DoD Corrosion Prevention and Control Program

Field Testing and Load Rating Report for Bridge No. 4, Composite Grid Reinforcement of Concrete Deck, at Fort Knox, Kentucky

Contractor’s Supplemental Report for Project F12-AR01

Brett Commander and Brice Carpenter

September 2016

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Field Testing and Load Rating Report for Bridge No. 4, Composite Grid Reinforcement of Concrete Deck, at Fort Knox, Kentucky

Contractor's Supplemental Report for Project F12-AR01

Brett Commander and Brice Carpenter

Bridge Diagnostics, Inc. (BDI)
1965 57th Court North
Suite 106
Boulder, CO 80301

Final report

Approved for public release; distribution is unlimited.

Prepared for Office of the Secretary of Defense (OUSD(AT&L))
3090 Defense Pentagon
Washington, DC 20301-3090

Under Contract W9132T-07-D-0007, Delivery Order 0022, “Demonstration and Validation of 3-D Gridform and Hybrid Composite Beams for Bridges” for Project F12-AR01, “Demonstration and Validation of 3-D Gridform for Bridges”

Monitored by Construction Engineering Research Laboratory
U.S. Army Engineer Research and Development Center
2902 Newmark Drive
Champaign, IL 61822
Abstract

The Army has 1,500 vehicular bridges on its installations, with many experiencing high maintenance or replacement costs due to corrosion of the steel structure or of the reinforcing bar in the concrete deck. Under the Department of Defense Corrosion Prevention and Control Program (Project F12-AR01), a span of Bridge No. 4 at Fort Knox, Kentucky, was selected to demonstrate and validate a fiber reinforced polymer (FRP) composite grid deck system, which does not use any reinforcement steel. The results of that project were published as ERDC/CERL TR-16-21 (September 2016). Bridge Diagnostics, Inc. (BDI) of Boulder, Colorado, was subcontracted to perform load testing and rating for the newly completed composite bridge deck system to confirm that the bridge span met its required load rating and performance criteria for deflection and strain. Structural testing of the bridge was performed at the end of construction and after 12 months of exposure. Results showed the bridge met all design specifications and load ratings.
Foreword

Bridge Diagnostics, Inc. (BDI) of Boulder, Colorado, was subcontracted by Mandaree Enterprise Corporation (MEC), of Warner Robins, Georgia, to perform load testing on Bridge 4 at Fort Knox, Kentucky. MEC was the prime contractor retained by the Construction Engineering Research Laboratory—Engineer Research and Development Center (ERDC-CERL) to supervise the installation and testing of two technologies on two different spans of Bridge 4.

One of the demonstrated technologies is a fiber reinforced polymer (FRP) composite deck system known as Gridform™. Gridform is a stay-in-place concrete bridge deck system that is designed to replace steel rebar in reinforced concrete bridge decks. The technology was selected for demonstration and validation under Project F12-AR01 of the Department of Defense Corrosion Prevention and Control (CPC) Program. The final technical report on that project was published as ERDC/CERL TR-16-21 (September 2016). This contract report, which is incorporated into TR-16-21 by reference, provides complete details on the subcontractor’s execution of the load-testing program.

The primary goal of BDI’s live load testing was to determine whether the FRP composite deck met design specifications for deflection and strain. Their report (reproduced in its entirety here) outlines the testing procedures used, provides a detailed discussion of the data collected, and summarizes the findings.

Richard G. Lampo
Project Manager and Materials Engineer
ERDC-CERL
Champaign, Illinois
Preface

Load testing was conducted by Bridge Diagnostics, Inc. as a subcontractor under Mandaree Enterprise Corporation’s Contract W9132T-07-D-0007, Delivery Order 0022, “Demonstration And Validation Of 3-D Gridform And Hybrid Composite Beams For Bridges” for Project F12-AR01, “Demonstration and Validation of 3-D Gridform for Bridges.” The work was conducted for the Office of the Secretary of Defense (OSD) under the Department of Defense (DoD) Corrosion Control and Prevention Program. The project monitor was Mr. Steven C. Sweeney, CEERD-CFM.

The work was monitored by the Engineering and Materials Branch of the Facilities Division (CEERD-CFM), U.S. Army Engineer Research and Development Center, Construction Engineering Research Laboratory (ERDC-CERL), Champaign, IL. At the time of publication, Ms. Vicki L. Van Blaricum was Chief, CEERD-CFM; Mr. Donald K. Hicks was Chief, CEERD-CF; and Mr. Kurt Kinnevan was the Technical Director for Adaptive and Resilient Installations, CEERD-CZT. The Deputy Director of ERDC-CERL was Dr. Kirankumar Topudurти, and the Director was Dr. Ilker Adiguzel.

The Commander of ERDC was COL Bryan S. Green, and the Director was Dr. Jeffery P. Holland.
FIELD TESTING AND LOAD RATING REPORT:
BRIDGE NO. 4 - STEEL GIRDER SPAN – FORT KNOX, KY

SUBMITTED TO:

Madaree Enterprise Corporation
812 Park Drive
Warner Robbins, GA 31088-5182
www.mandaree.com

SUBMITTED BY:

BRIDGE DIAGNOSTICS, INC.
1995 57th Court North, Suite 100
Boulder, CO 80301-2810
303.494.3230
www.bridgetest.com

Original Submission: January 2013 (Load Test I)
Final Submission: January 2014 (Load Test II)
EXECUTIVE SUMMARY

In September of 2012, Bridge Diagnostics, Inc. (BDI) was contracted by the Mandaree Enterprise Corporation (MEC) to perform diagnostic load tests and load rate Bridge No. 4 at Fort Knox Military Reservation, KY. Load tests were performed as a means of performance verification because the bridge was built with innovative structural materials and construction procedures. An initial diagnostic load test was performed on December 18th, 2012 immediately after the bridge was constructed. A load test and load rating report was submitted in January 2013 indicating the bridge had the desired load capacity. Per the project specifications, a repeat load test was performed one year later on December 18, 2013.

Bridge No.4 consists of two simply supported spans. One span is comprised of five rolled steel girders and a concrete deck reinforced with Strongwell GRIDFORM™ plastic reinforcement. The second span consists of five Hybrid Composite Beams that are composite with a conventionally reinforced concrete deck. This report focuses on the load tests and subsequent analysis and load rating of the steel girder span. Ratings were calculated for steel girders to determine whether or not the span, in its current state, could independently support the live load of the standard AASHTO HL-93 load configuration, the combined military M1070/M1000 load configuration, and the combined military M1070/M747 load configuration. Additionally, a controlling MLC vehicle classification was determined for the bridge for both a tracked and a wheeled configuration.

During both field test phases, the superstructure was instrumented with a combination of strain transducers, cantilevered displacement sensors, LVDT displacement sensors, and tiltmeter rotation sensors. Once the structure was instrumented, controlled load tests were performed with a variety of test vehicles crossing the structure along three different lateral positions. The initial 2012 tests were performed with a set of 3-axle dump trucks in both single and double truck configurations; while the final 2013 load tests were performed using the HETS/M1A1 military logistics vehicle and the M1A1 Abrams tank separately.

Response data obtained from the first round of load tests in 2012 was first evaluated for quality and subsequently used to verify and calibrate a finite-element model of the structure. The resulting calibrated model was then used to analyze the effects of the specified rating vehicles. All load rating factors were calculated using the guidelines specified in the AASHTO Manual for Bridge Evaluation – 2011 Edition and the KYDOT TM08-01 LRFR guidelines.

Since identical loading could not be provided for the two tests, the 2012 calibrated bridge model was used as the reference to evaluate and compare the structure’s performance after one year of use. It was found that the steel span’s behavior did not significantly change over the years’ time. The only notable change in behavior was a slight stiffening of the exterior girders caused by the installation of the bridge rails; which were not present during the first round of load tests. Based on these comparisons, the load ratings performed in 2012 were found to still be valid.

Structural analysis indicated that the steel girder span had some continuity with the hybrid girder span and that the steel girders were partially restrained at the abutment. Additionally, the exterior girders exhibited partial composite behavior with the deck due to the guardrail attachment detail and this behavior was amplified after the bridge rails were installed.

To be consistent with standard rating procedures and to simulate the bridge performance under ultimate load conditions, the partial continuity, the unintended composite behavior and
The contribution of the metal guardrail were eliminated from model for load rating purposes. To reiterate, the following load ratings assume a simply supported, non-composite girder condition. As shown in the following table, the girders met the all load rating criteria for all examined loads, with the Inventory and Operating rating factors all being above 1.0. The resulting bridge load limits were found to be $MLC_T^{139}$ and $MLC_W^{212}$ vehicle classifications for tracked and wheeled vehicles, respectively. Note that these MLC tonnages were based on the hybrid composite beam span, since that span’s ratings controlled the bridge’s critical load ratings.

### Controlling load rating factors & applicable rating weights/classifications.

<table>
<thead>
<tr>
<th>LOADING CONDITION</th>
<th>CRITICAL INVENTORY RF</th>
<th>CRITICAL INVENTORY WEIGHT</th>
<th>CRITICAL OPERATING RF</th>
<th>CRITICAL OPERATING WEIGHT</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93 (Strength)</td>
<td>2.42</td>
<td>-</td>
<td>3.14</td>
<td>-</td>
</tr>
<tr>
<td>HL-93 (Service)</td>
<td>2.85</td>
<td>-</td>
<td>3.70</td>
<td>-</td>
</tr>
<tr>
<td>HL-93 (Fatigue)</td>
<td>5.68</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>HETS M1070/ M1000</td>
<td>2.49</td>
<td>286</td>
<td>3.23</td>
<td>372</td>
</tr>
<tr>
<td>HETS M1070/ M747</td>
<td>2.14</td>
<td>225</td>
<td>2.77</td>
<td>291</td>
</tr>
<tr>
<td>M1A1 Tracked</td>
<td>-</td>
<td>-</td>
<td>1.94*</td>
<td>$MLC_T^{139}$*</td>
</tr>
<tr>
<td>MLC70 Wheeled</td>
<td>-</td>
<td>-</td>
<td>3.04*</td>
<td>$MLC_W^{212}$*</td>
</tr>
</tbody>
</table>

* MLC Ratings controlled by HCB span.

The steel girder span’s load rating was controlled by the positive moment capacity of the girder at midspan. The load ratings were acceptable with Inventory Rating Factors above 1.0 for all load configurations and conditions. All indications are that the examined load configurations can safely cross the bridge without speed restrictions.

This report contains details regarding the instrumentation and load testing procedures, a qualitative review of the load test data, a brief explanation of the modeling steps, and a summary of the load rating methods and results.
Submittal Notes:

This submittal includes the following files on CD:

1. **BDI-FortKnox_Testing_Documents_Round-1.pdf**
   This file provides pertinent details about the instrumentation plan and testing scenarios/procedures for Bridge No. 4 performed in the first round of testing (December 18, 2012).

2. **BDI-FortKnox_Testing_Documents_Round-2.pdf**
   This file provides pertinent details about the instrumentation plan and testing scenarios/procedures for Bridge No. 4 performed in the first round of testing (December 18, 2013).

3. **BDI_FortKnox_Steel_Submittal_V2_DRAFT.pdf**
   This is the BDI report in “pdf” format. It contains details regarding the testing procedures, provides a qualitative data evaluation, describes the modeling procedures and results, and provides the procedure and results of the field-calibrate rating performed on the steel girder span.

4. **BDI_FortKnox_Steel_Rating_Output_and_Summary_Files**
   The output files contain detailed information regarding the applied load and resistance factors, capacities, unfactored structural responses, and critical load rating results for each of the rated vehicles. The summary files contain the critical load rating results along with the controlling factored responses.

5. **BDI_FortKnox_Selected_Data-Round-1.xlsx**
   This spreadsheet contains selected data from the 2012 Round 1 test that has been extracted and formatted as a function of vehicle position for each truck path. Additionally, envelope data (minimum and maximum responses) for every gage location at each of the three lateral truck positions, including the two lanes and tandem tests, is provided in this file. It also gives the vehicle location for each min/max response as related to the test truck’s front axle.

6. **BDI_FortKnox_Selected_Data-Round-2.xlsx**
   This spreadsheet contains selected data from the 2013 Round 1 test that has been extracted and formatted as a function of vehicle position for each truck path. Additionally, envelope data (minimum and maximum responses) for every gage location at each of the three lateral truck positions, including the two lanes and tandem tests, is provided in this file. It also gives the vehicle location for each min/max response as related to the test truck’s front axle.
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1. STRUCTURAL TESTING PROCEDURES

Bridge No. 4 is a two span simply supported structure that carries Main Range Road over a stream in Fort Knox Military Reservation, KY. The structure’s South Span consists of five rolled W33x130 steel girders spaced at 6'-1.5” with a non-composite concrete deck containing Strongwell GRIDFORM™ plastic reinforcement. According to the structure’s plans, the steel girder span has an overall width of 27'-8” and a unit length of approximately 38'-6” (center-to-center of bearing). It should be noted that BDI did not perform an in-depth visual inspection of the structure during the load tests.

To evaluate the steel girder span’s response to live load, the steel span was instrumented with:

- 20 reusable, surface-mounted strain transducers (Figure 1.1 and Figure 1.2)
- 5 cantilevered displacement sensors (Figure 1.1)
- 6 LVDT displacement sensors (Figure 1.3)
- 2 surface-mounted tiltmeter rotation sensors (Figure 1.2).

The instrumentation plans for both rounds of tests, including sensor locations and IDs, have been provided in the attached drawings labeled “BDI-FortKnox_Testing_Documents_Round-1.pdf” & “BDI-FortKnox_Testing_Documents_Round-2.pdf”. The instrumentation plan view, sections/elevations for the steel girder span, and test vehicle paths for both test phases have been provided Figure 1.9 through Figure 1.18.

Once the instrumentation was installed, a series of controlled load tests were completed with various load configurations traveling across the structure at crawl speed (3 to 5 mph). During each controlled test, data was recorded from all sensors at sample rate of 40 Hz. Each test vehicle crossed the structure along three different lateral positions, referred to as Paths Y1, Y2, and Y3 in the southbound direction (further described in Figure 1.18).

During the first round of load tests (designated as Round 1 tests from here on), a 3-axle 71 kip dump was used for single lane loaded tests along all three paths. Next, a two lane test was also performed with a 3-axle 72 kip dump truck traveling next to the 3-axle 71 kip dump truck along Paths Y1 and Y3 respectively (Figure 1.5). Lastly, a tandem load test was conducted where the 72 kip truck pulled the 71 kip truck backwards along the center truck path as shown in Figure 1.6. This tandem test was performed to simulate a loading closer to that of the Military HETS vehicle load configuration. The 71 kip truck will be referred to as Truck 1 and the 72 kip truck will be referred to as Truck 2 for the remainder of this report. The Round 1 test vehicles’ gross weights, axle weights, and wheel rollout distance (required for tracking its position along the structure) are provided in Table 1.2. Vehicle “footprints” used in Round 1 are also shown in Figure 1.19 and Figure 1.20. The vehicle weights were obtained from certified scales and all vehicle dimensions were measured in the field at the time of testing.

During the second round of load tests, both a HETS/M1A1 military logistics vehicle and the M1A1 Abrams tank crossed the structure, separately, along all three test paths (Y1, Y2, & Y3). Recorded weights and axle spacing for these vehicles have been provided in Table 1.3 and Table 1.4.

During the load tests, the vehicles’ longitudinal position was wirelessly tracked so that the response data could later be viewed as a function of vehicle position rather than just an arbitrary
point in time. In all cases, the test vehicles were the only live loads applied to the structure while data was being recorded.

BDI would like to thank Mandaree Enterprise Corporation for scheduling, planning, organizing, and aid in implementing the testing project.

### Table 1.1 - Steel girder span description & testing info.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STRUCTURE NAME</strong></td>
<td>Bridge No. 4 - Steel Girder Span (South Span – Span 1)</td>
</tr>
<tr>
<td><strong>MANDAREE PROJECT NUMBER</strong></td>
<td>120246</td>
</tr>
<tr>
<td><strong>BDI PROJECT NUMBER</strong></td>
<td>120901-KY</td>
</tr>
<tr>
<td><strong>TESTING DATES</strong></td>
<td>Round 1 - December 18\textsuperscript{th}, 2012</td>
</tr>
<tr>
<td></td>
<td>Round 2 - December 18\textsuperscript{th}, 2013</td>
</tr>
<tr>
<td><strong>LOCATION/ROUTE</strong></td>
<td>Main Range Road, Fort Knox, KY</td>
</tr>
<tr>
<td><strong>STRUCTURE TYPE</strong></td>
<td>Span 1: Non-Composite Steel Girder</td>
</tr>
<tr>
<td><strong>TOTAL NUMBER OF SPANS</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>SPAN LENGTHS</strong></td>
<td>Span 1: 38’-6” (center-to-center of bearing)</td>
</tr>
<tr>
<td><strong>STRUCTURE/ROADWAY WIDTHS</strong></td>
<td>Structure: 27’-8”</td>
</tr>
<tr>
<td><strong>WEARING SURFACE</strong></td>
<td>N/A</td>
</tr>
<tr>
<td><strong>TOTAL SPANS TESTED</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>TEST REFERENCE LOCATION (BOW) (X=0,Y=0)</strong></td>
<td>North-west corner of the structure at the expansion joint and outside edge of the slab</td>
</tr>
<tr>
<td><strong>TEST VEHICLE DIRECTION</strong></td>
<td>Southbound</td>
</tr>
<tr>
<td><strong>TEST BEGINNING POINT</strong></td>
<td>Front axle 20 ft. north of Span 2 from test reference location (BOW)</td>
</tr>
<tr>
<td><strong>LOAD POSITIONS</strong></td>
<td>Truck Path Y1 – Passenger’s side wheel 3’-0” from BOW</td>
</tr>
<tr>
<td></td>
<td>Truck Path Y2 – Passenger’s side wheel 10’-3” from BOW</td>
</tr>
<tr>
<td></td>
<td>Truck Path Y3 – Driver’s side wheel 24’-8” from BOW</td>
</tr>
<tr>
<td><strong>NUMBER/TYPE OF SENSORS</strong></td>
<td>- 20 Strain Transducers</td>
</tr>
<tr>
<td></td>
<td>- 5 Cantilevered Displacement Sensors</td>
</tr>
<tr>
<td></td>
<td>- 6 LVDT Displacement Sensors</td>
</tr>
<tr>
<td></td>
<td>- 2 Tiltmeter Rotation Sensors</td>
</tr>
<tr>
<td><strong>SAMPLE RATE</strong></td>
<td>40 Hz</td>
</tr>
<tr>
<td><strong>NUMBER OF TEST VEHICLES</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>STRUCTURE ACCESS PROVIDED BY</strong></td>
<td>BDI</td>
</tr>
<tr>
<td><strong>VEHICLES PROVIDED BY</strong></td>
<td>Mandaree Enterprise Corporation</td>
</tr>
<tr>
<td><strong>TOTAL FIELD TESTING TIME</strong></td>
<td>2 days</td>
</tr>
<tr>
<td>ITEM</td>
<td>DESCRIPTION</td>
</tr>
<tr>
<td>------</td>
<td>-------------</td>
</tr>
<tr>
<td><strong>Test File Information:</strong></td>
<td></td>
</tr>
<tr>
<td>File Name</td>
<td>Lateral Position</td>
</tr>
<tr>
<td>FTK_1</td>
<td>Y1</td>
</tr>
<tr>
<td>FTK_2</td>
<td>Y1</td>
</tr>
<tr>
<td>FTK_3</td>
<td>Y1</td>
</tr>
<tr>
<td>FTK_4</td>
<td>Y2</td>
</tr>
<tr>
<td>FTK_5</td>
<td>Y2</td>
</tr>
<tr>
<td>FTK_6</td>
<td>Y2</td>
</tr>
<tr>
<td>FTK_7</td>
<td>Y2</td>
</tr>
<tr>
<td>FTK_8</td>
<td>Y3</td>
</tr>
<tr>
<td>FTK_9</td>
<td>Y3</td>
</tr>
<tr>
<td><strong>ROUND 1 Tests</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Single Truck Test: Truck 1</strong></td>
<td></td>
</tr>
<tr>
<td>FTK_10</td>
<td>Y3 &amp; Y1</td>
</tr>
<tr>
<td><strong>ROUND 1 Tests</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Two Lane Truck Test</strong></td>
<td></td>
</tr>
<tr>
<td>FTK_11</td>
<td>Y2</td>
</tr>
<tr>
<td>FTK_12</td>
<td>Y2</td>
</tr>
<tr>
<td><strong>ROUND 1 Tests</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Tandem Truck Test</strong></td>
<td></td>
</tr>
<tr>
<td>FtKnoxM1_1</td>
<td>Y1</td>
</tr>
<tr>
<td>FtKnoxM1_2</td>
<td>Y1</td>
</tr>
<tr>
<td>FtKnoxM1_3</td>
<td>Y2</td>
</tr>
<tr>
<td>FtKnoxM1_4</td>
<td>Y2</td>
</tr>
<tr>
<td>FtKnoxM1_5</td>
<td>Y3</td>
</tr>
<tr>
<td>FtKnoxM1_6</td>
<td>Y3</td>
</tr>
<tr>
<td><strong>ROUND 2 Tests</strong></td>
<td></td>
</tr>
<tr>
<td><strong>M1A1 Tank</strong></td>
<td></td>
</tr>
<tr>
<td>FtKnoxHETS_1</td>
<td>Y1</td>
</tr>
<tr>
<td>FtKnoxHETS_2</td>
<td>Y1</td>
</tr>
<tr>
<td>FtKnoxHETS_3</td>
<td>Y2</td>
</tr>
<tr>
<td>FtKnoxHETS_4</td>
<td>Y2</td>
</tr>
<tr>
<td>FtKnoxHETS_5</td>
<td>Y3</td>
</tr>
<tr>
<td>FtKnoxHETS_6</td>
<td>Y3</td>
</tr>
<tr>
<td><strong>Other Test Comments:</strong></td>
<td>Weather – Sunny, ~50°F</td>
</tr>
</tbody>
</table>
Figure 1.1 - Strain transducers and displacement sensor near midspan of steel girder (typical of both test phases).

Figure 1.2 - Strain transducer and rotation sensor installed near pier in steel girder span (typical of both test phases).
Figure 1.3 – LVDT displacement sensors installed near abutment in Span 1 to measure slab deflection independent of the girders’ deflection (typical of both test phases).

Figure 1.4 – Round 1 Tests - Truck 1 crossing bridge along Truck Path Y3 during testing.
Figure 1.5 – Round 1 Tests - Two lane truck test: Truck 1 along Path Y3 & Truck 3 along Path Y1.

Figure 1.6 – Round 1 Tests - Tandem truck test: Truck 2 pulling backwards Truck 1 along Path Y2.
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Figure 1.8 – Round 1 Tests – HETS vehicle with M1A1 crossing Truck Path Y1.
Figure 1.9 - Plan view of Bridge No. 4 – Steel Span - Instrumentation plan including gage types and locations.
(Both Rounds of Testing)
Figure 1.10 – Round 1 Tests - Cross-sectional View of Section D-D instrumentation showing sensor IDs, Channel IDs, & sensor locations.

Figure 1.11 – Round 2 Tests - Cross-sectional View of Section D-D instrumentation showing sensor IDs, Channel IDs, & sensor locations.
Figure 1.12 - Round 1 Tests Cross-sectional View of Section E-E instrumentation showing sensor IDs, Channel IDs, & sensor locations.

Figure 1.13 – Round 2 Tests - Cross-sectional View of Section E-E instrumentation showing sensor IDs, Channel IDs, & sensor locations.
Figure 1.14 – Round 1 Tests - Cross-sectional View of Section F-F instrumentation showing sensor IDs, Channel IDs, & sensor locations.

Figure 1.15 – Round 2 Tests - Cross-sectional View of Section F-F instrumentation showing sensor IDs, Channel IDs, & sensor locations.
Figure 1.16 – Round 1 Tests - Cross-sectional View of Section G-G instrumentation showing sensor IDs, Channel IDs, & sensor locations.

Figure 1.17 – Round 2 Tests - Cross-sectional View of Section G-G instrumentation showing sensor IDs, Channel IDs, & sensor locations.
Figure 1.18 - Test vehicle path locations for Bridge No. 4 (Both Rounds of Testing).
### Table 1.2 – Test vehicle information – Round 1 Tests.

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>TRUCK 1 TEST VEHICLE 3-AXLE DUMP TRUCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>70,920 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1: FRONT</td>
<td>20,430 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 2: REAR TANDEM PAIR 1</td>
<td>25,490 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 3: REAR TANDEM PAIR 2</td>
<td>25,000 lbs</td>
</tr>
<tr>
<td>SPACING: AXLE 1 - AXLE 2</td>
<td>16’-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 2 – AXLE 3</td>
<td>4’-5”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>BDI</td>
</tr>
<tr>
<td>WHEEL ROLLOUT DISTANCE</td>
<td>11.08’ per wheel revolution</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>TRUCK 2 TEST VEHICLE 3-AXLE DUMP TRUCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>71,940 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1: FRONT</td>
<td>18,720 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 2: REAR TANDEM PAIR 1</td>
<td>26,720 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 3: REAR TANDEM PAIR 2</td>
<td>26,500 lbs</td>
</tr>
<tr>
<td>SPACING: AXLE 1 - AXLE 2</td>
<td>16’-6”</td>
</tr>
<tr>
<td>SPACING: AXLE 2 – AXLE 3</td>
<td>4’-5”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>BDI</td>
</tr>
</tbody>
</table>
Table 1.3 – Test vehicle information – Round 2 Tests – HETS/M1A1.

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>HETS/M1A1 VEHICLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>216,730 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1</td>
<td>20,150 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 2</td>
<td>21,050 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 3</td>
<td>19,440 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 4</td>
<td>18,750 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 5</td>
<td>24,940 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 6</td>
<td>25,030 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 7</td>
<td>24,320 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 8</td>
<td>30,810 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 9</td>
<td>32,240 lbs</td>
</tr>
<tr>
<td>SPACING: AXLE 1 - AXLE 2</td>
<td>13'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 2 – AXLE 3</td>
<td>5'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 3 – AXLE 4</td>
<td>5'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 4 – AXLE 5</td>
<td>15'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 5 – AXLE 6</td>
<td>6'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 6 – AXLE 7</td>
<td>6'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 7 – AXLE 8</td>
<td>6'-0”</td>
</tr>
<tr>
<td>SPACING: AXLE 9 – AXLE 10</td>
<td>6'-0”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>BDI</td>
</tr>
<tr>
<td>WHEEL ROLLOUT DISTANCE</td>
<td>13.78’ per wheel revolution</td>
</tr>
</tbody>
</table>
### Table 1.4 – Test vehicle information – Round 2 Tests – M1A1 Abrams.

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>M1A1 ABRAMS TANK</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>70,920 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1</td>
<td>20,220 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 2</td>
<td>23,360 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 3</td>
<td>23,030 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 4</td>
<td>22,430 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 5</td>
<td>17,720 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 6</td>
<td>16,060 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 7</td>
<td>12,290 lbs</td>
</tr>
<tr>
<td>SPACING: AXLE 1 - AXLE 2</td>
<td>2’-5”</td>
</tr>
<tr>
<td>SPACING: AXLE 2 – AXLE 3</td>
<td>2’-5”</td>
</tr>
<tr>
<td>SPACING: AXLE 3 – AXLE 4</td>
<td>2’-5”</td>
</tr>
<tr>
<td>SPACING: AXLE 4 – AXLE 5</td>
<td>2’-5”</td>
</tr>
<tr>
<td>SPACING: AXLE 5 – AXLE 6</td>
<td>2’-6”</td>
</tr>
<tr>
<td>SPACING: AXLE 6 – AXLE 7</td>
<td>3’-6”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>BDI</td>
</tr>
<tr>
<td>WHEEL ROLLOUT DISTANCE</td>
<td>7.05’ per wheel revolution</td>
</tr>
</tbody>
</table>

**Figure 1.19 - Test vehicle footprint – Round 1 Tests- Truck 1.**
Figure 1.20 - Test vehicle footprint – Round 1 Tests – Truck 2.
2. Preliminary Investigation of Test Results

All of the field data was examined graphically to provide a qualitative assessment of the structure’s live-load response. Some indicators of data quality include reproducibility between tests along identical truck paths, elastic behavior (strains returning to zero after truck crossing), and any unusual-shaped responses that might indicate nonlinear behavior or possible gage malfunctions. This process can provide a significant amount of insight into how a structure responds to live-load, and is extremely helpful in performing an efficient and accurate structural analysis.

- Responses as a Function of Load Position: Data recorded from the wireless truck position indicator (BDI AutoClicker) was processed so that the corresponding response data could be presented as a function of vehicle position. This step was crucial during the model calibration process since it allowed the engineer to easily compare the measured and computed responses as the test loads moved across the structure. Please note that the test reference location (denoted at “Beginning of World” or “BOW) was located at the bridge’s north expansion joint and corresponds to the 0.0 ft load position in provided figures.

Once all data was processed as a function of load position, one file from each of the test types was selected as having the best apparent quality. Table 2.1 and Table 2.2 provide a list of the selected data for all test types for both rounds of load tests. Please note that the selected data was used to determine the response envelopes for all gages.

- Reproducibility and Linearity of Responses: Responses from identical truck passes were found to be very reproducible as shown in Figure 2.1 through Figure 2.6. In addition, all response data appeared to be linear with respect to load magnitude and truck position. Note that the majority of responses returned to zero indicating elastic behavior, with the exception of residual responses that occurred when each girder was loaded with a heavier loading than it had previously resisted. This response mechanism occurred during both rounds of tests and is discussed further in the next bullet point. Despite the initial loading offset observed in a few test files, all of the response histories had a similar degree of reproducibility and linearity, indicating that the data collected was of good quality.

- Movement at Bearings & Bridge Rails Observed under Heavy Loadings: Residual readings or “response offsets” were observed after each heavier test load crossed the structure. These offsets were primarily seen on directly loaded girders during the load tests in which a heavier load crossed the structure for the first time. This phenomenon occurred first with the single Round 1 dump trucks crossings; next with the Round 1 double dump truck configuration crossings; then with the Round 2 M1A1 tank crossings; and finally with the Round 2 HETS crossing. Note: each test load was heavier than the previous vehicle configuration that had crossed the structure. However, all of the structural responses were found to essentially return to zero in duplicate load tests, indicating that the girders returned to their original state once the initial movement had occurred. Figure 2.7 through Figure 2.10 provide strain and displacement histories from initial and duplicate tests that illustrate this observed behavior.

After reviewing the Round 1 test data, it was found that the initial residual responses under the dump truck configurations were most likely caused by movement at the bearing locations. This bearing movement simply occurred in the girders that had never been loaded to such a
large magnitude before that given load test. This behavior is common for simply supported structures with bearing conditions that can develop friction between the bearing plates and the bottom of the beams.

After reviewing the Round 2 test data, it was found that not only was there evidence of movement at the bearings, but there was also evidence of movement at the connection between the bridge rail and the deck edge. This additional movement was observed as much larger offsets along the exterior beams when compared to the interior beams. This data observation was supported by loud popping noises heard near these connections as the military vehicles crossed the structure; especially during the first tests run near the structure’s edges (Test Paths Y1 & Y3). This behavior was likely due to the fact that the bridge rails were not installed during the first round of tests, and the M1A1 and HETS crossings were the first time these details were heavily loaded.

In general, this type of behavior is common for newer structures and does not affect the structural capacity of the bridge.

- **Lateral Load Distribution in Steel Girders:** When evaluating a bridge for the purpose of developing a load rating, the bridge’s ability to laterally distribute load is an essential characteristic to quantify. Lateral distribution is most easily observed by plotting the response values from an entire gage line cross-section as shown in Figure 2.11 through Figure 2.14. The response values shown in these figures correspond to the longitudinal load positions producing the maximum responses for each truck path at a midspan gage line (Sections E-E). From these figures, it can be observed that a minimum of 4 girders provided significant contribution to each truck crossing along all lateral paths; indicating that the structure exhibited a substantial amount of lateral load distribution across its cross-section. This behavior is typical of beam-slab structures.

- **Observed Neutral Axis of Steel Girders:** It was found that the neutral axis of the steel girders were generally near the center of the webs, meaning the steel girders were acting non-compositively with the deck. Figure 2.15 and Figure 2.16 show that the top gage, centered on the girder’s top flange, experienced a high level of compression around the same magnitude as the gage on the bottom flange. This behavior indicates that the neutral axis was approximately centered between the gage pair. This observed non-composite behavior verified the structure’s design and was considered during the modeling and rating process.

- **Continuity at Supports:** It was noted that both spans were acting somewhat continuous across the pier, as shown in Figure 2.17 and Figure 2.18. The level of continuity was higher than expected since there was a physical gap between superstructures of each span. The observed continuity was likely due to the pile cap’s ability to transfer horizontal load and displacement across the pier at each span’s bearing seat. This condition can be considered irrelevant from a load rating perspective but must be considered for model calibration since it influenced the measure responses.

- **Slab and Girder Serviceability Check:** LVDT displacement sensors were mounted to 2x4s set between steel girders near steel girder span’s abutment in order to measure slab displacement independent of girder displacement. These measurements were used to check the AASHTO LRFD 2.6.2.6.2 deflection limit of L/800 where L equaled to the steel girder spacing of 73.5”. As shown in Figure 2.19, the peak slab displacement was 0.0045” for the double dump truck configuration and was up to 0.0075 for the M1A1 vehicle; which was well below the 0.09” AASHTO deflection limit. The midspan cantilevered displacement
sensors mounted on the girders recorded maximum deflections of approximately 0.19” under the double dump trucks (towing configuration) and up to 0.21” under the M1A1 Tank. This was also well below the 0.57” deflection limit specific to the 38’-6” span length.

As previously stated, all test data was initially processed and assessed for quality. Then, one set of test data for each truck path was selected for having the best apparent quality. This selected data was then used to verify and calibrate the finite-element (FE) model of the structure, which was in turn used to make further conclusions about the effects of the cracking as it becomes more severe. Table 2.1 provides a list of the data files that were used in the FE analysis.

Table 2.1 – Round 1 Tests - Bridge No. 4 Selected Truck Path File Information.

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>TRUCK PATH</th>
<th>SELECTED DATA FILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Truck Crossing – Truck 1</td>
<td>Y1</td>
<td>FTK_3.dat</td>
</tr>
<tr>
<td></td>
<td>Y2</td>
<td>FTK_7.dat</td>
</tr>
<tr>
<td></td>
<td>Y3</td>
<td>FTK_9.dat</td>
</tr>
<tr>
<td>Two Lane Truck Crossing</td>
<td>Truck 1 (Y3), Truck 2 (Y1)</td>
<td>FTK_10.dat</td>
</tr>
<tr>
<td>Tandem Truck Crossing</td>
<td>Truck 2 towing backwards Truck 1</td>
<td>FTK_12.dat</td>
</tr>
</tbody>
</table>

Table 2.2 – Round 2 Tests - Bridge No. 4 Selected Truck Path File Information.

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>TRUCK PATH</th>
<th>SELECTED DATA FILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1A1 Abrams Tank</td>
<td>Y1</td>
<td>FtKnoxM1_2.dat</td>
</tr>
<tr>
<td></td>
<td>Y2</td>
<td>FtKnoxM1_4.dat</td>
</tr>
<tr>
<td></td>
<td>Y3</td>
<td>FtKnoxM1_6.dat</td>
</tr>
<tr>
<td>HETS/M1A1 Vehicle</td>
<td>Y1</td>
<td>FtKnoxHETS_2.dat</td>
</tr>
<tr>
<td></td>
<td>Y2</td>
<td>FtKnoxHETS_4.dat</td>
</tr>
<tr>
<td></td>
<td>Y3</td>
<td>FtKnoxHETS_6.dat</td>
</tr>
</tbody>
</table>
Figure 2.1 – Round 1 Tests - Example of strain response reproducibility on steel girder span (Span 1).

Figure 2.2 – Round 2 Tests - Example of strain response reproducibility on steel girder span (Span 1).
Figure 2.3 – Round 1 Tests - Example of displacement response reproducibility on steel girder span (Span 1).

Figure 2.4 – Round 2 Tests - Example of displacement response reproducibility on steel girder span (Span 1).
Figure 2.5 – Round 1 Tests - Example of rotation response reproducibility on steel girder span (Span 1).

Figure 2.6 – Round 2 Tests - Example of rotation response reproducibility on steel girder span (Span 1).
Figure 2.7 - Round 1 Tests - Strain Response History – On steel girder near pier – Directly loaded by truck path Y3 – Highlighting movement at bearings under first heavy loading.

Strain response offset indicated friction resistance at bearing due to first occurrence of heavy direct loading.

Figure 2.8 - Round 2 Tests - Strain Response History – On steel girder near pier – Directly loaded by truck path Y3 – HETS vehicle induced minor offsets on first crossing.

Strain response offset indicated friction resistance at bearing and at the connection between the deck and the bridge rail due to heavier HETS vehicle.
Figure 2.9 - Round 1 Tests - Displacement Response History – On steel girder near midspan – Directly loaded by truck path Y3 - Highlighting movement at bearings under first heavy loading.

Displacement response offset indicated friction resistance at bearing due to first occurrence of heavy direct loading.

Figure 2.10 - Round 2 Tests - Displacement Response History – On steel girder near midspan – Directly loaded by truck path Y3 - HETS vehicle caused offsets.

Displacement response offset indicated movement at bearing and at the connection between the deck and the bridge rail due to heavier HETS vehicle.
Figure 2.11 – Round 1 Tests - Lateral Load Distribution – Peak midspan strain values – Paths Y1-Y3.

Figure 2.12 – Round 2 Tests - Lateral Load Distribution – Peak midspan strain values – Paths Y1-Y3 – HETS Vehicle.
Figure 2.13 – Round 1 Tests - Lateral Load Distribution – Peak midspan displacement values - Paths Y1-Y3.

Figure 2.14 – Round 2 Tests - Lateral Load Distribution – Peak midspan displacement values - Paths Y1-Y3 – HETS Vehicle.
Figure 2.15 – Round 1 Tests - Top & bottom strain response near midspan of steel girder span (Span 1).

Figure 2.16 – Round 2 Tests - Top & bottom strain response near midspan of steel girder span (Span 1).
Figure 2.17 – Round 1 Tests - Bottom flange strain near midspan – showing continuity between spans.

Compression on bottom flange at midspan, while truck was on adjacent span, indicated flexural continuity between spans.

Figure 2.18 – Round 2 Tests - Bottom flange strain near midspan – showing continuity between spans – HETS Vehicle.

Compression on bottom flange at midspan, while truck was on adjacent span, indicated flexural continuity between spans.
Figure 2.19 – Round 1 Tests - LVDT Displacement – showing slab deflection relative to steel girders.

Figure 2.20 – Round 2 Tests - LVDT Displacement – showing slab deflection relative to steel girders – HETS Vehicle.
3. MODELING, ANALYSIS, & DATA CORRELATION

The key objectives of calibrating a finite-element bridge model are to accurately simulate the behaviors recorded during load testing, and in turn utilize this model to accurately predict the structure’s response under standard and site-specific rating loads. The steel girder span and the hybrid girder span were modeled independently due to the variation in required modeling and analysis techniques. This section briefly describes the methods and findings of Bridge #4’s steel girder span modeling procedures. A list of modeling and analysis parameters specific to this bridge is provided in Table 3.1.

MODEL CALIBRATION PROCEDURES

First, geometric data collected from the construction plans and insight gained from the qualitative data investigation were used to create an initial, two-dimensional finite-element model of the steel girder span (Span 1) using BDI’s WinGEN modeling software, which is illustrated in Figure 3.1. While the analysis focused on only Span 1, a second span was included in the model to account for the continuity effects noted in the previous section. Once the initial model was created, the load test procedures were reproduced using BDI’s WinSAC structural analysis and data correlation software. This was done by moving a two-dimensional “footprint” of the test truck across the model in consecutive load cases that simulated the designated truck paths used in the field, also shown in Figure 3.1. The analytical responses of this simulation were then compared to the field responses to validate the model’s basic structure and to identify any gross modeling deficiencies.

The model was then calibrated until an acceptable match between the measured and analytical responses was achieved. This calibration involved an iterative process of optimizing material properties and boundary conditions until they were effectively quantified. This iterative process involved using engineering judgment and modeling experience to decide which parameters likely caused the differences between the measured and computed responses. Reasonable lower and upper bounds for each of these parameters were then input into the WinSAC software, which uses a least squares curve fitting approach to minimize the error between the data and the model. Many different phases of this iterative process were run to ensure that the assumptions used to calibrate the model were correct. In the case of this structure, the majority of the calibration effort was spent modeling the effective stiffness of the slab, the girders’ end-restraint at the
supports, the spans stiffness at its edges, and the continuity between spans at the pier. This was accomplished by obtaining calibrated values for the following items:

- **Girder Bearing Details**: These bearing details were modeled as a combination of a vertical spring and horizontal spring at the center of the bearing of each girder. The stiffness of these springs was adjusted until a realistic boundary condition was achieved that modeled well the vertical girder support and the girder’s end restraint.

- **Girder Continuity Over Piers and Connection to the Piers**: The continuity between spans was modeled using axial struts between the bearing springs at the pier and simulated the interaction between the girders and the pile cap. These elements were given an effective stiffness (calibrated using an adjusted elastic modulus) and an eccentricity that best simulated the observed span interaction. These elements were only given axial resistance (E, A) to simulate the connectivity between the adjacent beam bearings.

- **Deck Stiffness**: The stiffness of the deck was optimized to accurately simulate the observed lateral load distribution. This calibration was performed by adjusting the elastic modulus of the deck shell elements.

- **Edge Stiffness**: Design details indicated that the exterior girders were braced every 37.5” with a diagonally sliced W18x35 section that was welded to the girder web and bolted to an L-section encasing the bottom and side of the slab edge. Analysis showed that the bracing allowed the exterior girders to behave partially composite with the deck and that the L-section greatly stiffened the slab edge. The girder’s composite behavior was optimized by adjusting the girder’s neutral axis eccentricity relative to the deck’s neutral axis. The slab edge was optimized by adjusting the elastic modulus of the outermost deck shell elements.
<table>
<thead>
<tr>
<th><strong>ANALYSIS TYPE</strong></th>
<th>Linear-elastic finite element - stiffness method.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MODEL GEOMETRY</strong></td>
<td>2D composed of frame elements, shell elements, and springs.</td>
</tr>
</tbody>
</table>
| **NODAL LOCATIONS** | - Nodes placed at all four corners of each shell element.  
- Nodes placed at both ends of each girder element  
- Nodes placed at all spring locations. |
| **MODEL COMPONENTS** | - Shell elements representing the deck elements.  
- Frame elements representing the girder elements, diaphragm elements, and railing.  
- Frame elements representing the interaction between the girders’ bearing conditions at the interior pier.  
- Springs representing the girders’ bearing conditions at the pier and abutments. |
| **LIVE-LOAD** | 2-D footprint of test truck consisting of vertical point loads for each wheel. Truck paths simulated by series of load cases with truck footprint moving at 1 ft increments along a straight path. |
| **TOTAL NUMBER OF RESPONSE COMPARISONS** | Round 1 & 2 Tests  
- 15 strain gage locations x 303 load positions = 4,545 strain comparisons  
- 5 displacement gage locations x 303 load positions = 1,515 displacement comparisons  
- 2 rotation gage locations x 303 load positions = 606 rotation comparisons |
| **MODEL STATISTICS** | Nodes  
- 2133  
- 2666 Elements  
- 16 Cross-section/Material types  
- 303 Load Cases (Single Truck Tests)  
- 22 Gage locations |
MODEL CALIBRATION RESULTS

Following the optimization procedures, the Round 1 final model produced a 0.9814 correlation with the Truck 1 measured responses and a 0.9807 correlation with the Tandem Truck measured responses. This match can be considered an excellent match for any steel girder structure. Additional comparisons were made with the two lane truck responses. The model accuracy values used in the initial and final bridge models are provided in Table 3.2. The error parameters account for the response comparisons between the model and the collected data, and have been briefly described in Table 3.4 for clarification.

The following are bulleted observations/conclusions made during the Round 1 optimization process:

- **Effective Deck Stiffness:** It was found that in order to best match the non-composite behavior and the lateral load distribution an effective deck modulus of 5000 ksi was needed. While this stiffness value is relatively high for deck stiffness, it accounts for the fact that the bridge is new and has not been exposed to a high frequency of loads yet that could cause cracking.

- **Girder Continuity Over Pier and Connection to the Pier and Abutment:** It was found that horizontal resistance that the pier induced on the bottom of the beams was relatively low. The axial strut simulating the connection between the adjacent beam bearings equated to approximately 10 square inches of concrete at the interior beams and 100 square inches of concrete for the exterior beams. The presence of this element in the finite element model along with axial springs at the abutments allowed some of the end-restraint provided by the pier beam bearing to be transferred between spans. Springs at the abutment were given a moderate level of eccentric axial restraint to accurately simulate the girders’ bearing condition at the abutment.

- **Exterior Girder Behavior and Slab Edge Stiffness:** Exterior girder elements were given a small amount of eccentricity relative to the deck elements to accurately simulate the partial composite behavior between the deck and the exterior girders created by the girders’ outer bracing. The slab edge elements partially encased in the L-section across the span’s length were stiffened to account for added stiffness provided by the steel.

The data from Round 2 was first compared with the final Round 1 model. It was found that in general the behavior was nearly identical, with the exception of the edge stiffness (described below). Following a second set of optimization procedures, the Round 2 final model produced a 0.9810 correlation with the HETS measured responses and a 0.9873 correlation with the M1A1 Tank measured responses. The model accuracy values used in the initial and final bridge models are provided in Table 3.3.

The following are bulleted observations/conclusions made during the Round 2 optimization process:

- **Effective Deck Stiffness:** It was found that the deck was still stiffer than typically assumed. The effective deck modulus of 5000 ksi was therefore still found to best match the recorded structural behavior. While this stiffness value is relatively high for deck stiffness, it accounts for the fact that the bridge still has not been exposed to a high frequency of loads.
- **Girder Support Behavior:** The support behavior of this span was found to remain somewhat consistent between the two rounds of tests. The observed end restraint behavior was found to be a function of vehicle weight, which increased as the vehicle weight increased. This conclusion was based on the fact that as a heavier and heavier load was used during load testing the end-restraint was found to increase. This observation provides further evidence that the end restraint is caused by friction between the girders and their bearings. It should be noted that because this end-restraint behavior was primarily friction based that it was found to be a non-linear type of behavior (i.e., the end-restraint was mostly present in directly loaded areas and therefore could not be perfectly modeled using linear spring elements).

- **Exterior Girder Behavior and Slab Edge Stiffness:** The bridge rails, which were installed after the first round of tests, were found to stiffen the exterior girder further. The exterior girder elements were therefore given additional eccentricity relative to the deck elements to account for the partial composite behavior between the deck, the bridge rail, and the exterior girders created by the girders’ outer bracing.

The final models were found to closely match the member strains, displacements, and rotations as shown in the comparison plots provided in Figure 3.2 though Figure 3.11. Note that in these comparison plots the measured responses are represented as solid lines while the computed responses are represented as discrete markers. Additionally, the model’s midspan lateral distribution of displacements and strains closely matched that of the actual structure as shown in Figure 3.12 and Figure 3.13.

### Table 3.2 – Round 1 Tests - Bridge No. 4 - Span 1 – Model Calibration Results.

<table>
<thead>
<tr>
<th>ERROR PARAMETERS</th>
<th>2012 INITIAL MODEL TRUCK 1 TESTS</th>
<th>2012 FINAL MODEL TRUCK 1 TESTS</th>
<th>2012 FINAL MODEL TANDEM TRUCK TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute Error</td>
<td>140,108.7</td>
<td>20216.3</td>
<td>8,955.4</td>
</tr>
<tr>
<td>Percent Error</td>
<td>118.0%</td>
<td>5.5%</td>
<td>4.0%</td>
</tr>
<tr>
<td>Scale Error</td>
<td>34.0%</td>
<td>2.3%</td>
<td>6.1%</td>
</tr>
<tr>
<td>Correlation Coefficient</td>
<td>0.9006</td>
<td>0.9814</td>
<td>0.9807</td>
</tr>
</tbody>
</table>

### Table 3.3 – Round 2 Tests - Bridge No. 4 - Span 1 – Model Calibration Results.

<table>
<thead>
<tr>
<th>ERROR PARAMETERS</th>
<th>2012 FINAL MODEL COMPARISON WITH HETS TESTS</th>
<th>2013 FINAL MODEL HETS TESTS</th>
<th>2013 FINAL MODEL M1A1 TANK TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute Error</td>
<td>140,108.7</td>
<td>32,591.0</td>
<td>16,127.1</td>
</tr>
<tr>
<td>Percent Error</td>
<td>6.9%</td>
<td>4.1%</td>
<td>3.8%</td>
</tr>
<tr>
<td>Scale Error</td>
<td>15.9%</td>
<td>8.1%</td>
<td>6.9%</td>
</tr>
<tr>
<td>Correlation Coefficient</td>
<td>0.9758</td>
<td>0.9810</td>
<td>0.9873</td>
</tr>
</tbody>
</table>
The following table contains the equations used to compute each of the statistical error values:

### Table 3.4 - Error Parameter Descriptions

<table>
<thead>
<tr>
<th>ERROR FUNCTION</th>
<th>EQUATION</th>
<th>BRIEF DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute Error</td>
<td>$$\sum</td>
<td>\varepsilon_m - \varepsilon_c</td>
</tr>
<tr>
<td>Percent Error</td>
<td>$$\frac{\sum (\varepsilon_m - \varepsilon_c)^2}{\sum (\varepsilon_m)^2}$$</td>
<td>Sum of the response differences squared divided by the sum of the measured responses squared. Helps provide a better qualitative measure of accuracy.</td>
</tr>
<tr>
<td>Scale Error</td>
<td>$$\frac{\sum \max</td>
<td>\varepsilon_m - \varepsilon_c</td>
</tr>
<tr>
<td>Correlation Coefficient</td>
<td>$$\frac{\sum (\varepsilon_m - \varepsilon_m)(\varepsilon_c - \varepsilon_c)}{\sqrt{\sum (\varepsilon_m - \varepsilon_m)^2}(\varepsilon_c - \varepsilon_c)^2}}$$</td>
<td>Measure of the linearity between the measured and computed data. Helps determine the error with respect to response shape and phase.</td>
</tr>
</tbody>
</table>
Figure 3.2 – Round 1 Tests - Final steel girder span model – Truck 1 along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on D-D.

Figure 3.3 - Round 2 Tests - Final steel girder span model – HETS along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on D-D.
Figure 3.4 - Round 1 Tests - Final steel girder span model – Truck 1 along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on E-E.

Figure 3.5 - Round 2 Tests - Final steel girder span model – M1A1 Tank along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on E-E.
Figure 3.6 – Round 1 Tests - Final steel girder span model – Truck 1 along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on F-F.

Figure 3.7 – Round 2 Tests - Final steel girder span model – M1A1 Tank along Truck Paths Y1, Y2, & Y3 - Example strain comparison plot at gage point on F-F.
Figure 3.8 - Round 1 Tests - Final steel girder span model – Truck 1 along Truck Paths Y1, Y2, & Y3 - Example displacement comparison plot at gage point on E-E.

Figure 3.9 - Round 2 Tests - Final steel girder span model – M1A1 Tank along Truck Paths Y1, Y2, & Y3 - Example displacement comparison plot at gage point on E-E.
Figure 3.10 - Round 1 Tests - Final steel girder span model – Truck 1 - Example rotation comparison plot at gage point on Section D-D.

Figure 3.11 - Round 2 Tests - Final steel girder span model – M1A1 Tank - Example rotation comparison plot at gage point on Section D-D.
Figure 3.12 - Round 1 Tests - Final steel girder span model – Truck 1 – Lateral load distribution strain comparison along E-E (All Paths).

Figure 3.13 - Round 2 Tests - Final steel girder span model – M1A1 Tank - Lateral load distribution displacement comparison along E-E (All Paths).
4. LOAD RATING PROCEDURES & RESULTS

**RATING PROCEDURES**

Load rating was performed on the non-composite steel girders in accordance with AASHTO and KYDOT LRFR guidelines. Structural responses were obtained from a slightly modified version of the final calibrated model, and member capacities were based on design details and AASHTO LRFD specifications. The rating methods used in BDI’s approach closely match typical rating procedures, with the exception that a field-verified finite-element model analysis was used rather than a typical AASHTO beam-line/distribution factor approach. This section briefly discusses the methods and findings of the load rating procedures.

Once the analytical model was calibrated to produce an acceptable match to the measured responses, the model was adjusted to ensure the reliability of all optimized model parameters. This adjustment involved the identification of any calibrated parameters that could change over time or could become unreliable under heavy loads.

In the analysis of the Bridge No. 4’s steel girder span, the secondary stiffness parameters that were determined to be unreliable for long-term bridge behavior were the end-restraint provided at the abutment, the continuity between spans and the partially composite behavior of the exterior girders. This model adjustment along with making the steel girders completely non-composite generated much more realistic ultimate loading behavioral responses and in turn a more conservative load rating. Note that the only observed changes between the first and second round of testing/model calibration fit into this secondary stiffness category. Therefore the ratings previously provided in January 2013 were found to still be valid.

Member capacities were calculated based on design drawings and AASHTO specifications. Serviceability and fatigue capacities for the steel girder were based on the relevant stress limits specified by AASHTO. However, these stress limits were converted to moment capacities since the structural responses generated by BDI’s analysis software (WinSAC) are force and moment based. A summary of the calculated beam capacities used in strength, serviceability, and fatigue ratings, as well as relevant member properties have been provided in Table 4.1 and Table 4.2 while detailed capacity calculations have been provided in Appendix A.

**Table 4.1 Steel Girder Moment Capacities (ΦMn, k-in).**

<table>
<thead>
<tr>
<th>MEMBER / LOCATION</th>
<th>L_B (FT)</th>
<th>L_P (FT)</th>
<th>L_R (FT)</th>
<th>F_Y (KSI)</th>
<th>S_X (IN^3)</th>
<th>M_Y (KIP-IN)</th>
<th>M_N (KIP-IN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder/ Midspan (Strength)</td>
<td>14.25</td>
<td>5.9</td>
<td>24.24</td>
<td>50</td>
<td>406</td>
<td>20300</td>
<td>19001</td>
</tr>
<tr>
<td>Girder/ Midspan (Serviceability)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>50</td>
<td>406</td>
<td>-</td>
<td>16240</td>
</tr>
<tr>
<td>Girder/ Midspan (Fatigue)</td>
<td>-</td>
<td>-</td>
<td>(ΔF)_TH</td>
<td>406</td>
<td>-</td>
<td>6496</td>
<td></td>
</tr>
</tbody>
</table>

Note: (ΔF)_TH = 16
Table 4.2 Steel Girder Shear Capacity (ΦVn, kips).

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>WEB DEPTH, IN</th>
<th>WEB WIDTH, IN.</th>
<th>FV (KSI)</th>
<th>VN (KIP)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Girder</td>
<td>31.375</td>
<td>0.5625</td>
<td>50</td>
<td>512</td>
</tr>
</tbody>
</table>

Load ratings were performed using the final adjusted rating model according to the AASHTO Manual for Bridge Evaluation, 2011 Edition and KYDOT TM08-01 (see Table 4.3 for applied rating factors). It should be noted that all modeled elements making up the steel girders were considered for rating. Given the 27’-8” wide roadway, one-lane and two-lane loading conditions were considered for the HL-93 strength and serviceability ratings. Only the single lane loading condition was considered for the fatigue rating, HETS ratings, and the MLC ratings. A multiple presence factor of 1.2 was applied to the loads for single lane loaded strength and serviceability limit states.

Figure 4.1 shows the load configurations and weights for the AASHTO standard load rating vehicles, the two HETS military design loads, and the controlling MLC tracked and wheeled load configurations. As specified in TM08-01, the HL-93 vehicle loads were increased by 25%. All structural component dead loads were automatically applied by the modeling program’s self-weight function. In addition to the superstructure’s self-weight, a uniform weight of 60 psf was applied to the roadway as the future wearing surface as specified in TM08-01 and a load of 295 psf was applied as the barrier weight.

The HL-93 load condition was rated for the strength and serviceability limit states at both an inventory and operating level, and for the fatigue limit state at an inventory level. The HETS M1070/M1000 and HETS M1070/M747 loads were rated for the strength limit state at both an inventory and operating level since the design plans indicated the structure was designed for these loads. Finally, the controlling MLC vehicle classes for the tracked and wheeled load cases for the steel girder span (as well as the hybrid composite beam span) was determined by rating the structure for the strength limit state at an operating level for the M1A1 Abrams tracked load and for the MLC70 wheeled load. These loads were used to determine the MLC vehicle classes instead of performing an iterative process because of the manner in which the loads are applied to the finite element rating model. Generally, all MLC loads are applied to a girder line analysis with the use of the same distribution factor for each of the loads. However, when applied to the 2-D finite element model, the magnitude and distribution of the load over the vehicle’s size causes different load distribution characteristics on the bridge model that weren’t accounted for in the creation of the MLC vehicles. Therefore, the M1A1 Abrams tracked load and MLC70 wheeled load were chosen as baseline vehicles to maintain consistent distribution regardless of what load was applied to the vehicle footprints.
### Table 4.3 Applied LRFR Rating factors.

<table>
<thead>
<tr>
<th>Factor Type</th>
<th>Description</th>
<th>Factor Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>AASHTO Load Factors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength Limit State</td>
<td>Dead Load – DC</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>Dead Load – DW</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>Live Load – Inventory</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>Live Load – Operating</td>
<td>1.35</td>
</tr>
<tr>
<td>Service Limit State</td>
<td>Dead Load – DC</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Dead Load – DW</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Live Load – Inventory</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Live Load – Operating</td>
<td>1.0</td>
</tr>
<tr>
<td>Fatigue Limit State</td>
<td>Dead Load – DC</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Dead Load – DW</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Live Load – Inventory</td>
<td>0.75</td>
</tr>
<tr>
<td>Impact Factor (AASHTO Table 3.6.2.1.1)</td>
<td></td>
<td>33%</td>
</tr>
<tr>
<td>HL-93 Truck Load Amplification</td>
<td></td>
<td>25%</td>
</tr>
<tr>
<td>(TM08-01 3.6.1.2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatigue Impact Factor</td>
<td></td>
<td>15%</td>
</tr>
<tr>
<td>(AASHTO Table 3.6.2.1.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>AASHTO Strength Reduction Factors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexure (Moment) in Non-Composite Steel Sections</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>Shear in Non-Composite Steel Sections</td>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>
Figure 4.1 Bridge #4 load rating vehicle configurations.
**Rating Results**

The following is a summary of the controlling load rating factors and responses for all rated vehicles on the Bridge #4 steel girder span. Ratings were calculated using a post processor that assembled the critical responses generated from the bridge model for the different load components used in the AASHTO LRFR rating equation: component dead load (DC), superimposed dead load (DW), and live-load (LL).

The steel girders met both Inventory and Operating rating criteria (RF>1.0) for all load configurations and rating criteria for HL-93, as shown in Table 4.4 and Table 4.5. The critical rating factor for HL-93 loading condition was controlled by positive flexure of the center girder at midspan. The HL-93 load rating was controlled by the two lanes loaded condition of the Design Tandem + Lane load configuration centered on the bridge. Under the fatigue loading condition and all military loads, the controlling member was the first interior girder because the single lane edge loading most directly loaded this beam.

The steel girder span’s controlling tracked vehicle load limit was MLC184 based on an M1A1 Abrams operating rating of 2.55. However, the hybrid composite beam (HCB) span had a lower tracked MLC rating 139 tons and therefore controlled Bridge #4's load capacity. The controlling responses for the HCB span are provided in the tables below. Additionally, tonnage ratings are provided in Table 4.6 for all rated military loads.

Summary files and output files containing rating results of all rated elements for all loading conditions and applicable limit states have been included with this report in the submittal files. Please note that live load amplification factors including the impact factor, the TM08-01 based HL-93 load amplification, and the multiple presence factors were applied by BDI’s analysis software (WinGen and WinSAC) before the results were post processed. This means that these factors do not appear in the output files but were considered for rating. Alternatively, the live and dead load factors used in the rating calculations were applied within the analysis result post processor program and are listed in the output files. It is important to mention this because the impact factor listed in the post processed output is shown as 0.0 since it was already applied in the previous step.
Table 4.4 Controlling rating factors and responses - Girders in positive flexure.

<table>
<thead>
<tr>
<th>LOADING CONDITION</th>
<th>CONTROLLING LOCATION</th>
<th>DC MOMENT, KIP-IN</th>
<th>DW MOMENT, KIP-IN</th>
<th>LL MOMENT, KIP-IN</th>
<th>INVENTORY RF</th>
<th>OPERATING RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93 (Strength)</td>
<td>Center Steel Girder/ At Midspan</td>
<td>1441.90</td>
<td>754.34</td>
<td>2854.06</td>
<td>2.42</td>
<td>3.14</td>
</tr>
<tr>
<td>HL-93 (Service)</td>
<td>Center Steel Girder/ At Midspan</td>
<td>1441.90</td>
<td>754.34</td>
<td>2854.06</td>
<td>2.85</td>
<td>3.70</td>
</tr>
<tr>
<td>HL-93 (Fatigue)</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>0.0</td>
<td>0.00</td>
<td>1325.22</td>
<td>5.68</td>
<td>-</td>
</tr>
<tr>
<td>HETS M1070/ M1000</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>1392.20</td>
<td>778.57</td>
<td>2754.21</td>
<td>2.49</td>
<td>3.23</td>
</tr>
<tr>
<td>HETS M1070/ M747</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>1392.20</td>
<td>778.57</td>
<td>3236.24</td>
<td>2.14</td>
<td>2.77</td>
</tr>
<tr>
<td>M1A1 Tracked</td>
<td>Hybrid Span Deck Force/ At Midspan</td>
<td>1.36 kip/in</td>
<td>0.61 kip/in</td>
<td>4.64 kip/in</td>
<td>-</td>
<td>2.64</td>
</tr>
<tr>
<td>MLC70 Wheeled</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>1392.20</td>
<td>778.57</td>
<td>3382.00</td>
<td>-</td>
<td>3.52</td>
</tr>
</tbody>
</table>

Note: Dead Load Responses are unfactored. Live Load Responses have applicable multiple presence factors applied but not the impact factor. HL-93 responses account for 25% load amplification on the Truck Load.
Table 4.5 Controlling rating factors and responses - Girders in shear.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Controlling Location</th>
<th>DC Shear, kip</th>
<th>DW Shear, kip</th>
<th>LL Shear, kip</th>
<th>Critical Inventory RF</th>
<th>Critical Operating RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93 (Strength)</td>
<td>Center Steel Girder/ Near End</td>
<td>13.06</td>
<td>5.73</td>
<td>32.74</td>
<td>6.39</td>
<td>8.29</td>
</tr>
<tr>
<td>HETS M1070/ M1000</td>
<td>Interior Steel Girder/ Near End</td>
<td>13.01</td>
<td>5.77</td>
<td>33.00</td>
<td>6.34</td>
<td>8.20</td>
</tr>
<tr>
<td>HETS M1070/ M747</td>
<td>Interior Steel Girder/ Near End</td>
<td>13.01</td>
<td>5.77</td>
<td>37.25</td>
<td>5.62</td>
<td>7.28</td>
</tr>
<tr>
<td>M1A1 Tracked</td>
<td>Hybrid Girder/ GRFP Web</td>
<td>0.24 kip/in</td>
<td>0.20 kip/in</td>
<td>1.20 kip/in</td>
<td>-</td>
<td>1.94</td>
</tr>
<tr>
<td>MLC70 Wheeled</td>
<td>Hybrid Girder/ GRFP Web</td>
<td>0.19 kip/in</td>
<td>0.22 kip/in</td>
<td>0.98 kip/in</td>
<td>-</td>
<td>3.04</td>
</tr>
</tbody>
</table>

Note: Dead Load Responses are unfactored. Live Load Responses have applicable multiple presence factors applied. HL-93 responses account for 25% load amplification.

Table 4.6 Controlling tonnage rating factors for all military loads.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Controlling Location</th>
<th>Critical Inventory RF (Tons)</th>
<th>Critical Operating RF (Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HETS M1070/ M1000</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>286</td>
<td>372</td>
</tr>
<tr>
<td>HETS M1070/ M747</td>
<td>Interior Steel Girder/ At Midspan</td>
<td>225</td>
<td>291</td>
</tr>
<tr>
<td>M1A1 Tracked</td>
<td>Hybrid Girder GFRP Web</td>
<td>-</td>
<td>139</td>
</tr>
<tr>
<td>MLC70 Wheel</td>
<td>Hybrid Girder/ GFRP Web</td>
<td>-</td>
<td>212</td>
</tr>
</tbody>
</table>
5. CONCLUSIONS AND RECOMMENDATIONS

In general, the response data recorded during the load tests was found to be of good quality. Responses collected during the first occurrence of a heavier load indicated some girder movement at the bearing and bridge rail locations. This behavior was most likely a function of friction between the elastomeric bearing pads on the bottom of the beams. A small amount of movement can be expected with heavy vehicles and significant temperature changes. The responses were observed during the first crossing of each heavy vehicle and disappeared with repeated tests.

A two-dimensional finite element model of the steel girder structure was created using the collected structural information, and subsequently calibrated until an acceptable match between the measured and analytical responses was achieved. An excellent correlation between the measured and computed response was obtained during the modeling process. A comparison between the final 2012 calibrated model and the data collected during the second round of tests indicated that the structural behavior did not significantly change over the years’ time.

The bridge and the resulting model exhibited excellent lateral load transfer characteristics and a small level of continuity between spans. In general, the non-composite slab and girders behaved as expected. A small amount of friction induced end-restraint was observed during both rounds of tests. Additionally, the exterior girders behaved partially composite with the edge of the slab and guard rail due to the connection detail. It was assumed the end-restraint and partially composite behavior may not exist indefinitely, particularly with heavy loads approaching the bridges ultimate capacity. Therefore the calibrated model was altered by removing these secondary stiffness parameters to provide a more conservative load-rating model.

Load ratings were computed using an altered version of the calibrated model that was considered to be more reliable for rating. In all cases the rating results for the girders were controlled by flexure of the interior beams. The beams met rating criteria (RF>1.0) for all specified loads and rating levels. Additionally, the controlling MLC ratings were 183 tons for a tracked vehicle on the steel girder span. However, the HCB span was limited by a 139 ton tracked vehicle and 212 ton wheeled vehicle, which controlled Bridge #4’s MLC ratings.

For additional information about BDI’s integrated approach (testing, modeling and rating procedures), supporting documents are available at www.bridgetest.com. The load test, structural investigation, and load rating results presented in this report correspond to the structure at the time of testing. Any structural degradation, damage, and/or retrofits must be taken into account in future ratings.
LRFD NONCOMPOSITE GIRDER CAPACITY
Structure: Fort Knox Steel Beam Span (W33x130)
Location: Moment - At Midspan, Shear - Near Support

Beam Dimensions:
- Beam Depth: \( d = 33.125 \text{ in} \)
- Flange Thickness: \( t_f = 0.875 \text{ in} \)
- Flange Width: \( b_f = 11.5 \text{ in} \)
- Web Depth: \( D = d - 2t_f = 31.375 \text{ in} \)

Depth of Web in Compression: \( D_c = \frac{D}{2} = 15.6875 \text{ in} \)
Distance Between Web Centerlines: \( h = d - t_f = 32.25 \text{ in} \)

Web Width: \( t_w = 0.5625 \text{ in} \)
Unbraced Length: \( L_b = 14.25 \text{ ft} \)

Beam Properties:
- Steel Yield Strength: \( F_y = 50 \text{ ksi} \)
- Compression Flange Stress at Yield Onset: \( F_{uy} = 0.7 F_y = 35 \text{ ksi} \)
- Modulus of Elasticity: \( E = 290000 \text{ ksi} \)
- Major Axis Elastic Section Modulus: \( S_x = 406 \text{ in}^2 \)
- Major Axis Plastic Section Modulus: \( Z_x = 467 \text{ in}^3 \)
- Moment of Inertia of Compression Flange: \( I_{yc} = \frac{(t_f b_f^3)}{12} = 110.9 \text{ in}^4 \)
- Moment of Inertia of Tension Flange: \( I_{yt} = \frac{(t_f b_f^3)}{12} = 110.9 \text{ in}^4 \)
- Torsional Constant: \( J_{beam} = 7.37 \text{ in}^4 \)
- Radius of Gyration: \( r_t = 2.94 \text{ in} \)
Strength Limit State Moment Capacity Calculations:

\[
\text{CHECK}_{WS} = \left \{ \begin{array}{ll} 
\frac{2D_y}{t_W} & < 5.7 \sqrt{F_y} \cdot 1.0 \\
1 & \end{array} \right. - 1 \\
\text{CHECK}_{FS} = \left \{ \begin{array}{ll} 
\frac{1}{t_W} & \geq 0.3, 1.0 \\
1 & \end{array} \right. - 1
\]

(AASHTO LRFD 6.10.6.2-1

Capacity = if (CHECK\(_{WS} = 1 \land CHECK\(_{FS} = 1", \text{"Appendix A6 Capacity"} \), 6.10.2 Capacity") = "Appendix A6 Capacity"

Web Plastification Factor Calculations:

**Beam Yield Moment:**

\[M_y = F_y S_y = 20300 \text{ kip-in} \]

**Beam Plastic Moment:**

\[M_p = F_y Z_y = 23550 \text{ kip-in} \]

\[\beta = \frac{(2D_c t_w)}{(t_r t_c)} = 1.754 \]

\[\rho = 1 \]

**Hybrid Factor:**

\[f_h = \left\{ \begin{array}{ll}
12 + \beta (3.0 - \rho) & (12 + 2.0) \\
1 & \end{array} \right. \]

(AASHTO LRFD 6.10.1.10.1-1

**Depth of Web in Compression at Plastic Moment:**

\[D_{cp} = \left( \frac{D}{2} \right) \left[ \left( \frac{(F_y Z_y t_f - F_y Z_y t_c)}{F_y D t_w} \right) + 1 \right] = 15.675 \text{ in} \]

(AASHTO LRFD D6.3.2-1

**Limiting Slenderness Ratio for Compact Web:**

\[
\lambda_{pw1} = \frac{\frac{E}{F_y}}{0.54 \left( \frac{F_y}{F_{zh} M_y} \right) - 0.09} = 45.243
\]

(AASHTO LRFD A6.2.1-2

**Limiting Slenderness Ratio for Noncompact Web:**

\[
\lambda_{TW} = \frac{5.7 \sqrt{F_y}}{137.374} = 137.274
\]

\[
\lambda_{pw2} = \lambda_{TW} \left( \frac{D_{cp}}{D_c} \right) = 137.274
\]

\[
\lambda_{pw} = \min(\lambda_{pw1}, \lambda_{pw2}) = 45.243
\]

\[
\lambda_{W} = \frac{(2D_c)}{t_w} = 55.778
\]

Compact Check: if (\(\lambda_{pw} < \lambda_{W}" \), "COMPACT", "NONCOMPACT") = "NONCOMPACT"
Web Plastification Factor: AASHTO LRFD A6.2.2.4

\[ R_p = \left[ \text{Compact/Check} = "COMPACT", \left( \frac{M_p}{M_y} \right), \text{and} \left[ 1 - \left( \frac{M_p}{M_y} \right) \left( \frac{M_p}{M_y} \right) \right] \left( \frac{\lambda_{ty} - \lambda_{pw}}{\lambda_{pw}} \right) \left( \frac{M_p}{M_y} \right) \left( \frac{M_p}{M_y} \right) \right] = 1.133 \]

Local Buckling Resistance:

Compression Flange Slenderness Ratio:

\[ \lambda_{xf} = \frac{b_f}{(2 - t_f)} = 6.571 \] AASHTO LRFD A6.3.2.3

Compact Flange Limiting Slenderness Ratio:

\[ \lambda_{pf} = 0.38 \cdot \frac{E}{F_y} = 9.152 \] AASHTO LRFD A6.3.2.4

Flange Local Buckling Coefficient:

\[ k_c = 0.76 \] AASHTO LRFD A6.3.2.6

Noncompact Flange Limiting Slenderness Ratio:

\[ \lambda_{nf} = 0.95 \cdot \frac{E \cdot k_c}{F_y} = 23.839 \] AASHTO LRFD A6.3.2.5

Local Buckling Resistance:

\[ M_{lc1} = \min \left[ \lambda_{tf} < \lambda_{pf}, R_p M_y \left[ 1 - \left( \frac{F_{yw} S_x b}{R_p M_y (\lambda_{nf} - \lambda_{pf})} \right) \frac{R_p M_y}{M_y} \right] \right] = 23003.81 \text{ kip-in} \] AASHTO LRFD A6.3.2.1.2

Lateral Torsional Buckling Resistance:

Limiting Lengths:

\[ L_t = 1.95 \cdot \frac{E}{F_y} \cdot \sqrt{\frac{S_y b}{S_x h}} \left[ 1 + \sqrt{1 + 6.78 \left( \frac{(S_y b)}{S_x h} \right) \left( \frac{F_{yw} S_x b}{E \cdot b_{beam}} \right)} \right] = 24.244 \text{ ft} \] AASHTO LRFD A6.3.3.5

\[ L_p = 1.3 \cdot \frac{E}{F_y} = 5.8 \text{ ft} \] AASHTO LRFD A6.3.3.4

\[ L_b < L_p \Rightarrow M_{lc2a} = R_p M_y = 23003.81 \text{ kip-in} \] AASHTO LRFD A6.3.3.1

\[ L_p < L_b < L_t \Rightarrow M_{lc2b} = \left[ 1 - \left( \frac{F_{yw} S_x b}{R_p M_y} \right) \frac{(L_b - L_p)}{L_p} \right] \frac{R_p M_y}{M_y} = 19000.98 \text{ kip-in} \] AASHTO LRFD A6.3.3.2
\[ F_{cr} = \left( \frac{t_{b}^{2}E}{I_{b}} \right) \left[ 1 + 0.078 \left( \frac{I_{b}}{S_{y} t_{b}} \right) \left( \frac{L_{b}}{R} \right)^{2} \right] = 90.67 \text{ ksi} \]  
\[ M_{cr2c} = F_{cr} S_{x} = 36812.62 \text{ kip-in} \]

**Lateral Torsional Buckling Controlling Resistance:**

\[ M_{cr2c} = \max \{ (L_{b} < L_{p}, M_{cr2a}, (L_{b} > L_{p}, M_{cr2c}, M_{cr2b})) \} = 19000.98 \text{ kip-in} \]

**Tension Flange Yielding Resistance:**

**Tension Flange Yield Moment:**

\[ M_{yt} = F_{pe} M_{p} = 25003.81 \text{ kip-in} \]  
\[ \text{AASHTO LRFD A6.4.1} \]

**Nominal Moment Strength:**

\[ M_{n} = \min \{ M_{cr1}, M_{cr2}, M_{nt} \} = 19000 \text{ kip-in} \]

**Factored Moment Strength:**

\[ \phi M_{n} = 1.0 \]  
\[ 0.9 M_{n} = \phi_{f} M_{n} = 19000 \text{ kip-in} \]  
\[ \text{AASHTO LRFD A6.5.4.2} \]

**Serviceability Moment Resistance:**

\[ M_{service} = 0.8 F_{pe} F_{y} S_{x} = 16240 \text{ kip-in} \]

**Fatigue Moment Resistance:**

\[ \text{Fatigue strength} = 16 \text{ ksi} S_{x} = 6496 \text{ kip-in} \]

**Shear Strength Calculations:**

**Shear Buckling Coefficient:**

\[ k = 5 \]  
\[ \text{AASHTO LRFD 6.10.9.2} \]

**Depth to Thickness Ratio Limits:**

\[ \lim_{1} = 1.12 \left( \frac{E t_{w}}{F_{y}} \right) = 60.31 \]  
\[ \text{AASHTO LRFD 6.10.9.3.2-4} \]

\[ \lim_{2} = 1.40 \left( \frac{E t_{w}}{F_{y}} \right) = 75.39 \]  
\[ \text{AASHTO LRFD 6.10.9.3.2-5} \]

**Depth to Thickness Ratio:**

\[ \text{web ratio} = \frac{D}{t_{w}} = 55.78 \]  
\[ \text{AASHTO LRFD 6.10.9.3.2} \]
Web Shear Coefficient:
\[ C_v = \begin{cases} \text{if } \text{web ratio} < \text{lim}_1, & 1.0, \text{if } \text{web ratio} > \text{lim}_2 \end{cases} \]
\[ \left[ \frac{1.57}{D} \left( \frac{E \cdot k}{F_y} \right) \right] \left[ \frac{1.12}{D} \left( \frac{E \cdot k}{F_y} \right) \right] = 1 \]
\[ \text{AASHTO LRFD 6.10.9.3.2} \]

Plastic Shear Force:
\[ V_p = 0.58 \cdot F_y \cdot D \cdot t_w = 511.8 \text{ kip} \]
\[ \text{AASHTO LRFD 6.10.9.2-2} \]

Nominal Shear Strength:
\[ V_n = C_v \cdot V_p = 511.8 \text{ kip} \]
\[ \text{AASHTO LRFD 6.10.9.2-1} \]

Factored Shear Strength:
\[ \phi V_p = \phi V_n = 512 \text{ kip} \]
B. APPENDIX B - REFERENCES

For all references to BDI’s equipment, services, and analysis/rating methods please go to our website:
www.bridgetest.com


KYDOT, (2010). "Transmittal Memorandum 08-01"
The Army has 1,500 vehicular bridges on its installations, with many experiencing high maintenance or replacement costs due to corrosion of the steel structure or of the reinforcing bar in the concrete deck. Under the Department of Defense Corrosion Prevention and Control Program (Project F12-AR01), a span of Bridge No. 4 at Fort Knox, Kentucky, was selected to demonstrate and validate a fiber reinforced polymer (FRP) composite grid deck system, which does not use any reinforcement steel. The results of that project were published as ERDC/CERL TR-16-21 (September 2016). Bridge Diagnostics, Inc. (BDI) of Boulder, Colorado, was subcontracted to perform load testing and rating for the newly completed composite bridge deck system to confirm that the bridge span met its required load rating and performance criteria for deflection and strain. Structural testing of the bridge was performed at the end of construction and after 12 months of exposure. Results showed the bridge met all design specifications and load ratings.