DESTRUCTIVE TESTING OF COMPOSITE PRECAST CONCRETE DECK PANELS
AND BUILT-UP STEEL PLATE GIRDERs

by

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ABSTRACT

Destructive Testing of Composite Precast Concrete Deck Panels and Built-Up Steel Plate Girders

by

Wesley J. Cook, Master of Science
Utah State University, 2010

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Department: Civil and Environmental Engineering

The Utah Department of Transportation (UDOT) has implemented the use of precast concrete panels for bridge deck construction. A bridge utilizing these panels as a reconstruction method was decommissioned three years after the new deck installation, due to unrelated matters. Two sections of this bridge were salvaged and sent to Utah State University (USU) for destructive testing.

Each bridge section consisted of two built-up steel plate girders intact with the precast concrete deck panels. The precast panels were designed and constructed to achieve full composite action between the deck and built-up steel plate girders through the use of Nelson shear studs. Additionally, the precast panels span the transverse direction and as such have a transverse joint. Historic data has shown the transverse joint to be an area of concern for the functionality of the structural system.

Flexure, beam shear, and punching shear of the deck ultimate capacities were compared to those calculated in accordance to the AASTHO Load and Resistance Factor
Design (LRFD) Bridge Design Specifications. Various experimental tests considered the affects of the transverse joint on the elastic and plastic capacities and code adherence. Nine destructive tests were performed. The Nelson shear studs were found to be capable of achieving the ultimate capacities of all three types of performed tests and therefore a significant level composite action was attained throughout the experimental tests. The transverse joints show a slight decrease in flexural elastic capacity, no measurable influence on flexural plastic capacity and beam shear ultimate capacity, and a 40% decrease to ultimate punching shear capacity to the deck compared to punching shear capacity without a transverse joint.

(107 pages)
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Special thanks to Utah Transportation Center for funding, providing bridge sections, a testing facility, and making this large-scale destructive testing possible. I’d like to thank Kiewit for the tedious challenge of working with researchers in the removal and transportation process of the bridge sections.

This was a massive undertaking and could not have been done without the help of Travis Brackus in pushing testing forward and as a fellow researcher. Zane Wells needs to be thanked for all his assistance and willingness to do even the most arduous tasks at any hour.

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CONTENTS

Page

ABSTRACT

iii

ACKNOWLEDGMENTS

v

LIST OF TABLES

viii

LIST OF FIGURES

ix

CHAPTER

I. INTRODUCTION

1

II. LITERATURE REVIEW

5

III. LABORATORY TESTING

12

Description of Bridge

12

Specimen Description and Collection

15

Testing Facilities

18

Tests Performed

19

Flexure Tests

19

Flexural Setup

20

Testing Description

23

Experimental Bridge Section Condition

27

Flexure Experimental Results

30

Testing Damage and Failure Description

40

Beam Shear Tests

45

Beam Shear Setup

46

Beam Shear Experimental Results

49

Punching Shear Tests

53

Punching Shear Setup

54

Punching Shear Experimental Results

55

IV. BRIDGE SPECIFICATIONS

59

Flexural Capacity

59

Flexural Factored Loads

61

Beam Shear Capacity

64

Beam Shear Factored Loading

66

Punching Shear Capacity

67

Punching Shear Factored Loads

68

V. COMPARISON OF EXPERIMENTS AND BRIDGE SPECIFICATIONS

69
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BDI theoretical and measured neutral axes locations</td>
<td>29</td>
</tr>
<tr>
<td>2</td>
<td>Theoretical and measured neutral axes for initial 178 kN flexural test</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>Flexural tests ultimate capacities</td>
<td>31</td>
</tr>
<tr>
<td>4</td>
<td>Flexural test capacities</td>
<td>37</td>
</tr>
<tr>
<td>5</td>
<td>Shear test ultimate capacities</td>
<td>53</td>
</tr>
<tr>
<td>6</td>
<td>Ultimate punching shear capacities</td>
<td>58</td>
</tr>
<tr>
<td>7</td>
<td>Theoretical elastic and plastic equivalent point loads</td>
<td>70</td>
</tr>
<tr>
<td>8</td>
<td>Deflections: at equivalent tandem loads and loads at maximum allowable</td>
<td>72</td>
</tr>
<tr>
<td>9</td>
<td>Moment comparison of flexural experimental and theoretical capacities</td>
<td>72</td>
</tr>
<tr>
<td>10</td>
<td>Comparison of beam shear experimental and theoretical capacities</td>
<td>74</td>
</tr>
<tr>
<td>11</td>
<td>Comparison of punching shear experimental and theoretical capacities</td>
<td>78</td>
</tr>
</tbody>
</table>
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I-15 and 800 North bridge aerial</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>Steel plate girder as-built drawing</td>
<td>13</td>
</tr>
<tr>
<td>3</td>
<td>Nelson shear studs detail</td>
<td>14</td>
</tr>
<tr>
<td>4</td>
<td>Transverse joint connection detail</td>
<td>14</td>
</tr>
<tr>
<td>5</td>
<td>Bridge cross-section depiction (cm)</td>
<td>16</td>
</tr>
<tr>
<td>6</td>
<td>Salvaged bridge specimens</td>
<td>17</td>
</tr>
<tr>
<td>7</td>
<td>I-15 and 800 North bridge specimen 1 removal</td>
<td>18</td>
</tr>
<tr>
<td>8</td>
<td>Flexural testing applied point load locations</td>
<td>20</td>
</tr>
<tr>
<td>9</td>
<td>General testing setup</td>
<td>21</td>
</tr>
<tr>
<td>10</td>
<td>(left) Pin reaction and (right) roller reaction</td>
<td>22</td>
</tr>
<tr>
<td>11</td>
<td>(left) Ram setup, (middle) potentiometer setup, and (right) tiltmeter setup.</td>
<td>22</td>
</tr>
<tr>
<td>12</td>
<td>Girder D flexural instrumentation schematic</td>
<td>23</td>
</tr>
<tr>
<td>13</td>
<td>Girder D strain gages setup (cm)</td>
<td>24</td>
</tr>
<tr>
<td>14</td>
<td>Girder E flexural instrumentation schematic</td>
<td>25</td>
</tr>
<tr>
<td>15</td>
<td>Girder E strain gage setup (cm)</td>
<td>26</td>
</tr>
<tr>
<td>16</td>
<td>Girder C strain gage setup (cm)</td>
<td>27</td>
</tr>
<tr>
<td>17</td>
<td>Girder C flexural instrumentation schematic</td>
<td>27</td>
</tr>
<tr>
<td>18</td>
<td>Flexural test Girder E reaction forces versus applied load.</td>
<td>31</td>
</tr>
<tr>
<td>19</td>
<td>Flexural test girder C reaction forces versus applied load.</td>
<td>32</td>
</tr>
<tr>
<td>20</td>
<td>Plane sections remain plane for flexure test Girder C, typical.</td>
<td>33</td>
</tr>
<tr>
<td>21</td>
<td>Neutral axis location versus load plot for flexural tests.</td>
<td>34</td>
</tr>
</tbody>
</table>
Flexural test Girder D neutral axis location across length of girder. .......................... 34
Flexural test Girder E neutral axis location across length of girder. .......................... 35
Flexural test Girder C neutral axis location across length of girder. .......................... 35
Diaphragm strain gages. ................................................................................................. 37
Flexural test Girder E diaphragm strain versus load.................................................. 38
Post ultimate load typical shear pocket failure. ............................................................ 39
Load versus vertical deflection plot............................................................................... 40
Flexural test Girder C load versus transverse or lateral deflection.............................. 40
Post 1421 kN (320 kips) load damage.......................................................................... 42
Post ultimate load Girder D flexure test. ..................................................................... 43
Post ultimate load Girder E flexure test. ..................................................................... 43
Flexural test Girder C. ................................................................................................. 44
Beam shear test loading location profile....................................................................... 45
Beam shear loading locations. ...................................................................................... 46
Strain gage locations for Girder B beam shear (cm). .................................................. 47
Strain gage locations for Girder C beam shear (cm). .................................................. 48
Post ultimate load Girder B beam shear. ..................................................................... 50
Beam shear strain gages plotted below: (left) Girder B, (right) Girder C. ............... 50
Beam shear Girder B tension strain a series. ............................................................... 50
Beam shear Girder C compression strain aD1 series 1,3, and 5. ............................... 51
Post ultimate load Girder C beam shear. ..................................................................... 52
Beam shear vertical deflections under point load....................................................... 53
Punching shear loading locations. .............................................................................. 54
<table>
<thead>
<tr>
<th>Page</th>
<th>Topic</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>Post ultimate load Punching Shear 1</td>
</tr>
<tr>
<td>46</td>
<td>Post ultimate load Punching Shear 2</td>
</tr>
<tr>
<td>47</td>
<td>Post ultimate load Punching Shear 3</td>
</tr>
<tr>
<td>48</td>
<td>Post ultimate Load Punching Shear 4</td>
</tr>
<tr>
<td>49</td>
<td>Theoretical punching shear failure</td>
</tr>
<tr>
<td>50</td>
<td>Stress versus strain plot for coupon steel test</td>
</tr>
<tr>
<td>51</td>
<td>Flexural Girder C strain gages plots 138 series</td>
</tr>
<tr>
<td>52</td>
<td>Flexural Girder C strain gages plots 229 series</td>
</tr>
<tr>
<td>53</td>
<td>Flexural Girder C strain gages plots 320 series</td>
</tr>
<tr>
<td>54</td>
<td>Flexural Girder C strain gages plots 411 series</td>
</tr>
<tr>
<td>55</td>
<td>Flexural Girder C strain gages plots 502 series</td>
</tr>
<tr>
<td>56</td>
<td>Flexural Girder C strain gages plots 592 series</td>
</tr>
<tr>
<td>57</td>
<td>Flexural Girder C strain gages plots 683 series</td>
</tr>
<tr>
<td>58</td>
<td>Beam shear Girder B displacements</td>
</tr>
<tr>
<td>59</td>
<td>Beam shear Girder C displacements</td>
</tr>
<tr>
<td>60</td>
<td>Beam shear Girder B strain gages plots A series</td>
</tr>
<tr>
<td>61</td>
<td>Beam shear Girder C strain gages plots AT series</td>
</tr>
<tr>
<td>62</td>
<td>Beam shear Girder C strain gages plots AB series</td>
</tr>
<tr>
<td>63</td>
<td>Beam shear Girder C strain gages plots aD1 Series</td>
</tr>
<tr>
<td>64</td>
<td>Beam shear Girder C strain gages plots aD2 series</td>
</tr>
<tr>
<td>65</td>
<td>Calculation of ENA</td>
</tr>
<tr>
<td>66</td>
<td>Calculation of PNA (AASHTO, 2009)</td>
</tr>
<tr>
<td>67</td>
<td>Testing sequence</td>
</tr>
</tbody>
</table>
CHAPTER I
INTRODUCTION

Utah Department of Transportation (UDOT) has adopted an Accelerated Bridge Construction (ABC) program, which incorporates the use of precast concrete deck panels for rapid bridge deck construction of new and rehabilitation bridge deck installations. Precast concrete panels can significantly reduce construction time in comparison to alternate methods, and by achieving composite action of the deck and superstructure, provide increased structural integrity. Precast panels allow for offsite pouring and curing of concrete bridge deck sections and subsequently the panels are hauled to the bridge site and secured in-place. Deck erection can be completed in hours and the bridge can be placed in service in days, rather than potentially weeks for conventional cast-in-place concrete decks. This construction method is ideal to prevent extended road closures, especially in areas of high traffic volumes.

In addition to the expedited construction, another advantage of precast concrete panels is that they can be designed for composite action between the concrete deck and steel girder superstructure. Composite action is achieved at the concrete deck and steel girder interface, typically through the use of Nelson shear studs. Given sufficient shear studs, optimization of each construction material occurs as the concrete resists compression and the steel resists tension consistent with Euler-Bernoulli beam theory flexural forces. Furthermore, composite action theoretically shifts the elastic neutral axis higher in the steel member, increases the moment of inertia, decreases deflections, and/or results in a decreased steel section sizes for a potential cost savings. In post-yield or
plastification of a composite member, the ultimate load capacity significantly increases and can be utilized as an overstrength of the structural system.

Precast concrete deck panels have been periodically utilized nationwide but with numerous variations in the panel connections. UDOT is cognizant of advantages and disadvantages of the common post-tensioning transverse joint connection. The primary disadvantage is if a singular precast panel in the deck is faulty, damaged, or deteriorated, the concrete panels across an entire post-tensioned deck section must be replaced. Sullivan (2003) provides a conclusive summary of the structural advantages of post-tensioning: 1) provides a tight connection to prevent water and deicing salts from penetrating steel in the connection, 2) prevents transverse cracking due to tensile forces in the concrete, especially at transverse joints, and 3) assists in transferring load across deck panels. The ABC program for this project utilizes an alternative method for panel-to-panel connections to avoid the necessity of replacing an entire post-tensioned strip of the deck in the event of damage. The standard UDOT precast panel connection imbeds studded plates on the bottom of a female-female joint at discrete intervals where the panels butt-up, a prepared welding plate is welded to the female-female joint plates forming an inverted “A” shape connection cross-section. This joint as well as many others have been scrutinized and are an area of continued research, here at Utah State (Porter, 2009; Roberts, 2011).

UDOT provided two bridge sections from an exterior span (span 1) of an underpass of I-15 and 800 North in Salt Lake City. Each bridge section is composed of two built-up steel plate girders and precast concrete deck panels. The deck and
superstructure are connected with Nelson shear studs and were designed to achieve full composite action. The standard UDOT transverse female-to-female welded plate joints were utilized in the deck replacement in 2007. The superstructure dates to 1964 with the original underpass construction. These two isolated bridge sections were removed during demolition of the bridge and hauled to the Utah State University (USU) System Materials and Structural Health (SMASH) lab for testing. UDOT’s primary interests for this research project are; how have the panel connections performed in service? And what effects do the transverse joints have on the bridge flexure, beam shear, and punching shear capacities?

To address UDOT’s concerns, a total of nine destructive testing was performed on the bridge sections in flexure of the composite section, beam shear of the composite section, punching shear of the deck near and away from the transverse joints. During each test strains, load, reactions, deflections, and rotations, were monitored. This data was used to make comparisons to the current LRFD Bridge Design Specifications manual, American Institute of Steel Construction (AISC) design manual, and UDOT design standards for adherence and recommendations for improvements. In a related study (Brackus, 2010) compared the destructive test data to Finite-Element Modeling and also included a section on changes in modal frequencies for the incremental flexure test through failure.

From the flexural results the experimental elastic capacity was 10% more conservative than the LRFD calculated theoretical capacity. When experimentally tested over a transverse joint the elastic capacity was 3% unconservative compared to the
theoretical calculated capacity, plastic capacity are conservative by about 30% regardless of the transverse joint. Beam shear results show the LRFD Specifications were conservative by 0.7% on one test and 8% unconservative on another; the transverse joint had no measurable influence. Punching shear results show the LRFD Specifications were conservative for tests 2 and 4 by 43.8% and 89.3%, respectively, and tests 1 and 3 conducted over transverse joints were unconservative by 13.4% and 5.7%, respectively.
CHAPTER II

LITERATURE REVIEW

Composite beams were patented in 1926 by J. Kahn, according to (Viest, Fountian, and Singleton, 1958), but the full advantage of composite concrete steel girders was not incorporated into bridge design codes until AASHTO 1944. During the 1950s Ivan M. Viest performed numerous tests on composite girders with a compilation of design consideration into a text (Viest, Fountian, and Singleton, 1958). A generalized design description, is the transformation of sections to calculate the elastic neutral axis and composite material resistance to loads and deflections. In the 1960's several notable advancements were achieved in understanding composite behavior. Chapman (1965) observed, once moments exceeded elastic stresses in steel, yielding would occur and the neutral axis moves upward to a point, known as the plastic neutral axis. The fully plastic neutral axis could be attained contingent on sufficient number of shear studs. Thus, shear studs design was purposed to be formulated to resist shear flow and to ensure a ductile failure rather than a sudden catastrophic shear failure in slip. Ultimate failure of the composite system occurred from either insufficient shear studs or crushing of the concrete slab. Chapman and Balakrishnan (1965) tested seventeen composite girders and observed the slip on the concrete-steel shear connection interface. Point loads were used to maximize slip. Experiments showed if no slip was present the sections acted fully composite, as slip occurred at the steel-concrete interface composite action deteriorated. Documentation was also shown of the stresses on the composite section demonstrating the movement of the neutral axis from elastic to fully plastic.
The next major leap for composite sections was the expanded use of steel girders and prefabricated concrete deck panels in the 1970's. New bridges and bridge rehabilitation projects replaced deteriorated decks utilizing full depth prefabricated concrete panels in composite action. Issa et al. (1995a) conducted a survey sent to 50 plus transportation organizations. Results from the survey showed agencies which implemented full depth prefabricated concrete panels significantly reduced on-site construction time. Assortments of construction methods were used for installation and joining prefabricated panels. The transverse joint is of particular interest, it was speculated post-tensioning provided tightness and maintained compression in the transverse joints, Issa conducted several studies on aspects of precast concrete panels.

Issa et al. (1995b) followed up the transportation survey with targeted visual field inspections of full depth precast concrete decks to evaluate the performance of the various construction practices. It was observed that transverse joints preformed better with the addition of post-tensioning in the longitudinally direction. Without post-tensioning and depending on traffic volumes joints cracked, leaked causing corrosion of superstructure elements, spalled, and showed evidence of joint failure. Other conclusions and recommendations from the field inspections were; precast panels are an economical deck replacement alternative, haunches mitigate dimensional irregularity issues, and overlays protected precast panels and provided a smoother ride.

Issa et al. (1998) considered the effectiveness of the female-female grouted transverse joint with and without post-tensioning subjected to positive and negative moments, of full depth precast concrete bridge deck panels using finite element modeling
(FEM). Two models were utilized; first of a simply supported three girder composite action precast deck, the second was a three span continuous deck again using precast panels in fully composite action. Both models were analyzed with and without post-tensioning. Modeling demonstrated the presumption of tensile forces in the transverse joints when post-tensioning was not employed. Tensile forces in the transverse joint were more predominant at the midspan.

The thesis work of Sullivan (2003) examined transverse joints commonly used by DOTs across the nation under wheel loading and temperature gradients. The modes of failure when post-tensioning is not utilized are described in detail for male-female, female-female, and dapped end transverse joints. The three main modes of failure are rotational cracks, crushing of the grout or shear key, and dislocation at the panel interface. FEM showed the tensile cracks on both the top and bottom of the joints; further crack sizes were theoretically calculated. Tensile cracking below dap, flat, and male-female transverse joints were figured to be above the ACI limit of 0.013 inches.

Porter (2009) experimentally tested typical female-female shear keyway transverse joints employed by UDOT. Joints experimentally tested were post-tensioned, welded stud with differing configurations, and unreinforced. Joints were loaded to failure using a push off method, direct downward force over the joint. Results of the ultimate shear capacity were relative to the post-tension specimen (100%), continuous welded stud was 87%, welded stud 6” with 18” spacing and welded stud 6” with 24” spacing decreased to 73% and 44%, respectively. The unreinforced joint failed at 30%.
The transverse joint was described by Sullivan as the “weak link” in the composite precast concrete deck. However, if sufficient capacity can be attained in the transverse joint the shear studs become the next likely point of failure in the precast panel composite system. Lam and El-Lobedy (2005) depict the shear stud failure by experimental tests comparing to FEMs. Four concrete strengths were targeted; 7000, 5000, 4500, and 3000 psi, in push off tests, with 3/4” stud shanks and 4” height. The failure mode of high strength concrete was shearing off of the stud at the weld, the moderately high strength concrete had a combined failure of semi-conical concrete tension and stud yielding with a tendency towards the stud shearing, the moderate concrete strength again had a combined failure but more towards the concrete failure, last the low concrete strength failed in a concrete conical failure around the stud. Finite element analysis (FEA) was able to predict the modes of failure and compared “well” with the experimental data.

Chapman (1965) noted experimental results of shear studs on composite concrete slabs and steel girders. Vertical separation of the slab from the steel girder occurred nearest the quarter-span, which Liang et al. (2005) derived theoretical equations for forces and resistance requirements using FEM. Horizontal shear was closely examined by Issa et al. (2003) in which four sets of shear equations (Viest, 1956; Slutter and Driscoll, 1965; AASHTO, 1994; and AASHTO, 1995) where compared against each other, results show AASHTO LRFD being more conservative. Chapman concluded shear connectors could be placed in a gradient (more dense at the ends) or uniformly across the span in simply supported systems contingent on the total number of studs.
Shear connectors should be sufficient for ultimate load to ensure not to have a sudden diminution in carrying capacity and catastrophic failure.

If sufficient shear connection is provided at the steel concrete interface, a probable mode of failure is slender plate, out of plane buckling, creating a post-buckling tension fields in steel girders. Slender plate post-buckling is a long researched topic for which no complete explanation exists (Yoo and Lee, 2006). Yoo provides an in depth history of the subject to disprove incorrect assumptions. Using non-linear finite element modeling techniques, not available to former researchers, analysis was performed on four node quadrilateral shell elements. The study depicts the tensions/compression field in the prebuckling stage and ultimate load refuting past assumptions of stresses on the buckled plate. Results of the FEM were compared to (AASHTO, 2004) code. The code proved to be adequate in shear strength despite adaptation of misconceptions in slender plate buckling theory.

Baskar and Shanmugam (2003) experimentally tested composite concrete slabs-steel girders in combined bending and shear. Six composite plate girders were designed with the specific intention to fail after post-buckling of the plate girder occurred, mitigating the possibility of failure of other modes; lateral torsional buckling, local web buckling, shear connection slip. Horizontal and vertical web stiffeners were used to create a designated failure region in the steel plate girder. Depth to thickness ratio of each member was chosen to compare the effectiveness of plate girders web slenderness in tension field action and ultimately analyzed under maximum load. From Evans, Porter, and Rockey (1978) the tension field band increased at the bottom flange when subjected
to combined bending and shear. Baskar conferred the width of the tension field increases with a composite concrete deck thus increasing the post-buckling capacity of the steel member; additionally the composite action was more effective when the web was slender. The ultimate mode of failure was a sudden catastrophic full depth transverse crack in the concrete slab which is never a design practice.

Bechtel (2008) investigated the system capacity, scaled models, and simply supported skewed bridges in a sequence of tests leading to a full scale in situ destructive bridge test. In the preparation for the full scale test, Bechtel thesis and related studies provides ample information on skewed bridges and previous destructive tests. Skewed bridges have been shown to collect vertical loads at obtuse corners and are preferred for analyzing ultimate system capacity of multi-girder bridges.

Issa et al. (2003) experimentally tested a full scale two span three steel girder composite bridge system using prefabricated concrete deck panels. The test specimen consisted of precast concrete deck was anchored to the top flange of the steel girders (standard AISC W18x86) through the use of headed shear studs at a design interval, shear stud pockets and haunches where grouted, post-tensioning was utilized longitudinally. The system was loaded to three predetermined load sets in both positive and negative moments; Case 1 was for the (AASHTO, 1994) service load, Case 2 was a 2x service load as an overload capacity, and Case 3 loaded to 95% ultimate capacity based on the FEM. The testing results were compared to the FEM and the performance to the AASHTO Bridge Code. The FEM was an accurate depiction of the stresses induced in lower load sets, because the composite system did not fail at 95% ultimate capacity the
model again was consistent with the field tests. The composite bridge system was within the deflection criterion of the AASHTO code.
CHAPTER III
LABORATORY TESTING

Description of Bridge

The bridge sections were taken from Span 1 of 4 from the I-15 and 800 North bridge underpass outside of downtown Salt Lake City, see Figure 1 (Google Maps, 2009). The bridge was originally constructed in 1964 and was composed of four spans 10.9 m (35'-9'"), 21.3 m (69'-10'"), 21.3 (69'-10'"), and 10.9 m (35'-9'""). Each span superstructure was composed of six built-up steel plate girders at a 2.41 m (7'-10-3/4'"") spacing and were made of 248 MPa (36 ksi) steel. The built-up steel plate girders were constructed with 25.4 cm x 1.6 cm (10" x 5/8"") plates for the top and bottom flanges and 96.5 cm x 1.0 cm (38" x 3/8"") plates for the webs, welded plate stiffeners for diaphragms were constructed with two full depth, plates both sides of girder 11.4 cm x 1.3 cm (4-1/2"").

Figure 1  I-15 and 800 North bridge aerial.
x 1/2") five intermediate stiffeners between diaphragms are 11.4 cm x 0.8 cm (4-1/2" x 5/16") plates on only one side of girder full depth, and C-channel C15x33.9 diaphragms at ends and mid-span were bolt connected stiffeners, see Figure 2. Span 1 had a skew angle of 19°36'59". Span 1 also was super-elevated having Girder F, the south most girder, being 21.6 cm (8-1/2") lower than Girder A the north most girder. Nelson shear studs, 2.2 cm (7/8") diameter by 15 cm (6") length, were welded in sets of 3 at 46 cm (18") intervals across the top flanges of each girder connecting the girders and precast concrete deck panels, see Figure 3.

Figure 2  Steel plate girder as-built drawing.
Figure 3  Nelson shear studs detail.

Figure 4  Transverse joint connection detail.
The concrete deck was replaced in 2007 with an accelerated bridge construction (ABC) technique, using a precast concrete panel system. The precast concrete panels extended from parapet to parapet in the transverse direction, Span 1 contained 5 panels, 4 of which measured 2.41 m (7'-10-3/4") the fifth panel as a filler panel of 1.42 m (4'-8"). All panels were reinforced with two mats of #19 bar at 15 cm (#6 bar at 6") O.C. E.W. top and bottom. The specified minimum deck concrete strength was 27.6 MPa (4 ksi); subsequent coring samples from the deck showed the in situ concrete to have an average compressive strength of 57.2 MPa (8.3 ksi). The concrete panels were connected together using female-female transverse joints. The transverse joints consisted of 1.3 cm x 7.6 cm x 15.2 cm (½" x 3"x 6") with 2 welded shear studs 1.3 cm (½") diameter 10.2 cm (4") length embedded in the concrete at the bottom of the joint at 1.2 m (4') intervals and where panels butt-up a third plate 17.8 cm x 0.64 cm (7" x 1/4") is welded on both sides full 15.2 cm (6") length and 1.11 cm (7/16") slot weld, see Figure 4. The cross-section of the joint has an inverted “A” shape. The transverse joint was filled with non-shrink grout with the shear pockets and haunch once the panels were hauled and leveled in-place. A minimum 5.7 cm (2-1/4") asphalt overlay was specified, actual depths ranged between 5.7 cm (2-1/4") and 7.6 cm (3").

Specimen Description and Collection

Approximately three year after the 8th North and I-15 bridge deck replacement, UDOT contracted to expand the I-15 corridor north out of downtown Salt Lake City, as a result this bridge was scheduled for demolition. In connection with the demolition,
UDOT contractually required a certain number of salvaged structural members to be shipped to USU for testing. The two specimens collected comprised parts of Span 1 (Figure 6). Figure 7 shows the removal of Specimen 1. Figure 6 depicts the girder labeling A through F, the abutment end as End 1, the bent end as End 2, and the transverse joints 1 through 4. Reactions are given the nomenclature of B1, B2, C1, C2, D1, D2, E1 and E2 represented as girder letter and end number. This labeling system was used consistently herein and in the related study. The bridge sections were cut to a 5 m (10'-6") width to alleviate transportation issues. The deck overhang for each girder was approximately 38 cm (1'-3") Figure 5 depicts the cross-section of the bridge sections. Lifting slots were cut through the deck in four locations adjacent to the girders. Minor damage occurred to the underside of deck and diaphragm stiffeners as a result of incomplete severing from the bridge sections. The weights for the bridge sections were 300 kN (67,000 lbs) and 290 kN (65,000 lbs) for Specimens 1 and 2, respectively.

Figure 5  Bridge cross-section depiction (cm).
Figure 6  Salvaged bridge specimens.
All tests were performed in the USU SMASH lab with the necessary appurtenances for larger scale destructive tests, including flexure, beam shear and punching shear test. A brief overview of this facility is given herein for a scope and magnitude of the facilities required for full scale destructive bridge testing. The SMASH lab contains a strong floor area which is heavily reinforced concrete, 0.9 m (3 ft.) thick, with a crawl space underneath and a grid of conduit slots for anchorage of the reaction frame. The reaction frame is composed of two W12x290 columns and a beam W36x395
spanning the columns, the capacity of the reaction frame is above the largest hydraulic ram in the facility at 5.3 MN (1,200 kips).

**Tests Performed**

Three types of destructive testing were identified with the limited number of specimens; three flexural, two beam shear, and four punching shear. The primary of objectives of the research were to demonstrate the degree of composite action of the precast concrete panels and the built-up steel plate girders, performance of the UDOT designed welded transverse joint, and adherence to the LRFD Bridge Specifications. Selected tests were conducted directly over transverse joints, one for flexure, one for beam shear, and two for punching shear, for comparison of tests away from joints.

**Flexure Tests**

Three destructive flexural tests were performed, two on bridge section 1 and one on bridge section 2. The first of these was an incremental static load at the mid-span of Girder D, the second was a monotonic static load at the mid-span of Girder E, and the third was a monotonic static load over a transverse joint closest to the mid-span on Girder C, see Figure 8 for a depiction of the flexural test locations. Dynamic tests were conducted on the incremental flexure test in-between loads, description of the test and conclusions are in a correlated study by Brackus (2010).
Figure 8  Flexural testing applied point load locations.

Flexural Setup

The bridge section was positioned on the strong floor in a manner to allow the reaction frame to be placed over the anticipated point of applied load, see Figure 9 for a general testing setup. Reactions were constructed out of steel plates, load cells, spherical bearings, and a topping plate to account for the dissimilar sizes of the spherical bearing
and bridge bearing plate, see Figure 10. Steel plates of 5.0 cm (2") minimum thickness were used as base plates and the applied load anytime dissimilar materials were stacked to avoid stress concentrations on the load cells. The spherical bearings allowed the bridge bearings to rotate, as a pin or roller reaction, with the load cells placed underneath to monitor a purely vertical load. The super-elevation of the bridge sections from the field was reproduced in the lab through the use of steel plates. The asphalt overlay was removed for all but the first flexure test under the point load. A load cell and spherical bearing with required plates, see Figure 10, were placed to monitor the applied load and the spherical bearing maintained the vertical alignment. The ram operated through a hydraulic pump with manual controls of increased or decreased pressure.

Figure 9  General testing setup.
Potentiometers were used to measure changes in displacement at the quarter points and mid points of the flexural tests. Displacements and rotation data was collected to calibrate Finite-Element Models of the bridge sections, the modeling is presented in the correlated study by Brackus (2010). Potentiometers were placed inside HSS steel sections for protection against falling debris, chains and hooks enabled the sensors to be placed on the ground. The tiltmeters used to measure rotations, were bolted to the web stiffeners of Girders D and E above reactions. Locations and items monitoring varied with succeeding tests for both tiltmeters and potentiometers.
Strain gages were applied liberally with the intent to monitor the neutral axis location of the bridge section on individual girders and determine the extent of steel plastification for post-yield loads. Collection of strain data on the surface of the concrete was predicted to yield poor results, as such, strain gages were exclusively applied to the steel. Strain gages were applied in vertical and horizontal arrays on each beam with the locations varying on succeeding tests.

Testing Description

Bridge section 1 Girder D, incremental static flexure

The first flexure test on specimen 1 Girder D, had a point load applied directly over the mid-span of Girder D, see Figure 12 for a schematic of the instrumentation. The bridge section was setup on four reactions to simulate the bridge in the field, including

![Schematic diagram of Girder D flexural instrumentation](image)

Figure 12  Girder D flexural instrumentation schematic.
the super-elevation. Each reaction contained base plates, a load cell, a spherical bearing, and a topping plate. Potentiometers were used to measure changes in displacements at the quarter points and mid-points. Tiltmeters were anchored to D2 and E2. Strain gages were applied in vertical and horizontal arrays, see Figure 13 for a depiction of the Girders D strain gage placements.

The first loading increment was 178 kN (40 kips) then 356 kN (80 kips) in anticipation of quickly exiting the elastic range of materials. Succeeding tests applied additional load in 89 kN (20 kip) increments in attempt to capture changes in the modal frequencies with the onset of damage and plastification of the steel, without being onerous on the number of test needed. The load cell under the ram was monitored throughout the tests to observe when the predetermined load was achieved, once achieved the load was held constant for a 10 second period then released. A dynamic test or shacking of the bridge section was performed in-between each load increment. This cycle continued up through the ultimate capacity of Girder D and subsequent dynamic test.

Figure 13  Girder D strain gages setup (cm).
Bridge section 1 Girder E, monotonic static flexure

For Girder E the instrumentation was nearly identical to Girder D, with exceptions of the strain gage locations, see Figure 14 for an instrumentation schematic. The strain gage locations for Girder E are shown in Figure 15. The strain gage locations were altered from the first setup to account for flexural panel action in which tension and compression bands formed within the steel panel section, similar to tension field action, as evident in the diagonal paint peeling off the panels. In a vertical alignment of strain gages, this behavior of banding created equal strains in two functioning gages vertically separated on the same panel, and inhibited locating the neutral axis. To account for the steel panel banding effect in the second flexure test Girder E, strain gages were placed on the top on bottom flanges in attempt to monitor the neutral axis. In addition to changes in the strain gages locations, no dynamic tests were performed, thus the loading could be applied monotonically.

Figure 14 Girder E flexural instrumentation schematic.
Figure 15  Girder E strain gage setup (cm).

Bridge section 2 Girder C, monotonic static flexure

For flexural test Girder C several boundary conditions and instrumentation alterations were included with this monotonic test, see Figure 17 for an instrumentation schematic. It was noted that during the first two flexure tests, the girders had experienced some degree of lateral deflection, and it was certain the diaphragm transferred load based on the strain gage reading from both flexure tests, therefore the diaphragm at mid-span was removed in attempt to observe the transfer of load from the loaded girder to the unloaded girder. The bearing setup on B1 and C1 were changed to an actual roller, by placing each base plate on five solid bar steel batons, see Figure 10. From previous flexure tests the girder under load elongated axially, with only the friction of the bridge reaction on topping plate to resisting this force, the reactions rotated and slid nearly toppling on Girder E flexure test. A tiltmeter was placed at the mid-span, with the two potentiometers, to measure the rotation in the transverse direction. The second tiltmeter was situated on the stiffener of reaction C2. The strain gage locations are depicted in Figure 16. Gages were place in vertical and horizontal arrays. The horizontal strain gage arrays were used in attempt to map plasticity and the vertical arrays of three or four
provide the neutral axis location and support interpolation of points for plasticity mapping. Gages on the bottom of flange monitored the extreme steel strain. Again, the gages on the panel were limited due to the tension and compression banding inside the panels observed in flexure test Girder D.

**Experimental Bridge Section Condition**

There were essential three alternatives for full scale destructive bridge tests, first in situ testing, second, constructing full scale sections, and third, removal and
transporting of the large bridge sections to a laboratory off site. In situ testing is often impeded by very stringent restrictions on availability, time, and environmental impacts; as a result very few full scale destructive tests have been conducted. Constructing full scale bridge sections is often too onerous cost wise, it is rarely done. Bridge section removal and hauling also carries a predominate disadvantage of damage incurred in the removal and relocating process. With these destructive tests being the third case, it is necessary to discuss the damage sustain to the bridge sections of this project in the removal of the sections from service to the experimental test results.

In June 2009, four months prior to the bridge demolition in situ live load tests were conducted on this bridge in a joint venture of USU and Bridge Diagnostics, Inc. (2010). Based on the data collected, strain measurements of Girders B, C, D, and E on span 1 in three locations were used to determine the elastic neutral axis depth in the cross-section of each girder. The depth of the elastic neutral axis, a key indicator of composite action, showed the concrete precast panels to be acting fully composite with the built-up steel plate girders. The fully composite action was observed unanimously across the entire bridge with the exception of Girders B at a distance of 132 cm (4' 4") from End 1, acting completely non-composite. The location of the theoretical elastic neutral axis based on transformation of sections and parallel axis theorem reported by DBI was 98.6 cm (38.8") from the bottom of the cross-section for interior girders of Span 1. Extracting the elastic neutral axis from span 1 in the live load testing data, the in situ neutral axis ranged between 87.4 cm to 97.5 cm (34.4" to 38.4"), or 12.8% to 1.0% compared to the theoretical, see Table 1. Therefore, the precast concrete panels were
Table 1  BDI theoretical and measured neutral axes locations

<table>
<thead>
<tr>
<th>Theoretical ENA (cm)</th>
<th>Highest Live Load ENA (cm)</th>
<th>Lowest Live Load ENA (cm)</th>
<th>Difference Percent</th>
</tr>
</thead>
</table>

judged to be acting compositely with the built-up steel plate girders in the field prior to demolition.

For an initial analysis, only the first loading of the first incremental static flexural was considered for the experimental neutral axis location. Across the length of the girder (measured in five locations) the experimental neutral axis ranged between 62.5 cm to 69.6 cm (24.6” to 27.4”) measured from the bottom of the cross-section. The girder acting alone has a theoretical neutral axis of 49.2 cm (19.4”). The theoretical elastic neutral axis of the composite section changed due to the severing of the concrete deck section during removal from the site and differing tributary width. The laboratory theoretical composite elastic neutral axis, based on measured material properties, was 96.3 cm (37.9”), see Appendix B for calculations. The 178 kN (40 kips) loading experimental elastic neutral was 54% to 38% of the theoretical, see Table 2 for the neutral axis locations. Though the percentages are arbitrary, they allow for comparison of the BDI live load test and SMASH II lab measured neutral axes. From the comparison the precast concrete panels were acting partially composite with the built-up steel plate girders at the beginning of the experimental testing. The loss of fully composite action can be attributed to the removal and hauling of the bridge sections.
Table 2  Theoretical and measured neutral axes for initial 178 kN flexural test

<table>
<thead>
<tr>
<th>Theoretical Steel Girder ENA (cm)</th>
<th>Theoretical Composite ENA (cm)</th>
<th>Highest ENA at 40 kips Loading (cm)</th>
<th>Lowest ENA at 40 kips Loading (cm)</th>
<th>Difference Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>49.3</td>
<td>96.3</td>
<td>69.6</td>
<td>63.5</td>
<td>38% to 54%</td>
</tr>
</tbody>
</table>

**Flexure Experimental Results**

The experimental flexural results provides relevant items for discussion on the behavior of the bridge sections for Code comparison and adherence to published literature. The various subtopics addressed for flexural testing are; system behaviors, neutral axis location during elastic and plastic loadings, diaphragms, shear studs, deflections, and modes of failure. The ultimate loads are provided in Table 3 showing the applied ultimate load (Pult) and corresponding moments (Mult), using the general static equation PL/4.

The LRFD Bridge Specifications addresses individual component capacity only and currently neglects system capacity through redundant load paths, with the bridge sections being a subpart, system capacity from elastic to ultimate load is relevant to the results of the flexural testing. In Figure 18 and Figure 19, the reaction loads are plotted as a percent of the applied load. The non-loaded girder or off girder bore, during flexural test 2 at 100% P/Pult, a maximum of 6.9% of the total applied load, whereas the majority of applied loadings 0-95% P/Pult correlated with off girder reactions at near zero or negative. Additionally, the mid-span diaphragm see Figure 26, had strain magnitudes of $-12 \mu e \times 10^{-6}$ or 12 $\mu e$ compression and 57 $\mu e$ tension. These strain values show the
diaphragm in flexural Girder E test provided little transfer of load to the off girder. The diaphragm was removed for flexural test Girder C. None of the bridge sections exhibit significant ability to transfer load to the off girder and the bridge sections primarily acted as a single girder components for the flexural tests.

Table 3  Flexural tests ultimate capacities

<table>
<thead>
<tr>
<th>Girder D</th>
<th>Girder E</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point Load (Pult)</td>
<td>Moment (Mult)</td>
<td>Point Load (Pult)</td>
</tr>
<tr>
<td>(kN)</td>
<td>(kN-m)</td>
<td>(kN)</td>
</tr>
<tr>
<td>1486</td>
<td>4048</td>
<td>1333</td>
</tr>
</tbody>
</table>

Figure 18  Flexural test Girder E reaction forces versus applied load.
Figure 19  Flexural test girder C reaction forces versus applied load.

The partial loss of composite action due to removal and transportation of the bridge sections was constant for all flexure tests. Figure 20 illustrates plane sections remain plane as a typical of the strain versus depth of composite cross-section for flexural test Girder C under the applied load, additional typical strain plots are provided in Appendix A. Figure 21 shown the neutral axis in the cross-section under the point load, measured from the top of deck, versus percent point load over ultimate point load (P/Pult). In each flexural test, partial composite action proceeded toward fully composite as the load increased, as shown in Figure 21 with the movement of the neutral axes. Figure 22 depicts the neutral axis location in the first flexural test Girder D across the length of the girder at specific load increments, identified in Chapter 4. The load increments are the equivalent design load, the experimental first yield, and the theoretical plastic capacity. Comparable neutral axis movement is portrayed in Figure 23 for flexural test Girder E and Figure 24 for flexural test Girder C. Advancement of the
neutral axis toward the concrete deck was generally observed but predominantly in the region under the point load. Point load locations where 545 cm (214"), 545cm (214"), and 442cm (174") for flexure tests Girders D, E, and C, respectively. Sufficient strain data was collected, during the post elastic range of flexure test 1, to show the neutral axis up in the concrete deck at a minimum depth from the top of deck as 9.9 cm (3.9") measured from the top of deck down. Notwithstanding the damage incurred in transportation, the depth of this neutral axis at 9.9 cm (3.9") demonstrates a plastic moment capacity was achieved. Not the theoretical plastic moment capacity which required the entire steel cross-section to yield, but a near theoretical plastic capacity. Flexural tests Girders E and C exhibited similar elastic and post-yield neutral axes behavior, however, extensive yielding up the web left one strain gage in the elastic range for each test, two points of strain are require to interpolate the neutral axis, prevents this data from being plotted beyond 90% of P/Pult. It can be concluded from increased web yielding the neutral axis for tests Girders E and C moved higher into the concrete deck and achieved a greater plastic capacity than Girder D test.

Figure 20  Plane sections remain plane for flexure test Girder C, typical.
**Legend**

Steel Girder NA: Neutral axis of only the steel girder.
ENA: Elastic neutral axis of composite section.
PNA: Plastic neutral axis of composite section.

![Graph 1](image1.png)

**Figure 21** Neutral axis location versus load plot for flexural tests.

![Graph 2](image2.png)

**Figure 22** Flexural test Girder D neutral axis location across length of girder.
Figure 23  Flexural test Girder E neutral axis location across length of girder.

Figure 24  Flexural test Girder C neutral axis location across length of girder.

System performance of simple supported shewed bridges is generally observed to be; increased vertical reactions at obtuse corners, increased transfer of load across diaphragms, and torsion of the bridge (Bechtel, 2008). In comparing the reaction forces for the flexural tests, the acute corners of the loaded girders bore more of the applied load
than the obtuse corners. Supplementary to this, are the higher neutral axis locations toward the 0 cm end or E1 shown in Figure 23, the higher the neutral axis the more moment being resisted. Similar results can be deduced for flexural tests Girder D and C. To address this anomaly of the acute corner collecting more load than the obtuse corners, the correlated study by Brackus (2010) identifies through finite modeling consistent results. This irregularity is simple a function of the limited number of girders involved and the loading location. For flexural test Girder C a tiltmeter was secured to the bottom flange of the loaded girder at mid-span, providing rotational data by which lateral deflections is computed (Figure 25), but no equipment or sensors were situated to quantify the experimental torsion for the flexural tests. With none of the bridge sections exhibiting significant ability to transfer load to the off girder the discussion on system capacity is limited, however, this bridge testing is particularly relevant for two girder skewed bridges for system capacity. Two items are evident; first the skew alters reaction loads with symmetric loading, and second two girder system behavior is different than multi-girder.

In an effort to quantify the contribution of the diaphragm, two strain gages were placed on the mid-span of the C-channel diaphragm placed symmetrically about the horizontal axis, see Figure 25. The strain values in Figure 26 show the diaphragm acting primarily axially with increased bending at the onset of yielding in the girder. In a destructive bridge test in Ontario (Bakht and Jaeger, 1988), diaphragms were shown to participation very little, on the magnitude of 10 MPa (1.5 ksi). Similar diaphragm data was captured during flexural test Girder E, strain magnitudes of -12 με (10^-5) or 12 με
compression and 57 με tension, this is an equivalent stress of 2.5 MPa (0.3 ksi) and 11.4 MPa (1.7 ksi). In comparing the reaction force distributions and diaphragm strain little correlation can be drawn from the shift of force in the reactions, from negative to positive, through the diaphragms strains and conversely the shift in the strain from compression to tension in the reaction distribution. More can be drawn from the comparison of the first yield at 45% P/Pult, see Table 4, and the point of inflection for compression in the diaphragm. First yield altered the behavior of the diagram but the participation remains relatively insignificant compared to the applied load.

Table 4  Flexural test capacities

<table>
<thead>
<tr>
<th>Point Load (kN)</th>
<th>Girder D</th>
<th>Girder E</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>P/Pult</td>
<td>Point Load (kN)</td>
<td>P/Pult</td>
<td>Point Load (kN)</td>
</tr>
<tr>
<td>791</td>
<td>53%</td>
<td>604</td>
<td>45%</td>
</tr>
<tr>
<td>Elastic Capacity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1486</td>
<td>100%</td>
<td>1333</td>
<td>100%</td>
</tr>
<tr>
<td>Plastic Capacity (Pult)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 25  Diaphragm strain gages.
Figure 26  Flexural test Girder E diaphragm strain versus load.

From the neutral axis results and failure modes of the flexural tests the shear studs and grout pockets were of sufficient strength to withstand the ultimate load. Composite action is contingent on resistance to shear flow at the concrete deck and steel girder interface (Chapman, 1965). Flexural tests Girder E and C exhibited strains on the steel top indicative of the neutral axis being well into the concrete portion of the deck and approaching yield before the ultimate capacity was obtained. Flexural test Girder D in Figure 21 shows increased contribution from concrete deck to resist the applied load as the neutral axis moves to a plastic position. Post ultimate load, as the damage increased on the bridge section the grout pockets experience a typical failure mode of a cracking of the grout pocket over the top of the nelson stud, see Figure 27. With the documented achievement of plastic behavior and the type of ultimate failure, the shear studs were of sufficient strength to withstand the ultimate load.
Deflections at mid-spans were monitored for each flexural test and show consistency in elasticity of materials and post yield behavior for all the flexural tests. Prior to commencement of loading, a camber of 0.64 cm (0.25") at the mid-span was visible in all girders. In Figure 28 all three flexural test vertical deflections are shown as a percent of P/Pult. Slight variations in the transition from elastic to plastic behavior of P/Pult are attributed to damage of the concrete deck incurred from other destructive tests and location of neutral axis. Vertical deflections acted as expected and will be compared to the LRFD Bridge Specifications in Chapter 5. Transverse or lateral deflections at the mid-span of flexural test 3 are shown in Figure 25 and similarly will be compared to the LRFD Bridge Specifications in Chapter 5.

Figure 27  Post ultimate load typical shear pocket failure.
Figure 28  Load versus vertical deflection plot.

Figure 29  Flexural test Girder C load versus transverse or lateral deflection.

Testing Damage and Failure Description

The first flexural test was an incremental static point load applied at the mid-span on Girder D of Bridge Section 1. To summarize the incurred damage, the asphalt on the
deck crushed and extruded out from under the spherical bearing point load beginning at 444 kN (100 kips). Alligator cracks on the underside of the precast concrete deck panels initiated in panel directly under the load and propagated outward to other panels as the load increments increased. Asphalt crushing and alligator cracking were the only visible signs of damage up to 1243 kN (280 kips), including visible signs of yield or sagging of the girder. First yield occurred at a load of 791 kN (178 kips) as reported in Table 4. This indicated that significant yielding occurred without being visually detected. At a load of 1243 kN (280 kips) permanent deformation was apparent.

In the effort to reach a load of 1421 kN (320 kips), four attempts were required, due to malfunctioning of the hydraulic pump. In the process of each attempt, significant damage was sustained to the bridge section. At the load of 1274 kN (287 kips), a crack developed in the edge face of the pre-cast concrete panel directly under the load running horizontally about 2.5 cm (1") below the top of deck. Permanent deformation also increased with each attempt. At a load of 1305 kN (294 kips) a similar crack occurred in the edge face running horizontally. This crack ran parallel with the top rebar mat in the deck and eventually became the primary contributor to failure. At the fourth 1421 kN (320 kips) attempt this crack extended and separated vertically, as shown in Figure 30 left. Significant damage occurred during this loading, delamination and spalling on the underside of deck are shown in Figure 30 (right).

The ultimate load was ascertained on the subsequent 1510 kN (340 kip) attempt with the maximum achieved load of 1486 kN (334.6 kips). At the ultimate load, a crack through the top deck reinforcement propagated and separated vertically approximately
7.5 cm (3"), apparently caused by a rebar buckling. The mode of failure is described by the onsite of a horizontal concrete crack in the deck edge face across the top rebar mat, a likely result of insufficient development length on the transverse steel in the deck (top mat) to the deck edge. Delamination through the thickness of the deck eventually resulted in localized crushing of the concrete under the point load, followed by a drop in the neutral axis, with continued deflection and diminished capacity. Figure 31 shows the final condition of the flexural test on Girder D.

The second flexural test was a monotonic static point load applied at the mid-span on Girder E of Bridge Section 1. First yield occurred at a load of 604 kN (136 kips) as reported in Table 4. A deck edge crack formed in the concrete panel at a load of 99% of P/Pult signifying a splitting failure of the concrete. Due to the deck being previously damaged from flexural test 1 the deck contributed less to the load resistance, which resulted in diminished ultimate load and altered the mode of failure. The maximum load achieved was 1333 kN (300 kips), see Figure 32 for the final condition of flexural Girder E test.
The third flexural test was a monotonic static point load applied 103 cm (40.5") from mid-span on Girder C of Bridge Section 2. First yield occurred at a load of 742 kN (167 kips) as reported in Table 4. A conical edge crack formed in the concrete panel at a load of 1385 kN (312 kips). The width of the crack increased to approximately 2 cm (1") along the top mat of steel reinforcement at a load of 1532 kN (345 kips). Crushing of the concrete was the primary mode of failure at a maximum load of 1557 kN (351 kips). A picture taken during the testing Figure 33 (left) shows the cracking of the deck edge.
concrete just prior to the ultimate. Figure 33 (right), shows the final condition of flexural Girder C test.

From the experimental flexural testing, it has been shown that composite action of the precast concrete deck panels and the built-up steel plate girders was diminished in the removal and transportation process. Regardless of the degree of partial composite action, the neutral axes demonstrated composite action increased as load increased in the elastic range, especially where the load was applied. Neutral axis locations in the post-elastic region were indicative of plastic behavior. From the Girder D flexural test, with the development length rebar mode of failure suggest, an in situ ultimate test would achieve higher ultimate from more concrete contribution. Diaphragms transferred very little load to off girder and predominantly transferred an axial load. The diaphragms contributed to the resistance of lateral deflections. The non-loaded or off girder collected little load at the reactions and therefore exhibited poor system capacity. Additionally, the acute corners were shown to collect more of the vertical force than the obtuse corners, contrary to typical skewed bridge behavior. This anomaly is a result of the number of girders and

Figure 33  Flexural test Girder C.
loading location. Vertical deflections data was consistence in the elastic and post-elastic regions for all three flexural tests. The mode of failures differed slightly; Girder D failed as a result of insufficient development length on the transverse deck reinforcement and crushing of the concrete, Girders D and E failed by crushing of the concrete.

**Beam Shear Tests**

Two monotonic ultimate shear tests were performed, both on bridge section 2, the first on Girder B and the second on Girder C. The location of the point load was at a distance of the depth of the composite cross-section 128 cm (50"), away from the reaction, and directly over the steel girder, a depiction is shown in Figure 34. The asphalt overlay was removed in a manner consistent with other tests. The precast panel section above End 2 was a different length than End 1, as such, a transverse joint was located directly under the point load for the B2 or first shear test. For the second shear test, the transverse joint was outside the distance of a depth away, thus isolating the effect of the transverse joint on the failure modes of the ultimate two shear tests, see Figure 35 for a depiction of the loading locations for the beam shear tests.

![Figure 34 Beam shear test loading location profile.](image-url)
Figure 35  Beam shear loading locations.

Beam Shear Setup

Bridge section 2 Girder B, monotonic static shear

The first shear test occurred on the second bridge section over Girder B with the point load placed, as described above, Joint 4. The two reactions on End 1 were exchanged for roller supports with load cells and spherical bearings as described in the flexural tests. Potentiometers were used to measure deflections on Girder C at mid-span, Girder B Mid-span, Girder B quarter point closest to point load, on Girder B directly under the point load. Strain gages were placed in strategic locations to determine first yield of the steel web panel. Four arrays of strain gages were used, the first on the bottom flange horizontally between web stiffeners, second vertically from the bottom of
the bottom flange to the bottom of the top flange, third and fourth are diagonals from corner of web stiffener and flanges oriented in 45 degrees from horizontal, see Figure 36 for a depiction. These arrays were repeated on the second steel panel from reaction B2. The third and fourth steel panels were only given vertical arrays for the purpose of measuring the neutral axis location.

Bridge Section 2 Girder C, Monotonic Static Shear

The second shear test occurred on the second bridge section over Girder B with the point load was placed as described above, near C1. The two reactions on End 2 were exchanged for roller supports with load cells and spherical bearings as described in the flexural tests and the End 1 reactions acting as pins. As discussed in the third flexure test, Bridge Section 2 Girder C flexure, the lateral deflection on the girders was also evident on the shear tests. The potentiometer readings had components of deflection and
rotational movement. In attempt to decouple the lateral deflection from vertical deflection, a new instrumentation setup was used for the second shear test.

Potentiometers were used to measure deflections on Girder B at mid-span, Girder C Mid-span (one with the new setup and one without), Girder B quarter points, and a sixth used to measure the axial movement of Girder C at roller reaction C2. Again strain gages were strategically placed to determine first yield of the steel panel. Four arrays of strain gages were used with some alterations from the first test. The arrays were first on the top flange horizontally between web stiffeners, second mirror to the top of the bottom flange, the third array was on a quarter diagonal point oriented parallel to the diagonal, and the fourth was on the half diagonal point again oriented parallel to the diagonal, see Figure 37 for a depiction.

Figure 37 Strain gage locations for Girder C beam shear (cm).
Beam Shear Experimental Results

The first shear test was preformed directly over the transverse Joint 4 and Girder B of bridge Section 2. The monotonic loading data showed no signs of damage to the steel girder until 80% Reaction load/Reaction Ultimate load (R/Ult). At 80% R/Ult a strain gage showed plastic type behavior, see Figure 40, even though the strain was below yielding strain which is plausible considering the complexity of stresses and strains at yielding of a uniaxial strain gage, additional strain plots are provided in Appendix A. The ultimate load 1517 kN (342 kips) occurred shortly after the initiation of tension yielding (gages a1 and a4) in the web panel of the steel girder, and can be classified as an inelastic web buckling. Tension strut yielding and out of plane buckling began in the web at the top flange corner above the reaction and propagated diagonally to the bottom flange at the intermediate web stiffener. Soon after the ultimate load, a shallow crack in the deck developed in the edge face from the point load toward the reaction. The steel panel continued to buckle and deform under the applied load, gradually decreasing the capacity of the cross-section. The external force was applied until a diagonal tension strut was fully developed extending from stiffener to stiffener and the concrete deck crack had separation, due to de-bond of the grout pocket from the top of the first set of nelson shear studs. The final condition is shown in Figure 38.
Figure 38  Post ultimate load Girder B beam shear.

Figure 39  Beam shear strain gages plotted below: (left) Girder B, (right) Girder C.

Figure 40  Beam shear Girder B tension strain a series.
The second shear test was performed 128 cm (50") from End 1 over Girder C of bridge section 2. Monotonic loading data shows multiple damage locations prior to achieving the ultimate load. Negative strain measurements in the first steel girder panel near the reaction stiffener exceeded -1310 micro-strain ($\mu$e) or strain indicative of compression yielding, based on the modulus of elasticity, at a load of 1345 kN (303 kips). At an applied load of 1354 kN (305 kips) an adjacent strain gage showed evidence of compression yielding and a third gage at 1456 kN (328 kips) compression yielding near the top flange. The compression yield around the edges of the web panel is consistent with known mechanics web panel behavior (Yoo and Lee, 2006). Additionally, yielding of the bottom flange in the second panel, directly under the point load, and in the third steel panel occurred prior to the ultimate load. The ultimate load was recorded as 1470 kN (331 kips) before the inelastic web buckling failure. Extensive out of plane buckling continued as load was applied and mimicked the failure of the first shear test. The final condition of the shear test is shown in Figure 42.
The two shear test generated similar ultimate capacities despite several differences in deflection measurements and stiffness of the deck. Figure 43 shows the vertical deflection under the point load for shear tests. Girder B demonstrated a much more ductile failure and was less stiffness in the linear elastic region. Additionally, flexure type yielding was recorded in the strain data of the Girder C shear test prior to the ultimate load, more deflection plots are provided in Appendix A. In comparing the ultimate capacities of the two shear tests an 8.0% difference on the reaction force was determined. Girder B shear test being stiffer and loaded directly over a transverse joint. Girder C shear test was less stiff and later in the loading sequence of the bridge section testing. Thus, the difference can be attributed to the damage sustained during the Girder B shear and Girder C flexural testing, see Appendix B for a depiction of the testing sequence. It was determined, that neither the concrete deck nor the transverse joint had a significant influence on the final mode of failure. Tabulated ultimate shear capacities are provided in Table 5.
Table 5  Shear test ultimate capacities

<table>
<thead>
<tr>
<th>Shear Test Girder B</th>
<th>Shear Test Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reaction Force</td>
<td>Reaction Force</td>
</tr>
<tr>
<td>(Rult) (kN)</td>
<td>(Rult) (kN)</td>
</tr>
<tr>
<td>1330</td>
<td>1224</td>
</tr>
</tbody>
</table>

Figure 43  Beam shear vertical deflections under point load.

Punching Shear Tests

Punching shear tests were performed on the bridge sections after flexure and shear tests were complete. As the bridge was already in a damaged state, no attempt was made to collect additional information beside applied load and reactions. Four monotonic punching shear tests were conduction on undamaged portions of the deck. For two tests, the load was placed in the center of the panel in-between the girders and away from the transverse joints. For the other two tests, the load was placed on or adjacent to the transverse joint, see Figure 44 for a depiction of the loading locations.
Figure 44  Punching shear loading locations.

**Punching Shear Setup**

For all four the punching shear tests the setups were identical. Reactions and point load were constructed consistent with the previous flexure tests, including the super-elevation. Reactions containing bottom plates, load cells, spherical bearing, and topping plates. The point load was constructed with a spherical bearing, load cell, and ram including the necessary steel plates. It is important to reemphasize a 20 cm (8")
diameter spherical bearing was used as the point load bearing surface directly on the concrete of the deck.

**Punching Shear Experimental Results**

The first punching shear test was preformed directly over transverse Joint 4 of bridge Section 1. Visible minimal bulging on the underside of the deck was seen during the loading. Two concrete crushing noises were heard in sequence at the ultimate load after which the load capacity was significantly reduced. Post crushing noises the ram continued to apply load until considerable damage was visible. The plane of failure was conical shape around the transverse welded connection at about a 45° angle; however, this cone shape dissipated near the transverse joint and propagates along the joint, see Figure 45.

The second punching shear test was performed on the precast panel between End 1 and Joint 1 centered between Girders D and E. Over the course of loading minute deflection was visible on the underside of deck. Failure was an instantaneous audible crack with a distinct circular rupture plane on the underside of deck. Visible signs of failure were limited to the concrete cracking in the circular area that had punched through. Post rupture the ram force significantly diminished, to ensure the ultimate capacity was achieved more load was applied. The post rupture loading drastically increased visible damage, loading continued until yielding of rebar was visible. Spalling of concrete outside of the circular failure plane was a direct result of post rupture loading and consistent with other tests. Figure 46 shows the final condition of the punching shear
failure after the removal of the spalled concrete. The ultimate capacity conical failure is visible; the angle of conical failure was measured at 30 degrees from horizontal.

The third punching shear test was performed on the precast panel centered between Girders B and C adjacent to transverse Joint 3. Audible cracking of the concrete was heard periodically throughout this test especially from 75% of ultimate load to the maximum load. From observation of video recordings, vertical separation of the precast concrete panels at the transverse joint under load occurred prior to the circular or cone
shaped failure plane. Post ultimate loading contributed the overwhelming majority of damage, spalling, and separation of precast panels. Figure 47 shows the final condition of this punching shear failure.

The fourth punching shear test was performed on the precast panel between Joint 2 and Joint 3 off-centered between Girders B and C slightly to Girder B. A singular transverse crack on the underside of the deck from Girder B to the center of the panel was notable prior to the ultimate capacity. At failure the underside of the concrete deck slumped in a conical depression accompanied with an audible sloughing of material. The conical shape approached the edge of the steel Girder B and may have affected the failure load. The ultimate capacity conical failure was measured to be 30 degrees from horizontal. Figure 48 shows the final condition of this punching shear failure.

The punching shear ultimate capacities are tabulated below in Table 6. The tests are grouped 2 and 4 for center of precast panel loading locations and, 1 and 3 for the loads applied over or adjacent to a transverse joint. For the punching shear tests in the center or near center of panel the loads at failure were both substantially larger than those
over or near a transverse joint. Additionally, the modes of failure described in connection with tests 2 and 4 where similar, in that, both developed a circular rupture plane on the underside of deck and a conical punch through the cross-section originating at the point load. The increased capacity of test 4 over test 2 can be attributed to test 4’s circular failure region being inches from Girder B. The failure modes for tests 1 and 3 were dissimilar; however both modes of failure were directly affected by the proximity of transverse joint and ruptured prematurely compared to the center or near center of panel loading. Thus it is concluded; the decreased capacities for tests 1 and 3 are ascribe to the transverse joint. The transverse joint constitutes a reduction to the punching capacity of 40%.

Table 6  Ultimate punching shear capacities

<table>
<thead>
<tr>
<th>Center or Near Center of Precast Panel</th>
<th>Above or Adjacent to Transverse Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Punching Shear 2 (kN)</td>
<td>Punching Shear 4 (kN)</td>
</tr>
<tr>
<td></td>
<td>Punching Shear 1 (kN)</td>
</tr>
<tr>
<td></td>
<td>Punching Shear 3 (kN)</td>
</tr>
<tr>
<td>616</td>
<td>811</td>
</tr>
<tr>
<td></td>
<td>371</td>
</tr>
<tr>
<td></td>
<td>404</td>
</tr>
</tbody>
</table>
CHAPTER IV
BRIDGE SPECIFICATIONS

The AASHTO LRFD Bridge Specifications (AASHTO, 2009) is the primary source used to investigate the adherence of design for the bridge experimental tests described in Chapter 3. The specifications layout the criteria by which the bridge sections are compared in Chapter 5. Experimental tests consisted of three flexural, two beam shear, and four punching shear tests. For the two limit states strength and service no factors (φ) were considered. Each set of tests are discussed individually to ascertain theoretical values, capacities, and code requirements, herein as Chapter 4.

Flexural Capacity

The LRFD flexure analysis is restricted in discussion to the steel girder superstructure and composite deck systems in strength and service limit states. The restrictions are based on project constraints of having removed bridge sections as specimens and the quantity of specimens. Within applicable areas, the various subtopics to flexure addressed are; system capacity, shear studs, diaphragms, and what can be referred to as the four points of interest. The four points of interest deal with specific loadings in the flexural testing and constitute a substantial part of the flexural tests.

In regard to system capacity LRFD Bridge Specifications addresses individual component capacity only and currently neglects system capacity through redundant load paths. Redundancy in load path is a standard practice in current bridge design adopted as a result of failure of fracture critical bridges, by definition lacked redundancy. The
interaction between individual components is accounted by use of distribution factors. Distribution factors (LRFD 4.6.2.2) fundamentally reduce the beam design loadings (flexure and shear) by transfer of load to adjacent girders. These criteria are based on the number and proximity of adjacent girders, span length, and thickness of deck. An increased adjustment is required for shear at obtuse corners (LRFD Table 4.6.2.2.3c-1). The bridge specimens fail to meet the first criterion of multiple girders, and therefore limit the analyses compared to LRFD distribution factors. The experimental data yielded results contrary to load increases at obtuse corners and is a likely candidate for future research at this institution.

Shear studs (LRFD 6.10.10.4) have two generally considered modes of failure; yielding of the steel stud, and crushing of the concrete around the stud (LRFD 6.10.10.4.3-1). In the event insufficient shear studs or strength concrete is provided, composite action between the concrete deck and steel girders cannot be achieved. From the modes of flexure and beam shear failures, the shear studs did not inhibit the load attained during experimental testing and therefore are considered adequate in number and design.

Diaphragms or cross-bracing (LRFD 6.7.4) is not specifically required nor is a design procedure detailed in the LRFD Bridge Design Specifications. Diaphragms are secondary members or primary for horizontally curved girder bridges. The design is based on the investigation of constructability, transfer of lateral loads, stability of flanges, distribution of load, etc. For the flexural tests, only one test (Girder C) was outfitted with the ability to measure the lateral deflection, this test also removed the center diaphragm.
to compare the transfer of load to the off girder. Additionally, strain data from the
diaphragm on flexure test Girder E showed predominantly axial force. This suggests the
diaphragms of a multi-girder system would increase the weak axis or lateral bending
stiffness of the girders, even if the transfer of load across the diaphragm is small as
reported for this and other destructive bridge tests.

Precast concrete flexurally discontinuous transverse joints (LRFD 9.7.5.2) have
no limitations on the types and materials used for this joint. The LRFD commentary
provides some insight to the functionality and past experiences with precast deck slab
transverse joints, but the only current stipulation is to be owner approved.

Flexural Factored Loads

For the purpose of flexural testing four points of interest or loadings are
considered in detail; the tandem truck equivalent moment, theoretical first yield or elastic
capacity, the theoretical plastic moment capacity equivalent moment, and the ultimate
load. The first theoretical point of interest is a code stipulated controlling design load,
from this critical loading a maximum moment is determined and is compared with an
equivalent moment produced by a point load using beam theory. The second loading of
interest, is the point of first yield theoretically calculated using the elastic method for a
moment and equivalent point load moment. Third, the theoretical plastic moment
capacity is again a moment in which the steel has yielded displacing the neutral axis
upward on the cross-section and sufficient hinging of the beam causes collapse. The
theoretical plastic moment capacity also correlates to an equivalent moment produced by a point load and is the theoretical ultimate load.

The first equivalent moment based on LRFD Bridge Design Specifications for the critical loading scenario is the tandem truck (LRFD 3.6.1.2) centered over the mid-span for flexure tests Girder D and Girder E. The third flexure test (Girder C) the loading location is 0.4L and generates a slightly different maximum moment. The single girder required moment capacity is two (25 kip) axle loads with (4') separation, divided by 2 for the wheel load and increased by 33% for a dynamic allowance (LRFD 3.6.2), equates a moment of 358 kN-m (264 k-ft) for flexure tests 1 and 2; flexure test 3 required moment capacity is 377 kN-m (278 k-ft). Thus the first point loads of interest are the tandem truck at a point load of 131 kN (29.6 kips) for flexure tests Girder D and Girder E and 143 kN (32.3 kips) for Girder C.

The second point of interest is the point load required to produce a moment equal to the elastic (at yield) capacity of a single girder with composite deck. This capacity is based on elastic theory of transformation of sections and parallel tributary width, in this case ½ the deck width. The elastic capacity calculations are provided in Appendix B, the required moment to produce yielding is 2034 kN-m (1500 k-ft). Thus, the second point loads of interest are the 742 kN (167 kips) for flexure tests Girder D and Girder E and 786 kN (177 kips) for Girder C.

The third point of interest is the point load required to produce a moment equal to the plastic capacity of a single girder with composite deck. The plastic moment capacity based on LRFD Bridge Design Specifications is limited by satisfying (LRFD 6.10.7)
Equation 1 to use nominal plastic capacity as the theoretical plastic moment. LRFD Appendix D6.1 provides a detailed method for solving the theoretical plastic moment. For a single girder and concrete tributary width the theoretical plastic moment is 3101 kN-m (2,287 k-ft) or an equivalent point load of 1137 kN (256 kips) for flexure tests Girder D and Girder E and an equivalent point load of 1176 kN (265 kips) for Girder C, calculations following the LRDF Appendix D6.1 are provided in Appendix B. The plastic moment capacity utilizing the plastic stress distribution method determines the plastic neutral axis with the assumption that the steel material is elastic than perfectly plastic. The neutral axes from elastic to plastic (Chapman, 1965) has been shown to move upward on the cross-section as yielding occurs, also the beam exhibits additional capacity. The theoretical plastic moment is considered the ultimate capacity. A summary of the points of interest are compiled and tabularized in Chapter 5, Table 7.

Vertical deflections for the bridge sections tested according to the LRDF Bridge Specifications are limited to L/800 for bridges with vehicular loads and without pedestrian walkways and cantilever ends (LRFD 2.5.2.6.2). For the bridge sections tested the maximum allowable deflection is calculated as 1.36 cm (0.54"). No maximum allowable lateral deflection is stipulated in the current specifications.

\[ If \ D_p \leq 0.1D_t, then \ M_n = M_p \]  \hspace{1cm} (1)

where:

- \( D_p \) = Distance from top of deck to PNA (in.); 2.63"
- \( D_t \) = Total depth of composite section (in.); 47"
- \( M_n \) = Nominal Flexural Resistance (k - ft)
- \( M_p \) = Plastic Moment Capacity (k - ft); 2,287 k - ft

LRFD 6.10.7.1.2-1 Nominal Flexural Resistance
Beam Shear Capacity

Both the AISC and AASHTO LRFD Bridge Design Specifications assume only the steel web resists shear forces though not explicitly stated in the AASHTO code, it is in AISC (2006). AISC (2006) section 13.1b “A conservative approach to shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete contribution and serves to simplify design.” By ignoring the concrete deck and steel flanges, the only parameters governing shear for composite beams are the web properties. Equation 2 provides the nominal shear resistance as the shear buckling resistance. The shear buckling resistance \( V_{\text{cr}} \) is solved through the plastic shear force \( V_p \) and determined as the ratio \( (C) \) between the two. The plastic shear force is a function of steel yield strength and the cross-sectional area of the web, see Equation 3. The ratio \( C \) is calculated using Basler’s model for tension field action. The \( C \) ratio for this case falls into a category of inelastic web buckling, see Equation 4. Tension field action is not considered for the end panels, but because stiffeners are provided and the aspect ratio is such, that the interior panels can have tension field influences for the ultimate capacity. These design parameters were not tested due to insufficient testing specimens. Bases on these calculations, the shear capacity is reduced slightly (less than 0.5%) due to web slenderness, and is therefore, small enough that it can be neglected. Using Equations 2 through 5 the theoretical nominal shear capacity was calculated as 1390 kN (313 kips) and is valid for only steel end panels. Additional, stiffeners (LRFD 6.10.2.1.1) are not required by the specifications base on Equation 6, but were part of the existing bridge.
\[ V_n = V_{cr} = CV_p \]  

where:

- \( V_p \) = Plastic shear force (kips)
- \( V_n \) = Nominal Shear resistance of web panel (kips)
- \( V_{cr} \) = Shear – buckling resistance (kips)
- \( C \) = ratio of \( V_{cr} \) to shear yield strength

**LRFD 6.10.9.3.3-1 Nominal Shear Resistance of Web End Panel**

\[ V_p = 0.58F_{yw}Dt_w; 314 \text{ kips} \]  

where:

- \( F_{yw} \) = Web minimum yield strength (ksi); 38 ksi
- \( D \) = Depth of web (in); 38"
- \( t_w \) = Web thickness (in); 0.375"

**LRFD 6.10.9.3.3-2 Plastic Shear Force**

\[ \text{If } 1.12 \sqrt{\frac{Ek}{F_{yw}}} \leq \frac{D}{t_w} \leq 1.4 \sqrt{\frac{Ek}{F_{yw}}}, \text{then } C = \frac{1.12}{\frac{D}{t_w}} \sqrt{\frac{Ek}{F_{yw}}}; C = 0.996 \]  

where:

- \( E \) = Modulus of Elasticity (ksi); 29,000 ksi
- \( D \) = See Equation 3
- \( F_{yw} \) = See Equation 3
- \( t_w \) = See Equation 3
- \( C \) = See Equation 2
- \( k \) = See Equation 5

**LRFD 6.10.9.3.2-5 Ratio of Shear Buckling Resistance to Shear Yield Strength**

\[ k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \]  

where:

- \( d_o \) = Stiffener Spacing (in); 35.75"
- \( k \) = shear – buckling coefficient; 10.65
- \( D \) = See Equation 3

**LRFD 6.10.9.3.2-7 Shear Buckling Coefficient**
If \( \frac{D}{t_w} \geq 150 \); then longitudinal stiffeners required \quad (6) \\

where:

\( D = \text{Depth of steel Section}; \) 38 in. \\
\( t_w = \text{Web Thickness}; \) 0.375 in.

LRFD 6.10.2.1-1 Stiffeners

In determining the theoretical shear capacity an alternative value can be considered based on the specified web steel 248 MPa (36 ksi) requirements in replace of the coupon steel tested yield of 262 MPa (38 ksi) properties, steel coupon test results are provided in Appendix A. This approach delves into the ideological realm of the basis and intent of the code, by allowing the overstrength of the material to provide excess, unaccounted for shear capacity. If specified properties are used, the theoretical calculated shear capacity is 1321 kN (298 kips).

**Beam Shear Factored Loading**

The required capacity based on the LRFD Bridge Design Specifications considers either the design truck or the tandem load. For the bridge span unlike flexure the design truck creates a higher shearing force at 128 cm (50") from the end support, where the back axle lands exactly over this point. Thus, the design truck loading for shear is 200 kN (45.1 kips) for the axle or 100 kN (22.6 kips) for the wheel load and a dynamic impact factor allowance by 33% renders a calculated load of 133 kN (30 kips) for design.
Punching Shear Capacity

Punching shear is more commonly a design parameter for footing and pile caps. The equations used for determining the material shear resistance is similar for ACI-08 and AASHTO LRFD Bridge Design Specifications (2009) and are used in this scenario for deck punching shear. The LRFD Specifications recommendations using Equation 7 shown below, for Two-Way Action design also known as punching shear. This equation is based on three primary parameters, the material property of concrete compressive strength and the depth and width of the concrete section. The width is determined based on an assumed failure plane which identifies the critical section as the section through which tensile forces occur at an angle of 45 degrees from the load, see Figure 49, LRFD (C9.7.2.1) states the failure inclination is experimentally less than 45 degrees but is still used as a source of conservatism. This distance is averaged from the top of concrete to the bottom rebar centroid (distance d). The depth is again based on flexural design which dependent on the compressive strength of concrete and the moment arm from the bottom rebar centroid to the Whitney concrete compression block centroid.

\[
V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f'_c} b_o d_v \leq 0.126\sqrt{f'_c} b_o d_v
\]  

(7)

where:

- \(\beta_c\) = Ratio of long side - short side through which load acts; 1.0
- \(f'_c\) = compressive strength of concrete (ksi); 8.3 ksi
- \(b_o\) = perimeter of critical section (in); 44.77"
- \(d_v\) = effective shear depth (in); 5.94"

\(V_n\) = Nominal Punching Shear (kips); 96.5 kips

LRFD (5.13.3.6.3-1) Punching Shear Two-Way Action
Figure 49  Theoretical punching shear failure.

The material properties for punching shear capacity can be estimated in a couple of ways. First, the designer can use the actual concrete strength determined from core samples and second the UDOT minimum specified concrete strength. The values shown with the variables Equation 7 are based on the concrete strength of the core samples, the estimated punching shear capacity of the deck is 428.5 kN (96.5 kips). If the UDOT minimum specified for 27.6 MPa (4.0 ksi) concrete strength is used, $d_p = 5.6''$ and the punching shear capacity is 243.3 kN (54.8 kips). These values show a drastic overstrength in the estimated capacity based actual concrete strength provided and the punching shear designed capacity.

**Punching Shear Factored Loads**

The LRFD Bridge Design Specifications design load for punching shear is half of the maximum axle load or 71 kN (16 kips). The dynamic load allowance (LRFD 3.6.2) increases this value to 94 kN (21 kips).
CHAPTER V

COMPARISON OF EXPERIMENTS AND BRIDGE SPECIFICATIONS

The AASHTO LRFD Bridge Design Specifications is the primary tool used to layout the criteria in Chapter 4 to compare the bridge experimental tests described in Chapter 3. Experimental tests consisted of three flexural, two beam shear, and four punching shear. The specifications considered service and strength limit states for the superstructure specimens collected from 8th North and I-15 underpass in Salt Lake City.

Flexure Comparison

The experimental flexural testing investigated the five various subtopics; system capacity, shear studs, diaphragms, and four loading magnitudes or points of interest as an integral part of composite action. The bridge sections failed to meet the criterion of multiple girders system and as such, displayed poor transfer of load to the unloaded girder; therefore little investigation was performed for the system capacity. Shear studs, a critical component of composite action, have been concluded to be adequate in strength and design from the experimental tests. Diaphragms were shown primarily to transfer axial forces or resist lateral deflection; by connecting all girders together of the original six girders in situ bridge system the lateral stiffness would increase 3 fold, mitigating lateral deflection. Vertical deflections are considered in conjunction with the service limit state. The four loading magnitudes equate to the service limit state of the equivalent design loading, tandem truck, and the strength limit state for
Table 7 Theoretical elastic and plastic equivalent point loads

<table>
<thead>
<tr>
<th>Girder D</th>
<th>Girder E</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point Load</td>
<td>Point Load</td>
<td>Point Load</td>
</tr>
<tr>
<td>(kN) 742</td>
<td>(kN) 742</td>
<td>(kN) 768</td>
</tr>
<tr>
<td>Plastic Capacity</td>
<td>Plastic Capacity</td>
<td>Plastic Capacity</td>
</tr>
<tr>
<td>(kN) 1137</td>
<td>(kN) 1137</td>
<td>(kN) 1176</td>
</tr>
</tbody>
</table>

elastic moment capacity, plastic moment capacity, and the ultimate load. A summary of theoretical values are provided in Table 7, was described in Chapter 4.

Experimental results in Chapter 3 provided extensive discussion on composite action of the precast concrete deck and built-up steel plate girders through the location of the neutral axis. It was shown that during specimen collection and transportation the bridge sections sustained damage with the relocation of the neutral axes signifying partial composite action at the beginning of testing. Regardless of the damage incurred, as the load increased the neutral axis progressed toward the theoretical ENA, and as the beam plasticized the neutral axis approached the theoretical PNA. Having the theoretical and experimental neutral axis location compared in Chapter 3, the discussion herein will address service limit state deflection and strength limit state loading capacities.

The LRFD Bridge Design Specifications maximum deflection (Δ) is 1.36 cm (0.54") at the service limit equivalent point load, see Table 8. At the design tandem truck equivalent point load the experimental deflection ranges between 25% and 32% (Δ/Δmax) for the flexural tests. Additionally, it required 33% to 39% of P/Pult to achieve the maximum allowable deflection. The relative nature is of importance for the bridge
sections tested, in the serving of the bridge section deck during the removal process, 34% of the deck weight was eliminated (23% of the total weight). The service limit state is intended to account for all dead loads and live; therefore the experimental deflection is relative to the actual dead and live load tested. Conversely, the deck would increase the moment of inertia and contribute to the load resistance; additionally, the barrier, which may be included in the deflection calculations, would also resist the service limit state deflection. In the design of steel members it is common for deflection to control flexural design; by accomplishing partial composite action the deflections are significantly reduced. With a percent increase required in the weight to return to the original condition and a multiplier required to achieve the maximum allowable deflection it can be concluded that the partial composite action reduces the vertical deflection and the service limit state deflection criterion was satisfied.

The second and third points of interest are the point loads required to produce a moment equal to the elastic and plastic moment capacities of a single girder with composite deck. The actual or experimental capacities are shown in Table 9 in correlation with the theoretical capacities and the percent difference. Table 9 is converted from point loads to moments by the general statics equation PL/4.

From the elastic capacity flexural tests Girder E and Girder C, the bridge section yielded prior to the theoretical, whereas Girder D experienced an increased capacity. Decreases in the elastic capacity were to be expected with the neutral axis being lower than theoretically calculated. The 7% increase in yield capacity of flexural test Girder D demonstrates more concrete than just the tributary width, contributed to resistance of
Table 8  Deflections: at equivalent tandem loads and loads at maximum allowable

<table>
<thead>
<tr>
<th>Girder D</th>
<th>Girder E</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent Tandem Point Load (kN)</td>
<td>P/P_{ult}</td>
<td>Equivalent Tandem Point Load (kN)</td>
</tr>
<tr>
<td>131</td>
<td>9%</td>
<td>131</td>
</tr>
<tr>
<td>(\Delta) (cm)</td>
<td>(\Delta/\Delta_{allow})</td>
<td>(\Delta) (cm)</td>
</tr>
<tr>
<td>0.37</td>
<td>27%</td>
<td>0.44</td>
</tr>
</tbody>
</table>

| \(\Delta_{allow}\) 1.36 (cm) | \(\Delta_{allow}\) 1.36 (cm) | \(\Delta_{allow}\) 1.36 (cm) |
| Load (kN) | P/P_{ult} | Load (kN) | P/P_{ult} | Load (kN) | P/P_{ult} |
| 574 | 39\% | 440 | 33\% | 576 | 37\% |

Table 9  Moment comparison of flexural experimental and theoretical capacities

<table>
<thead>
<tr>
<th>LRFD</th>
<th>Girder D</th>
<th>Girder E</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical Moment</td>
<td>Experimental Difference</td>
<td>Experimental Difference</td>
<td>Experimental Difference</td>
</tr>
<tr>
<td>Elastic Capacity</td>
<td>Moment</td>
<td>Percent</td>
<td>Moment</td>
</tr>
<tr>
<td>(kN-m) 2021</td>
<td>(kN-m) Increase 7%</td>
<td>(kN-m) Decrease 19%</td>
<td>(kN-m) 1952</td>
</tr>
<tr>
<td>Plastic Capacity</td>
<td>(kN-m) Increase 31%</td>
<td>(kN-m) Increase 17%</td>
<td>(kN-m) 4097</td>
</tr>
</tbody>
</table>

the load. Because of bridge specimen 1 deck damages sustained from Girder D flexural test, Girder E test experienced less than the tributary width of concrete to resist the load hence a moderate decrease in elastic capacity. Girder C test would be expected therefore to act similar flexural test Girder D, both having the similar concrete deck conditions. Girder C experienced a decrease in elastic capacity. The incremental static testing of Girder D exacerbated damage by the cyclic nature of incremental testing and would be
less likely to have an increased capacity than Girder C which was tested monotonically. The main differences between the two tests are the type of loading, which is expected to be a benefit for Girder C, and the applied load being directly over a transverse joint for Girder C test. Thus, the 10% differential of Girder D to Girder C elastic capacities can be ascribed to the transverse joint. Regardless of the decreased capacity of the flexural test over the transverse joint all test were able to sustain design loading in the elastic range. In order to produce a moment equivalent to the applied elastic capacity point load 4.0, 3.1, or 3.8 tandem loads are required for Girders D, E, and C, respectively.

The plastic capacity of the flexural tests experienced universal increases over the theoretical values, see Table 9. The damage sustained in the removal of specimens and transportation which caused a drop in neutral axis location has less effect on the ultimate load. With extensive yielding through the depth of the web and sufficient restraint again shear flow at the concrete deck steel girder interface, the neutral axis is expected to rise into the deck mitigating effects of a lower elastic neutral axis. In the theoretical calculation of the plastic moment, steel is assumed to act elastic than perfectly plastic. A36 steel is known to have an ultimate strength of 400 MPa (58 ksi) or higher, with the girders being A36 steel, the ultimate capacities are conceivability 31% or 32% higher than the theoretical. The smaller increase to Girder E is attributed to the deck being damaged from Girder D destructive flexure test. The limiting factor for each flexural test is the concrete deck. In order to produce a moment equivalent to the applied ultimate point load 7.5, 6.8, or 7.6 tandem loads are required for Girders D, E, and C, respectively.
Shear Comparison

In the previous chapters, the ultimate beam shear capacities and modes of failure have been described and design values based on the AASHTO LRFD Bridge Design Specifications have been computed. The ultimate failure tests investigated two distinctly different cross-sectional properties in order to isolate the potential effects of the precast panel transverse joint. From the experimental data consistent with the theoretical mode of failure the concrete deck and therefore transverse joint had minimal influence on the ultimate shear capacity. The mode of failure was strictly a function of the cross-section web area and the steel yield strength.

The theoretical capacity was calculated as 1321 kN (297 kips) based on the specified steel yield strength or 1390 kN (313 kips) based on the experimentally determined steel properties. According to the LRFD Specifications, a reduction factor can be applied to the theoretical capacity based on web slenderness; however this value was calculated as less than 0.5% of the total capacity and was therefore negligible. The experimental shear reaction forces are 1330 kN (300 kips) and 1220 kN (276 kips) for shear test 1 and 2, respectively. In comparison of theoretical and experimental the experimental is 0.7% greater than and 8% less than for the specified steel yield strength,

<table>
<thead>
<tr>
<th>LRFD</th>
<th>Girder B</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>Difference</td>
</tr>
<tr>
<td>Theoretical</td>
<td>ReAction Load (Pult)</td>
<td>Percent</td>
</tr>
<tr>
<td>Reaction Load (Pult)</td>
<td>(kN)</td>
<td>Increase</td>
</tr>
<tr>
<td>1321</td>
<td>1330</td>
<td>0.7%</td>
</tr>
</tbody>
</table>
and 4-12% less than theoretical for the actual steel yield strength. Indicating the ultimate shear capacity was accurately predicted by the code based on the overstrength of steel, alternatively the code was less conservative when using the actual steel properties.

The design truck equivalent wheel load with dynamic allowance is 133 kN (30 kips) and the experimental failures occurred at 1330 kN (300 kips) and 1220 kN (276 kips) for tests 1 and 2, respectively. The experimental applied load is the equivalent of 10 design trucks, and overwhelmingly surpassing code loadings requirements for shear.

**Punching Shear Comparison**

In Chapter 3, two-way action or punching shear was described for experimental testing of the ultimate bridge section failures and in Chapter 4, the design values based on the AASHTO LRFD Bridge Design Specifications were calculated. From the ultimate failure tests two groups were organized for a comparison of the loading location, center or near center of precast panel and over or adjacent to a transverse joint. Ultimate capacities were reported as; 616.0 kN (138.7 kips) and 811.1 kN (182.7 kips) for the center or near center of panel loading location, 371 kN (83.6 kips) and 403.9 kN (91.0 kips) for the load location over or adjacent to a transverse joint. The first set both reported a circular rupture plane on the underside of deck and a conical type failure angle of 30 degrees from horizontal through the cross-section. The largest recorded value 811.1 kN (182.7 kips) may have been influenced by the proximity of the steel girder inches from the circular rupture plane, in an arching effect. The second set modes of failure were directly affected by the proximity of the transverse joint and deviated from a
circular failure plane on the underside of the deck. The LRFD Specifications are congruent with a circular point load creates a circular failure plane for the underside of deck; the angle of the cone failure plane through the cross-section is theoretically 45 degrees. The punching shear capacity was calculated to be 428.5 kN (96.5 kips) based on the measured concrete compressive strength of 57.2 MPa (8.3 ksi), if the UDOT minimum specified 27.6 MPa (4.0 ksi) concrete strength had been used the punching shear capacity calculation the value would be 243.3 kN (54.8 kips).

From LRFD code and using actual concrete compressive strength of 57.2 MPa (8.3 ksi) the calculated punching shear was 428.5 kN (96.5 kips) which conservatively compares with the measured value of 616.0 kN (138.7 kips). The discrepancy between the two values can be imputed to the angle of conical rupture. Theoretically concrete ruptures in tension or at a 45 degree angle, for both tests uninhibited by the transverse joint the observed angle of rupture was 30 degrees. The decreased angle increases the perimeter of the critical section, consistent with LRFD C.9.7.2.1, thus the experimental punching shear capacity was well above the design capacity. Theoretically, the typical 45 degree failure plan of concrete can become shallower by incorporation of flexural forces or force resistance contribution from reinforcement in the horizontal plane. Both scenarios are possible for the experimental tests described above. It is deduced from the shallower rupture angle the experimental ultimate capacity is greater than the code design value and the code is conservative.

The two tests conducted over or adjacent a transverse joint have poor correlation with theoretical understanding of material behavior. These experimental failures
occurred prematurely due to the lack of transfer of load across the transverse joints. Transverse joint contribute to early failures in two significant areas, first reinforcement is discontinuous across the joint and the Whitney stress block therefore cannot be developed, second materials/ material strengths are dissimilar in the grout filled female-female joints. Not only was rebar discontinuous but due to clear cover requirements and constructability, rebar terminations were 38 mm (1.5") or more from the edge of precast panel.

In a subjective fashion which is highly open to scrutiny the punching shear capacity can be estimated for a concrete strength of 27.6 MPa (4.0 ksi) as specified in the UDOT contract documents. The theoretical punching shear capacities based on actual concrete strength and minimum required concrete strength 57.2 MPa (8.3 ksi) and 27.6 MPa (4.0 ksi) were determined to be 428.5 kN (96.5 kips) and 243.3 kN (54.8 kips), respectively. This is a 56.8% reduction in the available capacity. Applying the same percent reduction to the lower of the experimental punching shear capacities gives 210.8 kN (47.5 kips). With the removal of concrete overstrength the punching shear capacity is still above the required two-way action capacity for the given tire contact area (LRFD 3.6.1.2.5) plus dynamic load allowance (LRFD 3.6.2) for the extreme Strength I Limit State at 165.3 kN (37.2 kips).

The design truck wheel load with dynamic allowance is 94 kN (21 kips) and the experimental failures occurred at 371 kN (83.6 kips), 811kN (183 kips), 404 kN (91 kips), and 616 kN (139 kips) for test 1, 2, 3 and 4, respectively. The experimental applied load is the equivalent of 3.9, 6.5, 4.3, and 8.6, design trucks, and overwhelmingly
surpassing LRFD Bridge Design Specifications loading requirements for punching shear. The transverse joint constitutes a reduction to the punching capacity of 40%.

Table 11  Comparison of punching shear experimental and theoretical capacities

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<thead>
<tr>
<th>LRFD Theoretical</th>
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<th>Punching Shear 2</th>
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</thead>
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<tr>
<td></td>
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<td>Difference</td>
<td>Experimental</td>
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<tr>
<td>Point Load (kN)</td>
<td>Point Load (kN)</td>
<td>Percent</td>
<td>Point Load (kN)</td>
</tr>
<tr>
<td>428.5</td>
<td>371</td>
<td>13.4%</td>
<td>616</td>
</tr>
</tbody>
</table>

Punching Shear 3

<table>
<thead>
<tr>
<th>Experimental</th>
<th>Difference</th>
<th>Experimental</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
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<td>Point Load (kN)</td>
<td>Percent</td>
<td>Point Load (kN)</td>
<td>Percent</td>
</tr>
<tr>
<td>404</td>
<td>5.7%</td>
<td>811</td>
<td>89.3%</td>
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</tbody>
</table>
CHAPTER VI
CONCLUSIONS

Two bridge sections, each consisting of two girders and intact concrete deck, removed from I-15 and 800 North underpass outside downtown Salt Lake City were experimentally tested for composite behavior of precast concrete deck panels and built-up steel plate girders. The effects of the female-female concrete panel transverses joints were considered for flexural, beam shear, and punching shear destructive testing. Experimental results were compared with theoretical calculated capacities using the AASHTO LRFD Bridge Design Specifications (2009).

From experimental observations the bridge sections sustained damage in the removal and transportation process as concluded with the location of the experimental neutral axis between the girder neutral axis and the theoretical elastic neutral axis. Regardless of the partial composite behavior of the bridge section system caused by damage sustained in the removal and transportation process, all tests were found to be compliant with the LRFD Bridge Specifications. A bulleted summary of results are provided below for flexural, beam shear, two-way action also known as punching shear, and the effects of the transverse joint on each type of destructive test performed:

1. Flexural Results
   a. In general the experimental elastic capacity was 10% more conservative than the LRFD calculated theoretical capacity. When experimentally tested over a transverse joint the experimental elastic capacity was 3% unconservative compared to the theoretical calculated capacity.
b. Two experimental flexural tests exceeded the theoretical calculated plastic capacity by, or are conservative by, 31% and 32%.

c. In order to produce a moment equivalent to the applied elastic capacity point load 4.0, 3.1, or 3.8 tandem loads are required for Girders D, E, and C, respectively.

d. An equivalent point load of 7.5 and 7.6 design tandem loads are required to equal the ultimate load. The limiting factor for each flexure test was the concrete deck.

c. Deflections were 27%, 32%, and 25% of maximum allowable at the design load 131 kN, 131 kN, and 143 kN for the three flexural tests performed. The LRFD Bridge Design Specifications stipulated maximum allowable deflection is calculated as 1.36 cm was experimentally achieved at point loads of 574 kN, 440 kN, and 576 kN for each flexural test.

2. Beam Shear Results

a. The theoretical ultimate beam shear capacity was calculated as 1321 kN (297 kips). The experimental ultimate capacities were 1330 kN (300 kips) and 1220 kN (276 kips), thus the LRFD Specifications were conservative by 0.7% on one test and 8% unconservative on another.

b. The experimental applied load was the equivalent of 10 design trucks.

3. Punching Shear Results
a. The theoretical ultimate punching shear capacity was calculated as 428.5 kN (96.5 kips). The experimental capacities were 371 kN (83.6 kips), 811 kN (183 kips), 404 kN (91 kips), and 616 kN (139 kips) for tests 1, 2, 3 and 4, respectively. Thus, the LRFD Specifications were conservative for tests 2 and 4 by 43.8% and 89.3%, respectively, and tests 1 and 3 were unconservative by 13.4% and 5.7%, respectively.

b. The transverse joint directly affects the ultimate capacity of punching shear and results in premature failures. The transverse joint constitutes a reduction to the punching shear capacity of 40%.

c. The ultimate capacity of punching shear over a transverse joint for the testing specimens was 3.9 to 4.2 design trucks wheel loads more than required by LRFD Bridge Design Specifications.

4. Transverse Joint Effects

a. The transverse joint was 3% less than the theoretical calculated elastic capacity of the flexural test.

b. The transverse joint had no measurable influence on the plastic capacity of the flexural test.

c. The transverse joint had no significant influence on the beam shear tests.

d. The transverse joint decreased the punching shear capacity by 40% and was 13% less than theoretically calculated.
REFERENCES


Appendix A: Strain and Deflection Plots

Figure 50 Stress versus strain plot for coupon steel test.

Figure 51 Flexural Girder C strain gages plots 138 series.
Figure 52  Flexural Girder C strain gages plots 229 series.

Figure 53  Flexural Girder C strain gages plots 320 series.
Figure 54  Flexural Girder C strain gages plots 411 series.

Figure 55  Flexural Girder C strain gages plots 502 series.
Figure 56  Flexural Girder C strain gages plots 592 series.

Figure 57  Flexural Girder C strain gages plots 683 series.
Figure 58  Beam shear Girder B displacements.

Figure 59  Beam shear Girder C displacements.
Figure 60  Beam shear Girder B strain gages plots A series.

Figure 61  Beam shear Girder C strain gages plots AT series.
Figure 62  Beam shear Girder C strain gages plots AB series.

Figure 63  Beam shear Girder C strain gages plots aD1 Series.
Figure 64  Beam shear Girder C strain gages plots aD2 series.
### Appendix B: Neutral Axis Calculations

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<thead>
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<th>Term</th>
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<th>Units</th>
<th>Term</th>
<th>Value</th>
<th>Units</th>
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<td>$F_{yx}$</td>
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<td></td>
<td>$A_y$</td>
<td>119.7 in$^2$</td>
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<tr>
<td>$e$</td>
<td>4.4 in$^2$</td>
<td></td>
<td>$y$</td>
<td>37.88 in</td>
<td>from top of bottom</td>
</tr>
<tr>
<td>$f_c$</td>
<td>8.3 ksi</td>
<td></td>
<td>$y$</td>
<td>9.12 in</td>
<td>from top of deck</td>
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<td></td>
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<td></td>
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<td></td>
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<td>$V_{rib}$</td>
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<th>Value</th>
<th>Units</th>
<th>Term</th>
<th>Value</th>
<th>Units</th>
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<td></td>
<td>$A_y$</td>
<td>6.25 in$^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$b_w$</td>
<td>10 in</td>
<td></td>
<td>$y$</td>
<td>18.3125 in</td>
<td></td>
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<tr>
<td></td>
<td>$t_w$</td>
<td>0.625 in</td>
<td></td>
<td>$I_{pant}$</td>
<td>0.203451 in$^4$</td>
<td></td>
</tr>
<tr>
<td>Web</td>
<td>$F_{yw}$</td>
<td>38 ksi</td>
<td></td>
<td>$A_y$</td>
<td>14.25 in$^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$D$</td>
<td>38 in</td>
<td></td>
<td>$y$</td>
<td>0 in</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$t_w$</td>
<td>0.375 in</td>
<td></td>
<td>$I_{pant}$</td>
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<td>$F_{xw}$</td>
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<td>$A_y$</td>
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<td></td>
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<td>Flange</td>
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<td></td>
<td>$y$</td>
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<td></td>
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<td>$E_b$</td>
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<td>29000 ksi</td>
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<td></td>
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<td>5.26 ratio</td>
<td></td>
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<tr>
<td>Transformed Concrete</td>
<td></td>
<td>92.99 in$^2$</td>
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<table>
<thead>
<tr>
<th>First Yield Moment</th>
<th>18002 k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Yield Moment</td>
<td>1500 k-ft</td>
</tr>
</tbody>
</table>

Figure 65 Calculation of ENA.
The calculation of PNA (AASHTO, 2009) involves the following steps:

1. **Equation for Moment:***
   
   \[ M_p = \left( \frac{\gamma^2 P_s}{2r_s} \right) + \left[ P_{t+} d_t + P_{d+} d_d + P_{c+} d_c + P_{p+} d_p \right] P_t + P_v + P_z + P_n \geq \left( \frac{c_s}{t_s} \right) P_s \quad Y = c_{st} \]

2. **Table of Values:**

<table>
<thead>
<tr>
<th>Term</th>
<th>Value</th>
<th>Units</th>
<th>Term</th>
<th>Value</th>
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<th>Case</th>
<th>Value</th>
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<td>A_t</td>
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<td>P_t</td>
<td>264</td>
<td>Kips</td>
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<td></td>
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<tr>
<td></td>
<td>c_d</td>
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<td>P_s</td>
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<td>P_t</td>
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<td>541.5</td>
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<tr>
<td></td>
<td>t_s</td>
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<td>P_t</td>
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<td>Kips</td>
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<td>in</td>
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<td>in</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td>in</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web</td>
<td>F_{yw}</td>
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<td>( \delta_3 )</td>
<td>5.44</td>
<td>in</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>38 in</td>
<td>( \delta_4 )</td>
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<td>in</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>t_w</td>
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<td>in</td>
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<td>( \delta_6 )</td>
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<td>b_t</td>
<td>10 in</td>
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<td></td>
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</tr>
</tbody>
</table>

3. **Moments Calculation:**

   \[ M_p = 27449 \text{ k-in} \quad 2287 \text{ k-ft} \]

Figure 66  Calculation of PNA (AASHTO, 2009).
TESTING SEQUENCE

1 FLEXURE TEST D
2 FLEXURE TEST E
3 PUNCHING SHEAR 1
4 PUNCHING SHEAR 2
5 BEAM SHEAR B
6 FLEXURE TEST C
7 BEAM SHEAR C
8 PUNCHING SHEAR 3
9 PUNCHING SHEAR 4

Figure 67  Testing sequence.