DoD Corrosion Prevention and Control Program

Field Testing and Summary Report for Road 5 (Morris Road) Over Road 3 (Toftoy Throughway) at Redstone Arsenal, AL

Contractor’s Supplemental Report for Project F09-AR16

Brett Commander and Scott Aschermann

August 2016

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Field Testing and Summary Report for Road 5 (Morris Road) Over Road 3 (Toftoy Throughway) at Redstone Arsenal, AL

Contractor’s Supplemental Report for Project F09-AR16

Brett Commander and Scott Aschermann

Bridge Diagnostics, Inc. (BDI)
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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 Purchase Order BDI0001 from Mandaree Enterprise Corporation under Contract W9132T-06-D-0001, Delivery Order 0067, “Lightweight FRP Composite Bridge Deck Replacement of a Concrete Deck at Redstone Arsenal, Alabama” for Project F09-AR16, “Demonstration and Validation of a Lightweight Composite Bridge Deck Technology as an Alternative to Reinforced Concrete”

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

Cyclic loading and weathering of reinforced concrete bridge decks causes corrosion of reinforcement steel, which leads to cracking, potholes, and other problems. Under the Department of Defense Corrosion Prevention and Control Program (Project F09-AR16), a deteriorated concrete bridge at Redstone Arsenal, Alabama, was selected to demonstrate and validate a glass-fiber reinforced polymer (GFRP) composite deck system, which does not use any reinforcement steel. The results of that project were published as ERDC/CERL TR-16-6 (August 2016). Upon completion of the new GFRP composite deck system, Bridge Diagnostics, Inc. (BDI) was contracted to perform load testing to confirm that the bridge meets the structure’s original 36-ton (HS-20) load rating and performance criteria for deflection and strain. This report documents the load test methods used by BDI and the results. The test results indicate that the demonstrated GFRP composite deck system met the strength design specifications and passed the deflection criteria.

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Foreword

Bridge Diagnostics, Inc. (BDI) was subcontracted by Mandaree Enterprise Corporation (MEC), of Warner Robins, Georgia, to perform load testing on Bridge 18, Morris Road (Road 5) Over Toftoy Throughway (Road 3) at Redstone Arsenal, Huntsville, Alabama. 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 a glass-fiber reinforced polymer (GFRP) composite deck system to repair the reinforced concrete deck on Bridge 18, which had begun to fracture and spall as a result of reinforcement steel corrosion. The technology was selected for demonstration and validation under Project F09-AR16 of the Department of Defense Corrosion Prevention and Control (CPC) Program. The final technical report on that project was published as ERDC/CERL TR-16-16 (August 2016). The current report, which is incorporated into the technical report by reference, provides complete details on the contractor’s execution of the load-testing program.

The primary goal of BDI’s live load testing was to determine whether the GFRP 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. under Purchase Order BDI0001 from Mandaree Enterprise Corporation under Contract W9132T-06-D-0001, Delivery Order 0067, “Lightweight FRP Composite Bridge Deck Replacement of a Concrete Deck at Redstone Arsenal, Alabama” for Project F09-AR16, “Demonstration and Validation of a Lightweight Composite Bridge Deck Technology as an Alternative to Reinforced Concrete.” 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. Richard G. Lampo, 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 Topudurti, 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 SUMMARY REPORT:
ROAD 5 (MORRIS RD) OVER ROAD 3 (TOFTOY THWY) REDSTONE ARSENAL, AL

SUBMITTED TO:
Mandaree Enterprise Corporation
812 Park Drive
Warner Robins, GA 31088
478.329.8233

SUBMITTED BY:
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1965 57th Court North, Suite 106
Boulder, CO 80301
303.494.3230
www.bridgetest.com

April, 2010
EXECUTIVE SUMMARY

In January of 2010, Bridge Diagnostics, Inc. (BDI) was contracted by Mandaree Enterprise Corporation (MEC) to perform load testing on Bridge 18, Morris Road (Road 5) over Toftoy Throughway (Road 3) at Redstone Arsenal, Huntsville, Alabama. The primary goal of the live load testing was to obtain and then utilize field measurements to determine whether or not the FRP composite deck met design specifications for deflection and strain. This report outlines the testing procedures, provides a detailed discussion of the data collected, and summarizes the subsequent findings.

The field work phase of this project was completed on March 25th, 2010 despite inclement weather including rain and wind. The BDI Wireless Structural Testing System (STS-WiFi) was used for measuring strains at 14 locations and displacements at 9 locations on the deck and superstructure while it was subjected to a moving truck load at several lateral positions. The response data was examined and evaluated in a qualitative manner, and then extrapolated to determine the responses induced by an AASHTO HS-20 design vehicle plus impact (33% per LRFD).

Please note that the results that follow are based on an estimated load distribution between the test truck’s three axles, and may not be entirely accurate. Because individual axle weights could not be obtained (only the truck’s gross weight), the results can only provide a reasonable measure of the deck meeting design criteria rather than an absolute conclusion, and must be treated as so.

The test vehicle was weighed offsite, and was determined to have a gross weight of 78.66 kips. Based on a thorough examination of the field data, it was estimated that the front axle weighed approximately 22 kips, leaving each rear axle at approximately 28.3 kips. In order to extrapolate to the HS-20 design live loading, all front axle responses were multiplied by a factor of 0.364 (8 kips / 22 kips) and all rear axle responses were multiplied by a factor of 1.13 (32 kips / 28.3 kips).

The design specification for strain, provided to BDI by ZellComp, stated that “the strains in the panels under full dead load and design live load shall not exceed twenty (20) percent of the strain at the ultimate capacity of the FRP material” and that “the strains in the panels under dead load alone shall not exceed ten (10) percent of the strains at the ultimate capacity of the FRP material.” Based on the live load test results and subsequent calculations, maximum strains incurred by HS-20 live loading and dead load would be in the range of 800-900 µε, which is significantly smaller than the 3,220 µε limit. Even taking into account possible test truck axle-load distribution discrepancies, it is certain that the FRP deck panels met the strength design specifications.

The design specification for deflection stated that “the deck deflection due to live loads plus impact shall be limited to L/500, where L is the distance between the centerline of adjacent girders.” Based on the live load test results and subsequent calculations, deck deflection incurred by HS-20 live loading plus impact would be 0.14 inches, which is approximately 87% of the deflection limit of 0.16 inches. While there is some question as to the actual load test axle weights, it is likely that the estimated weights were within 10 percent. Therefore, the load test results indicated that the deck passed the deflection criteria.
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1. STRUCTURAL TESTING PROCEDURES

Bridge 18 at Redstone Arsenal is a bridge deck replacement project located at the intersection of Morris and Toftoy Roads in Huntsville, Alabama. The superstructure consists of four original wide flange beams (W33x118) and a new 7” ZellComp Composite FRP Deck with a ½” bituminous wearing surface. The four bridge spans are continuous, and divided into two end spans of 40’-0”, an interior span of 64’-4½”, and an interior span of 52’-4½”. The overall bridge width is 22’-5”, with a curb-to-curb roadway width of 20’-0”.

The superstructure (beams and deck) was instrumented with 14 strain transducers and 9 Linear Varying Differential Transformer (LVDT) displacement sensors, as shown in Figure 1.1 through Figure 1.9. The intent of the instrumentation was to measure the deck strain and deflection relative to the steel girders. LVDT’s were mounted to tri-pods, which in turn were placed on the concrete sloped apron of the abutment. Semi-static load tests were performed with a 3-axle dump truck (speed approximately 5 mph) at three lateral load positions. Strain measurements and truck position were recorded continuously at a sample rate of 40Hz as the test truck was driven across the bridge at crawl speed.

Information specific to this load test can be found in Table 1.1 and the field notes in Appendix B. The test vehicle’s gross-weight and wheel rollout distance (required for tracking its position across the structure) are provided in Table 1.2. A “footprint” of the vehicle is also shown in Figure 1.10 for reference. The vehicle weight was obtained off-site and provided to BDI by Angelo Iafrate Construction and MEC.

Please see Appendix C for an outline of the general field testing procedures, and Appendix D for the specifications on the strain transducers and the wireless structural testing system.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>DESCRIPTION</th>
</tr>
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<tbody>
<tr>
<td>STRUCTURE NAME</td>
<td>Redstone Arsenal FRP Bridge</td>
</tr>
<tr>
<td>BDI Reference Number</td>
<td>100101AL</td>
</tr>
<tr>
<td>TESTING DATE</td>
<td>March 25th, 2010</td>
</tr>
<tr>
<td>CLIENT’S STRUCTURE ID #</td>
<td>Bridge 18</td>
</tr>
<tr>
<td>LOCATION/ROUTE</td>
<td>Road 5 (Morris) over Road 3 (Toftoy)</td>
</tr>
<tr>
<td></td>
<td>Redstone Arsenal, Huntsville, Alabama</td>
</tr>
<tr>
<td>STRUCTURE TYPE</td>
<td>Steel girders with FRP composite deck</td>
</tr>
<tr>
<td>BEAMS/BEAM SPACING</td>
<td>4, 4-span continuous W33x118 / 3 spaces @ 6’-8”</td>
</tr>
<tr>
<td>DECK</td>
<td>7” ZellComp Composite FRP, continuous</td>
</tr>
<tr>
<td>TOTAL NUMBER OF SPANS</td>
<td>4</td>
</tr>
<tr>
<td>SPAN LENGTHS</td>
<td>End Spans: 40’-0”</td>
</tr>
<tr>
<td></td>
<td>Interior Spans: 64’-4½” and 52’-4½”</td>
</tr>
<tr>
<td>SKEW</td>
<td>None</td>
</tr>
<tr>
<td><strong>Structure/Roadway Widths</strong></td>
<td>Structure: 22’-5” / Roadway: 20’-0”</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>--------------------------------------</td>
</tr>
<tr>
<td><strong>Wearing Surface</strong></td>
<td>½” bituminous</td>
</tr>
<tr>
<td><strong>Other Structure Info</strong></td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Spans Tested</strong></td>
<td>Southernmost span</td>
</tr>
<tr>
<td><strong>Test Reference Location</strong></td>
<td>Southeast corner, inside edge of curb along face of abutment wall</td>
</tr>
<tr>
<td><strong>Test Vehicle Direction</strong></td>
<td>North</td>
</tr>
<tr>
<td><strong>Test Beginning Point</strong></td>
<td>Front axle at X = -10’ from (0,0)</td>
</tr>
<tr>
<td><strong>Lateral Load Positions</strong></td>
<td>Y1 = 3’-2”(P), Y2 = 6’-8”(P), Y3 = 9’-4”(P) Measurements to field “BOW”</td>
</tr>
<tr>
<td><strong>Number/Type of Sensors</strong></td>
<td>14 – 3” strain gages 9 – LVDT displacement gages</td>
</tr>
<tr>
<td><strong>Sample Rate</strong></td>
<td>40 Hz</td>
</tr>
<tr>
<td><strong>Number of Test Vehicles</strong></td>
<td>1</td>
</tr>
<tr>
<td><strong>Structure Access Type</strong></td>
<td>Ground</td>
</tr>
<tr>
<td><strong>Structure Access Provided by</strong></td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Traffic Control Provided by</strong></td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Total Field Testing Time</strong></td>
<td>1 Day</td>
</tr>
<tr>
<td><strong>Field Notes</strong></td>
<td>See Appendix B</td>
</tr>
<tr>
<td><strong>Additional NDT Info</strong></td>
<td>N/A</td>
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### Test File Information

<table>
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<tr>
<th><strong>File Name</strong></th>
<th><strong>Lateral Position</strong></th>
<th><strong>Field Comments</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Red_1.dat</td>
<td>Y1</td>
<td>Truck ~8” east</td>
</tr>
<tr>
<td>Red_2.dat</td>
<td>Y1</td>
<td>Good</td>
</tr>
<tr>
<td>Red_3.dat</td>
<td>Y1</td>
<td>Good</td>
</tr>
<tr>
<td>Red_4.dat</td>
<td>Y2</td>
<td>Truck ~6” east, dynamics</td>
</tr>
<tr>
<td>Red_5.dat</td>
<td>Y2</td>
<td>Truck ~3”-4” east</td>
</tr>
<tr>
<td>Red_6.dat</td>
<td>Y2</td>
<td>Good</td>
</tr>
<tr>
<td>Red_7.dat</td>
<td>Y3</td>
<td>Good</td>
</tr>
<tr>
<td>Red_8.dat</td>
<td>Y3</td>
<td>Truck ~6” west</td>
</tr>
<tr>
<td>Red_9.dat</td>
<td>Y3</td>
<td>Good</td>
</tr>
</tbody>
</table>

### Comments

Fairly large, longitudinal crack or seam observed in the wearing surface – see Figure 1.11 and Figure 1.12
Figure 1.1 Typical strain gage and LVDT on bottom of beam.

Figure 1.2 Typical strain gages and LVDT at top of beam and edge of FRP deck panel.
Figure 1.3 Typical strain gage and LVDT at midspan of FRP deck panel.
Figure 1.4 Typical FRP deck instrumentation between beams.

Figure 1.5 Overall view of entire instrumentation setup.
Figure 1.6 Instrumentation plan with truck positions and gage locations.
Figure 1.7 Elevation view with longitudinal gage locations.
Figure 1.8 Cross-section with gage locations and truck positions.

Figure 1.9 Cross-section with gage ID’s.
Table 1.2 Dump truck test vehicle information.

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>TANDEM REAR AXLE DUMP TRUCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>78,660lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1</td>
<td>Unknown 7’-2”</td>
</tr>
<tr>
<td>WEIGHT/WIDTH – AXLE 2</td>
<td>Unknown 7’-3”</td>
</tr>
<tr>
<td>WEIGHT/WIDTH – AXLE 3</td>
<td>Unknown 7’-3”</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’-8”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>Angelo Iafrate Construction</td>
</tr>
<tr>
<td>AUTOCLICKER POSITION</td>
<td>Manual clicking</td>
</tr>
<tr>
<td>WHEEL ROLLOUT 5 REVS</td>
<td>53’-11”</td>
</tr>
<tr>
<td>WHEEL CIRCUMFERENCE</td>
<td>10.78’</td>
</tr>
<tr>
<td># CRAWL SPEED Passes</td>
<td>9</td>
</tr>
<tr>
<td># HIGH SPEED Passes/SPEED</td>
<td>0   N/A (incomplete roadway)</td>
</tr>
<tr>
<td>VEHICLE PROVIDED BY</td>
<td>Angelo Iafrate Construction</td>
</tr>
</tbody>
</table>

Figure 1.10 Dump truck vehicle footprint.
Figure 1.11 Longitudinal crack or seam observed in wearing surface.

Figure 1.12 Longitudinal crack or seam observed in wearing surface.
2. Qualitative Review of Test Data

All of the field data was first examined graphically to determine its quality and to provide a qualitative assessment of the structure's live-load response. Some of the indicators of data quality included reproducibility between identical truck crossings, elastic behavior (strains returning to zero after truck crossing), symmetry of measurement responses, and any unusual-shaped responses that might indicate nonlinear behavior or possible gage malfunctions.

General Observations of Test Results:

- Reproducibility and Linearity: Responses from identical truck paths were very reproducible as shown in Figure 2.1. In addition, all strains and displacements appeared to be linear with respect to load magnitude (truck position). Note that responses did not return to zero, but this was a result of the beams being continuous over several spans and the test truck not being driven all the way off of the bridge. For all tests, the test truck was stopped near the end of Span 2, which resulted in the strain and displacement response histories displaying negative moment at the end of the tests. Figure 2.1 shows the response histories for midspan deck displacement gages in the middle bay and side bay for three identical truck paths along Path Y3. All of the response histories had a similar degree of reproducibility and linearity indicating that the data collected was of very good quality.

- Midspan FRP Deck Strains: Midspan, flexural deck strains were measured at two locations: in the middle bay directly centered between the two interior beams, and in one side bay directly centered between an interior beam and the corresponding exterior beam. Truck Paths Y1 and Y3 were located so as to generate maximum strains at these specific locations. In general, recorded maximum live load strains were in the range of 500-600µu, which corresponds to about 550-700µu for HS-20 design loading (without impact). Figure 2.2 and Figure 2.3 show the flexural deck strains in the two previously described locations for truck Path Y1, while Figure 2.4 shows the deck strain in the middle bay generated by truck Path Y3. Note that deck strain responses are extremely sensitive to variations in lateral truck position, and therefore do not appear quite as reproducible as displacement responses for the same truck paths.

- Midspan FRP Deck Deflections: Midspan deck deflections were measured at the same locations as the midspan deck strains: in the middle bay directly centered between the two interior beams, and in one side bay directly centered between an interior beam and the corresponding exterior beam. However, in order to isolate the relative deck deflection from the total bridge deflection (deck and beams), beam deflections were also measured. By subtracting the average of the two adjacent beam deflections from the total measured deflection at a given midspan deck location, the relative deflection of the deck itself was obtained. Figure 2.5 through Figure 2.8 shows the maximum total midspan deck deflection and respective average beam deflection for each truck path.
Figure 2.1 Reproducibility of deck displacement measurements – Path Y3.

Figure 2.2 Deck strain measurements at interior bay for 3 truck passes along Path Y1.
Figure 2.3 Deck strain measurements at exterior bay for 3 truck passes along Path Y1.

Figure 2.4 Deck strain measurements at interior bay for 3 truck passes along Path Y3.
Figure 2.5 Deck deflection and average beam deflection at outer bay for Truck Path Y1.

Figure 2.6 Deck deflection and average beam deflection at inner bay for Truck Path Y1.
Figure 2.7 Deck deflection and average beam deflection at inner bay for Truck Path Y2.

Figure 2.8 Deck deflection and average beam deflection at inner bay for Truck Path Y3.
3. **DATA ANALYSIS AND CALCULATION PROCEDURES**

The goal of load testing this particular structure was to obtain live load strains and deflections that could provide a measure of performance for the FRP deck and verify that it met required design specifications. By quantifying deck strains and deflections under a known loading, strains and deflections under HS-20 design loading could be extrapolated.

It is important to note however, that only a gross weight of the testing vehicle was available and not individual axle weights. This ultimately means that the axle-load distribution had to be estimated based on the collected field data. And because precise weights could not be achieved, these procedures can only provide a reasonable estimate of the deck meeting its design criteria.

1) **CALCULATION OF DEFLECTION AND STRAIN LIMITS:**

The first step in the process was to calculate the deflection and strain limits as outlined in MEC’s Statement of Work (SOW). The design specification for deflection stated that “the deck deflection due to live loads plus impact shall be limited to L/500, where L is the distance between the centerline of adjacent girders.” With a center to center girder spacing of 80 inches, the maximum allowable deflection under HS-20 live load plus impact was 0.16 inches.

The design specification for strain stated that “the strains in the panels under full dead load and design live load shall not exceed twenty (20) percent of the strain at the ultimate capacity of the FRP material” and that “the strains in the panels under dead load alone shall not exceed ten (10) percent of the strains at the ultimate capacity of the FRP material.” In a Quality Assurance Test Summary Report, written by Creative Pultrusions and provided to BDI by ZellComp, the flexural strength of the FRP material was 44,760 psi with a flexural stiffness modulus of 2.78E+06 psi. Assuming the FRP material remains linear until failure (ultimate strength), the stress-strain relationship shown in Equation 1 yielded an ultimate strain of 16,100µε. This meant that the dead load strain in the panels could not exceed 1,610µε, and the combined dead load and live load (plus impact) strains could not exceed 3,220µε.

\[ \sigma = E \cdot \varepsilon \]

**Equation 1**

2) **CALCULATION OF ACTUAL DEFLECTION AND STRAIN:**

The second step was to review the field data and calculate the maximum deflections and strains under the test truck loading. Deck deflections were calculated by taking the measured total deflection at the midspan of a given deck panel, and subtracting from it the average deflection of the two adjacent steel girders. This process was automated in MS Excel for each data file, and the results of which are provided in the document `Ext_Data.xls`. Also included in the MS Excel document are the nine data files, which have been extracted in terms of truck position.

Maximum live load strains were taken directly from the field data, and increased by 33% to account for impact and dynamic effects. Dead load strain values were slightly more complicated to calculate, and required a multistep process as outlined below:
1. Calculated an adjusted gross cross-sectional Moment of Inertia, $I_x$, based on the ratios of member stiffness values to the bottom plate stiffness value.

2. Calculated a uniform distributed dead load, $\omega$, based on the self-weight of the deck panels (17.5 psf) and wearing surface ($\frac{1}{2}\" \times 140$ pcf).

3. Calculated the middle bay and side bay midspan positive moments based on the assumption of a 3-span continuous beam, Equation 2 and Equation 3.

$$M_{\text{middle}} = 0.025\omega L^2$$  \hspace{1cm} \text{Equation 2}

$$M_{\text{side}} = 0.08\omega L^2$$  \hspace{1cm} \text{Equation 3}

4. Calculated the corresponding dead load stresses based on Equation 4.

$$\sigma = \frac{M \cdot y}{I_x}$$  \hspace{1cm} \text{Equation 4}

5. Calculated the dead load strains based on Equation 1.

3) **Extrapolation to HS-20 Design Live Load and Design Verification:**

Extrapolating the maximum deflections and strains produced by the test truck to those that would be produced with an HS-20 was as simple as multiplying the calculated values by the ratio of the axle weights of the two trucks. Because the maximum deflections and strains were produced by the rear axles of the test truck, the calculated test truck values were multiplied by a factor of 1.13 (32 kip rear axle weight of an HS-20 divided by 28.3 kip rear axle weight of the test truck). This process produced design-load deflections and strains below the respective specification limits. Table 3.1 shows the maximum deflections and strains produced by the test truck for each of the nine truck passes. Calculations for the extrapolated HS-20 deflections and strain are provided in Table 3.2 along with the corresponding limit checks.

Of moderate concern to BDI was the fact that the individual axles of the test truck were not weighed in the field and therefore had to be estimated based on the test response data. Although the assumed axle distribution was likely close, it could still possibly provide false evidence for the deck meeting (or not meeting) the specified criteria. To help compensate for this possible error, and to provide additional supporting evidence, the deflection calculation was performed backwards in an attempt to “back-out” the axle load distribution that would have caused the deck to fail the deflection criteria. Note that this procedure was not done for the strain criteria because the test strains were substantially lower than the maximum allowable strains and therefore not of concern.
The result of this “backward” calculation showed that in order for the deck to fail the deflection criteria, the test truck’s rear axles would have weighed 25.3 kips and the front axle would have weighed 28.1 kips. This is a very unlikely distribution for a three-axle dump truck, which supports the conclusion that the deck met the design specification for deflection. For reference, Figure 3.1 shows the axle configuration and load distribution of an HS-20 loading vehicle.

Table 3.1 Maximum FRP deck displacements and strains.

<table>
<thead>
<tr>
<th></th>
<th>Test Y1-1</th>
<th>Test Y1-2</th>
<th>Test Y1-3</th>
<th>Test Y2-1</th>
<th>Test Y2-2</th>
<th>Test Y2-3</th>
<th>Test Y3-1</th>
<th>Test Y3-2</th>
<th>Test Y3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max Disp</td>
<td>Max Strain</td>
<td>Max Disp</td>
<td>Max Strain</td>
<td>Max Disp</td>
<td>Max Strain</td>
<td>Max Disp</td>
<td>Max Strain</td>
<td>Max Disp</td>
</tr>
<tr>
<td>Middle</td>
<td>0.073</td>
<td>420</td>
<td>0.078</td>
<td>461</td>
<td>0.078</td>
<td>480</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Side</td>
<td>0.085</td>
<td>500</td>
<td>0.091</td>
<td>501</td>
<td>0.092</td>
<td>504</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle</td>
<td>0.045</td>
<td>116</td>
<td>0.045</td>
<td>116</td>
<td>0.049</td>
<td>125</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Side</td>
<td>0.010</td>
<td>26</td>
<td>0.011</td>
<td>25</td>
<td>0.009</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle</td>
<td>0.095</td>
<td>548</td>
<td>0.095</td>
<td>505</td>
<td>0.095</td>
<td>515</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Side</td>
<td>0.011</td>
<td>6</td>
<td>0.011</td>
<td>4</td>
<td>0.011</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2 HS-20 Deflection and strain limit check.

<table>
<thead>
<tr>
<th></th>
<th>Deflection Criteria</th>
<th>Strain Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Relative Displacement</td>
<td>0.095</td>
<td>548</td>
</tr>
<tr>
<td>Plus Impact @ 33%</td>
<td>0.126</td>
<td>729</td>
</tr>
<tr>
<td>Extrapolated to HS-20 (x 1.13)</td>
<td>0.143</td>
<td>824</td>
</tr>
<tr>
<td>Live-load Limit</td>
<td>0.160</td>
<td>3217</td>
</tr>
<tr>
<td>Pass/Fail</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td>Rating Factor</td>
<td>1.12</td>
<td>3.91</td>
</tr>
</tbody>
</table>

Figure 3.1 HS-20 vehicle configuration.
4. **CONCLUSIONS AND RECOMMENDATIONS**

Load tests indicated that the bridge was performing in a normal manner; all responses were linear-elastic and there were no signs of distress. In addition, all of the response histories had a high degree of reproducibility, indicating that the data collected was of very good quality.

The goal of load testing this particular structure was to obtain live load strains and deflections that could provide a measure of performance for the FRP deck and verify whether it met the required design specifications. By quantifying actual deck strains and deflections under a known loading, theoretical strains and deflections under HS-20 design loading were extrapolated and directly compared to the design limits.

The results of these procedures indicated that the deck met the design criteria for both strain and deflection. Note however, that because individual axles were not weighed in the field and resulting axle-load distributions had to be estimated for calculation purposes, the final results only indicate a high probability of success rather than an absolute certainty of success.

As a whole, the load test was very successful, and the new FRP deck performed as designed.
A. APPENDIX A – HAND CALCULATIONS (SCANNED)

Bottom Plate:  \( E = 2500 \text{ ksi} \)

**Dim:**  \( 6'' \times 0.5'' \)

Bottom Flange:  \( E = 2600 \text{ ksi} \)

**Dim:**  \( 4'' \times 0.17'' \)

\[ A_{d} = (4'')(\frac{36}{25}) \times 0.17'' = 5.76'' \times 0.17'' \]

Web:  \( E = 2600 \text{ ksi} \)

**Dim:**  \( 0.5'' \times 5.33'' \)

\[ A_{d} = (0.5)(\frac{26}{25}) \times 5.33'' = 0.52'' \times 5.33'' \]

Top Flange:  \( E = 3600 \text{ ksi} \)

**Dim:**  \( 4'' \times 0.5'' \)

\[ A_{d} = (4'')(\frac{36}{25}) \times 0.5'' = 5.76'' \times 0.5'' \]

Top Plate:  \( E = 1500 \text{ ksi} \)

**Dim:**  \( 6'' \times 0.5'' \)

\[ A_{d} = (6'')(\frac{15}{25}) \times 0.5'' = 4.5'' \times 0.5'' \]

\[ I_{x} = 104.5 \text{ in}^4 \]

\[ A = 13.03 \text{ in}^2 \]

\[ \bar{y} = 3.45'' \text{ up from bottom face} \]
Strain & Deflection Cals

→ Truck Info:
  • Gross Weight = 78,660 lbs
  • Front Axle = 22,000 lbs (28%)
  • Rear Tandem = 56,660 lbs (72%)

→ Deflection Criteria Check
  Test Max = 0.015” w/ a 28.3 k axle

  Design Check: 0.015 \left( \frac{32}{28.3} \right) (1.13) < 0.14” < 0.16” OK

  Test Axle Weight to Fail:
  \[ W = \frac{0.015”}{0.16”} \left( \frac{32}{113} \right) = 25.3 k \text{ rear axle} \]
  \[ = 50.5 k \text{ tandem} \]
  \[ = 28.1 k \text{ front} \] \text{ NOTE*}

→ Strain Criteria Check
  DL Deck = 17.5 psf
  DL wearing surface = (140 psf) (0.32) = 5.33 psf
  \[ W_{DL} = \left( \frac{0.3”}{12} \right) (17.5 + 5.33) = 15.56 \text{ lbs} \]
  \[ M_{\text{middle,DL}} = 0.025 W L^2 = 0.025 (15.56)(6.67)^2 = 17.3 \text{ lb-ft} \]
  \[ M_{\text{side,DL}} = 0.025 W L^2 = 17.3 \left( \frac{0.025}{0.025} \right) = 55.3 \text{ lb-ft} \]
\[ \sigma_{\text{middle,DL}} = \frac{My}{I} = \frac{(17.3 \text{ ft})(12)(3.45 \text{ in})}{104.51 \text{ in}^4} = 6.85 \text{ psi} \]

\[ \epsilon_{\text{middle,DL}} = \frac{\sigma}{E} = \frac{6.85 \text{ psi}}{29.1 \times 10^6 \text{ psi}} \times 10^4 = 2.7 \times 10^{-4} \text{ in/in} \]

\[ \sigma_{\text{side,DL}} = \frac{My}{I} = \frac{(55.3)(12)(3.45)}{104.51} = 21.9 \text{ psi} \]

\[ \epsilon_{\text{side,DL}} = \frac{\sigma}{E} = \frac{21.9}{2.5} = 8.8 \times 10^{-3} \text{ in/in} \]

\[ \epsilon_{\text{total max}} = \epsilon_{\text{LL}} + \epsilon_{\text{DL}} = 54.8 \left( \frac{32}{26.3} \right)(1.33) + 2.7 \]

\[ = 9.27 \times 10^{-4} \text{ in/in} \leq 3.22 \times 10^{-4} \text{ in/in} \text{ OK} \]
Pass/Fail Criteria

→ Deflection: \( \frac{1}{500} \)

\[ 6\frac{\text{in}}{} = 80\text{"} \]

\[ \frac{1}{500} = \frac{80}{500} = 0.16\" \]

→ Strain: \( \varepsilon_{DL} + \varepsilon_{LL} \leq 0.20(\varepsilon_{\text{ultimate}})\), \( \varepsilon_{PL} \leq 0.10(\varepsilon_{\text{ultimate}})\)

\[ F_u = 441,760 \text{ psi} \]
\[ E = 2.76 \times 10^{6} \text{ psi} \]

\[ \sigma = E \varepsilon \rightarrow \varepsilon = \frac{\sigma}{E} = \frac{441,760}{2.76 \times 10^6} \times 10^6 = 16.100 \mu e \]

- Dead Load

\[ \varepsilon_{DL} \leq 0.10(16.100) = 1.610 \mu e \]

- Dead Load + Live Load

\[ \varepsilon_{DL} + \varepsilon_{LL} \leq 0.20(16.100) = 3.220 \mu e \]
Creative Pultrusions
Quality Assurance
Test Summary Report

Part Id: CP216.102          Date Tested: December 21, 2009
Description: 7" Bridge Deck          Date Produced: December 18, 2009
Production Order: 

<table>
<thead>
<tr>
<th>Mechanical Properties</th>
<th>ASTM Test</th>
<th>Results</th>
<th>Minimum Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength Lengthwise</td>
<td>D638</td>
<td>45,304, 2.685 psi</td>
<td>psi</td>
</tr>
<tr>
<td>Tensile Modulus Lengthwise</td>
<td>D638</td>
<td>4.32E+06, 2.37E+05</td>
<td>psi</td>
</tr>
<tr>
<td>Flexural Strength Lengthwise</td>
<td>D790</td>
<td>44,760, 3.113 psi</td>
<td>psi</td>
</tr>
<tr>
<td>Flexural Modulus Lengthwise</td>
<td>D790</td>
<td>2.78E+06, 2.31E+05</td>
<td>psi</td>
</tr>
</tbody>
</table>

See Drawing below for sample location.

Tested By: D. Crawford
Quality Assurance Specialist

Approved By: D. Allison
Quality Assurance Supervisor
B. APPENDIX B – FIELD NOTES (SCANNED)

Figure B.1 Scanned Notes- Page 1.
MEASUREMENTS AND TESTING PROCEDURES (ABOVE)

BEGINNING OF WORLD (BOW)  VERIFY NORTH ON PLANS: YES  NO
(X=0, Y=0, location)
Front Face of abut wall inside edge of barrier → SE corner

BOW PHOTOS: □  ROAD MARKINGS PHOTOS: □

ROADWAY WIDTH (curb-to-curb): 19' 10"  SYMMETRIC: YES  NO

STRUCTURE WIDTH (Out-to-Out): 20' 8" out-to-out of barriers

WEARING SURFACE: Gravel compacted  THICKNESS:

STARTING TEST POSITION: - 10' (no ½ rev) DIRECTION: x North

VEHICLE ROLL OUT (5 REV/S): ≤3’-11"  A/C LOCATION: None

*****MAKE SURE YOU PUT THE A/C ON THE SAME WHEEL AS WAS USED TO MEASURE THE ROLL OUT***** Manual

VEHICLE MEASUREMENTS:

\[
\begin{array}{c}
1'1" \\
7'2"
\end{array}
\]

AXLE WEIGHTS:

\[
\begin{array}{c}
1'8" \\
10'11" \\
7'3"
\end{array}
\]

FRONT: REAR: GROSS: 75,600

VEHICLE PROVIDED BY: Angelo Infante Construction

TRAFFIC CONTROL PROVIDED BY: None

ACCESS PROVIDED BY: None

Figure B.2 Scanned Notes- Page 2.
**Lateral Testing Positions:** (Referenced from BOW)

<table>
<thead>
<tr>
<th>Y1:</th>
<th>Y2:</th>
</tr>
</thead>
<tbody>
<tr>
<td>3'-2'' (P)</td>
<td>6'-8'' (P)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Y3:</th>
<th>Y4:</th>
<th>Y5:</th>
<th>Y6:</th>
</tr>
</thead>
<tbody>
<tr>
<td>9'-10'' (P)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Lateral Positions Checked By:**  

**Testing Operations (WinSTS)**

**Verify Gage ID & # of Channels with WinSTS:**

**Run WinSTS to Verify Responses:**

**Weather Conditions & Ambient Temperature:** Breezy, rainy, & cold! 50°F

**Running the Field Tests**

STS Operator: Dan  
TRUCK OPERATOR: Scott

**Controlled Semi-Static Tests**

**Sample Rate:** 40 Hz  
**Gain:** 3.2

<table>
<thead>
<tr>
<th>File Name</th>
<th>Lateral Position</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bed-1. dat</td>
<td>Y1</td>
<td>≈ 6'' toward barrier, forgot to stop test! Got stuck on the way back</td>
</tr>
<tr>
<td>-2.dat</td>
<td>Y1</td>
<td>Good, truck right on line</td>
</tr>
<tr>
<td>-3.dat</td>
<td>Y1</td>
<td>Good</td>
</tr>
<tr>
<td>-4.dat</td>
<td>Y2</td>
<td>Backing when came onto bridge, tire width toward barrier (East)</td>
</tr>
<tr>
<td>-5</td>
<td>Y2</td>
<td>Good, ≈ 3'-4'' toward barrier (East)</td>
</tr>
<tr>
<td>-6</td>
<td>Y2</td>
<td>Good, right on</td>
</tr>
<tr>
<td>-7</td>
<td>Y3</td>
<td>Good, see note below</td>
</tr>
<tr>
<td>-8</td>
<td>Y3</td>
<td>Good, about 6'' west, so 7'-10''</td>
</tr>
<tr>
<td>-9</td>
<td>Y3</td>
<td>Good, see note below</td>
</tr>
</tbody>
</table>

**Note:** Y3 passes 6'' off due to big hole in the approach slab.  

Figure B.3 Scanned Notes- Page 3.
Figure B.4 Scanned Notes- Page 4.
C. APPENDIX C - FIELD TESTING PROCEDURES

BACKGROUND

The motivation for developing a relatively easy-to-implement field-testing system was to allow short and medium span bridges to be tested on a routine basis. Original development of the hardware was started in 1988 at the University of Colorado under a contract with the Pennsylvania Department of Transportation (PennDOT). Subsequent to that project, the Integrated Technique was refined on another study funded by the Federal Highway Administration (FHWA) in which 35 bridges located on the Interstate system throughout the country were tested and evaluated. Further refinement has been implemented over the years through testing and evaluating hundreds of bridges, lock gates, and other structures.

STRUCTURAL TESTING HARDWARE

The real key to being able to complete the field-testing quickly is the use of strain transducers (rather than standard foil strain gages) that can be attached to the structural members in just a few minutes. These sensors were originally developed for monitoring dynamic strains on foundation piles during the driving process. They have been adapted for use in structural testing through special modifications, have very high accuracy, and are periodically re-calibrated to NIST standards. Please refer to Appendix D for specifications on the BDI Strain Transducers.

In addition to the strain sensors, the data acquisition hardware has been designed specifically for structural live load testing which means it is extremely easy to use in the field. Please see Appendix E for specifications on the BDI Structural Testing System. Briefly, some of the features include military-style connections for quick assembly and self-identifying sensors that dramatically reduce bookkeeping efforts. The WinSTS testing software has been written to allow easy hardware configuration and data recording operation. Other enhancements include the BDI AutoClicker which is an automatic load position indicator that is mounted directly on the vehicle. As the test truck crosses the structure along the preset path, a communication radio sends a signal to the STS that receives it and puts a mark in the data. This allows the field strains to be compared to analytical strains as a function of vehicle position, not only as a function of time. Refer to Appendix F for the AutoClicker specifications. The end result of using all of the above-described components is a system that can be used by people other than computer experts or electrical engineers. Typical testing times with the STS is usually anywhere from 20 to 60 channel tests being completed in one day, depending on access and other field conditions.

The following general directions outline how to run a typical diagnostic load test on a short-to medium-span highway bridge up to about 200 ft (60m) in length. With only minor modifications, these directions can be applied to railroad bridges (use a locomotive rather than a truck for the load vehicle), lock gates (monitor the water level in the lock chamber), amusement park rides (track the position of the ride vehicle) and other structures in which the live load can be applied easily. The basic scenario is to first instrument the structure with the required number of sensors, run a series of tests, and then removing all the sensors. These procedures can often be completed within one working day depending on field conditions such as access and traffic.
**INSTRUMENTATION OF STRUCTURE**

This outline is intended to describe the general procedures used for completing a successful field test on a highway bridge using the BDI-STS. For a detailed explanation of the instrumentation and testing procedures, please contact BDI and request a copy of the Structural Testing System (STS) Operation Manual.

**ATTACHING STRAIN TRANSDUCERS**

Once a tentative instrumentation plan has been developed for the structure in question, the strain transducers must be attached and the STS prepared for running the test. There are several methods for attaching the strain transducers to the structural members depending on whether they are steel, concrete, timber, FRP, or other. For steel structures, quite often the transducers can be clamped directly to the steel flanges of rolled sections or plate girders. If significant lateral bending is assumed to be present, then one transducer may be clamped to each edge of the flange. In general, the transducers can be clamped directly to painted surfaces. The alternative to clamping is the tab attachment method that involves cleaning the mounting area and then using a fast-setting cyanoacrylate adhesive to temporarily install the transducers. Small steel “tabs” are used with this technique and they are removed when testing is completed, and touch-up paint can be applied to the exposed steel surfaces.

Installation of transducers on pre-stressed concrete (PS/C) and FRP members is usually accomplished with the tab technique outlined above, while readily-available wood screws and a battery-operated hand drill are used for timber members. Installing transducers on reinforced concrete (R/C) is more complex in that gage extensions are used and must be mounted with concrete studs.

If the above steps are followed, it should be possible to mount each transducer in approximately five to ten minutes. The following figures illustrate transducers mounted on both steel and reinforced concrete members.
Figure C.1 Strain Transducers Mounted on Steel Girder

Figure C.2 Transducers w/Gage Extensions Mounted On R/C Slab
ASSEMBLY OF SYSTEM

Once the transducers have been mounted, they are connected to the four-channel STS units which are also located on the bridge. The STS units can be easily clamped to the bridge girders, or if the structure is concrete and no flanges are available to set the STS units on, transducer tabs glued to the structure and plastic zip-ties or small wire can be used to mount them. Since the transducers will identify themselves to the system, there is no special order that they must be plugged into the system. The only information that must be recorded is the transducer serial number and its location on the structure. Signal cables are then used to connect STS units together either in series or in a “tree” structure through the use of cable splitters. If several gages are in close proximity to each other, then the STS units can be plugged directly to each other without the use of a cable.

Once all of the STS units have been connected together, only one cable must be run and connected to the STS Power Supply located near the PC. Once power and communication cables are connected, the system is ready to acquire data. One last step entails installing the AutoClicker on the test vehicle as seen in Figure C.3.

Figure C.3 AutoClicker Mounted on Test Vehicle
**ESTABLISHING LOADING VEHICLE POSITIONS**

Once the structure is instrumented and the loading vehicle prepared, some reference points must be established on the deck in order to determine where the vehicle will cross. This process is important so that future analysis comparisons can be made with the loading vehicle in the same locations as it was in the field. Therefore, a “zero” or initial reference point is selected and usually corresponds to the point on the deck directly above the abutment bearing and the centerline of one of the fascia beams. All other measurements on the deck will then be related to this zero reference point. For concrete T-beams, box beams, and slabs, this can correspond to where the edge of the slab or the beam web meets the face of the abutment. If the bridge is skewed, the first point encountered from the direction of travel is used. In any case, it should be a point that is easily located on the drawings for the structure.

Once the zero reference location is known, the lateral load paths for the vehicle are determined. Often, the painted roadway lines are used for the driver to follow if they are in convenient locations. For example, for a two-lane bridge, a northbound shoulder line will correspond to Y1 (passenger-side wheel), the center dashed line to Y2 (center of truck), and the southbound shoulder line to Y3 (driver’s side wheel). Often, the structure will be symmetrical with respect to its longitudinal center line. If so, it is good practice is to take advantage of this symmetry by selecting three Y locations that are also symmetric. This will allow for a data quality check since the response should be very similar, say, on the middle beam if the truck is on the left side of the bridge or the right side of the bridge. In general, it is best to have the truck travel in each lane (at least on the lane line) and also as close to each shoulder or sidewalk as possible. When the deck layout is completed, the loading vehicle’s axle weights and dimensions are recorded.

**RUNNING THE LOAD TESTS**

After the structure has been instrumented and the reference system laid out on the bridge deck, the actual testing procedures are completed. The WinSTS software is initialized and configured. When all personnel are ready to commence the test, traffic control is initiated and the Run Test option is selected which places the system in an activated state. When the truck passes over the first deck mark, the AutoClicker is tripped and data is being collected at the specified sample rate. An effort is made to get the truck across with no other traffic on the bridge. When the rear axle of the vehicle completely crosses over the structure, the data collection is stopped and several strain histories evaluated for data quality. Usually, at least two passes are made at each “Y” position to ensure data reproducibility, and then if conditions permit, high speed or dynamic tests are completed.

The use of a moving load as opposed to placing the truck at discrete locations has two major benefits. First, the testing can be completed much quicker, meaning there is less impact on traffic. Second, and more importantly, much more information can be obtained (both quantitative and qualitative). Discontinuities or unusual responses in the strain histories, which are often signs of distress, can be easily detected. Since the load position is monitored as well, it is easy to determine what loading conditions cause the observed effects. If readings are recorded only at discreet truck locations, the risk of losing information between the points is great. The advantages of continuous readings have been proven over and over again.

When the testing procedures are complete, the instrumentation is removed and any touch-up work completed.
D. APPENDIX D – EQUIPMENT SPECIFICATIONS

**SPECIFICATIONS: BDI STRAIN TRANSDUCERS**

- **Effective gage length:** 3.0 in (76.2 mm). Extensions available for use on R/C structures.
- **Overall Size:** 4.4 in x 1.2 in x 0.5 in (110 mm x 33 mm x 12 mm).
- **Cable Length:** 10 ft (3 m) standard, any length available.
- **Material:** Aluminum
- **Circuit:** Full wheatstone bridge with four active 350Ω foil gages, 4-wire hookup.
- **Accuracy:** ± 2%, individually calibrated to NIST standards.
- **Strain Range:** Approximately ±4000 με.
- **Force req’d for 1000 με:** Approximately 9 lbs. (40 N).
- **Sensitivity:** Approximately 500 με/mV/V.
- **Weight:** Approximately 3 oz. (88 g).
- **Environmental:** Built-in protective cover, also water resistant.
- **Temperature Range:** -60°F to 250°F (-50°C to 120°C) operation range.
- **Cable:** BDI RC-187: 22 gage, two individually-shielded pairs w/drain.
- **Options:** Fully waterproofed, Heavy-duty cable, Special quick-lock connector.
- **Attachment Methods:** C-clamps or threaded mounting tabs & quick-setting adhesive.
**SPECIFICATIONS: BDI WIRELESS STRUCTURAL TESTING SYSTEM**

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<thead>
<tr>
<th>Channels</th>
<th>4 to 128 (Expandable in multiples of 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardware Accuracy</td>
<td>± 0.2% (2% for Strain Transducers)</td>
</tr>
<tr>
<td>Sample Rates</td>
<td>0.1 – 500 Hz (Internal over-sampling rate is 19.5-312 kHz)</td>
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<tr>
<td>Max Test Lengths</td>
<td>21 minutes at 100 Hz. 128K samples per channel maximum test lengths</td>
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<tr>
<td>Gain Levels</td>
<td>1, 2, 4, 6, 16, 32, 64, 128</td>
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<tr>
<td>Digital Filter</td>
<td>Fixed by selected sample rate</td>
</tr>
<tr>
<td>Analog Filter</td>
<td>200 Hz, -3db, 3rd order Bessel</td>
</tr>
<tr>
<td>Max. Input Voltage</td>
<td>10.5 Volts DC</td>
</tr>
<tr>
<td>Battery Power</td>
<td>9.6 NiMH rechargeable battery (Programmable low-power sleep mode)</td>
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<tr>
<td>Alternative Power</td>
<td>9-48 Volts DC input</td>
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<td>Excitation Voltages</td>
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<tr>
<td>Standard:</td>
<td>5 Volts DC</td>
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<tr>
<td>LVDT/Other:</td>
<td>5.5 Volts DC</td>
</tr>
<tr>
<td>A/D Resolution</td>
<td>0.3uV bit (24-bit ADC)</td>
</tr>
<tr>
<td>PC Requirements</td>
<td>Windows XP or higher</td>
</tr>
<tr>
<td>PC Interface</td>
<td>Wi-Fi Ethernet 802.11b: 10/100 Mbps</td>
</tr>
<tr>
<td>Auto Zeroing</td>
<td>Sensors automatically zero before each test</td>
</tr>
<tr>
<td>Enclosures</td>
<td>Aluminum splash resistant</td>
</tr>
<tr>
<td>Sensor Connections</td>
<td>All aluminum military grade, circular bayonet “snap” lock</td>
</tr>
<tr>
<td>Vehicle Tracking</td>
<td>BDI AutoClicker, switch closure detection</td>
</tr>
<tr>
<td>Sensors</td>
<td>BDI Intelliducer Strain Transducer</td>
</tr>
<tr>
<td></td>
<td>Also supports, LVDT’s, foil strain gages, accelerometers, Load Cell’s and other various DC output sensors</td>
</tr>
<tr>
<td></td>
<td>Single RS232 serially-interfaced sensor</td>
</tr>
<tr>
<td>On-Board PC</td>
<td></td>
</tr>
<tr>
<td>Processor:</td>
<td>520 MHz Intel XScale PXA270</td>
</tr>
<tr>
<td>RAM:</td>
<td>64MB</td>
</tr>
<tr>
<td>Dimensions</td>
<td></td>
</tr>
<tr>
<td>Base Station:</td>
<td>10” x 6” x 4”</td>
</tr>
<tr>
<td>STS 4-Channel Node:</td>
<td>11” x 3.5” x 3.23”</td>
</tr>
</tbody>
</table>
### Specifications: BDI AutoClicker

<table>
<thead>
<tr>
<th>Specification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 Handheld Radios</td>
<td>Motorola P1225 2-Channel (or equal) modified for both “Rx” and “Tx”.</td>
</tr>
<tr>
<td>Power</td>
<td>9V battery</td>
</tr>
<tr>
<td>Mounting</td>
<td>Universal front fender mounting system</td>
</tr>
<tr>
<td>Target</td>
<td>Retroreflective tape mounted on universal wheel clamp</td>
</tr>
<tr>
<td>Bands/Power</td>
<td>VHF/1 Watt or UHF/2 Watt</td>
</tr>
<tr>
<td>Frequencies</td>
<td>User-specified</td>
</tr>
<tr>
<td>Data Acquisition System Requirements</td>
<td>TTL/CMOS input (pull-up resistor to 5V)</td>
</tr>
<tr>
<td>Output</td>
<td>Isolated contact closure (200V 0.5A max switch current)</td>
</tr>
</tbody>
</table>
E. APPENDIX G - REFERENCED MATERIAL


Earney, T. Patrick. “Girder-End Cracking in Prestressed I-Girders”. University of Missouri, Columbia, MO.
### Distribution of Reports:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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<tbody>
<tr>
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<td>4</td>
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<td>10</td>
</tr>
</tbody>
</table>

### REPORT OF CONCRETE AND MORTAR COMpressive STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39

<table>
<thead>
<tr>
<th>PROJECT NAME:</th>
<th>Morris Road Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>PROJECT NUMBER:</td>
<td>HV10003</td>
</tr>
<tr>
<td>LOCATION:</td>
<td>Redstone Arsenal, AL</td>
</tr>
<tr>
<td>CLIENT:</td>
<td>Angelo Iafrate Construction</td>
</tr>
<tr>
<td>CONTRACTOR:</td>
<td>Mandaree Enterprise Corporation</td>
</tr>
<tr>
<td>CONCRETE SUPPLIER:</td>
<td>Mixed on site</td>
</tr>
</tbody>
</table>

### DESIGN & SPECIFICATION DATA

<table>
<thead>
<tr>
<th>MIX ID</th>
<th>SPECIFIED STRENGTH (psi)</th>
<th>SPECIFIED SLUMP (in)</th>
<th>SPECIFIED AIR CONTENT (%)</th>
<th>TEMPERATURE (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/A</td>
<td>1,000</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
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</tbody>
</table>

### FIELD & PLACEMENT DATA

<table>
<thead>
<tr>
<th>METHOD OF PLACEMENT:</th>
<th>EXTRA WATER ADDED AT JOB SITE:</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/A</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

### COMPRESSIVE STRENGTH DATA

<table>
<thead>
<tr>
<th>SPECIMEN ID</th>
<th>DATE SAMPLED</th>
<th>DATE TESTED</th>
<th>AGE (days)</th>
<th>TEST SPECIMEN SIZE</th>
<th>TOTAL LOAD (pounds)</th>
<th>TEST STRENGTH (psi)</th>
<th>TYPE OF FRACTURE</th>
<th>TESTED BY</th>
<th>TESTING LAB</th>
</tr>
</thead>
<tbody>
<tr>
<td>9958 A</td>
<td>01/14/10</td>
<td>01/28/10</td>
<td>14</td>
<td>6.01</td>
<td>28.37</td>
<td>147,760</td>
<td>5,210</td>
<td>1</td>
<td>HB</td>
</tr>
<tr>
<td>9958 B</td>
<td>01/14/10</td>
<td>02/11/10</td>
<td>28</td>
<td>6.02</td>
<td>28.46</td>
<td>168,060</td>
<td>5,910</td>
<td>1</td>
<td>DP</td>
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<tr>
<td>9958 C</td>
<td>01/14/10</td>
<td>02/11/10</td>
<td>28</td>
<td>6.02</td>
<td>28.46</td>
<td>181,480</td>
<td>6,380</td>
<td>1</td>
<td>DP</td>
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</table>

The average 28-day compressive strength (6145 psi) is 615% of the design compressive strength.
**REPORT OF CONCRETE AND MORTAR COMpressive STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39**

**PROJECT NAME:** Morris Road Bridge  
**PROJECT NUMBER:** HV10003  
**CONCRETE REPORT NO.:** C-02  
**SAMPLE NO.:** 9959

**LOCATION:** Redstone Arsenal, AL  
**CLIENT:** Angelo Iafrate Construction  
**CONTRACTOR:** Mandaree Enterprise Corporation  
**CONCRETE SUPPLIER:** Mixed on site

### Design & Specification Data

<table>
<thead>
<tr>
<th>MIX ID</th>
<th>SPECIFIED STRENGTH (psi)</th>
<th>SPECIFIED SLUMP (in)</th>
<th>SPECIFIED AIR CONTENT (%)</th>
<th>TEMPERATURE (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/A</td>
<td>1,000</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**MIX TYPE:**  
- NORMAL WEIGHT
- LIGHT WEIGHT
- HEAVY WEIGHT
- MORTAR
- GROUT

### Field & Placement Data

<table>
<thead>
<tr>
<th>BATCH TIME:</th>
<th>SAMPLE TIME:</th>
<th>TRUCK NUMBER:</th>
<th>TICKET NUMBER:</th>
<th>SIZE OF LOAD (cy):</th>
<th>SAMPLED BY:</th>
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</thead>
<tbody>
<tr>
<td>Unknown</td>
<td>Unknown</td>
<td>N/A</td>
<td>N/A</td>
<td>Unknown</td>
<td>Contractor</td>
</tr>
</tbody>
</table>

**METHOD OF PLACEMENT:**  
- CHUTE
- PUMP
- OTHER

**EXTRA WATER ADDED AT JOB SITE:**  
- Yes
- No
- Unknown

**SLUMP (in):** (report to the nearest ¼-inch)  
**AIR CONTENT (%):** (report to the nearest 0.1%)  
**CONCRETE TEMP (°F):** (report to the nearest 1°F)  
**AIR TEMP (°F):** (report to the nearest 1°F)  
**UNIT WEIGHT (pcf):** (report to the nearest 1 pcf)  
**METHOD OF PLACEMENT:**  
**MIX ID:**  
**CHUTE:**  
**PUMP:**  
**OTHER:**

### Field & Placement Data

- **SLUMP:** 6.01
- **AIR CONTENT:** N/A
- **CONCRETE TEMP:** 28.46
- **AIR TEMP:** N/A
- **UNIT WEIGHT:** N/A

**Temperature:**  
- **CONCRETE TEMP:** 28.46
- **AIR TEMP:** N/A

**Extra Water Authorized By:**

### Location of Concrete or Mortar Placement:

- **Non-shrink grout used to camber bridge deck**

### Location of Concrete or Mortar Sample:

- **Location:** Redstone Arsenal, AL

### Molded in accordance with ASTM C31:

- **YES**
- **NO**
- **UNKNOWN**

### REMARKS:

- Cylinders cast by contractor and field cured for 14 days. Contractor stated that compressive strength of non-shrink grout had to achieve 1,000 psi.

### Compressive Strength Data

<table>
<thead>
<tr>
<th>SPECIMEN ID</th>
<th>DATE SAMPLED</th>
<th>DATE TESTED</th>
<th>AGE (days)</th>
<th>TEST SPECIMEN SIZE</th>
<th>TOTAL LOAD (pounds)</th>
<th>TEST STRENGTH (psi)</th>
<th>TYPE OF FRACTURE</th>
<th>TESTED BY</th>
<th>TESTING LAB</th>
</tr>
</thead>
<tbody>
<tr>
<td>9959 A</td>
<td>01/15/10</td>
<td>01/29/10</td>
<td>14</td>
<td>6.01</td>
<td>28.37</td>
<td>135,350</td>
<td>4,770</td>
<td>HB</td>
<td>HSV</td>
</tr>
<tr>
<td>9959 B</td>
<td>01/15/10</td>
<td>02/12/10</td>
<td>28</td>
<td>6.02</td>
<td>28.46</td>
<td>176,250</td>
<td>6,190</td>
<td>DP</td>
<td>HSV</td>
</tr>
<tr>
<td>9959 C</td>
<td>01/15/10</td>
<td>02/12/10</td>
<td>28</td>
<td>6.02</td>
<td>28.46</td>
<td>200,170</td>
<td>7,030</td>
<td>DP</td>
<td>HSV</td>
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</table>

The average 28-day compressive strength (6610 p.s.i.) is 661% of the design compressive strength.

### Type of Fracture

<table>
<thead>
<tr>
<th>Date of 1st issue:</th>
<th>2/11/10</th>
<th>ResultsReviewed by:</th>
<th>Heath Black, PE</th>
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<tbody>
<tr>
<td>Date of 2nd issue:</td>
<td>2/22/10</td>
<td>ResultsReviewed by:</td>
<td>Heath Black, PE</td>
</tr>
<tr>
<td>Date of 3rd issue:</td>
<td></td>
<td>ResultsReviewed by:</td>
<td></td>
</tr>
<tr>
<td>Final issue:</td>
<td></td>
<td>ResultsReviewed by:</td>
<td></td>
</tr>
</tbody>
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### Distribution of Reports:

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>1</td>
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<td>2</td>
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<td>8</td>
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<tr>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
</tr>
</tbody>
</table>
REPORT OF CONCRETE AND MORTAR COMpressive STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39

PROJECT NAME: Morris Road Bridge
PROJECT NUMBER: HV10003
LOCATION: Redstone Arsenal, AL
CLIENT: Angelo Iafrate Construction
CONTRACTOR: Angelo Iafrate Construction
CONCRETE SUPPLIER: DCA / USA

Distribution of Reports:

1 6
2 7
3 8
4 9
5 10

DATE RCVD: 2/1/10

DESIGN & SPECIFICATION DATA

MIX ID 10C401H52
SPECIFIED STRENGTH (psi) 4,000
SPECIFIED SLUMP (in) 3 - 5
SPECIFIED AIR CONTENT (%) 3 - 6
TEMPERATURE (°F) 50 - 90

FIELD & PLACEMENT DATA

BATCH TIME: 9:19
SAMPLE TIME: 9:45
TRUCK NUMBER: 707
TICKET NUMBER: 50524
SIZE OF LOAD (cy): 8.25
SAMPLED BY: DP

METHOD OF PLACEMENT: CHUTE PUMP OTHER
EXTRA WATER ADDED AT JOB SITE: YES NO UNKNOWN

SLUMP (in): 3 1/2
AIR CONTENT (%): 3.8
CONCRETE TEMP (°F): 55
AIR TEMP (°F): 34
UNIT WEIGHT (pcf): N/A

UNIT WEIGHT (pcf): N/A

Location of Concrete or Mortar Placement:

Barrier rails for west side of bridge

100 ft S of N end of barrier rails

Molded in accordance with ASTM C31:

COMPRESSION STRENGTH DATA

SPECIMEN ID DATE SAMPLED DATE TESTED AGE TEST SPECIMEN SIZE TOTAL LOAD (pounds) TEST STRENGTH (psi) TYPE OF FRACTURE TESTED BY TESTING LAB
9960 A 01/29/10 02/01/10 3 4.01 12.63 36,730 2,910 1 DP HSV
9960 B 01/29/10 02/05/10 7 4.00 12.57 66,640 5,300 1 DP HSV
9960 C 01/29/10 02/26/10 28 4.00 12.57 80,220 6,380 1 DP HSV
9960 D 01/29/10 02/26/10 28 4.00 12.57 80,000 6,360 1 DP HSV
9960 E 01/29/10 02/26/10 28 4.00 12.57 81,430 6,480 1 DP HSV
9960 F 01/29/10 HOLD H

The average 28-day compressive strength (6407 p.s.i.) is 160% of the design compressive strength.

TYPE OF FRACTURE:
1: Cone both ends
2: Cone one end w/ vertical cracks
3: Columnar vertical cracking
4: Diagonal fracture, no cracking
5: Side fractures, top or bottom
6: Side fractures, end pointed

Date of 1st issue: 2/8/10
Results Reviewed by: Heath Black, PE

Date of 2nd issue: 2/26/10
Results Reviewed by: Heath Black, PE

Final issue: Results Reviewed by:
REPORT OF CONCRETE AND MORTAR COMPRESSION STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39

### Design & Specification Data

<table>
<thead>
<tr>
<th>MIX ID</th>
<th>SPECIFIED STRENGTH (psi)</th>
<th>SPECIFIED SLUMP (in)</th>
<th>SPECIFIED AIR CONTENT (%)</th>
<th>TEMPERATURE (°F)</th>
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<tbody>
<tr>
<td>10C40H152</td>
<td>4,000</td>
<td>3 - 5</td>
<td>3 - 6</td>
<td>50 - 90</td>
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</table>

### Field & Placement Data

- **Method of Placement:**
  - **SLUMP (in):**
  - **AIR CONTENT (%):**
  - **CONCRETE TEMP (°F):**
  - **UNIT WEIGHT (pcf):**
    - N/A

### Location of Concrete or Mortar Placement:

- **Barrier rails for bridge:**
  - **Location of Concrete or Mortar Sample:**
    - **70 to 120 feet south of NW Corner**

### COMpressive Strength Data

<table>
<thead>
<tr>
<th>SPECIMEN ID</th>
<th>DATE SAMPLED</th>
<th>DATE TESTED</th>
<th>AGE (days)</th>
<th>TEST SPECIMEN SIZE</th>
<th>TOTAL LOAD (pounds)</th>
<th>TEST STRENGTH (psi)</th>
<th>TYPE OF FRACtURE</th>
<th>TESTED BY</th>
<th>TESTING LAB</th>
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<tbody>
<tr>
<td>10038 A</td>
<td>03/01/10</td>
<td>03/04/10</td>
<td>3</td>
<td>4.00</td>
<td>12.57</td>
<td>45,660</td>
<td>3,630</td>
<td>DP</td>
<td>HSV</td>
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<tr>
<td>10038 B</td>
<td>03/01/10</td>
<td>03/08/10</td>
<td>7</td>
<td>4.01</td>
<td>12.63</td>
<td>53,970</td>
<td>4,270</td>
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<td>HSV</td>
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<tr>
<td>10038 C</td>
<td>03/01/10</td>
<td>03/29/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>65,040</td>
<td>5,150</td>
<td>DP</td>
<td>HSV</td>
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<tr>
<td>10038 D</td>
<td>03/01/10</td>
<td>03/29/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>67,080</td>
<td>5,310</td>
<td>DP</td>
<td>HSV</td>
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<td>10038 E</td>
<td>03/01/10</td>
<td>03/29/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>68,220</td>
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<td>DP</td>
<td>HSV</td>
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<tr>
<td>10038 F</td>
<td>03/01/10</td>
<td>HOLD</td>
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</table>

The average 28-day compressive strength (5287 p.s.i.) is 132% of the design compressive strength.
Distribution of Reports:

<table>
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REPORT OF CONCRETE AND MORTAR COMPRESSIVE STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39

PROJECT NAME: Morris Road Bridge
PROJECT NUMBER: HV10005
LOCATION: Redstone Arsenal, AL
CLIENT: Angelo Iafrate Construction
CONTRACTOR: Angelo Iafrate Construction
CONCRETE SUPPLIER: DCA / USA
SUPERINTENDENT: Kelly Crane

DATE RCVD: 3/5/10

DCM / USA

DATE 1ST ISSUE: 04/01/10
Date of 3rd issue: 03/07/10
Date of 2nd issue: 03/04/10

WEATHER: Cloudy / Cool

SLUMP (in):
ASTM C 143
ASTM C 231
ASTM C 1064

UNIT WEIGHT (pcf):
ASTM C 138

Location of Concrete or Mortar Placement:
Location of Concrete or Mortar Sample:

Barrier rails for bridge
East side rail bottoms beginning at NE Corner and extending 70' south
Molded in accordance with ASTM C31:

REMARKS:

COMPRRESSIVE STRENGTH DATA

<table>
<thead>
<tr>
<th>SPECIMEN ID</th>
<th>DATE SAMPLED</th>
<th>DATE TESTED</th>
<th>AGE (days)</th>
<th>TEST SPECIMEN SIZE</th>
<th>TOTAL LOAD (pounds)</th>
<th>TEST STRENGTH (psi)</th>
<th>TYPE OF FRACTURE</th>
<th>TESTED BY</th>
<th>TESTING LAB</th>
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<tbody>
<tr>
<td>10049 A</td>
<td>03/04/10</td>
<td>03/07/10</td>
<td>3</td>
<td>4.01</td>
<td>12.63</td>
<td>51,280</td>
<td>4,060</td>
<td>1</td>
<td>DP HSV</td>
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<tr>
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<td>03/11/10</td>
<td>7</td>
<td>4.01</td>
<td>12.63</td>
<td>55,230</td>
<td>4,370</td>
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<td>DP HSV</td>
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<td>10049 C</td>
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<td>04/01/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>55,230</td>
<td>4,370</td>
<td>1</td>
<td>DP HSV</td>
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<td>10049 D</td>
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<td>04/01/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>55,230</td>
<td>4,370</td>
<td>1</td>
<td>DP HSV</td>
</tr>
<tr>
<td>10049 E</td>
<td>03/04/10</td>
<td>04/01/10</td>
<td>28</td>
<td>4.01</td>
<td>12.63</td>
<td>55,230</td>
<td>4,370</td>
<td>1</td>
<td>DP HSV</td>
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<tr>
<td>10049 F</td>
<td>03/04/10</td>
<td>HOLD</td>
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<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

The 7-day compressive strength (4370 p.s.i.) is 100% of the design compressive strength.

TYPE OF FRACTURE:
1: Cone both ends
2: Cone one end w/ vertical cracks
3: Columnar vertical cracking
4: Diagonal fracture, no cracking
5: Side fractures, top or bottom
6: Side fractures, end pointed
### Distribution of Reports:

<table>
<thead>
<tr>
<th># of Issue</th>
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<th>2</th>
<th>3</th>
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</table>

### REPORT OF CONCRETE AND MORTAR COMPRRESSIVE STRENGTH USING CYLINDRICAL SPECIMENS – ASTM C39

**Project Name:** Morris Road Bridge  
**Project Number:** HV10003  
**Location:** Redstone Arsenal, AL  
**Client:** Angelo Iafrate Construction  
**Concrete Supplier:** DCA / USA  
**Design & Specification:** Angelo Iafrate Construction  
**Superintendent:** Kelly Crane

### Distribution of Reports:

| Sample No. |  
|------------|---|
| Total      | 1 |

### Field & Placement Data:

- **Date of 1st issue:** 3/9/10  
- **Date of 2nd issue:** N/A  
- **Date of 3rd issue:** N/A  
- **Final issue:** N/A

### Concrete Report No.:

- **Sample No.:** 10057  
- **Sample Date:** 3/9/10  
- **Date Rcvd:** 3/10/10  
- **Weather:** Cloudy / Cool

### Design & Specification Data:

- **Specified Strength (psi):** 4,000  
- **Specified Slump (in):** 3 - 5  
- **Specified Air Content (%):** 3 - 6  
- **Temperature (°F):** 50 - 90

### Field & Placement Data:

- **Batch Time:** 7:32  
- **Sample Time:** 7:55  
- **Truck Number:** 293  
- **Ticket Number:** 51655  
- **Size of Load (cy):** 8.5  
- **Sampled By:** DP

### Method of Placement:

- **SLUMP (in):** 2 3/4  
- **Air Content (%):** N/A

### Concrete Temp (°F):

- **ASTM C 391**  
- **ASTM C 391**  
- **ASTM C 391**

### Location of Concrete or Mortar Placement:

- **Location of Concrete or Mortar Sample:** (should describe the the exact location of the sampled material, e.g. Line A at 5)  
- **Location of Concrete or Mortar Placement:** (should describe the total placement area, e.g. between Column Lines A, E, 1, and 3 or Column Line A from 1 to 3)

### Barrier rails for bridge

- **East side rail 70' south of north end

### Molded in accordance with ASTM C39:

- **YES**

### REMARKS:

- Results Reviewed by: Heath Black, PE

### Compressive Strength Data:

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<tr>
<th>Specimen ID</th>
<th>Date Sampled</th>
<th>Date Tested</th>
<th>Age (days)</th>
<th>Test Specimen Size Diameter (in)</th>
<th>Total Load (pounds)</th>
<th>Test Strength (psi)</th>
<th>Type of Fracture</th>
<th>Tested By</th>
<th>Testing Lab</th>
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<td>HSV</td>
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</tr>
</tbody>
</table>

The 7-day compressive strength (4490 p.s.i.) is 112% of the design compressive strength.
4. TITLE AND SUBTITLE
Field Testing and Summary Report for Road 5 (Morris Road) Over Road 3 (Toftoy Throughway) at Redstone Arsenal, AL: Contractor’s Supplemental Report for Project F09-AR16

6. AUTHOR(S)
Brett Commander and Scott Aschermann

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
Bridge Diagnostics, Inc. (BDI)
1965 57th Court North
Suite 106
Boulder CO 80301

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Research and Development Center
Construction Engineering Research Laboratory
PO Box 9005
Champaign, IL 61826-9005

14. ABSTRACT
Cyclic loading and weathering of reinforced concrete bridge decks causes corrosion of reinforcement steel, which leads to cracking, potholes, and other problems. Under the Department of Defense Corrosion Prevention and Control Program (Project F09-AR16), a deteriorated concrete bridge at Redstone Arsenal, Alabama, was selected to demonstrate and validate a glass-fiber reinforced polymer (GFRP) composite deck system, which does not use any reinforcement steel. The results of that project were published as ERDC/CERL TR-16-6 (August 2016). Upon completion of the new GFRP composite deck system, Bridge Diagnostics, Inc. (BDI) was contracted to perform load testing to confirm that the bridge meets the structure’s original 36-ton (HS-20) load rating and performance criteria for deflection and strain. This report documents the load test methods used by BDI and the results. The test results indicate that the demonstrated GFRP composite deck system met the strength design specifications and passed the deflection criteria.

15. SUBJECT TERMS
Glass fibers; Concrete–Service life; Concrete bridges–Floors; Polymeric composites; Fibrous composites; Concrete bridges–Maintenance and repair; Reinforced concrete—Corrosion; Materials—Dynamic testing

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(Include area code)