The most notable trends from this study were that girder spacing is the most significant parameter in shear distribution factors and span length has little or no effect. Results from their study indicated that the modified Henry’s methods did not perform better than the existing AASHTO LRFD methods for lateral load distribution of shear.

Ebeido and Kennedy (1995) performed a study that considered the effects of skew, girder spacing, aspect ratio and diaphragms on the lateral load distribution for shear in composite bridges. Analytical results for finite element models were compared with laboratory test results from six scale bridges to validate the performance of the models. Using the same finite element modeling techniques, a parametric study was performed by varying the parameters under investigation. Their results indicated that skew was a major contributor in the lateral load distribution for shear. Interior girders and girders closest to the acute corner experienced an increase while girders closest to the obtuse corner experienced a decrease; both of these trends were further amplified at skews greater than 30°. The effect of girder spacing on shear distribution factor was shown to vary depending on the loading configuration. Eccentric loadings resulted in a decrease in distribution factor with an increase in girder spacing whereas a uniform load yielded the opposite trend. The later is in agreement with the results from Huo et al. (2005). An increase in aspect ratio resulted in decrease in distribution factor for all skew angles with a more substantial decrease observed for skew angles above 30°. The effect of diaphragms was investigated as a ratio of the transverse stiffness to the longitudinal stiffness with an increase in the ratio resulting in an increase in distribution in the girders closest to the obtuse corner and a decrease in the interior girder and girder closest to the acute corner. Ebeido and Kennedy presented empirical equations for calculating shear distribution factors for interior girders, a girder close to obtuse corner and a girder close to the acute corner, each with variations for skew angle. The empirical equations were further subdivided in a concentric load case, an eccentric load case, and a dead load case.
2.3.6 – Summary of Lateral Load Distribution

In the design of slab-girder bridges, methods for determining the lateral load distribution to supporting members is essential for the design of adequate and economical structures. Refined methods of analysis such as FEM and grillage have been proven to be the most accurate in predicting these distributions, but are often time consuming and not practical in the design phase. Simplified methods of distribution have become the norm for design and have proven to be adequate, but in many cases yield overly conservative or unconservative results and are deemed inapplicable for a wide number of bridges. For these reasons researchers have continued to study lateral load distribution with the goal of developing new and improved methods for prediction.

For this research effort, existing methods and research will be considered for the evaluation of the lateral load distribution behavior of SPS in bridge applications. The objective is to either assess the validity of existing provisions for SPS or develop a new method applicable to SPS.

2.4 – Dynamic Load Allowance (DLA) or Impact Factors (IM)

The response of a bridge structure subjected to live load is primarily a function of the interaction of the deck and supporting girders with some influence from secondary members. When this live load is in motion, the response is significantly increased above that of the same static load. This increased response is referred to as dynamic amplification and is the result of a number of factors including the dynamic characteristics of the bridge, the roadway roughness, and the dynamic characteristics of the load or truck (Barker and Puckett 1997). These factors are a general categorization of the critical parameters of influence; research has shown the list of parameters to be much more extensive. A summary of these findings is presented in the following sections.

2.4.1 – Source of amplification

Paultre et al. (1992) are credited with performing one of the most extensive literature reviews on bridge dynamics and dynamic amplification. Their review examined historical research efforts on the subject of bridge dynamics from numerous countries around the world and highlighted the critical factors from each. Much of the research highlighted was
instrumental in the development of worldwide bridge design codes including the AASHTO and OHBDC codes. One issue observed within the study was the difference in predictions from the various design codes worldwide; parameters critical to dynamic amplification were not consistent between the various codes. Paultre et al. provided a summary of the critical factors affecting dynamic amplification.

The most notable parameter was considered to be the fundamental frequency of the bridge; typical highway bridges were found to have frequencies similar to the resonant frequencies of commercial vehicles, 2-5 Hz. Roadway roughness was also found to have a significant influence on dynamic amplification because a rough surface tends to induce vibration in the vehicle which amplifies the force effect. Other parameters of influence included vibration resulting from tire pressure and vehicle suspension. On the contrary, conventional construction materials and geometry were not seen to have a significant influence on dynamic amplification; this is of relevance to this research because the limited applications do not allow for SPS to be categorized as a conventional construction material.

Paultre et al. (1992) suggested that the best method for the investigation of dynamic effects is through full-scale testing under traffic or controlled truck load because analytical methods cannot accurately account for all of the parameters involved in the bridge-vehicle interaction. However, analytical methods were shown to be capable of predicting vibration frequencies with good accuracy (Cantieni 1984), a key parameter in the dynamic response of a bridge.

2.4.2 – Dynamic Amplification – Experimental Methods

Due to the complex nature of bridge-vehicle interaction it is often difficult to quantify all of the parameters involved in the relationship. This is especially true in experimental methods, as certain parameters are often difficult to measure. Researchers that utilize experimental work usually try to focus on one or a few specific parameters that influence the dynamic response of a bridge structure. This section highlights a few of these works.

One of the parameters with the most influence on dynamic amplification is known to be roadway roughness. As a vehicle travels along a rough roadway surface, vibrations and bounce are introduced into the vehicle; while traveling across the bridge this vehicle motion increases the loading on the structure as it accelerates in the vertical direction. Park et al. (2005) studied
the effects of roadway roughness on dynamic amplification on a series of bridges in Korea. In
their study, the roadway profile was measured with surveying tools along the length of the span
in the traveling lane. A rough linear correlation between dynamic amplification with roadway
roughness was observed, demonstrating that an increase in roughness results in an increase in
dynamic amplification. Linear relationships were observed for comparisons to both the
roughness coefficient and the international roughness index. Park et al. acknowledged that the
relatively low correlation coefficients ($R^2$) demonstrated that the parameters were directly
related, but were not the only parameters in the relationship. No other parameters were
considered in this study.

In companion papers by Brady et al. (2006) and Brady and O’Brien (2006), the influence
of vehicle speed on dynamic amplification was studied by considering one and two vehicle
crossings. The studies included comparisons of an analytical point load model, field testing and
detailed finite element models. The analytical model was primarily used to evaluate general
behavior. The results demonstrated that the dynamic amplification is a function of the ratio of
the load circular frequency to the bridge first circular frequency, with the magnitude a function
of damping. Comparison of the field test results to the finite element models demonstrated
reasonable agreement, but required a complicated representation of the roadway surface and
loading vehicle. A comparison of an analytical point load model to the FE model proved
ineffective as the magnitude of the dynamic response could not be replicated, even though trends
were similar. The most significant trend observed in these studies was that the maximum
dynamic amplification occurred at a number of critical frequency ratios, implying that there are a
number of critical velocities for a given bridge structure. In addition, it was shown that the
dynamic amplification for two vehicles is less than that of a single vehicle; this suggests that the
maximum static and dynamic effects for the multi-lane loaded case do not occur simultaneously.
This observation was also in agreement with results from the study by Paultre et al. (1995).

As the interest in composite materials has continued to increase, researchers have begun
to investigate their behavior for use in bridge applications. The use of fiber reinforced polymers
(FRP) in bridge structures has allowed for the design of lighter structures when compared to
conventional materials, without sacrificing strength and durability. The reduction in weight and
lower damping allows for the potential of an increased dynamic response, or increased dynamic
amplification. A study by Aluri et al. (2005) focused on the dynamic behavior of three FRP
bridges through field testing. All of the bridges investigated utilized a FRP deck, but three different support structures were used including a truss structure, FRP stringers, and steel plate girders. The goals of the study were to determine if the dynamic behavior was significantly different than that of conventional materials, whether design code provisions were adequate for FRP systems, and if vibrations were within acceptable limits. The dynamic amplification for both the truss and plate girder bridges were within the AASHTO limits, but the FRP stringer bridge was significantly outside the limits. The excessive dynamic amplification in the FRP stringer bridge was attributed to the decreased stiffness of the FRP stringers as compared to the steel stringers in the other bridges. Also noted was that the measured damping ratios for the GFRP bridges were much lower than those presented in literature (Paultre et al. 1992) for bridges made of conventional materials; this lower level of damping was determined to cause excessive vibrations measured during testing.

2.4.3 – Dynamic Amplification – Analytical Methods

Paultre et al. (1992) commented that the use of analytical models such as the finite element method cannot account for all of the parameters involved in dynamic behavior of bridges. With the improvement in computing technology and finite element software researchers have been able to develop much improved models. These models are often able to consider a large number of the parameters highlighted in the discussion by Paultre et al.; the models are usually validated by some form of experimental testing for completeness. This section highlights some of the research on dynamic amplification using analytical modeling methods.

Using purely analytical models, Huang et al. (1993) performed a parametric study on the dynamic behavior of multi-girder concrete bridges. The parameters investigated during the study included load position, vehicle weight, number of loaded lanes, transverse rigidity and girder spacing. In their study the bridge was modeled as a linear grillage system with a roughened surface and subjected to a non-linear vehicle model. The effect of load position was demonstrated to have a dramatic effect on impact as the least heavily loaded girders experienced the largest amplification. Variations in vehicle weight were shown to significantly affect dynamic response with larger amplifications observed for lighter vehicles; the additional effect diminishes for heavier vehicles. The effect of number of lanes loaded appeared to be contrary to other literature in that an increase in dynamic effect was observed with more trucks on the
bridge. For short span bridges an increase in transverse rigidity was shown to more evenly distribute load and as a result dynamic amplification increased in adjacent girders. An increase in girder spacing was shown to result in a decrease in dynamic response. The trend is analogous to the heavier truck scenario because the wider girder spacing results in a larger distribution, increasing the load in a given girder and lowering the dynamic effect. In general, all of the effects considered were shown to have some effect on dynamic amplification, but an increase in span length significantly reduced these effects in all cases. This study provided some interesting insight into some of the factors affecting dynamic amplification, but is a purely analytical model without experimental validation.

Zhu and Law (2003) investigated the dynamic behavior of orthotropic plates subjected to moving loads. Their study was based on analytical models for orthotropic plate theory and energy principles, but focused specifically on bridge structures. The analytical models were first validated with data from other research and were then extended to a slab-girder bridge by considering the entire system as an equivalent orthotropic plate. Their study included an investigation of the effect of load positioning, vehicle speed, and multi-lane loading on dynamic response. The load positioning study demonstrated a commonly observed phenomenon; the members further from the loading were less influenced, but experienced a higher amplification. Zhu and Law suggested that this phenomenon was directly related to the magnitude of the torsional rigidity ($D_{xy}$) with respect to the transverse rigidity ($D_y$) of the structure. The effect of multi-lane loading was shown not to affect dynamic response while vehicle speed was shown to increase it; the effect of vehicle speed was highly dependent on positioning. In an attempt to further simplify the model, an equivalent beam model was developed for comparison. The beam model provided reasonable response predictions along the deck centerline, but underestimated the response along the edges.

Most research related to dynamic response in bridges has been focused on the response of the supporting members with little attention given to the deck. Broquet et al. (2004) performed a parametric study of the dynamic behavior of concrete bridges with a specific focus on the dynamic response within the deck. Finite element models were developed for both the bridges and the loading vehicle. The bridge model included a representation for the roadway surface and the vehicle model utilized a series of lumped masses, springs and dampers to allow for consideration of the bridge-vehicle interaction. Parameters considered included vehicle speed,
vehicle mass, roadway surface roughness, and bridge cross-section type. Broquet et al. concluded that of the parameters considered, roadway surface roughness was the only one with a significant influence on the dynamic response with a bridge deck. They also suggested that these results be further validated with field testing as the study was conducted entirely with finite element modeling.

2.4.4 – Dynamic amplification in other bridge types

Typically research on dynamic response to live load has focused on slab-girder bridges, but Laman et al. (1999) studied the dynamic response for through-truss bridges subjected to live loads. In this configuration the truss and stringers share in load carrying. Their study considered the dynamic response of three through-truss bridges with consistent configurations subjected to various test trucks. The testing indicated that the dynamic response of the most heavily loaded members is less than code provisions. Also noted was that the dynamic amplification of the truss members was similar to that of the stringers.

Similarly, Huang (2005) investigated the behavior of half-through arch bridges. In through arch type bridges the arch structure serves as the primary supporting system for the deck, resulting in the elimination or limiting of longitudinal stiffening requirements. This configuration makes arch type bridges more suitable for longer span bridges and significantly different than slab-girder construction. Of importance for this study is that in an arch configuration the ratio of the live load-to-dead load is relatively larger when compared to that of slab-girder bridges. The results from study indicated that the dynamic amplification for the through arch bridges were in line with that of slab-girder bridges in magnitude, but predominantly controlled by span length and rise-to-span ratio.

A study by Zhang et al. (2003) investigated the dynamic response of composite concrete-steel cellular bridges using analytical models. The model consisted of a 3-D finite element model of the bridge with the vehicle represented as pairs of concentrated forces moving along the deck surface. The vehicle representation neglected both the mass and dynamic behavior of the truck and assumed constant contact with the deck surface. The parameters that proved to be most influential in the study were fundamental frequency of the bridge, span length, vehicle speed and vehicle positioning. Impact factors for moment, shear, and deflection from the model indicated that code provisions significantly overestimated impact in some cases and
underestimated it in others. The authors proposed upper bound equations for moment, shear and deflection based on first fundamental frequency and also on span length similar to those in early versions of the OHBDC and the AASHTO Standard specification, but more representative of the smaller response for the composite concrete-steel cellular bridges studied.

A recent study performed by Kwasniewski et al. (2006a) utilized detailed finite element models to analyze bridge vehicle interaction. In their study, detailed models for the bridge and truck were developed to allow for simulation of a vehicle crossing a bridge structure. This study differed significantly from those of other researchers in that the truck model was extremely sophisticated compared to the spring-mass-damper models developed in previous studies. Some of the detail within the truck model included 3D representations of the tires and contact, 3D suspension components, and calibration of the truck model based on measured acceleration histories. Results from the model were shown to provide good agreement with experimental results from field tests. The main drawback to such a representation is that it required the use of a supercomputer.

2.4.5 – Dynamic amplification for design

Design code provisions in the United States and Canada have adopted a simple approach to estimating dynamic amplification effects. These methods allow designers to quickly determine dynamic amplification without detailed knowledge of the bridge configuration. The simple methods do not account for the parameters that significantly influence dynamic amplification; these methods can also be overly conservative for some cases. The simplified methods for the AASHTO Standard specification, the AASHTO LRFD, and the CHBDC are summarized in Table 5. A predecessor of the CHBDC (CSA 1988) utilized an approach based on the fundamental frequency of the bridge as shown in Figure 15. The methods presented are on a percent basis and represent the additional load, above the static response, due to the dynamic amplification response as shown in Eq. 17, where $U_{L+1}$ represents the live load effect including dynamic amplification (i.e. shear or moment), $U_L$ is the live load effect, and $IM$ is the dynamic amplification represented as a fraction.

$$U_{L+1} = U_L (1 + IM) \quad \text{Eq. 17}$$
Table 5 – Summary of Dynamic Load Allowance (Impact Factors)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Component</th>
<th>IM</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO Standard</td>
<td>All components – all limit states</td>
<td>( \frac{50}{L + 125} ) (100%) ≤ 30%</td>
</tr>
<tr>
<td>AASHTO LRFD</td>
<td>Deck joints – all limit states</td>
<td>75%</td>
</tr>
<tr>
<td></td>
<td>All other components</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fatigue and fracture limit states</td>
<td>15%</td>
</tr>
<tr>
<td></td>
<td>All other limit states</td>
<td>33%</td>
</tr>
<tr>
<td>CHBDC</td>
<td>Deck joint</td>
<td>50%</td>
</tr>
<tr>
<td></td>
<td>Where only one axle of CL-W truck is used (except deck joints)</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>Where any two axles of CL-W truck, or axles 1,2, and 3, are used</td>
<td>30%</td>
</tr>
<tr>
<td></td>
<td>Where three axles of CL-W truck, or axles 1,2, and 3, or more than three axles are used</td>
<td>25%</td>
</tr>
</tbody>
</table>

Figure 15 – 1988 CAN/CSA DLA vs. Frequency

2.4.6 – Summary of dynamic load allowance

As part of the design load, the dynamic amplification is critical to the design of a slab-girder bridge. Researchers have demonstrated that the dynamic amplification can primarily be attributed to the interaction between the traveling vehicle and the bridge. Parameters such as
bridge fundamental frequency, roadway roughness, vehicle speed and vehicle suspension dynamics were shown to be the parameters with the most influence on dynamic amplification. Using both experimental and analytical techniques, researchers have studied bridge dynamics by focusing on some of these parameters, but the underlying theme is that none of these parameters can be considered alone.

In the evaluation of SPS for bridge structures, the initial focus will be on the applicability of existing design provisions. It is expected that due to the low weight of the system, the dynamic response on a SPS bridge will be somewhat different than that of conventional systems such as a concrete deck on steel girders. The investigation of DLA will focus primarily on the frequency of SPS bridge structures to assess whether the low weight of the deck system significantly influences the dynamic characteristics.

2.5 – Bridge Deck Analysis

When comparing conventional bridge superstructure designs for systems such as a concrete deck on steel or concrete girders to the design of a SPS bridge superstructure, the primary differences arise as a result of the deck behavior and deck-girder interaction. These differences primarily include the deck design, connections within the deck, connections of the deck to the girders, and the transmission of loads from the deck to the girders. Outside of these differences, the design of a SPS bridge follows conventional practice. This section presents an overview of the key issues related to the design of a bridge deck with a focus on those that related to SPS decks.

2.5.1 – SPS Deck Design

For the majority of bridges in North America, concrete is the material of choice for bridge decks. The use of concrete is primarily a function of cost considerations, ease of construction, material availability and previous experience. Other deck systems include wood plank, orthotropic steel systems, and open/filled steel grid systems, but these systems are typically utilized for specific applications. In recent years there has also been a strong interest in the use of structural composite deck systems such as modular fiber reinforced polymer (FRP) decks, but the majority of these decks have been utilized as one-time applications, typically as part of a validation program. The limited application of FRP bridge decks appears to be
primarily a function of high initial material cost, limited design experience base and a lack of design provisions.

With the exception of composite decks, current provisions exist for all of the deck systems previously highlighted. While all of these systems represent viable alternatives to concrete, their use is typically application specific. Numerous commonalities exist between a SPS deck and these alternative deck systems, but the behavior of SPS cannot be directly accommodated by any of the current provisions. When comparing a SPS deck to these conventional systems, the form can be most closely compared to an orthotropic steel deck. A SPS bridge deck can be considered analogous to an orthotropic deck in that it utilizes metal face plates, but differs dramatically in that the main structural resistance of a SPS deck is derived from the face plates and core, whereas the orthotropic deck derives its resistance from the deck plate and discretely spaced stiffening elements acting together as a unit. The continuity of the core allows SPS deck to maintain isotropic material properties. When compared to FRP deck systems, the polymer core material of SPS provides the common characteristics between the two. While there are some underlying similarities between these deck systems, the primary distinction lies in the isotropy of the SPS deck because most composite deck systems exhibit orthogonal mechanical properties as a result of their manufacturing processes.

AASHTO design provisions focus primarily on the design of concrete deck systems, but do provide recommendations for the design of steel and aluminum orthotropic decks. These orthotropic deck systems are likely the most similar to SPS because of the deck is comprised of metal plates, but at the same time these systems are dramatically different in that the support for the metal plates in an orthotropic system is provided by a grid of structural members whereas the plates in SPS are supported by the polymer core [Figure 16].
2.5.2 – AASHTO Deck Design Considerations

As a result of the multitude of bridge types, there is no single design method available for the design of bridge decks. The AASHTO design specifications (AASHTO 2002; AASHTO 2004) allow for any method of analysis that satisfies the requirements of equilibrium and compatibility and also utilizes the stress-strain relationships for the material under investigation. Both AASHTO design specifications provide methods and guidelines for the design of typical deck types including reinforced concrete, wood, steel grid, steel orthotropic and aluminum orthotropic. This section presents a brief summary of the design methodologies recommended for the design of these deck systems. In the following summary the design methodologies will be divided and classified based on bridge type and applicable design methodology.

2.5.2.1 – Decks in Slab-Girder Bridges

With slab-girder bridges representing the vast majority of bridges in the United States, the AASHTO design provisions provide more guidance for the design of this type of deck than for less common systems. The types of decks that fall into this category include cast-in-place reinforced concrete, precast/post-tensioned concrete, steel open grid, prefabricated glulam wood,
stressed laminated wood, and spike laminated wood decks. Design of these types of deck systems is typically accomplished utilizing one of three available methods including the linear elastic (equivalent strip) method, yield-line method, or the empirical method (Barker and Puckett 2007). Each of these methods is considered applicable to decks in slab-girder bridges, but the resulting deck designs vary significantly between the methods.

2.5.2.1.1 – Linear Elastic Method (Equivalent Strip)

The equivalent strip method reduces the deck design down to that of a one-way slab section supported (or strip width) on rigid supports [Figure 17]. The equivalent strip methodology allows for direct analysis of the local effect within the deck by eliminating the global effects resulting from the stringer deflection. This local effect within the deck can be attributed primarily to bending in the span direction as a result of wheel loads on the deck between supports (Barker and Puckett 2007).

Strip widths [Table 6] are typically a function of girder spacing or deck span and are intended to include the effects of flexure in the secondary direction and torsion (AASHTO 2004). The force effects are determined from the controlling load case by treating the slab section as a continuous beam over rigid supports. A design load per unit width is determined by distributing the determined force effect over the strip width, allowing for the section to be designed using conventional methods.

Figure 17 – Equivalent Strip Representations
A similar approach is employed in the Standard specification (AASHTO 2002), but the design loads for unit width section are calculated directly with the equations provided based on the effective span length [AASHTO Section 3.24]. A further description of the Standard specification method will not be presented due to its similarity to the equivalent strip method used in the LRFD specification.
2.5.2.1.2 – Yield-Line Method

The yield-line method for deck design is based on the assumption that the system behaves inelastically and has adequate ductility to allow plastic hinges to form prior to failure. Failure is assumed to occur when a failure mechanism forms [Figure 18]. Yield-line analysis has often been used to design reinforced concrete slabs, but the solution often requires a trial and error approach. The resulting solution is highly dependent on the selected failure pattern and yields an upper-bound to the true failure load, meaning that the calculated load to cause failure will be equal to or greater than the true failure load (MacGregor and Wight 2005). This potential for overestimation can yield an unconservative deck design. While this approach is valid for design, it is only considered relevant for ultimate and extreme limit states and would require other methods to ensure serviceability. For these reasons and the low probability of forming a collapse mechanism in a SPS deck, the yield-line method will not be considered further.

![Figure 18 – Generic Yield-Line Failure Mechanism](image)

2.5.2.1.3 – Empirical Method

The empirical method is used solely for the design of reinforced concrete decks, deck sizing and reinforcement selection, without design validation so long as the structure satisfies the design criteria specified within the AASHTO LRFD specification [AASHTO Section 9.7.2]. This method is based on the theory that wheel loads are resisted by internal arching actions as a result of complex internal membrane stresses and has been demonstrated to provide a factor of safety of over 8 when compared to measured data (AASHTO 2004). While this method of analysis is
valid for reinforced concrete deck design, it cannot be extended to other deck systems and will not be considered further.

2.5.2.2 – Bridges with Orthotropic Type Bridge Decks

While the majority of bridges in the United States utilize concrete decks, a number of cases exist where concrete is not a viable option due to weight requirements. These bridges often employ lightweight systems such as steel/aluminum orthotropic, and grid decks (filled/partially filled and unfilled composite with reinforced concrete deck). As a result of higher material strength, these decks utilized less material and in-turn yield lighter decks. The drawbacks to these systems are that they can be rather costly due to material cost and labor, labor intensive and can also be prone to fatigue failures.

2.5.2.2.1 – Orthotropic Decks

Steel orthotropic decks have been used in numerous long span bridges in the United States and around the world as a result of their effective load carrying capacity and light weight. Unfortunately, many of these decks have been prone to fatigue failures as a result of welding details subjected to significant stresses. These fatigue issues have occurred more frequently in the more structurally efficient closed-rib configurations than in the open-rib configurations [Figure 19]. The AASHTO LRFD does not directly provide a method for design of orthotropic decks, but rather specifies design criteria and critical considerations for the design.
2.5.2.2.2 – Grid Decks

For grid deck systems other than those with no concrete, the AASHTO LRFD specification provides equations for determining orthogonal design moments based on orthotropic plate theory (Higgins 2003). Additionally, the specification allows the designer to use other methods of analysis such as orthotropic plate theory or equivalent grillage.

2.5.2.3 – AASHTO Limit States

When considering the design of a bridge deck, the AASHTO specifications require that the limit states considered in the design process include checks on strength, serviceability, fatigue/fracture, and extreme event. The strength limit state requires adequate resistance to the applied loads from the structural member. Whereas the service limit state places restriction on stresses, deflections, and crack widths. The fatigue and fracture limit state restricts stresses resulting from a design to ensure adequacy over the life of a bridge. The extreme event limit state ensures the survival of a bridge structure during major events such as earthquake, flood, and collision. While these limit states are valid for all bridge types, the focus herein will be on the limit states of strength, serviceability and fatigue. The extreme event limit state is beyond the scope of this research.
2.5.2.3.1 – AASHTO Strength Limit State

Currently the AASHTO specifications do not provide significant guidance with respect to the strength design of deck systems with the exception of reinforced concrete decks. The guidelines provided for grid decks and orthotropic decks are mainly related to determining force effects and detail requirements. As a result, none of these provisions will be considered applicable to the strength design of a SPS deck.

2.5.2.3.2 – AASHTO Serviceability Limit State

The AASHTO serviceability criterion for bridge decks is a topic that has received much attention, but all aspects of these criteria are not fully understood (AASHTO 2002; AASHTO 2004). Serviceability criteria include durability, inspection considerations, maintainability, ride quality, and deformation or deflection and are intended to provide experience-related provisions that may not be accounted for in other limit states. These limit states are of importance for bridge decks because they can control, especially for lightweight and metal deck systems. The following section provides a brief summary of the key deflection limits presented in both of the AASHTO specifications.

2.5.2.3.2.1– Deflection Limits

The deflection limit state stipulated in the AASHTO (2002; 2004) specifications have evolved over the years, but can be traced back to the railway bridge specification of 1871. Modifications were made by the Bureau of Public Roads in the 1930’s to the form that is present in the current code provisions (Wright and Walker, Mertz, AASHTO 2004). The development of these criteria was based on limiting vibrations to minimize user discomfort, but did not provide a measure of user comfort.

Over the years, these criteria have continued to be included within the AASHTO specifications with minimal modification primarily as a result of satisfactory performance. Current AASHTO (2004) provisions have made these deflection limits optional for all members with the exception of orthotropic decks, metal grid decks, and other lightweight metal and concrete bridge decks. Within the AASHTO Standard (2002) specification these limits are not made explicitly optional, but recommended. The deflection limits presented in the AASHTO LRFD are intended for comparison to the controlling static live load increased by the dynamic
load allowance of 33%. The controlling load case is specified to be the larger of the design truck alone or 25% of the design truck along with the lane load. The Standard specification maintains similar provisions, but limits the loading to that of a HS-20 truck amplified by the variable impact \[ Eq. 18 \]. A summary of the relevant deflection limits is presented in Table 7 as a function of L, the span length in the direction of interest.

\[
\text{IM} = \frac{50}{L + 125} \leq 0.30
\]

\[ Eq. 18 \]

<table>
<thead>
<tr>
<th>Member</th>
<th>AASHTO LRFD (R) = required</th>
<th>Sections</th>
<th>AASHTO Standard</th>
<th>Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete and Steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simple or continuous spans</td>
<td>L/800a</td>
<td>2.5.2.6.2</td>
<td>L/800</td>
<td>8.9.2; 10.6</td>
</tr>
<tr>
<td></td>
<td>L/1000a*</td>
<td></td>
<td>L/1000*</td>
<td></td>
</tr>
<tr>
<td>Cantilever arm</td>
<td>L/300a</td>
<td>2.5.2.6.2</td>
<td>L/300</td>
<td>8.9.2; 10.6</td>
</tr>
<tr>
<td></td>
<td>L/375a*</td>
<td></td>
<td>L/375*</td>
<td></td>
</tr>
<tr>
<td>Steel box, tube, and I-sections</td>
<td>*limiting stress (applicable to I-sections only)</td>
<td>6.10.4.2</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Metal grid decks and other lightweight metal and concrete decks</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• No pedestrian traffic</td>
<td>L/800 (R)</td>
<td>9.5.2</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>• Limited pedestrian traffic</td>
<td>L/1000 (R)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Significant pedestrian traffic</td>
<td>L/1200 (R)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Orthotropic Bridge</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Ribs, beams, girders</td>
<td>N/A</td>
<td>2.5.2.6.2</td>
<td>L/500</td>
<td>10.41.4.8.1</td>
</tr>
<tr>
<td>• Deck plate between ribs</td>
<td>L/300 (R)</td>
<td></td>
<td>L/300</td>
<td></td>
</tr>
<tr>
<td>• Ribs</td>
<td>L/1000 (R)</td>
<td></td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>• Ribs (relative deflection between adjacent ribs)</td>
<td>0.10 in. (R)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Simple or continuous spans</td>
<td>L/425</td>
<td>2.5.2.6.2</td>
<td>L/500</td>
<td></td>
</tr>
<tr>
<td>• Wood planks and panels (relative deflection between adjacent edges)</td>
<td>0.10 in.</td>
<td></td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

*pedestrian use

2.5.2.3.3 – AASHTO Fatigue Limit State

The fatigue and fracture limit state within the AASHTO specifications are intended to limit crack growth and prevent fracture in steel elements, components, and connections resulting
from the repetitive loading over the design life of the bridge (Barker and Puckett 2007). When considering steel structures, fatigue is of great concern primarily as a result of the welded and bolted connections required to provide continuity in the structure. The connections are typically adequate in strength, but can be prone to fatigue cracking and failures as a result of the stress concentrations (Boresi and Schmidt 2003; Salmon and Johnson 1996). The following section presents the fatigue limits within the AASHTO specifications.

2.5.2.3.3.1 – Fatigue and Fracture Limits

The AASHTO LRFD and the Standard specifications provide different means of determining the fatigue limit, but both limit the allowable stress range based on the design detail category and number of anticipated cycles. The limits within the Standard specification are presented as an upper bound whereas the LRFD utilizes a S-N curve with distinct behavior categories, infinite and finite life. The LRFD approach can be represented in equation form [Eq. 19] where \((\Delta F)_n\) represents the nominal fatigue resistance, A is the detail category constant, N is the number of stress-range cycles, and \((\Delta F)_{TH}\) is the constant amplitude fatigue threshold stress. A summary of the Standard specification limits is presented in Table 8. It should be noted that in addition to the limiting stress range, the Standard specification also limits the maximum stress based on the material type and shape of the base section [AASHTO Section 10.32].

\[
(\Delta F)_{n} = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2} (\Delta F)_{TH}
\]

Eq. 19

The AASHTO LRFD limits for fatigue are intended to safeguard against a truck that is assumed to produce a loading that is two times that of the design truck. This scaled fatigue truck loading is accounted for by reducing the fatigue resistance by \(\frac{1}{2}\) [Eq. 19]. The applied load, design truck with 14 ft rear axle spacing, is increased by a dynamic load allowance of 15% and subjected to the fatigue load factor of 75% to account for the low probability of the exclusion vehicle being representative of daily traffic on the bridge (Barker and Puckett 2007). The Standard specification maintains similar provisions, but limits the loading to an unfactored HS-20 truck.
### Table 8 – AASHTO Standard Specification Allowable Fatigue Stress Ranges

<table>
<thead>
<tr>
<th>Category</th>
<th>100,000 cycles</th>
<th>500,000 cycles</th>
<th>2,000,000 cycles</th>
<th>&gt; 2,000,000 cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R</td>
<td>N</td>
<td>R</td>
<td>N</td>
</tr>
<tr>
<td>A</td>
<td>63 (49)c</td>
<td>50 (39)c</td>
<td>37 (29)c</td>
<td>29 (23)c</td>
</tr>
<tr>
<td>B</td>
<td>49</td>
<td>39</td>
<td>29</td>
<td>23</td>
</tr>
<tr>
<td>B’</td>
<td>39</td>
<td>31</td>
<td>23</td>
<td>18</td>
</tr>
<tr>
<td>C</td>
<td>35.5</td>
<td>28</td>
<td>21</td>
<td>16</td>
</tr>
<tr>
<td>D</td>
<td>28</td>
<td>22</td>
<td>16</td>
<td>13</td>
</tr>
<tr>
<td>E</td>
<td>22</td>
<td>17</td>
<td>13</td>
<td>10</td>
</tr>
<tr>
<td>E’</td>
<td>16</td>
<td>12</td>
<td>9.2</td>
<td>7</td>
</tr>
<tr>
<td>F</td>
<td>15</td>
<td>12</td>
<td>12</td>
<td>9</td>
</tr>
</tbody>
</table>

R – Redundant load path structure
N – Nonredundant load path structure
\(^c\) - unpainted weathering steel designed according to FHWA technical advisory
\(^d\) - transverse stiffener welds on girder webs or flanges

The AASHTO (2002; 2004) specifications contain numerous limits on fatigue, but this section only highlights those specifically related to steel members as this is the primary component in SPS decks. Attention will also be given to connection limits as highlighted in the work by Martin and Murray (2005).

#### 2.5.3 – Considerations for SPS Deck Design

Of primary interest to this research program is the development of design recommendations for SPS decks in bridge applications. Based on the current AASHTO specifications, no provisions exist that can accommodate SPS bridge decks. The configuration of a SPS bridge deck allows for some overlap with some conventional systems such as orthotropic and grid decks, but the isotropic mechanical properties of SPS result in dramatically different behavior. As a result of these differences, current AASHTO provisions will not be extended to the design of SPS, but the limits imposed on these deck systems will be considered in the development of design recommendations. Of specific interest will be the strength,
serviceability, and fatigue limit states. As previously highlighted, the extreme event limit state will not be considered herein.

When evaluating potential design criteria for SPS decks the following limits were considered: 1) control of deformations, 2) control of first yield or initiation of cracking, 3) allowance for limited spread of yield through the depth, 4) allowance for localized penetration of yield to the mid-plane of the plate (Aalami and Williams 1975) and 5) shear failure of deck subjected to a patch loading. While criteria 3) and 4) are valid for the analysis of a deck, an effort will be made to ensure that these are not the critical failure modes since the elastomer core does not provide significant strength in bending (IE 2002). The formation of plastic hinges in the structure would likely result in an abrupt collapse mechanism. For these reasons the design procedure for SPS decks will focus on limiting deformations/deflections and controlling first yield and initiation of cracking to meet serviceability criteria with additional checks on strength criteria. While criterion 5) is not expected to limit the design as a result of the low anticipated shear stresses carried by the core (Zenkert 1995), this assumption will be considered and validated in the development of the design criteria for SPS decks.

The main difficulty that arises when evaluating SPS is the variation in material properties through the depth of the plate. The control of deflections is stiffness based and does not require knowledge of component dimensions. However, dimensions are required for the determination of component stresses. This variation does not allow for a direct correspondence between the deflection/deformation limit and that for the control of yielding and initiation of cracking. To accommodate this difficulty, the approach that will be used in the development of SPS design recommendations will be to select a stiffness to limit deflections and then size the plate components to prevent yielding and cracking. A further description of the proposed design approach is presented in Chapter 4.

2.6 – Literature Review Summary

For SPS to be considered a viable bridge solution in the United States, an understanding of its global behavior is essential. Of particular importance is the lateral load distribution behavior and dynamic response to moving loads; as both of these parameters are essential in the design of the primary load carrying members. Also of importance is the development of a simple design method for the deck panels. Researchers have studied all of these parameters for a
wide variety of bridge types and configurations, but none have evaluated the behavior of SPS. The review of relevant literature presented in this chapter is primarily aimed at highlighting the critical parameters of influence on lateral load distribution and dynamic amplification and also the key issues for the design of SPS decks.
Chapter 3 – Analytical Models

With the Shenley Bridge being the only SPS bridge in service to date, there are limited opportunities for experimentally investigating the global behavior of SPS bridges. As a result, there is a need for analytical models capable of predicting the global response of SPS bridges. These analytical models are extended to the determination of lateral load distribution and dynamic amplification characteristics of SPS bridges. In addition, these models are utilized in the analysis of SPS decks to develop deck design recommendations. The work presented in this chapter focuses on the finite element method and a stiffened plate approach for investigating the global behavior of SPS bridges, but the majority of the emphasis is on the finite element method.

3.1 – Finite Element Modeling of Bridge Structures

Finite element analysis of structures, in general, has become more commonplace in recent years, primarily due to improved computer performance and the availability of commercial software packages. In the analysis of bridges FEM has, in many instances, taken the place previously reserved for the grillage method. When applied to bridges, FEM can serve as both an analysis and design tool and does not require an estimation of equivalent section properties as in the grillage method. Typically, slab-girder bridges have been designed using conventional methods such as the beam-line method with distribution factors, but these methods have often yielded overly conservative results (Eom and Nowak 2001; Kim and Nowak 1997) and in turn, uneconomical designs. The use of FEM has been proven more accurate than the beam-line method, even when simplified models are considered. The pitfall is that improper use of the FEM can also yield incorrect results.

The following sections do not include an extensive literature review on FEM, as this can be found in most textbooks on the subject, but will focus on literature and methods used for the analysis of bridges.

3.1.1 – Simplified Models vs. Detailed Models

In developing a finite element model, the generation of the geometric model is often the most time consuming. Model generation is typically carried out through a graphical user interface, a series of commands in an input file or some combination of the two. This requires
advanced planning of the model. The addition of more details to the model directly correlates to additional time spent in the model development phase. As a result, it is in the best interest of the modeler to make use of symmetry and include only elements that contribute to the behavior of interest. When considering the global behavior, the objective is to develop the simplest model that can accurately represent the global response; this is not necessarily the case when considering local behavior where the elimination of components can affect accuracy.

3.1.2 – Slab-Girder Bridge System

In the evaluation of slab-girder bridges researchers have developed models that include only the slab and girder while others have developed models that include secondary elements such as diaphragms, bracing, parapets and wearing surface. The selection of elements to include is highly dependent on the behavior of interest. For an analysis of lateral load distribution it has been shown that the primary factors that influence this phenomenon are girder spacing, span length, deck thickness, and longitudinal stiffness, whereas secondary elements such as a wearing surface are not typically modeled. Research has shown that parapets result in an edge stiffening effect and end diaphragms help distribute load. When considering dynamic response of a structure, mass, stiffness and geometry have been demonstrated to be important as they relate directly to the natural frequency of the structure. As a result, it is prudent to include elements that are considered to be direct contributors to the dynamic response or at a minimum their mass and a representation of their stiffness.

3.1.3 – Slab-Girder FEA Models

The majority of three-dimensional modeling techniques for slab-girder bridges have utilized the eccentric beam, detailed beam, or solid modeling techniques for the analysis. The eccentric beam formulation is the simplest of the methods because it considers the deck as either shell or solid elements and the beam as an offset beam element connected to the deck with rigid link elements to ensure composite action. The detailed beam utilizes a similar representation for the deck, but the girder flanges and web are represented by shell and/or beam elements; rigid links are also used to ensure composite action between the deck and beam. Solid modeling utilizes solid (brick) elements to represent the entire structure and ensures composite action through node connectivity.
Researchers have considered many variations of these formulations. Some of formulations used in the analysis of slab-girder bridges are presented in Figure 20. The elements used in these formulations are fairly standard in typical finite element software. Comparisons of the different idealizations have been shown to yield different results (Chan and Chan 1999) with the most accurate results coming from the representations most similar to the actual bridge configuration. These differences are typically a result of geometric or compatibility errors between connected elements within the models (Chung and Sotelino 2006). In addition, the level of complexity in model generation can vary dramatically from one idealization to the next. The eccentric beam model is the simplest followed by the detailed beam model and finally the solid modeling technique.

Chan and Chan (1999) proposed an idealization which considered the girder and the portion of deck just above the girder as one member and developed an element based on compatibility conditions. The models developed performed well when compared to laboratory measurements and other FE models and proved more accurate than the grillage and semi-continuum methods. Their model was also in agreement with the eccentric beam idealization presented by Memari and West (1991). Since the element developed is not included in typical FEA software, the authors developed their own program for the analysis of slab-girder bridges using this element.
Chung and Sotelino (2006) performed a study on the various modeling techniques in an attempt to identify the sources of incompatibilities and geometric errors. Their study focused primarily on the detailed beam and eccentric beam models by considering idealizations that utilized shell elements for the deck and shell and/or beam elements for the girders connected with rigid links. Solid elements were not considered for the deck due to their higher computational cost; use of these elements was expected to require multiple element layers through the deck for an accurate analysis due to their internal linear strain variation. Their comparisons considered four variations in the girder model (Figure 21); the girder was analyzed as a series of shells (model 1), shell web with beam flanges (model 2), beam web with shell flanges (model 3), and a beam element for the entire member (model 4). The two models that were shown to produce the most accurate results when compared to a known solution were models 3 and 4. Model 4 was considered to be the most efficient because it was the simplest to
develop and yielded accurate results. Models 1 and 2 were shown to have an incompatibility between the degrees of freedom (DOF) of the interconnected elements; this arises from the incompatibility between a shell “drilling” DOF and a bending DOF of a connected shell or beam element. The incompatibility within models 1 and 2 disappeared at higher levels of mesh refinement, but also at a higher computational cost. An additional incompatibility between axial displacements of the shell and beam elements was also observed; this incompatibility was a result of the constraint elements (rigid links) enforcing rigid rotation between elements with different order shape functions. The authors concluded that quadratic beam elements would solve this issue, this was also the recommendation proposed by Gupta and Ma (1977). Models 3 and 4 along with a detailed solid model were compared to laboratory test results; each model yielded results in agreement with test data, but the computational cost was significantly lower for model 4. The authors recommended the use of this model type for the analysis of slab-girder bridges based on its simplicity and the fact that no accuracy was lost with the simplification.
Other researchers (Barr et al. 2001; Chan and Chan 1999; Shahawy and Huang 2001; Zokaie 1992) have used variations of the modeling techniques described above with the general configuration remaining the same, deck modeled with shell elements and beams modeled as shell and/or beam elements. Others (Eom and Nowak 2001; Tarhini and Frederick 1992) have utilized solid modeling techniques, but these have been proven to be no more accurate than the simplified models and at an increased computational cost. One of the goals of this research is to develop simplified models capable of predicting the global behavior of a SPS bridge, for this reason only model 4, as proposed by Chung and Sotelino (2006), will be considered herein. This model has been proven to be accurate and with a low level of complexity in development.
3.1.4 – Finite Element Software

Numerous finite element software packages are available on the market today, each with various analysis capabilities. Some of the software is categorized as finite element software, but in reality are structural analysis packages with some finite element capabilities. For this research the ANSYS software package was selected because of its wide acceptance, availability and fairly simple pre-processing interface. ANSYS has an extensive library of elements, numerous solver options, and is capable of both linear and non-linear analyses. The capabilities of ANSYS are more than adequate for representation of the behavior of a bridge.

3.1.5 – Element Selection

Proper element selection is critical to the development of a good finite element model. The elements selected have to be capable of representing the behavior of the members they represent and compatible with the elements they are in contact with. Often, element validation is one of the first steps in the development of a finite element model. Most software packages have already performed some sort of validation on an element, but it is possible that the parameters for validation differ significantly from the intended application.

Within this study the primary elements of consideration are shell elements for the bridge deck, beam elements for the girders, and constraint elements for composite action. A more detailed investigation of element type is presented in 3.2.1 – Element Selection and Validation.

3.1.6 – Equivalent Nodal Load

One difficulty with the use of finite element methods for analysis is the proper application of loads to the system. For FEM, the analysis is performed with respect to the nodal degrees of freedom; this means that all of the applied loadings and boundary conditions must be applied to the nodes. Most finite element software programs do not have a simple method for applying loads within an element boundary; so, without a high mesh density of a model it becomes a challenge to apply loads to the structure unless they happen to fall on a node. Many researchers, however, have failed to comment on their load application method while others (Chen 1995; Conner and Huo 2006) have used the concept of tributary area to distribute the loads from within an element to the nodes. Eom and Nowak (2001) used a linear distribution technique based on the relative distance of the load from a given node. While these methods
appear to be different in form, they actually yield the same resultant and are statically equivalent, meaning that the moment is the same about an arbitrary point. This is the same premise upon which consistent or work-equivalent loads is based; this method assumes that the load is distributed amongst the nodes based on the element shape function (Cook et al. 2001).

The use of equivalent nodal loads based on shape functions allows for the applied load to be uncoupled from the mesh configuration. Loads can be of any form and distributed properly by using work equivalence principles (Eq. 20 and Eq. 21).

$R_c = \int_S N^T t \, dS \quad \text{Eq. 20}$

$R_c = \sum_{i=1}^{K} N_i^T \, p_i \quad \text{Eq. 21}$

Chung and Sotelino (2006) considered the load from a tire to be a uniformly distributed patch loading; this patch was then further discretized into a series of uniformly distributed concentrated loads. This research will focus primarily on concentrated and patch loadings representative of truck wheel loads.

3.1.7 – Boundary Conditions

Proper selection and application of boundary conditions is essential to the accuracy of a finite element analysis. Boundary conditions are especially important when the comparisons are made to known results; but even more so when no results are available for comparison. The latter may be the case for a parametric study using the finite element method. Improper selection of boundary restraints can lead to either an under or over constrained model and incorrect results.

Typical analysis procedures consider restraints in a simplified form such as pins, rollers, and fixed, but in actual structures these conditions do not truly exist. In reality the degree of restraint is somewhere in between that of the theoretical idealizations. This difference automatically induces some error in the model because it is often very difficult to quantify the degree of fixity. In addition, the location of the restraint is of importance because theoretical models typically assume a different location of restraint than actual conditions. An example of this is in elastic beam theory for a simply supported beam; here the assumption is that the beam is simply supported at the neutral axis, but in an actual structure the beam is likely to be
supported on the bottom (Figure 22). It has been reported that slight differences in boundary conditions can have considerable effects on the result of an analysis (Bakht and Jaeger 1988; Schulz et al. 1995). Bakht and Jaeger demonstrated that girder restraint can reduce live load moments by up to 20%.

![Figure 22 – Comparison of Beam Boundary Condition Representations](image)

Similar to loading conditions, many researchers have failed to comment on the location of supports within their models. Others (Mabsout et al. 1998) have neglected this effect altogether and assumed the supports to be at the centroid of the beam member. This technique introduces an error in the results when compared to actual conditions, especially when comparing to field measurements where supports are at the bottom of the girders. To combat this error, some researchers have utilized various techniques to apply the boundary conditions at the actual locations within the real structure (Chung et al. 2006; Chung and Sotelino 2006; Eom and Nowak 2001; Samaan et al. 2002). When utilizing the detailed beam model or solid modeling this can be accommodated by applying the necessary restraint on the nodes of the element at the bottom of the structure. For the eccentric beam model, this requires the addition of a dummy node or element at which the restraint can be applied. This dummy node is then tied back to the rest of the model, typically with constraints.
3.1.8 – Summary of Finite Element Modeling for Slab-Girder Bridges

The finite element has been proven to be a powerful tool for the analysis of bridge structures. Researchers over the years have successfully utilized various finite element methods and techniques for the analysis of slab-girder bridges; their success has been based largely on the improvements in modeling techniques as FEM has become more common practice. This research will draw from these experiences and lessons for the development of similar finite element models for SPS bridges. The goal is to develop simple models capable of representing the global behavior of a SPS bridges.

3.2 – Development of Finite Element Model for SPS

With little published research and test results on SPS, the use of finite element modeling is crucial to understanding the global behavior of a SPS bridges. The following section discusses the development of simplified finite element models capable of analyzing the global behavior of a SPS bridge. The simple models are then compared to measured lab results and field measurements for validation. An additional validation of the simplified models is presented in Chapter 5 – Lateral Load Distribution of SPS Bridges.

3.2.1 – Element Selection and Validation

Within most finite element software packages there are numerous element types each with their own limits of applicability. To develop a simple finite element model it is important that the elements selected are capable of mimicking the true behavior of the structure. To create the simplest model only the deck and the girder members were considered. In addition to these members, a mechanism to provide composite action between the deck and girders was also needed.

3.2.1.1 – Deck Element

With the deck of a SPS bridge being very thin, the logical element to use would be a shell element; a solid element could also be used, but would require a large number of elements to maintain a reasonable aspect ratio. For this reason shell elements were the only elements considered for the deck. Within ANSYS there exist a large number of structural shell elements with various capabilities, relevant shell elements are shown in Table 9.
Table 9 – Comparison of ANSYS Shell Elements

<table>
<thead>
<tr>
<th>Element</th>
<th>Description</th>
<th>Capabilities</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHELL 63</td>
<td>4 node element with 6 DOFs per node</td>
<td>Elastic shell with bending and membrane capabilities</td>
<td>Mindlin*</td>
</tr>
<tr>
<td>SHELL 91</td>
<td>8 node element with 6 DOFs per node</td>
<td>Non-linear layered structural shell capable of modeling sandwich structures</td>
<td>Mindlin</td>
</tr>
<tr>
<td>SHELL 93</td>
<td>8 node element with 6 DOFs per node</td>
<td>Structural shell with bending and membrane capabilities</td>
<td>Mindlin</td>
</tr>
<tr>
<td>SHELL 99</td>
<td>8 node element with 6 DOFs per node</td>
<td>Linear layered structural shell capable of modeling sandwich structures</td>
<td>Mindlin</td>
</tr>
</tbody>
</table>

* special formulation to exclude shear deformations

Prior to the selection of the primary element to represent the deck, the performance of each element was evaluated versus a known solution based on plate theory. The plates were all assumed to be clamped on all edges, have the same material through the thickness, and be subjected to a uniform pressure on the plate surface. A comparison of the performance for each element with various aspect ratios is shown in Figure 23. From the comparison it is evident that all of the elements are capable of predicting the behavior over a wide range of aspect ratios. The SHELL 99 element was chosen to represent the SPS deck because it allows for representation of each layer of the sandwich plate separately without the need to determine equivalent mechanical properties of the entire plate. Both the SHELL 91 and the SHELL 93 would have also been adequate; these elements are used for modeling the deck plate behavior in Chapter 4 – SPS Deck Design.
3.2.1.2 – Girder Element

Numerous elements have been used successfully to represent girders including line (beam), shell, and solid elements. The work of Chung and Sotelino (2006) demonstrated that line elements adequately represent the behavior and are also the easiest to implement. For this research only line elements were considered to represent the girders. Similar to the shell elements previously discussed, there are a number of beam elements within ANSYS with various capabilities, the relevant elements are listed in Table 10.

Table 10 – Comparison of ANSYS Beam Elements

<table>
<thead>
<tr>
<th>Element</th>
<th>Description</th>
<th>Capabilities</th>
<th>Beam Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEAM 4</td>
<td>2 node element with 6 DOFs per node</td>
<td>3-D Elastic Beam</td>
<td>Euler-Bernoulli</td>
</tr>
<tr>
<td>BEAM 188</td>
<td>2 node element with 6 DOFs per node</td>
<td>3-D Linear Finite Strain Beam: includes shear deformations</td>
<td>Timoshenko</td>
</tr>
<tr>
<td>BEAM 189</td>
<td>3 node element with 6 DOFs per node</td>
<td>3-D Quadratic Finite Strain Beam: includes shear deformations</td>
<td>Timoshenko</td>
</tr>
</tbody>
</table>
The primary difference between the BEAM 4 and the BEAM 18X family of elements is that the former does not include the effects of shear deformation while the latter does. When considering members with thin sections such as plate girders, this effect could be substantial. Additionally, the BEAM 18X elements allows for the use of the Section tool within ANSYS; the section tool allows for consideration of the actual element shape as defined by the user. The section tool also creates a sub-mesh within each beam element, allowing for a more detailed analysis of the member behavior. ANSYS (2004) recommends that the slenderness ratio ($\frac{\text{length}}{\text{area}}$) for the member be greater than 30 for proper convergence of the solution; the cases considered in this research exceed this value. For these reasons the BEAM 18X elements were selected over the BEAM 4 element for this research.

When selecting between the BEAM 188 and 189 elements, the only significant difference is that the formulation for the latter includes an additional node in the middle of the element. The BEAM 188 element also has an option for an additional node at the element center, but this node is considered internally in the formulation and is not used in the model generation. The solutions for the two elements match when the additional node is included for the BEAM 188 element. For this research the BEAM 188 element has been selected because of its simplicity, the generation of a two-node element is more direct than that for a three-node element and the difference in behavior can be accommodated by including the additional node option.

### 3.2.1.3 – Composite Connection Element

There are a number of techniques available to model the composite action between the deck and the girders. The most tedious is through the use of coincident nodes, where the deck elements and girder elements are aligned and modeled such that the nodes for the deck also serve as the nodes for the girders. This method typically introduces some errors because the node are typically located at the element centroid making it difficult to properly represent the geometry; this difficulty can be accommodated by using offset nodes within the elements, but this is also complicated.

Another option is through the use of constraint equations within the model. This technique utilizes master-slave relationships between the element degrees of freedom without a physical connection. The multi-point constraint element (MPC 184) utilizes these same principles, but creates a physical connection between the nodes of interest using a rigid element
with no mass. MPC 184 elements allow for full composite action between the deck and the girder as the elements deform and rotate as one unit Figure 24, but this can also introduce error into the model due to incompatible shape functions (Gupta and Ma 1977). This incompatibility is eliminated by introducing a center node in the beam formulation (Miller 1980) as in the BEAM 18X elements. For this research the MPC 184 elements are used to model the composite action between the deck and the girders; if the system were non-composite the constraint equation method would be required.

![Figure 24 – Shell and Beam Element Deformation with Rigid Link](image)

**Figure 24 – Shell and Beam Element Deformation with Rigid Link**

### 3.2.2 – Details of Simplified FE Models

The initial model considered for the analysis of SPS included only the deck and the girders connected with rigid links along the span [Figure 25] and is referred to as the “base” model. A second model added the end diaphragms and bracing to the model [Figure 26] and is referred to as the “expanded” model. While a third model increased the stiffness of the second model by including a vertical plate to partially represent the cold-formed angle between the SPS deck and the girders and is referred to as the “complete” model [Figure 27]. A summary of the differences between the models is presented in Table 11. Each of the iterations resulted in increased complexity, but each was still significantly less complex than the solid models developed by Intelligent Engineering which will be referred to as the “detailed” model. The additional iterations allowed for evaluation of the influence of these secondary members on the global behavior and were intended to help determine if their inclusion was necessary. No
consideration was given to other components of the bridge such as the additional cold-formed angles spanning transversely between the girders at the panel joints, longitudinal plates along the exterior edge of the panels, guardrails, or the wearing surface. These members were assumed to be of minimal influence on the lateral load distribution characteristics of SPS.

Table 11 – Element Summary for Simplified Finite Element Models

<table>
<thead>
<tr>
<th>Model</th>
<th>Longitudinal Girders</th>
<th>Deck</th>
<th>Bracing</th>
<th>Longitudinal Cold-Formed Angle</th>
<th>Transverse Cold-Formed Angles</th>
</tr>
</thead>
<tbody>
<tr>
<td>“Base”</td>
<td>Beam 188</td>
<td>Shell 99</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>“Expanded”</td>
<td>Beam 188</td>
<td>Shell 99</td>
<td>Beam 188</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>“Complete”</td>
<td>Beam 188</td>
<td>Shell 99</td>
<td>Beam 188</td>
<td>Beam 188</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Figure 25 – ANSYS Representation of “Base” Model
Figure 26 - ANSYS Representation of “Expanded” Model

Figure 27 - ANSYS Representation of “Complete” Model
The boundary conditions for each model were assumed to be simple supports located at the bottom of the girders and at the center of the bearings. To model the simple supports, additional dummy nodes were created at the location of the support and connected to the centroid of the beam elements with rigid links. Boundary constraints were then applied directly to the dummy nodes, eliminating the error introduced with restraint applied at the girder nodes.

The loads on the system were applied using the principles of equivalent nodal loading. Equivalent nodal loading is based on the same methods used in the element formulation process and utilizes the element shape function to distribute a load on an element to the nodes. This method also allowed for the loads to be uncoupled from the mesh density and has been proven very effective in other research (Chen 1995; Chung and Sotelino 2006; Conner and Huo 2006; Eom and Nowak 2001). For this research point loads were used to represent the applied loads on the system.

3.2.3 – Comparison of Simplified Models to Measured Results

To validate the models and modeling techniques, comparisons were made to measured results for both the Virginia Tech laboratory specimen and two different field tests of the Shenley Bridge. Martin and Murray (2005) reported results on deflections and strains for four load cases. Testing was performed in a controlled environment where the location of load application, displacement measurement, and strain measurements were accurately known. The first round of field testing on the Shenley Bridge was performed by Intelligent Engineering using a static truck with deflections measured using surveying techniques and strains recorded electronically. The second round of field testing utilized displacement and strain measurements from a truck moving slowly across the bridge at various locations as described in previous sections. Each of the tests has certain inherent flaws, but each provided multiple data points for validation of the simplified FE models. The primary indicators in the model validation were comparisons of deflections and strains.

3.2.3.1 – Virginia Tech Laboratory Specimen

The Virginia Tech laboratory specimen is a half-scale SPS bridge with a simple span of 40 ft and a width of 14.8 ft [Figure 28]. Figure 29 shows a typical cross-section of the bridge which is similar to the layout of the Shenley Bridge with the SPS deck welded to cold-formed
angles which in turn are bolted to the longitudinal girders. The SPS deck is comprised of eight panel sections with plate and core thicknesses of 1/8 in. and 3/4 in., respectively.

Figure 28 – Virginia Tech Half-Scale Laboratory Specimen

The bridge was subjected to four load configurations [Figure 30 - Figure 33] by Martin and Murray (2005) during an experimental investigation. The bridge was tested with the entire span simply supported for Load Cases 1 and 2 whereas the bridge was configured as a cantilever for Load Cases 3 and 4. Each of the tests was performed to evaluate a specific condition, but Load Cases 1 and 2 are more representative of the configuration of a real bridge. For these reasons only Load Cases 1 and 2 will be given further consideration in this research. For both of the load cases considered, deflections and strains were recorded during testing at multiple locations as shown in Figure 34. These measured displacements and strains are the basis of comparison to the simplified finite element models. For the laboratory bridge only the “base” model was considered for comparison because the intent was to prove that a simple model was
more than adequate to model the bridge’s global behavior. Additionally, a finite element model developed by Intelligent Engineering are also be used for comparison to the simplified models because these models have already been validated with measured data. This model utilizes a solid modeling technique and considers all of the members within the structure and is referred to as the “detailed” model. A description of this model is presented in Martin and Murray (2005).

Figure 30 – Loading Configuration for Load Case C1 – Lab Specimen (simple supports)
Figure 31 – Loading Configuration for Load Case C2 – Lab Specimen (simple supports)

<table>
<thead>
<tr>
<th>Girder 1</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
<th>Girder 5</th>
<th>Girder 6</th>
<th>Girder 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint 1</td>
<td>Joint 2</td>
<td>Joint 3</td>
<td>Joint 4</td>
<td>Joint 5</td>
<td>Joint 6</td>
<td>Joint 7</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
</tbody>
</table>

Figure 32 – Loading Configuration for Load Case C3 – Lab Specimen (cantilever)

<table>
<thead>
<tr>
<th>Girder 1</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
<th>Girder 5</th>
<th>Girder 6</th>
<th>Girder 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint 1</td>
<td>Joint 2</td>
<td>Joint 3</td>
<td>Joint 4</td>
<td>Joint 5</td>
<td>Joint 6</td>
<td>Joint 7</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
</tbody>
</table>
Figure 33 – Loading Configuration for Load Case C4 – Lab Specimen (cantilever)

Figure 34 – Displacement and Strain Instrumentation Layout – Lab Specimen
3.2.3.1.1 – Load Case C1 Comparison

Load Case C1 was designed to observe the composite behavior of panel-to-girder connection subject to a scaled CLW truck loading based on the CHBDC (CSA 2000). The loads were positioned symmetrically with respect to the bridge centerline and applied through three separate load frames allowing for different loads at each axle location [Figure 30]. The peak loads applied to load frames 1, 2, and 3 were 14.8 kips, 72.3 kips, and 94.9 kips respectively. Comparisons of the deflections and strains at the maximum load were used to investigate the behavior. A representation of the finite element model for Load Case C1 is illustrated in Figure 35. A summary of the deflections of key locations is shown in Table 12 and graphically in Figure 36, also presented are the predictions from the “detailed” FE models; the percent difference is referenced to the measured results in both cases. The deflected shape is shown in Figure 37.

![Figure 35 – FE Model of Half-Scale Laboratory Specimen (Load Case C1)](image)

### Table 12 – Comparison of Vertical Deflections for Load Case C1 under Maximum Load

<table>
<thead>
<tr>
<th>Displacement Transducer</th>
<th>Measured (in.)</th>
<th>“Base” Model Predictions (in.)</th>
<th>% difference</th>
<th>“Detailed” Model Predictions (in.)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>-0.797</td>
<td>-0.824</td>
<td>3.3%</td>
<td>-0.752</td>
<td>-5.7%</td>
</tr>
<tr>
<td>4</td>
<td>-0.795</td>
<td>-0.824</td>
<td>3.7%</td>
<td>-0.752</td>
<td>-5.4%</td>
</tr>
<tr>
<td>7</td>
<td>-1.07</td>
<td>-1.09</td>
<td>1.9%</td>
<td>-1.04</td>
<td>-2.5%</td>
</tr>
<tr>
<td>8</td>
<td>-1.03</td>
<td>-1.09</td>
<td>6.1%</td>
<td>-1.04</td>
<td>1.5%</td>
</tr>
<tr>
<td>11</td>
<td>-0.765</td>
<td>-0.764</td>
<td>-0.2%</td>
<td>-0.722</td>
<td>-5.6%</td>
</tr>
<tr>
<td>12</td>
<td>-0.715</td>
<td>-0.764</td>
<td>6.9%</td>
<td>-0.722</td>
<td>1.0%</td>
</tr>
</tbody>
</table>
The deflection predictions from both the simplified “base” model and the “detailed” model are equally acceptable. None of the predictions exceed 10% in difference with respect to the measured values as indicated by the error bars in Figure 36. The key difference between the
two models is evident when comparing the computational cost; the “detailed” model has a much higher cost due to the large number of degrees of freedom, but does not appear to yield more accurate results for this load case.

As a result of the numerous simplifications in the “base” model, the strain response was not expected to agree as well as the deflections. The exclusion of the cold-formed angles would be expected to result in a loss of accuracy in the model, especially in the region where the angles were located. Elimination of the cross bracing was not expected to significantly alter the response for this load case because the load was positioned symmetrically, so each girder experienced the same load. A summary of the strain response through the cross-section depth for Girder 1 is presented in Table 13 for measured data, “base” model and “detailed” model.

Table 13 - Comparison of Strains for Load Case C1 under Maximum Loading

<table>
<thead>
<tr>
<th>Depth (in.)</th>
<th>Measured $\varepsilon_x$ ($\mu\varepsilon$)</th>
<th>&quot;Base&quot; Model $\varepsilon_x$ ($\mu\varepsilon$)</th>
<th>&quot;Detailed&quot; Model $\varepsilon_x$ ($\mu\varepsilon$)</th>
<th>Measured $\varepsilon_x$ ($\mu\varepsilon$)</th>
<th>&quot;Base&quot; Model $\varepsilon_x$ ($\mu\varepsilon$)</th>
<th>&quot;Detailed&quot; Model $\varepsilon_x$ ($\mu\varepsilon$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-264</td>
<td>-329</td>
<td>-336</td>
<td>-300</td>
<td>-279</td>
<td>-336</td>
</tr>
<tr>
<td>-1.00</td>
<td>-237</td>
<td>-340</td>
<td>-300</td>
<td>-236</td>
<td>-290</td>
<td>-300</td>
</tr>
<tr>
<td>-7.70</td>
<td>-37</td>
<td>-64</td>
<td>-64</td>
<td>-52</td>
<td>-</td>
<td>-64</td>
</tr>
<tr>
<td>-7.89</td>
<td>-24</td>
<td>-52</td>
<td>-35</td>
<td>-20</td>
<td>-101</td>
<td>-36</td>
</tr>
<tr>
<td>-30.94</td>
<td>793</td>
<td>790</td>
<td>791</td>
<td>808</td>
<td>829</td>
<td>790</td>
</tr>
</tbody>
</table>

The “base” model reasonably predicts the strain response at the bottom of the girder, but begins to diverge at the depth of the cold-formed angle connecting the SPS deck to the girders [Figure 38 and Figure 39]. This trend is in agreement with expectations and would indicate that further refinement may be necessary for an improved prediction of the composite behavior. It should be noted that the predictions from the “detailed” model also diverge in this region suggesting that further refinement may not improve the prediction. Both models yield reasonable predictions of the response through the depth of the girder, but also diverge at the top of the plate.
3.2.3.1.2 – Load Case C2 Comparison

Load Case C2 was designed to observe the torsional characteristics of the bridge. The applied loads were a scaled wheel line of the CLW truck. The loads were positioned directly over Girder 1 as in Load Case C1 and applied through three load frames. The peak loads applied
to load frames 1, 2, and 3 were 8.2 kips, 40.7 kips, and 52.6, kips respectively. A representation of the finite element model for Load Case C2 is illustrated in Figure 40. Comparisons of the deflections and strains at the maximum load were used to investigate the behavior. A summary of the deflections of key locations is shown in Table 14 and graphically in Figure 41. Also presented are the predictions from the “detailed” FE model. The deflected shape is shown in Figure 42.

![Figure 40– FE Model of Half-Scale Laboratory Specimen (Load Case C2)](image)

**Table 14– Comparison of Wire Pot Deflections for Load Case C2 under Maximum Load**

<table>
<thead>
<tr>
<th>Displacement Transducer</th>
<th>Measured (in.)</th>
<th>“Base” Model Predictions (in.)</th>
<th>% difference</th>
<th>“Detailed” Model Predictions (in.)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.469</td>
<td>0.499</td>
<td>6.5%</td>
<td>0.060</td>
<td>-87.1%</td>
</tr>
<tr>
<td>4</td>
<td>-1.25</td>
<td>-1.42</td>
<td>13.4%</td>
<td>-1.20</td>
<td>-4.0%</td>
</tr>
<tr>
<td>7</td>
<td>0.614</td>
<td>0.609</td>
<td>-0.8%</td>
<td>0.137</td>
<td>-77.6%</td>
</tr>
<tr>
<td>8</td>
<td>-1.71</td>
<td>-1.83</td>
<td>6.9%</td>
<td>-1.71</td>
<td>-0.2%</td>
</tr>
<tr>
<td>11</td>
<td>0.449</td>
<td>0.428</td>
<td>-4.7%</td>
<td>0.130</td>
<td>-71.0%</td>
</tr>
<tr>
<td>12</td>
<td>-1.21</td>
<td>-1.28</td>
<td>5.5%</td>
<td>-1.29</td>
<td>6.3%</td>
</tr>
</tbody>
</table>
The “base” model tends to provide good predictions of the response at all locations with no difference greater than \(\sim 13\%\) when compared with the measured results. The “detailed” model predictions are not in agreement for the odd numbered deflectometers, which are all located on the side of the specimen furthest from the applied load [Figure 31]. Martin and
Murray (2005) attributed this phenomenon to an eccentricity that could not be accounted for, but upon further evaluation this is not likely the case because the “base” model was capable of predicting the same trend as the measured results. The source of the error is more likely the result of a modeling error due to the stiff deck elements or incorrect output interpretation.

Similar to Load Case C1, the strain response for the “base” model on Load Case C2 was not expected to agree as well as the deflections due to the numerous simplifications within the model. For this load case, the exclusion of the bracing would be expected to have more influence because the load was applied eccentrically and the bracing would assist in the load transfer between the girders. A summary of the strain response through the cross section depth for both girders is presented in Table 15 for measured data, the “base” model and the “detailed” model. The “base” model results more closely match the experimental results through the depth of the girder Load Case C1, but diverge above the top of the girder [Figure 43]. The “detailed” model predicts the strain response at the bottom of the girder well for Girder 1, but also loses accuracy towards the top of the cross section. Additionally, the “detailed” model proves to be inaccurate for Girder 2, which is located further from the applied load [Figure 44], this trend is similar to that observed in the deflection predictions further suggesting a possible error in the model.

Table 15- Comparison of Strains for Load Case C2 under Maximum Loading

<table>
<thead>
<tr>
<th>Depth (in.)</th>
<th>ε_x (με)</th>
<th>ε_x (με)</th>
<th>ε_x (με)</th>
<th>ε_x (με)</th>
<th>ε_x (με)</th>
<th>ε_x (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-199</td>
<td>-260</td>
<td>-326</td>
<td>-93</td>
<td>-107</td>
<td>-167</td>
</tr>
<tr>
<td>-1.00</td>
<td>-175</td>
<td>-277</td>
<td>-291</td>
<td>-82</td>
<td>-102</td>
<td>-150</td>
</tr>
<tr>
<td>-7.70</td>
<td>22</td>
<td>-48</td>
<td>-49</td>
<td>-61</td>
<td>-75</td>
<td>-48</td>
</tr>
<tr>
<td>-7.89</td>
<td>18</td>
<td>15</td>
<td>-27</td>
<td>-49</td>
<td>-57</td>
<td>-26</td>
</tr>
<tr>
<td>-30.94</td>
<td>792</td>
<td>807</td>
<td>814</td>
<td>65</td>
<td>75</td>
<td>340</td>
</tr>
</tbody>
</table>
The comparisons of the “base” model predictions to experimental results from a series of laboratory tests demonstrated that the model is capable of predicting the response with reasonable accuracy. When compared to the predictions using the “detailed” model, there was no significant gain in accuracy when using the “detailed” model, and in many instances the
“base” model provided better predictions. For these reasons the modeling technique used in the development of the “base” model were deemed acceptable for the analysis of SPS bridges.

3.2.3.2 – Shenley Bridge Test (November 2003)

The simplified models were then extended to the analysis of the Shenley Bridge for comparison to the field test results. The first field test was performed in November 2003 as part of the bridge acceptance process (IE 2004) and included displacement and strain measurements for five load cases using a multi-axle truck with a total weight of 117 kips (519 kN) [Figure 45]. The deflections during the testing were measured optically using two automatic levels positioned at both ends of the bridge and strains were measured with conventional strain gages. For each of the load cases, the deflections were measured for the two outside girders at the location of the SPS panel intersections [Figure 46]. The strain gages were positioned on the top and bottom girder flanges at midspan to measure global effects and at other locations on the SPS panels to measure local effects.

<table>
<thead>
<tr>
<th>Axle Weight</th>
<th>20.5 k</th>
<th>21.1 k</th>
<th>22.1 k</th>
<th>20.8 k</th>
<th>22.0 k</th>
<th>10.1 k</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>13&quot;</td>
<td>11&quot;</td>
<td>7&quot;</td>
<td>13&quot;</td>
<td>58&quot;</td>
<td>84&quot;</td>
</tr>
<tr>
<td></td>
<td>71&quot;</td>
<td>72&quot;</td>
<td>391&quot;</td>
<td>53&quot;</td>
<td>179&quot;</td>
<td></td>
</tr>
</tbody>
</table>

*Figure 45 – Loading Truck Configuration for November 2003 Field Test of Shenley Bridge*
The deflections during the testing were measured optically using two automatic levels positioned at both ends of the bridge and strains were measured with conventional strain gages. Details on the strain measurement methods were not presented, but it is assumed that the data was recorded electronically. Measurements were made for only the two exterior girders.

**3.2.3.2.1 – Shenley Field Test Comparisons (November 2003)**

The location of the nodes for the simplified FE models were not the same as the locations of the measured deflections due to the mesh density; as a result, no direct comparison of the measured and predicted results is presented in this section. Comparisons were made by considering the deflection response along the length of each girder which would be expected to be a smooth curve along the span. This method allowed for comparison of the deflected shape of the measured response, “detailed” model and “base” model. The following sections present the results from three of the five load cases. The two cases not presented yield similar results because the loading configurations overlap the configurations presented. The deflection response for the excluded load cases, Load Cases 4 and 5, are presented in Appendix A without discussion.

**Load Case 1**

For Load Case 1, the truck was centered over the interior girder and positioned longitudinally such that the tandem axle straddled STN 13650 [Figure 47]. The deflection
response comparisons are presented in Figure 48 and Figure 49 for girders A and C, respectively. Due to the symmetry of the loading about the bridge longitudinal centerline the deflections for the two girders should be identical. This is the case for both of the finite element models, but not for the measured results. The difference in the response for the two girders suggests that the truck may not have been exactly centered. Additionally, the lack of a continuously smooth curve in the measured data indicates some inherent error or inaccuracy. This error can likely be attributed to the level of accuracy in measurement technique used to determine the deflection, especially considering that the measurements were taken under true static conditions. When considering the overall behavior it is difficult to assess the performance of the “base” model because of the scatter in the measured data. Overall the “base” model reasonably predicts the deflection trend for both of the girders as a larger number of the predictions are within 10% (error bars) of the measured response. When comparing the prediction response of the “base” and “detailed” models the former appears stiffer for this load case, as demonstrated by the smaller deflections [Figure 48 and Figure 49]. Considering that the “detailed” model utilized stiff solid elements throughout the model, it is expected that the behavior would be less flexible than the “base” model, which made use of thin flexible shell elements.
Load Case 2

For Load Case 2 the truck was centered between the interior girder (girder A) and the exterior girder (girder B) at the same longitudinal position as Load Case 1 [Figure 50]. This configuration was tested to evaluate the torsional response of the bridge. For this configuration,
Girder A was the most heavily loaded as is illustrated by the difference in magnitude of the deflection for the two exterior girders [Figure 51 and Figure 52]. The trend of scatter in the measured data is also present for this load case further highlighting the inaccuracy in measurement. When comparing the response prediction of the “base” model to the “detailed” model, Figure 51 suggests that the “base” model best predicts the response of the most heavily loaded girder, while Figure 52 suggests that the “detailed” model best predicts the response of the least loaded girder. This trend would indicate that for an eccentric loading, the “detailed” model better distributes the load amongst the girders than the “base” model. Similar to Load Case 1, it is difficult to assess the performance of the “base” model due to the scatter in the measured response, but for Girder A the model does a reasonable job of predicting the response for the majority of the data points as demonstrated by the 10% error bars. For Girder C this assessment does not hold partly due to the scatter in the measured data, but also due to the relatively low order of magnitude of the response.

*Figure 50 – Load Case 2 Configuration for Shenley Bridge Field Test – November 2003*
Figure 51 - Comparison of Girder A Deflection for Load Case 2

Figure 52 - Comparison of Girder C Deflection for Load Case 2

Load Case 3

For Load Case 3 the truck was centered laterally on the bridge centerline with the tandem axle centered longitudinally between STN 11250 and STN 13650 [Figure 53]. This load case is similar to Load Case 1, but with a different longitudinal positioning of the truck. For this
configuration the two exterior girders are expected to share equally in the resistance due to symmetry. This expectation is validated by the similar deflection response of the two girders [Figure 54 and Figure 55]. Similar to the previous load cases, the measured response does not provide a smooth continuous curve along the span making it difficult to assess the accuracy of the predictions, but both FE models appear to reasonably predict the deflection response of both girders for Load Case 3. Based on the previously observed trends this was expected because the loading was applied symmetrically to the bridge. The perceived level of accuracy of the “base” model for this load case is in agreement with the accuracy levels for the other load cases suggesting that the model is reasonable for predicting the deflection response.

![Diagram of Load Case 3 Configuration for Shenley Bridge Field Test – November 2003](image)

*Figure 53 – Load Case 3 Configuration for Shenley Bridge Field Test – November 2003*
Based on the comparisons presented in this section, the “base” model is capable of predicting the global behavior of an actual bridge. In the previously considered cases, the limiting factor was the inaccuracy in the measured results and for this reason the “expanded” model and the “complete” model were not included in the comparisons. In general, the “detailed” model was expected to yield lower deflection predictions than the “base” model.
because of the inherently stiff elements used, but this phenomenon appears to have been
eliminated as a result of the high mesh refinement. Validations of the “expanded” and
“complete” models were not presented in this section as the “base” model provided reasonably
good correlation with the experimental results from the Half-Scale specimen and the static test
results of the Shenley Bridge. A validation of these models will be presented in Chapter 5 –
Lateral Load Distribution of SPS Bridges as they are extended to the analysis of lateral load
distribution behavior.

3.3 – Stiffened Plate Model for Bridge Structures

In bridges, the finite element method appears to be the dominant method of analysis
likely due to its ease of use, speed of solution, and graphical representation. Even before the
popularity of the finite element method, other methods such as grillage and finite strip were the
norm for the analysis bridge structures.

The use of plate methods is rarely utilized for the analysis of bridge structures,
specifically slab-girder bridges, as is evident from the limited literature available on the subject.
This trend is likely due to the complicated and cumbersome nature of the analysis procedures.
The solution, if not already established, requires solving the differential equation of a plate and
satisfying the boundary conditions, which can both be quite challenging depending on the
configuration and loading. The difficulty usually results from the selection of a general
deflection shape function that satisfies both the differential equation and the boundary
conditions. Solutions for complex plate problems often require approximate methods of
analysis. In addition, the primary result from a plate analysis is the deflected shape function;
other parameters such as stress, strain, moments, and shears are derived from this function
through kinematic and constitutive relationships. These relationships will not be presented or
derived here, but can be found in most plates and shells textbooks (Timoshenko 1959; Ugural
1999).

Researchers in other fields such as aerospace have studied similar structures that include
a plate supported by elastic beams; this type of structure is called a stiffened plate structure. A
few researchers have developed solutions for bridge structures, but these solutions are often
tailored for a specific case. The following discussion presents a summary of some of the pertinent literature related to the application of plate solutions to bridge structures.

Kukreti and Cheraghi (1993) presented a general solution method using an energy approach. Their solution considered plates supported both longitudinally and transversely by beams, subjected to an arbitrary loading. This was an improvement over previous work (Kukreti and Rajapaksa 1990) which was limited based on whether the loading was symmetric or asymmetric. The results from the improved plate model were in very good agreement with results from a finite element model developed for comparison. It should be noted that both solutions neglected the eccentricity of the beams; the beam and plate nodes were modeled as coincident.

Qiao et al. (2000) presented series solutions for an FRP deck supported by FRP stringers. The solutions were divided into a symmetric and asymmetric loading case and utilized an orthotropic plate model where the stringers were incorporated into the plate. This technique considered the plate and stringers as a single continuum with representative orthogonal mechanical properties. Their model provided reasonable predictions of deflection response when compared to experimental and finite element results. The use of orthotropic plate modeling is not likely applicable to SPS bridges due to the large girders at wide spacing (Szilard 2004), but the principles described by the authors may be considered further.

Other researchers have utilized other approaches for the analysis of stiffened plates such as the analog equation method (AEM) (Sapountzakis and Katsikadelis 2002) and element-free Galerkin method (EFG) (Peng et al. 2005), but these methods are very complex and beyond the interest of this research. The method that will be considered in this research is the Ritz energy approach. The Ritz method (Ugural 1999) will be considered primarily due to its ease of use and flexibility to be tailored to numerous applications. This method will be used to represent slab-girder bridges as plates continuous over elastic support beams that are eccentric from the slab.

3.3.1 – Energy Method for Plates Continuous over Elastic Supports

A direct method for the analysis of stiffened plates is through the use of energy methods of analysis. This approach typically requires that a suitable shape function be selected that satisfies the differential plate equation Eq. 2 and the geometric boundary conditions, but does not require that the force boundary conditions be satisfied exactly. The accuracy and convergence of
the solution is directly related to the shape function selected; poor selection of the shape function results in slow convergence or none at all, while the selection of a shape function that satisfies all boundary conditions will result in rapid convergence.

The Ritz method is based on the theory of minimum potential energy, similar to that of the finite element method. The general process involves selection of a shape function that satisfies the geometric boundary conditions and is capable of approximately satisfying the other boundary conditions; the shape function selected is written in terms of unknown coefficients. The shape function is then used in the equations for the system energy which can include the plate strain energy, beam strain energy, and external work due to applied loads. Once the total energy of the system is developed the principle of minimum potential energy is applied with respect to the unknown coefficients of the shape function, allowing for determination of these unknowns. A more detailed presentation of the Ritz method can be found in most plates and shells textbooks (Timoshenko 1959; Ugural 1999).

3.3.2 – Development of Plate Solutions for SPS Bridge

The use of plate theory in the analysis of SPS bridges requires an extension of classical plate theory principles and numerous assumptions. These assumptions include, but are not limited to representation of the SPS deck with an equivalent homogenous deck, idealization of the girders as equivalent stiffening elements, and idealization of the boundary conditions. In essence, the bridge system is reduced to an equivalent plate with stiffening elements along the span at various transverse locations representative of the longitudinal girders [Figure 56].
3.3.2.1 – Equivalent Plate Properties

Sandwich construction has been used in the aerospace industry for many years and as a result, multiple representations of equivalent properties have been developed. This section presents the equations used for the determination of equivalent flexural rigidities of layer plates. The various predictions were validated by using them in a known solution and comparing to a simple FE model. Table 16 presents a summary of the relevant equations used to determine the equivalent flexural rigidity of a layered plate where the term $t$ represents the thickness and the subscripts $p$ and $c$ denote the plate and polymer core, respectively as shown in Figure 57.

Figure 56 – Representation of Slab Girder Bridge as an Equivalent Stiffened Plate

Figure 57 – Sandwich Plate Geometry
To validate the equivalent properties, the known solution for a square plate subjected to a uniformly distributed load with all edges fully restrained were compared for each model. The plate used in the comparison had a total thickness of 1.88 in. comprised of two ¼ in. steel plates and a 1.38 in. thick polymer core. The plates were 100 square inches and subjected to a uniformly distributed load of 0.01 ksi. A comparison of the central deflection predictions is presented in Table 17. The Shell 99 element was previously determined to be the best element for modeling SPS and is also used here as the measure of performance. The FE models using the Shell 63 and Shell 93 elements were based on an equivalent plate with the equivalent flexural rigidities converted back to an equivalent Young’s Modulus. Each of the predictions performs reasonably well compared to the exact solution; but a comparison between models suggests that the equation proposed by Plantema is most suitable for SPS because the predictions are agreement with all of FE models. The equation proposed by Ugural, Vinson, and Ventsel/Krauthammer also performed well, but is not in full agreement with layered shell element prediction. Intelligent Engineering’s equation overestimates the deflection likely due to the exclusion the Poisson’s ratio in the equivalent flexural rigidity. Based on these comparisons the equivalent flexural rigidity proposed by Plantema will be used for the stiffened plate models in this research.

### Table 16 – Summary of Equivalent Flexural Rigidity Prediction Methods

<table>
<thead>
<tr>
<th>Source</th>
<th>Formula</th>
</tr>
</thead>
</table>
| (Ugural 1999; Ventsel and Krauthammer 2001; Vinson 1999) | \[
D_t = \frac{2}{3} E_p \left[ \frac{\left(\frac{t_c}{2} + t_p\right)^3 - \left(\frac{t_c}{2}\right)^3}{\left(1 - \nu_p^2\right)} \right] + E_c \left(\frac{t_c}{2}\right)^3
\]
| (Plantema 1966) | \[
D_t = \frac{2E_p t_p}{\left(1 - \nu_p^2\right)} \left(\frac{t_c}{2} + \frac{t_p}{2}\right)^2
\]
| (IE 2006) | \[
D_t = \frac{E_p t_p^2}{2t_p} \left(\frac{t_c}{2} + \frac{t_p}{2}\right)^2
\]
Table 17 – Comparison of Equivalent Flexural Rigidities

<table>
<thead>
<tr>
<th>Model</th>
<th>$D_t$ (kip-in)</th>
<th>Exact (in.)</th>
<th>ANSYS Finite Element Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>SHELL 63 (in.)</td>
</tr>
<tr>
<td>Ugural; Vinson; Ventsel/ Krauthammer</td>
<td>11012</td>
<td>-0.1162</td>
<td>-0.1159</td>
</tr>
<tr>
<td>Plantema</td>
<td>10898</td>
<td>-0.1175</td>
<td>-0.1171</td>
</tr>
<tr>
<td>Intelligent Engineering</td>
<td>10000</td>
<td>-0.1280</td>
<td>-0.1276</td>
</tr>
</tbody>
</table>

3.3.2.2 – Representation of Equivalent Stiffening Elements

When considering the contribution of the stiffening elements to the global behavior of a plate the effects of flexural, shear, torsional, and axial resistance must be included. The energy method presented by Kukreti and Cheraghi included strain energy from both the plate and girders with the girder contribution comprised of the flexural, axial, torsional, and shear strain energies [Eq. 22].

$$U_T = \frac{TS}{Plate\ Strain\ Energy} + \frac{TBX}{Long.\ Beam\ Strain\ Energy} + \frac{TBY}{Trans.\ Beam\ Strain\ Energy} + \frac{TP}{Work\ done\ by\ Load} \quad Eq.\ 22$$

where

$$TS = \int_0^a \int_0^b \frac{D}{2} \left( \frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right)^2 - 2(1-\nu_s) \left( \frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} - \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 \right) dxdy$$

$$TBX = \int_0^a \int_0^b \frac{E_y I_y}{2} \left( \frac{\partial^2 w}{\partial x^2} \right)^2 + \int_0^a \int_0^b \frac{GJ_y}{2} \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 + \int_0^a \frac{E_y I_y}{(2GA_x)} \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 dx$$

$$TBY = \int_0^b \int_0^a \frac{E_x I_y}{2} \left( \frac{\partial^2 w}{\partial y^2} \right)^2 + \int_0^b \int_0^a \frac{GJ_y}{2} \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 + \int_0^b \frac{E_y I_y}{(2GA_y)} \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 dy$$

$$TP = \int_0^a \int_0^b q(x,y)w(x,y) dxdy$$

Kukreti and Cheraghi’s representation considered the energies of the plate, longitudinal beams, and transverse beams, but was validated with a finite element model and no experimental tests. The finite element model considered the beam to be concentric with the deck. This representation is similar to those proposed in early finite element models of slab-girder bridges.
prior to the development of the eccentric beam model. A problem with this representation is that it does not include the additional stiffness provided by the eccentric beam acting compositely with the deck. Their representation would be more similar to a non-composite system where the beam and deck act independently and the added stiffness results primarily from the moment of inertia of the beam. Based on this observation it is proposed for this research that an equivalent moment of inertia be used to account for the additional stiffness provided by an offset composite beam.

The development of an equivalent moment of inertia requires a series of assumptions about the behavior of the composite system. The first assumption is that the section behaves as if fully composite with no slip between the girders and the plate. Secondly, the effective width used is one half of the girder spacing on either side of the center of the girder flange. Based on the research of Martin and Murray (2005) it was suggested that the effective width of the laboratory specimens extended to at least half of the girder spacing on either side of the girder web, which was greater than the overhang for the exterior edge. The final assumption is that the polymer core does not contribute to the section properties due to the low modulus, but the space occupied by the core is included.

The derivation of this equivalent stiffness is based on the principles of the plastic neutral axis and effective width. In a typical plate analysis the stiffness of the plate alone is accounted for in the flexural rigidity, but for a composite section analysis all of the stiffness terms are with respect to the plastic neutral axis of the section [Figure 58]. These differences make it necessary to consider the stiffness provided by the composite section without the contribution of the deck. Based on elementary mechanics of materials principles, the moment of inertia of a composite member is comprised of the summation of the moments of inertia of sections about their own centroid ($I$) plus their area times the square of the distance between their centroid and the plastic neutral axis ($Ad^2$) [Eq. 23]. This analysis would provide the stiffness of the composite system assuming it to behave as one section with respect to the plastic neutral axis, but for this analysis the location of interest is with respect to the centroid of the plate. Since the stiffness of the plate term is already included the only portion of interest would be the contribution of the longitudinal beams. Referencing the moment of inertia of the beams with respect to the plate centroid would result in an overly stiff section and would not be realistic, while referencing with respect to the PNA would require backing out the plate stiffness from the analysis. A more
realistic determination would reference the stiffness provided by the beam and plate to the plastic neutral axis; this would include both $I$ and $Ad^2$ terms for the beam and also the $Ad^2$ term for the plate which is not considered in a plate analysis.

$$I_{\text{composite}} = \sum_{\text{elements}} \left( \overline{I} + Ad^2 \right)$$  \hspace{1cm} \text{Eq. 23}

3.3.2.3 – Boundary Conditions

For simple span bridges the boundary conditions typically consist of pin and roller restraints that are coincident with the bottom of the supporting girders at the location of the bearings. This configuration is significantly different from the idealization of the boundary restraints of a stiffened plate structure, where the restraints are assumed to be along the plate edges at the centroid of the plate. The difference in idealization of support conditions has been shown to induce significant error in the finite element and similarly would also be expected to induce some error in the proposed equivalent plate method. The difference between the methods is that the error within the finite element method can be rectified by offsetting the support nodes to the bottom of the members, whereas no such simplification exists for plates. For this analysis it is assumed that because the plates are supported by the girders, the boundary conditions of the girder are transferred directly to the plate at the location of application. While this assumption is not entirely true and may induce some error in the analysis, it should hold reasonably true for a composite bridge system because displacement response of the girders and deck are coupled.
The error produced as a result of the boundary conditions will not be investigated in this research, but will be considered in the interpretation of the results.

3.3.2.4 – Comparison of Stiffened Plate Results to Experimental Results

The greatest challenge in the analysis procedure was in the selection of a suitable shape function to represent the behavior and boundary conditions of the stiffened plate or bridge structure. Results from the modeling performed by Kukreti and Cheraghi (1993) suggested that the shape function [\( Eq. \ 24 \)] used in their research was acceptable for a simply supported bridge with two edges free, but required multiple terms to converge to the exact solution. In their shape function the \( A_{mn} \) terms represent unknown coefficients that are solved for, \( y \) represents the direction between the free edges, \( x \) represents the direction between the simple supports, \( b \) represents the transverse width, and \( a \) represents the span length [Figure 59]. Kukreti and Cheraghi’s suggested shape function is applicable to all loading configurations and not limited to either symmetric or anti-symmetric configurations.

\[
\begin{align*}
    w(x, y) &= \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} A_{mn} \frac{y^{n-1}}{b^{n-1}} \sin \left( \frac{m \pi}{2b} x \right) \\
    &\quad \text{Eq. 24}
\end{align*}
\]

![Figure 59 - Stiffened Plate Configuration](image-url)
Based on the numerous approximations used within the analysis procedure, it is expected that the predicted response will not exactly match test results, but should provide a reasonable approximation of the behavior. When comparing deflections, the predicted response is expected to yield results in best agreement with the measured response because the energy is based on derivatives of the shape function. A decrease in accuracy is expected with a comparison of moments and shears as a result of the successive derivatives of an already approximate solution. For these reasons only a comparison of the deflections between the plate models and experimental results will be considered in these initial stages of this research. A validation of the stiffened plate approach will be presented in Chapter 5 – Lateral Load Distribution of SPS Bridges

3.3.3 – Summary of Stiffened Plate Modeling

While classical plate methods have not been frequently utilized for the analysis of bridge structures, their use is expected to provide additional insight into the behavior of SPS in bridges. The simplicity of the Ritz method should allow for the solution of a stiffened plate subjected to an arbitrary loading such as a truck. The expectation is that this method will allow for consideration of the global behavior of a SPS bridge and provide another means of analyzing lateral load distribution behavior. A further extension of the analysis may include a parametric study of the variables influencing the design of a SPS bridge.

3.4 – Summary of Analytical Modeling

Of primary interest in this research is the development of simple models capable of predicting global behavior of SPS bridges. Chapter 3 focuses on presenting the analytical methods used to investigate the key design parameters of SPS highlighted in Chapter 2. While the stiffened plate model appears to represent a viable option for analyzing behavior, the majority of the focus is on the finite element method as a means of performing these analyses. The stiffened plate model is investigated, but is considered only as a secondary option.
Chapter 4 – SPS Deck Design

The design of a SPS bridge differs from that of a conventional system in only a few aspects, a critical one being the design of the deck panel. While many of the connection design details of SPS construction are still under investigation and subject to modification, the structural configuration of the panel cross section is not, but a method for designing it is. The current inventory of SPS bridges have relied heavily on finite element models and “in-house” experience for the deck design, but the development of a practical design methodology is essential for more widespread acceptance of the technology. Bridge engineers are not likely to perform a rigorous finite element analysis when considering their various design options. This section aims to provide a general design methodology for the panel sections that can be utilized for typical bridge applications.

4.1 – Deck Analysis Model

When considering the available methods of analysis for bridge deck systems, two readily available methods include treatment of the deck as a grid or as a plate. The former divides the deck into a series of longitudinal and transverse grid (or beam) members as in the grillage analysis method; whereas, the latter accounts for continuity of the deck and also considers the coupled response from the orthogonal directions. The isotropic mechanical properties of SPS make the plate analysis method a more logical choice for the analysis SPS bridge decks.

This representation rapidly increases in complexity as boundary conditions and applied loads deviate from the limited known solutions. When considering the interaction of the deck with the other structural elements such as stringers, the complexity is even further increased. This interaction requires the uni-directional response of the stringers [Eq. 1 - pg. 15] to be coupled with the multi-directional response of the deck or plate [Eq. 2 - pg. 15]. As a result of these complexities, this method of analysis is not frequently used in design.

While the use of classical plate theory is not typical in design, it has been used in the development of some of the AASHTO design equations (Higgins 2003) and is also recommended as a suitable method for the analysis of bridge decks within the AASHTO specifications (AASHTO 2002; AASHTO 2004). This method will be extended to SPS bridge decks through the finite element method by considering the deck region between girders as a
plate with variable boundary conditions subjected to bridge dead and live design loads [Figure 60]. Closed-form plate solutions are available for certain boundary and loading conditions, but these solutions are not easily adaptable to variations from these conditions. In addition, these series solutions often require the summation over numerous terms before convergence is achieved (Timoshenko 1959). While these classical plate solutions have some limitations, they will be included herein when deemed applicable such as for cases with known solutions. The intent of this proposed approach is to develop practical design recommendations that allow for sizing of the panel cross-sections adequate for the strength, serviceability, and fatigue limit states as defined in the AASHTO LRFD design specification.

4.1.1 – Validation of Deck Plate Behavior Approximation

Prior to accepting this analysis methodology, comparisons were made to results from two static tests of SPS bridge deck panels. The first comparison was made to a stand alone SPS panel section while the second comparison was made to a panel within a half-scale bridge. The
following sections provide comparisons of the measured response to that of the proposed model and also provide a basis for treatment of the boundary conditions for future analysis.

4.1.1.1 – SPS Static Panel Comparison

To validate the proposed design approach, a comparison was made to the results from the full scale static panel section tested by Martin and Murray (2005). The section consisted of a SPS deck panel core and plate thicknesses of 1.5 in. and 0.25 in., respectively. The panel was supported on W27x84 stringers, which were in turn supported on plates clamped to the floor. Load Case E1-B was considered for comparison since the load was applied between the supports [Figure 61] and is representative of the type of loading scenario under investigation. In summary, the panel section was incrementally loaded in 5 kip increments to the factored load of a CLW design truck axle (77.78 kips) with key deflections and strains recorded at each increment. A more detailed description of the testing procedure is presented in the work by Martin and Murray (2005).

To determine if the proposed model is valid for analyzing a SPS deck, a series of plate models were evaluated under various boundary conditions [Table 18]. Of particular interest was the response as a function of the variation in boundary conditions at the location of the stringers. The selected boundary conditions would be expected to yield an upper bound on the maximum midspan deflection for Case I and a lower bound on the deflection for Case III with the actual deflection somewhere in between. These two cases equate to plates with all edges simply supported and all edges fully restrained, respectively. Case II was selected because it represents a reasonable representation of the likely boundary conditions as a result of the continuity over the support. Similarly, Case IV serves as a more accurate representation of the actual geometry, but is presented only for comparative purposes.

A finite element model was created in ANSYS with Shell 91 elements considering only the deck section between the stringer supports while excluding the cold-formed angles and stringers [Figure 60]. The Shell 91 element is a layered element that allows for representation of each of the steel and polyurethane layers, this element type was previously demonstrated to yield results consistent with classical plate methods when properly refined (3.2 – Development of Finite Element Model for SPS). The loads were applied to the plate in 5 kip increments as two uniform pressures distributed over a rectangular area of 200 in² (10 x 20 in.) to approximate the
actual pressure, which was applied using a concrete filled tire section. These pressures were assumed to act directly on the plate as the concrete-filled tire was not included in the model.

![Elevation View](image)

**Figure 61 – Full Scale Static SPS Test Layout (Load Case E1-B)**

**b) Plan View Schematic**

*Table 18 – Summary of Boundary Conditions for Full Scale Panel Comparison*

<table>
<thead>
<tr>
<th>Boundary Condition Case</th>
<th>Boundary Condition at Stringer Location</th>
<th>Boundary Condition at Other Edges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case I</td>
<td>Simple</td>
<td>Simple</td>
</tr>
<tr>
<td>Case II</td>
<td>Fixed</td>
<td>Simple</td>
</tr>
<tr>
<td>Case III</td>
<td>Fixed</td>
<td>Fixed</td>
</tr>
<tr>
<td>Case IV</td>
<td>Simple (includes overhang)</td>
<td>Simple</td>
</tr>
</tbody>
</table>
Figure 62 illustrates the geometric configuration and the deflection contours for the simply supported plate configuration. Of particular interest in the investigation was the peak deflection that resulted from the symmetric loading condition of Load Case E1-B. Figure 63 presents the load displacement relationships at the plate center for each of the boundary condition cases and the measured response and predicted response from Intelligent Engineering (Martin and Murray 2005). A comparison of the measured response for the static panel to the predicted response of the plate model suggests that the SPS deck is best represented by simple support conditions (Case I). While the idealized boundary conditions are a simplification of the actual boundary conditions, it is to be expected that the cold-form angles would only provide a limited degree of rotational restraint as a result of their thin cross-section.

Figure 62 – Sample ANSYS Geometry/Loading Configuration and Deflection Response
4.1.1.2 – SPS Half Scale Bridge Comparison

In an effort to assess the shear resistance provided by SPS decks, an additional static test of the half-scale bridge was performed [Figure 64 and Figure 65]. While the intent of this test was to provide a validation of the shear capacity, it also provided an additional comparison for the proposed model. A brief description of the bridge is presented in 3.2.3.1 – Virginia Tech Laboratory Specimen with a more detailed description presented by Martin and Murray (2005). It should be noted that all previous tests on this bridge consisted of loading along the girders with no loads applied directly on the unsupported deck panel locations (Martin and Murray 2005).

The test consisted of loading the bridge section at the center of Panel 3 with a rectangular loading patch (9 in. x 14 in.). Panel 3 was selected as the testing location to allow for some distance between the panel-to-girder connection fatigue rupture that occurred in a separate test. The panel section was instrumented with strain gauges and displacement transducers at the center of the panel, panel support locations, and end support locations to ensure that the panel was not yielded during the testing and also to provide additional data for validation of the analysis method [Figure 65]. The instrumentation locations were selected based on the
assumption that the panel section would behave like a plate, but considered both simply supported and fully restrained boundary conditions with gauges placed at locations expected to experience high strains.

Three tests were performed and each consisted of slowly loading the panel section until the measured strain approached the yield strain, assumed to be 1,724 με based of Grade 50 steel. Each test was terminated when strains approached the yield strain. The limiting strain occurred at mid-panel in the longitudinal direction (SG 1). The load deflection results for the three tests are presented in Figure 66 along with the FEA plate predictions for the simply supported (Case I) and fully restrained longitudinal edge boundary (Case II) conditions. The FE modeling approach was identical to that used in the panel section validation, but utilized a single loading patch located at the middle of the panel section without the bearing pad. The panel section considered in the model spanned 5.09 ft in the transverse direction and 5 ft in the longitudinal direction, representing the region of deck spanning between the longitudinal girders and the transverse cold-formed angles. These load deflection comparisons would suggest that the panel sections tend to behave more like simply supported plates than those with full fixity at the girder supports, similar to the observed behavior of the SPS panel section.
Figure 64 – Half-Scale Static Panel Test Set-up

Figure 65 – Plan and Elevation of Virginia Tech Half-Scale SPS Bridge (Static Panel Test)
A comparison of the load-strain response at the mid-panel would suggest that some degree of fixity is likely present at the supports. The longitudinal strain gauge (SG 1) exhibits a linear elastic response in agreement with the simply supported case [Figure 67]. Similarly, the transverse strain gauge (SG 2) exhibits a nearly linear response, but appears to be in better agreement with the predictions for the case with the longitudinal edges fully restrained [Figure 68]. This trend would suggest that the supports parallel to the span direction provide some degree of fixity while the supports parallel to the transverse span provide minimal rotational restraint. This difference is likely the result of the additional rotational stiffness provided by the longitudinal stringers.

The load-strain response for all the gauges at the panel support locations (SG 6 - SG 9) of Test 3 is presented in Figure 69. Similar behavior was observed for Tests 1 and 2 and these comparisons are presented in Appendix B.1 – Additional Half-Scale Test Results [and Figure 169]. The measured strain values greater than zero indicate that there is some rotational restraint provided by all of the supports; a simply supported boundary would yield no strain. The magnitude of the strain at these locations suggests that restraint provided by these supports is less than full restraint because they are well below the limiting yield strain (1,724 με), which would not be the case for fully restrained plate. The discrepancy for the transverse edge support can
likely be attributed to the gauge location falling within the longitudinal area of influence of the applied load and the continuity restraint provided by the rest of the deck in the longitudinal direction, but could also be a result of the cold-formed angle not actually behaving as a support as a result of its relatively low flexural stiffness. While these issues surrounding the boundary conditions are relevant to the behavior of SPS decks their influence will be accounted for by utilizing a bounded solution approach.

Figure 67 – Load vs. Mid-panel Longitudinal Strain (SG 1) – Half-Scale Bridge

Test 1, Test 2, Test 3, SS Plate (Case I), Fixed at Beams (Case II)
Figure 68 – Load vs. Mid-panel Transverse Strain (SG 2) – Half-Scale Bridge

Figure 69 – Load vs. Panel Edge Strain (SG 6-9) Locations – Half-Scale Bridge (Test 3)
4.1.1.3 – Summary of the Design Model Validation

The two panel sections used in the validation analysis were considered to be realistic representations of the “as-built” conditions of a typical SPS bridge deck panel and as a result were suitable for validation of the proposed plate model. The results from both of the comparisons suggest that the behavior of a SPS bridge deck panel is similar to that of a simply supported plate, but exhibit a small degree of fixity at the support locations. This behavior is expected because the cold-formed angle supporting the deck panels are relatively thin and would not be expected to provide a high degree of rotational restraint. Based on these comparisons, the behavior will be assumed to be similar to that of a plate with all edges simply supported, but consideration will also be given to the fixed edge boundary conditions of Case II as this is the likely lower bound on the response.

4.1.2 – SPS Deck Design Approach

In the development of the design procedure for SPS, the controlling design limit state was initially unknown. This uncertainty caused some difficulty in that it was impossible to size the SPS components without knowing whether the design is controlled by stiffness limits or stress limits. The approach employed in the development of the design procedure assumed that stiffness was likely the controlling limit state as a result of the thinness of a typical SPS deck.

The design approach adopted herein consists of developing a stiffness-controlled limit state based on the optional live load deflection limits within the AASHTO LRFD (2004) specification and stress checks for the Strength I, Service II, and Fatigue limit states. This approach was not fixed, but would be deemed acceptable so long as the stiffness based design did not result in an exceedance of the remaining limit states. To determine the controlling limit states and upper bounds, a series of plates with varying properties, boundary conditions, and subjected to multiple loading scenarios were analyzed using the finite element method. A description of the variations in properties, boundary conditions, and loading scenarios is presented in the following sections.

4.1.2.1 – Equivalent Plate Properties

When considering the behavior of a plate, the deflection response is inversely proportional to the flexural rigidity or equivalent plate stiffness [Eq. 25]. For homogenous
plates, a given flexural rigidity can be achieved through combinations of total plate thickness and material properties, $E$ and $\nu$. When considering the equivalent flexural rigidity of a sandwich plate, infinite combinations of face plate and core thickness are available to yield the corresponding flexural rigidity [Figure 70]. This flexibility in component selection allows for thickness to be selected to meet stiffness and strength requirements.

Of primary importance to the design of a SPS deck is a proper estimation of equivalent properties. These equivalent properties are essential for proper selection of the steel and polymer thicknesses. A comparison of a number of methods for calculating flexural rigidity of a plated were presented in 3.3.2.1 – Equivalent Plate Properties and it was determined that models proposed by Plantema (1966), Ventsel and Krauthammer (2001), Ugural (1999), Vinson (1999), and McFarland (1972) yielded similar results when compared to the finite element predictions. All of the models considered allow for the calculation of an equivalent flexural rigidity for a layered plate [Eq. 26], but the model proposed by Ventsel and Krauthammer also provides a method for calculating the equivalent Poisson’s ratio [Eq. 27]. The equivalent Poisson’s ratio can then be used in conjunction with the equivalent flexural rigidity to determine the equivalent modulus of elasticity ($E_{\text{equiv}}$) for a given thickness plate based on the relationship in Eq. 28. The calculated equivalent modulus of elasticity can in turn be used to select an equivalently stiff plate of the same thickness. For these reasons, the method proposed by Ventsel and Krauthammer are used to calculate the equivalent properties of SPS plates.

For the analysis of SPS decks these equivalent properties allowed for treatment of the section as a homogenous section for stiffness based response such as deflection, rotation, and strain, but cannot be extended to the determination of internal stresses as a result of the difference in material stiffness between the layers.
Figure 70 - Variation in Flexural Rigidity with core and plate thicknesses

Equation 25

$$\nabla^4 w(x, y) = \frac{p(x, y)}{D}
$$

Equation 26

$$D_t = \frac{2}{3} \left[ \frac{E_p}{\left(1-v_p^2\right)} \left(\frac{t_p}{2}\right)^3 - \left(\frac{t_c}{2}\right)^3 \right] + \frac{E_c}{\left(1-v_c^2\right)} \left(\frac{t_c}{2}\right)^3$$

Equation 27

$$\nu_{eq} = \frac{2}{3D_t} \left[ \frac{E_p\nu_p}{\left(1-v_p^2\right)} \left(\frac{t_p}{2}\right)^3 - \left(\frac{t_c}{2}\right)^3 \right] + \frac{E_c\nu_c}{\left(1-v_c^2\right)} \left(\frac{t_c}{2}\right)^3$$

Equation 28

$$D_t = \frac{E_{equiv\, total}^3}{12\left(1 - \nu_{eq}^2\right)} \Rightarrow E_{equiv} = \frac{12D_t\left(1 - \nu_{eq}^2\right)}{t_{total}^3}$$
4.1.2.2 – Boundary Conditions

One of the most difficult tasks in the development of a representative deck model is in the selection of boundary conditions. Within an actual slab-girder bridge, the deck is typically continuous over the bridge span and over the supporting girders. The continuity in the longitudinal span does not pose any significant difficulty because it is reasonable to assume that the deck maintains the same support conditions as the supporting members, but the restraint provided by the deck being continuous over elastic supports presents a challenge. The restraint provided can be considered a combination of in-plane restraint provided by the deck continuity, rotational restraint from the girders and deck continuity, and out-of-plane restraint from the elastic supports.

While it is possible to quantify the restraint provided by the deck continuity and elastic supports, the results cannot be assumed to be applicable to all cases. Fortunately, these restraints can be bounded by considering two support scenarios. Both scenarios neglect the out of plane deformations of the elastic supports, but the first scenario also neglects the contributions of the elastic girders and plate continuity and considers the deck to be a plate simply supported on all edges. This scenario serves as an upper bound on the deflection response in that this configuration will be the most flexible. The second scenario assumes in-plane and rotational restraint provided by the deck continuity and elastic supports respectively, to be infinite (i.e. fixed support). In reality the true degree of restraint is somewhere between these two scenarios because both are somewhat difficult to achieve. The boundary condition scenarios used in the development of the SPS deck design will be those highlighted in the comparison to the full-scale static panel and the half-scale SPS bridges. The first boundary condition scenario will consist of simple supports along all four edges whereas the second scenario will utilize fixed supports at the location of the supporting stringers and simple supports on the other two edges [Figure 71].

As a result of the combination of the deck being continuous over the supports and the apparent torsional flexibility of the supporting cold-formed angle, the behavior of the plate is expected to lie within these bounds. All of the FE plate models maintained an aspect ratio of 1:3 (transverse span to longitudinal span) to prevent the longitudinal boundary conditions from influencing the transverse behavior (Timoshenko 1959).
4.1.2.3 – Loading Scenarios

The load scenarios for the development of SPS deck design recommendations were selected to mimic possible loading scenarios that would induce worst case loading effects while considering the design loads specified within AASHTO LRFD (2004). For all of the scenarios, dead load was not considered with the exception of self-weight, where applicable. The dead loads were excluded primarily because they are expected to be small compared to magnitude of live load. Table 19 summarizes the force effect combinations, load factors, and dynamic load allowance values considered for each of the limit states considered. For the cases that include self-weight, the load results from gravity acting on the deck, 384 in/s², assuming consistent unit mass densities of $740 \times 10^9$ kip·s/in⁴ and $144 \times 10^9$ kip·s/in⁴ for the steel and polymer core, respectively.
### Table 19 – Summary of Loading Conditions for Key Limit States

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Load Factor Dead/Live</th>
<th>DLA</th>
<th>Load Description*</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability (Deflection)</td>
<td>NA/1.0</td>
<td>33%</td>
<td>Design Truck</td>
<td>Figure 73 or Figure 74</td>
</tr>
<tr>
<td>Serviceability (Service II)</td>
<td>1.0/1.3</td>
<td>33%</td>
<td>Design Lane +25% of Design Truck</td>
<td>Figure 72 + (Figure 73 or Figure 74)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Self-weight + Design Lane + Design Truck</td>
<td>Figure 72 + (Figure 73 or Figure 74)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Self-weight + Design Lane + Design Tandem</td>
<td>Figure 72 + (Figure 76 or Figure 77)</td>
</tr>
<tr>
<td>Strength (Strength I)</td>
<td>1.25/1.75</td>
<td>33%</td>
<td>Design Lane + Design Truck</td>
<td>Figure 72 + (Figure 73 or Figure 74)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Self-weight + Design Lane + Design Truck</td>
<td>Figure 72 + (Figure 73 or Figure 74)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Self-weight + Design Lane + Design Tandem</td>
<td>Figure 72 + (Figure 76 or Figure 77)</td>
</tr>
<tr>
<td>Fatigue</td>
<td>NA/0.75</td>
<td>15%</td>
<td>Design Truck</td>
<td>Figure 73 or Figure 74</td>
</tr>
</tbody>
</table>

*Design Lane = 0.64 klf (over a transverse width of 10')
*Design Truck = 16 kips (over 10” x 20” tire patch)
*Design Tandem = 12.5 kips (over 20” x 10” tire)

---

*Figure 72 - Design Lane Load*
Figure 73 - Single Tire of HL-93 Design Truck

Figure 74 – Rear Axle of HL-93 Design Truck
Figure 75 – Side-by-Side Rear Tires of HL-93 Design Trucks

Figure 76 - Design Tandem
4.1.2.3.1 – Serviceability Limit State (Deflection Control)

As previously highlighted [2.5.2.3.2.1 – Deflection Limits], the deflection control limit, L/800, specified within the AASHTO LRFD specification (AASHTO 2004) was used to establish a limiting stiffness to minimize relative deflections between girders resulting from tire loadings. This criterion is considered optional for most deck systems, but the specification stipulates that the criterion is required for lightweight metal decks [AASHTO LRFD Section 2.5.2.6.2], of which SPS can be loosely classified. While this criterion is primarily based on limited research on human tolerance to vibrations (Wright and Walker 1972), it has yielded historically acceptable designs. A further investigation of the deflection control limit is definitely warranted, but is beyond the scope of this research.

The serviceability limit state for deflection was evaluated by considering the stiffness or flexural rigidity ($D_t$) required to satisfy the limiting deflection criterion. The investigation considered a variety of plates with flexural rigidities ranging from 1,000-38,000 kip-in., subjected to the loading combinations specified under the Serviceability (Deflection) limit state in Table 19. Simply supported (Case I) and fixed (Case II) boundary conditions were considered for all of these plates. As highlighted, numerous combinations of plate and core thicknesses can be utilized to achieve the flexural rigidity desire and for simplicity a face plate size of 5/16 in.
was selected. The total thickness, equivalent Poisson’s ratio, and equivalent modulus were determined by solving Eq. 26-Eq. 28 after assuming a modulus of elasticity values of 29,878 ksi and 109 ksi for the plates and core, respectively. A summary of the plate sizes and equivalent properties is shown in Table 20.

Shell 93 elements with equivalent plate properties were used for the analysis; these elements were previously shown to yield accurate results with proper refinement [3.1.5 – Element Selection]. Predicted deflections at midspan versus girder spacing for the simply supported and fixed edge boundary conditions are shown in Figure 78 and Figure 79 for a range of flexural rigidities. These figures allow for the determination of a plate stiffness required to satisfy the limiting deflection, L/800. Also shown in these figures is the limiting deflection, L/800, to illustrate the plate stiffness that would be required to satisfy this limit.
Table 20 – Plate Properties for Serviceability (Deflection) Limit State

<table>
<thead>
<tr>
<th>Equivalent Modulus of Elasticity (ksi)</th>
<th>Total Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29814</td>
<td>0.72</td>
</tr>
<tr>
<td>28944</td>
<td>0.91</td>
</tr>
<tr>
<td>27831</td>
<td>1.06</td>
</tr>
<tr>
<td>26776</td>
<td>1.18</td>
</tr>
<tr>
<td>25826</td>
<td>1.29</td>
</tr>
<tr>
<td>24977</td>
<td>1.38</td>
</tr>
<tr>
<td>24215</td>
<td>1.47</td>
</tr>
<tr>
<td>23528</td>
<td>1.55</td>
</tr>
<tr>
<td>22905</td>
<td>1.63</td>
</tr>
<tr>
<td>22336</td>
<td>1.70</td>
</tr>
<tr>
<td>21814</td>
<td>1.77</td>
</tr>
<tr>
<td>21332</td>
<td>1.84</td>
</tr>
<tr>
<td>20886</td>
<td>1.90</td>
</tr>
<tr>
<td>20471</td>
<td>1.96</td>
</tr>
<tr>
<td>20083</td>
<td>2.02</td>
</tr>
<tr>
<td>19720</td>
<td>2.08</td>
</tr>
<tr>
<td>19379</td>
<td>2.13</td>
</tr>
<tr>
<td>19058</td>
<td>2.18</td>
</tr>
<tr>
<td>18754</td>
<td>2.23</td>
</tr>
<tr>
<td>18466</td>
<td>2.29</td>
</tr>
<tr>
<td>18193</td>
<td>2.33</td>
</tr>
<tr>
<td>17934</td>
<td>2.38</td>
</tr>
<tr>
<td>17687</td>
<td>2.43</td>
</tr>
<tr>
<td>17451</td>
<td>2.47</td>
</tr>
<tr>
<td>17226</td>
<td>2.52</td>
</tr>
<tr>
<td>17010</td>
<td>2.56</td>
</tr>
<tr>
<td>16803</td>
<td>2.61</td>
</tr>
<tr>
<td>16605</td>
<td>2.65</td>
</tr>
<tr>
<td>16414</td>
<td>2.69</td>
</tr>
<tr>
<td>16230</td>
<td>2.73</td>
</tr>
<tr>
<td>16054</td>
<td>2.77</td>
</tr>
<tr>
<td>15883</td>
<td>2.81</td>
</tr>
<tr>
<td>15719</td>
<td>2.85</td>
</tr>
<tr>
<td>15560</td>
<td>2.89</td>
</tr>
<tr>
<td>15407</td>
<td>2.92</td>
</tr>
<tr>
<td>15259</td>
<td>2.96</td>
</tr>
<tr>
<td>15115</td>
<td>3.00</td>
</tr>
<tr>
<td>14976</td>
<td>3.03</td>
</tr>
</tbody>
</table>

Equivalent Poisson’s Ratio = 0.287
Figure 78 – Midspan Deflection vs. Girder Spacing – Serviceability (Deflection) – SS

Figure 79 - Midspan Deflection vs. Girder Spacing – Serviceability (Deflection) – Fixed
For the two load cases considered [Table 20], the controlling case resulted from the design truck alone. In addition, none of the plates approached yielding, verifying that the elastic solution was valid for the applied loads. This allowed for the determination of a relationship between the stiffness required to satisfy the AASHTO serviceability criterion for deflection and the plate span, or girder spacing. The resulting relationships [Figure 80] are linear correlations for both the simple (Case I) [Eq. 29] and fixed (Case II) [Eq. 30] boundary conditions.

These relationships allow for a direct determination of the flexural rigidity required for a given girder spacing. The resulting design satisfies the AASHTO LRFD deflection criterion, assuming simple and fixed restraint provided by the supporting girders. It would be expected that the simple support would always yield the largest deflections and should be used for a conservative design, but depending on the rotational resistance of the supporting members the fixed edge supports may better represent the behavior.

![Figure 80](image_url)

**Figure 80 – Required Flexural Rigidity vs. Girder Spacing**
\[ D_t = 3738 \cdot S - 2551 \quad (S \text{ in } ft) \quad \text{Eq. 29} \]
\[ D_t = 1598 \cdot S - 1569 \quad (S \text{ in } ft) \quad \text{Eq. 30} \]

4.1.2.3.2 – Serviceability Limit State (Limiting Stress – Service II)

The serviceability criterion for deflection was based purely on the stiffness of the section without regard to the material composition of the plate. This same approach cannot be employed with the serviceability criterion based on limiting stress because of the difference in material properties of the components. When considering the assumption that plane sections remain plane after bending, the result is a linear strain distribution through the depth of the section. This relationship cannot be extended to the determination of stresses as a result of the difference in modulus of elasticity between the deck components. The resulting behavior is larger bending stresses within the steel face plates than within the core. This increase in stress is further amplified when comparing sections of equivalent stiffness, but comprised of different component thicknesses and subjected to the same loading. A section with thinner face plates will experience larger stresses than an equivalently stiff plate with thicker face plates as a result of reduction in area over which to distribute the internal force couple.

To account for this behavior in design and satisfy the Service II criterion, plates must be selected such that no yielding occurs under the prescribed loading conditions [Table 19]. As a result of variability and uncertainty in steel plate cost and availability, a blank approach was employed in the analysis for this serviceability criterion. While the required stiffness has been established based on the AASHTO LRFD deflection criterion, the component thicknesses to yield these stiffnesses are unlimited. Steel face plate sizes, ranging from 1/8 – 1/2 in. (1/16 in. increments), were selected for analysis and the corresponding core thicknesses were determined based on Eq. 26 using the required flexural rigidity as function of girder spacing relationships [Eq. 29 and Eq. 30]. Each of the plates was then subjected to the various loading conditions specified in Table 19. An illustration of the peak stresses within the steel face plate is shown in Figure 81 and Figure 82. It should be noted that the curves represent the peak stresses resulting from the worst case loading scenarios and that those for the fixed edge condition result from negative bending (tension in the top) at the fixed support location. The curves do not appear smooth because of the transitions between the controlling loading conditions resulting from the changes in girder spacing. More detailed comparisons of the response from each of the loading
scenarios as well as the response due to variations in plate thickness are presented in Appendix B.2 – Service II Results.

![Service II Stress Limits vs Girder Spacing for Simple Support Boundary](image1)

**Figure 81 – Peak Normal Stress Comparison of Service II Limit State (Simple Support)**

![Service II Stress Limits vs Girder Spacing for Fixed Boundary](image2)

**Figure 82 – Peak Normal Stress Comparison of Service II Limit State (Fixed Support)**

Based on the peak stresses observed within the plate sizes selected, it is evident that the Service II limit does not control for SPS plates for either of the boundary conditions. There is
likely be some plate thickness less than 1/8 in. that would result in yielding of the plate, but this section would require plates that are beyond practical limits of availability.

4.1.2.3.3 – Strength Limit State (Limiting Stress – Strength I)

Currently, there is little information available about the ultimate strength of SPS and the controlling failure limit state. Martin and Murray’s (2005) attempts to fail a SPS deck resulted in reaching the capacity of the testing frame prior to failure. The expectation is that the mode of failure would likely result from one of two failure modes, a flexural failure mode resulting from the steel plate becoming fully plastic or a shear failure through the core. The following sections present an analysis of the flexural and shear strengths of SPS bridge decks.

4.1.2.3.3.1 – Flexural Strength of SPS Bridge Deck

In flexure, sandwich behavior assumes that the core material provides no resistance to bending and that the face plates carry all of the flexural stresses. In actuality the core material likely provides some resistance to bending, but additional resistance would not be expected to be significant due to the low stiffness.

Similar to the Service II limit state, the analysis of SPS for the Strength I limit state was based on plates with various boundary conditions subjected to different factored loading scenarios. In essence, the plates underwent the same loading scenarios as the Service II case, but with larger loads. The peak stresses within the steel face plate for the Strength I limit state are shown in Figure 83 and Figure 84. Similar to the Service II comparison, it should be noted that the curves represent the peak stresses resulting from the worst case loading scenarios and that those for the fixed edge condition result from negative bending (tension in the top) at the fixed support location while those for the simple support edge conditions result from stresses at the middle of the plate. More detailed comparisons of the response from each of the loading scenarios as well as the response due to variations in plate thickness are presented in Appendix B.3 – Strength I Results. Based on the peak stresses observed all of the plates considered are more than adequate to resist the prescribed Strength I loads as is demonstrated by the lack of the onset of yielding on the extreme faces.
4.1.2.3.3.2 – Shear Strength of SPS Bridge Decks

Little is known about the shear resistance of SPS, but the lack of apparent shear failures (Martin and Murray 2005) and the inherent shear resistance of steel suggest that shear is not
likely to control the strength limit state. Based on its position with respect to the neutral axis, the core material is assumed to resist the majority of the transverse shear stress with a parabolic decrease in stress observed away from the neutral axis. Sandwich theory simplifies the behavior and assumes that the core material resists the shear, with no contribution from the face plates. The resistance provided then becomes a function of the shear stiffness and thickness of the core material, larger and less stiff core materials resulting in more shear deformation. When compared to the flexural deformations and stresses, those resulting from shear deformation are typically negligible (Cook et al. 2001; Ventsel and Krauthammer 2001; Zenkert 1995). A comparison of the peak flexural and transverse shear stresses predicted from the finite element models are presented in Figure 85 and Figure 86 for the Strength I limit state. It should be noted that the peak stresses presented for shear do not necessarily correspond to the same geometric configurations that yield the peak flexural stresses. This occurs because peak normal stresses occur for the thinnest plate whereas the peak transverse shear would be expected to occur with the thinnest core, which would correspond to a thick plate for the same flexural rigidity. Transverse shear stresses for all of the loading scenarios are presented in Appendix B.3 – Strength I Results for each of the plate thicknesses considered.

![Maximum Normal/Transverse Shear Stress vs. Girder Spacing](image)

*Figure 85 – Comparison of Peak Normal and Shear Stresses – Strength I (Simple Support)*
Rather than attempting to quantify the shear resistance of SPS decks, a validation approach was employed to ensure that the system would provide adequate resistance to the anticipated loading. The static test of the full scale panel reached a total load of 260 kips (130 kips per load patch) without experiencing a shear failure. This test was performed on a deck with 0.25 in. plates and 1.5 in. core (Martin and Murray 2005).

To demonstrate the shear resistance of the thinnest plate under consideration, 1/8 in., a panel within the half-scale bridge was tested. The original intent of the validation was to load the deck to twice the Strength I design load (74.5 kips), but the core thickness within the deck was not adequate to achieve the desired loading without yielding the face plates. This behavior was anticipated prior to testing and was deemed acceptable due to the lack of specimens available for testing. As a result of the thin core material, the deck was only loaded to approximately 24 kips to prevent yielding of the deck plate. It should be noted that while the tested specimen is considered to be a half-scale specimen, the provided core thickness of 0.75 in. is only 28% of the core thickness (2.67 in.) that would be recommended to satisfy the AASHTO LRFD deflection criterion as presented in 4.1.2.3.1 – Serviceability Limit State (Deflection Control). The thickness selected based on this stiffness requirement would be expected to be
capable of resisting the desired loading without yield of the face plates assuming the boundary conditions were simple supports.

Figure 66 [pg. 117] illustrates the measured load deflection response at mid-panel for all three tests. The results have been adjusted to yield the relative panel deflection by subtracting the measured girder and cold-form angle deflections. A comparison to the simply supported plate prediction indicates that the panel tends to behave like a simply supported plate, but the measured strain at the location of the cold-formed angles [Figure 69] demonstrates that the panel exhibits some degree of fixity. The observed fixity is likely a result of the deck continuity and the torsional restraint provided by the supporting cold-formed angles and girders.

While these observations are important to understanding the deck behavior, the test objective was to validate the adequacy of the shear resistance. Based on the low expected shear stresses within the panel from sandwich theory (Ventsel and Krauthammer 2001; Zenkert 1995) and the absence of an apparent shear failure during the panel tests, shear will not be considered to be a limiting criterion for the design of SPS decks. It should be noted that the core thicknesses of 1.5 in. and 0.75 in. for the static panel and half-scale bridge, respectively, are thinner than core thicknesses that would be proposed based on the proposed stiffness design approach.

4.1.2.3.4 – Fatigue Limit State (Limiting Stress)

The final limit state considered for the design of SPS decks was the fatigue limit state. While this limit state was not expected to limit the design of the actual deck plate, the work by Martin and Murray (2005) indicated that the connection details are likely the limiting factor in the design. Similar to the Service II and Strength I limit states, the deck was subjected to the loading highlighted in Table 19 [pg. 125]. The key difference is that the resulting stress is a predicted stress range due to live load rather than a peak stress.

Comparisons of the peak stress range for both the simple and fixed boundary conditions are presented in Figure 87 to Figure 90. Also presented are the limiting stress ranges for an infinite life based on the AASHTO LRFD (2004) constant amplitude fatigue threshold stress, $(\Delta F)_{TH}$. The values presented are divided by two to account for the specified design load being two times the design truck (AASHTO LRFD Section 6.6.1.2.5). Category A represents the fatigue resistance of the base metal of the deck whereas Category E represents the resistance of the connections tested by Martin and Murray (2005). Based on the stress limits and peak stress
ranges it is apparent that there exists the possibility of exceeding the fatigue limit for Category A when the face plates become too thin [Figure 88 and Figure 90]. This can be easily rectified by limiting the plate thickness to one for which the fatigue limit is not exceeded. Plates greater than 1/8 in. appear to be adequate for fatigue [Figure 90]. Based on the predicted stress ranges it is also apparent that the connection is more critical than the base metal. While the design of the connection is beyond the scope of this research, the results from this analysis suggest that stresses within the connection can be limited by varying the plate size and properly locating the connection in a region that does not experience high stresses as a result of tire loadings.

A rough example is presented in Figure 91, which illustrates the peak tensile stress range at midspan across the transverse girder span for both the simple and fixed support conditions. The example presented is for a girder spacing of 9 ft where the two tires of the design truck controls with a plate thickness of 1/4 in. and a core thickness of 1.5 in. based on the relationship in Eq. 29 [pg. 133]. Figure 92 illustrates that for the fixed edge condition the peak tensile stresses occur at the face of the support whereas the peak tensile stresses occurs under the loading for the simple support condition. It should be noted that this procedure is presented for illustrative purposes and does not assume that the location recommended is the ideal location, but rather the location based on the loading scenarios considered. An overlay of the influence surfaces for each of the boundary conditions would be more useful in providing the ideal connection location. As a result of the uncertainty, this type of analysis is critical when considering the design and placement of the connection details. A further investigation of this behavior is warranted, but beyond the scope of this research.
Figure 87 - Peak Normal Stress Range - Fatigue Limit State (Simple Support)

Fatigue Stress Range Limits vs Girder Spacing for Simple Support Boundary

Figure 88 - Peak Normal Stress Range - Fatigue Limit State (Fixed Support)

Fatigue Stress Range Limits vs Girder Spacing for Fixed Boundary
Figure 89 - Peak Plate Normal Stress Range - Fatigue Limit State (Simple Support)

Figure 90 - Peak Plate Normal Stress Range - Fatigue Limit State (Fixed Support)
4.1.2.3.5 – Summary of Design Limit States

The design limit states presented are based on the treatment of the SPS deck as an equivalent plate with both simply supported and fixed supported edges at the locations of the supporting stringers. This approach is assumed to provide a bounded solution because the true
support conditions resulting from deck continuity and stringer torsional resistance is unknown. Based on the plate analyses it became evident that the controlling limit state for the design of SPS decks is the serviceability limit state, with the deflection limit of L/800 specified within the AASHTO LRFD. For deck configurations where the deck plate was less than 1/8 in., the fatigue limit can control when the supports provide full fixity of the edge, but this can easily be accommodated with thicker deck plate. For the fatigue limit state, the connection details appear to be the controlling limit state, but this beyond the scope of this research.

4.1.3 – Methods for Design and Analysis

Based on the plate analyses presented, the design approach employed for SPS will be based primarily on the deflection control serviceability limit state. This method considers two edge conditions for the support at the location of the stringers, but only the simple support will be considered to allow for a conservative design. Sizing of the plate cross-section to satisfy the deflection control serviceability limit assuming simple supports [Eq. 29] will result in thicker face plates for a given total thickness compared to the fixed edge condition. These thicker faceplates will also reduce peak stresses within the deck regardless of the true boundary conditions. For all designs the deck plate thickness should not be less than 3/16 in. to ensure that fatigue of the base material does not occur during the life of the structure. A brief summary of the design procedure is outlined in the following steps.

1. Select girder spacing (as in typical design)
2. Determine the required flexural rigidity assuming simple support conditions
   Girder spacing - \( S \) (ft) \( \rightarrow \) Flexural rigidity – \( D_t \) (kip·in) [Eq. 29]
3. Select plate and core thicknesses to achieve the required flexural rigidity [Eq. 26]
   – Note: ensure that plate thickness is not less than 3/16 in.

4.1.3.1 – Plate Sizing

The sizing of the deck components is fairly straightforward, but for a given faceplate thickness requires the solution of a cubic polynomial [Eq. 31]. This analysis is simplified by solving the equation numerically or graphically in any available numerical software or calculator. To aid in the sizing of the deck an EXCEL worksheet was developed that allows for the user to determine the required core material thickness by providing material properties,
required flexural rigidity, and the desired face plate thickness. While this tool cannot be easily incorporated into this work, the resulting design graph and a sample screenshot of the user input are presented in Figure 93 and Figure 94, respectively. The tool allows the user to select a girder spacing and face plate thickness and then automatically calculates the required core thickness to satisfy the stiffness requirement based on the relationship of Eq. 29. While the EXCEL file cannot be included within the document, it will be available as a separate file (Flexural Rigidity Design Tool.xls). An instructions tab provides the user with detailed steps for using the worksheet.

\[
\frac{2}{3} \left[ \frac{E_p \left( t_p + \frac{t_c}{2} \right)^3}{1 - \nu_p^2} - \left( \frac{t_c}{2} \right)^3 \right] + \frac{E_c \left( \frac{t_c}{2} \right)^3}{1 - \nu_c^2} - D_{t,\text{required}} = 0
\]

Eq. 31

**Figure 93 – Required Core Thickness vs. Girder Spacing and Flexural Rigidity**
### Calculations using Krauthammer

**GIVENS**

- $E_{pl,1} = 29877.77$
- $E_{core} = 126.78$
- $E_{pl,2} = 29877.77$
- $G_{pl,1} = 11607.53$
- $G_{core} = 46.61$
- $G_{pl,2} = 11607.53$
- $\nu_{pl,1} = 0.287$
- $\nu_{core} = 0.36$

**Graph Entries (do not erase)**

- Girder Spacing (ft) = 6
- Plate Thickness = 0.3125 in.
- Flexural Rigidity Required = 19881 kip-in
- Core Thickness Required = 1.65 in.

**Required Modulus**

- Actual Core Thickness = 1.65 in.
- Actual Flexural Rigidity = 19881 kip-in
- Actual Equivalent Modulus = 18511 ksi

### Table 1

<table>
<thead>
<tr>
<th>Plate</th>
<th>0.1875</th>
<th>0.2500</th>
<th>0.3125</th>
<th>0.3750</th>
<th>0.4375</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>2</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>3</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>4</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>5</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>6</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>7</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>8</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>9</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>10</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>11</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>12</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>13</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>14</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>15</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
<tr>
<td>16</td>
<td>0.1875</td>
<td>0.2500</td>
<td>0.3125</td>
<td>0.3750</td>
<td>0.4375</td>
</tr>
</tbody>
</table>

- Solve Eqn
- Calculated

**Figure 94 – Section Design Spreadsheet (Screenshot)**
4.1.3.2 – Classical Plate Method

While there exists some uncertainty in the true nature of the boundary conditions for a SPS bridge deck, the principles employed in the plate analysis approach can be extended to classical plate theory. For the case of a simply supported plate [Figure 71], closed form solutions are readily available for a wide range of loading conditions primarily in the form of series solutions (McFarland 1972; Timoshenko 1959; Ugural 1999; Ventsel and Krauthammer 2001). For the second scenario under consideration, two edges simply supported/two edges fully restrained [Figure 71], additional complexity is introduced and as result closed form solutions are typically limited to few loading scenarios. The solution to this second scenario would likely require the use of an energy approach to achieve convergence.

When considering a simply supported plate subjected to a patch loading, Navier’s solutions provides the most direct solution. A further description of the theory behind Navier’s solution can be found in a textbook on the analysis of plates and shells (McFarland 1972; Timoshenko 1959; Ugural 1999; Ventsel and Krauthammer 2001). The use of Navier’s solution for sandwich plates was presented by Zenkert (1995) and has been extended herein to accommodate a patch loading. The treatment by Zenkert includes a contribution for the shear flexibility of the core material [Eq. 32]. Table 21 presents a summary of the relationships between deflected surface and internal moments and face plate stresses with the dimension terms illustrated in Figure 95.
\[ w(x, y) = w_b(x, y) + w_c(x, y) \]

where
\[
w_b(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} \frac{4P_{pack} \left(1 - \nu_{eq}^2\right)}{\pi^2 mncd} \sin \left( \frac{m\pi}{a} \right) \sin \left( \frac{n\pi y}{b} \right) \sin \left( \frac{m\pi x}{a} \right) \sin \left( \frac{n\pi y}{b} \right) \left[ D \left( \frac{m\pi}{a} \right)^2 + \left( \frac{n\pi}{b} \right)^2 \right] \cdot \sin \left( \frac{m\pi}{a} \right) \sin \left( \frac{n\pi y}{b} \right) \quad \text{Eq. 32} \]

\[
w_c(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} \frac{4P_{pack} \sin \left( \frac{m\pi}{a} \right) \sin \left( \frac{n\pi y}{b} \right) \sin \left( \frac{m\pi x}{a} \right) \sin \left( \frac{n\pi y}{b} \right)}{\pi^2 mncd} \left[ S_{eq} \left( \frac{m\pi}{a} \right)^2 + \left( \frac{n\pi}{b} \right)^2 \right] \cdot \sin \left( \frac{m\pi}{a} \right) \sin \left( \frac{n\pi y}{b} \right) \]

**Table 21 – Summary of Relationships Between Plate Deflected Surface and Moment/Stress**

<table>
<thead>
<tr>
<th>Moment about transverse axis</th>
<th>[ M_x(x, y) = \frac{D}{(1 - \nu_{eq}^2)} \left( \frac{d^2}{dx^2} w_b(x, y) + \nu_{eq} \frac{d^2}{dy^2} w_b(x, y) \right) ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment about longitudinal axis</td>
<td>[ M_y(x, y) = \frac{D}{(1 - \nu_{eq}^2)} \left( \frac{d^2}{dy^2} w_b(x, y) + \nu_{eq} \frac{d^2}{dx^2} w_b(x, y) \right) ]</td>
</tr>
<tr>
<td>Transverse shear force on longitudinal face</td>
<td>[ T_x(x, y) = S_{eq} \left( \frac{d}{dx} w_s(x, y) \right) ]</td>
</tr>
<tr>
<td>Transverse shear force on transverse face</td>
<td>[ T_y(x, y) = S_{eq} \left( \frac{d}{dy} w_s(x, y) \right) ]</td>
</tr>
<tr>
<td>Normal stress in longitudinal direction</td>
<td>[ \sigma_x(x, y) = \frac{M_x(x, y) \zeta_x E_{plate}}{D} ]</td>
</tr>
<tr>
<td>Normal stress in transverse direction</td>
<td>[ \sigma_y(x, y) = \frac{M_y(x, y) \zeta_y E_{plate}}{D} ]</td>
</tr>
<tr>
<td>Transverse shear stress on longitudinal face</td>
<td>[ \tau_{xz}(x, y) = \frac{T_x(x, y)}{D} \left[ E_{plate} t_f \frac{d_{plate}}{2} + E_{core} \frac{t^3_c}{4} \left( 4 - z^2 \right) \right] ]</td>
</tr>
<tr>
<td>Transverse shear stress on transverse face</td>
<td>[ \tau_{yz}(x, y) = \frac{T_y(x, y)}{D} \left[ E_{plate} t_f \frac{d_{plate}}{2} + E_{core} \frac{t^3_c}{4} \left( 4 - z^2 \right) \right] ]</td>
</tr>
</tbody>
</table>
Table 22 presents a comparison of deflection and normal stresses from the finite element solutions to those from the classical plate solution [Eq. 32] summed over 50 terms (m and n) for two plate configurations. The comparisons each consider a sandwich plate subjected to a patch loading at mid-panel with all four edges simply supported. Each plate was subjected to a concentrated wheel load (16 kips) distributed over a patch area of 20 in. x 10 in.

### Table 22 – Comparison of FEA Solution with Classical Plate Solution

<table>
<thead>
<tr>
<th>Case</th>
<th>CASE I</th>
<th>CASE II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Span (a) (ft)</td>
<td>Plate Solution</td>
<td>30</td>
</tr>
<tr>
<td>Transverse Span (b) (ft)</td>
<td>Plate Solution</td>
<td>10</td>
</tr>
<tr>
<td>Plate Thickness (t_p) (in.)</td>
<td>Plate Solution</td>
<td>0.25</td>
</tr>
<tr>
<td>Core Thickness (t_c) (in.)</td>
<td>Plate Solution</td>
<td>2.66</td>
</tr>
<tr>
<td>Mid-panel deflection (\Delta) (in.)</td>
<td>Plate Solution</td>
<td>0.1106</td>
</tr>
<tr>
<td>Longitudinal Normal Stress (\sigma_x) (ksi)</td>
<td>Plate Solution</td>
<td>6.28</td>
</tr>
<tr>
<td>Longitudinal Normal Stress (\sigma_y) (ksi)</td>
<td>Plate Solution</td>
<td>7.03</td>
</tr>
</tbody>
</table>
The deflection results from the two models are in excellent agreement while predicted stresses also agree, but to a lesser extent. This difference can be attributed to the extraction method for stresses within the ANSYS software. The key advantage of classical plate methods of analysis is that they can be implemented using any commercially available software such as EXCEL, Mathcad, and Mathematica. The classical plate approach was employed within this research effort as an additional tool for validating results. A sample Mathcad worksheet is included in Appendix B.4—Classical Plate Analysis Worksheet.

4.2—Deck Design Summary

For SPS to be accepted by designers there exists the need for a practical design methodology, because the system differs from conventional decks, and current design approaches cannot be applied. The design method developed in this section treated the SPS deck as a plate with variable boundary conditions, subjected to bridge design loads. The development of the design method considered the applicable limit states within the AASHTO LRFD (AASHTO 2004) including serviceability, strength, and fatigue and determined that the design is controlled by the deflection criterion within the serviceability limit state. Based on this determination, sizing of SPS decks was determined to be a function of girder spacing. The design method presented provides designers with a simplified method for sizing the SPS deck cross section, but does not give direct consideration to the deck connections which are beyond the scope of this work.
Chapter 5 – Lateral Load Distribution of SPS Bridges

The lateral load distribution behavior of a bridge is a critical component for an economical design. Assumptions behind the principles of lateral load distribution were discussed in Chapter 2, but essentially a 3D structure is reduced to a 2D structure as loads are assumed to be transmitted laterally from the bridge deck to the supporting members. The transmission of loads is expected to be a function of the relative stiffness of the interacting deck and support members (Barker and Puckett 2007). While this methodology is a simplification from the true behavior, it has proven to be an effective method for the design of bridge.

Researchers have investigated lateral load distribution behavior using simplified methods, semi-analytical methods, rigorous analyses, and field testing with varying conclusions. Most researchers have concluded that the parameter with the greatest influence on lateral load distribution behavior is girder spacing, while other parameters such as span length, longitudinal girder stiffness, and deck thickness also contribute, but to a lesser extent. Other considerations such as secondary members have been shown to have an influence, but the degree of influence is much debated. A further discussion of the influence of parameters is presented in 2.3.4.2 – Parameter specific effect on lateral load distribution. This chapter presents the analysis of the lateral load distribution characteristics of SPS bridges through a combination of field investigations, finite element modeling and parametric investigations.

5.1 – SPS Lateral Load Distribution Considerations

When considering the key design aspects of slab-girder bridges, lateral load distribution is the means by which loads are transmitted to the girders. The majority of the provisions within the AASHTO LRFD (AASHTO 2004) are tailored to the design of bridges with conventional materials such as reinforced concrete, prestressed concrete, steel and timber with none specifically devoted or inclusive of new materials such as SPS or FRP. This lack of inclusion is primarily the result of the limited usage of these types of materials and the relatively recent recognition of these materials as viable alternatives to the conventional materials.

While the current provisions within the AASHTO LRFD are not inclusive of these new materials, the primary concern that arises when considering these new deck systems is the apparent reduction in stiffness when compared typical concrete decks. Typical concrete bridge
deck designs are not often controlled by stiffness requirements as most designs typically adhere
to minimum thickness requirements or are designed to satisfy strength requirements. These
congealed deck thicknesses typically range from ~ 6-8 in. depending on the design methodology
employed [2.5.2.1 – Decks in Slab-Girder Bridges] and regional practices. The main difference
with SPS decks is that their designs are intended to minimize weight which is accomplished by
making the deck as thin as practical. Based on the results from the deck design methodology,
stiffness tends to control the design of SPS bridge decks [Chapter 4]. This design approach
results in decks that are adequate for strength, but considerably more flexible than their concrete
counterpart. A comparison is presented in Table 23, demonstrating the relative difference in
stiffness between a concrete deck and variations in SPS deck configurations, all comparisons are
made to an 8 in. unit width concrete section with an assumed compressive strength of 4 ksi, a
typical design configuration for a slab-girder bridge in Virginia.

Results from the NCHRP 12-26 project (Zokaie 1992; Zokaie 2000) suggested that slab
thickness influenced lateral load distribution, but to a lesser extent than some of the other
parameters. It was demonstrated that an increase in slab thickness resulted in a decrease in
lateral load distribution, but the analysis only considered reinforced concrete deck ranging in
thickness from 6-9 in. As demonstrated in Table 23, the stiffness for an equivalent SPS deck
would be significantly lower than concrete decks in this range of thicknesses. Based on the
results of the NCHRP 12-26 project and this difference in stiffness, it would be expected that
SPS bridge decks would exhibit different lateral load distribution characteristics than reinforced
concrete decks. For these reasons the lateral load distribution behavior of SPS bridges warranted
further investigation.
Table 23 – SPS Deck Stiffness Relative to an 8 in. Reinforced Concrete Deck Stiffness

<table>
<thead>
<tr>
<th>SPS Plate Thickness (in.)</th>
<th>SPS Core Thickness (in.)</th>
<th>$E_{SPS1}$ (kip-in²)*</th>
<th>$E_{R,C1}$ (kip-in²)</th>
<th>Stiffness Ratio ($E_{SPS}/E_{R,C}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1875</td>
<td>1.85</td>
<td>137,000</td>
<td>1,846,000</td>
<td>7%</td>
</tr>
<tr>
<td>0.25</td>
<td>1.51</td>
<td>136,000</td>
<td>1,846,000</td>
<td>7%</td>
</tr>
<tr>
<td>0.375</td>
<td>1.05</td>
<td>136,000</td>
<td>1,846,000</td>
<td>7%</td>
</tr>
<tr>
<td>0.1875</td>
<td>2.39</td>
<td>219,000</td>
<td>1,846,000</td>
<td>12%</td>
</tr>
<tr>
<td>0.25</td>
<td>1.98</td>
<td>218,000</td>
<td>1,846,000</td>
<td>12%</td>
</tr>
<tr>
<td>0.375</td>
<td>1.44</td>
<td>218,000</td>
<td>1,846,000</td>
<td>12%</td>
</tr>
<tr>
<td>0.1875</td>
<td>2.84</td>
<td>302,000</td>
<td>1,846,000</td>
<td>16%</td>
</tr>
<tr>
<td>0.25</td>
<td>2.37</td>
<td>301,000</td>
<td>1,846,000</td>
<td>16%</td>
</tr>
<tr>
<td>0.375</td>
<td>1.76</td>
<td>301,000</td>
<td>1,846,000</td>
<td>16%</td>
</tr>
<tr>
<td>0.1875</td>
<td>3.22</td>
<td>384,000</td>
<td>1,846,000</td>
<td>21%</td>
</tr>
<tr>
<td>0.25</td>
<td>2.71</td>
<td>384,000</td>
<td>1,846,000</td>
<td>21%</td>
</tr>
<tr>
<td>0.375</td>
<td>2.04</td>
<td>384,000</td>
<td>1,846,000</td>
<td>21%</td>
</tr>
<tr>
<td>0.1875</td>
<td>3.57</td>
<td>466,000</td>
<td>1,846,000</td>
<td>25%</td>
</tr>
<tr>
<td>0.25</td>
<td>3.01</td>
<td>466,000</td>
<td>1,846,000</td>
<td>25%</td>
</tr>
<tr>
<td>0.375</td>
<td>2.28</td>
<td>464,000</td>
<td>1,846,000</td>
<td>25%</td>
</tr>
</tbody>
</table>

* $E_{SPS}$ calculated based on Eq. 28

5.2 – Investigation Method for Lateral Load Distribution of SPS Bridges

As highlighted in the previous section, researchers have employed numerous methods for investigating lateral load distribution behavior. Some of these methods will be utilized in the investigation of lateral load distribution of SPS bridges. The methods that will be employed herein include field measurement, finite element modeling, stiffened plate modeling, parametric investigation, and available simplified formulas for conventional bridge systems. With the Shenley Bridge being the only SPS bridge structure constructed to date, the analysis presented in the following sections primarily focus on analytical methods with a heavy emphasis on the finite element approach.

5.3 – Field Test of the Shenley Bridge

The field testing of the Shenley Bridge was performed to assess the in-service behavior of a SPS bridge under live load conditions. This testing provided a means to measure lateral load distribution behavior for interior and exterior girders under single and multiple lane loaded conditions. This measured distribution behavior provided in-service data upon which the
analytical models could be compared against to further assess their validity. In addition, this field test allowed for the investigation of the dynamic behavior of a SPS bridge subjected to moving truck loads; the dynamic behavior discussed further in Chapter 6. This section presents the background, results and preliminary findings from the field investigation of the Shenley Bridge performed in June 2005, after approximately 19 months in service.

5.3.1 – Bridge Description

A detailed description of the Shenley Bridge is presented in Chapter 1, but a condensed version is presented again for reference. The Shenley Bridge [Figure 96] was constructed in Saint-Martin, Québec, Canada during the fall of 2003 utilizing a SPS deck as the bridge’s riding surface. The Shenley Bridge has a span of 73.8 ft and a transverse width of 23.3 ft and was designed by IE with detailed finite element analyses, conforming to the Canadian Bridge Design Specification (CHBDC) (CSA 2000). Ten prefabricated SPS panels, eight full size (7.9 ft x 23.3 ft) panels and two scaled (5.4 ft x 23.3 ft) end panels, were used in the construction of the bridge (IE 2004). The panels were made composite with the three longitudinal support girders through slip critical bolts connecting the girder flanges to cold-formed angles welded to the SPS panels. Additionally, the panels were made continuous with slip-critical bolts on the underside between panels and transverse welds along the panel seams on the topside. A cross-section and a plan schematic of the bridge are presented in Figure 97 and Figure 98, respectively.
5.3.2 – Instrumentation

The bridge was instrumented primarily for the investigation of lateral load distribution, dynamic load allowance, and composite action. Displacements and strains were measured at midspan and near the supports on the extreme tension surface of the girders. Additionally, strains on the bottom side of the top flange of the girders were monitored to observe composite behavior [Figure 99 and Figure 100].
Displacements were measured using deflectometers attached to the bottom flange of the three girders with “C”-clamps, and in turn anchored in the creek bed below with wire attached to a concrete-filled masonry block [Figure 101]. The tips of the deflectometers were pre-deflected (pretensioned) before loading allowing for a measurement of relative deflection, an increase in girder deflection resulting in a proportional decrease in tip deflection. The deflectometers were calibrated in the Virginia Tech Structures Laboratory to within 0.001 in. prior to arrival on-site.
Vishay Micro-Measurements strain gauges (CEA-06-125UN-120 [1/8 in. – 120 Ω - G.F. = 2.064]) were used during the testing. At the locations where the strain gauges were to be placed, the surfaces were ground down to remove scale and cleaned to provide a proper surface for mounting the gauges. Each of the gauges was mounted to the girder surface using Vishay Measurements Group M-Bond 200 adhesive [Figure 102].

All of the data was collected using a portable data acquisition system, CR9000 (Campbell Scientific 2006), capable of high speed monitoring [Figure 103]. During testing the acquisition system was powered by battery to help minimize noise in the data.
5.3.3 – Load Cases

The live load testing of the Shenley Bridge was performed using a three-axle dump truck [Figure 104] provided by the Ministry of Transportation of Quebec (MTQ). The testing included both quasi-static and dynamic testing of the bridge. A schematic of the loading scenarios is presented in Figure 105. A further discussion of the dynamic loading scenarios is presented in Chapter 6.

Figure 104 – Three-Axle Dump Truck used for Shenley Bridge Live Load Testing

Figure 105 – Loading Configurations for Shenley Bridge Live Load Tests
Four loading configurations were considered for the single truck configuration and two were considered for the paired truck configuration. These configurations were intended to produce the worst case loading scenarios for the interior and exterior girders. The two paired truck loading configurations were not actually performed, but employ superposition of the single truck configurations. This method was deemed acceptable in the absence of a second truck because the bridge remained in the elastic range throughout the testing. All of the quasi-static tests were performed at “crawl” speeds of 5 mph (8 kph) or less, to minimize the dynamic amplification and allow the truck to follow lines marked on the bridge deck. Displacement and strain data were recorded at a rate of 20 Hz per channel to cover the entire spectrum of the truck crossings. For each load configuration a total of 5 repetitions for each crossing were performed.

5.3.4 – Measured Results

The live load testing of the Shenley Bridge yielded a large volume of data that required interpretation and analysis. Each of the truck crossings was evaluated to determine if data was recorded and there were no significant variations from the other crossings in the same loading configuration. Some of these variations included noise in the data, measured data not starting from the original baseline, and data not being recorded for a given instrument. Data that included these variations were either manually repaired or eliminated from consideration if the variations were substantial. Figure 106 demonstrates a typical displacement response with dramatic spikes in the data for two of the runs, these spikes were attributed to noise and were manually removed from the data set. Additionally, the baseline for the response starts below zero; for this type of scenario the data is adjusted and relative displacements are considered.

Each of the runs lasted the duration of the truck crossing, but the data recording duration was not always consistent with some runs yielding more data than others. To account for this in the data interpretation, the peak deflection was used to align the data from each run with the peak from the first run in the series. The peak deflection method was also used to align the strain data because the time basis for the strains and displacements was the same.
In general, the displacement measurements were more consistent between crossings for a given load configuration and displayed less noise than the strain measurements, but both instruments were susceptible to the variations previously described. Figure 107 and Figure 108 show typical representations for displacement and strain response under quasi-static loadings. A notable trend when considering the strain response is the difference in strains between gauges at the same height, but on opposite sides of the web. The difference can be attributed to a torsional effect during the loading as a result of the load not being symmetric. The complete set of displacement and strain responses are not presented in the main body, but the unadjusted curves are presented in Appendix A – Additional Results.
Figure 107 - Typical Displacement Response for Quasi-Static Load Case C - Interior girder

Figure 108 - Typical Strain Response for Quasi-Static Load Case C - Interior girder
5.3.5 – Lateral Load Distribution Factors

For a multiple girder bridge system, lateral load distribution represents the fraction of a force effect such as moment or shear that is resisted by a given girder. The magnitude of this fractional force effect is difficult to quantify, especially in an uncontrolled setting such as field testing. Researchers have utilized various techniques for the determination of lateral load distribution from field measurements, but the most widely accepted methods consider displacement and/or strain response in the determination. Displacements and strains are known to be directly related to the desired force effect by derivatives of the equation of the elastic curve for a beam. The displacements and strains recorded during testing were primarily located at midspan and as a result would be related to the lateral load distribution for moment rather than shear. Determination of shear distribution factors would require measurement of the maximum shear strain response or end reactions, but that was beyond the scope of the field test objectives. The method used in this research to calculate lateral load distribution or distribution factors \((DF)\) is a ratio of the maximum displacement/strain response in a girder to the summation of the displacement/strain response in all girders (Eq. 33). This ratio is then multiplied by the number of trucks \((N)\) used during testing to make the distribution factor on a per truck basis. This definition for distribution factor has been widely used by researchers conducting live-load tests (Stallings and Yoo 1993; Waldron et al. 2005).

\[
DF_i = \frac{\Delta_{\text{max},i}}{\sum_{i=1}^{\# \text{girders}} \Delta_{\text{max},i}} \cdot N_{\text{trucks}} = \frac{\varepsilon_{\text{max},i}}{\sum_{i=1}^{\# \text{girders}} \varepsilon_{\text{max},i}} \cdot N_{\text{trucks}} \quad \text{Eq. 33}
\]

Table 24 presents a summary of the calculated distribution factors for displacements and strains for each girder and all of the single truck loading configurations. The values presented represent the average for all of the repetitions in a given load case which results in a better representation of behavior than a single data point. The distribution factors can be considered representative of the amount of loading that is transferred to a given girder from the deck; higher distribution factors indicating a more heavily loaded girder. With the truck factor \((N)\) included the distribution factors represent the fraction of a single truck load resisted by a girder for a load configuration, the same basis as the distribution factors from the AASHTO LRFD specification.
Table 24 – Summary of Lateral Load Distribution Factors for Single Truck Loading

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Girder A (exterior)</th>
<th>Girder B (interior)</th>
<th>Girder C (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.654 0.676</td>
<td>0.321 0.279</td>
<td>0.024 0.045</td>
</tr>
<tr>
<td>B</td>
<td>0.146 0.166</td>
<td>0.372 0.412</td>
<td>0.482 0.422</td>
</tr>
<tr>
<td>C</td>
<td>0.479 0.497</td>
<td>0.359 0.386</td>
<td>0.162 0.117</td>
</tr>
<tr>
<td>E</td>
<td>0.328 0.327</td>
<td>0.380 0.427</td>
<td>0.291 0.246</td>
</tr>
</tbody>
</table>

Similarly, Table 25 presents the calculated distribution factors for the paired truck loading configurations. It is easily observed that the results for the paired truck configuration are larger than the single truck; this is expected because the presence of two trucks increases the load on the entire structure and in turn requires each of the girders to resist a higher percentage of a single truck weight.

Table 25 – Summary of Lateral Load Distribution Factors for Paired Truck Loading

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Girder A (exterior)</th>
<th>Girder B (interior)</th>
<th>Girder C (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A + B</td>
<td>0.849 0.894</td>
<td>0.689 0.676</td>
<td>0.462 0.430</td>
</tr>
<tr>
<td>C + A⁻¹</td>
<td>0.496 0.520</td>
<td>0.679 0.660</td>
<td>0.824 0.820</td>
</tr>
</tbody>
</table>

The live load testing yielded displacement and strain results for all three girders in the Shenley Bridge; these results are essential in determining the relative response of each girder, but not in the determination of the distribution that would be used in design. Design practice would utilize the worst case distribution factor based on girder location, with the possibility of different distribution factors for interior and exterior girders.

A comparison of the critical distribution factors to code provisions for interior and exterior girders as well as single and paired loadings is presented in Table 26, Figure 109 and Figure 110. The code predictions presented are considering the deck to be concrete because no provisions exist for SPS decks within any of the specifications. A concrete deck is the most common system used in the United States and serves as a good baseline for comparison. The lever rule is also presented because code provisions allow it as an alternative method for bridges.
outside the range of applicability stipulated. It also serves as an upper bound since no lateral load distribution is permitted across interior girders which are assumed to be hinged. For completeness the distribution assuming an infinitely stiff deck is presented to highlight the lower bound. The distribution factors presented from the measured results represent the fraction of a single truck resisted by a girder [Eq. 35 - pg. 213]. This allows for direct comparison between the code predictions and also between the single and two truck loading configurations.

*Table 26 – Critical Distribution Factors for Shenley Bridge vs. Code Provisions*

<table>
<thead>
<tr>
<th>Girder</th>
<th>Load Case</th>
<th>Test Results</th>
<th>Predictions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Δ</td>
<td>ε</td>
</tr>
<tr>
<td>One Truck</td>
<td>Ext. A</td>
<td>0.654</td>
<td>0.676</td>
</tr>
<tr>
<td></td>
<td>Int. E</td>
<td>0.380</td>
<td>0.427</td>
</tr>
<tr>
<td>Two Trucks</td>
<td>Ext. A+B</td>
<td>0.849</td>
<td>0.894</td>
</tr>
<tr>
<td></td>
<td>Int. A+B</td>
<td>0.689</td>
<td>0.676</td>
</tr>
</tbody>
</table>

*Figure 109 - Lateral Load Distribution Factors for Single Truck Load (Shenley Bridge)*
The distribution factors from the field tests are all within the limits of the code provisions with the exception of the exterior girder subjected to the paired truck loading. The CHBDC provision underestimates the distribution factor for this case by about 10%, but all of the other provisions are conservative. The conservative estimates of distribution factors indicate that the current AASHTO methods are likely adequate for SPS, assuming the behavior to be the same as a concrete deck. This trend is expected because research has indicated that the factor with the most influence on distribution factors is girder spacing; other factors of influence include span length, longitudinal stiffness, deck thickness, but all to a lesser extent.

In all cases considered, the distribution factors approach that of the lever rule predictions more so than that of the equal distribution. This would indicate that the deck is fairly flexible, but not so much that no load is transferred across in interior girder.

While all of the trends tend to indicate that current provisions for lateral load distribution can be applied to SPS bridges, it should be noted that the live load test is only a single data point. Also of note is that the Shenley Bridge utilizes a three girder system with relatively high girder spacing, both of which are not common in the state of Virginia and many other states. For these reasons it would be inappropriate to definitively state that current AASHTO provisions for a concrete deck on steel girders can be directly applied to SPS. Further study of lateral load
distribution through field testing and finite element modeling is expected to yield more data to make this judgment.

5.4 – Validation of Analytical Models

The field testing of the Shenley Bridge suggested that the distribution behavior of SPS bridges might be different than that of a reinforced concrete deck, but the limited program did not allow for a definitive assessment. For these reasons the extension of the finite element model to the determination of lateral load distribution behavior was employed. The models presented in Chapter 3 – Analytical Models were utilized for this analysis.

5.4.1 – Finite Element Model

Based on the results from the validation of the SPS finite element models, it would be expected that the “base” model would be capable of mimicking the response from the live load test of the Shenley Bridge, but the results were somewhat inconclusive as a result of the scatter in the measured data from the initial field test (IE 2004). For this reason all three models were compared to the results from the live load testing of the Shenley Bridge. Comparisons of the deflection and strain response are presented in the following sections. Similar to the results from the laboratory specimen, it is expected that the models will be capable of predicting global response, but may encounter difficulty near the depth of the cold-formed angles because of the modeling simplifications.

5.4.1.1 – Shenley Field Test (June 2005) Comparisons (Deflection and Strain)

5.4.1.1.1 – Load Case A Comparison

Load Case A was intended to produce the produce the worst case loading scenario for the exterior girder (girder A) because the truck was positioned as close to the barrier rail as feasible [Figure 105]. A comparison of the measured deflection to each of the simplified models is presented in Table 27. All of the simplified models predict the deflection of the most heavily loaded girder, girder A, within 5%, but experience a loss of accuracy for the other girders further away from the load. For the interior girder the “base” model results in a difference of about 22% when compared to the measured results, but the difference is reduced as the model complexity is increased. This reduction is likely a result of the improved load sharing caused by the addition
of the cross-bracing addition in the “expanded” model. The “complete” model appears to result in a minor loss of accuracy compared to the “expanded” model; this behavior can be attributed to the increased stiffness of the bridge with the addition of elements used to represent the cold-formed angles. The deflection of the opposite exterior girder (girder C) is considerably lower than that of the other two girders and as a result the percent difference is high. Each of the simplified models predicts some uplift of this girder, but the measured response indicates the opposite. This difference could be attributed to a number of factors, but these differences will not be considered further due to the relatively low magnitude of the response compared to the other girders.

Table 27 – Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case A

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (in.)</th>
<th>Base (in.)</th>
<th>Difference</th>
<th>Expanded (in.)</th>
<th>Difference</th>
<th>Complete (in.)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-0.616</td>
<td>-0.620</td>
<td>1%</td>
<td>-0.599</td>
<td>-3%</td>
<td>-0.592</td>
<td>-4%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-0.302</td>
<td>-0.236</td>
<td>-22%</td>
<td>-0.254</td>
<td>-16%</td>
<td>-0.251</td>
<td>-17%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-0.023</td>
<td>0.037</td>
<td>-263%</td>
<td>0.035</td>
<td>-251%</td>
<td>0.033</td>
<td>-243%</td>
</tr>
</tbody>
</table>

Table 28 and Table 29 present comparisons of the measured strain response to the predicted strain response for the simplified models. The strain response is also presented on a relative basis; the predictions appear inaccurate due to the differences between the measured and predicted response, but many of these differences can likely be attributed to errors associated with strain measurements in general such as system noise.

The trend for the bottom flange strain response is somewhat different than the deflection response in that the increase in model complexity generally results in an improvement in accuracy for all of the girders. Similar behavior is also present in the strain response at the top flange, but the “expanded” model better predicts the behavior than the “complete” model. This trend differs from expectations because the “complete” mode would be expected to more closely match actual data since it includes the contribution of the cold-formed angles. Similar to the deflection comparison the strain in the top flange of girder C will not be given further consideration due to the low magnitude.
Table 28 - Bottom Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case A

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>243</td>
<td>316</td>
<td>30%</td>
<td>284</td>
<td>17%</td>
<td>284</td>
<td>17%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>97</td>
<td>110</td>
<td>13%</td>
<td>117</td>
<td>21%</td>
<td>117</td>
<td>21%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-11</td>
<td>-14</td>
<td>27%</td>
<td>-8</td>
<td>-27%</td>
<td>-8</td>
<td>-27%</td>
</tr>
</tbody>
</table>

Table 29 – Top Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case A

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-53</td>
<td>-85</td>
<td>60%</td>
<td>-53</td>
<td>0%</td>
<td>-49</td>
<td>-8%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-22</td>
<td>-28</td>
<td>27%</td>
<td>-20</td>
<td>-9%</td>
<td>-18</td>
<td>-18%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-2</td>
<td>14</td>
<td>-800%</td>
<td>15</td>
<td>-850%</td>
<td>14</td>
<td>-800%</td>
</tr>
</tbody>
</table>

Without consideration of the response of girder C, all of the simplified models perform reasonably well when compared with the measured results. The “base” model appears to be more flexible than the other two models as is evident by the larger deflections and higher strains in the most heavily loaded girder. When considering the “expanded” and “complete” models, both perform well overall when compared to the measured results, but the latter appears slightly less accurate and is not capable of accurately modeling the behavior at the level of the cold-formed angles.

5.4.1.1.2 – Load Case B Comparison

Load Case B was designed to be part of a tandem loading configuration with the truck positioned 4 ft from the wheel line of Load Case A [Figure 105]. The loading scenario resulted in significant load being applied to the interior (girder B) and exterior (girder C) girders. A comparison of the measured deflections to each of the simplified models is presented in Table 30. Each of the girders provides some resistance to the applied load as demonstrated by the
relative deflections of the girders. The inclusion of the cross-bracing in the “expanded” and “complete” models appears to improve the load sharing between the girders. This sharing was not evident in Load Case A because the loads were positioned far enough from the opposite exterior girder to not influence the response. Similar behavior is also observed in the comparison of the strain response [Table 31 and Table 32].

When comparing the predicted deflection response of the most heavily load girder (girder C), all of the models predict the response within 15%, but the “expanded” model provides the best prediction. The accuracy of the predictions for the interior girder is somewhat reduced, but still reasonable, within about 15%, when considering the predictions of the “expanded” and “complete” models. For the opposite exterior girder (girder A) the deflection predictions for the “base” model most closely agree with the measured deflections, but the model sacrifices accuracy in the other two girders when compared to the other models. The strain comparisons are similar in trend, but the relative difference between the measured and predicted response is relatively high compared to the deflection response; this difference is likely attributed to normal error in strain measurements as previously described.

A consistent trend in all of the models is that the deflections for girder C were underestimated while those for girders A and B were all overestimated. These differences could be attributed to inaccuracies in the models, but a more likely scenario is a difference between the actual and modeled load position and configuration.

Table 30 - Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case B

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (in.)</th>
<th>Base (in.)</th>
<th>Difference</th>
<th>Expanded (in.)</th>
<th>Difference</th>
<th>Complete (in.)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-0.107</td>
<td>-0.115</td>
<td>7%</td>
<td>-0.145</td>
<td>36%</td>
<td>-0.144</td>
<td>35%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-0.285</td>
<td>-0.383</td>
<td>35%</td>
<td>-0.331</td>
<td>16%</td>
<td>-0.328</td>
<td>15%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-0.370</td>
<td>-0.316</td>
<td>-15%</td>
<td>-0.338</td>
<td>-9%</td>
<td>-0.334</td>
<td>-10%</td>
</tr>
</tbody>
</table>
Table 31 - Bottom Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case B

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (µε)</th>
<th>Base Model (µε)</th>
<th>Difference</th>
<th>Expanded Model (µε)</th>
<th>Difference</th>
<th>Complete Model (µε)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>44</td>
<td>43</td>
<td>-2%</td>
<td>65</td>
<td>48%</td>
<td>65</td>
<td>48%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>122</td>
<td>222</td>
<td>82%</td>
<td>173</td>
<td>42%</td>
<td>173</td>
<td>42%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>127</td>
<td>144</td>
<td>13%</td>
<td>153</td>
<td>20%</td>
<td>153</td>
<td>20%</td>
</tr>
</tbody>
</table>

Table 32 - Top Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case B

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (µε)</th>
<th>Base Model (µε)</th>
<th>Difference</th>
<th>Expanded Model (µε)</th>
<th>Difference</th>
<th>Complete Model (µε)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-52</td>
<td>-11</td>
<td>-79%</td>
<td>-11</td>
<td>-79%</td>
<td>-10</td>
<td>-81%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-26</td>
<td>-44</td>
<td>69%</td>
<td>-15</td>
<td>-42%</td>
<td>-13</td>
<td>-50%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-43</td>
<td>-42</td>
<td>-2%</td>
<td>-30</td>
<td>-30%</td>
<td>-28</td>
<td>-35%</td>
</tr>
</tbody>
</table>

5.4.1.1.3 – Load Case C Comparison

For Load Case C the test truck was positioned such that one wheel line was directly over the interior girder and the other wheel line closest to girder A [Figure 105]. This configuration was intended to produce a larger force effect in the interior girder. Based on the measured results, the exterior girder (girder A) experienced a greater force effect than the interior girder (girder B) [Table 33, Table 34, and Table 35]. This behavior is likely the result of load sharing between the interior and the exterior girder; the interior girder shedding load to an already heavily loaded exterior girder. For all of the models the predictions reasonably match the measured results in the most heavily loaded girders; the refined models also result in improved predictions over the “base” model for all of the girders.
### Table 33 - Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case C

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (in.)</th>
<th>Base (in.)</th>
<th>Difference</th>
<th>Expanded (in.)</th>
<th>Difference</th>
<th>Complete (in.)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-0.436</td>
<td>-0.383</td>
<td>-12%</td>
<td>-0.399</td>
<td>-9%</td>
<td>-0.394</td>
<td>-10%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-0.326</td>
<td>-0.359</td>
<td>10%</td>
<td>-0.317</td>
<td>-3%</td>
<td>-0.314</td>
<td>-4%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-0.145</td>
<td>-0.072</td>
<td>-51%</td>
<td>-0.099</td>
<td>-32%</td>
<td>-0.099</td>
<td>-32%</td>
</tr>
</tbody>
</table>

### Table 34 - Bottom Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case C

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>157</td>
<td>182</td>
<td>16%</td>
<td>183</td>
<td>17%</td>
<td>183</td>
<td>17%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>115</td>
<td>203</td>
<td>77%</td>
<td>163</td>
<td>42%</td>
<td>163</td>
<td>42%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>32</td>
<td>24</td>
<td>-25%</td>
<td>46</td>
<td>44%</td>
<td>46</td>
<td>44%</td>
</tr>
</tbody>
</table>

### Table 35 - Top Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case C

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-66</td>
<td>-52</td>
<td>-21%</td>
<td>-36</td>
<td>-45%</td>
<td>-33</td>
<td>-50%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-20</td>
<td>-41</td>
<td>105%</td>
<td>-16</td>
<td>-20%</td>
<td>-14</td>
<td>-30%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-15</td>
<td>-4</td>
<td>-73%</td>
<td>-5</td>
<td>-67%</td>
<td>-4</td>
<td>-73%</td>
</tr>
</tbody>
</table>

#### 5.4.1.1.4 – Load Case E Comparison

Load Case E positioned the test truck such that the wheel lines straddled the interior girders [Figure 105]. This configuration resulted in the interior girder being the most heavily loaded, but also resulted in significant load within the exterior girders. Based on the measured distribution factors, Load Case E resulted in the greatest distribution factor for the interior girder.
[Table 26] even though Load Case C produced the largest deflection in the girder. A comparison of the measured deflection and strain response indicates a relatively uniform distribution of loading within the system [Table 36, Table 37, and Table 38]. For both deflection and strain, the “base” model has trouble predicting the response, especially for the most heavily loaded interior girder. The “expanded” and “complete” models yield improved predictions in both the deflection and strain response for all of the girders.

**Table 36 - Deflections for Shenley Bridge Live Load Test vs. FEA – Load Case E**

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (in.)</th>
<th>Base (in.)</th>
<th>Difference</th>
<th>Expanded (in.)</th>
<th>Difference</th>
<th>Complete (in.)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-0.266</td>
<td>-0.220</td>
<td>-17%</td>
<td>-0.250</td>
<td>-6%</td>
<td>-0.247</td>
<td>-7%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-0.309</td>
<td>-0.400</td>
<td>29%</td>
<td>-0.340</td>
<td>10%</td>
<td>-0.337</td>
<td>9%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-0.235</td>
<td>-0.193</td>
<td>-18%</td>
<td>-0.224</td>
<td>-5%</td>
<td>-0.221</td>
<td>-6%</td>
</tr>
</tbody>
</table>

**Table 37 - Bottom Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case E**

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>92</td>
<td>94</td>
<td>2%</td>
<td>111</td>
<td>21%</td>
<td>111</td>
<td>21%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>131</td>
<td>234</td>
<td>79%</td>
<td>179</td>
<td>37%</td>
<td>180</td>
<td>37%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>71</td>
<td>80</td>
<td>13%</td>
<td>100</td>
<td>41%</td>
<td>100</td>
<td>41%</td>
</tr>
</tbody>
</table>

**Table 38 - Top Flange Strain for Shenley Bridge Live Load Test vs. FEA – Load Case E**

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured (με)</th>
<th>Base Model (με)</th>
<th>Difference</th>
<th>Expanded Model (με)</th>
<th>Difference</th>
<th>Complete Model (με)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder A (exterior)</td>
<td>-32</td>
<td>-27</td>
<td>-16%</td>
<td>-22</td>
<td>-31%</td>
<td>-20</td>
<td>-38%</td>
</tr>
<tr>
<td>Girder B (interior)</td>
<td>-21</td>
<td>-46</td>
<td>119%</td>
<td>-13</td>
<td>-38%</td>
<td>-12</td>
<td>-43%</td>
</tr>
<tr>
<td>Girder C (exterior)</td>
<td>-30</td>
<td>-23</td>
<td>-23%</td>
<td>-19</td>
<td>-37%</td>
<td>-18</td>
<td>-40%</td>
</tr>
</tbody>
</table>
5.4.1.2 – Summary of Model Comparison and Selection

Based on the comparison presented in the previous section it appears that the “expanded” and “complete” models more accurately predicted the response from the live load test of the Shenley Bridge. For this reason only these two models will be considered further. The negligible difference between the predicted response of the “expanded” and “complete” model allows for selection of either model, but the “complete” models results in additional complexity without significant gain and for this reason will no longer be considered. The modeling configuration that will be used for the remainder of this research will be the “expanded” model.

5.4.1.3 – Shenley Field Test (June 2005) Comparisons (Distribution Factors)

The “expanded” finite element model provided reasonable predictions of the deflection and strain response for each of the loading scenarios and would be expected to equally predict distribution behavior. Table 39 is a summary of the critical measured distribution factors and corresponding distribution factors calculated using Eq. 33 with the response \( R_i \) represented by the midspan nodal deflection and midspan strain on the tension flange of the girders. Also presented in Table 39 is a comparison to the AASHTO LRFD, AASHTO Standard, CHBDC, and lever rule. In the cases considered, the predictions from the finite element model produced conservative or about equal results when compared to the measured results [Figure 111 and Figure 112]. These results would suggest that for the Shenley Bridge current code provisions are adequate for determining lateral load distribution, assuming the behavior of SPS to be similar to that of a conventional concrete deck bridge, but it should be noted that these results are for a single bridge and this matter warrants further investigation.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Girder</th>
<th>Load Case</th>
<th>Measured Results</th>
<th>Finite Element Model</th>
<th>Code Predictions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>( \Delta ) &amp; ( \varepsilon )</td>
<td>( \Delta ) &amp; ( \varepsilon )</td>
<td>LRFD Spec</td>
</tr>
<tr>
<td>One Truck</td>
<td>Ext.</td>
<td>A</td>
<td>0.654 0.676</td>
<td>0.732 0.722</td>
<td>0.849</td>
</tr>
<tr>
<td></td>
<td>Int.</td>
<td>E</td>
<td>0.380 0.427</td>
<td>0.418 0.460</td>
<td>0.526</td>
</tr>
<tr>
<td>Two Trucks</td>
<td>Ext.</td>
<td>A+B</td>
<td>0.849 0.894</td>
<td>0.912 0.890</td>
<td>0.942</td>
</tr>
<tr>
<td></td>
<td>Int.</td>
<td>A+B</td>
<td>0.689 0.676</td>
<td>0.716 0.741</td>
<td>0.889</td>
</tr>
</tbody>
</table>

Table 39 - Comparison of Critical Distribution Factors for Shenley Bridge to FE Model
Figure 111 – FEA Distribution Factors vs. Measured and Code (Single Truck)

Figure 112 – FEA Distribution Factors vs. Measured and Code (Paired Trucks)
5.4.2 – Stiffened Plate Model

Similar to the validation for the finite element model, a validation of the stiffened plate representations using the energy approach was carried out using the commercial software package Mathematica 5.2. Kukreti and Cheraghi’s (1993) shape function was used to develop the energy equation for the entire stiffened plate system. The energy of the entire system was then numerically minimized, based on the Ritz method, to determine the unknown coefficients of the shape function. These coefficients were then substituted into the shape function which was in turn used to calculate deflections.

5.4.2.1 – Shenley Field Test Comparisons (June 2005)

Three equivalent plate models were compared to the field test results of the Shenley Bridge. The first model, model A, was the simplest and included only the energy contribution of the deck and the longitudinal girders over the entire span length. The second model, model B, was further expanded to include the contribution of the lateral bracing along the span. Lateral bracing between the supports in the Shenley Bridge consists of a series of angle sections while the end diaphragms consist of built-up I-sections. For simplification, members with a stiffness equivalent to the internal braces were used at all of the lateral brace locations including the location of the end diaphragms. The final model, model C, extends model B to account for the actual simple span length between the bearings, center-to-center of bearings, rather than the entire span of the bridge. A comparison of the three models to the measured results from the June 2005 Shenley Bridge field tests are presented in the following sections.

5.4.2.1.1 – Load Case A

Load Case A results in the worst case loading scenario for the exterior girder (girder A) and as a result the greatest deflection for that girder. Each of the stiffened models provide excellent predictions for girder A and very reasonable predictions for the interior girder as shown in Table 40 and Figure 113. Similar to the predictions from the FE models, the predictions for the opposite exterior girder (girder C) are not in agreement. This is due to the relatively low magnitude of the deflection and differences in the load sharing behavior of the actual bridge and the simplified models. Based on the results presented, all of the models provide good predictions
of the deflection response, but significant improvement is observed with the inclusion of the lateral bracing members.

Table 40 – Comparison of Equivalent Plate Solution for Load Case A

<table>
<thead>
<tr>
<th>Girder</th>
<th>A (exterior)</th>
<th>B (interior)</th>
<th>C (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured (in.)</td>
<td>-0.616</td>
<td>-0.302</td>
<td>-0.023</td>
</tr>
<tr>
<td>Model A (in.)</td>
<td>-0.682</td>
<td>-0.240</td>
<td>0.036</td>
</tr>
<tr>
<td>% difference</td>
<td>11%</td>
<td>-21%</td>
<td>-258%</td>
</tr>
<tr>
<td>Model B (in.)</td>
<td>-0.669</td>
<td>-0.265</td>
<td>0.049</td>
</tr>
<tr>
<td>% difference</td>
<td>9%</td>
<td>-12%</td>
<td>-314%</td>
</tr>
<tr>
<td>Model C (in.)</td>
<td>-0.554</td>
<td>-0.215</td>
<td>0.039</td>
</tr>
<tr>
<td>% difference</td>
<td>-10%</td>
<td>-29%</td>
<td>-271%</td>
</tr>
</tbody>
</table>

Figure 113 – Comparison of Stiffened Plate Model Deflection Predictions for Load Case A

5.4.2.1.2 – Load Case B

The configuration of Load Case B results in significant deflections in all three girders, but girders B and C are the most heavily loaded as shown in Table 41 and Figure 114. For this load case, model A appears to be too flexible when compared to the measured deflections of the most heavily loaded girder. This can likely be attributed to the limited load sharing capacity due to the exclusion of the lateral bracing. A comparison of the deflection predictions for models B
and C highlights the contribution of the lateral bracing. Model B provides excellent predictions of the deflections for the two exterior girders, but significantly overestimates the deflection of the interior girder. Similar behavior is observed in the predictions from model C, but the reduction in length results in an overall decrease in deflection as would be expected. Based on the results presented both models B and C provide reasonable predictions of deflection whereas model A appears to be too inconsistent when compared to the measured results.

Table 41 – Comparison of Equivalent Plate Solution for Load Case B

<table>
<thead>
<tr>
<th></th>
<th>Girder A (exterior)</th>
<th>Girder B (interior)</th>
<th>Girder C (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured (in.)</td>
<td>-0.107</td>
<td>-0.285</td>
<td>-0.370</td>
</tr>
<tr>
<td>Model A (in.)</td>
<td>-0.049</td>
<td>-0.536</td>
<td>-0.296</td>
</tr>
<tr>
<td>% difference</td>
<td>-54%</td>
<td>88%</td>
<td>-20%</td>
</tr>
<tr>
<td>Model B (in.)</td>
<td>-0.106</td>
<td>-0.424</td>
<td>-0.353</td>
</tr>
<tr>
<td>% difference</td>
<td>0%</td>
<td>49%</td>
<td>-5%</td>
</tr>
<tr>
<td>Model C (in.)</td>
<td>-0.079</td>
<td>-0.365</td>
<td>-0.283</td>
</tr>
<tr>
<td>% difference</td>
<td>-26%</td>
<td>28%</td>
<td>-24%</td>
</tr>
</tbody>
</table>

Figure 114 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case B
5.4.2.1.3 – Load Case C

The configuration of Load Case C results in significant deflections in all three girders similar to Load Case B, but for this load case girders A and B are the most heavily loaded as shown in Table 42 and Figure 115. For this load case, model A is still somewhat inconsistent and overestimates the deflection for the interior girder and underestimates the deflection for the exterior girders. Both models B and C result in an improvement in the prediction over model A, but suggest a more even deflection response for girders A and B. This trend differs from the measured response where girder A is shown to be more heavily loaded. It should be noted that the stiffened plate model predictions are in agreement with those of the FE model suggesting that either the models are not capable of accurately predicting the response or there is a load positioning error within the field measurements [Table 43]. Similar to Load Case B, models B and C provide reasonable predictions of deflection for Load Case C whereas model A is somewhat inconsistent when compared to the measured results.

<table>
<thead>
<tr>
<th>Table 42 - Comparison of Equivalent Plate Solution for Load Case C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
</tr>
<tr>
<td>Measured (in.)</td>
</tr>
<tr>
<td>Model A (in.)</td>
</tr>
<tr>
<td>% difference</td>
</tr>
<tr>
<td>Model B (in.)</td>
</tr>
<tr>
<td>% difference</td>
</tr>
<tr>
<td>Model C (in.)</td>
</tr>
<tr>
<td>% difference</td>
</tr>
</tbody>
</table>
Table 43 - Equivalent Plate Solution (Load Case C) vs. “Expanded” FE Model

<table>
<thead>
<tr>
<th>Girder</th>
<th>A (exterior)</th>
<th>B (interior)</th>
<th>C (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FE Model (in.)</td>
<td>-0.399</td>
<td>-0.317</td>
<td>-0.099</td>
</tr>
<tr>
<td>Model A (in.)</td>
<td>-0.353</td>
<td>-0.504</td>
<td>-0.025</td>
</tr>
<tr>
<td>% difference</td>
<td>-11%</td>
<td>59%</td>
<td>-75%</td>
</tr>
<tr>
<td>Model B (in.)</td>
<td>-0.403</td>
<td>-0.406</td>
<td>-0.074</td>
</tr>
<tr>
<td>% difference</td>
<td>1%</td>
<td>28%</td>
<td>-25%</td>
</tr>
<tr>
<td>Model C (in.)</td>
<td>-0.325</td>
<td>-0.349</td>
<td>-0.054</td>
</tr>
<tr>
<td>% difference</td>
<td>-19%</td>
<td>10%</td>
<td>-46%</td>
</tr>
</tbody>
</table>

Figure 115 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case C

5.4.2.1.4 – Load Case E

The configuration of Load Case E produces a somewhat uniform deflection distribution amongst all three girders with the deflection of the interior girder being the greatest as shown in Table 44 and Figure 116. For this load case, model A remains inconsistent, overestimating the deflection for the interior girder and underestimating the deflection for the exterior girders. Model B provides good predictions for the exterior girders, but overestimates the interior girder deflection. Model C results in an overall reduction in deflection due to the shorter span length, bringing the interior girder deflection more in line with the prediction, but also reducing the
exterior girder deflections. None of the models perform exceptionally well, but model C yields the most consistent predictions for all of the girders.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Measured (in.)</th>
<th>Model A (in.)</th>
<th>% difference</th>
<th>Model B (in.)</th>
<th>% difference</th>
<th>Model C (in.)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (exterior)</td>
<td>-0.266</td>
<td>-0.154</td>
<td>-42%</td>
<td>-0.222</td>
<td>-16%</td>
<td>-0.173</td>
<td>-35%</td>
</tr>
<tr>
<td>B (interior)</td>
<td>-0.309</td>
<td>-0.582</td>
<td>88%</td>
<td>-0.448</td>
<td>45%</td>
<td>-0.388</td>
<td>26%</td>
</tr>
<tr>
<td>C (exterior)</td>
<td>-0.235</td>
<td>-0.145</td>
<td>-38%</td>
<td>-0.213</td>
<td>-10%</td>
<td>-0.166</td>
<td>-30%</td>
</tr>
</tbody>
</table>

Figure 116 - Comparison of Stiffened Plate Model Deflection Predictions for Load Case E

5.4.2.2 – Summary of Stiffened Plate Comparisons

A comparison of the predicted deflections from the equivalent plate models demonstrate that the models are capable of predicting the overall behavior, but are somewhat lacking with respect to accuracy. Model A appears is somewhat inconsistent when compared to the measured results because the deflections are consistently higher for the most heavily loaded girders and typically lower for the remaining girders. Model B includes the contribution of the bracing which appears to aid in the load distribution between the girders. This behavior is in agreement
with the conclusions of other researchers suggesting that lateral bracing significantly contributes to the lateral load distribution behavior. Predictions from Model C are similar to, but lower than those of Model B due to the reduction in span length. The predictions from model C tend to yield more consistent predictions for each load case, but not always the most accurate prediction for a given girder.

5.4.2.3 – Comparison of Stiffened Plate Distribution Factors

Figure 117 – Figure 120 present comparisons of the predicted distribution factors from the stiffened plate models to those from the field testing and the finite element models. Considering the overall behavior, the stiffened plate models provide reasonable predictions, but is somewhat inconsistent for certain cases. For this reason these models will not be considered further in this research, but the model does warrant further investigation, especially considering the numerous simplifications utilized in their development.

![Analytical Distribution Factor Predictions vs. Measured](Image)

*Figure 117 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case A)*
Figure 118 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case B)

Figure 119 – Distribution Factor Comparison – Measured, FEM, Plate (Load Case C)
When comparing the differences between SPS construction and a conventional material such as concrete, the difference that stands out with respect to lateral load distribution behavior is deck thickness and stiffness, as highlighted in Table 23 (pg. 153). This difference would be expected to yield larger distribution factors for SPS as a result of the increased flexibility or decreased stiffness as compared to a concrete deck. The field investigation suggested that this assumption might be true as the measured results approached the predictions of the lever rule, but this assessment was not sufficient to truly characterize the lateral load distribution behavior of SPS. As a result of there being only one SPS bridge, the primary mechanism available for investigating this behavior is finite element modeling.

The final phase of the investigation of lateral load distribution was an investigation of the parameters believed to be of influence. This was accomplished through a finite element investigation of various hypothesized SPS bridges. The parameters investigated were based on the literature presented in Chapter 2 and included girder spacing, span length, deck thickness, and number of loaded lanes. It should be noted that the longitudinal stiffness parameter was not explicitly considered as a result of the influence of the designer’s choice on member selection.
and design, but will be considered in the interpretation as the design approach employed was consistent for all of the bridges investigated.

5.5.1 – Investigated Parameters

The parameters selected were assumed to have some degree of influence on lateral load distribution primarily based on the results of the NCHRP Project 12-26 (Zokaie 1992). A summary of all of the parameters investigated is in Table 45. The parameters were selected based of assumed practical limits of short-medium span bridges and also considering the limitation ranges for distribution factors within the AASHTO LRFD (2004). The combinations of parameters resulted in 150 total models for consideration.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Parameter Increments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Length (ft)</td>
<td>40</td>
<td>120</td>
<td>40 60 80 100 120</td>
</tr>
<tr>
<td>Girder Spacing (ft)</td>
<td>4</td>
<td>12</td>
<td>4 6 8 10 12</td>
</tr>
<tr>
<td>Deck Plate Thickness (in.)</td>
<td>3/16</td>
<td>3/8</td>
<td>3/16 1/4 3/8</td>
</tr>
<tr>
<td>Deck Core Thickness (in.)</td>
<td>1.05</td>
<td>3.57</td>
<td>Determined using deck design methodology (based on girder spacing)</td>
</tr>
<tr>
<td>Number of Loaded Lanes</td>
<td>1</td>
<td>2</td>
<td>1 and 2 lanes</td>
</tr>
</tbody>
</table>

5.5.2 – Parametric Model Development

The modeling approach used for the parametric investigation was based on the “expanded” model since it was proven to be accurate in predicting lateral load distribution behavior. The main difficulty that results from a parametric investigation is the time required to construct the models. In addition, for a bridge the exact loading location is unknown prior to the analysis, requiring the loading to be placed at multiple locations to yield the worst case loading scenario.

To carry out this challenge a parametric bridge model was created using the ANSYS Parametric Design Language (ADPL). The model allows the user to provide a minimal number of input parameters and then constructs the bridge model, applies a series of loads, solves the
model, and writes the results to an output file. A generic flow diagram of the parametric process is illustrated in Figure 121.

Figure 121 – ANSYS Parametric Bridge Model Flow Diagram

5.5.2.1 - Parametric Model Capabilities

The parametric model was designed for right bridges (no skew) with a rectangular mesh pattern and is capable of analyzing multiple point load configurations. The current configuration is for one or two trucks configured in opposing directions, limiting the loading scenarios to the two lane loading case. For the case of one truck on the bridge, the parametric model allows the truck to travel along the entire length of the bridge and also transversely across the entire width, limited by the bounds imposed by the user [Figure 122]. For the two truck loading configuration, both trucks can be incremented along the entire length of the bridge, but are limited to half the bridge width in the transverse direction [Figure 123]. These loading options allow for the trucks to be positioned incrementally to yield the greatest response in any girder for the case of one or two truck loadings, but also require a large number of load cases and long solution times to cover the entire bridge at reasonably small increment lengths.
5.5.2.2 – Parametric Investigation Methodology and Assumptions

To ensure that the behavior would not be adversely influenced by an unreasonable structural behavior such as excessive deflection or stresses, each bridge was individually designed following the bridge design approach given in AASHTO LRFD. With no provisions currently in AASHTO LRFD for the design of a SPS bridge deck system, a number of assumptions had to be made. Presented below is a summary of the design methodology and key assumptions.
1. Select Girder Spacing
2. Select Span Length
3. Select Plate Size
4. Select/Design Core Thickness based on Deck Design Methodology (Chapter 4)
   - Core thickness directly related to girder spacing
5. Design Girders*
   - Assume two-lane bridge (2-12 ft lanes plus overhang)
   - Overhang assumed to be ≤ 30% of girder spacing
   - All girders designs equal based on the worst case (interior or exterior)
   - Distribution factors determined using equations for concrete deck on steel girders (properties and thickness of SPS deck)
   - Dead loads
     - Self-weight (deck and girders)
     - Additional attachments = 5% of non-composite girder self-weight
     - Parapet weight = 0.6 k/ft
     - Future wearing surface = 30 psf
   - Offset distance represented by the longitudinal cold-formed angle between the deck and girder assumed to be constant = 7.95 in.
   - Deck and girders assumed to be fully composite with entire deck effective (effective width = girder spacing)
   - Girder were designed such that section maintained 15-20% reserve capacity based on the controlling design limit state (the section was not optimized for economy)
   - Girders designed such that no stiffeners were required
     * An example of the girder design worksheet utilized is in Appendix C.3 – Bridge Design Worksheet.
6. Position truck loads
   - Single truck loading
     - Truck positioned longitudinally using influence lines
       - Incremented longitudinally 12 in. in both directions from critical position
     - Truck incremented transversely across the bridge (within 1 ft of edge)
       - Increment length between 9-10 in.
   - Two truck loading
     - Trucks positioned longitudinally using influence line (no increment)
     - Truck incremented transversely across half of the bridge (within 1 ft of edge)
       - Increment length between 9-10 in.
5.5.3 – Analysis of Results – Parametric Investigation of Lateral Load Distribution

As a result of the numerous bridges and multiple load cases considered, the data extracted from the parametric analysis was voluminous (>13,000 data sets) and is not included herein in its entirety, a sample data set is included in Appendix C.4 – Parametric Results Output. In summary, the output results included general information about the bridge structure (span, girder spacing, plate/core thickness, number of lanes loaded, transverse width, and overhang) in addition to key parameters for determining distribution factors (midspan girder deflection and bottom flange strain for all girders) as well as additional model result data to ensure that key limit states were not exceeded (peak overall deflections and stresses).

The deflection and strain data were used to determine distribution factors for each of the bridges based on Eq. 33 [pg. 162]. A comparison of the range of distribution factors versus girder spacing is illustrated in Figure 124. Based on the output data, the distribution factors are categorized based on location (interior or exterior) and load configuration (single lane double lane loaded), similar to the divisions imposed within the AASHTO LRFD (AASHTO 2004). From the analysis results, the critical distribution factors for each scenario combination (i.e. interior single truck, exterior single truck, interior two trucks, exterior two trucks) were collected for each bridge. The following section is an analysis of the influence of the key parameters on lateral load distribution behavior.

![Figure 124 – Distribution Factor for All Models and Loadings vs. Girder Spacing](image)

**Figure 124 – Distribution Factor for All Models and Loadings vs. Girder Spacing**
5.5.3.1 – Parameter Influence

Previous research has indicated that the parameter with the greatest influence on lateral load distribution behavior is girder spacing with parameters such as span length, longitudinal girder stiffness and deck thickness contributing to a lesser extent (Zokaie 1992). Based on these findings, the data was analyzed to determine if the influence of these factors was any different for a SPS bridge system. A summary of the correlation coefficients with respect to the distribution factors for the key parameters is presented in Table 46 for interior girders and Table 47 for exterior girders, where the correlation coefficient represents the strength and direction of a linear relationship between two variables \((x \text{ and } y)\) as defined by Eq. 34. A correlation coefficient close to one indicates strong correlation while a correlation close to zero indicates weak correlation between variables.

\[
R(x, y) = \frac{\sum(x - \bar{x})(y - \bar{y})}{\sqrt{\sum(x - \bar{x})^2 \sum(y - \bar{y})^2}} \quad \text{Eq. 34}
\]

### Table 46 – Correlation Coefficients for Interior Girder

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Single Truck Interior (Displacement)</th>
<th>Single Truck Interior (Strain)</th>
<th>Two Trucks Interior (Displacement)</th>
<th>Two Trucks Interior (Strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Spacing (S)</td>
<td>0.932</td>
<td>0.961</td>
<td>0.982</td>
<td>0.990</td>
</tr>
<tr>
<td>Span (L)</td>
<td>-0.259</td>
<td>-0.205</td>
<td>-0.111</td>
<td>-0.070</td>
</tr>
<tr>
<td>Total Deck Thickness ((t_p))</td>
<td>0.706</td>
<td>0.720</td>
<td>0.746</td>
<td>0.752</td>
</tr>
<tr>
<td>Plate Flexural Rigidity ((D_t))</td>
<td>0.932</td>
<td>0.961</td>
<td>0.982</td>
<td>0.990</td>
</tr>
<tr>
<td>Longitudinal Stiffness ((K_g))</td>
<td>0.115</td>
<td>0.190</td>
<td>0.270</td>
<td>0.310</td>
</tr>
</tbody>
</table>

### Table 47 – Correlation Coefficients for Exterior Girder

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Single Truck Exterior (Displacement)</th>
<th>Single Truck Exterior (Strain)</th>
<th>Two Trucks Exterior (Displacement)</th>
<th>Two Trucks Exterior (Strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Spacing (S)</td>
<td>0.980</td>
<td>0.983</td>
<td>0.997</td>
<td>0.996</td>
</tr>
<tr>
<td>Span (L)</td>
<td>-0.061</td>
<td>-0.054</td>
<td>-0.022</td>
<td>-0.002</td>
</tr>
<tr>
<td>Total Deck Thickness ((t_p))</td>
<td>0.791</td>
<td>0.772</td>
<td>0.783</td>
<td>0.773</td>
</tr>
<tr>
<td>Plate Flexural Rigidity ((D_t))</td>
<td>0.980</td>
<td>0.983</td>
<td>0.997</td>
<td>0.996</td>
</tr>
<tr>
<td>Longitudinal Stiffness ((K_g))</td>
<td>0.343</td>
<td>0.352</td>
<td>0.370</td>
<td>0.380</td>
</tr>
</tbody>
</table>
The correlation coefficients confirm the expectation that the parameter with the greatest influence is girder spacing, for both interior and exterior girders. There also appears to be a correlation between distribution behavior and plate flexural rigidity, but this is as expected because of the linear relationship between girder spacing and flexural rigidity used in the deck design (Chapter 4). This is also true for deck thickness, upon which the flexural rigidity is based. For this reason it is difficult to assess the influence of plate stiffness on distribution behavior based solely on correlation coefficients. The correlation with span length appears only to be relevant for an interior girder subjected to a single lane loading; this correlation is relatively weak when compared to girder spacing. The longitudinal stiffness parameter also suggests some correlation, but this parameter was not intentionally varied in the parametric investigation.

5.5.3.1.1 – Girder Spacing

A comparison of the critical distribution factors vs. girder spacing is illustrated in Figure 125. For both the interior and exterior girders the two truck loading configuration controls for all of the bridges considered. The relationships appear near linear as demonstrated by the correlation coefficient being close to unity.

![Lateral Load Distribution Factor vs. Girder Spacing](image)

Figure 125 – Lateral Load Distribution Factors (Critical) vs. Girder Spacing
5.5.3.1.2 – Span Length

Comparisons of distribution factors versus span length are presented in Figure 126 and Figure 127 for interior and exterior girders, respectively. When comparing the influence of span length graphically, there is an apparent decrease in distribution factor for the interior girders and almost no change for exterior girders for increases in span length. A further consideration of the interior girders subjected to one [Figure 128] and two trucks [Figure 129] further highlights this relationship. The trends for the interior girders indicate than an increase in span length results in a decrease in distribution, with the relationship more pronounced for the single truck loading configuration than for the two truck loading.

![Lateral Load Distribution Factor vs. Span Length (Interior Girder)](image)

*Figure 126 – Interior Distribution Factors vs. Span Length*
Figure 127 – Exterior Distribution Factors vs. Span Length

Figure 128 – Interior Distribution Factors vs. Span Length (Variable Spacing – 1 truck)
5.5.3.1.3 – Plate Rigidity and Deck Thickness

The influence of deck stiffness would be expected to significantly influence lateral load distribution behavior as a result of the significant difference in deck stiffness when compared to a traditional reinforced concrete deck. A comparison of distribution factors versus flexural rigidity at first appears to validate this assumption [Figure 130 and Figure 131], but it should be noted that the relationship used to determine flexural rigidity [Eq. 29] is directly related to the girder spacing. As a result, it is apparent that this relationship is directly influenced by girder spacing due to its strong correlation with lateral load distribution. This makes it difficult to directly assess the influence of deck stiffness on lateral load distribution behavior. Similar relationships exist for deck thickness [Figure 132 and Figure 133], but the relationships are not linear as those for flexural rigidity. However, it should be noted that a low correlation between deck thickness and girder spacing was observed in the NCHRP 12-26 project; variations of 10% were reported over the range of slab thickness considered (Zokaie 1992). This stronger apparent correlation with SPS is likely due to the difference in deck design methodology when compared to a concrete deck.
**Figure 130 – Interior Distribution Factors vs. Plate Flexural Rigidity**

**Figure 131 – Exterior Distribution Factors vs. Plate Flexural Rigidity**
Figure 132 – Interior Distribution Factors vs. Plate Thickness

Lateral Load Distribution Factor vs. Deck Thickness (Interior Girder)

![Graph showing Lateral Load Distribution Factor vs. Deck Thickness (Interior Girder)]

- Single Truck Interior (Displacement)
- Single Truck Interior (Strain)
- Two Trucks Interior (Displacement)
- Two Trucks Interior (Strain)

Figure 133 – Exterior Distribution Factors vs. Plate Thickness

Lateral Load Distribution Factor vs. Deck Thickness (Exterior Girder)

![Graph showing Lateral Load Distribution Factor vs. Deck Thickness (Exterior Girder)]

- Single Truck Exterior (Displacement)
- Single Truck Exterior (Strain)
- Two Trucks Exterior (Displacement)
- Two Trucks Exterior (Strain)
5.5.3.1.4 – Longitudinal Stiffness Parameter

The influence of the longitudinal stiffness parameter is somewhat difficult to assess primarily because it is strongly influenced by the designer’s selection and design of longitudinal girders. All of the designs employed in this parametric study were arrived at using the same methodology, but there was no attempt made to optimize the designs for economy. Comparison of the lateral load distribution factors for the interior and exterior girder versus the longitudinal stiffness parameter are shown in Figure 134 and Figure 135, respectively. Based on these trends, a relationship with lateral load distribution is not apparent, but consideration of the correlation coefficients suggests than an increase in the longitudinal stiffness parameter, \( K_g \), results in an increase in lateral load distribution. The trends indicate that as the stiffness of the supporting members increase, less of the applied load is shared amongst the girders.

![Figure 134 – Interior Distribution Factors vs. Longitudinal Stiffness (Kg)](image_url)

**Figure 134 – Interior Distribution Factors vs. Longitudinal Stiffness (Kg)**
5.5.3.2 – Summary of Parameter Influence on Lateral Load Distribution

Based on the observed relationships between lateral load distribution and the parameters under consideration, girder spacing maintains the greatest degree of influence. Other parameters shown to influence distribution behavior included span length and the longitudinal girder stiffness parameter, but each of these parameters influenced the distribution behavior significantly less than girder spacing. The plate flexural rigidity and deck thicknesses also appeared to have a significant influence on lateral load distribution behavior, but the degree of influence could not be ascertained due to the coupling with girder spacing as a result of the deck design methodology. These findings suggest that the distribution behavior of SPS may not be significantly affected by secondary parameters and may more closely relate to girder spacing than any other parameter, especially considering the coupled effect of girder spacing and deck flexural rigidity. For this reason, the majority of the remaining comparisons will be made using the girder spacing parameter.
5.5.3.3 – Comparison to Code Predictions and Upper Limits Solutions

When considering the influence of each of the parameters on the distribution behavior of SPS it is apparent that girder spacing is the parameter with the most influence. This interpretation suggests that current AASHTO LRFD provisions may be applicable to SPS. A comparison of each of the key scenarios is presented in Figure 136 to Figure 139. It should be highlighted that all of the code equations utilize semi-empirical equations that were derived for concrete bridge decks, with the exception of the exterior girder subjected to a single lane loading which is based on the lever rule. Also presented are upper bound solutions as determined by the lever rule with multiple presence factors included. The key difference between the lever rule calculation for the code prediction and the upper bound is that the former uses a 2 ft clear distance between the guardrail edge and the load, whereas the upper bound considers a 1 ft clear distance identical to the bounds of the parametric investigation. The 1 ft clear distance was utilized to yield the largest practical distribution factors in the exterior girder as demonstrated in the live load testing of the Shenley Bridge.

![Figure 136 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule (Interior Girder – One Truck)](image-url)
Figure 137 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule (Interior Girder – Two Trucks)

Figure 138 – SPS Distribution Factors vs. AASHTO LRFD and Lever Rule (Exterior Girder – One Truck)
For all of the cases considered, the AASHTO LRFD provides conservative estimates of the distribution behavior of SPS bridges. The interpretation would suggest that AASHTO LRFD can be used to determine distribution factors for SPS bridges in a conservative fashion. This assessment is better illustrated in Figure 140 with conservative estimates falling below the correlation line and unconservative estimates lying above. While a primary objective of this research is to characterize the lateral load distribution behavior of SPS bridges, it is not expected that the behavior will match code provision, considering that these equations were developed empirically for design based on concrete deck bridges.
Similarly, the lever rule provides upper bound limits on the distribution behavior because of the assumption that no load is transferred across interior girders; this behavior is equivalent to that of a very flexible deck. For many instances the lever rule is overly conservative [Figure 141]. However, this method is often used in design without regard for the degree of conservativeness, as in the case of steel grid decks (AASHTO 2004) and can surely be utilized for SPS bridges.
When comparing SPS to conventional bridge decks such as reinforced concrete, the expectation would be that the greater flexibility of SPS would result in larger distribution factors. To establish this comparison, a series of finite element models were developed to determine the distribution characteristics of a comparable reinforced concrete deck. These models were developed on the same basis as the SPS bridges, but utilized an 8 in. thick concrete deck for all of the configurations. The deck thickness was selected such that it would satisfy the empirical design approach within the AASHTO LRFD (AASHTO 2004). Since the original SPS bridge designs were performed as if the decks behaved like concrete, no designs were performed for the girders in this bridge population. The decks of each of the SPS parametric bridges were simply replaced with an 8 in. thick reinforced concrete deck. As a result, one concrete bridge overlaps with three of the SPS bridges for a given girder spacing and span length.

The critical distribution factors for each of the bridge configurations were then determined utilizing the relationships of Eq. 33 [pg. 162]. Due to the similarity of results from the displacement and strain distribution factors only those calculated from deflection are presented for simplicity. A comparison of the distribution factors for the concrete deck bridges
and those for the SPS bridges are presented in Figure 142 to Figure 145 for the interior and exterior girders subjected to both single truck and double truck loadings. From these comparisons it is evident that the distribution behavior for a SPS bridge results in larger distribution factors than a comparable reinforced concrete bridge. While the distribution factors are different, they do not differ by a large degree as might be expected for the difference in stiffness. A comparison of the ratio of distribution factors from each of the two deck types is illustrated in Figure 146 and Figure 147, demonstrating that the increase in distribution is no larger than 20%. Figure 148 demonstrates that the distribution factors from SPS parametric investigation are similar to those reported in the NCHRP 12-26 project, but not always larger. The comparison would suggest that a typical SPS bridge does not differ significantly from the average bridge considered in the NCHRP 12-26 project.

![Figure 142 –Distribution Factors for SPS vs. Reinforced Concrete (Interior Girder)](image)

*Figure 142 –Distribution Factors for SPS vs. Reinforced Concrete (Interior – One Truck)*
Figure 143 – Distribution Factors for SPS vs. Reinforced Concrete (Interior – Two Trucks)

Figure 144 – Distribution Factors for SPS vs. Reinforced Concrete (Exterior – One Truck)
Figure 145 – Distribution Factors for SPS vs. Reinforced Concrete (Exterior – Two Trucks)

Figure 146 – Ratio of SPS LDF to R/C LDF (Single Truck)
Figure 147 – Ratio of SPS LDF to R/C LDF (Two Trucks)

Figure 148 – SPS Distribution Factors to NCHRP Project 12-26
5.5.3.5 – Comparison to NCHRP 12-62 Method

While the research results from the NCHRP Project 12-26 were shown to represent an improvement over the methods included in the AASHTO Standard specification (AASHTO 1996), their use has been the subject of much debate in recent years. Researchers and designers have considered the limited range of applicability, their complexity and the iterative process to be weaknesses in the methodology (Phuvoravan et al. 2004; Puckett et al. 2006). The NCHRP 12-62 project was initiated to develop simple methods capable of estimating distribution behavior, such as modified versions of the lever rule and Henry’s method. Comparisons for each of the key scenarios are presented in Figure 149-Figure 152. These comparisons are presented alongside the current AASHTO LRFD and lever rule estimations for reference.

![Figure 149 – SPS Distribution Factors vs. NCHRP Project 12-62 (Interior – One Truck)](image-url)

*Figure 149 – SPS Distribution Factors vs. NCHRP Project 12-62 (Interior – One Truck)*
Figure 150 – SPS Distribution Factor vs. NCHRP Project 12-62 (Interior – Two Trucks)

Figure 151 – SPS Distribution Factors vs. NCHRP Project 12-62 (Exterior – One Truck)
For the majority of the SPS bridges, the NCHRP 12-62 equations provide conservative estimates of the distribution factors with the exception of a few bridges [Figure 153]. While these equations do not necessarily represent an improvement in accuracy over the current AASHTO LRFD code equations, their application is much simpler.
5.6 – Summary of SPS Lateral Load Distribution Behavior

This section presented the results from the investigation of lateral load distribution behavior of SPS. The initial expectation was that the distribution behavior of SPS might vary from that of conventional reinforced concrete deck systems as a result of the difference in deck flexibility between the two. Evaluation methods included a field investigation of the Shenley bridge for determination of in place distribution behavior, validation of a finite element model for predicting lateral load distribution, and a parametric study of the lateral load distribution characteristics of SPS bridges.

The results of both the field testing and the parametric model validated the expectation that the lower flexibility of SPS influenced lateral load distribution behavior, but not to a significant degree. The degree of influence was determined by comparisons of equivalent concrete deck bridges as well as results from the NCHRP 12-26 project, the foundation of the current AASHTO LRFD.

Conservative estimates using the current AASHTO LRFD code equations demonstrated that SPS bridges can be accommodated by current provisions without modification. However, the comparisons suggest that the current provisions can also yield overly conservative estimates.
and may warrant refinement. Also presented in this section are comparisons with the equations proposed by the NCHRP 12-62 project which have been proposed as a replacement to the current AASHTO LRFD equations. While these methods do not provide a better estimate of distribution behavior for SPS bridges, they do provide a simplification over the current methods.

While both of these provisions were demonstrated to provide conservative predictions of the distribution factors for SPS, the distribution factors for SPS decks were typically larger than those for concrete decks. The difference would not result in a different in girder design, but would result in larger stresses with the girders for the same design.

Based on these comparisons, it is recommended that the current AASHTO LRFD provisions be used for determination of the distribution behavior of SPS. If the recommendations of the NCHRP 12-62 are accepted as a replacement for the current methods, then it would also be acceptable to use those recommendations for SPS bridges.
Chapter 6 - Dynamic Response of SPS Bridge

The dynamic response of bridge structures has been studied by many researchers, but none have developed methods for characterizing the response for bridge-vehicle interaction. The concern for SPS bridges is the dynamic response could be greater than a conventional system because of the system’s lightweight. A more descriptive discussion of bridge dynamic behavior is presented in Chapter 2.

This chapter presents the results of a survey of some the key dynamic characteristics of SPS bridges. Included in this survey are the measured results from dynamic testing of the Shenley bridge and vibration response results from a parametric investigation of SPS bridges. The objective is to assess whether the dynamic response of SPS bridges significantly differs from that of conventional systems such as those with heavier reinforced concrete decks.

6.1 – Field Test of the Shenley Bridge

A description of the Shenley Bridge was presented in Chapter 1 and a description of the testing program was presented in Chapter 5. For this reason only a brief extension of the testing program will be highlighted as it pertains to the dynamic testing. The objective of the dynamic tests was to evaluate the dynamic response of the bridge subjected to moving loads. These tests were intended to demonstrate the dynamic amplification observed over that of static conditions.

The dynamic testing only considered a single truck positioned in three configurations, one configuration was not considered for safety reasons due to the proximity to the guardrail. A graphical presentation of the load cases considered is illustrated in Figure 105. The dynamic tests were performed at speeds of about 50 mph to mimic the behavior of a truck traveling at the highest speed attainable in the approach roadway section. Displacement and strain data were recorded at a rate of 200 Hz per channel to cover the entire spectrum of the truck crossings. Similar to the quasi-static tests, a total of 5 repetitions for each crossing were performed.

6.1.1 – Measured Results

The dynamic testing yielded a large volume of data similar to the quasi-static testing, which required interpretation and analysis. Each of the truck crossings was evaluated to determine if data was recorded and there were no significant variations from the other crossings.
in the same loading configuration. Similar to the quasi-static tests, data that included these variations were either manually repaired or eliminated from consideration if the variations were substantial. A typical displacement response from the dynamic testing is illustrated in Figure 154. The displacement response for all the load cases is presented in Appendix C.2 – Shenley Bridge Field Test Results - Dynamic Tests. The response for strain was similar, but exhibited significant noise in many cases and for that reason is not presented graphically.

![Twanger 1 (Midspan Girder A) Displacement - Loading C](image)

**Figure 154 – Typical Displacement Response for Dynamic Field Test of Shenley Bridge**

### 6.1.1.1 – Dynamic Load Allowance (Impact)

From the live load test of the Shenley Bridge, the dynamic effects considered were the additional deflection and/or strain beyond the static response under the same loading [*Figure 155 and Figure 156*]. In the determination of dynamic amplification or dynamic load allowance (*DLA*), displacement and strain data for the dynamic tests were compared with the data for the static test under the same load configuration. *DLA* is considered in this research to be the additional effect observed under dynamic loading conditions when compared to an equivalent static response and is determined as shown in *Eq. 35.*

\[
DLA = \frac{R_{\text{dynamic}} - R_{\text{static}}}{R_{\text{static}}} = \frac{\Delta_{\text{dynamic}} - \Delta_{\text{static}}}{\Delta_{\text{static}}} = \frac{\varepsilon_{\text{dynamic}} - \varepsilon_{\text{static}}}{\varepsilon_{\text{static}}} \quad \text{Eq. 35}
\]
Comparisons of the dynamic load allowance results for the Shenley Bridge live load test are presented in Table 48 along with the AASHTO and CHBDC specified values. Similar to the method for lateral load distribution, a total of 5 repetitions for each crossing were performed.
The dynamic load allowance values for the cases presented include a maximum and average value for both strain and deflection; the maximum DLA values utilize the minimum static response from all five repetitions and the average DLA values use the average of the static response for all of the static repetitions.

Each of the code predictions utilizes a blanket approach to determine the dynamic amplification and gives no consideration to the type of bridge. The AASHTO standard specification does take into consideration the bridge length which has been shown to be related to fundamental frequency, but still maintains an upper limit of 25% amplification. Two out of the three load cases considered result in conservative predictions when compared with the field measurements. For Load Case E all of the codes underestimate the amplification effects. The CHBDC yields the closest prediction, but it should be noted that the CHBDC prediction is based on a larger truck with more axles and is also overly conservative for the other load cases.

<table>
<thead>
<tr>
<th>Table 48 – Summary of Dynamic Load Allowance for Shenley Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case B (Girder C)</td>
</tr>
<tr>
<td>Max</td>
</tr>
<tr>
<td>Δ ε</td>
</tr>
<tr>
<td>Avg (%)</td>
</tr>
<tr>
<td>Standard Deviation</td>
</tr>
<tr>
<td>LRFD</td>
</tr>
<tr>
<td>Std. Spec</td>
</tr>
<tr>
<td>CHBDC</td>
</tr>
</tbody>
</table>

The underestimation of the code predictions for Load Case E is more pronounced with the deflection than with the strain response. This trend is opposite of what was observed in the determination of distribution factors where the strain response was consistently higher. All five of the static tests for Load Case E resulted in similar deflection and strain values, but the dynamic tests displayed some inconsistencies between runs as shown in Table 49. The deflection and strain values for Runs 1, 3, and 5 are significantly larger than those for Runs 2 and 4, resulting in relatively large dynamic load allowance values for the odd cases. This difference in dynamic response is the primary cause of the large standard deviation for the dynamic load allowance for Load Case E, but the large values cannot be easily explained from the test data. The source of the relatively large dynamic load allowance values for runs 1, 3, and 5 are rather
difficult to explain as a result of the numerous parameters of influence, but one likely source is inconsistencies in load positioning during testing. Another potential source of the additional amplification is due to bounce induced by the deck and roadway approach not being level and support deflection, causing a premature excitation of the truck before it arrived at the critical location. This effect was not quantified during the testing, but is highlighted based on observations during the testing. Other parameters often associated with dynamic amplification include roadway roughness and vehicle suspension response, but these were not quantified in the field testing and as a result cannot be evaluated. It is likely that the roadway surface of the approach span and the support conditions had some influence on the dynamic amplification, but the current data is not sufficient to analyze the influence. These discrepancies suggest that further investigation of dynamic load allowance is warranted.

Table 49 – Summary of Variation in Dynamic Response for Load Case E

<table>
<thead>
<tr>
<th>Run</th>
<th>Δ(dynamic in.)</th>
<th>DLA Max (%)</th>
<th>DLA Avg (%)</th>
<th>ε(dynamic με)</th>
<th>DLA Max (%)</th>
<th>DLA Avg (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 1</td>
<td>0.488</td>
<td>60%</td>
<td>58%</td>
<td>198</td>
<td>50%</td>
<td>48%</td>
</tr>
<tr>
<td>Run 2</td>
<td>0.377</td>
<td>24%</td>
<td>22%</td>
<td>151</td>
<td>15%</td>
<td>14%</td>
</tr>
<tr>
<td>Run 3</td>
<td>0.465</td>
<td>52%</td>
<td>50%</td>
<td>174</td>
<td>36%</td>
<td>35%</td>
</tr>
<tr>
<td>Run 4</td>
<td>0.374</td>
<td>23%</td>
<td>21%</td>
<td>150</td>
<td>14%</td>
<td>13%</td>
</tr>
<tr>
<td>Run 5</td>
<td>0.496</td>
<td>63%</td>
<td>61%</td>
<td>200</td>
<td>53%</td>
<td>51%</td>
</tr>
<tr>
<td>Average - All Runs</td>
<td>0.440</td>
<td>44%</td>
<td>42%</td>
<td>133.7</td>
<td>34%</td>
<td>32%</td>
</tr>
<tr>
<td>Average – Runs 1,3,5</td>
<td>0.483</td>
<td>58%</td>
<td>56%</td>
<td>148</td>
<td>47%</td>
<td>45%</td>
</tr>
<tr>
<td>Average – Runs 2,4</td>
<td>0.376</td>
<td>23%</td>
<td>22%</td>
<td>112</td>
<td>15%</td>
<td>13%</td>
</tr>
</tbody>
</table>

6.1.1.2 – Vibration Characteristics of Shenley Bridge

The primary source of the dynamic amplification is often difficult to quantify mainly because of the large number of parameters involved. One parameter most often associated with dynamic amplification is the fundamental frequency of the bridge (Paultre et al. 1992). To date the Shenley Bridge is the only bridge with frequency data available. Two separate vibration tests were performed to measure the vibration characteristics of the bridge. The first test was performed as part of the validation testing program prior to application of the wearing surface (IE 2004), whereas the second was performed concurrently with the live load testing program after ~19 months in service (Murray 2005). The fundamental frequencies for the first and second
test were 5.8 (November 2003) and 4.5 Hz (June 2005), respectively. These frequencies are well within the expected range of typical short-medium span highway bridges of this span length (Paultre et al. 1992). A comparison of the measured frequency is also within reasonable agreement with the simplified models as proposed by other researchers based on span length [Table 50] (Cantieni 1984; Chan and O'Connor 1990; Tilly 1986). The comparisons suggest that the dynamic behavior of SPS bridges may not be significantly different than that of typical slab girder bridges, but the limited comparative population does not allow for a definitive conclusion.

Table 50 – Comparison of Frequency Predictions for Shenley Bridge

<table>
<thead>
<tr>
<th>Prediction Equation</th>
<th>Predicted Frequency (f)</th>
<th>Prediction/Measured (November 2003)</th>
<th>Prediction/Measured (June 2005)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cantieni</td>
<td>$f = 95.4L^{-0.935}$</td>
<td>5.22</td>
<td>0.90</td>
</tr>
<tr>
<td>Tilly</td>
<td>$f = 82L^{-0.9}$</td>
<td>4.98</td>
<td>0.86</td>
</tr>
<tr>
<td>Chan and O’Connor</td>
<td>$f = \frac{100}{L}$</td>
<td>4.44</td>
<td>0.77</td>
</tr>
</tbody>
</table>

Note: L in meters

6.2 – Finite Element Model for Dynamic Characterization

A detailed model capable of capturing true vehicle-bridge interaction would likely be the best method for investigating the dynamic behavior, but this approach is beyond the scope of this research effort. The method that is employed herein focuses on expanding the previously developed finite element models to predict natural frequencies of SPS bridges. The fundamental frequency of a bridge has been shown to be a key parameter in the dynamic response of a bridge to truck loadings. This will approach also allows for a comparative analysis with conventional bridges to determine if the vibration characteristics of SPS bridges differ significantly.

6.2.1 – Validation of Finite Element Models for Modal Analysis

The first step in the extension of the finite element models to the evaluation of the natural frequencies of SPS was the validation of the models with measured data. The objective was to include in the model only members and elements that influence the response without significantly increasing the complexity of the model. This validation was performed using a trial and error approach to determine the influence of various members on the predicted natural
frequency response. The measured natural frequencies from the acceptance (IE 2004) and live load testing (Murray 2005) of the Shenley Bridge were used for the validations.

The “expanded” finite element model was previously shown to yield good predictions of the global response of the Shenley Bridge; therefore it is assumed that this model is inclusive of all of the primary members that provide flexural stiffness. This is not to say that the other members do not provide additional stiffness, but rather that their contribution is relatively small compared to the members included in the model. Based on this assumption and the relationships between the first flexural frequency and stiffness \[ Eq. 36 \] and \[ Eq. 37 \], it is assumed that the mass contribution from elements not included in the “expanded” model can be included without consideration of their stiffness contribution i.e., maintaining constant stiffness and increasing the mass.

\[
[M]\ddot{u} + [K]u = 0 \quad \Rightarrow \quad (-\omega^2[M] + [K])\phi_i = \{0\} \\
\text{where} \\
\{u\} = \{\phi\}_i \cos \omega t \quad \text{for a linear system} \\
f = \frac{\omega}{2\pi} \
\text{Eq. 36} \\
\text{Eq. 37}
\]

To determine the natural frequency of the Shenley Bridge, these additional elements were included, as distributed masses, one at a time to determine their influence on the frequency response as determined from the “expanded” model. Table 51 summarizes the various elements considered in the validation of the “expanded” model for vibration response.

While the stiffness of these mass elements were not included in the models, reasonable effort was made to locate the mass as close to their true geometric position as possible. This effort was limited somewhat by mesh configuration, because the masses were applied as nodal mass elements. The longitudinal cold-formed angles were distributed at the level of the deck along the length of each of the girders. Similarly the longitudinal edge plates and guardrail masses were distributed along the two edges of the deck. The transverse cold-formed angles between deck panel joints and a 2 in. wearing surface were assumed to be uniformly distributed over the deck surface. For all of the models the boundary conditions were idealized as simple supports.
Table 51 – Summary of Element Consideration for Modal Analysis

<table>
<thead>
<tr>
<th>Model</th>
<th>Longitudinal Girders</th>
<th>Deck</th>
<th>Bracing</th>
<th>Longitudinal Cold-Formed Angles</th>
<th>Edge Plates</th>
<th>Transverse Cold-formed angles</th>
<th>Guardrail</th>
<th>Wearing surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>6</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td></td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

* – stiffness contribution including (model as elements)
X – mass contribution included (modeled as distributed mass elements)

Table 52 presents a summary of the frequencies from the model as well as the measured response from the corresponding field investigation. These frequencies correspond to the first flexural frequencies [Figure 157]. Also included is the total mass of each model to highlight its influence on natural frequency. These comparisons suggest that model 5 best predicts the response of the Shenley Bridge without the wearing surface. This model captures the mass of all of the elements, but only considers the stiffness of the elements included in the “expanded” model. This agreement validates the initial assumption that the members significantly contributing to the model stiffness have been included, but does not account for the likelihood that the boundary restraints are not true simple supports. When considering the comparison of model 6 with the measured response with the addition of the wearing surface the results do not agree as well as the case without the wearing surface. This difference is somewhat difficult to explain, but could be attributed to a number of factors. Considering the bridge as a single degree of freedom system with a constant stiffness, it would be expected that the change in stiffness would be proportional to a change in the inverse of the square root of mass as shown in Eq. 38. A comparison of the change in frequency with the change in mass demonstrates that this relationship does not hold for the case with the wearing surface included. Considering the nature of construction, the actual wearing surface thickness is probably somewhat different than the planned thickness, thicker in this case. In addition, the temperature at testing likely influenced the vibration response with higher frequencies measured during the initial testing (November) than the live load test (June). This behavior was demonstrated in a long term bridge monitoring study by Zhao and DeWolf (2002). Based on these comparisons, these simplified models
provided good estimates of the natural frequency of the Shenley Bridge and will be considered
valid for further investigating vibration response of other SPS bridges.

Table 52 – Comparison of Natural Frequencies of FEA Models to Measured

<table>
<thead>
<tr>
<th>Model</th>
<th>FEA (Hz)</th>
<th>Measured (Hz)</th>
<th>% difference</th>
<th>Model Mass (kip-sec²/in)</th>
<th>% change between models Mass¹/²</th>
<th>% change between models FEA frequency</th>
<th>% change between model Measured frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.7</td>
<td>5.8¹</td>
<td>16%</td>
<td>0.224</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>6.6</td>
<td>6.4</td>
<td>14%</td>
<td>0.233</td>
<td>-1%</td>
<td>-2%</td>
<td>0%</td>
</tr>
<tr>
<td>3</td>
<td>6.4</td>
<td>6.1</td>
<td>10%</td>
<td>0.242</td>
<td>-3%</td>
<td>-2%</td>
<td>0%</td>
</tr>
<tr>
<td>4</td>
<td>6.1</td>
<td>5.9</td>
<td>5%</td>
<td>0.266</td>
<td>-5%</td>
<td>-5%</td>
<td>0%</td>
</tr>
<tr>
<td>5</td>
<td>5.9</td>
<td>5.0</td>
<td>2%</td>
<td>0.287</td>
<td>-4%</td>
<td>-4%</td>
<td>0%</td>
</tr>
<tr>
<td>6</td>
<td>5.0</td>
<td>4.5²</td>
<td>11%</td>
<td>0.397</td>
<td>-15%</td>
<td>-15%</td>
<td>-22%</td>
</tr>
</tbody>
</table>

¹ – measured natural frequency without wearing surface (IE 2004)
² – measured natural frequency with wearing surface (Murray 2005)
* all % changes between models are with respect to Model 1

Figure 157 – Mode Shape for Shenley Bridge Model 5

\[ f = \frac{1}{2\pi} \sqrt{\frac{k}{m}} \]

Eq. 38
6.2.2 – Parametric Investigation of SPS Natural Frequencies

The lone SPS bridge in-service, the Shenley Bridge, presents a challenge for investigating dynamic characteristics. To compensate for this shortcoming a limited parametric investigation was initiated to aid in the evaluation of the dynamic characteristics of SPS bridges. The parametric study was performed in conjunction with the parametric investigation for lateral load distribution behavior. This allowed for the consideration of the influence of span length, girder spacing, and deck geometry on the vibration characteristics of SPS bridges. These variations translate into the consideration of various mass and stiffness contributions on vibration characteristics. A summary of the parameters and ranges is presented in 5.5 – Parametric Investigation of Lateral Load Distribution of SPS Bridges. The objective was to determine if variations in the bridge design resulted in bridges with fundamental frequencies significantly different than those of conventional designs.

6.2.2.1 – Parametric Model Assumptions

The models assumed that the majority of the stiffness resulted from the longitudinal girders, deck, and lateral bracing with the remaining elements primarily contributing mass. These were the same considerations used in the validation of the “expanded” model for the Shenley Bridge [6.2.1 – Validation of Finite Element Models for Modal Analysis]. The assumptions used in the extension of the “expanded” model are listed below.

1. The expanded model was used
   - Model included deck, longitudinal girders, and lateral bracing
   - Steel \( \rho_{mass} = 7.40 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \); Polymer Core \( \rho_{mass} = 1.44 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \)
2. Longitudinal cold-formed angles were assumed distributed along the deck at the location of the connection (interior and edge)
   - ½ in. thick equal leg cold-formed angles \( \rho_{mass} = 7.40 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \)
3. Guardrail were assumed to be distributed mass along the edges of the deck
   - Area = 16 in\(^2\) \( \rho_{mass} = 7.40 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \)
4. Additional masses were assumed uniformly distributed over the deck area
   - 2 in. thick wearing surface \( \rho_{mass} = 2.26 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \)
   - 2 ea. ½ in. thick equal leg cold-formed angles between panels (8 ft panel widths) \( \rho_{mass} = 7.40 \times 10^{-7} \text{kip-s}^2/\text{in}^4 \)
6.2.2.2 – Parametric Results

For each of the parametric bridges investigated, the resulting output included the first six modal frequencies and mode shapes, as well as the total mass of the structure. This allowed for visual inspection of the mode shapes to ensure that the mode corresponding to the first flexural frequency was evaluated. For all of the models considered, the first mode corresponded to the first flexural mode as illustrated in Figure 157.

A comparison of the first flexural frequencies vs. span length for each of the models is illustrated in Figure 158; the parameters and results are further summarized in Table 53 [pg. 302]. To provide a frame of reference the models proposed by Cantieni (1984), Tilly (1986) and Chan and O’Connor (1990) are also presented. Based on these comparisons it appears that vibration response of SPS does not significantly differ from that of conventional systems.

![Figure 158 – Natural Frequency of SPS Parametric Bridges vs. Span Length](image)

To validate this assessment, the concrete models developed for the comparison of distribution factors [5.5.3.4 – Comparison to Equivalent Design Reinforced Concrete Model] were analyzed for vibration response. These models utilized the same design as the SPS bridges, but utilized an 8 in. concrete deck in lieu of SPS. This change results in an increase in total mass
for all of the cases considered because of the increased deck mass [Figure 159]. A comparison of the frequency vs. span for the two deck types validates this expectation [Figure 160], as the increase mass of the concrete decks result in a decrease in frequency for all spans. This decrease is less substantial for longer spans as the influence of girder mass dominates the response. It should be noted that the prediction equations were developed based on regression analyses of measured data with significant scatter (Taly 1998). When considering the comparisons between the reinforced concrete and the SPS bridges, the SPS bridges are consistently in better agreement with the prediction models indicating that these bridges are representative of the “average” bridges considered in the development of the equations.

![Span Length-Bridge Mass Relationship](image)

**Figure 159 – Span Length vs. Mass Comparison of SPS and R/C Bridge Models**
This scatter in measured frequency is further illustrated in a comparison of the SPS response to the reported results of a number of researchers [Figure 161]. The population includes a variety of slab-girder bridge types, but the vast majority consists of reinforced concrete decks supported by either steel or concrete girders. A greater degree of variation is evident for spans less than about 80 ft. This is of significance because the first natural frequency of typical commercial trucks occurs in the range of 2-5 Hz (Paultre et al. 1992), which based on the span to frequency relationships [Table 50] could begin to impact bridges at this span length and greater. For the shorter spans, the bridge-vehicle interaction response would not be expected to be significant because the natural frequencies are not overlapping.
6.3 – Summary of the Dynamic Characteristics of SPS Bridges

This chapter presents the results of dynamic portion of the live load testing of the Shenley Bridge which included the determination of the dynamic load allowance. For one of the load cases the DLA was greater than the AASHTO code predictions, suggesting that the lightweight deck might have some influence on the dynamic characteristic of these types of bridges.

In the absence of additional bridges for comparison, an investigation of the vibration response of SPS bridges was performed using the finite element models developed for predicted global response. These models were first validated with measured results from vibration testing of the Shenley Bridge and then extended in a parametric investigation of SPS bridges. While the results of the investigation could not be directly correlated to dynamic load allowance, they did indicate that the vibration response of SPS bridges is not significantly different than that of conventional systems.
The results presented in this section can be considered a limited investigation of the dynamic characteristics of SPS because the investigations primarily focused on bridge behavior without significant consideration of the bridge-vehicle interaction. While the scope of the investigation is somewhat limited, the results of this investigation suggest that the dynamic behavior of SPS does not significantly vary from that of bridges constructed with more conventional materials. This is likely the result of the mass and stiffness of the girders controlling the dynamic response more so than the deck. These findings would suggest that the current provisions for dynamic load allowance within AASHTO LRFD can be applied to SPS bridges. A further investigation of the bridge-vehicle interaction response would provide additional insight into this finding, but is beyond the scope of this research.
Chapter 7 – Conclusions and Recommendations

7.1 – Project Review and Summary

With SPS being relatively new to the bridge market there have been only a limited number of applications. As a result of these limited applications there is no standard design procedure for SPS bridge deck systems. The few designs that have been done used detailed finite element models. This research effort focused on evaluating the global behavior of SPS bridges with a focus on critical design parameters such as lateral load distribution, dynamic load allowance, and deck design procedures. This chapter summarizes the findings related to these global characteristics and highlights the need for additional research.

7.2 – Deck Design Procedure

The deck design procedure developed in this research was based on the analysis of SPS bridge decks as a plate spanning between girders. This approach was first validated by comparison to experimental data from both field and laboratory testing and then extended to consider the design limit states within the AASHTO LRFD including serviceability, strength, and fatigue. Due to the uncertainty in the true nature of the boundary conditions, the analysis was performed using a bounded approach that considered the upper and lower limits on the support conditions. Based on this analysis approach the following conclusions can be made:

- Considering the AASHTO LRFD limit states, the design of SPS deck is controlled by stiffness requirements. The flexural rigidity can be related directly to the girder spacing to satisfy the deflection limits [Eq. 29].
- The stress criteria for the serviceability limit state should not control under service loading conditions (including typical dead load and service live loads).
- The stress criteria for the strength limit state should not control under factored loading conditions (including typical dead and factored live loads).
- The stress range due to the AASHTO fatigue design loading was shown not to control for steel face plate thicknesses greater than 1/8 in.. It is recommended that face plate
thicknesses greater than or equal to 1/8 in. be selected to prevent fatigue from controlling the design.

7.3 – Lateral Load Distribution Behavior of SPS

The lateral load distribution characteristics of SPS were evaluated using a combination of field testing, finite element modeling, and classical plate analysis. Based on the analyses performed it was determined that a minimal number of member contributions were necessary for predicting the global response of SPS bridges. Therefore, these models included deck, girders, and lateral bracing elements. A parametric investigation of lateral load distribution along with the field results allowed for the following conclusions to be drawn.

• The lateral load distribution characteristics of SPS bridges are not significantly different from that of reinforced concrete deck bridges:
  1. The distribution factors for an equivalent concrete deck were lower than those of SPS bridges, but not to a significant degree.
  2. The difference between a concrete deck bridge and SPS bridges is the deck stiffness which was shown to be of minimal significance on the lateral load distribution.
• The lateral load distribution behavior of SPS can be accounted for by using the current AASHTO LRFD provisions for concrete decks supported by steel girders.
  1. These provisions were conservative for all of the cases considered.
• The lateral load distribution behavior of SPS can also be accounted for by using the recommendations of the NCHRP 12-62.
  1. These provisions are currently under review as a replacement to the existing provisions in the AASHTO LRFD (status TBD).
• The lever rule can be used to determine the distribution factors for SPS bridges and for all cases yields conservative results. Also, the lever rule is more conservative than the distribution factors in the AASHTO LRFD.
7.4 – Dynamic Load Allowance of SPS

Due to the lightweight of SPS it was believed that the dynamic characteristics would differ significantly from those of conventional systems. These dynamic characteristics were evaluated through field testing of a SPS bridge and modeling of the vibration characteristics of SPS bridges. The following conclusion can be made based on the investigation:

- The field testing of the Shenley Bridge demonstrated that the measured dynamic load allowance values were reasonably accommodated by the AASHTO LRFD provisions, with the exception of one load case.
  - The exact cause of the divergence for this one load case could not be determined, but it is believed that cause was related to approach settlement, roadway roughness and load positioning during the testing.
- The finite element models developed for evaluating global response were capable of characterizing the vibration response of SPS bridges, but required additional elements to represent the mass contributions of the longitudinal and transverse cold-formed angles, guardrails, edge plates, and wearing surface
- A parametric investigation indicated that the vibration response of SPS does not differ significantly from slab-girder bridges with similar spans.
  - This agreement can be attributed to the strong influence of girder mass and stiffness on vibration response.
  - The largest divergence was observed for the shorter spans, where the likelihood of significant bridge-vehicle interaction is low due to the high bridge frequencies.
- Current provisions for dynamic load allowance with AASHTO LRFD can be extended to SPS bridges.
  - These provisions utilize a blanket approach to dynamic amplification with no consideration of deck type. The minimal difference in vibration characteristics between SPS and conventional decks further validate this assessment.

7.5 – Design Recommendations for SPS Bridge

When considering the design of SPS bridges versus that of a conventional bridge such as a steel girder bridge with a reinforced concrete deck, the key differences are primarily in the
design of the deck. A general procedure for the design of a SPS bridge superstructure is presented below. The procedure does not consider all aspects of the bridge design, but rather those that were considered in this research.

1. For the desired girder spacing, determine the flexural rigidity required to satisfy the AASHTO LRFD deflection limit per Eq. 29 [pg. 133].

2. Select the face plate thickness based on economy and/or availability and determine the core thickness necessary to yield the required flexural rigidity from Step 1.
   a. Designer must select a face plate thickness of 1/8 in. or greater to ensure that the fatigue limit state does not control the design of the base metal. For plate plate thicknesses less than 1/8 in. a refined analysis may be required.
   b. The core thickness required is determined from the solution of Eq. 31 [pg. 145] or the use of the Flexural Rigidity Design Tool.xls worksheet.

3. Determine the design loads utilizing the beam line method of analysis with the applicable lateral load distribution factors and dynamic load allowance. Design the girders using the conventional methods within the AASHTO LRFD specification.
   a. Calculate the distribution factors for SPS using the equations for a concrete deck supported by steel girders. Utilize the properties of the SPS deck as determined by Eq. 27 and Eq. 28 [pg. 122] in the calculation of the distribution factors.
   b. Use a dynamic load allowance of 33% for the amplification of the beam line loads.

7.6 – Recommendations for Additional Research

The SPS technology appears to be a viable alternative for short to medium span bridges, but a number of aspects related to this technology warrant further investigation. Based on the analysis performed as part of this research the following observations and recommendations are made related to the implementation of SPS in bridges.

7.6.1 – Additional Experimental Validation of Research Findings

The procedures developed from the analytical investigations were shown to yield conservative designs, but additional field investigations of the lateral load distribution
characteristics, dynamic load allowance, vibration characteristics, and deck response of in-service SPS bridges would provide further validation of the analyses and allow for additional refinement. Additional live load testing would allow for the consideration of additional parameters, specific to SPS bridges, which may influence lateral load distribution and dynamic characteristics. The testing would also allow for an assessment of the influence of temperature, roadway surface profile, support conditions, vehicle speed, and vehicle characteristics on the dynamic behavior of SPS bridges. It would also provide additional validation of the deck response when subjected to static and dynamic truck loads.

In addition to the field testing, further laboratory investigations of the shear capacity of SPS is warranted. Measurement of shear strength of SPS cross-sections would allow for the determination of the boundary for a shear-critical configuration, ensuring that shear failures will not control the design.

7.6.2 – Supplementary Recommendations

The following recommendations do not directly relate to this research, but may warrant consideration in future research investigations.

- Evaluation of the bond characteristics of the steel-polymer interface.
  - Investigate the effects of thermal ratcheting from temperature induced residual stresses under typical highway bridge temperatures.
  - Determine the influence of reduced bond on local and global behavior.
- Evaluate the design approach to determine the optimal configuration for panel-to-panel and panel-to-girder connections for typical bridge applications. Proper fatigue design of these connections is essential to ensure a reasonable service life of a SPS bridge structure.
  - Determine the fatigue resistance of the connections considered for design.
  - Investigate optimal panel-to-panel connection location to minimize fatigue issues.
Summary of Definitions

AASHTO – American Association of State and Highway Transportation Officials
CHBDC – Canadian Highway Bridge Design Code
DF – lateral load distribution factor
DLA – dynamic load allowance
FEM – Finite Element Method
IM – impact or dynamic load allowance
LRFD – Load and Resistance Factored Design
OHBDC – Ontario Highway Bridge Design Code
SPS – Sandwich Plate System
A – beam cross sectional area
A – detail category constant (AASHTO LRFD Fatigue)
$A_b$ – cross sectional area of girder
$A_d$ – cross sectional area of deck
$A_x$ – shear area of longitudinal beam
$A_y$ – shear area of transverse beam
D – constant related to bridge type (AASHTO Standard Specification)
D – flexural rigidity of a plate
$D'$ – flexural rigidity of a plate $= D_t [1 - \nu_{eq}^2]$ (Zenkert 1995)
$D_t$ – equivalent flexural rigidity of a layered plate
$D_x$ – longitudinal flexural rigidity of a plate
$D_y$ – transverse flexural rigidity of a plate
$D_{xy}$ – torsional rigidity of a plate
E – modulus of elasticity
$E_b$ - modulus of elasticity of beam (in direction considered)
$E_c$ – modulus of elasticity of core material
$E_p$ – modulus of elasticity of plate material
$E_{R/C}$ – modulus of elasticity of reinforced concrete deck
$E_{SPS}$ – equivalent modulus of elasticity of SPS deck
Exp1, Exp2, Exp3, D – Bridge type constants (alternative DF) for NCHRP Project 12-62
$F_m$ – equal distribution modification factor for moment (CHBDC)
$F_v$ – equal distribution modification factor for shear (CHBDC)

$G$ – shear modulus of beam (in direction considered)

$I$ – moment of inertia

$\bar{I}_b$ – moment of inertia of girder about neutral axis of girder

$\bar{I}_d$ – moment of inertia of deck about neutral axis of deck

$I_x$ – moment of inertia of longitudinal beam about primary bending axis

$I_y$ – moment of inertia of transverse beam about primary bending axis

$J_x$ – polar moment of inertia of longitudinal beam

$J_y$ – polar moment of inertia of transverse beam

$K_g$ – longitudinal stiffness parameter

$[K]$ – structure stiffness matrix

$L$ – span length (ft)

$L$ – span length of deck [Equivalent strip width] (ft)

$M_{fb}$ – applied moment due to factored loads in transverse beam

$M_{ft}$ – applied transverse moment in the deck plate due to the factored loads as a result of the plate carrying wheel loads to adjacent longitudinal ribs

$M_t$ – maximum moment (CHBDC)

$M_{rh}$ – factored moment resistance of transverse beam

$M_{rt}$ – factored moment resistance of deck plate in carrying wheel loads to adjacent ribs

$M_{rr}$ – flexural resistance of longitudinal rib

$M_{lu}$ – local flexural moment of longitudinal rib due to factored loads

$M^+$ = positive moment region of slab

$M^-$ = negative moment region of slab

$[M]$ – structure mass matrix

$N$ – number of girders in bridge

$N$ – number of stress-range cycles (AASHTO LRFD Fatigue)

$N_b$ – number of beams in bridge

$N_g$ – number of girders in bridge for NCHRP Project 12-62

$N_l$ – maximum number of design lanes (assumed lane width of 10ft) for NCHRP Project 12-62

$N_{lanes}$ – number of design lanes considered in the analysis for NCHRP Project 12-62

$N^T$ – transpose of element shape function
N\text{trucks} – number of trucks on bridge for given loading

P = axle load (kip)

P_{\text{patch}} = \text{Force distributed over patch area (kip)}

P_r – nominal tensile resistance of deck

P_u – factored resistance of decks subjected to global tension

Q_i – force effect

R(x,y) – correlation coefficient

R_{\text{dynamic}} – strain under static loading

R_e – equivalent nodal load

R_L – multiple presence factor

R_n – nominal resistance

R_{\text{static}} – deflection under static loading

S = transverse girder spacing (ft)

S_b = spacing of grid bars (in.)

S_{eq} = \text{Shear Stiffness (kip/in)} = G \cdot \frac{t_s^2}{t_c}

TBX – longitudinal beam(s) strain energy

TBY – transverse beam(s) strain energy

TP – equivalent work done by applied loading

TS – plate strain energy

U_L – static live load effect

U_{L+1} – live load effect including dynamic amplification

U_T – energy of stiffened plate system

V_t – maximum shear (CHBDC)

W_c – curb-to-curb distance (ft) for NCHRP Project 12-62

X = distance from load to point of support (ft)

a – longitudinal dimension of plate (span)

a_m – calibration constant for NCHRP Project 12-62

b – transverse dimension of plate (width)

b_f – flange width

b_m – calibration constant for NCHRP Project 12-62

c – half of tire patch in longitudinal direction (in.)
$d$ – half of tire patch in transverse direction (in.)

$d_b$ – distance from centroid of girder to plastic neutral axis of composite section

$d_d$ – distance from centroid of deck to plastic neutral axis of composite section

$d_e$ – distance from exterior web of exterior beam to the interior edge of curb or traffic barrier

$d_{plate}$ – distance between midplane of face plates

$f_i$ – $i$th natural frequency (cycles per unit time)

$h$ – height of deck (in.)

$h_c$ – thickness of core

$k$ – stiffness

$m$ – multiple presence factor

$n$ – number of design lanes

$p(x,y)$ – loading function

$p_i$ – concentrated load at arbitrary location

$t$ – distributed load

$t_c$ – thickness of core material

$t_f$ – flange thickness

$t_d$ – thickness of deck

$t_p$ – thickness of plate material (total thickness)

$t_w$ – web thickness

$w(x,y)$ or $w$ – out of plane deflected shape

$w_h(x,y)$ – out of plane deflected shape (homogenous solution)

$w_p(x,y)$ – out of plane deflected shape (particular solution)

$x$ – direction axis along x-axis

$x_1$ – longitudinal distance to center of tire patch along x-axis (in.)

$x_k$ – distance from end of plate to transverse beam location

$y$ – direction axis along y-axis

$y_1$ – transverse distance to center of tire patch along y-axis (in.)

$y_k$ – distance from plate edge to longitudinal beam location

$z$ – distance from midplane surface of deck to location of interest (in.)

$z_x$ – distance from midplane surface of deck to extreme face (in.)

$\Delta_{dynamic}$ – deflection under dynamic loading
$\Delta F_n$ – nominal fatigue resistance (ksi)

$\Delta F_{TH}$ – constant amplitude fatigue threshold stress (ksi)

$\Delta_{\text{max}}$ – maximum deflection in a girder

$\Delta_{\text{static}}$ – deflection under static loading

$\phi$ – strength reduction factor

$\{\phi\}_i$ – eigenvector representing the mode shape of the $i$th natural frequency

$\gamma_l$ – load factor

$\gamma_s$ – live load distribution simplification factor (DSF)

$\varepsilon_{\text{dynamic}}$ – strain under dynamic loading

$\varepsilon_{\text{max}}$ – maximum strain in a girder

$\varepsilon_{\text{static}}$ – strain under static loading

$\varepsilon_y$ – yield strain

$\zeta$ – longitudinal position of center of distributed load from end of plate ($x = 0$)

$\eta$ – transverse position of center of distributed load from edge of plate ($y = 0$)

$\eta_i$ – load modifier relating to ductility, redundancy, and operational importance

$\mu$ – length of distributed loading (longitudinal direction)

$\nu$ – width of distributed loading (transverse direction)

$\nu_c$ – Poisson’s ratio for core material

$\nu_{eq}$ – Poisson’s ratio for equivalent sandwich plate

$\nu_p$ – Poisson’s ratio for plate material

$\sigma_p$ – strength at proportional limit stress

$\sigma_y$ – yield stress

$\sigma_{10\%\varepsilon}$ – stress at 10% strain

$\varphi_1$ – empirical deck overhang coefficient

$\varphi_2$ – empirical span coefficient

$\omega_i$ – $i$th natural circular frequency (radians per unit time)
References


241


Appendix A – Additional Results

A.1 – Shenley Bridge Test (November 2003)

Figure 162 – Load Case 4 Configuration for Shenley Bridge Field Test – November 2003

Figure 163 - Comparison of Girder A Deflection for Load Case 4
Figure 164 - Comparison of Girder C Deflection for Load Case 4

Figure 165 – Load Case 5 Configuration for Shenley Bridge Field Test – November 2003
Figure 166 - Comparison of Girder A Deflection for Load Case 5

Figure 167 - Comparison of Girder C Deflection for Load Case 5
Appendix B – Additional Deck Analysis Results

B.1 – Additional Half-Scale Test Results

Figure 168– Load vs. Panel Edge Strain (SG 6-9) Locations – Half-Scale Bridge (Test 1)

Figure 169– Load vs. Panel Edge Strain (SG 6-9) Locations – Half-Scale Bridge (Test 2)
B.2 – Service II Results

Figure 170 – Stress Comparison for Service II Limit State (Simple Support) All Load Cases

Figure 171 – Stress Comparison for Service II Limit State (Fixed Support) All Load Cases
Figure 172 – Plate Stress for Service II Limit State (Simple Support) All Load Cases

Figure 173 – Plate Stress for Service II Limit State (Fixed Support) All Load Cases
B.3 – Strength I Results

**Figure 174**– Stress Comparison for Strength I Limit State (Simple Support) All Load Cases

**Figure 175**– Stress Comparison for Strength I Limit State (Fixed Support) All Load Cases
**Figure 176**– Plate Stress for Strength I Limit State (Simple Support) All Load Cases

**Figure 177**– Plate Stress for Strength I Limit State (Fixed Support) All Load Cases
Figure 178 – Transverse Shear Stress Comparison for Strength I Limit State (Simple Support) All Load Cases

Figure 179 – Transverse Shear Stress Comparison for Strength I Limit State (Fixed Support) All Load Cases
B.4 – Classical Plate Analysis Worksheet

**Given**

USER Input

**Plate Dimensions**

<table>
<thead>
<tr>
<th>b(\text{:=}) 5ft</th>
<th>girder spacing (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a(\text{:=}) 3-b</td>
<td>Tire Load</td>
</tr>
<tr>
<td>P(\text{:=}) 16kip</td>
<td>Patch Loading Area (2c x 2d)</td>
</tr>
<tr>
<td>c(\text{:=}) 20/2</td>
<td></td>
</tr>
<tr>
<td>d(\text{:=}) 10/2</td>
<td></td>
</tr>
<tr>
<td>x(_1) := a/2</td>
<td></td>
</tr>
<tr>
<td>y(_1) := b/2</td>
<td></td>
</tr>
</tbody>
</table>

**Loading Data**

---

**SPS Deck Properties**

\[ E_{\text{plate}} := 29877.77 \text{ksi} \quad \nu_{\text{plates}} := .287 \quad t_{\text{plates}} := .375 \text{in} \]

\[ E_{\text{core}} := 108.78 \text{ksi} \quad \nu_{\text{core}} := .36 \quad t_{\text{core}} := 1.2356 \text{in} \]

\[ \frac{E_{\text{plate}}}{2(1 + \nu_{\text{plates}})} \]

\[ \frac{E_{\text{core}}}{2(1 + \nu_{\text{core}})} \]

\[ d_{\text{plate}} := t_{\text{core}} + t_{\text{plates}} \]

\[ z_x := \frac{t_{\text{core}}}{2} + t_{\text{plates}} \]

\[ \nu_{\text{eq}} = 0.287089 \]

\[ \nu_{\text{eq}} = 0.287089 \]

\[ t_{\text{plates}}^2 + t_{\text{core}}^3 \]

\[ \frac{E_{\text{eq}}}{2(1 + \nu_{\text{eq}})} \]

\[ S_{\text{eq}} := \frac{G_{\text{eq}}}{t_{\text{core}}} \]

---

**Note:** In text by Zenkert the calculations for Flexural Rigidity to not incorporated \(1/(1-n^2)\) [pg 137]. This term is included within the differential equations separately. For the below calculation the following substitution will be used for clarity.

\[ D := \frac{1}{1-\nu_{\text{eq}}^2} \]

\[ D = 14812 \text{kip in} \]

\[ D_{\text{xy}} := \frac{D}{1 + \nu_{\text{eq}}} \]

\[ D_{\text{xy}} = 11508 \text{kip-in} \]

---

**ScaleFactor**

\[ D_{\text{xy}} := 1 \]

---

---
\[
\begin{align*}
\text{Cont'd} \\
\ m &= 1..50 \
\ n &= 1..50 \\
\text{wbending}(x,y) &= -1 \sum_m \sum_n \left[ \left( \frac{4P}{\pi^2 \cdot m \cdot n \cdot c \cdot d} \right) \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \sin \left( \frac{m\pi \cdot c}{a} \right) \sin \left( \frac{n\pi \cdot d}{b} \right) \left( 1 - \nu_{eq}^2 \right) \right] \left( \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \right) \\
\text{wbending} \left( \frac{a}{2}, \frac{b}{2} \right) &= -0.056578 \ln \\
\text{wshear}(x,y) &= -1 \sum_m \sum_n \left[ \left( \frac{4P}{\pi^2 \cdot m \cdot n \cdot c \cdot d} \right) \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \sin \left( \frac{m\pi \cdot c}{a} \right) \sin \left( \frac{n\pi \cdot d}{b} \right) \right] \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \\
\text{wshear} \left( \frac{a}{2}, \frac{b}{2} \right) &= -0.000274 \ln \\
\text{wpatch}(x,y) &= -1 \sum_m \sum_n \left[ \left( \frac{4P}{\pi^2 \cdot m \cdot n \cdot c \cdot d} \right) \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \sin \left( \frac{m\pi \cdot c}{a} \right) \sin \left( \frac{n\pi \cdot d}{b} \right) \left( 1 - \nu_{eq}^2 \right) \right] \left( \sin \left( \frac{m\pi \cdot x}{a} \right) \sin \left( \frac{n\pi \cdot y}{b} \right) \right) \\
\text{wpatch} \left( \frac{a}{2}, \frac{b}{2} \right) &= -0.056853 \ln
\end{align*}
\]
Appendix C – Additional Lateral Load Distribution Results

C.1 – Shenley Bridge Field Test Results - Static Tests

Figure 180 – Static Displacement Response- Girder A (Load Case A)

Figure 181 – Static Displacement Response- Girder B (Load Case A)
Figure 182 – Static Displacement Response - Girder C (Load Case A)

Figure 183 – Static Strain Response - Girder A (Load Case A)
Figure 184 – Static Strain Response- Girder B (Load Case A)

Figure 185 – Static Strain Response- Girder C (Load Case A)
Figure 186 – Static Displacement Response- Girder A (Load Case B)

Figure 187 – Static Displacement Response- Girder B (Load Case B)
Figure 188 – Static Displacement Response- Girder C (Load Case B)

Figure 189 – Static Strain Response- Girder A (Load Case B)
Figure 190 – Static Strain Response - Girder B (Load Case B)

Figure 191 – Static Strain Response - Girder C (Load Case B)
Figure 192 – Static Displacement Response - Girder A (Load Case C)

Figure 193 – Static Displacement Response - Girder B (Load Case C)
Figure 194 – Static Displacement Response- Girder C (Load Case C)

Figure 195 – Static Strain Response- Girder A (Load Case C)
Figure 196 – Static Strain Response- Girder B (Load Case C)

Figure 197 – Static Strain Response- Girder C (Load Case C)
Figure 198 – Static Displacement Response- Girder A (Load Case E)

Figure 199 – Static Displacement Response- Girder B (Load Case E)
Figure 200 – Static Displacement Response- Girder C (Load Case E)

Figure 201 – Static Strain Response- Girder A (Load Case E)
Figure 202 – Static Strain Response- Girder B (Load Case E)

Figure 203 – Static Strain Response- Girder C (Load Case E)
C.2 – Shenley Bridge Field Test Results - Dynamic Tests

Figure 204 – Dynamic Displacement Response – Girder A (Load Case B)

Figure 205 – Dynamic Displacement Response – Girder B (Load Case B)
Figure 206 – Dynamic Displacement Response – Girder C (Load Case B)

Figure 207 – Dynamic Displacement Response – Girder A (Load Case C)
Figure 208 – Dynamic Displacement Response – Girder B (Load Case C)

Figure 209 – Dynamic Displacement Response – Girder C (Load Case C)
Figure 210 – Dynamic Displacement Response – Girder A (Load Case E)

Figure 211 – Dynamic Displacement Response – Girder B (Load Case E)
Figure 212 – Dynamic Displacement Response – Girder C (Load Case E)
C.3 – Bridge Design Worksheet

GEOMETRIC PARAMETERS

General Parameters:

\[ N_G := 4 \]
\[ N_L := 2 \]
\[ L_{\text{span}} := 80 \text{ ft} \]
\[ w_{\text{oh}} := 2.4 \text{ ft} \]
\[ w_{\text{shdr\_lf}} := 0.0 \text{ ft} \]
\[ w_{\text{shdr\_rt}} := 0.0 \text{ ft} \]
\[ w_p := 0.0 \text{ ft} \]
\[ w_{\text{lane}} := 12 \text{ ft} \]

\[ w_{\text{bridge}} := w_{\text{lane}} N_L + w_{\text{shdr\_lf}} + w_{\text{shdr\_rt}} + w_p^2 + w_{\text{oh}}^2 \]

\[ w_{\text{roadway}} := w_{\text{lane}} N_L + w_{\text{shdr\_lf}} + w_{\text{shdr\_rt}} \]

\[ S := \frac{(w_{\text{bridge}} - 2 \cdot w_{\text{oh}})}{(N_G - 1)} \]  

Girder Spacing

\[ w_{\text{bridge}} = 28.8 \text{ ft} \]

\[ w_{\text{roadway}} = 24 \text{ ft} \]

\[ S = 8 \text{ ft} \]
MATERIAL PROPERTIES

Deck Properties:

\[ E_{\text{top\_pl}} := 29000 \text{ ksi} \]

Top plate Young's Modulus

\[ \nu_{\text{top\_pl}} := 0.28 \]

Poisson's ratio for top plate

\[ \gamma_{\text{top\_pl}} := 490 \frac{\text{lbf}}{\text{ft}^3} \]

Specific gravity of top plate

\[ E_{\text{core}} := 108.78 \text{ ksi} \]

Polyurethane core Young's Modulus

\[ \nu_{\text{core}} := 0.36 \]

Poisson's ratio for polyurethane

\[ \gamma_{\text{core}} := 71.75 \frac{\text{lbf}}{\text{ft}^3} \]

Specific gravity of core

\[ E_{\text{bot\_pl}} := E_{\text{top\_pl}} \]

Bottom plate Young's Modulus

\[ \nu_{\text{bot\_pl}} := \nu_{\text{top\_pl}} \]

Poisson's ratio for bottom plate

\[ \gamma_{\text{bot\_pl}} := \gamma_{\text{top\_pl}} \]

Specific gravity of bottom plate

\[ F_{\text{y\_plate}} := 50 \text{ ksi} \]

Yield strength of top plate

Girder Properties:

\[ E_{\text{girder}} := 29000 \text{ ksi} \]

Young's Modulus of girder

\[ \nu_{\text{girder}} := 0.28 \]

Poisson's ratio of girder

\[ \gamma_{\text{girder}} := 490 \frac{\text{lbf}}{\text{ft}^3} \]

Specific gravity of girder

\[ F_{\text{y\_girder}} := 50 \text{ ksi} \]

Yield strength of girder
## Preliminary Member Sizing

### Preliminary Flexural Considerations

**Cross-Sectional Proportion Limits**

### Web Proportions

\[
\frac{D}{t_w} \leq 15 \quad \text{web slenderness limit}
\]

### Flange Proportions

\[
\frac{b_f}{2t_f} \geq 12.6 \quad \text{compactness limit}
\]

\[
b_f \geq \frac{D}{6}
\]

\[
t_f \geq 1.1t_w
\]

\[
0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10
\]

### Plate Thicknesses:

- \(t_{\text{top\_pl}} := 0.1875\text{ in}\)
- \(t_{\text{core}} := 3.57\text{ in}\)
- \(t_{\text{bot\_pl}} := t_{\text{top\_pl}}\)

\[t_{\text{deck\_total}} := t_{\text{top\_pl}} + t_{\text{core}} + t_{\text{bot\_pl}}\]

\(\text{offset\_plate} := 7.95\text{ in}\)

### General Considerations for Limit States

**Ductility** \(\eta_D := 1.0\)

**Redundancy** \(\eta_R := 1.0\)

**Operational Importance** \(\eta_I := 1.0\)

For loads which a maximum value of \(\gamma_i\) is appropriate:

\[
\eta = \eta_D \cdot \eta_R \cdot \eta_I \geq 0.95
\]

For loads which a minimum value of \(\gamma_i\) is appropriate

\[
\eta = \frac{1}{\eta_D \cdot \eta_R \cdot \eta_I} \leq 1.0
\]

\[
\eta := \max \left( \frac{1}{\eta_D \cdot \eta_R \cdot \eta_I} \leq 1.0, \eta_D \cdot \eta_R \cdot \eta_I \geq 0.95 \right)
\]

\(\eta = 1\)

### Trial Depth Selection:

Suggested minimum span-to-depth ratio for a simple span design

\[
D := 0.033L_{\text{span}} \quad D = 31.68\text{ in} \quad \text{depth of girder portion of composite section}
\]

\[
D_{\text{total}} := 0.04L_{\text{span}} \quad D_{\text{total}} = 38.4\text{ in} \quad \text{depth of composite girder cross section (including deck)}
\]

Suggested optimal depth for the hybrid configuration chosen: (Horton, 2002)

\[
D_{\text{opt}} := \frac{L_{\text{span}}}{30} \quad D_{\text{opt}} = 32\text{ in}
\]
**Equivalent Plate Properties (layered plate)**

\[ z_1 := \frac{t_{\text{top\_pl}} + t_{\text{core}}}{2} \]

\[ D_{\text{deck}} := \frac{2}{3} \left[ \frac{E_{\text{top\_pl}} \left( \frac{t_{\text{core}}}{2} + t_{\text{top\_pl}} \right)^3}{\left( 1 - \nu_{\text{top\_pl}} \right)^2} - \left( \frac{t_{\text{core}}}{2} \right)^3 \right] + \frac{E_{\text{core}} \left( \frac{t_{\text{core}}}{2} \right)^3}{\left( 1 - \nu_{\text{core}} \right)^2} \]

\[ E_{\text{deck\_eq}} := \frac{D_{\text{deck}} \cdot 12 \left( 1 - \nu_{\text{top\_pl}} \right)^2}{t_{\text{deck\_total}}^3} \]

Equivalent flexural rigidity of layered plate (Plantema)

\[ D_{\text{deck}} = 4.234 \times 10^4 \text{ kip in} \]

Equivalent Young's modulus of layered plate

\[ E_{\text{deck\_eq}} = 7.594 \times 10^3 \text{ ksi} \]

**Girder Flange:**

Considerations:
- Use a minimum of 11.8 in. (300 mm) [based on allowing for room to accommodate shear studs in R/C deck] - Assume similar for SPS to accommodate connections
- Provide at least 3/4 in. (19 mm) in thickness to avoid weld distortion
- Do not exceed 1.97 in. for Thermo Mechanical Controlled Processing (TMCP)

**Girder Web:**

Considerations:
- Consider unstiffened webs for span lengths less than 115 ft (35 m) - will use up to 120 ft
- Consider unstiffened webs if the web depth is less than 51 in. (1295 mm)
- For webs depths between 51-71 in. (1295-1803 mm) subtract 0.08-.12 in. (2-3 mm) from the unstiffened web design and provide stiffeners as required
- Provide the thinnest web possible for webs greater than 71 in. in depth

**SUMMARY OF TRIAL GIRDER DIMENSIONS (Input)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top flange width</td>
<td>16 in</td>
</tr>
<tr>
<td>Top flange thickness</td>
<td>1.2 in</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>1.2 in (b_f.b.)</td>
</tr>
<tr>
<td>Bottom flange thickness</td>
<td>1.2 in (t_f.b.)</td>
</tr>
<tr>
<td>Web depth</td>
<td>41 in (D_w)</td>
</tr>
<tr>
<td>Web thickness</td>
<td>.6 in (t_w)</td>
</tr>
<tr>
<td>Total Girder Depth</td>
<td>43.4 in (D_girder)</td>
</tr>
</tbody>
</table>

\[ D_{\text{opt}} = 32\text{in} \]

\[ D_{\text{w}} = 41\text{in} \]

\[ D_{\text{girder}} = 43.4\text{in} \]
NON COMPOSITE SECTION PROPERTIES

overall depth of section \( d := t_{f.t.} + D_w + t_{f.b.} \)

area of top flange \( A_{f.t.} := b_{f.t.} \cdot t_{f.t.} \)

area of bottom flange \( A_{f.b.} := b_{f.b.} \cdot t_{f.b.} \)

area of web \( A_w := D_w \cdot t_w \)

total girder area \( A_g := A_{f.t.} + A_{f.b.} + A_w \)

girder self-weight \( w_{tg} := (A_{f.t.} + A_{f.b.} + A_w) \cdot \gamma_{girder} \)

distance to centroid of top flange \( d_{f.t.} := t_{f.t.} + D_w + 0.5t_{f.t.} \)

distance to centroid of bottom flange \( d_{f.b.} := 0.5t_{f.b.} \)

distance to centroid of web \( d_w := t_{f.b.} + 0.5D_w \)

\[
d_{\text{Bot Steel NC}} := \frac{A_{f.t.} \cdot d_{f.t.} + A_{f.b.} \cdot d_{f.b.} + A_w \cdot d_w}{A_g}
\]

\[
d_{\text{Top Steel NC}} := d - d_{\text{Bot Steel NC}}
\]

\[
I_{f.t.} := b_{f.t.} \cdot \frac{t_{f.t.}^3}{12} + A_{f.t.} \cdot (d_{f.t.} - d_{\text{Bot Steel NC}})^2
\]

\[
I_{f.b.} := b_{f.b.} \cdot \frac{t_{f.b.}^3}{12} + A_{f.b.} \cdot (d_{\text{Bot Steel NC}} - d_{f.b.})^2
\]

\[
I_w := t_w \cdot \frac{D_w^3}{12} + A_w \cdot (d_w - d_{\text{Bot Steel NC}})^2
\]

\[
I_{NC} := I_{f.t.} + I_{f.b.} + I_w
\]

\[
S_{\text{Top Steel NC}} := \frac{I_{NC}}{d_{\text{Top Steel NC}}}
\]

\[
S_{\text{Bot Steel NC}} := \frac{I_{NC}}{d_{\text{Bot Steel NC}}}
\]
LATERAL LOAD DISTRIBUTION FACTORS

Range of applicability
* Assume SPS bridge deck behaves similar to a reinforced concrete deck supported by steel girders
* Range of applicability will not be enforced (this is the purpose of the assessment)

\[ \begin{align*}
3.5 \text{ ft} & \leq S \leq 16 \text{ ft} \\
20 \text{ ft} & \leq L \leq 240 \text{ ft} \\
10,000 \text{ in}^4 & \leq K_g \leq 7,000,000 \text{ in}^4 \\
4.5 \text{ in} & \leq t_g \leq 12 \text{ in} \\
N_b & \geq 4
\end{align*} \]

\( K_g \) term

\[ e_g := \frac{d_{\text{Top, Steel NC}} + \text{offset plate} + t_{\text{bot, pl}} + t_{\text{core}}}{2} \]

\[ n := \frac{E_{\text{girder}}}{E_{\text{deck, eq}}} \]

\[ K_g := n \left( I_{\text{NC}} + A_g e_g \right)^2 \]

**Interior Beam - Moment**

*One lane loaded*

\[ DFM_1 := 0.06 + \left( \frac{S}{14 \text{ ft}} \right)^{0.4} \left( \frac{S}{L_{\text{span}}} \right)^{0.3} \left( \frac{K_g}{12 \text{ in/ft} \cdot L_{\text{span}} \cdot t_{\text{deck, total}}} \right)^{0.1} \]

DFM1 = 0.534

*Two or more design lanes*

\[ DFM_2 := 0.075 + \left( \frac{S}{9.5 \text{ ft}} \right)^{0.6} \left( \frac{S}{L_{\text{span}}} \right)^{0.2} \left( \frac{K_g}{12 \text{ in/ft} \cdot L_{\text{span}} \cdot t_{\text{deck, total}}} \right)^{0.1} \]

DFM2 = 0.749

**Fatigue**

For single-lane loading to be used for fatigue design, remove the multiple presence factor of 1.20

\[ DFM_{1, \text{fatigue}} := \frac{DFM_1}{1.20} \]

DFM1_fatigue = 0.445

**Interior Beam - Shear**

*One lane loaded*

\[ DFV_1 := 0.36 + \frac{S}{25 \text{ ft}} \]

DFV1 = 0.68

*Two or more design lanes*

\[ DFV_2 := 0.2 + \frac{S}{12 \text{ ft}} - \left( \frac{S}{35 \text{ ft}} \right)^2 \]

DFV2 = 0.814
Exterior Beam - Moment (Diaphragm Effect)

This section is not completely automated and must be modified to account for the number of girders in the bridge (Diaphragm and cross-bracing effects).

Truck width \( w_{truck} := 6.12 \text{ in} \) \( w_{truck} = 72 \text{ in} \)

Spacing between trucks \( s_{truck} := 6.12 \text{ in} \) \( s_{truck} = 72 \text{ in} \)

Number of design lanes \( N_{DL} := \text{floor} \left( \frac{w_{roadway}}{12 \text{ ft}} \right) \) \( N_{DL} = 2 \) make an integer (next lowest)

Spacing between parapet and truck
\( S_{truck \_parapet} := 2.12 \text{ in} \) \( S_{truck \_parapet} = 24 \text{ in} \)

Truck reaction location

Distance from c.g. of each truck pattern to c.g. of girder pattern (center of bridge for symmetric girders) \( j := 1..N_L \)
\[ e_r_j := \frac{w_{roadway}}{2} - S_{truck \_parapet} - \frac{w_{truck}}{2} - \left( w_{truck} + s_{truck} \right) (j - 1) \]
\[ e_{r_1} = 84 \text{ in} \]
\[ e_{r_2} = -60 \text{ in} \]
\[ e_{r_3} = \text{in} \]

For more than three design lanes continue with the sequence

Distance between bridge girders and center of bridge (starting with exterior girder and moving in) \( i := 1..N_G \)
\[ x_1 := \frac{w_{bridge}}{2} - w_{oh} - S \left( i - 1 \right) \]
\[ x_1 = 144 \text{ in} \] distance to girder 1 from bridge CL
\[ x_2 = 48 \text{ in} \] distance to girder 2 from bridge CL
\[ x_3 = -48 \text{ in} \] distance to girder 3 from bridge CL
\[ x_4 = -144 \text{ in} \] distance to girder 4 from bridge CL
\[ x_5 = \text{in} \] distance to girder 5 from bridge CL
\[ x_{ext} = 144 \text{ in} \] distance to exterior girder from CL

For more than 5 girders continue the sequence (\( x_i \) where \( i \) is the number of girders)
**Multiple Presence Factors** \( t := 1..N_{DL} \)

\[
m_t := \begin{cases} 
1.2 & \text{if } t = 1 \\
1.0 & \text{if } t = 2 \\
0.85 & \text{if } t = 3 \\
0.65 & \text{if } t \geq 4 
\end{cases}
\]

**Reaction** \( p := 1..N_L \)

\[
R_p := \frac{p}{N_G} + \sum_{f=1}^{N_G} \frac{e_r f}{\left(\sum_{i=1}^{N_G} (x_i)^2\right)}
\]

**Distribution Factor (included multiple presence factor)**

\[
h := 1..N_{DL}
\]

\[
D_{m_h} := R_h \cdot m_h
\]

**Exterior Beam - Moment (Lever Rule)**

This section is not completely automated and must be modified to account for bridge configuration.

*One Lane Loaded:* \( \text{edge}_\text{dist} := 2\text{ft} \) assumed to be 2ft from edge (rail) AASHTO Sect. 3.6.1.3.1

\[
\text{DFMLever}_1 := \begin{cases} 
\frac{1}{2} \left( \frac{S + w_{oh} - \text{edge}_\text{dist}}{S} \right) & \text{if } \text{edge}_\text{dist} + w_{\text{truck}} > w_{oh} + S \\
\frac{\left[ \frac{1}{2} \left( S + w_{oh} - \text{edge}_\text{dist} \right) + \frac{1}{2} \left( S + w_{oh} - \text{edge}_\text{dist} - w_{\text{truck}} \right) \right]}{S} & \text{otherwise}
\end{cases}
\]

\[
\text{DFMLever}_1 := \text{DFMLever}_1 \cdot m_1 \\
\text{DFMLever}_1 = 0.81
\]

*Two or more lanes loaded:*

\[
d_e := w_{oh} - w_p \\
d_e = 2.4\text{ft}
\]

\[
e := 0.77 + \frac{d_e}{9.1\text{ft}} \\
e = 1.034
\]

\[
\text{DFMLever}_2 := e \cdot \text{DFM}_2 \\
\text{DFMLever}_2 = 0.774
\]
Exterior Beam - Shear

One Lane Loaded:

\[ DF_{E1} := DFMLever_{1} \quad \text{DF}_{E1} = 0.81 \]

Two or more lanes loaded:

\[ e_{1} := 0.6 + \frac{d_{e}}{10 \text{-ft}} \quad e_{1} = 0.84 \]

\[ DF_{E2} := e_{1} \cdot DF_{2} \quad \text{DF}_{E2} = 0.684 \]

Skew Correction Factors

Not applicable to this case, but will include for completeness

Moment Skew Correction Factor:

If \( \theta \) is less than 30\(^{\circ} \)

\[ \theta := 0 \]

If \( \theta \) is greater than 60\(^{\circ} \) use \( \theta = 60\(^{\circ} \)

\[ c_{1} := 0.25 \left( \frac{K_{g}}{12 \cdot L_{\text{span}} \cdot t_{\text{deck total}}} \right)^{0.25} \left( \frac{S}{L_{\text{span}}} \right)^{0.5} \]

Manual input for \( \theta = 0 \)

\[ \text{skew\_corr\_M} := 1 - c_{1} \cdot \tan(\theta)^{1.5} \quad \text{skew\_corr\_M} = 1 \]

Shear Skew Correction Factor:

\[ \text{skew\_corr\_V} := 1.0 + 0.2 \left( \frac{12 \cdot L_{\text{span}} \cdot t_{\text{deck total}}}{K_{g}} \right)^{0.3} \cdot \tan(\theta) \quad \text{skew\_corr\_V} = 1 \]
**Governing Distribution Factors (including skew correction):**

**Interior Max:**

- **Moment:** \( DFM_I := (\text{skew}_-\text{corr}_-\text{M}) \cdot \max(DFM_1, DFM_2) \)  \( DFM_I = 0.749 \)
- **Shear:** \( DFV_I := (\text{skew}_-\text{corr}_-\text{V}) \cdot \max(DFV_1, DFV_2) \)  \( DFV_I = 0.814 \)

**Interior Fatigue:**

- **Moment:** \( DFM_{I,F} := (\text{skew}_-\text{corr}_-\text{M}) \cdot (DFM_{1,\text{fatigue}}) \)  \( DFM_{I,F} = 0.445 \)
- **Shear:** \( DFV_{I,F} := (\text{skew}_-\text{corr}_-\text{V}) \cdot \frac{DFV_1}{m_1} \)  \( DFV_{I,F} = 0.567 \)

**Exterior Max:**

- **Moment:** \( DFM_{E} := (\text{skew}_-\text{corr}_-\text{M}) \cdot \max(D_m, DFMLever_{1}, DFMLever_{2}) \)  \( DFM_{E} = 0.81 \)
- **Shear:** \( DFV_{E} := (\text{skew}_-\text{corr}_-\text{V}) \cdot \max(DFV_{E1}, DFV_{E2}) \)  \( DFV_{E} = 0.81 \)

**Exterior Fatigue:**

- **Moment:** \( DFM_{E,F} := (\text{skew}_-\text{corr}_-\text{M}) \cdot \max \left( \frac{D_m_{1}}{m_1}, DFMLever_{1,f} \right) \)  \( DFM_{E,F} = 0.675 \)
- **Shear:** \( DFV_{E,F} := (\text{skew}_-\text{corr}_-\text{V}) \cdot DFMLever_{1,f} \)  \( DFV_{E,F} = 0.675 \)

\[
DF_{\text{Diaphragm}_-\text{Lanes}} := \begin{cases} 
"\text{More than 3 design lanes - adjust DF for diaphragm}" & \text{if } N_L > 3 \\
"\text{OK}" & \text{otherwise}
\end{cases}
\]

\( DF_{\text{Diaphragm}_-\text{Lanes}} = "\text{OK}" \)

\[
DF_{\text{Diaphragm}_-\text{Beams}} := \begin{cases} 
"\text{More than 5 beams - adjust DF for diaphragm}" & \text{if } N_L > 3 \\
"\text{OK}" & \text{otherwise}
\end{cases}
\]

\( DF_{\text{Diaphragm}_-\text{Beams}} = "\text{OK}" \)
Design Loads

Component Dead Loads:

DC1 - acts on non-composite section

\[ DC_{1\text{girder}} = \text{w}t_g \]

\[ DC_{1\text{attach}} = 5\% \cdot DC_{1\text{girder}} \]

Interior Girder

\[ DC_{1\text{deck_int}} := \left( \gamma_{\text{top_pl}} \cdot t_{\text{top_pl}} + \gamma_{\text{core}} \cdot t_{\text{core}} + \gamma_{\text{bot_pl}} \cdot t_{\text{bot_pl}} \right) \cdot S \]

\[ DC_{1\text{int}} := DC_{1\text{deck_int}} + DC_{1\text{girder}} + DC_{1\text{attach}} \]

Exterior Girder

\[ DC_{1\text{deck_ext}} := \left( \gamma_{\text{top_pl}} \cdot t_{\text{top_pl}} + \gamma_{\text{core}} \cdot t_{\text{core}} + \gamma_{\text{bot_pl}} \cdot t_{\text{bot_pl}} \right) \cdot \left( \frac{S}{2} + w_{\text{oh}} \right) \]

\[ DC_{1\text{ext}} := DC_{1\text{deck_ext}} + DC_{1\text{girder}} + DC_{1\text{attach}} \]

Superimposed Component Dead Loads (Long-Term):

DC2 - acts on long-term composite section. Assumed to be carried equally by all girders

\[ \text{w}t_{\text{parapet}} := 0.6 \text{ kip/ft} \]

\[ DC_{2\text{parapet}} := \frac{\text{w}t_{\text{parapet}}}{N_G} \]

\[ DC_{2\text{parapet}} = 0.15 \text{ kip/ft} \]

Superimposed Component Dead Loads (Long-Term):

DW - wearing surface load acts on long-term composite section. Assumed to be carried equally by all girders

\[ \text{w}t_{\text{future_ws}} := 30 \text{ psf} \]

\[ DW := \frac{\text{w}t_{\text{future_ws}} \cdot w_{\text{roadway}}}{N_G} \]

\[ DW = 0.18 \text{ kip/ft} \]
COMPOSITE SECTION PROPERTIES

Only short-term composite section properties are considered because no creep anticipated in SPS deck

overall depth of composite section
\[ d_{\text{comp}} := t_{f.t.} + D_w + t_{f.b.} + \text{offset plate} + t_{\text{top pl}} + t_{\text{core}} + t_{\text{bot pl}} \]
\[ d_{\text{comp}} = 55.295\text{in} \]

**Interior Beams**

Assume that the entire width of the deck is effective (only core material is transformed)

\[ b_{\text{e int}} := S \quad b_{\text{e int}} = 96\text{in} \]
\[ n_{\text{core}} := \frac{E_{\text{girder}}}{E_{\text{core}}} \quad n_{\text{core}} = 266.593 \]

\[ A_{\text{top pl int}} := t_{\text{top pl}} b_{\text{e int}} \quad A_{\text{top pl int}} = 18\text{in}^2 \]
\[ y_{\text{bar top pl}} := \frac{t_{\text{top pl}}}{2} \quad y_{\text{bar top pl}} = 0.094\text{in} \]
\[ A_{\text{core trans int}} := \frac{t_{\text{core}} b_{\text{e int}}}{n_{\text{core}}} \quad A_{\text{core trans int}} = 1.286\text{in}^2 \]
\[ y_{\text{bar core}} := t_{\text{top pl}} + \frac{t_{\text{core}}}{2} \quad y_{\text{bar core}} = 1.972\text{in} \]
\[ A_{\text{bot pl int}} := t_{\text{bot pl}} b_{\text{e int}} \quad A_{\text{bot pl int}} = 3.851\text{in} \]
\[ y_{\text{bar bot pl}} := t_{\text{top pl}} + t_{\text{core}} + \frac{t_{\text{bot pl}}}{2} \quad y_{\text{bar bot pl}} = 34.795\text{in} \]

area of top flange (previous calculation)
\[ A_{f.t.} = 19.2\text{in}^2 \]
\[ A_{f.b.} = 19.2\text{in}^2 \]

area of bottom flange (previous calculation)
\[ A_w = 24.6\text{in}^2 \]
\[ A_g = 63\text{in}^2 \]

area of web (previous calculation)
\[ A_{f.t.} = 12.495\text{in} \]
\[ A_{f.b.} = 34.795\text{in} \]

total girder area (previous calculation)
\[ A_{f.t.} = 54.695\text{in} \]
centroid of girder wrt comp section axis (top of composite section)

\[ y_{\text{bar, Comp}} := t_{\text{top, pl}} + t_{\text{core}} + t_{\text{bot, pl}} + \text{offset, plate} + d_{\text{Top, Steel, NC}} \]

\[ y_{\text{bar, Comp}} = 33.595 \text{ in} \]

\[ A_{Y_1} := A_{\text{top, pl int}} \cdot y_{\text{bar, top, pl}} \]

\[ A_{Y_1} = 1.688 \text{ in}^3 \]

\[ A_{Y_2} := A_{\text{core, trans, int}} \cdot y_{\text{bar, core}} \]

\[ A_{Y_2} = 2.536 \text{ in}^3 \]

\[ A_{Y_3} := A_{\text{bot, pl int}} \cdot y_{\text{bar, bot, pl}} \]

\[ A_{Y_3} = 69.323 \text{ in}^3 \]

\[ A_{\text{total, trans}} := A_g + A_{\text{top, pl int}} + A_{\text{core, trans, int}} + A_{\text{bot, pl int}} \]

\[ A_{\text{total, trans}} = 100.286 \text{ in}^2 \]

\[ y_{\text{bar, Top, Comp}} := \frac{(A_{Y_1} + A_{Y_2} + A_{Y_3} + A_{Y_4} + A_{Y_5} + A_{Y_6})}{A_{\text{total, trans}}} \]

\[ y_{\text{bar, Top, Comp}} = 22.132 \text{ in} \]

\[ y_{\text{bar, Bot, Comp}} := d_{\text{comp}} - y_{\text{bar, Top, Comp}} \]

\[ y_{\text{bar, Bot, Comp}} = 33.163 \text{ in} \]

\[ I_{\text{deck}} := \frac{1}{12} \left( b_{\text{e, int}}^3 t_{\text{top, pl}} + \frac{b_{\text{e, int}}^3 t_{\text{core}}}{n_{\text{core}}} + b_{\text{e, int}}^3 t_{\text{bot, pl}} \right) \]

\[ I_{\text{deck}} = 1.471 \text{ in}^4 \]

\[ A_{d_{\text{top, pl}}} := (A_{\text{top, pl int}}) \left( y_{\text{bar, top, pl}} - y_{\text{bar, Top, Comp}} \right)^2 \]

\[ A_{d_{\text{core}}} := (A_{\text{core, trans, int}}) \left( y_{\text{bar, core}} - y_{\text{bar, Top, Comp}} \right)^2 \]

\[ A_{d_{\text{bot, pl}}} := (A_{\text{bot, pl int}}) \left( y_{\text{bar, bot, pl}} - y_{\text{bar, Top, Comp}} \right)^2 \]

\[ A_{d_{\text{plate}}} := A_{d_{\text{top, pl}}} + A_{d_{\text{core}}} + A_{d_{\text{bot, pl}}} \]

\[ I_{\text{comp}} := I_{\text{NC}} + A_g \left( y_{\text{bar, girder, Comp}} - y_{\text{bar, Top, Comp}} \right)^2 + I_{\text{deck}} + A_{d_{\text{plate}}} \]

\[ I_{\text{comp}} = 4.411 \times 10^4 \text{ in}^4 \]

\[ d_{\text{Top, Steel, Comp}} := \begin{cases} d - y_{\text{bar, Bot, Comp}} & \text{if } y_{\text{bar, Top, Comp}} \geq t_{\text{deck, total}} + \text{offset, plate} \\ t_{\text{deck, total}} + \text{offset, plate} - y_{\text{bar, Top, Comp}} & \text{otherwise} \end{cases} \]

\[ S_{\text{Top, Steel, Comp}} := \frac{I_{\text{comp}}}{d_{\text{Top, Steel, Comp}}} \]

\[ S_{\text{Top, Steel, Comp}} = 4.308 \times 10^3 \text{ in}^3 \]

\[ S_{\text{Bot, Steel, Comp}} := \frac{I_{\text{comp}}}{y_{\text{bar, Bot, Comp}}} \]

\[ S_{\text{Bot, Steel, Comp}} = 1.33 \times 10^3 \text{ in}^3 \]
**Exterior Beams**

Assume that the half of span width and entire overhang are effective (only core material is transformed)

\[ b_{e\_ext} := \frac{S}{2} + w_{oh} \quad b_{e\_ext} = 76.8\text{in} \]

\[ n_{core} := \frac{E_{girder}}{E_{core}} \quad n_{core} = 266.593 \]

\[ A_{top\_pl\_ext} := t_{\text{top pl}} \cdot b_{e\_ext} \quad A_{top\_pl\_ext} = 14.4\text{in}^2 \]

\[ A_{core\_trans\_ext} := \frac{t_{\text{core}} \cdot b_{e\_ext}}{n_{core}} \quad A_{core\_trans\_ext} = 1.028\text{in}^2 \]

\[ A_{bot\_pl\_ext} := t_{\text{bot pl}} \cdot b_{e\_ext} \quad A_{bot\_pl\_ext} = 14.4\text{in}^2 \]

area of top flange (previous calculation)

area of bottom flange (previous calculation)

area of web (previous calculation)

total girder area (previous calculation)

centrifugal of girder wrt comp section axis (top of composite section)

\[ A_{y1\_ext} := A_{top\_pl\_ext} \cdot y_{\text{bar top pl}} \quad A_{y1\_ext} = 1.35\text{in}^3 \]

\[ A_{y2\_ext} := A_{core\_trans\_ext} \cdot y_{\text{bar core}} \quad A_{y2\_ext} = 2.029\text{in}^3 \]

\[ A_{y3\_ext} := A_{bot\_pl\_ext} \cdot y_{\text{bar bot pl}} \quad A_{y3\_ext} = 55.458\text{in}^3 \]

\[ A_{y4\_ext} := A_{f.t.} \cdot y_{\text{bar top flg}} \quad A_{y4\_ext} = 239.904\text{in}^3 \]

\[ A_{y5\_ext} := A_{w} \cdot y_{\text{bar web}} \quad A_{y5\_ext} = 855.957\text{in}^3 \]

\[ A_{y6\_ext} := A_{f.b.} \cdot y_{\text{bar bot flg}} \quad A_{y6\_ext} = 1.05 \times 10^3\text{in}^3 \]

\[ A_{\text{total trans ext}} := A_{g} + A_{top\_pl\_ext} + A_{core\_trans\_ext} + A_{bot\_pl\_ext} \quad A_{\text{total trans ext}} = 92.828\text{in}^2 \]
\[ y_{\text{bar Top Comp ext}} := \frac{(Ay_{1\text{ ext}} + Ay_{2\text{ ext}} + Ay_{3\text{ ext}} + Ay_{4\text{ ext}} + Ay_{5\text{ ext}} + Ay_{6\text{ ext}})}{A_{\text{total trans ext}}} \]

\[ y_{\text{bar Top Comp ext}} = 23.752\text{in} \]

\[ y_{\text{bar Bot Comp ext}} := d_{\text{comp}} - y_{\text{bar Top Comp ext}} \quad \quad y_{\text{bar Bot Comp ext}} = 31.543\text{in} \]

\[ I_{\text{deck ext}} := \frac{1}{12} \left( b_{e\text{ ext}} t_{\text{top pl}} \frac{3}{n_{\text{core}}} + b_{e\text{ ext}} t_{\text{bot pl}} \frac{3}{n_{\text{core}}} \right) \quad I_{\text{deck ext}} = 1.177\text{in}^4 \]

\[ \text{Ad}_{\text{top pl ext}} := \left( A_{\text{top pl ext}} \right) \left( y_{\text{bar top pl}} - y_{\text{bar Top Comp ext}} \right)^2 \]

\[ \text{Ad}_{\text{core ext}} := \left( A_{\text{core trans ext}} \right) \left( y_{\text{bar core}} - y_{\text{bar Top Comp ext}} \right)^2 \]

\[ \text{Ad}_{\text{bot pl ext}} := \left( A_{\text{bot pl ext}} \right) \left( y_{\text{bar bot pl}} - y_{\text{bar Top Comp ext}} \right)^2 \]

\[ \text{Ad}_{\text{plate ext}} := \text{Ad}_{\text{top pl ext}} + \text{Ad}_{\text{core ext}} + \text{Ad}_{\text{bot pl ext}} \]

\[ I_{\text{comp ext}} := I_{\text{NC}} + A_g \left( y_{\text{bar girder Comp}} - y_{\text{bar Top Comp ext}} \right)^2 + I_{\text{deck ext}} + \text{Ad}_{\text{plate ext}} \]

\[ I_{\text{comp ext}} = 4.09 \times 10^4 \text{in}^4 \]

\[ d_{\text{Top Steel Comp ext}} := \begin{cases} 
& d - y_{\text{bar Bot Comp ext}} \quad \text{if} \quad y_{\text{bar Top Comp ext}} \geq I_{\text{deck total}} + \text{offset plate} \\
& I_{\text{deck total}} + \text{offset plate} - y_{\text{bar Top Comp ext}} \quad \text{otherwise}
\end{cases} \]

\[ S_{\text{Top Steel Comp ext}} := \frac{I_{\text{comp}}}{d_{\text{Top Steel Comp ext}}} \quad S_{\text{Top Steel Comp ext}} = 3.72 \times 10^3 \text{in}^3 \]

\[ S_{\text{Bot Steel Comp ext}} := \frac{I_{\text{comp}}}{y_{\text{bar Bot Comp ext}}} \quad S_{\text{Bot Steel Comp ext}} = 1.398 \times 10^3 \text{in}^3 \]
Live Load Force Effects (Based on Influence lines for simple span)

\[ \text{IM}_{\text{comp}} := 33\% \quad \text{IM}_{\text{fatigue}} := 15\% \]

\[ w_{\text{uni\_lane}} := 0.64 \frac{\text{kip}}{\text{ft}} \]

\[ x := \max \left( \frac{L_{\text{span}}}{2} - 30 \text{ ft}, 0 \text{ ft} \right) \]

determines if back wheel of fatigue truck is positioned on bridge with 30 ft spacing

\[ M_{\text{uni\_lane}} := \frac{w_{\text{uni\_lane}} L_{\text{span}}^2}{8} \]

\[ M_{\text{uni\_lane}} = 512\text{kip-ft} \]

\[ M_{\text{truck}} := 32\text{kip} \left( \frac{L_{\text{span}}}{4} \right) + 8\text{kip} \left( \frac{L_{\text{span}}}{2} - 14 \text{ ft} \right) \frac{1}{2} + 32\text{kip} \left( \frac{L_{\text{span}}}{2} - 14 \text{ ft} \right) \frac{1}{2} \]

\[ M_{\text{truck}} = 1160\text{kip-ft} \]

\[ M_{\text{tandem}} := 25\text{kip} \left( \frac{L_{\text{span}}}{4} \right) + 25\text{kip} \left( \frac{L_{\text{span}}}{2} - 4 \text{ ft} \right) \frac{1}{2} \]

\[ M_{\text{tandem}} = 950\text{kip-ft} \]

\[ M_{\text{fatigue}} := 32\text{kip} \left( \frac{L_{\text{span}}}{4} \right) + 8\text{kip} \left( \frac{L_{\text{span}}}{2} - 14 \text{ ft} \right) \frac{1}{2} + 32\text{kip} \left( x \right) \frac{1}{2} \]

\[ M_{\text{fatigue}} = 904\text{kip-ft} \]

\[ V_{\text{uni\_lane}} := \frac{w_{\text{uni\_lane}} L_{\text{span}}}{2} \]

\[ V_{\text{uni\_lane}} = 25.6\text{kip} \]

\[ V_{\text{truck}} := 32\text{kip} \left( 1 + 32\text{kip} \left( \frac{-1}{L_{\text{span}}} \cdot 14 \text{ ft} + 1 \right) + 8\text{kip} \left[ \frac{-1}{L_{\text{span}}} \left( 14 \text{ ft} + 14 \text{ ft} \right) + 1 \right] \right) \]

\[ V_{\text{truck}} = 63.6\text{kip} \]

\[ V_{\text{tandem}} := 25\text{kip} \left( 1 + 25\text{kip} \left( \frac{-1}{L_{\text{span}}} \cdot 4 \text{ ft} + 1 \right) \right) \]

\[ V_{\text{tandem}} = 49\text{kip} \]

\[ V_{\text{fatigue}} := 32\text{kip} \left( 1 + 8\text{kip} \left[ \frac{-1}{L_{\text{span}}} \left( 14 \text{ ft} \right) + 1 \right] \right) \]

\[ V_{\text{fatigue}} = 39\text{kip} \]
**Interior Girder Live Loads**

\[ V_{\text{Int LL IM}} := DFV_I \left[ \max(V_{\text{truck}} \cdot V_{\text{tandem}}) \left( 1 + \frac{IM_{\text{comp}}}{100} \right) + V_{\text{uni lane}} \right] \]

\[ V_{\text{Int Fatigue IM}} := DFV_I \left[ V_{\text{fatigue}} \left( 1 - \frac{IM_{\text{fatigue}}}{100} \right) \right] \]

\[ M_{\text{Int LL IM}} := DFM_I \left[ \max(M_{\text{truck}}, M_{\text{tandem}}) \left( 1 + \frac{IM_{\text{comp}}}{100} \right) + M_{\text{uni lane}} \right] \]

\[ M_{\text{Int Fatigue IM}} := DFM_I \left[ M_{\text{fatigue}} \left( 1 - \frac{IM_{\text{fatigue}}}{100} \right) \right] \]

**Exterior Girder Live Loads**

\[ V_{\text{Ext LL IM}} := DFV_E \left[ \max(V_{\text{truck}} \cdot V_{\text{tandem}}) \left( 1 + \frac{IM_{\text{comp}}}{100} \right) + V_{\text{uni lane}} \right] \]

\[ V_{\text{Ext Fatigue IM}} := DFV_E \left[ V_{\text{fatigue}} \left( 1 - \frac{IM_{\text{fatigue}}}{100} \right) \right] \]

\[ M_{\text{Ext LL IM}} := DFM_E \left[ \max(M_{\text{truck}}, M_{\text{tandem}}) \left( 1 + \frac{IM_{\text{comp}}}{100} \right) + M_{\text{uni lane}} \right] \]

\[ M_{\text{Ext Fatigue IM}} := DFM_E \left[ M_{\text{fatigue}} \left( 1 - \frac{IM_{\text{fatigue}}}{100} \right) \right] \]

**Interior Dead Loads**

\[ M_{\text{DC1 int}} := \frac{DC_{\text{1 int}} \cdot L_{\text{span}}}{8} \]

\[ M_{\text{DC2 int}} := \frac{DC_{\text{2 parapet}} \cdot L_{\text{span}}}{8} \]

\[ M_{\text{DW int}} := \frac{DW \cdot L_{\text{span}}}{8} \]

\[ V_{\text{DC1 int}} := \frac{DC_{\text{1 int}} \cdot L_{\text{span}}}{2} \]

\[ V_{\text{DC2 int}} := \frac{DC_{\text{2 parapet}} \cdot L_{\text{span}}}{2} \]

\[ V_{\text{DW int}} := \frac{DW \cdot L_{\text{span}}}{2} \]
Exterior Dead Loads

\[ M_{DC1_{\text{ext}}} = \frac{DC_{1_{\text{ext}}} \cdot L_{\text{span}}^2}{8} \quad \text{MDC}_{1_{\text{ext}}} = 368 \text{kip-ft} \]

\[ M_{DC2_{\text{ext}}} = \frac{DC_{2_{\text{ext}}} \cdot L_{\text{span}}^2}{8} \quad \text{MDC}_{2_{\text{ext}}} = 120 \text{kip-ft} \]

\[ M_{DW_{\text{ext}}} = \frac{D \cdot L_{\text{span}}^2}{8} \quad \text{MDW}_{\text{ext}} = 144 \text{kip-ft} \]

\[ V_{DC1_{\text{ext}}} = \frac{DC_{1_{\text{ext}}} \cdot L_{\text{span}}}{2} \quad \text{VDC}_{1_{\text{ext}}} = 18 \text{kip} \]

\[ V_{DC2_{\text{ext}}} = \frac{DC_{2_{\text{ext}}} \cdot L_{\text{span}}}{2} \quad \text{VDC}_{2_{\text{ext}}} = 6 \text{kip} \]

\[ V_{DW_{\text{ext}}} = \frac{D \cdot L_{\text{span}}}{2} \quad \text{VDW}_{\text{ext}} = 7 \text{kip} \]
**Plastic-Moment Capacity (Interior)**

Assume no net axial force

Neglect polymer core contribution to plastic moment capacity

Top and bottom rebar location equivalent to top and bottom plate location (Ps=0)

Determine location of PNA

\[
P_{tp} := t_{top\_pl} \cdot b_{e\_int} \cdot F_y\_plate
\]

\[
P_{tp} = 900\text{kip}
\]

\[
P_{rt} := P_{tp}
\]

\[
P_{bp} := t_{bot\_pl} \cdot b_{e\_int} \cdot F_y\_plate
\]

\[
P_{bp} = 900\text{kip}
\]

\[
P_{rb} := P_{bp}
\]

\[
P_{tf} := t_{f.t.} \cdot b_{f.t.} \cdot F_y\_girder
\]

\[
P_{tf} = 960\text{kip}
\]

\[
P_{tc} := P_{tf}
\]

\[
P_{w} := D_w \cdot t_w \cdot F_y\_girder
\]

\[
P_{w} = 1.23 \times 10^3\text{kip}
\]

\[
P_{bf} := t_{f.b.} \cdot b_{f.b.} \cdot F_y\_girder
\]

\[
P_{bf} = 72\text{kip}
\]

\[
P_{t} := P_{bf}
\]

\[
P_{s} := 0\text{-kip}
\]

\[
P_{s} = 0\text{kip}
\]

\[
P_{ten} := P_{tf} + P_{w} + P_{bf}
\]

\[
P_{ten} = 2262\text{kip}
\]

\[
P_{comp} := P_{tp} + P_{bp}
\]

\[
P_{comp} = 1800\text{kip}
\]

\[
C_{rb} := t_{top\_pl} + t_{core} + \frac{t_{bot\_pl}}{2}
\]

\[
C_{rt} := \frac{t_{top\_pl}}{2}
\]

\[
t_s := t_{deck\_total}
\]

\[
C_{rb} = 3.851\text{in}
\]

\[
C_{rt} = 0.094\text{in}
\]

\[
t_s = 3.945\text{in}
\]

CASE :=

<table>
<thead>
<tr>
<th>CASE</th>
<th>Return Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;CASE I&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE II&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE III&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE IV&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE V&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE VI&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
<tr>
<td>&quot;CASE VII&quot;</td>
<td>if $P_t + P_w + P_c + P_s + P_{rb} + P_{rt}$</td>
</tr>
</tbody>
</table>

"ERROR" otherwise

CASE = "CASE II"
This section applies only to Case IV and Case II
Modify for other cases

\[ Y_{\text{bar}} := \begin{cases} \text{return } C_{rb} & \text{if CASE} = "\text{CASE IV}" \\ \text{return } \frac{tf.t.}{2} \left( \frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1 \right) & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ y_{rt} := \begin{cases} \text{return } Y_{\text{bar}} - \frac{t_{\text{top pl}}}{2} & \text{if CASE} = "\text{CASE IV}" \\ \text{return } \frac{t_{\text{top pl}}}{2} + t_{\text{core}} + t_{\text{bot pl}} + \text{offset plate} + \frac{tf.t.}{2} & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ y_{w} := \begin{cases} \text{return } t_s + \text{offset plate} + tf.t. + \frac{D_w}{2} - Y_{\text{bar}} & \text{if CASE} = "\text{CASE IV}" \\ \text{return } \frac{D_w}{2} + \frac{tf.t.}{2} & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ y_{t} := \begin{cases} \text{return } t_s + \text{offset plate} + tf.t. + D_w + \frac{tf.b.}{2} - Y_{\text{bar}} & \text{if CASE} = "\text{CASE IV}" \\ \text{return } D_w + \frac{tf.t.}{2} + \frac{tf.b.}{2} & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ y_{c} := \begin{cases} \text{return } t_s + \text{offset plate} + \frac{tf.t.}{2} - Y_{\text{bar}} & \text{if CASE} = "\text{CASE IV}" \\ \text{return } 0 & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ y_{rb} := \begin{cases} \text{return } 0 & \text{if CASE} = "\text{CASE IV}" \\ \text{return } \frac{t_{\text{bot pl}}}{2} + \text{offset plate} + \frac{tf.t.}{2} & \text{if CASE} = "\text{CASE II}" \end{cases} \]

\[ M_p := \begin{cases} \text{return } \left( \frac{Y_{\text{bar}}^2 - P_s}{2\times tf.t.} \right) + \left( P_{rt}y_{rt} + P_c y_c + P_w y_w + P_t y_t \right) & \text{if CASE} = "\text{CASE IV}" \\ \text{return } \left( \frac{P_c}{2\times tf.t.} \right) \left[ Y_{\text{bar}}^2 + \left( tf.t. - Y_{\text{bar}} \right)^2 \right] + \left( P_{rt}y_{rt} + P_{rb} y_{rb} + P_w y_w + P_t y_t \right) & \text{if CASE} = "\text{CASE II}" \\ "\text{ERROR}" & \text{otherwise} \end{cases} \]

\[ \begin{align*} Y_{\text{bar}} &= 0.289\text{in} & y_{w} &= 21.1\text{in} & y_{c} &= 0 \end{align*} \]
\[ \begin{align*} y_{rt} &= 12.401\text{in} & y_{t} &= 42.2\text{in} & y_{rb} &= 8.644\text{in} \end{align*} \]
\[ M_p = 4025\text{kip-ft} \]
I-SECTION FLEXURAL MEMBERS

General:
All types of I-section flexural members shall be designed as a minimum to satisfy:
The cross section proportion limits specified in Article 6.10.2;
The constructability requirements specified in Article 6.10.3;
The service limit state requirements specified in Article 6.10.4;
The fatigue and fracture limit state requirements specified in Article 6.10.5;
The flexural strength limit state requirements specified in Article 6.10.6.
The shear strength limit state requirements specified in Article 6.10.9

1. Cross Section Proportional Limits

Web Proportions:
Web slenderness limit: \( \frac{D_w}{t_w} \leq 150 \)

Check_Web_Slenderness := "OK" if \( \frac{D_w}{t_w} \leq 150 \)
"Limit not met" otherwise

Check_Web_Slenderness = "OK"

Allows for easier proportioning of the web in preliminary design
Satisfies elastic buckling of the web as a column subjected to a radial transverse compression from the curvature of the flanges
Allows web-bend buckling to be disregarded in design of composite sections in positive flexure

Flange Proportions:

Flange weld/distortion/Compactness limit: \( \frac{b_f}{2\cdot t_f} \leq 12.0 \)

Check_Flange_Limit := "OK" if max \( \left( \frac{b_{f.t.}}{2\cdot t_{f.t.}}, \frac{b_{f.b.}}{2\cdot t_{f.b.}} \right) \leq 12.0 \)
"Limit not met" otherwise

Check_Flange_Limit = "OK"

Limits distortion of flange when welded to the web

Local buckling limit
Web depth to flange width aspect ration: \[ b_f \geq \frac{D_w}{6} \]

Check_Flange_Web_Limit := "OK" if \( \min(b_{f.t} \cdot b_{f.b}) \geq \frac{D_w}{6} \)

"Limit not met" otherwise

\textbf{Check_Flange_Web_Limit = "OK"}

Controls strength and moment-rotation characteristics of the I-section.
Ensures stiffened web panels to reach requirement for post-buckling shear.

Web shear buckling limit: \[ t_f \geq 1.1 \cdot t_w \]

Check_Shear_Buckling_Limit := "OK" if \( \min(t_{f.t} \cdot t_{f.b}) \geq 1.1 \cdot t_w \)

"Limit not met" otherwise

\textbf{Check_Shear_Buckling_Limit = "OK"}

Ensures some restraint will be provided by the flanges against web shear buckling.
Satisfies assumed boundary conditions for web-flange juncture in the web-bend buckle and compression flange local buckling formulas.

Flange Proportion limit: \[ 0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10 \]

Tension flange: \[ I_{yt} := \frac{t_{f.b} \cdot b_{f.b}^3}{12} \]
Compression flange: \[ I_{yc} := \frac{t_{f.t} \cdot b_{f.t}^3}{12} \]

Check_Flange_Proportion_Limit := "OK" if \( 0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10 \)

"Proportion not within bounds" otherwise

\textbf{Check_Flange_Proportion_Limit = "OK"}

Establishes I-section proportional limits in order to ensure validity of equations in specification.
Ensures more efficient flange proportions and prevents the use of sections that ay be particularly difficult to handle during construction.

2. Constructability

Constructability will not be check for the SPS design - the panels will either be composite with the girder for the panel on girder configuration or made composite in the transverse panel design.
3. Service Limit State: Control of Permanent Deflections

Flexure check for top flange steel: \( f_t \leq 0.95 \cdot R_h \cdot F_y \)

The compression-flange flexural stresses resulting from the unfactored loads are as follows:

- \( M_{DC1_{int}} = 414.687 \text{kip ft} \)
  - \( f_{DC1_{cf}} = \frac{M_{DC1_{int}}}{S_{Top\_Steel\_NC}} \) \( f_{DC1_{cf}} = 5.256 \text{ksi} \)

- \( M_{DC2_{int}} = 120 \text{kip ft} \)
  - \( f_{DC2_{cf}} = \frac{M_{DC2_{int}}}{S_{Top\_Steel\_Comp}} \) \( f_{DC2_{cf}} = 0.334 \text{ksi} \)

- \( M_{DW_{int}} = 144 \text{kip ft} \)
  - \( f_{DW_{cf}} = \frac{M_{DW_{int}}}{S_{Top\_Steel\_Comp}} \) \( f_{DW_{cf}} = 0.401 \text{ksi} \)

- \( M_{Int\_LL\_IM} = 1.255 \times 10^3 \text{kip ft} \)
  - \( f_{LL\_IM_{cf}} = \frac{M_{Int\_LL\_IM}}{S_{Top\_Steel\_Comp}} \) \( f_{LL\_IM_{cf}} = 3.495 \text{ksi} \)

Applying SERVICE II Factors:

\( f_{serviceII_{cf}} := \eta \left( 1.0 f_{DC1_{cf}} + 1.0 f_{DC2_{cf}} + 1.0 f_{DW_{cf}} + 1.3 f_{LL\_IM_{cf}} \right) \)

\( f_{serviceII_{cf}} = 10.535 \text{ksi} \)

Hybrid Factor:

The compression flange has the same yield strength as the the web, there the hybrid factor is:

This analysis assumes that the NA is further from the bottom flange than the top flange

\( f_n := F_y \cdot f_{girder} \)
\( \rho := \min \left( \frac{F_y \cdot f_{girder}}{f_n}, 1 \right) \)
\( D_n := y_{bar\_Bot\_Comp} - f_{f.b.} \)
\( A_{fn} := A_{f.b.} \)
\( \beta := \frac{2 \cdot D_n \cdot f_w}{A_{fn}} \)
\( R_h := \frac{12 + \beta \left( 3 \cdot \rho - \rho^3 \right)}{12 + 2 \beta} \)
\( R_h = 1 \)

Flexure check for top flange steel:

\( \text{Check\_Comp\_Flange\_Service\_II} := \)

"OK" if \( f_{serviceII_{cf}} \leq 0.95 \cdot R_h \cdot F_y \cdot f_{girder} \)

"Permanent deflection limitation exceeded" otherwise

\( \text{PR\_COMP\_FLANGE\_SERVICE\_II} = 22.179\% \)
Flexure check for bottom flange steel: \( f_r + \frac{f_i}{2} \leq 0.95R_hF_yf \)

The compression-flange flexural stresses resulting from the unfactored loads are as follows:

\[
\begin{align*}
M_{DC1_{\text{int}}} &= 414.687 \text{kip ft} \\
\frac{f_{DC1_{\text{tf}}}}{S_{\text{Bot Steel NC}}} &= \frac{M_{DC1_{\text{int}}}}{S_{\text{Bot Steel NC}}} \\
f_{DC1_{\text{tf}}} &= 5.256 \text{ksi} \\
M_{DC2_{\text{int}}} &= 120 \text{kip ft} \\
\frac{f_{DC2_{\text{tf}}}}{S_{\text{Bot Steel Comp}}} &= \frac{M_{DC2_{\text{int}}}}{S_{\text{Bot Steel Comp}}} \\
f_{DC2_{\text{tf}}} &= 1.083 \text{ksi} \\
M_{DW_{\text{int}}} &= 144 \text{kip ft} \\
\frac{f_{DW_{\text{tf}}}}{S_{\text{Bot Steel Comp}}} &= \frac{M_{DW_{\text{int}}}}{S_{\text{Bot Steel Comp}}} \\
f_{DW_{\text{tf}}} &= 1.299 \text{ksi} \\
M_{\text{Int LL IM}} &= 1.255 \times 10^3 \text{kip ft} \\
\frac{f_{LL_{\text{IM tf}}}}{S_{\text{Bot Steel Comp}}} &= \frac{M_{\text{Int LL IM}}}{S_{\text{Bot Steel Comp}}} \\
f_{LL_{\text{IM tf}}} &= 11.323 \text{ksi} \\
\end{align*}
\]

Applying SERVICE II Factors:

\[
f_{\text{serviceII}_{\text{tf}}} := \eta \left( 1.0f_{DC1_{\text{tf}}} + 1.0f_{DC2_{\text{tf}}} + 1.0f_{\text{DW}_{\text{tf}}} + 1.3f_{\text{LL}_{\text{IM tf}}} \right)
\]

\[f_{\text{serviceII}_{\text{tf}}} = 22.357 \text{ksi}\]

Flange lateral bending stresses are assumed to be negligible for straight (nonskewed) bridges [Barker & Puckett]

\[f_i := 0 \text{ ksi}\]

Flexure check for bottom flange steel:

\[
\begin{align*}
\text{Check Ten Flange Service II} := & \begin{cases} 
\text{"OK"} & \text{if } f_{\text{serviceII}_{\text{tf}}} + \frac{f_i}{2} \leq 0.95R_hF_y\text{girder} \\
\text{"Permanent deflection limitation exceeded" otherwise} 
\end{cases} \\
\text{Check Ten Flange Service II} = \text{"OK"}
\end{align*}
\]

\[
\begin{align*}
\text{PR TEN FLANGE SERVICE II} := & \frac{f_{\text{serviceII}_{\text{tf}}} + \frac{f_i}{2}}{0.95R_hF_y\text{girder}} \\
\text{PR TEN FLANGE SERVICE II} = & 47.068\%
\end{align*}
\]
4. Fatigue and Fracture Limit State

Fatigue can be an issue for SPS (connections and weathering steel), but will not be assessed for this analysis. The fatigue and fracture resistance will be assumed to be adequate.

5. Flexural Strength Limit State

Compact Sections:
Sections that satisfy the following requirements shall qualify as compact sections:
The specified minimum yield strength of the flanges does not exceed 70 ksi

The web satisfies the requirements of article 6.10.2.1.1

The section satisfies the slenderness limit:

\[ \frac{2D_{cp}}{t_w} \leq 3.76 \sqrt{\frac{E_s}{F_{yc}}} \]

Depth of web in compression at the plastic moment:

\[ D_{cp\_case I} = \frac{D_w}{2} \left[ F_{y\_girder} - F_{y\_girder} - P_s - F_{y\_plate} \left( t_{top\_pl} b_{e\_int} + t_{top\_pl} b_{e\_int} \right) + 1 \right] \]

\[ D_{cp} := \begin{cases} D_{cp\_case I} & \text{if CASE = "CASE I"} \\ 0 & \text{otherwise} \end{cases} \]

Section :=

"Compact" if \( \left( \frac{2D_{cp}}{t_w} \leq 3.76 \sqrt{\frac{E_{girder}}{F_{y\_girder}}} \right) \left( F_{y\_girder} \leq 70\text{ksi} \right) \left( \frac{D_w}{t_w} \leq 150 \right) \)

"Non Compact" otherwise

**Flexural Strength Limit State Check**

\[ M_u + \frac{1}{3} f_{\frac{f}{f}} S_{xt} \leq \phi_f M_n \]

\[ \phi_f := 1.0 \]

\[ M_n := \begin{cases} M_p & \text{if } D_p \leq 0.1D_t \\ M_p \left[ 1.07 - 0.7 \left( \frac{D_p}{D_t} \right) \right] & \text{otherwise} \end{cases} \]

\[ M_n = 4025\text{kip} \cdot \text{ft} \]
\[ M_{AD} := \left( \frac{1.25 \text{MDC}_{1 \text{int}}}{S_{\text{Bot Steel NC}}} + \frac{-1.25 \text{MDC}_{2 \text{int}} - 1.5 \text{MDW}_{\text{int}}}{S_{\text{Bot Steel Comp}}} \right) S_{\text{Bot Steel Comp}} \]

\[ M_{AD} = 4448 \text{kip} \cdot \text{ft} \]

\[ M_{yt} := (1.25 \text{MDC}_{1 \text{int}} + 1.25 \text{MDC}_{2 \text{int}} + 1.5 \text{MDW}_{\text{int}} + M_{AD}) \]

\[ F_{yt} := F_{y \text{ girder}} \]

\[ S_{xt} := \frac{M_{yt}}{F_{yt}} \]

\[ M_{u \text{ strength}_1} := 1.25 \text{MDC}_{1 \text{int}} + 1.25 \text{MDC}_{2 \text{int}} + 1.5 \text{MDW}_{\text{int}} + 1.75 \text{M}_{\text{Int LL ILM}} \]

\[ M_{u \text{ strength}_1} = 3.081 \times 10^3 \text{kip} \cdot \text{ft} \]

Check Strength I Flexure := "OK" if \( M_{u \text{ strength}_1} + \frac{1}{3} f_l S_{xt} \leq \phi_f M_n \)

"Flexural resistance not adequate" otherwise

Check Strength I Flexure = "OK"

\[ PR_{\text{Strength I Flexure}} := \frac{M_{u \text{ strength}_1} + \frac{1}{3} f_l S_{xt}}{\phi_f M_n} \]

PR Strength I Flexure := 76.539%

**Ductility Requirement**

\[ D_p = 0.289 \text{in} \]

\[ D_t = 55.295 \text{in} \]

Check Ductility := "OK" if \( D_p \leq 0.42 D_t \)

"Brittle failure" otherwise

Check Ductility = "OK"

\[ PR_{\text{Ductility}} := \frac{D_p}{0.42 D_t} \]

PR Ductility := 1.243%
6. Shear Strength Limit State

**Shear Strength Limit State Check**

\[ V_r = \phi_V V_n \quad \phi_V := 1.0 \]

Plastic Shear Force

\[ V_p := 0.58 F_y \cdot D_w \cdot t_w \quad V_p = 713.4 \text{kip} \]

Shear Buckling coefficient for unistiffened web:

\[ k := 5 \]

\[
C := \begin{cases} 
1.0 & \text{if } \frac{D_w}{t_w} < 1.12 \sqrt{\frac{E_{girder} \cdot k}{F_y}} \\
1.12 \frac{E_{girder} \cdot k}{F_y} & \text{if } 1.12 \sqrt{\frac{E_{girder} \cdot k}{F_y}} \leq \frac{D_w}{t_w} \leq 1.40 \sqrt{\frac{E_{girder} \cdot k}{F_y}} \\
1.12 \frac{E_{girder} \cdot k}{F_y} & \text{if } \frac{D_w}{t_w} > 1.40 \sqrt{\frac{E_{girder} \cdot k}{F_y}}
\end{cases}
\]

\[ V_{cr} := C \cdot V_p \quad V_{cr} = 629.677 \text{kip} \]

\[ V_{r\_unstiffened} := \phi_V V_{cr} \quad V_{r\_unstiffened} = 629.677 \text{kip} \]

**End Panel at Abutment**

Shear from factored loads at abutment location (end of beam):

\[ V_{u\_strength\_I} := 1.25 V_{DC1\_int} + 1.25 V_{DC2\_int} + 1.5 V_{DW\_int} + 1.75 V_{Int\_LL\_IM} \]

\[ V_{u\_strength\_I} = 171.648 \text{kip} \]

Check_Shear := "OK for unstiffened Design" if \( V_{u\_strength\_I} \leq V_{r\_unstiffened} \)

"Increase Shear Resistance (add stiffeners)" otherwise

**Check_Shear = "OK for unstiffened Design"**

\[ \text{PR\_Shear} := \frac{V_{u\_strength\_I}}{V_{r\_unstiffened}} \quad \text{PR\_Shear} = 27.26\% \]

299
SUMMARY OF CHECKS (Interior Girder)

DF_Diaphragm_Lanes = "OK"
Determine if DF calcs for diaphragm effects have correct # of beams and lanes

DF_Diaphragm_Beams = "OK"

Check_Flange_Limit = "OK"
Check_Web_Slenderness = "OK"
Check_Flange_Web_Limit = "OK"
Check_Shear_Buckling_Limit = "OK"

ENSURE THAT NO RESERVE % (PR) is GREATER THAN 85% ---> Try to keep MAX PR close to 85% if possible

Check_Comp_Flange_Service_II = "OK"
PR_COMP_FLANGE_SERVICE_II = 22.179%

Check_Ten_Flange_Service_II = "OK"
PR_TEN_FLANGE_SERVICE_II = 47.068%

Section = "Compact"

Check_Strength_I_Flexure = "OK"
PR_Strength_I_Flexure = 76.539%

Check_Ductility = "OK"
PR_Ductility = 1.243%

Check_Shear = "OK for unstiffened Design"
PR_Shear = 27.26%

CASE = "CASE II"

SUMMARY OF TRIAL GIRDER DIMENSIONS (Input)

- Top flange thickness: $t_{f.t.} = 1.2\text{in}$
- Top flange width: $b_{f.t.} = 16\text{in}$
- Web depth: $D_w = 41\text{in}$
- Web thickness: $t_w = 0.6\text{in}$
- Bottom flange width: $b_{f.b.} = 16\text{in}$
- Bottom flange thickness: $t_{f.b.} = 1.2\text{in}$
- Total Girder Depth: $D_{girder} = 43.4\text{in}$
### C.4 – Parametric Results Output

**Sample Results**

<table>
<thead>
<tr>
<th>Run Designation</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
<th>120(L-40)-0.375(TPL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_1$ Hz</td>
<td>2.97</td>
<td>2.97</td>
<td>2.97</td>
<td>2.97</td>
<td>2.97</td>
<td>2.97</td>
<td>2.97</td>
</tr>
<tr>
<td>$t_2$ Hz</td>
<td>3.35</td>
<td>3.35</td>
<td>3.35</td>
<td>3.35</td>
<td>3.35</td>
<td>3.35</td>
<td>3.35</td>
</tr>
<tr>
<td>$t_3$ Hz</td>
<td>8.48</td>
<td>8.48</td>
<td>8.48</td>
<td>8.48</td>
<td>8.48</td>
<td>8.48</td>
<td>8.48</td>
</tr>
<tr>
<td>$t_4$ Hz</td>
<td>8.58</td>
<td>8.58</td>
<td>8.58</td>
<td>8.58</td>
<td>8.58</td>
<td>8.58</td>
<td>8.58</td>
</tr>
<tr>
<td>$t_5$ Hz</td>
<td>8.68</td>
<td>8.68</td>
<td>8.68</td>
<td>8.68</td>
<td>8.68</td>
<td>8.68</td>
<td>8.68</td>
</tr>
<tr>
<td>$t_6$ Hz</td>
<td>8.70</td>
<td>8.70</td>
<td>8.70</td>
<td>8.70</td>
<td>8.70</td>
<td>8.70</td>
<td>8.70</td>
</tr>
<tr>
<td><strong>mass</strong> kip-sec/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total Span</strong> ft</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td><strong>Transverse Span</strong> ft</td>
<td>26.4</td>
<td>26.4</td>
<td>26.4</td>
<td>26.4</td>
<td>26.4</td>
<td>26.4</td>
<td></td>
</tr>
<tr>
<td><strong># Girder</strong></td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td><strong># Trucks</strong></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td><strong>Plate Thickness</strong> in.</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td>0.375</td>
<td></td>
</tr>
<tr>
<td><strong>Core Thickness</strong> in.</td>
<td>1.05</td>
<td>1.05</td>
<td>1.05</td>
<td>1.05</td>
<td>1.05</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td><strong>Truck X Coord</strong> in.</td>
<td>892</td>
<td>892</td>
<td>892</td>
<td>892</td>
<td>892</td>
<td>892</td>
<td>892</td>
</tr>
<tr>
<td><strong>Truck Y Coord</strong> in.</td>
<td>108</td>
<td>108</td>
<td>108</td>
<td>108</td>
<td>108</td>
<td>108</td>
<td></td>
</tr>
<tr>
<td><strong>Truck X Coord</strong> in.</td>
<td>836</td>
<td>836</td>
<td>836</td>
<td>836</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Truck Y Coord</strong> in.</td>
<td>246</td>
<td>246</td>
<td>246</td>
<td>246</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>UL (Order 1)</strong> in.</td>
<td>-0.400</td>
<td>-0.737</td>
<td>-0.740</td>
<td>-0.743</td>
<td>0.014</td>
<td>0.016</td>
<td>0.017</td>
</tr>
<tr>
<td><strong>UL (Order 2)</strong> in.</td>
<td>-0.477</td>
<td>-0.609</td>
<td>-0.611</td>
<td>-0.613</td>
<td>-0.108</td>
<td>-0.108</td>
<td>-0.107</td>
</tr>
<tr>
<td><strong>UL (Order 3)</strong> in.</td>
<td>-0.552</td>
<td>-0.481</td>
<td>-0.483</td>
<td>-0.484</td>
<td>-0.231</td>
<td>-0.231</td>
<td>-0.231</td>
</tr>
<tr>
<td><strong>UL (Order 4)</strong> in.</td>
<td>-0.626</td>
<td>-0.359</td>
<td>-0.355</td>
<td>-0.358</td>
<td>-0.231</td>
<td>-0.231</td>
<td>-0.231</td>
</tr>
<tr>
<td><strong>UL (Order 5)</strong> in.</td>
<td>-0.701</td>
<td>-0.231</td>
<td>-0.231</td>
<td>-0.231</td>
<td>-0.481</td>
<td>-0.481</td>
<td>-0.484</td>
</tr>
<tr>
<td><strong>UL (Order 6)</strong> in.</td>
<td>-0.778</td>
<td>-0.108</td>
<td>-0.107</td>
<td>-0.107</td>
<td>-0.609</td>
<td>-0.613</td>
<td>-0.613</td>
</tr>
<tr>
<td><strong>UL (Order 7)</strong> in.</td>
<td>-0.855</td>
<td>0.014</td>
<td>0.016</td>
<td>0.017</td>
<td>-0.737</td>
<td>-0.740</td>
<td>-0.743</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>606</td>
<td>606</td>
<td>606</td>
<td>606</td>
<td>606</td>
<td>606</td>
<td>606</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>48</td>
<td>48</td>
<td>48</td>
<td>48</td>
<td>48</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>96</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>144</td>
<td>144</td>
<td>144</td>
<td>144</td>
<td>144</td>
<td>144</td>
<td>144</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td></td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>192</td>
<td>192</td>
<td>192</td>
<td>192</td>
<td>192</td>
<td>192</td>
<td>192</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>240</td>
<td>240</td>
<td>240</td>
<td>240</td>
<td>240</td>
<td>240</td>
<td>240</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
<td>696</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>288</td>
<td>288</td>
<td>288</td>
<td>288</td>
<td>288</td>
<td>288</td>
<td>288</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>41</td>
<td>70</td>
<td>70</td>
<td>70</td>
<td>70</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>48</td>
<td>56</td>
<td>56</td>
<td>56</td>
<td>56</td>
<td>56</td>
<td>56</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>56</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>56</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>68</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>75</td>
<td>9</td>
<td>8</td>
<td>8</td>
<td>55</td>
<td>55</td>
<td>55</td>
</tr>
<tr>
<td><strong>Girder X Coord</strong> in.</td>
<td>83</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>68</td>
<td>68</td>
<td>68</td>
</tr>
<tr>
<td><strong>Girder Y Coord</strong> in.</td>
<td>-0.877</td>
<td>-0.796</td>
<td>-0.801</td>
<td>-0.796</td>
<td>-0.796</td>
<td>-0.801</td>
<td></td>
</tr>
<tr>
<td><strong>X Location (Max U2)</strong> in.</td>
<td>710.2</td>
<td>710.2</td>
<td>710.2</td>
<td>710.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Y Location (Max U2)</strong> in.</td>
<td>302.4</td>
<td>302.4</td>
<td>302.4</td>
<td>302.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Max mx</strong> in.</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
<td></td>
</tr>
<tr>
<td><strong>Max my</strong> in.</td>
<td>8.3</td>
<td>8.3</td>
<td>8.3</td>
<td>8.3</td>
<td>8.3</td>
<td>8.3</td>
<td></td>
</tr>
<tr>
<td><strong>Max xy</strong> in.</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td><strong>Max ox</strong> in.</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Max oy</strong> in.</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Max ox</strong> in.</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Max oy</strong> in.</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

301
Appendix D – SPS Vibration Parametric Results
Table 53 – Tabular Results of Vibration Parametric Investigation
Natural
Bridge
Span Bridge Face Plate
Core
Girder
Frequency
Mass
Length Width Thickness Thickness
#
Spacing
(Hz)
(kip-sec2)/in (ft)
(ft)
(in.)
(in.)
Girders
(ft)
9.57
0.240
40
27.6
0.1875
2.39
5
6
9.72
0.244
40
26
0.1875
3.22
3
10
9.77
0.246
40
27.6
0.25
1.98
5
6
9.94
0.247
40
26
0.25
2.71
3
10
9.60
0.248
40
26.4
0.1875
1.85
7
4
9.85
0.254
40
26.4
0.25
1.51
7
4
9.42
0.259
40
26
0.375
2.04
3
10
9.73
0.261
40
28.8
0.1875
2.84
4
8
9.81
0.263
40
27.6
0.375
1.44
5
6
9.95
0.265
40
28.8
0.25
2.37
4
8
9.98
0.272
40
26.4
0.375
1.05
7
4
10.02
0.281
40
28.8
0.375
1.76
4
8
9.19
0.283
40
30
0.1875
3.57
3
12
9.12
0.285
40
30
0.25
3.01
3
12
9.17
0.299
40
30
0.375
2.28
3
12
6.41
0.391
60
26
0.1875
3.22
3
10
5.98
0.394
60
27.6
0.1875
2.39
5
6
6.61
0.395
60
26
0.25
2.71
3
10
6.17
0.402
60
27.6
0.25
1.98
5
6
6.27
0.415
60
28.8
0.1875
2.84
4
8
6.74
0.415
60
26
0.375
2.04
3
10
6.46
0.421
60
28.8
0.25
2.37
4
8
5.64
0.422
60
26.4
0.1875
1.85
7
4
6.29
0.427
60
27.6
0.375
1.44
5
6
5.84
0.432
60
26.4
0.25
1.51
7
4
6.57
0.445
60
28.8
0.375
1.76
4
8
6.26
0.450
60
30
0.1875
3.57
3
12
6.45
0.453
60
30
0.25
3.01
3
12
6.02
0.459
60
26.4
0.375
1.05
7
4
6.27
0.472
60
30
0.375
2.28
3
12
4.87
0.555
80
26
0.1875
3.22
3
10
4.94
0.559
80
26
0.25
2.71
3
10
4.66
0.579
80
27.6
0.1875
2.39
5
6
4.41
0.582
80
27.6
0.25
1.98
5
6
5.07
0.586
80
26
0.375
2.04
3
10
4.78
0.613
80
28.8
0.25
2.37
4
8
4.54
0.616
80
27.6
0.375
1.44
5
6
5.02
0.624
80
28.8
0.1875
2.84
4
8
4.66
0.638
80
30
0.1875
3.57
3
12
4.81
0.642
80
30
0.25
3.01
3
12
4.91
0.645
80
28.8
0.375
1.76
4
8
4.94
0.670
80
30
0.375
2.28
3
12
4.45
0.675
80
26.4
0.375
1.05
7
4
4.99
0.707
80
26.4
0.1875
1.85
7
4
4.82
0.711
80
26.4
0.25
1.51
7
4
4.00
0.725
100
26
0.1875
3.22
3
10
3.95
0.727
100
26
0.25
2.71
3
10

302

Load Designation
40(L)-6(S)-0.1875(TPL)
40(L)-10(S)-0.1875(TPL)
40(L)-6(S)-0.25(TPL)
40(L)-10(S)-0.25(TPL)
40(L)-4(S)-0.1875(TPL)
40(L)-4(S)-0.25(TPL)
40(L)-10(S)-0.375(TPL)
40(L)-8(S)-0.1875(TPL)
40(L)-6(S)-0.375(TPL)
40(L)-8(S)-0.25(TPL)
40(L)-4(S)-0.375(TPL)
40(L)-8(S)-0.375(TPL)
40(L)-12(S)-0.1875(TPL)
40(L)-12(S)-0.25(TPL)
40(L)-12(S)-0.375(TPL)
60(L)-10(S)-0.1875(TPL)
60(L)-6(S)-0.1875(TPL)
60(L)-10(S)-0.25(TPL)
60(L)-6(S)-0.25(TPL)
60(L)-8(S)-0.1875(TPL)
60(L)-10(S)-0.375(TPL)
60(L)-8(S)-0.25(TPL)
60(L)-4(S)-0.1875(TPL)
60(L)-6(S)-0.375(TPL)
60(L)-4(S)-0.25(TPL)
60(L)-8(S)-0.375(TPL)
60(L)-12(S)-0.1875(TPL)
60(L)-12(S)-0.25(TPL)
60(L)-4(S)-0.375(TPL)
60(L)-12(S)-0.375(TPL)
80(L)-10(S)-0.1875(TPL)
80(L)-10(S)-0.25(TPL)
80(L)-6(S)-0.1875(TPL)
80(L)-6(S)-0.25(TPL)
80(L)-10(S)-0.375(TPL)
80(L)-8(S)-0.25(TPL)
80(L)-6(S)-0.375(TPL)
80(L)-8(S)-0.1875(TPL)
80(L)-12(S)-0.1875(TPL)
80(L)-12(S)-0.25(TPL)
80(L)-8(S)-0.375(TPL)
80(L)-12(S)-0.375(TPL)
80(L)-4(S)-0.375(TPL)
80(L)-4(S)-0.1875(TPL)
80(L)-4(S)-0.25(TPL)
100(L)-10(S)-0.1875(TPL)
100(L)-10(S)-0.25(TPL)


<table>
<thead>
<tr>
<th>Value1</th>
<th>Value2</th>
<th>Value3</th>
<th>Value4</th>
<th>Value5</th>
<th>Value6</th>
<th>Value7</th>
<th>Value8</th>
<th>Value9</th>
<th>Value10</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.06</td>
<td>0.760</td>
<td>100</td>
<td>26</td>
<td>0.375</td>
<td>2.04</td>
<td>3</td>
<td>10</td>
<td>100(L)-10(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.73</td>
<td>0.801</td>
<td>100</td>
<td>27.6</td>
<td>0.1875</td>
<td>2.39</td>
<td>5</td>
<td>6</td>
<td>100(L)-6(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.72</td>
<td>0.809</td>
<td>100</td>
<td>27.6</td>
<td>0.25</td>
<td>1.98</td>
<td>5</td>
<td>6</td>
<td>100(L)-6(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.90</td>
<td>0.811</td>
<td>100</td>
<td>28.8</td>
<td>0.1875</td>
<td>2.84</td>
<td>4</td>
<td>8</td>
<td>100(L)-8(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.84</td>
<td>0.814</td>
<td>100</td>
<td>28.8</td>
<td>0.25</td>
<td>2.37</td>
<td>4</td>
<td>8</td>
<td>100(L)-8(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.74</td>
<td>0.832</td>
<td>100</td>
<td>30</td>
<td>0.1875</td>
<td>3.57</td>
<td>3</td>
<td>12</td>
<td>100(L)-12(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.75</td>
<td>0.834</td>
<td>100</td>
<td>31.2</td>
<td>0.25</td>
<td>3.01</td>
<td>3</td>
<td>12</td>
<td>100(L)-12(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.56</td>
<td>0.840</td>
<td>100</td>
<td>27.6</td>
<td>0.375</td>
<td>1.44</td>
<td>5</td>
<td>6</td>
<td>100(L)-6(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.89</td>
<td>0.852</td>
<td>100</td>
<td>28.8</td>
<td>0.375</td>
<td>1.76</td>
<td>4</td>
<td>8</td>
<td>100(L)-8(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.86</td>
<td>0.868</td>
<td>100</td>
<td>30</td>
<td>0.375</td>
<td>2.28</td>
<td>3</td>
<td>12</td>
<td>100(L)-12(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.67</td>
<td>0.924</td>
<td>100</td>
<td>26.4</td>
<td>0.1875</td>
<td>1.85</td>
<td>7</td>
<td>4</td>
<td>100(L)-4(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.64</td>
<td>0.932</td>
<td>100</td>
<td>26.4</td>
<td>0.25</td>
<td>1.51</td>
<td>7</td>
<td>4</td>
<td>100(L)-4(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.70</td>
<td>0.958</td>
<td>100</td>
<td>26.4</td>
<td>0.375</td>
<td>1.05</td>
<td>7</td>
<td>4</td>
<td>100(L)-4(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.25</td>
<td>0.911</td>
<td>120</td>
<td>26</td>
<td>0.1875</td>
<td>3.22</td>
<td>3</td>
<td>10</td>
<td>120(L)-10(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.27</td>
<td>0.915</td>
<td>120</td>
<td>26</td>
<td>0.25</td>
<td>2.71</td>
<td>3</td>
<td>10</td>
<td>120(L)-10(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.36</td>
<td>0.944</td>
<td>120</td>
<td>26</td>
<td>0.375</td>
<td>2.04</td>
<td>3</td>
<td>10</td>
<td>120(L)-10(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>1.003</td>
<td>120</td>
<td>27.6</td>
<td>0.1875</td>
<td>2.39</td>
<td>5</td>
<td>6</td>
<td>120(L)-6(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>1.012</td>
<td>120</td>
<td>27.6</td>
<td>0.25</td>
<td>1.98</td>
<td>5</td>
<td>6</td>
<td>120(L)-6(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.15</td>
<td>1.013</td>
<td>120</td>
<td>28.8</td>
<td>0.1875</td>
<td>2.84</td>
<td>4</td>
<td>8</td>
<td>120(L)-8(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.16</td>
<td>1.019</td>
<td>120</td>
<td>28.8</td>
<td>0.25</td>
<td>2.37</td>
<td>4</td>
<td>8</td>
<td>120(L)-8(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.11</td>
<td>1.026</td>
<td>120</td>
<td>30</td>
<td>0.25</td>
<td>3.01</td>
<td>3</td>
<td>12</td>
<td>120(L)-12(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.10</td>
<td>1.035</td>
<td>120</td>
<td>30</td>
<td>0.1875</td>
<td>3.57</td>
<td>3</td>
<td>12</td>
<td>120(L)-12(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.09</td>
<td>1.035</td>
<td>120</td>
<td>27.6</td>
<td>0.375</td>
<td>1.44</td>
<td>5</td>
<td>6</td>
<td>120(L)-6(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.08</td>
<td>1.056</td>
<td>120</td>
<td>28.8</td>
<td>0.375</td>
<td>1.76</td>
<td>4</td>
<td>8</td>
<td>120(L)-8(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.21</td>
<td>1.068</td>
<td>120</td>
<td>30</td>
<td>0.375</td>
<td>2.28</td>
<td>3</td>
<td>12</td>
<td>120(L)-12(S)-0.375(TPL)</td>
<td></td>
</tr>
<tr>
<td>2.98</td>
<td>1.141</td>
<td>120</td>
<td>26.4</td>
<td>0.1875</td>
<td>1.85</td>
<td>7</td>
<td>4</td>
<td>120(L)-4(S)-0.1875(TPL)</td>
<td></td>
</tr>
<tr>
<td>3.02</td>
<td>1.155</td>
<td>120</td>
<td>26.4</td>
<td>0.25</td>
<td>1.51</td>
<td>7</td>
<td>4</td>
<td>120(L)-4(S)-0.25(TPL)</td>
<td></td>
</tr>
<tr>
<td>2.97</td>
<td>1.194</td>
<td>120</td>
<td>26.4</td>
<td>0.375</td>
<td>1.05</td>
<td>7</td>
<td>4</td>
<td>120(L)-4(S)-0.375(TPL)</td>
<td></td>
</tr>
</tbody>
</table>
Vita

Devin Harris was born on October 10, 1977 in Philadelphia, Pennsylvania to Jerome and Petra Harris. He spent his childhood in a number of locations in the United States and the Caribbean including: California, Puerto Rico, Jamaica, and Barbados. Devin attended Pine Crest Preparatory School as a boarding student in Ft. Lauderdale, Florida and graduated in 1995.

Devin enrolled as an undergraduate at the University of Florida in the fall of 1995. In June of 1999 he graduated with a Bachelor of Science degree in Civil Engineering. Upon graduation he began employment with ExxonMobil (formerly Exxon Company USA) in New Orleans, Louisiana as a drilling engineer.

After three and a half years of working for ExxonMobil, Devin enrolled at Virginia Polytechnic Institute and State University in the spring of 2003 to pursue graduate studies. He completed his M.S. studies under the guidance of Dr. Carin Roberts-Wollmann with a thesis titled Characterization of Punching Shear Capacity of Thin UHPC Plates.

Devin completed his Ph.D. studies with a dissertation titled Lateral Load Distribution and Deck Design Recommendations for the Sandwich Plate System (SPS) in Bridge Applications during the Fall of 2007 under the guidance of Dr. Tommy Cousins and Dr. Thomas M. Murray.