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Demonstration and Validation of a Composite Grid Reinforcement System for Bridge Decks

Final Report on Project F12-AR01

Steven C. Sweeney, Richard G. Lampo, James Wilcoski, Christopher Olaes, and Larry Clark

September 2016

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and Control Program

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Final report

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Under Project F12-AR01, “Demonstration and Validation of 3-D Gridform for Bridges”

Abstract

The Department of Defense (DoD) maintains a large array of road networks that include vehicular bridges. Moving people, materials, and equipment is critical to the DoD mission. Many of these bridges are in dire need of major repairs or replacement due to corrosion and material degradation. The application of corrosion-resistant technology can extend the service life of bridges and reduce maintenance costs. This DoD Corrosion Prevention and Control Program project demonstrated and validated the performance characteristics of the fiber-reinforced polymer (FRP) composite, three-dimensional Gridform product for reinforcing concrete bridge decks that was designed to solve many of the installation and life-cycle problems associated with steel-reinforced bridge decks. The Gridform technology replaced the existing steel-reinforced concrete deck on one span of Bridge No. 4 at Fort Knox, Kentucky. The newly replaced span's performance was compared to a second span that was newly replaced with a concrete deck using traditional steel rebar reinforcement. Structural testing and corrosion monitoring and analysis of the bridge was performed. Results show that using Gridform technology could provide needed load capacity and improved corrosion protection for DoD bridges, while maintaining structural capability. The technology's return on investment (ROI) is 10.31.

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Preface

This demonstration was performed for the Office of the Secretary of Defense (OSD) under Department of Defense (DoD) Corrosion Control and Prevention Project F12-AR01, “Demonstration and Validation of 3-D Gridform for Bridges.” The proponent was the U.S. Army Office of the Assistant Chief of Staff for Installation Management (ACSIM), and the stakeholder was the U.S. Army Installation Management Command (IMCOM). The technical monitors were Daniel J. Dunmire (OUSD(AT&L)), Bernie Rodriguez (IMPW-FM), and Valerie D. Hines (DAIM-ODF).

The work was performed 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. Significant portions of this work were performed by Mandaree Enterprise Corporation, Warner Robins, GA. At the time this report was prepared, Vicki L. Van Blaricum was Chief, CEERD-CFM; Donald K. Hicks was Chief, CEERD-CF; and Kurt Kinnevan, CEERD-CZT, was the Technical Director for Adaptive and Resilient Installations. The Deputy Director of ERDC-CERL was Dr. Kirankumar Topudurti, and the Director was Dr. Ilker Adiguzel.

The contributions of Brett Commander of Bridge Diagnostics Inc. are gratefully acknowledged. Also, the following personnel are gratefully acknowledged for their support and assistance in this project:

- Mr. Jay Schmidt – Directorate of Public Works, Fort Knox, KY
- Mr. John Wiseman – Directorate of Public Works, Fort Knox, KY
- Mr. Rodney Mason – Installation Range Control Officer, Fort Knox, KY
- Mr. Butch Faust – All Cities Enterprises, Fort Knox, KY

COL Bryan S. Green was the Commander of ERDC, and Dr. Jeffery P. Holland was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
gallons (U.S. liquid)	3.785412 E-03	cubic meters
in.	0.0254	meters
mils	0.0254	millimeters
square feet	0.09290304	square meters

1 Introduction

1.1 Problem statement

The Army has installations around the world, and many of these have bridges as a significant part of their infrastructure. These bridges, like those in our national highway system, are experiencing significant deterioration from corrosion of the steel structures and/or the steel reinforcement in the concrete. Federal Highway Administration (FHWA) Report RD-01-156 (Koch et al. 2002) states that approximately one-quarter of the direct cost of corrosion of bridges is made up of maintenance and capital costs for steel reinforcement. Maintaining serviceable bridges is essential to providing access to the facilities on the post and to remote training areas that would otherwise be in accessible due to rivers, streams, trains, roads, and other geographical obstacles to transportation. Thus the cost for maintenance and replacement of bridge infrastructure has a big impact on the Army and its operations.

The current technology employed in the existing bridge infrastructure typically has a 50-year design life; however, according to the Illinois and New York state departments of transportation—two states where road salts are used extensively for deicing—the average service life of a steel-reinforced concrete bridge deck is 25 years. (Hastak, Halpin, and Hong 2004). The inventory for the Army's bridge safety program shows that more than 80% of its bridges are standard steel, concrete or steel and concrete construction (Dean 2008). Bridges are exposed to all the climate conditions as well as heavy industrial contaminates. Bridges are exposed to all climate conditions and often are exposed to heavy industrial contaminates as well. Both design and construction experience show that this exposure is currently an added problem for corrosion because it results in cracking and spalling of concrete beams and corrosion of steel beams. In addition, bridges located in northern regions are frequently exposed to deicing salts in winter weather, and in coastal areas, they are exposed to splash zone sea water—both conditions accelerate corrosion problems.

New technologies employing corrosion-resistant composite materials are still under development and evaluation as replacements for steel and concrete. The validation and implementation of these technologies will allow Department of Defense (DoD) installations to utilize them for replacing or rehabilitating corroding bridge structures. Use of the new technologies could reduce maintenance costs, sustain the mission, and prevent premature failure of infrastructure.

This DoD Corrosion Prevention and Control (CPC)-funded project was a collaboration between the Engineer Research and Development Center–Construction Engineering Research Laboratory (ERDC-CERL) and the Fort Knox Directorate of Public Works (DPW). The Fort Knox DPW has an ongoing initiative to replace or rehabilitate bridges throughout the installation's vast training range that are severely corroded. Fort Knox is a training base for the Army's mobile armor combat, and its bridges must carry some of the Army's heaviest vehicles. These vehicles include the M1A1 Abrams Battle Tank, M2A3 Bradley, and Heavy Equipment Transporter (HET). When carrying the M1A1, a HET has a combined weight of at least 105 tons. The HETS and M1A1 were used in the second load test of the bridge to validate the ability of the grid reinforced concrete deck to perform to those demanding requirements.

Bridge No. 4 (Figure 1) in the Fort Knox training range was one of the bridges scheduled for rehabilitation of its corroded support beams and deteriorating bridge deck. With Bridge No. 4 having two spans, it was an excellent candidate for concurrent demonstrations of a hybrid-composite beam (HCB) and the grid composite deck reinforcement technologies.

Figure 1. Bridge No. 4 at Fort Knox.

1.2 Objective

The objective of this project was to demonstrate and validate the performance characteristics of a commercially available fiber-reinforced polymer (FRP) composite grid element as an alternative to conventional steel bar as the reinforcing material in concrete bridge decks.

1.3 Approach

The selected demonstration structure was Bridge No. 4, a two-span bridge on the training range at Fort Knox where the support beams and bridge deck were scheduled for replacement. This bridge served as the site for two separately funded but concurrent CPC projects—the one documented in this report and Project F12-AR15, documented in ERDC/CERL TR-16-22 (Sweeney et al. 2016). The demonstration documented here involves the use of composite-gridform concrete-reinforcement technology (GRIDFORM™)[1] for use as a bridge deck supported by conventional steel beams. The other span of the bridge (used in Project F12-AR15) demonstrated HCBs on one span of the bridge to support a standard steel-reinforced concrete deck.

After demolition of Bridge No. 4 deck and support beams, the bridge was then restored utilizing the new technologies to demonstrate their capabilities. Sensors were installed to evaluate corrosion rates on the new bridge structure, and load tests were performed to assess the structural performance of the new bridge technologies.

1.4 Metrics

The corrosion potential of the site was determined using the combination of exposed atmospheric coupons, collected weather data and embedded corrosion sensors. The atmospheric coupon rack was built and tested in accordance with ASTM G1-03 "Standard Practice for Preparing, Cleaning and Evaluating Corrosion Test Specimens" with the exception of the silver coupons. The silver coupons were tested in accordance with ASTM B825, "Standard Test Method for Coulometric Reduction of Surface Films on Metallic Tests." The results from testing the atmospheric coupons and the collected weather data were analyzed using ISO 9223:2012, "Corrosion of

[1] GRIDFORM (appearing in this report as Gridform) is a registered trademark of the Strongwell Corporation of Bristol, Virginia.

Metal and Alloys – Corrosivity of Atmospheres – Classification, Determination and Estimation." A summary of the results of the analysis are provided in section 3.1.1. Details of the corrosion potential analysis are presented in Appendix B of this report.

Ease of installation was also observed during construction, and empirical information collected from the construction contractor.

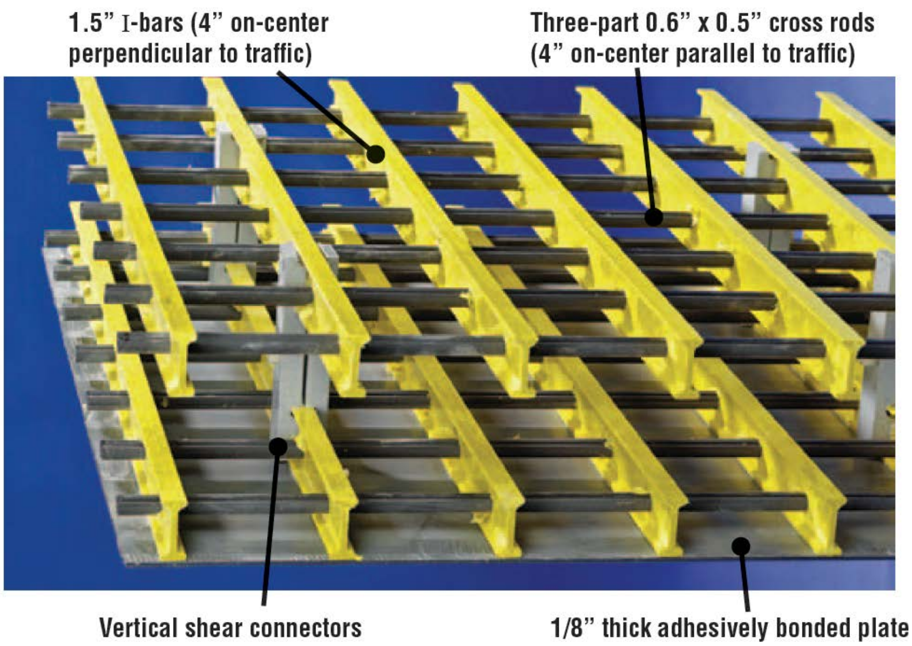
Bridge load tests were conducted immediately after construction and one year after construction to ensure that the composite-gridform concrete-reinforcement technology meets the original design requirements.

2 Technical Investigation

2.1 Technology overview

A Gridform bridge deck system is designed to replace steel rebar in reinforced concrete bridge decks. Gridform consists of two layers of pultruded grating members that are separated by FRP shear connectors with nylon bolts. The grating is formed from I-shaped cross-sections (called I-bars) that are situated in the lengthwise direction 4 in. on center, with cross rods in the perpendicular direction also on 4 in. spacing. Gridform also has a 1/8 in. pultruded FRP plate bonded to the bottom grating layer that creates a stay-in-place concrete form which simplifies installation and reduces construction time (Figure 2; Bank, Olivia, and Brunton 2011).

Figure 2. Section of the 3-D Gridform reinforcement.



Gridform was designed to solve many of the installation and life-cycle problems associated with steel-reinforced bridge decks. This lightweight, high-strength system eliminates time-consuming and labor-intensive steps such as setting forms and tying rebar (Bank, Oliva, and Brunton 2011). Observed failure loads from manufacturer laboratory testing are listed in Table 1.

Table 1. Manufacturer's laboratory test results for FRP I-bar concrete bridge deck panels (Strongwell Corp.).

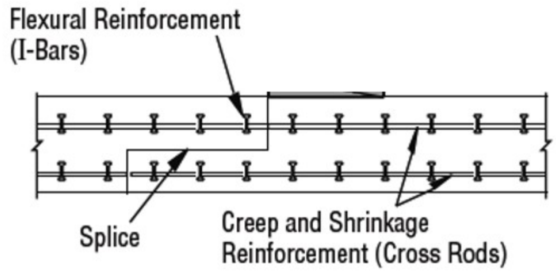
FRP I-bar Size and Spacing	Slab Depth	Slab Span	Slab Length	Slab Width	Concrete Compressive Strength, f'c (psi)	Predicted Failure Loads: Flexure, ACI 440 (kips)	Predicted Failure Loads: Punching Shear, UW-Madison (kips)	Predicted Failure Loads: Flexural Shear, ACI 318 (kips)	Tested Failure Loads (kips)
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	7.625"	6'-4"	8'-0"	7'-0"	4350	97.3	115.3	122.2	125
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	6'-6"	7'-6"	6'-6"	5347	86	119.5	127.6	119.3
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	6'-6"	7'-6"	6'-6"	5343	93.5	120.5	127.5	120.6
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	6'-6"	7'-6"	6'-6"	5507	94.7	121.7	129.5	121.8
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	7'-6"	8'-6"	7'-8"	6854	107.6	121.9	158.3	121
1-1/2" I-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	8'-6"	9'-6"	8'-8"	4652	89.8	107.2	141.8	109.4
2" T-bars at 4" o.c., 1/2" dia cross rods at 4" o.c.	8"	8'-6"	9'-6"	8'-8"	4630	101.9	114.2	140.1	115.7

2.2 Field work

Bridge No. 4 is an 82 ft long bridge divided into two spans (see Engineering drawings in Appendix A). The south-facing span's deck was replaced with the FRP I-bar reinforcement on top of conventional steel girders, and the north-facing span was replaced with conventional rebar reinforcement deck on top of HCBs.

The FRP reinforcement sections were delivered to the site precut by the manufacturer (approximately 4 x 27 ft) to fit the dimensions of the bridge, with each layer on each section offset to create a splice joint (Figure 3). The two end sections did not contain the layer offset. Figure 4–Figure 6 depict different views of section installation, showing that only one crane lift is needed for placement. Once the sections were lowered onto the beams, the sections could be manipulated into position by hand. Ten sections were installed in the 38.5 ft span. The FRP reinforcement was fastened to the longitudinal steel beams by sandwiching two FRP blocks around the steel beam top flange and the bottom layer FRP I-bars, compressed by stainless-steel bolts, as shown in Figure 8 and Figure 9. Concrete was poured onto the deck using a conventional concrete truck and pump, followed by a concrete vibrator and hand spreading concrete over the FRP reinforcement (Figure 10–Figure 11).

Figure 3. Gridform cross-section showing panel overlap at the splice.



Installation of the FRP reinforcement sections, not including the concrete pour, was completed in 4 hours by a three-person crew and a crane operator. The sections used in this project were light enough to be manipulated into place by hand (Figure 7), once they were positioned by the crane. The labor savings attributable to the section's light weight is a major advantage during installation of this technology. The construction contractor estimated that deck construction time was reduced by at least 80% on this span. More rapid installation of the panels allowed much faster placement of the concrete, saving further time and labor. The Bridge No. 4 contractors estimated that the reduced manpower requirement and speed of installation led to a 75% reduction in total labor costs for this deck, compared to the adjacent steel reinforced concrete deck documented in the companion demonstration report, ERDC/CERL TR-16-22 (Sweeney et al. 2016). The finished bridge deck is shown in Figure 12.

Figure 4. Crane lift of FRP reinforcement panels.

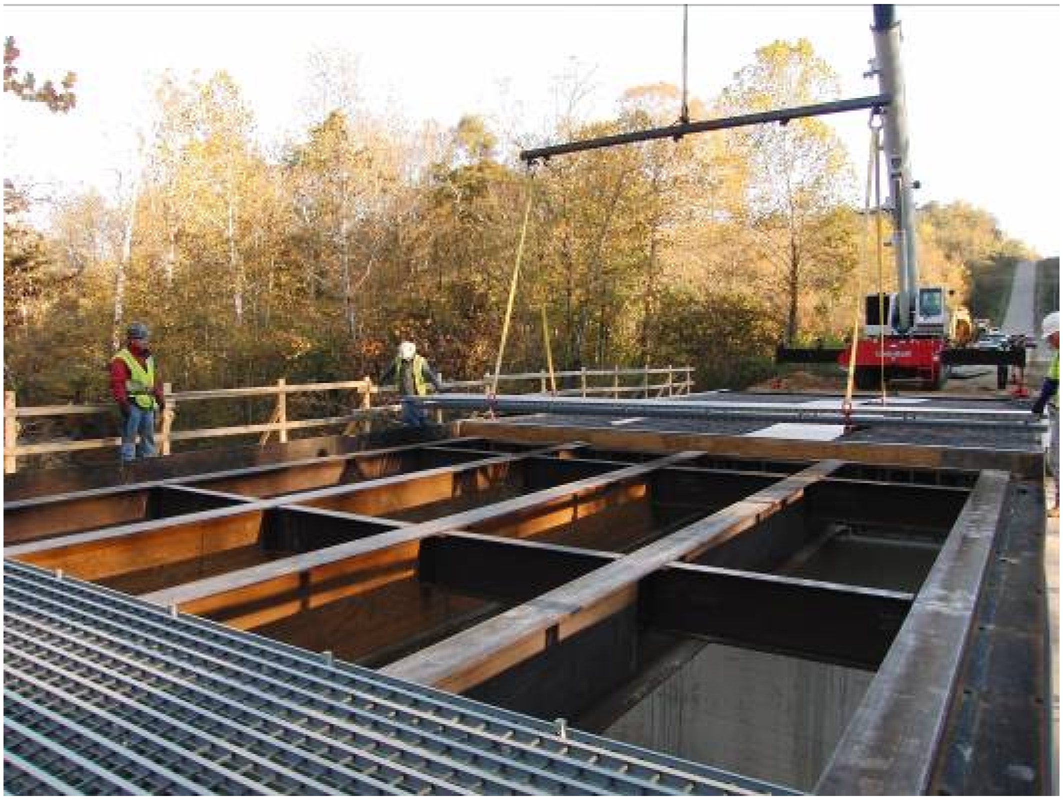


Figure 5. Placing FRP reinforcement panels by crane.

Figure 6. Guiding the FRP reinforcement panel installation.

Figure 7. Manipulating the FRP reinforcement panels by hand.

Figure 8. Fastening the bottom layer of FRP I-bars to the steel beam using FRP blocks and stainless steel bolts, shown from above.

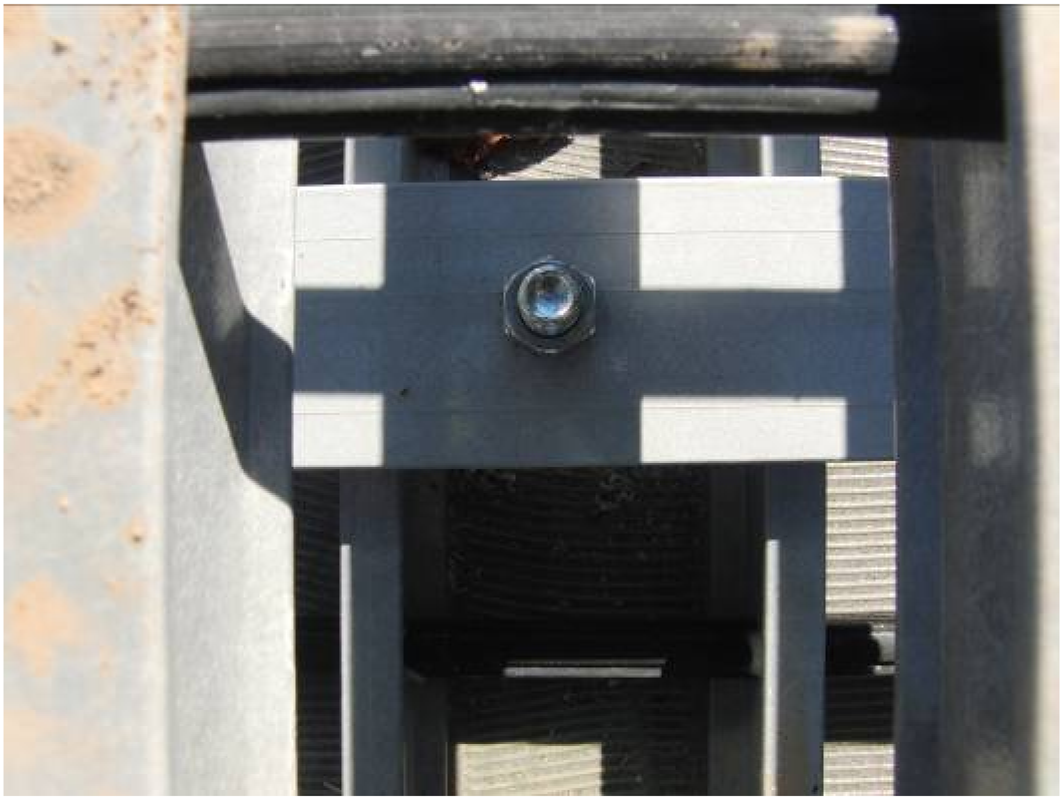


Figure 9. Fastening the FRP reinforcement panels to the top flange of the steel beam, using FRP blocks and stainless steel bolts (shown from below).

Figure 10. Concrete pouring and vibrating.

Figure 11. Spreading of concrete over the FRP reinforcement.

Figure 12. Finished bridge deck with the Gridform section in the foreground, and the conventional deck in the background (break is at the guardrail gap).

2.3 Commissioning and monitoring

The Fort Knox DPW contractor, All Cities Enterprises, completed the construction of Bridge No. 4, but monitoring could not begin until after the approaches and guard rails were complete. MEC monitored the bridge for one year. The evaluation of the FRP reinforcement was accomplished by conducting two load tests on Bridge No. 4. Construction of Bridge No. 4, including the deck using the FRP reinforcement, was finished in October 2012, but the guard rails and approaches were not finished until December 2012. Only at that time could the initial load test be scheduled and conducted. The final load test was conducted in December 2013. This evaluation was to assess the performance of the FRP reinforcement and to verify the bridge meets design load requirements. The load tests were performed by Bridge Diagnostics, Inc. of Boulder, Colorado. Details of the monitoring and testing are available in the subcontractor's report, ERDC/CERL CR-16-4 (Commander and Carpenter 2016).

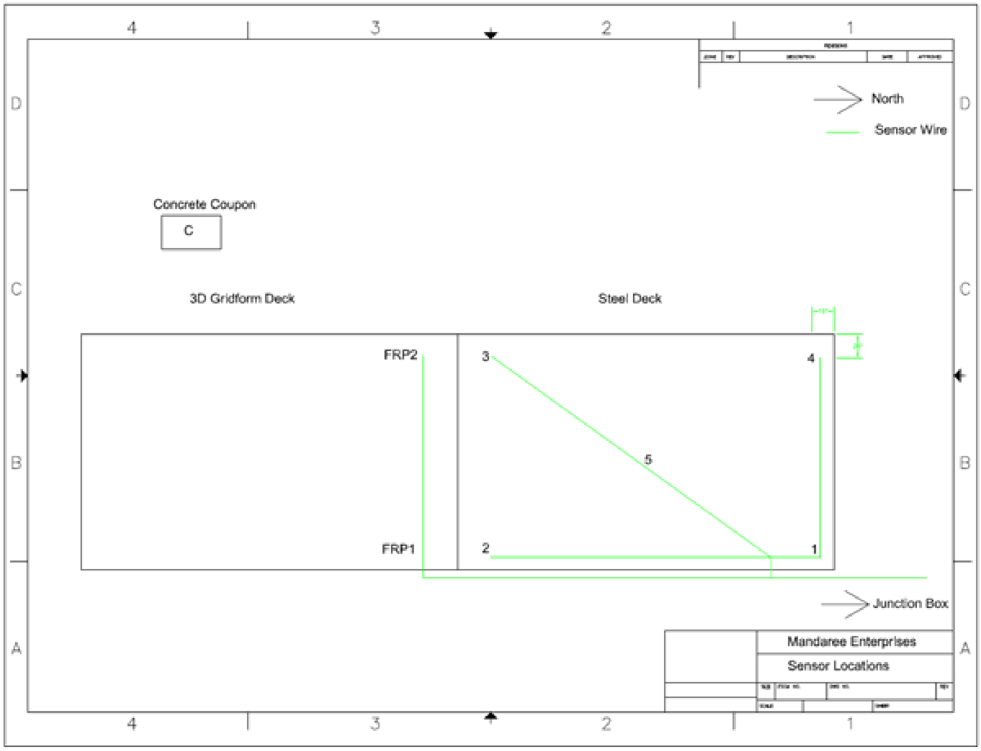
An evaluation of the corrosion potential of the site was done with the use of a weather station, corrosion sensors embedded in the concrete deck, and an atmospheric corrosion test rack. The weather station measured temperature, relative humidity, wind speed and direction, and rainfall. The weather station was powered by a solar panel and a rechargeable battery. A data logger was used to store the measurements which were recorded every 12 hours by the rain gage and every 15 minutes for the remaining sensors.

The corrosion sensors were installed in the bridge deck to measure chloride penetration, corrosion potential and corrosion rate. Measurements from the sensors were taken quarterly for a 12-month period. The selected sensors were Rohback Cosasco 900 Concrete Multi-Depth Sensors, Borin Stelth 7 sensors, and Rohback Cosasco 800 LPR Corrosion Rate sensors. The positions of the sensors are given in Table 2 and Figure 13.

Table 2. Corrosion sensor location key (see Figure 13 for location of sensors by span).

Location	Rohback Cosasco 900 Chloride	Rohback Cosasco 800 Corrater	Borin Stelth 7 Potential
1	X	X	X
2		X	X
3		X	X
4	X	X	X
5	X	X	XX
FRP1	X		
FRP2	X		
Salt Coupon	X	X	

Figure 13. Corrosion sensor location plan view.



The site was also visited quarterly, during which times weather station data was downloaded and readings were taking from the corrosion sensors. At the 6-month and 12-month points, coupons were retrieved from the corrosion test rack and assessed by a laboratory. Establishment of the site corrosion potential will be used to evaluate the potential future performance of the FRP reinforcement material. The corrosion potential of the site was determined using the combination of exposed atmospheric coupons, collected weather data, and embedded corrosion sensors. The atmospheric coupon rack was built and tested in accordance with ASTM G1-03, "Standard Practice for Preparing, Cleaning and Evaluating Corrosion Test Specimens." Appendix B provides a summary of the data recorded for the corrosion potential and an interpretation of the results to form the site corrosion potential.

3 Discussion

3.1 Results

3.1.1 Site corrosion potential

A summary of the results and classification from ISO 9223:2012 are listed in Table 3–Table 6.

The results from the ISO 9223:2012 analysis of weather data and mass loss testing suggest the Fort Knox Bridge No. 4 site is a C3 classification of atmospheric corrosion severity. Although the steel coupon testing resulted in a C2 classification, the results were on the upper limit of the category. The potential for atmospheric corrosion at the site is considered medium. The embedded corrosion sensors show no corrosion in the bridge deck over the 12 months of monitoring. Road salts are reportedly used on this bridge, given its location at the base if a steep hill. It make take more than a year for the chlorides to penetrate into the new concrete.

Copper experienced a high mass loss in comparison to the other metals in both the 6-month and 12-month tests (Table 4 and Table 5). Results from the 12-month testing suggest that the 2024 and 7075 aluminum alloys experienced an extremely high mass loss due to corrosion. These results are inconsistent with the other alloys and the results from the weather data analysis; therefore, the mass loss test from the 12-month 7075 and 2024 coupon have been omitted from the atmospheric corrosion severity classification of the site (Table 6).

Table 3. Summary of weather data collected December 2012 - December 2013.

	Wind Direction, ø	Wind Speed, mph	Gust Speed, mph	Temp, °F	RH, %
Average	192	0.25	2.5	56	83
Std Deviation	100	0.91	3.6	18	18.9

Table 4. Summary of results from the 6-month ASTM G1 mass loss test and corrosion classification per ISO 9223:2012.

	1010 Steel	CDA101	Al6061-T6	Al2024-T3	Al7075-T6
Weight loss [g]	0.104	0.417	0.005	0.003	0.005
Rcorr [g/m2y]	37.71	151.66	1.95	0.94	1.74
Classifiction	C2 (Low)	CX (Extreme)	C3 (Med)	C3 (Med)	C3 (Med)

Table 5. Summary of results from the 12-month ASTM G1 mass loss test and corrosion classification per ISO 9223:2012.

	1010 Steel	CDA101	Al6061-T6	Al2024-T3	Al7075-T6
Weight loss [g]	0.984	0.143	0.006	0.294	0.192
Rcorr [g/m2y]	178.9	25.91	1.05	53.36	34.96
Classifiction	C2 (Low)	C5 (Very High)	C3 (Med)	CX (Extreme)	CX (Extreme)

Table 6. Atmospheric corrosion severity classification from weather data and ISO 9223:2012 response equation calculations.

	Steel	copper	aluminum	zinc
Rcorr [um/y]	9.67	0.88	0.04	0.54
Classifiction	C2 (Low)	C3 (Med)	-	C2 (Low)

3.1.2 Load testing

Load tests were accomplished as part of a concurrent OSD demonstration of the longitudinal structural beams. Specific details and results of the tests are discussed in the related contractor's report, published as ERDC/CERL CR-16-4 (Commander and Carpenter 2016). The load testing did not evaluate the deck, but both decks were subjected to the same loads by a Heavy Equipment Transport Semitrailer (HETS) that was hauling an M1A1 Abrams tank. Neither deck showed any visual signs of failure from the load tests. The FRP reinforcement panels met or exceeded manufacturer's design load ratings and performed similarly to the reinforced-concrete deck on the adjacent span. By achieving this level of performance, the bridge can be used for its intended purpose at the design loads without any modification or load-limit reductions.

3.1.3 Installation methods

Field experience verified that use of this composite grid reinforcing system in a concrete bridge deck improved ease of installation and reduced labor requirements, as anticipated. The Gridform sections used in this project were light enough to be adjusted into place by hand once they were placed into their approximate positions by crane, eliminating the need for extended or multiple crane lifts. As estimated by the Bridge No. 4 contractor, these benefits reduced the construction time for the deck by approximately 80%, with a pursuant total 75% reduction in labor costs as compared to work performed on the adjacent steel-reinforced concrete deck that is documented in ERDC/CERL TR-16-22 (Sweeney et al. 2016).

3.2 Lessons learned

Even though the Gridform product is a novel emerging technology, the project team and contractors encountered no problems or unusual field-work requirements during project execution. The ease of installation claimed by the manufacturer was validated in this work through a side-by-side comparison with work required for installing a standard steel-reinforced concrete deck on the other span of Bridge No. 4, as part of the companion CPC-funded demonstration of HCBs as covered in ERDC/CERL TR-16-22 (Sweeney et al. 2016).

4 Economic Summary

4.1 Costs and assumptions

Although formal documentation of contractor productivity statistics were specified as a deliverable product of this work, ERDC-CERL was unable to obtain detailed written records through the prime contractor for this demonstration. However, as reported in section 3.1.3, field observations and discussions with the construction subcontractor indicated that construction of the bridge deck using the Gridform system resulted in a 75% reduction in total labor cost to install, compared to conventional steel reinforcing. Consequently, some of the assumptions and costs developed in the original project management plan (PMP) have been changed, but are noted in text that follows. Actual project costs for conducting the demonstration and evaluation project are listed in Table 7 and Table 8.

Table 7. Breakdown of total project costs.

Description	Amount, $K
Labor	47.6
Support from Fort Knox for bridge construction	305.0
Cost for Gridforms	100.0
Cost for Chloride Sensors	10.5
Contract for monitoring and testing	101.9
Travel	25.0
Reporting	20.0
Air Force and Navy participation	
Total	610.0

Table 8. Project field demonstration costs.

Item	Description	Amount, $K
1	Labor for project management and execution	49.2
2	Travel for project management	11.8
3	Cost for materials	5.2
4	Cost for corrosion analysis	7.6
5	Cost for load tests	28.1
	Total	101.9

Costs for the return on investment (ROI) computation are based on the assumption that 30 bridge decks on Army installations will be repaired in

Year 2, using this technology. These bridges currently have reinforced concrete bridge decks over steel girder substructures and an average span of approximately 60 feet. They are in various state of disrepair, with many needing deck replacements. This analysis considers 30 bridges having 60-foot spans and 35-foot widths. Thus, each bridge would be figured at 2,100 square feet (sf). Service life projection for FRP decks has been set at 75 years (O'Connor 2003). The average service life of a reinforced concrete deck in locations where road salts are used is 25 years (Hastak, Halpin, and Hong 2004).

The costs for replacement of an existing steel-reinforced concrete bridge deck include costs for disposal, construction, and traffic control incurred during construction. The following costs adjusted to 2008 dollars were used: disposal = $12.40/sf, construction = $42.00/sf, and traffic control = $24.00/sf. (The traffic control delay is 25 days based on discussion with Fort Knox DPW regarding normal deck replacement schedules.)

The construction cost for an FRP composite bridge deck, adjusted to 2008 constant dollars, is $61.55/sf. The original assumption was $77.30/sf, but the previous assumption is reduced here due to the observed labor cost reductions. Disposal costs are the same as for the concrete bridge deck. Using a construction time reduced by two-thirds as compared to reinforced concrete construction, the traffic delay costs for FRP is $8.00/sf.

The following preventive maintenance requirements can be expected for steel-reinforced concrete decks (in 2008 dollars) according to the Virginia Transportation Research Council:

1. Joint sealing costs of $55 per linear foot can be expected to replace pourable joint sealant every 6 years. Maintenance costs for 30 bridges (assumed length of 60 ft each) total $99,000.
2. Pothole repair costs of $150,000 for the first year after deck construction and escalating $10,000 a year until the bridge is replaced.

Finally, a benefit has been added to the FRP option for avoidance of traffic control and delayed costs for the maintenance. For the pot hole fixing, 3 days of traffic cost avoidance (equal to a savings of $2.88/sf) is saved for each bridge for the first year. This results in savings of $181,440 for the first year for 30 bridges, and the savings is escalated by $13,600 per year until the steel-reinforced concrete bridge deck is redone after 25 years. For joint sealing costs, two days of traffic interruption is avoided at six-year intervals, resulting in a savings of $120,960 for each bridge.

4.2 Projected return on investment (ROI)

Over 30 years, using the methods prescribed by Office of Management and Budget (OMB) Circular A-94 (OMB 1992) and the above revised assumptions, the projected ROI for this demonstration is 10.31 (Table 9). The original ROI in the PMP was projected at 10.55.

Table 9. Project return on investment calculation.

Investment Required			610
Return on Investment Ratio	10.31	Percent	1031%
Net Present Value of Costs and Benefits/Savings	4,509	10,800	6,291

A Future Year	B Baseline Costs	C Baseline Benefits/Savings	D New System Costs	E New System Benefits/Savings	F Present Value of Costs	G Present Value of Savings	H Total Present Value
1							
2	4,939		5,163		4,509	4,314	-196
3	150			181		271	271
4	160			195		271	271
5	170			209		270	270
6	180			222		268	268
7	190			236		265	265
8	299			370		390	390
9	210			263		257	257
10	220			277		252	252
11	230			290		247	247
12	240			304		241	241
13	250			317		235	235
14	359			452		315	315
15	270			345		223	223
16	280			358		216	216
17	290			372		210	210
18	300			385		203	203
19	310			399		196	196
20	419			534		246	246
21	330			426		183	183
22	340			440		176	176
23	350			453		169	169
24	360			467		163	163
25	370			481		157	157
26	4,939					850	850
27	150			181		53	53
28	160			195		53	53
29	170			209		53	53
30	180			222		53	53

5 Conclusions and Recommendations

5.1 Conclusions

The 3D FRP reinforcement is a strong and lightweight alternative to traditional steel reinforcement for concrete bridge decks. Both the FRP composite and the steel-reinforced deck did not show any signs of failure during the load testing that included loads from a Heavy Equipment Transport Semitrailer (HETS) that was hauling an M1A1 Abrams tank.

The FRP composite reinforcement is not susceptible to the effects of corrosion induced by the presence of chlorides from road salts that can penetrate into the concrete deck and cause corrosion of standard steel reinforcement.

Installation of the concrete bridge deck FRP reinforcement system is greatly facilitated by the lightweight nature of the panels. Large panels can be placed with a single lift of a crane and then maneuvered into position by hand. This advantage reduced the construction time for the Fort Knox bridge deck by approximately 80%. Quicker installation of the panels led to a much faster rate of placement for the concrete, saving additional time and labor. The construction subcontractor estimated that with the reduced need for manpower and the speed of installation, total deck construction labor costs were reduced by more than 75% for the deck when compared to the adjacent steel reinforced deck. The ROI ratio calculated for this technology over 30 years is 10.31.

5.2 Recommendations

5.2.1 Applicability

As the DoD continues to replace aging bridges and construct new roadways, FRP reinforcement provides a corrosion-resistant technology that extends the life cycle of the structure, reduces maintenance costs, and improves the logistics of construction while maintaining performance standards.

The benefits expressed in the above paragraph make the FRP reinforcement panel decks a good choice for any bridge in the DoD inventory. In addition, this technology's benefits would be especially appropriate for bridges in highly corrosive environments such as those were road salts are routinely used or salt spray is present. It would also be especially appropriate for high-traffic bridges, bridges in difficult-to-reach locations, and critical-path bridges.

5.2.2 Implementation

Per Unified Facilities Criteria (UFC) 3-301-01, "Structural Engineering," the design of highway bridges shall be in accordance with AASHTO bridge design specifications (AASHTO 2007). There is an AASHTO specification for "LRFD[1] Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings" (AASHTO 2009); however, this specification does not specifically cover the Gridform design, which is a proprietary modification of a conventional rebar system design.

The manufacturer of the Gridform system, Strongwell Corporation, has developed a software program and a design manual specifically for their system (Bank, Oliva, and Brunton 2011). The program was developed using code from the following specifications: (1) AASHTO "LRFD Bridge Design Specifications (2007); (2) "LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings" (AASHTO 2009); and (3) "Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars, ACI 440.1R-06 (ACI 2006).

Use of the manufacturer's software and design manual (Bank, Oliva, and Brunton 2011) will enable safe and reliable use of the Gridform reinforcing system for replacing concrete bridge decks in locations with highly corrosive environments (e.g., coastal sites or where road deicing salts are used extensively) such as Bridge No. 4 at Fort Knox.

There also is currently no guidance in any UFC or Unified Facilities Guide Specification (UFGS) documents with regard to FRP composite bridge decks. Under an FY15-funded project, "Composites for Bridge Applications," ERDC-CERL is currently developing a new UFC for the use of FRP composites in bridge structures. FRP composite reinforcing elements, such as the Gridform system for bridge decks, will be included in this new guidance. Publication of this new UFC is expected in 2017. This guidance

[1] LRFD stands for "Load and Resistance Factor Design."

should facilitate wider use of FRP composites for bridge applications in advance of future AASHTO guidance.

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Appendix A: Engineering Drawings for Bridge No. 4, Fort Knox, Kentucky

Appendix A contains the engineer design and specification drawings as prepared by Bridge Diagnostics, Inc. for both of the demonstrated and evaluated spans of Bridge No. 4 at Fort Knox, Kentucky.

Bridge Diagnostics, Inc. did not produce separate drawings for each span; therefore, all the drawings are included in Appendix A, covering both the composite grid reinforcement system span (this report) and the hybrid composite bridge beams span (ERDC/CERL TR-16-22).

Figure A1. General structural and bridge notes for Bridge No. 4, Fort Knox.

BRIDGE NUMBER 4, FORT KNOX MILITARY RESERVATION

FORT KNOX, KENTUCKY

GENERAL NOTES – STRUCTURAL

CONCRETE NOTES

1. ALL CONCRETE SHALL BE 4,000 PSI COMPRESSIVE STRENGTH AT 28 DAYS. SLAB CONCRETE SHALL BE AIR-ENTRANED WITH AIR RATIO TARGET 6% -1 +2% AND USE 1" MAXIMUM COARSE AGGREGATE; TARGET W/C RATIO 0.45 OR LESS.
2. CONCRETE SHALL BE DESIGNED, MIXED, PLACED, TESTED AND CURED IN ACCORDANCE WITH AMERICAN CONCRETE INSTITUTE BUILDING CODE (318-08). SLUMP SHALL BE 1" MINIMUM; 3" MAXIMUM. USE SUPER PLASTICIZER TO ADJUST SLUMP AS NECESSARY. DO NOT ADD WATER TO THE MIX AFTER THE TRUCK HAS LEFT THE BATCH PLANT.
3. ALL REINFORCING STEEL SHALL BE NEW BILLET GRADE 60 AND CONFORM TO ASTM 615 SPECIFICATIONS. REINFORCING STEEL PLACEMENT SHALL BE IN ACCORDANCE WITH CONCRETE REINFORCING STEEL INSTITUTE (CRSI) MANUAL OF STANDARD PRACTICE.
4. LAP ALL REINFORCING STEEL 30 DIAMETERS OR 12" MINIMUM UNLESS OTHERWISE NOTED.
5. REINFORCING STEEL BENDS SHALL BE MADE AS SHOWN ON DRAWING AND/OR IN ACCORDANCE WITH THE AMERICAN CONCRETE INSTITUTE (ACI) CODE.
6. ALL ITEMS TO BE CAST INTO CONCRETE SUCH AS REINFORCING DOWELS, BOLTS, ANCHORS, PIPES, SLEEVES, ETC., SHALL BE SECURELY AND ACCURATELY POSITIONED INTO THE FORMS PRIOR TO PLACING THE CONCRETE.
7. USE HILTI HY150 EPOXY TO SET REINFORCING DOWELS INTO EXISTING CONCRETE.

STEEL NOTES

1. ALL WF STRUCTURAL STEEL SHALL CONFORM TO ASTM A992 STEEL SPECIFICATIONS. ALL OTHER STEEL SHAPES SHALL CONFORM TO ASTM A36 STEEL SPECIFICATIONS. ALL STEEL SPECIFICATIONS ARE AS OUTLINED IN AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) "MANUAL OF STEEL CONSTRUCTION".
2. ALL BOLTED STRUCTURAL STEEL CONNECTIONS SHALL BE 3/4" DIAMETER HIGH STRENGTH BOLTS IN ACCORDANCE WITH ASTM A325 UNLESS OTHERWISE NOTED.
3. ALL CONNECTION BOLTS ARE TO HAVE TENSION INDICATORS AND ARE TO BE TIGHTED ACCORDINGLY.
4. ALL SHOP AND FIELD WELDING SHALL BE PERFORMED BY A QUALIFIED WELDER IN ACCORDANCE WITH A.W.S. D1.1 SPECIFICATIONS, LATEST EDITION. USE E70XX ELECTRODE WELDS WITH SIZE AND LENGTH AS SHOWN ON DRAWINGS.
5. ALL STRUCTURAL STEEL MEMBERS, BOLTS, WASHERS, GUARDRAILS, BRACKETS, ANGLES, & DIAPHRAMS SHALL BE "CORTEN" STEEL WITH NO SHOP OR FIELD PAINTING REQUIRED.

GENERAL NOTES – BRIDGE

SPECIFICATIONS:

REFERENCES TO THE SPECIFICATIONS ARE TO THE CURRENT EDITION OF THE KENTUCKY DEPARTMENT OF HIGHWAYS SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION INCLUDING ANY CURRENT SUPPLEMENTAL SPECIFICATIONS. ALL REFERENCES TO THE AASHTO SPECIFICATIONS ARE TO THE CURRENT EDITION OF THE AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, WITH INTERIMS.

DESIGN LOAD AND METHOD:

THE BRIDGE IS DESIGNED AS A ONE LANE BRIDGE SUPPORTING EITHER A 115 TON M1070/M1000 HETS OR A 105 TON M1070/M747 HETS, WHICHEVER PRODUCES THE GREATER STRESS. SEE THE LOADING DIAGRAM BELOW FOR PLACEMENT OF THE VEHICLES TO PRODUCE MAXIMUM MOMENTS. ALL REINFORCED CONCRETE MEMBERS ARE DESIGNED BY THE LOAD FACTOR METHOD AS SPECIFIED IN THE CURRENT AASHTO SPECIFICATIONS.

WIND LOAD:

THIS BRIDGE IS DESIGNED FOR A WIND LOAD BASED ON A WIND VELOCITY OF 100 MPH. THE BRIDGE IS DESIGNED FOR A 60 PSF WEARING SURFACE LOAD.

CONCRETE:

CLASS AA CONCRETE (4000 PSI) IS TO BE USED THROUGHOUT FOR THIS PROJECT. ABUTMENT CAPS AND BACKWALLS, PIER CAP, AND THE SUPERSTRUCTURE SHALL ALL BE CLASS AA CONCRETE.

REINFORCEMENT:

DIMENSIONS SHOWN FROM THE FACE OF CONCRETE TO BARS ARE TO FACE OF BARS UNLESS OTHERWISE SHOWN. SUPERSTRUCTURE REINFORCING FOR SPAN 1 SHALL BE A STRONGWELL GRIDFORM SYSTEM USING TWO ROWS OF PLASTIC MEMBERS. THIS DECK SHALL BE NON-COMPOSITE CONSTRUCTION SUPPORTED ON STEEL WIDE FLANGE BEAMS.

BEVELED EDGES:

BEVEL ALL EXPOSED EDGES 7/8 ", UNLESS OTHERWISE NOTED.

SHOP DRAWINGS:

SUBMIT SHOP DRAWINGS THAT ARE REQUIRED BY THE PLANS AND SPECIFICATIONS DIRECTLY TO THE CONSULTANT. IF CHANGES IN THE DESIGN PLANS ARE PROPOSED BY A FABRICATOR OR SUPPLIER, SUBMIT THOSE CHANGES TO THE CONSULTANT. SUBMIT ALL FINAL, APPROVED SHOP DRAWINGS TO THE DIVISION OF STRUCTURAL DESIGN.

DIMENSIONS:

DIMENSIONS ARE FOR A NORMAL TEMPERATURE OF 60 DEGREES FAHRENHEIT. LAYOUT DIMENSIONS ARE HORIZONTAL DIMENSIONS.

PAYMENT FOR HYBRID BEAMS

FIVE HYBRID BEAMS SHALL BE USED IN SPAN 2 TO SUPPORT THE COMPOSITE REINFORCED CONCRETE SUPERSTRUCTURE. THE BASIS FOR PAYMENT FOR THE HYBRID BEAMS SHALL BE AT THE CONTRACT UNIT PRICE PER LINEAR FOOT OF BEAM, IN ACCORDANCE WITH THE SPECIFICATIONS.

TEMPORARY SUPPORTS:

TEMPORARY SUPPORTS OR SHORING WILL NOT BE PERMITTED UNDER THE GIRDERS WHEN POURING THE CONCRETE FLOOR SLAB OR WHEN TAKING "TOP OF BEAM" ELEVATIONS.

CONSTRUCTION IDENTIFICATION:

THE NAMES OF THE PRIME CONTRACTOR AND THE SUB-CONTRACTOR SHALL BE IMPRINTED IN THE CONCRETE WITH ONE INCH LETTERS AT A LOCATION DESIGNATED BY THE ENGINEER. THE CONTRACTOR SHALL FURNISH ALL PLANS, EQUIPMENT AND LABOR NECESSARY TO DO THE WORK FOR WHICH NO DIRECT PAYMENT WILL BE MADE.

SUPERSTRUCTURE SLAB

THE SUPERSTRUCTURE SLAB IS TO BE POURED CONTINUOUSLY OUT TO OUT, BEFORE ALLOWING ANY CONCRETE TO SET.

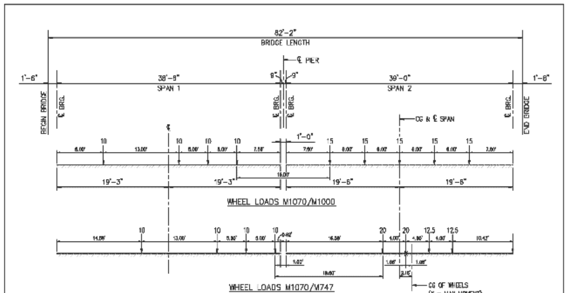
COMPLETION OF THE STRUCTURE

THE CONTRACTOR IS REQUIRED TO COMPLETE THE STRUCTURE IN ACCORDANCE WITH THE PLANS AND SPECIFICATIONS. MATERIAL, LABOR, OR CONSTRUCTION OPERATIONS, NOT OTHERWISE SPECIFIED, ARE TO BE INCLUDED IN THE BID ITEM MOST APPROPRIATE TO THE WORK INVOLVED. THIS MAY INCLUDE COFFERDAMS, SHORING, EXCAVATIONS, BACKFILLING, REMOVAL OF ALL OR PARTS OF EXISTING STRUCTURES, PHASE CONSTRUCTION, INCIDENTAL MATERIALS, LABOR, OR ANYTHING ELSE REQUIRED TO COMPLETE THE STRUCTURE.

SHEET INDEX	
Sheet No.	Description
S0.1	Cover Sheet & Notes
S1.0	Span & Deck Design
S2.0	Abutment 1 Layout & Reinforcement
S3.0	Abutment 2 Layout & Reinforcement
S4.0	Pier Layout & Reinforcement
S5.0	Framing Plan & Details
S6.0	Slab Reinforcement & Elevation
S7.0	Gerneral Details

ABBREVIATIONS – LEGEND

BM .... BENCH MARK
BKWL .... BACKWALL
BRG .... BEARING
BT/ .... BOTTOM
℄ .... CENTER
C/C .... CENTER TO CENTER
CLR .... CLEAR
CONC .... CONCRETE
CJ .... CONSTRUCTION OR CONTROL JOINT
DIA .... DIAMETER
DIM .... DIMENSION
EA .... EACH
EL .... ELEVATION
ELEV .... ELEVATION
EJ .... EXPANSION JOINT
EX .... EXISTING
EW .... EACH WAY
EF .... EACH FACE
GALV .... GALV
HORZ .... HORIZONTAL
ID .... INSIDE DIAMETER
JT .... JOINT
LEN .... LENGTH
MAX .... MAXIMUM
O/O .... OUT TO OUT
MIN .... MINIMUM
MISC .... MISCELLANEOUS
NTS .... NOT TO SCALE
OAL .... OVER ALL LENGTH
OC .... ON CENTER
OD .... OUTSIDE DIAMETER
PERF .... PERFORATED
℘ .... PLATE
R .... RADIUS
RCP .... REINFORCED CONCRETE PIPE
REINF .... REINFORCED
STA .... STATION
T/ .... TOP OF
TYP .... TYPICAL
W/ .... WITH



LOADING LAYOUT
N.T.S.

REVISIONS
NO. | BY | DESCRIPTION | DATE | CHK
1 | -- | -- | -- | --

BTM Engineering, Inc.
Consulting Engineers, Landscape Architects, Planners & Surveyors
3001 Taylor Springs Drive Louisville, Kentucky 40220
(502) 459-8402 (502) 459-8427 Fax
www.btmeng.com

DATE
SIGNATURE
DATE
SIGNATURE

TITLE: BRIDGE NUMBER 4 FORT KNOX
BRIDGE STRUCTURAL DESIGN
COVER SHEET & GENERAL NOTES
BTM PROJECT NO.: 120246
SITE INFORMATION:
OWNER/DEVELOPER:
TITLE TITLE
ADDRESS 1
ADDRESS 2
DRAWN BY: DSR
CHECKED BY: RTT
DATE: 07-2-12
DRAWING: S-NOTES.dwg
SCALE: AS NOTED
SHEET S0.1

Figure A2. Engineer drawing details for Span 1 and Span 2 of Bridge No. 4 at Fort Knox.

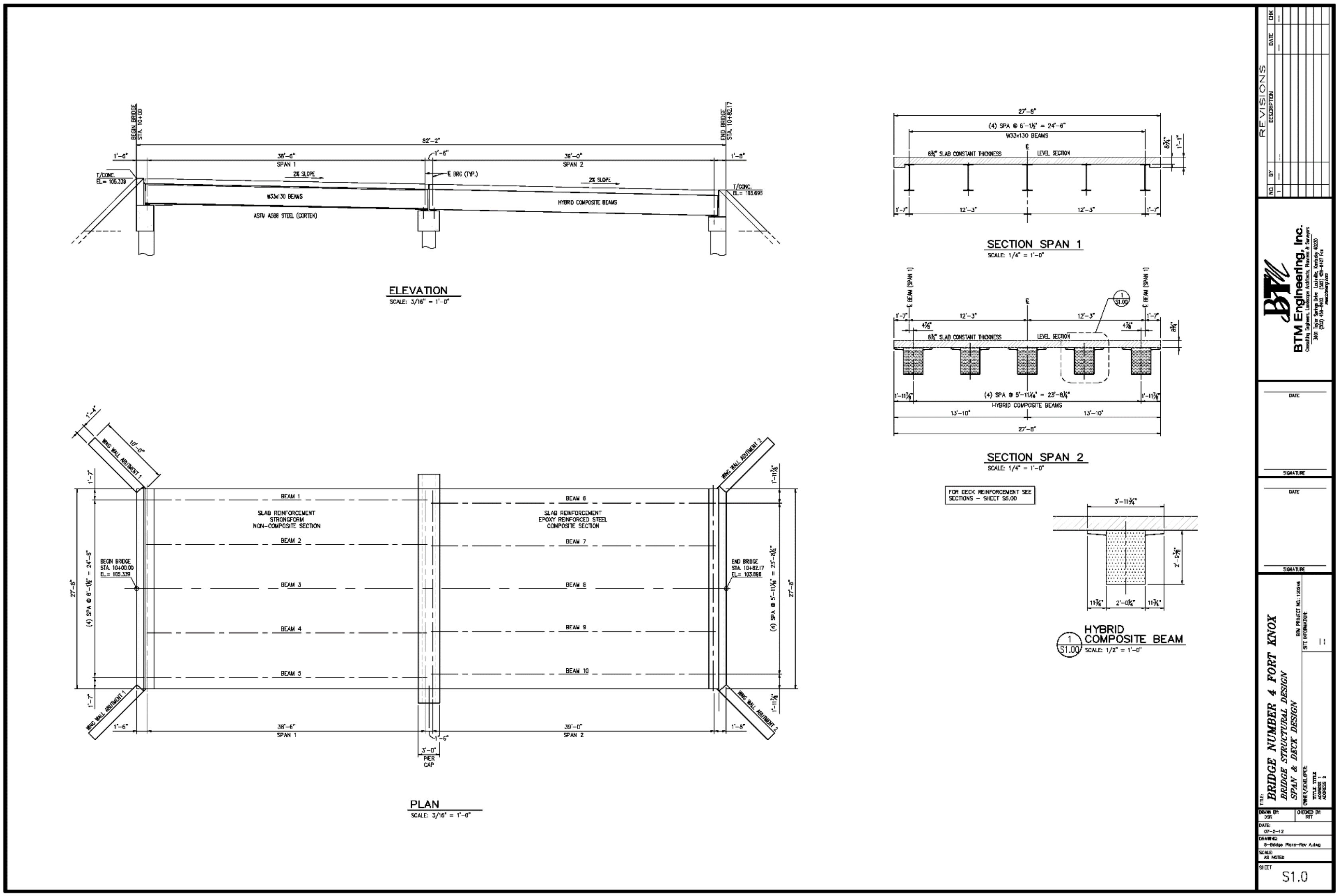
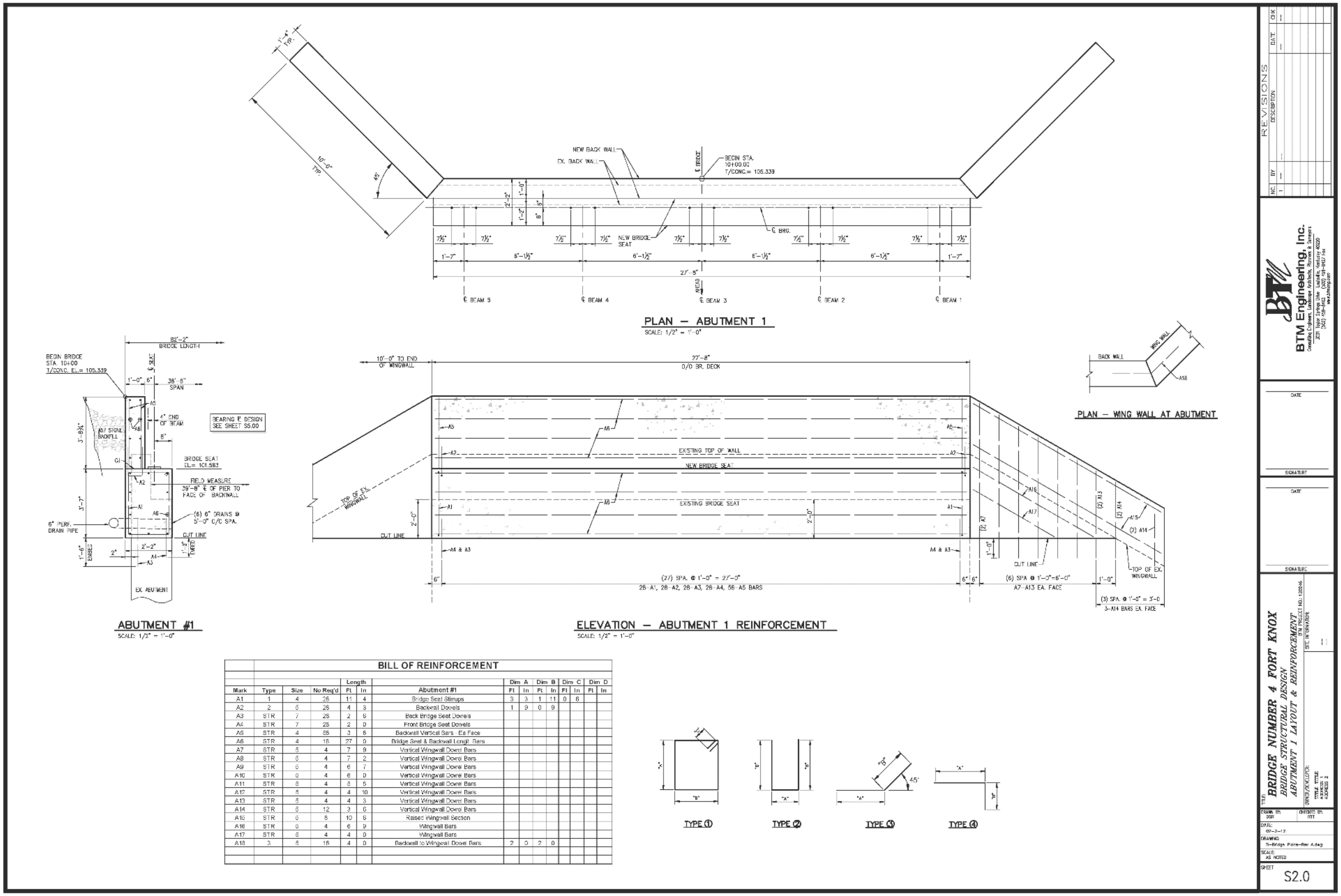


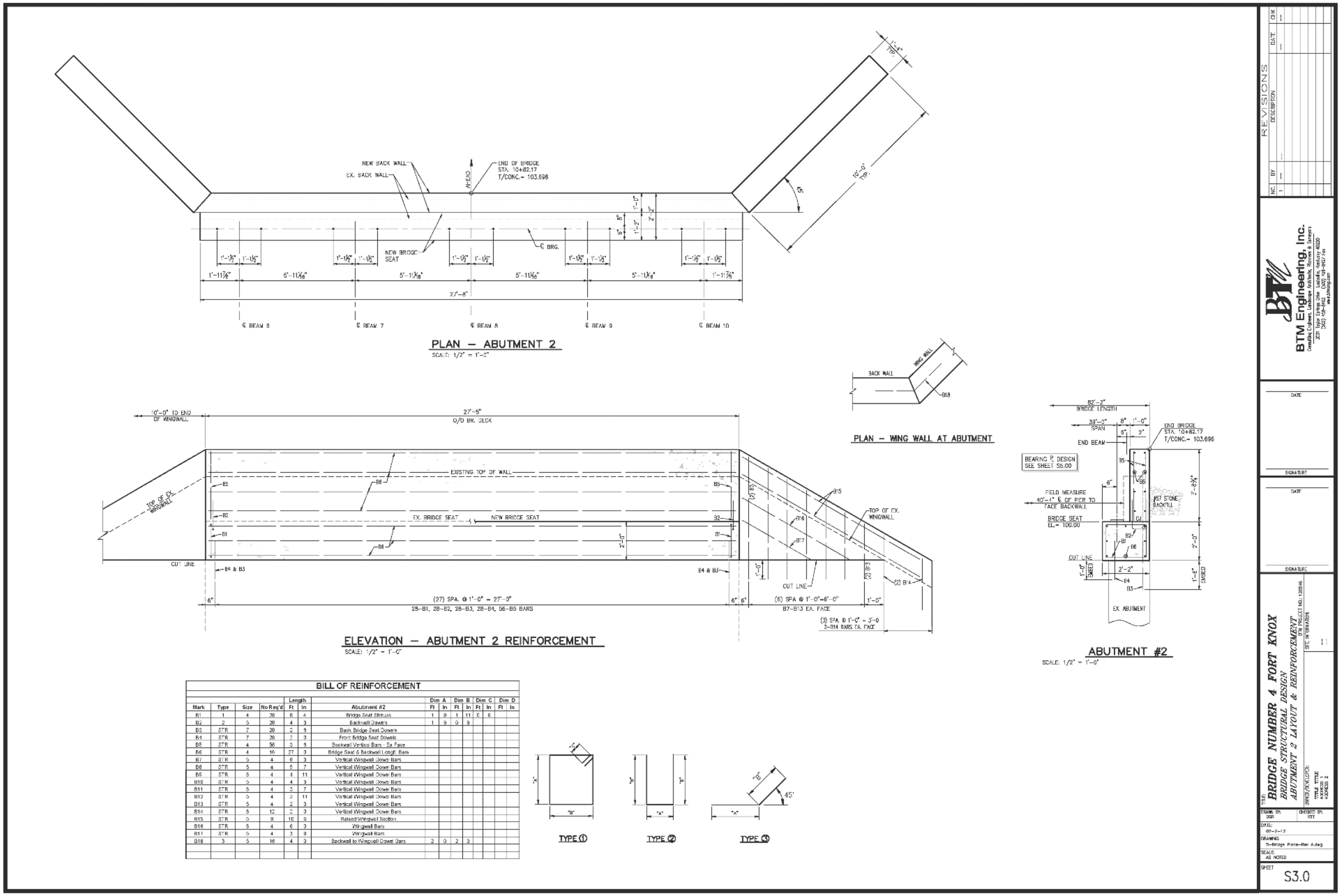
Figure A3. Engineer drawing for Abutment 1 on Bridge No. 4, Fort Knox.



BILL OF REINFORCEMENT

				Length			Dim A		Dim B		Dim C		Dim D	
Mark	Type	Size	No Req'd	Ft	In	Abutment #1	Ft	In	Ft	In	Ft	In	Ft	In
A1	1	4	28	11	4	Bridge Seat Stirrups	3	3	1	11	0	6		
A2	2	5	28	4	3	Backwall Dowels	1	9	0	9				
A3	STR	7	28	2	6	Back Bridge Seat Dowels								
A4	STR	7	28	2	0	Front Bridge Seat Dowels								
A5	STR	4	56	3	6	Backwall Vertical Bars - Ea Face								
A6	STR	4	16	27	0	Bridge Seat & Backwall Longit. Bars								
A7	STR	5	4	7	9	Vertical Wingwall Dowel Bars								
A8	STR	5	4	7	2	Vertical Wingwall Dowel Bars								
A9	STR	5	4	6	7	Vertical Wingwall Dowel Bars								
A10	STR	5	4	6	0	Vertical Wingwall Dowel Bars								
A11	STR	5	4	5	5	Vertical Wingwall Dowel Bars								
A12	STR	5	4	4	10	Vertical Wingwall Dowel Bars								
A13	STR	5	4	4	3	Vertical Wingwall Dowel Bars								
A14	STR	5	12	3	6	Vertical Wingwall Dowel Bars								
A15	STR	5	8	10	6	Raised Wingwall Section								
A16	STR	5	4	6	9	Wingwall Bars								
A17	STR	5	4	4	0	Wingwall Bars								
A18	3	5	16	4	0	Backwall to Wingwall Dowel Bars	2	0	2	0				

Figure A4. Engineer drawing for Abutment 2 on Bridge No. 4, Fort Knox.



BILL OF REINFORCEMENT

Mark	Type	Size	No Req'd	Length Ft	Length In	Abutment #2	Dim A Ft	Dim A In	Dim B Ft	Dim B In	Dim C Ft	Dim C In	Dim D Ft	Dim D In
B1	1	4	28	8	4	Bridge Seat Stirrups	1	9	1	11	0	6		
B2	2	5	28	4	3	Backwall Dowels	1	9	0	9				
B3	STR	7	28	2	9	Back Bridge Seat Dowels								
B4	STR	7	28	2	0	Front Bridge Seat Dowels								
B5	STR	4	56	3	9	Backwall Vertical Bars - Ea Face								
B6	STR	4	16	27	0	Bridge Seat & Backwall Longit. Bars								
B7	STR	5	4	6	3	Vertical Wingwall Dowel Bars								
B8	STR	5	4	5	7	Vertical Wingwall Dowel Bars								
B9	STR	5	4	4	11	Vertical Wingwall Dowel Bars								
B10	STR	5	4	4	3	Vertical Wingwall Dowel Bars								
B11	STR	5	4	3	7	Vertical Wingwall Dowel Bars								
B12	STR	5	4	2	11	Vertical Wingwall Dowel Bars								
B13	STR	5	4	2	3	Vertical Wingwall Dowel Bars								
B14	STR	5	12	2	0	Vertical Wingwall Dowel Bars								
B15	STR	5	8	10	6	Raised Wingwall Section								
B16	STR	5	4	6	0	Wingwall Bars								
B17	STR	5	4	3	9	Wingwall Bars								
B18	3	5	16	4	0	Backwall to Wingwall Dowel Bars	2	0	2	0				

Figure A5. Engineer drawing for bridge pier on Bridge No. 4, Fort Knox.

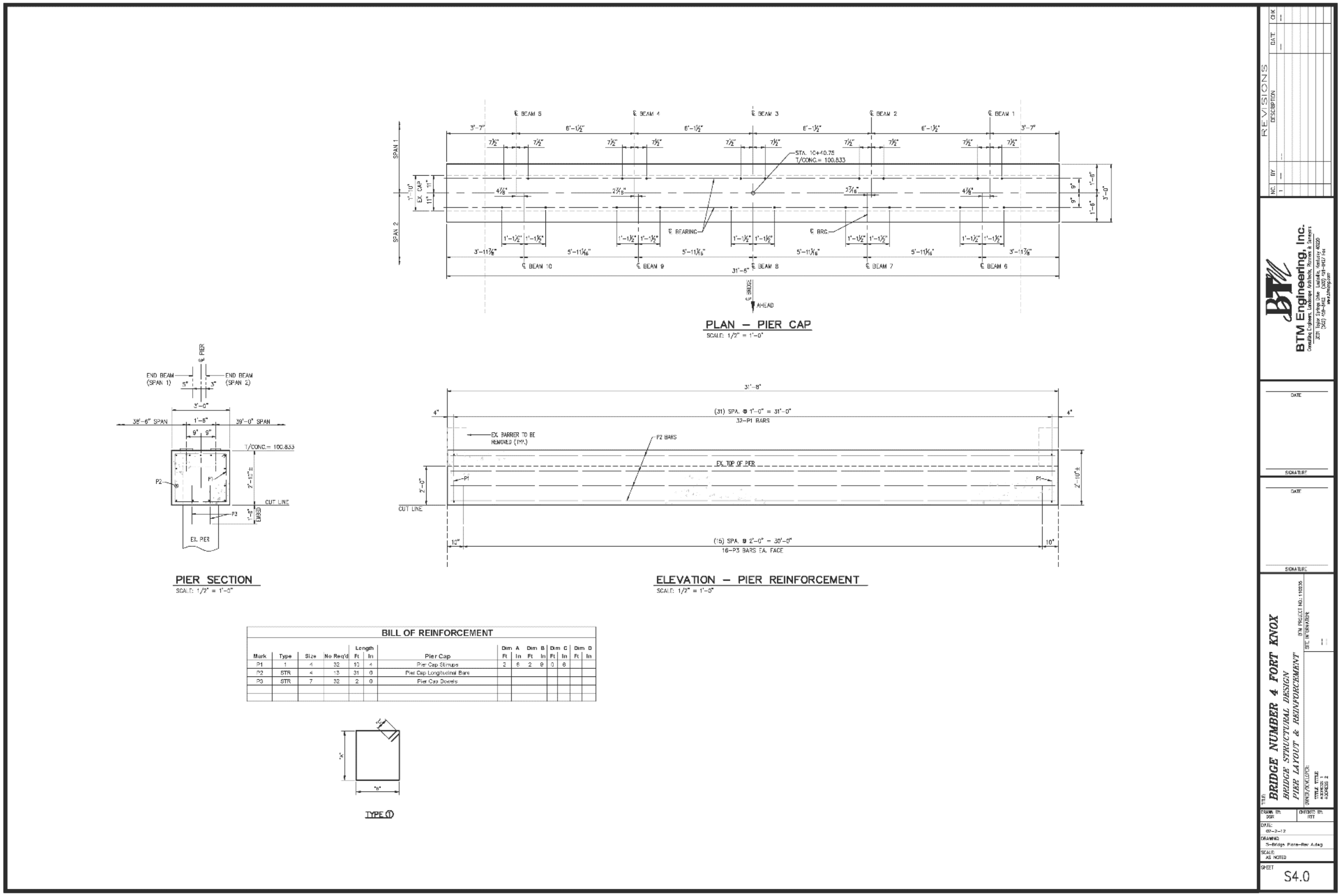


Figure A6. Engineer drawings for various design components on both spans of Bridge No. 4 at Fort Knox.

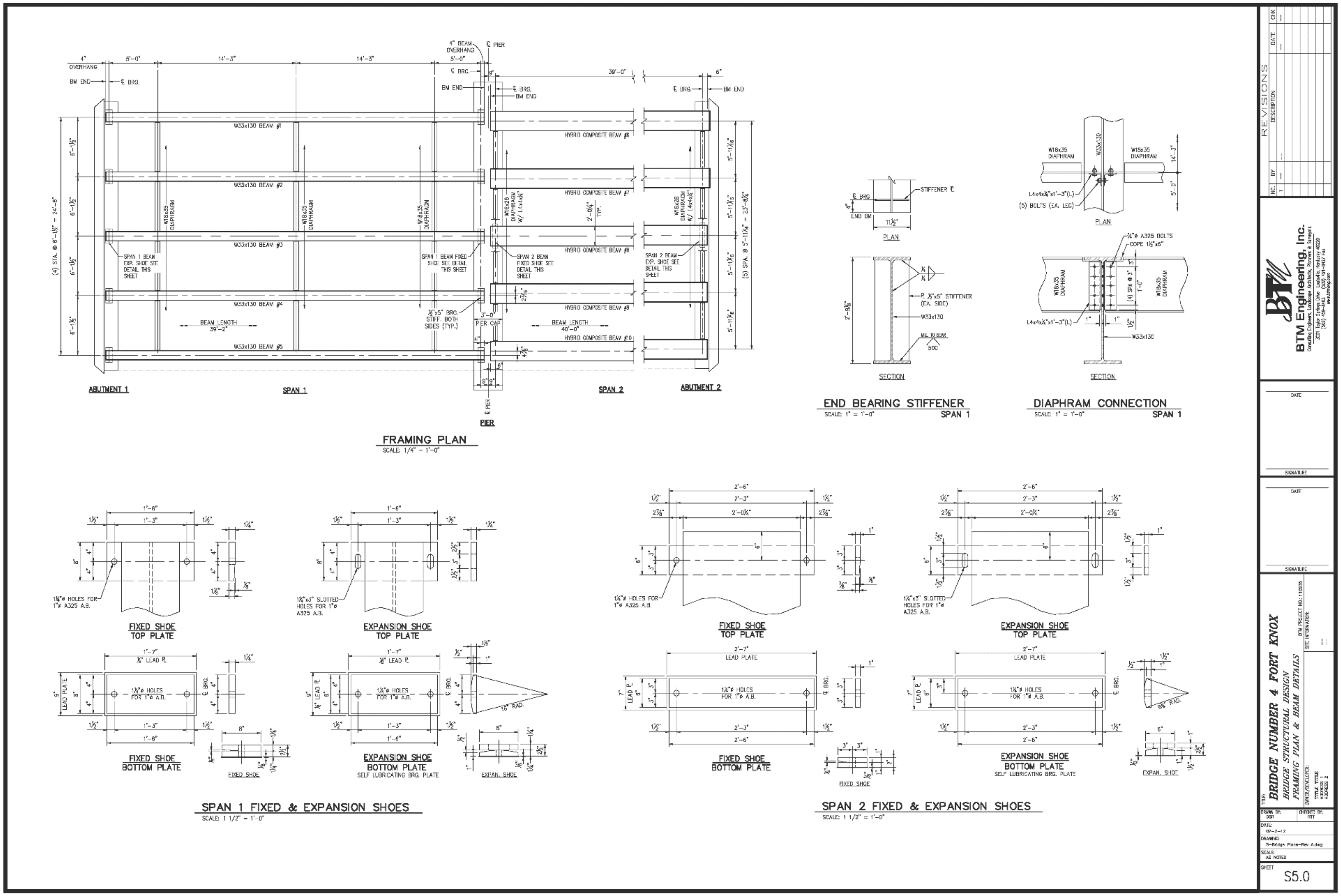


Figure A7. Engineer drawing for deck plans for both spans of Bridge No. 4 at Fort Knox.

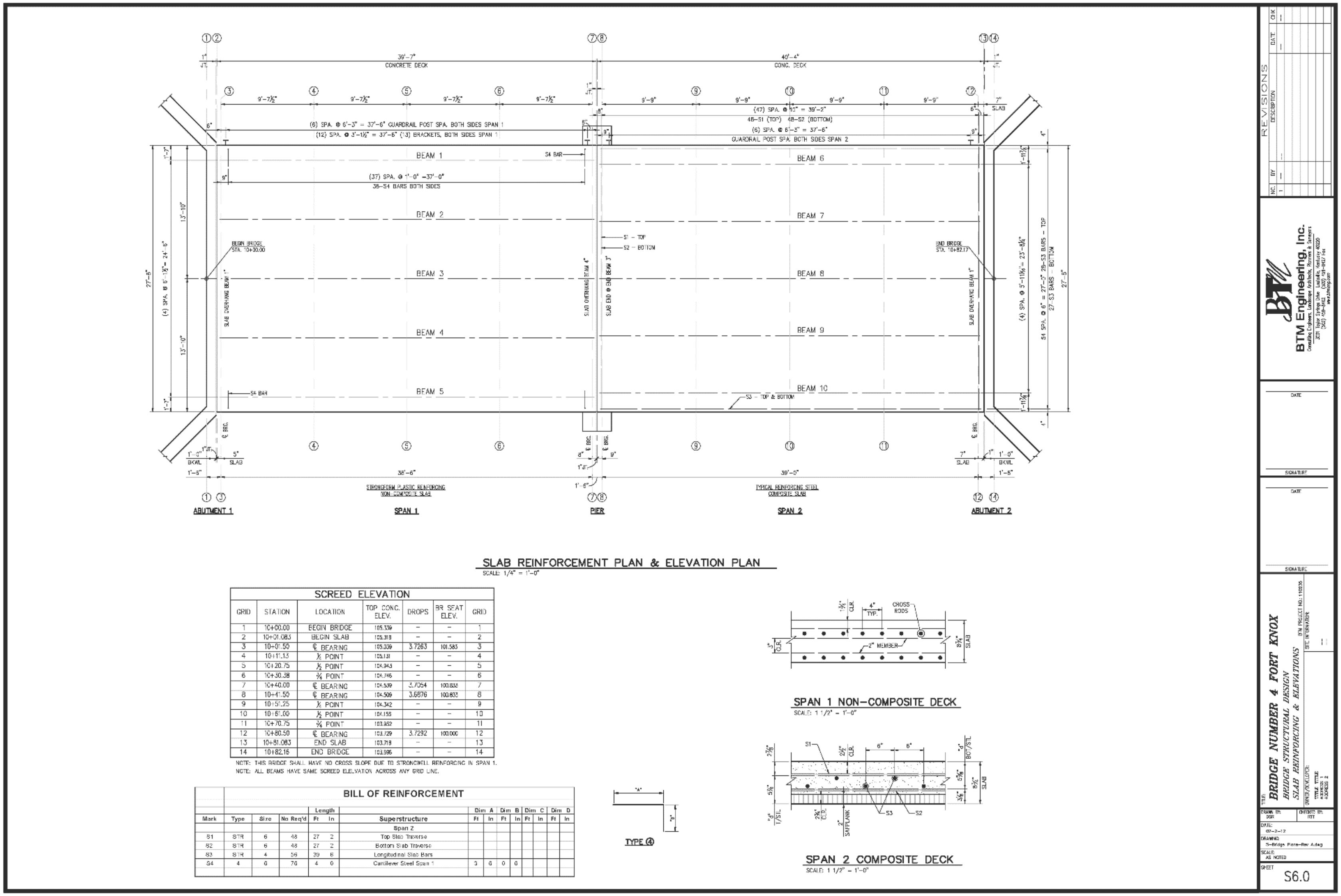
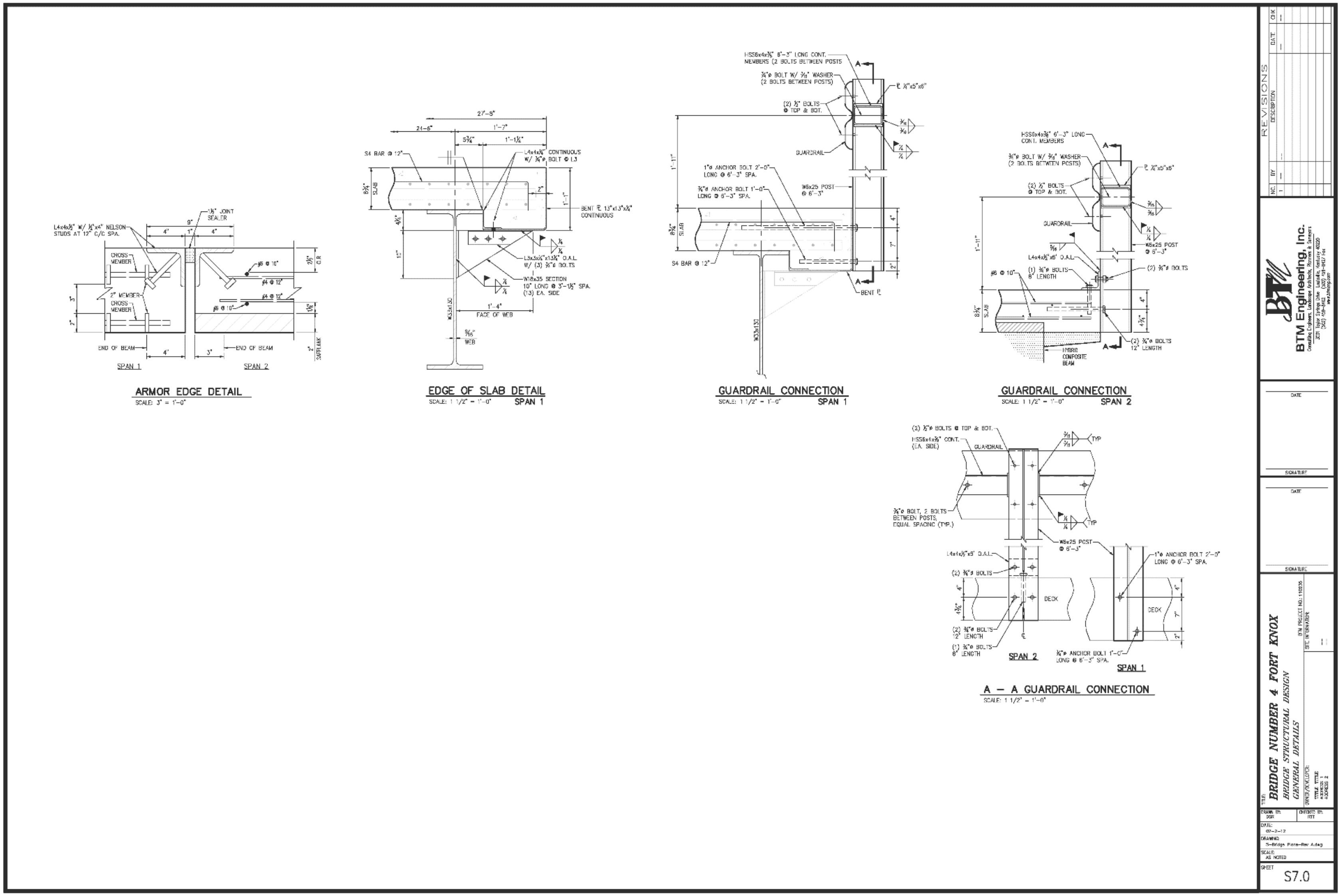


Figure A8. Engineer drawing for edge details and guardrail connections for both spans of Bridge No. 4 at Fort Knox.



Appendix B: Corrosion Potential Assessment for Bridge No. 4, Fort Knox, Kentucky

Classification method

A corrosion severity classification for the Bridge No. 4 site at Fort Knox was developed for use in evaluating the materials used in this project. This was accomplished at the site through placement of a portable weather station (collecting weather data for one year), an atmospheric corrosion test rack, (equipped with sensors to monitor corrosion and chlorides were inserted in the bridge deck), and quarterly site visits (performed visual inspections).

Monitoring

Weather station

A weather station was installed to measure and record environmental characteristics throughout the exposure period as shown in Figure B1. The station measured temperature, relative humidity, wind speed and direction, and rainfall. The weather station was powered by a solar panel and a rechargeable battery. A data logger was used to store the measurements which were recorded every 12 hours by the rain gage and every 15 minutes for the remaining sensors. Data was downloaded manually during each quarterly inspection through the use of a laptop computer. The data logger has a storage capacity to continue storing data at 15-minute intervals for approximately 2.5 years. Upon reaching full capacity, the data logger will truncate the oldest data point to create room for new, incoming data.

Figure B1. Weather station.

Sensors

Sensors were installed in the bridge deck to measure chloride penetration, corrosion potential, and corrosion rate (Figure B2). Measurements from the sensors were taken quarterly for a 12-month period.

The Rohback Cosasco 900 Concrete Multi-Depth Sensor was utilized to accomplish the chloride measurements. Four sets of electrodes are spaced by 1 in. intervals to provide four separate measurements at different depths from each sensor. The 900 sensors were mounted such that the first electrode was 1 in. from the surface of the concrete. Two sensors were positioned in the span with the RFP reinforcement, and three sensors were positioned in the control span adjacent to it. The Rohback Cosasco Aquamate was used to collect the corrosion rate measurements.

The Borin Stelth 7 sensor was used to measure corrosion potential in the bridge deck. The Stelth 7 sensor is an IR-Free probe with a silver-silver chloride (Ag-AgCl) electrode. Corrosion potential sensors measure a voltage difference between the sensor electrode and reinforcement rebar; therefore six Stelth 7 sensors were installed only throughout the control span of the bridge. Two ground wires were installed for redundancy. Measurements from each ground should theoretically be identical. An Extech 540 multimeter was used to collect the corrosion potential measurements.

Rohback Cosasco 800 LPR Corrosion Rate sensors were used to measure the instantaneous corrosion rate of reinforcing steel in concrete by the method of Linear Polarization Resistance (LPR). The electrodes of the LPR probe are manufactured using carbon steel. The LPR sensor utilizes the solution resistance compensation (SRC) method which makes a separate measurement and correction for the effect of the resistivity of the concrete and eliminates the need for a third electrode that is typically used in LPR sensors. Five LPR sensors were positioned throughout the control span of the bridge. The Rohback Cosasco Aquamate was used to collect the imbalance (Imb) measurements.

Sensor types and locations are in BDI's full report, contained in ERDC/CERL CR-16-4 (Commander and Carpenter 2016).

Coupons to simulate chloride penetration

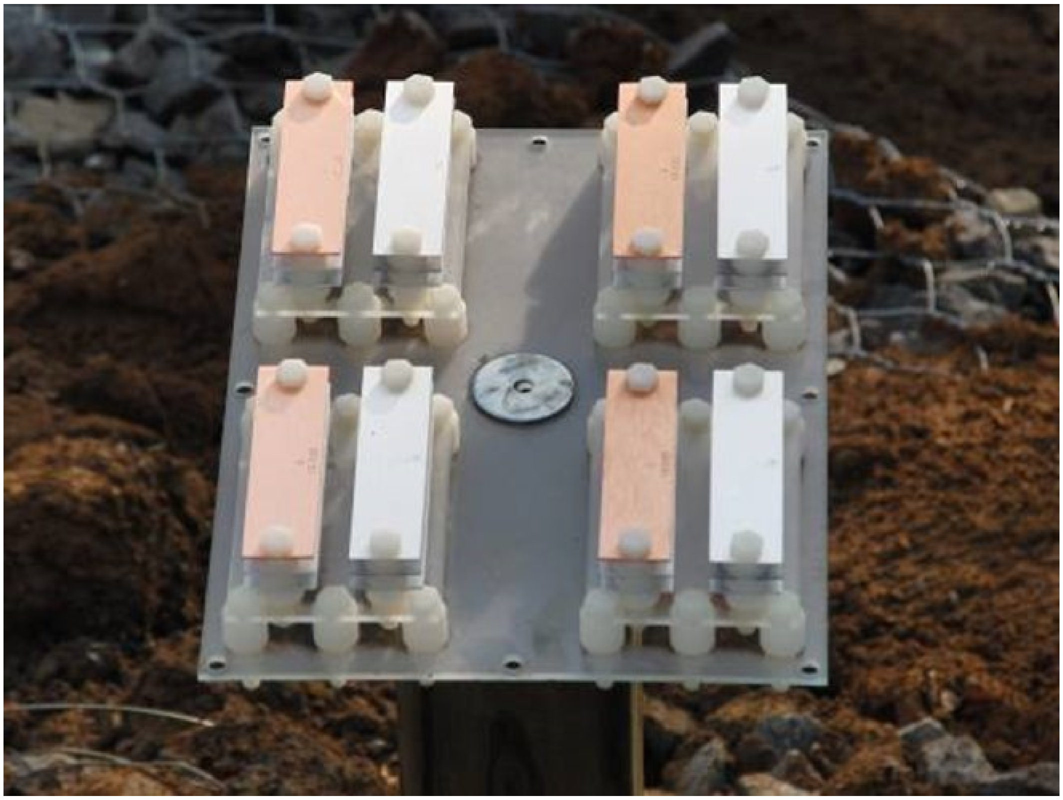
A concrete coupon was formed in a 5-gallon bucket to provide a method to simulate chloride penetration. The bucket was filled approximately halfway with a concrete mix including one cup of sodium chloride. A corrosion ladder was situated in the form such that the chloride enriched concrete covered the first two set of electrodes of the chloride sensor. A corrater was also submerged in the concrete. The concrete was provided 24 hours to cure before filling the rest of the form with standard concrete. Figure B2 shows the chloride-enriched concrete covering the corrater and half of the chloride ladder sensor. Figure B3 shows the cured concrete coupon. Measurements were collected during the quarterly inspections.

Figure B2. Coupon preparation.

Figure B3. Finished coupon.

An atmospheric coupon rack to determine the relative corrosivity of the site was installed facing 90 degrees from vertical at the bridge site (Figure B4). The corrosion coupons included silver, copper, 1010 steel, and three aluminum alloys (2024 T3, 6061 T6, and 7075 T6).The coupons measured 1 in. wide by 4 in. long by 1/16 in. thick. These coupons were collected after 6 months and 12 months of exposure. The mass of each coupon was recorded before being exposed to the test environment. The silver coupon was tested for chlorides in accordance with ASTM B825. The remaining coupons were analyzed for mass loss in accordance with ASTM G1-03.

Figure B4. Atmospheric corrosion test rack.



Assessments for weather, sensors, and corrosion coupon rack

Weather assessment

The weather data was analyzed using response functions from the ISO 9223:2012 Corrosion of Metal and Alloys – Corrosivity of Atmospheres – Classification, Determination and Estimation. SO_2 measurements were not collected; however due to the location of Bridge No. 4, it was assumed that deposition of SO_2 would be equal to zero milligrams per square meter, per day. The amount of Cl deposition was calculated from the ASTM B825, "Standard Test Method for Coulometric Reduction of Surface Films on Metallic Test" for the silver mass-loss coupon. The equations used are shown in Figure B6. Corrosion classifications per ISO 9223:2012 are shown in Table B1.

Figure B6. ISO 9223:2012 response equations for four standard metals.

Equation (1) for carbon steel:

$$r_{\text{corr}} = 1{,}77 \cdot P_{\text{d}}^{0{,}52} \cdot \exp(0{,}020 \cdot \text{RH} + f_{\text{St}}) + 0{,}102 \cdot S_{\text{d}}^{0{,}62} \cdot \exp(0{,}033 \cdot \text{RH} + 0{,}040 \cdot T) \qquad (1)$$

$f_{\text{St}} = 0{,}150 \cdot (T - 10)$ when $T \leq 10$ °C; otherwise $-0{,}054 \cdot (T - 10)$

$N = 128$, $R^2 = 0{,}85$

Equation (2) for zinc:

$$r_{\text{corr}} = 0{,}0129 \cdot P_{\text{d}}^{0{,}44} \cdot \exp(0{,}046 \cdot \text{RH} + f_{\text{Zn}}) + 0{,}0175 \cdot S_{\text{d}}^{0{,}57} \cdot \exp(0{,}008 \cdot \text{RH} + 0{,}085 \cdot T) \qquad (2)$$

$f_{\text{Zn}} = 0{,}038 \cdot (T - 10)$ when $T \leq 10$ °C; otherwise, $-0{,}071 \cdot (T - 10)$

$N = 114$, $R^2 = 0{,}78$

Equation (3) for copper:

$$r_{\text{corr}} = 0{,}005\,3 \cdot P_{\text{d}}^{0{,}26} \cdot \exp(0{,}059 \cdot \text{RH} + f_{\text{Cu}}) + 0{,}010\,25 \cdot S_{\text{d}}^{0{,}27} \cdot \exp(0{,}036 \cdot \text{RH} + 0{,}049 \cdot T) \qquad (3)$$

$f_{\text{Cu}} = 0{,}126 \cdot (T - 10)$ when $T \leq 10$ °C; otherwise, $-0{,}080 \cdot (T - 10)$

$N = 121$, $R^2 = 0{,}88$

Equation (4) for aluminium:

$$r_{\text{corr}} = 0{,}004\,2 \cdot P_{\text{d}}^{0{,}73} \cdot \exp(0{,}025 \cdot \text{RH} + f_{\text{Al}}) + 0{,}0018 \cdot S_{\text{d}}^{0{,}60} \cdot \exp(0{,}020 \cdot \text{RH} + 0{,}094 \cdot T) \qquad (4)$$

$f_{\text{Al}} = 0{,}009 \cdot (T - 10)$ when $T \leq 10$ °C; otherwise $-0{,}043 \cdot (T - 10)$

$N = 113$, $R^2 = 0{,}65$

where

r_{corr}	is first-year corrosion rate of metal, expressed in micrometres per year (µm/a);
T	is the annual average temperature, expressed in degrees Celsius (°C);
RH	is the annual average relative humidity, expressed as a percentage (%);
P_{d}	is the annual average SO_2 deposition, expressed in milligrams per square metre per day [mg/(m^2·d)];
S_{d}	is the annual average Cl^- deposition, expressed in milligrams per square metre per day [mg/(m^2·d)].

Table B1. Corrosion rates, r_{corr}, for the first year of exposure for the different corrosivity categories.

Corrosivity category	Corrosion rates of metals r_{corr}				
	Unit	Carbon steel	Zinc	Copper	Aluminium
C1	g/(m²·a)	$r_{corr} \leq 10$	$r_{corr} \leq 0,7$	$r_{corr} \leq 0,9$	negligible
	µm/a	$r_{corr} \leq 1,3$	$r_{corr} \leq 0,1$	$r_{corr} \leq 0,1$	—
C2	g/(m²·a)	$10 < r_{corr} \leq 200$	$0,7 < r_{corr} \leq 5$	$0,9 < r_{corr} \leq 5$	$r_{corr} \leq 0,6$
	µm/a	$1,3 < r_{corr} \leq 25$	$0,1 < r_{corr} \leq 0,7$	$0,1 < r_{corr} \leq 0,6$	—
C3	g/(m²·a)	$200 < r_{corr} \leq 400$	$5 < r_{corr} \leq 15$	$5 < r_{corr} \leq 12$	$0,6 < r_{corr} \leq 2$
	µm/a	$25 < r_{corr} \leq 50$	$0,7 < r_{corr} \leq 2,1$	$0,6 < r_{corr} \leq 1,3$	—
C4	g(m²·a)	$400 < r_{corr} \leq 650$	$15 < r_{corr} \leq 30$	$12 < r_{corr} \leq 25$	$2 < r_{corr} \leq 5$
	µm/a	$50 < r_{corr} \leq 80$	$2,1 < r_{corr} \leq 4,2$	$1,3 < r_{corr} \leq 2,8$	—
C5	g/(m²·a)	$650 < r_{corr} \leq 1\,500$	$30 < r_{corr} \leq 60$	$25 < r_{corr} \leq 50$	$5 < r_{corr} \leq 10$
	µm/a	$80 < r_{corr} \leq 200$	$4,2 < r_{corr} \leq 8,4$	$2,8 < r_{corr} \leq 5,6$	—
CX	g/(m²·a)	$1\,500 < r_{corr} \leq 5\,500$	$60 < r_{corr} \leq 180$	$50 < r_{corr} \leq 90$	$r_{corr} > 10$
	µm/a	$200 < r_{corr} \leq 700$	$8,4 < r_{corr} \leq 25$	$5,6 < r_{corr} \leq 10$	—

NOTE 1 The classification criterion is based on the methods of determination of corrosion rates of standard specimens for the evaluation of corrosivity (see ISO 9226).

NOTE 2 The corrosion rates, expressed in grams per square metre per year [g/(m²·a)], are recalculated in micrometres per year (µm/a) and rounded.

NOTE 3 The standard metallic materials are characterized in ISO 9226.

NOTE 4 Aluminium experiences uniform and localized corrosion. The corrosion rates shown in this table are calculated as uniform corrosion. Maximum pit depth or number of pits can be a better indicator of potential damage. It depends on the final application. Uniform corrosion and localized corrosion cannot be evaluated after the first year of exposure due to passivation effects and decreasing corrosion rates.

NOTE 5 Corrosion rates exceeding the upper limits in category C5 are considered extreme. Corrosivity category CX refers to specific marine and marine/industrial environments (see Annex C).

Table B2. Description of typical atmospheric environments related to the estimation of corrosivity categories.

Corrosivity category[a]	Corrosivity	Typical environments — Examples[b]	
		Indoor	Outdoor
C1	Very low	Heated spaces with low relative humidity and insignificant pollution, e.g. offices, schools, museums	Dry or cold zone, atmospheric environment with very low pollution and time of wetness, e.g. certain deserts, Central Arctic/Antarctica
C2	Low	Unheated spaces with varying temperature and relative humidity. Low frequency of condensation and low pollution, e.g. storage, sport halls	Temperate zone, atmospheric environment with low pollution (SO_2 < 5 µg/m^3), e.g. rural areas, small towns Dry or cold zone, atmospheric environment with short time of wetness, e.g. deserts, subarctic areas
C3	Medium	Spaces with moderate frequency of condensation and moderate pollution from production process, e.g. food-processing plants, laundries, breweries, dairies	Temperate zone, atmospheric environment with medium pollution (SO_2: 5 µg/m^3 to 30 µg/m^3) or some effect of chlorides, e.g. urban areas, coastal areas with low deposition of chlorides Subtropical and tropical zone, atmosphere with low pollution
C4	High	Spaces with high frequency of condensation and high pollution from production process, e.g. industrial processing plants, swimming pools	Temperate zone, atmospheric environment with high pollution (SO_2: 30 µg/m^3 to 90 µg/m^3) or substantial effect of chlorides, e.g. polluted urban areas, industrial areas, coastal areas without spray of salt water or, exposure to strong effect of de-icing salts Subtropical and tropical zone, atmosphere with medium pollution
C5	Very high	Spaces with very high frequency of condensation and/or with high pollution from production process, e.g. mines, caverns for industrial purposes, unventilated sheds in subtropical and tropical zones	Temperate and subtropical zone, atmospheric environment with very high pollution (SO_2: 90 µg/m^3 to 250 µg/m^3) and/or significant effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline

Section 3.1.1 of this report contains a summary of selected weather data collected from December 2012–December 2013 (Table 1), and the results from the response equation calculations (Table 4).

Sensor corrosion assessment

Data from sensors installed on the bridge were collected after 1, 4, 7, 10, and 13 months (Tables B3–B7). The zero, Corr, and Imb values at the bottom of each table represent instrument calibration check readings for a dummy probe provided by the CORRATER instrument manufacturer. The check values (5±1 mpy [mils per year; 1 mpy = 0.001 in. per year] for corrosion rate and 0±1 for imbalance) indicated that the instrument was functioning properly.

The CORRATER LPR probes at locations 1, 2, 3, and 5 all indicated very general and low general corrosion rates, ranging from 0–0.03 mpy. The imbalance readings (qualitatively indicative of pitting tendency) ranged from 0–0.02. Both of these sets of data indicate very low corrosion activity over the 13-month test period. This is not surprising because the corrosion rate of steel in highly alkaline, uncontaminated concrete (pH ~ 13) is negligible due to the formation of a complex passive film (mixture of α and γ iron-oxide and magnetite). With sufficient concrete cover over the steel rebar and less severe corrosive environments, it can take more than a decade for corrosion rates to increase appreciably. The CORRATER probe at location number 4 indicated erratic corrosion rates; for example, ranging from "off scale" at 1 month, increasing to 13.8 mpy at 4 months, accelerating to 48.9 mpy at 7 months, then decreasing dramatically to 0.49 mpy at 10 months, and finally off scale again at 13 months. The imbalance readings were 0.39, 0.65, 0.48, 0.36, and 0.91, respectively. The imbalance readings were all lower than the corresponding general corrosion rates; thus, qualitatively indicating low pitting tendency. The check readings all indicated that the Aquamate CORRATER instrument was functioning properly. The results for the artificially-contaminated concrete block “salt coupon” are shown graphically in Figure 4. It is apparent that some corrosion activity was indicated at 4 months with an increase in pitting tendency at 7 and 10 months and a decrease at 13 months. Although the general corrosion rate appears to be increasing steadily, the actual rates (e.g., 0.04 mpy) are negligible.

For the chloride-ladders, the corrosion rates varied from 0–0.04 mpy and the imbalance readings from 0 to 0.07. The chloride-ladder at location 4 appeared to show the most activity. Although the imbalance readings were greater than the corrosion rates, all of the values were very small, indicating low corrosion activity. Similarly, the galvanic current measurements related to chloride ingress also indicated no significant penetration. The artificially-contaminated concrete block salt coupon exhibited the most activity at 1 and 4 months but this decreased dramatically at 7, 10, and 13 months, possibly due to a drying out effect.

Table B3. Sensor data after 1 month.

Month 1 Data												
Location	1		2		3		4		5		Salt Coupon	
	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
Corrator	0.02	0	0	0	0.01	0.01	ovr	0.39	0.02	0.02	0.01	0

Location	1		4		5		FR1		FR2		Salt Coupon	
Chloride Ladders												
Corrosion	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
H	0	0	0	0.01	0	0	0	0	0	0	0	0
I	0	0	0	0.01	0	0	0	0	0	0	1.17	2.95
L	0	0	0	0.03	0	0	0	0.06	0	0	0.33	1.3
Imbalance												
H	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0.01	0	0.01	0	0	0	0.01	0	0	0.12	2.04
L	0	0	0	0	0	0	0	0	0	0	0.06	1.08

Location	1	2	3	4	5	6
Borin [mV - Ground #1]						
Orange	-242.1	-233.3	-276.1	-184.5	-291.1	-271.4
Blue	-556.6	-240.6	-102.8	-98.6	-490.3	-276.7
Black	-242.2	-233.3	-276.1	-184.5	-291.1	-271.4
Yellow	198.0	219.9	181.4	234.3	195.2	-212.3
Red	-556.7	-240.6	-102.8	-98.6	-490.3	-276.7
Borin [mV - Ground #2]						
Orange	-242.1	-233.3	-276.1	-184.5	-291.1	-271.4
Blue	-556.6	-240.6	-102.8	-98.6	-490.3	-276.7
Black	-242.2	-233.3	-276.1	-184.5	-291.1	-271.4
Yellow	198.0	219.9	181.4	234.3	195.2	-212.3
Red	-556.7	-240.6	-102.8	-98.6	-490.3	-276.7

Instrument Calibartion		
	Corr	Imbal
Zero	5.07	0.03
Zero	4.99	0.03

Table B4. Sensor data after 4 months.

Month 4 Data												
Location	1		2		3		4		5		Salt Coupon	
	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
Corrator	0.02	0	0	0	0.01	0.01	13.8	0.65	0.02	0.02	0.01	0
Location	1		4		5		FR1		FR2		Salt Coupon	
Chloride Ladders												
Corrosion	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
H	0	0	0	0.03	0	0	0	0	0.04	0.07	0.04	0.54
I	0	0	0	0.01	0	0	0	0	0	0.01	0	0.01
L	0	0	0	0	0	0	0	0	0	0.06	0	0
Imbalance												
H	0	0	0	0	0	0	0	0	0	0	0.2	0.26
I	0	0	0	0.02	0	0	0	0	0	0	0	0
I	0	0.01	0	0	0	0	0	0	0	0	0	0
L	0	0	0	0	0	0	0	0	0	0	0	0

Location	1	2	3	4	5	6
Borin [mV - Ground #1]						
Orange	-330.0	-313.0	-210.2	-138.9	-264.7	-242.0
Blue	-699.6	-346.2	-197.9	-68.7	-609.0	-107.1
Black	-33.0	-313.0	-210.2	-138.9	-264.7	-242.0
Yellow	196.1	203.6	237.9	282.1	230.6	286.7
Red	-699.6	-346.2	-197.9	-68.7	-609.0	-107.1
Borin [mV - Ground #2]						
Orange	-330.0	-313.0	-210.2	-138.9	-264.7	-242.0
Blue	-699.6	-346.2	-197.9	-68.7	-609.0	-107.1
Black	-33.0	-313.0	-210.2	-138.9	-264.7	-242.0
Yellow	196.1	203.6	237.9	282.1	230.6	286.7
Red	-699.6	-346.2	-197.9	-68.7	-609.0	-107.1

Instrument Calibartion		
	Corr	Imbal
Zero	5.07	0.03
Zero	4.99	0.02

Table B5. Sensor data after 7 months.

Month 7 Data												
Location	1		2		3		4		5		Salt Coupon	
	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
Corrator	0.03	0	0.02	0.01	0.02	0	48.9	0.48	0.03	0.02	0.02	0.07

Location	1		4		5		FR1		FR2		Salt Coupon	
Chloride Ladders												
Corrosion	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
H	0	0	0	0.06	0	0.01	0	0	0	0	0	0
I	0	0	0	0.05	0	0	0	0	0.01	0	0	0
L	0	0	0.01	0.05	0	0	0	0.01	0.03	0	0	0
Imbalance												
H	0	0	0	0.04	0	0.02	0	0	0	0	0	0
I	0	0	0	0	0	0.01	0	0	0	0	0	0
I	0	0	0	0	0	0.01	0	0	0	0	0	0
L	0	0	0	0	0	0	0	0.06	0	0	0	0

Location	1	2	3	4	5	6
Borin [mV - Ground #1]						
Orange	-141.7	-51.1	-67.7	-66.0	-431.2	-161.2
Blue	-81.7	13.4	-145.2	19.7	-528.6	-186.7
Black	-141.7	-51.1	-67.7	-66.0	-431.2	-161.2
Yellow	287.0	342.1	351.7	377.5	308.0	365.6
Red	-81.7	13.4	-145.2	19.7	-528.6	-186.7
Borin [mV - Ground #2]						
Orange	-141.7	-51.1	-67.7	-66.0	-431.2	-161.2
Blue	-81.7	13.4	-145.2	19.7	-528.6	-186.7
Black	-141.7	-51.1	-67.7	-66.0	-431.2	-161.2
Yellow	287.0	342.1	351.7	377.5	308.0	365.6
Red	-81.7	13.4	-145.2	19.7	-528.6	-186.7

Instrument Calibartion		
	Corr	Imbal
Zero	5.07	0.04
Zero	4.99	0.02

Table B6. Sensor data after 10 months.

Month 10 Data												
Location	1		2		3		4		5		Salt Coupon	
	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
Corrator	0.02	0.01	0.02	0.01	0.02	0.01	0.49	0.36	0.02	0.01	0.03	0.08

Location	1		4		5		FR1		FR2		Salt Coupon	
Chloride Ladders												
Corrosion	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
H	0	0	0	0.07	0	0	0	0	0	0	0	0
I	0	0	0	0.06	0	0	0	0	0	0	0	0.03
L	0	0	0	0.05	0	0	0	0	0	0	0	0
Imbalance												
H	0	0	0	0.04	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
L	0	0	0	0	0	0	0	0	0	0	0	0

Location	1	2	3	4	5	6
Borin [mV - Ground #1]						
Orange	-10.1	0.8	98.0	-73.1	-189.2	-43.9
Blue	-9.7	10.3	-197.0	10.4	-435.0	33.3
Black	-10.1	0.8	98.0	-73.1	-189.2	-43.9
Yellow	331.1	304.2	340.0	369.4	280.1	386.2
Red	-9.7	10.3	-197.0	10.4	-435.0	33.3
Borin [mV - Ground #2]						
Orange	-10.1	0.8	98.0	-73.1	-189.2	-43.9
Blue	-9.7	10.3	-197.0	10.4	-435.0	33.3
Black	-10.1	0.8	98.0	-73.1	-189.2	-43.9
Yellow	331.1	304.2	340.0	369.4	280.1	386.2
Red	-9.7	10.3	-197.0	10.4	-435.0	33.3

Instrument Calibartion		
	Corr	Imbal
Zero	5.07	0.04
Zero	4.99	0.02

Table B7. Sensor data after 13 months.

Month 13 Data												
Location	1		2		3		4		5		Salt Coupon	
	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
Corrator	0	0	0.01	0	0	0	OVR	0.91	0	0	0.04	0.01

Location	1		4		5		FR1		FR2		Salt Coupon	
Chloride Ladders												
Corrosion	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal	Corr	Imbal
H	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
L	0	0	0	0	0	0	0	0	0	0	0	0
Imbalance												
H	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
I	0	0	0	0	0	0	0	0	0	0	0	0
L	0	0	0.01	0	0	0	0	0	0	0	0	0

Location	1	2	3	4	5	6
Borin [mV - Ground #1]						
Orange	-187.9	-272.0	-241.3	-100.0	-410.6	-108.8
Blue	-90.6	-104.2	-86.1	-112.1	-574.0	-120.6
Black	-187.9	-272.0	-241.3	-100.0	-410.6	-108.8
Yellow	25.8	186.1	246.0	315.6	185.8	325.5
Red	-90.6	-104.2	-86.1	-112.1	-574.0	-120.6
Borin [mV - Ground #2]						
Orange	-187.9	-272.0	-241.3	-100.0	-410.6	-108.8
Blue	-90.6	-104.2	-86.1	-112.1	-574.0	-120.6
Black	-187.9	-272.0	-241.3	-100.0	-410.6	-108.8
Yellow	25.8	186.1	246.0	315.6	185.8	325.5
Red	-90.6	-104.2	-86.1	-112.1	-574.0	-120.6

Instrument Calibartion		
	Corr	Imbal
Zero	5.07	0.03
Zero	4.99	0.02

The reference electrodes each had two, built-in steel coupons. Figure B7 shows the potentials of these coupons versus time for the six test locations. Again the noble potentials (e.g., around -0.100 mV vs. Ag/AgCl/Sat KCl) are qualitatively indicators of steel that is likely in a passive condition, while active potentials (e.g., more negative than say, -0.250 mV vs. Ag/AgCl/sat KCl) indicate higher probability of corrosion activity. The potentials and trends indicated by the reference electrode built-in coupons did not exactly match the actual steel rebar measured potentials.

Figure B8 represents a plot of the potentials of the rebar (versus Ag/AgCl/sat. KCl reference electrodes) in the steel-reinforced section of the bridge at the six test locations. The initial noble potential values would suggest the passive condition of steel rebar in the highly alkaline (pH ~ 13) concrete environment. There was a general potential shift towards more active values over 10 months, and then a drift toward more noble potentials at 13 months. While active potentials typically suggest increased corrosion activity (i.e., possible loss of passivity at the corresponding areas), the actual corrosion rates indicated by the CORRATER LPR probes were low in all cases except at location 4, where rates appeared to increase and then decrease very dramatically.

The corrosion potentials measured with respect to the reference electrodes indicated corrosion activity ranging from passive to active behavior. However, very low corrosion rates were indicated by the corrosion rate sensors, typically less than 0.1 mpy and with very low pitting propensity. The primary reason for this observation is that insufficient chloride has migrated through the concrete bridge deck to stimulate detectable corrosion attack during the 13-month study. This is not surprising because it takes many years, and often, decades, for a significant amount of chloride to permeate through good quality concrete; a thicker concrete cover also impedes chloride migration. (See Figures B7 and B8).

The concrete test block salt coupon artificially contaminated with chloride indicated generally greater corrosion activity at 1 and 4 months compared to the bridge deck. However, this activity diminished at 7, 10, and 13 months, probably due to drying out of the test block. (Figure B9).

Corrosion will eventually be detected when enough chloride has reached the sensors embedded in the bridge deck concrete. The greater the amounts and frequency of deicing road salt usage, the shorter the chloride permeation time leading to significant corrosion. Even then, it could take many years. Therefore, it is recommended that monitoring of the corrosion sensors embedded in the concrete bridge deck at Fort Knox be continued (for example every 5 years), to confirm their veracity.

Figure B7. Coupon potentials vs. time at the six test locations on the steel-reinforced section of the bridge.

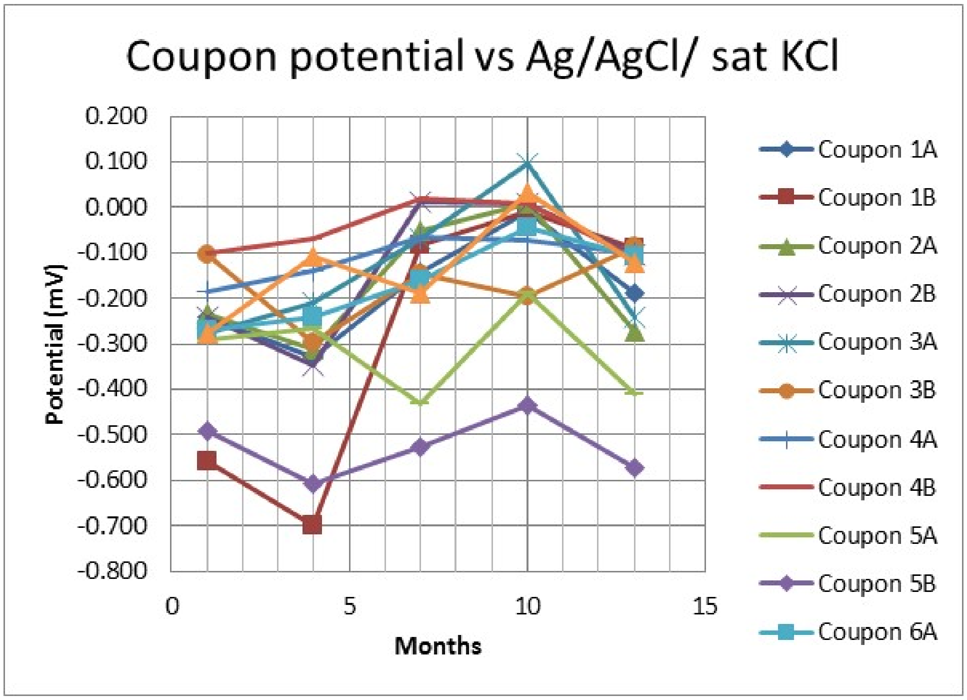


Figure B8. Rebar potential vs. time at the six test locations on the steel-reinforced section of the bridge.

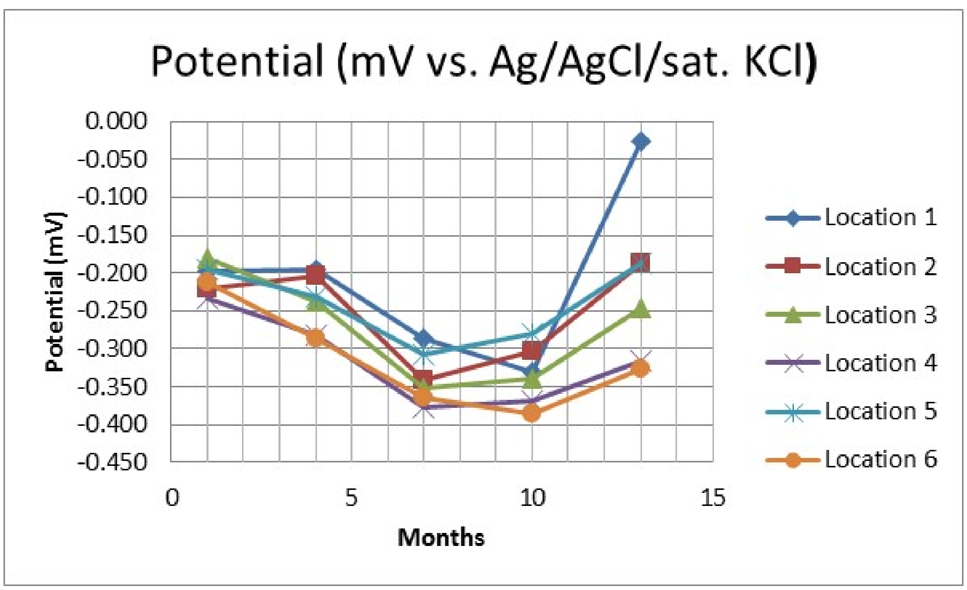
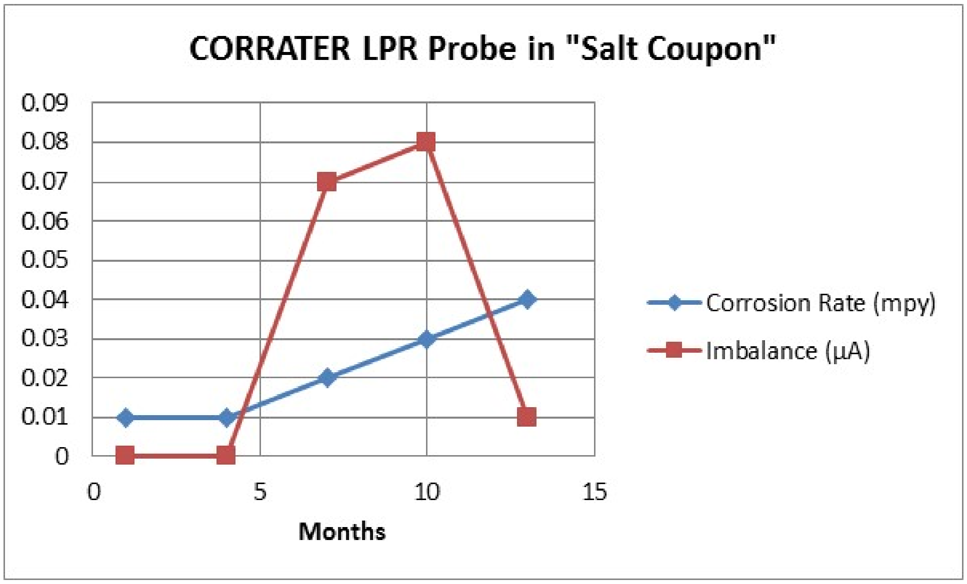


Figure B9. Corrosion rate and imbalance readings vs. time for CORRATER probe in artificially-contaminated concrete block "salt coupon."



Corrosion coupon rack assessment

The atmospheric corrosion coupon rack placed at the site had coupons removed at 6 and 12 months. These coupons were sent to a certified lab and mass loss was measured per ASTM G1-03 on the AL 6061 T6, AL 2024 T3, AL 7075 T6, C 1010, and CDA 101. The silver test coupon had Coulometric Reduction of Surface Films done per ASTM B 825-13. These test results are included as Attachments 1 and 2 at the end of this appendix. A summary of the results and classification according to the categories listed in Table 2 from ISO 9223:2012 are shown in Table 2 and Table 3 in Section 3.1.1 of this report. The copper experienced a high mass loss in comparison to the other metals. The results from the 12-month testing suggest that the 2024 and 7075 aluminum alloys experienced an extremely high mass loss due to corrosion. These results are inconsistent with the other alloys and the results from the weather data analysis; therefore, the mass loss test from the 12 month 7075 and 2024 coupon have been omitted from the atmospheric corrosion severity classification of the site.

Corrosion severity site classification

The results from the ISO 9223:2012 analysis of weather data and mass loss testing suggest the Fort Knox Bridge No. 4 site is a C3 classification of atmospheric corrosion severity. Although the steel coupon testing resulted in a C2 classification, the results were on the upper limit of the category.

The potential for corrosion at the site is considered medium. The corrosion sensors show no corrosion in the bridge and validate this classification as being much less than severe.

Attachment 1

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ACCELERATED ENVIRONMENTAL TEST REPORT

Ref. D201577 **Date** November 29, 2013 **Page** 1 **of** 2

Christopher Olaes
Mandaree Enterprise Corp.
812 Park Dr.
Warner Robins, Georgia 31088

Purchase Order #: 2012-22

Procedure

Test Performed	Method
Mass Loss Evaluation	ASTM G1-03 (2011)
Test Material	**Requirements**
Metal Test Coupons	None Specified

Results

Sample ID	Part Number	Initial Weight [g]	Final Weight [g]	Δ Weight [g]	Weight Loss %
AL6061 T6	COR123400304100	9.30895	9.30359	0.00536	0.06
AL2024 T3	COR122990304100	8.98105	8.97847	0.00258	0.03
AL7075 T6	COR123470304100	9.36348	9.35870	0.00478	0.05
CDA1010	COR124140304100	32.60544	32.50175	0.10369	0.32
CDA101	COR124140304100	28.64740	28.23034	0.41706	1.46

ISO 9001

Prepared by: F. Lopez
Supervisor

Approved by: E. W. Sproat
Group Manager

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ACCELERATED ENVIRONMENTAL TEST REPORT

Ref. D201577 **Date** November 29, 2013 **Page** 2 **of** 2

Christopher Olaes
Mandaree Enterprise Corp.
812 Park Dr.
Warner Robins, Georgia 31088

Purchase Order #: 2012-22

Procedure

Test Performed	Method
Coulometric Reduction of Surface Films on Metallic Surfaces	ASTM B 825-13

Test Material	Requirements
Silver Test Coupon	None Specified

Results

Sample ID	Part Number	Results
Ag	COR117520304100	Reduction Time = 1025 Seconds Total Reduction Charge = 1.943 Coulombs

ISO 9001

Prepared by: F. Lopez, Supervisor

Approved by: Gene Price, P.E., Senior Engineer

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Attachment 2

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MATERIALS TEST REPORT

Ref. D209559 **Date** February 18, 2014 **Page** 1 **of** 7

Christopher Olaes
Mandaree Enterprise Corp.
812 Park Dr.
Warner Robins, Georgia 31088

Purchase Order #: 2012-22

Procedure

Test Performed	Method
Mass Loss Evaluation	ASTM G1-03 (2011)
Test Material	Requirements
Metal Test Coupons	None Specified

Results

Sample ID	Part Number	Initial Weight [g]	Final Weight [g]	Δ Weight [g]	Weight Loss %
AL6061 T6	COR123400304100	9.5998	9.5940	0.00536	0.06
AL2024 T3	COR122990304100	9.1249	8.8314	0.00258	0.03
AL7075 T6	COR123470304100	9.6156	9.4233	0.00478	0.05
C1010	COR124140304100	28.5495	27.5658	0.10369	0.32
CDA101	COR124140304100	32.3241	32.1816	0.41706	1.46

ISO 9001

Prepared by: T. Burris
Materials Testing

Approved by: F. Lopez
Supervisor

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MATERIALS TEST REPORT

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Figure 1: AL6061 T6 Sample prior to Corrosion Removal

Figure 2: AL6061 T6 after Corrosion Removal

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Figure 3: AL2024 T3 Sample prior to Corrosion Removal

Figure 4: AL2024 T3 after Corrosion Removal

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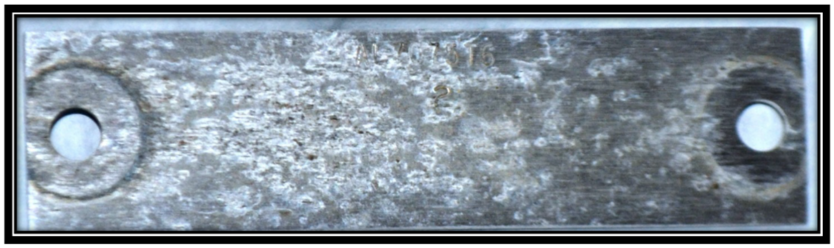

Figure 5: AL7075 T6 Sample prior to Corrosion Removal

Figure 6: AL7075 T6 after Corrosion Removal

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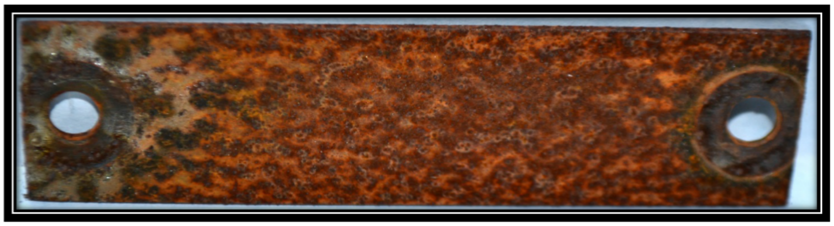

Figure 7: C1010 Sample prior to Corrosion Removal

Figure 8: C1010 after Corrosion Removal

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Figure 9: CDA101 Sample prior to Corrosion Removal

Figure 10: CDA101 after Corrosion Removal

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MATERIALS TEST REPORT

Ref. D209559 **Date** February 18, 2014 **Page** 7 **of** 7

Christopher Olaes
Mandaree Enterprise Corp.
812 Park Dr.
Warner Robins, Georgia 31088

Purchase Order #: 2012-22

Procedure

Test Performed	Method
Coulometric Reduction of Surface Films on Metallic Surfaces	ASTM B 825-13

Test Material	Requirements
Silver Test Coupon	None Specified

Results

Sample ID	Part Number	Results
Ag	COR117520304100	Reduction Time = 1,660 Seconds Total Reduction Charge = 3.146 Coulombs

ISO 9001

Prepared by: F. Lopez
Supervisor

Approved by: Gene Price, P.E.
Senior Engineer

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ATS905B, 01/2010

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14. ABSTRACT

The Department of Defense (DoD) maintains a large array of road networks that include vehicular bridges. Moving people, materials, and equipment is critical to the DoD mission. Many of these bridges are in dire need of major repairs or replacement due to corrosion and material degradation. The application of corrosion-resistant technology can extend the service life of bridges and reduce maintenance costs. This DoD Corrosion Prevention and Control Program project demonstrated and validated the performance characteristics of the fiber-reinforced polymer (FRP) composite, three-dimensional Gridform product for reinforcing concrete bridge decks that was designed to solve many of the installation and life-cycle problems associated with steel-reinforced bridge decks. The Gridform technology replaced the existing steel-reinforced concrete deck on one span of Bridge No. 4 at Fort Knox, Kentucky. The newly replaced span's performance was compared to a second span that was newly replaced with a concrete deck using traditional steel rebar reinforcement. Structural testing and corrosion monitoring and analysis of the bridge was performed. Results show that using Gridform technology could provide needed load capacity and improved corrosion protection for DoD bridges, while maintaining structural capability. The technology's return on investment (ROI) is 10.31.

15. SUBJECT TERMS

Structural dynamics; Bridges–Live loads; Bridges–Design and construction; Composite materials–Testing; Corrosion-resistant materials; Corrosion Prevention and Control (CPC) Program; Gridform; Fort Knox, Kentucky

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