

# Instrumentation and Monitoring of Rustic Road Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS)



Prepared by  
Andrew Boeckmann  
Eric Lindsey  
Sam Runge  
J. Erik Loehr

University of Missouri-Columbia Department of Civil and Environmental Engineering



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16. Abstract An instrumentation and monitoring system was implemented for a geosynthetic reinforced soil (GRS) integrated bridge system (IBS) constructed in Boone County, Missouri in 2014. The project location is subjected to relatively frequent flash flooding, which was a significant consideration in the design of the bridge and the design of the monitoring system. The monitoring system includes 26 surveying points on the bridge exterior to monitor external movement; settlement plates and inclinometers to monitor vertical and horizontal exterior movement, respectively; earth pressure cells to monitor total stresses within the abutment backfill; and vibrating wire piezometers to monitor pore pressures and drainage within the abutment backfill. The GRS-IBS was monitored for a period of 19 months after construction. The monitoring period included several high-water events, but none overtopped the bridge. The results indicate satisfactory performance, including negligible external and internal movements and rapid backfill drainage in response to groundwater level increases.			
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prepared for

**Missouri Department of Transportation**

by

**Andrew Boeckmann, Eric Lindsey,  
Sam Runge, and J. Erik Loehr**

**University of Missouri  
Department of Civil and Environmental Engineering**

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## **Disclaimer**

The opinions, findings, and conclusions expressed in this document are those of the investigators. They are not necessarily those of the Missouri Department of Transportation, U.S. Department of Transportation, or Federal Highway Administration. This information does not constitute a standard or specification.

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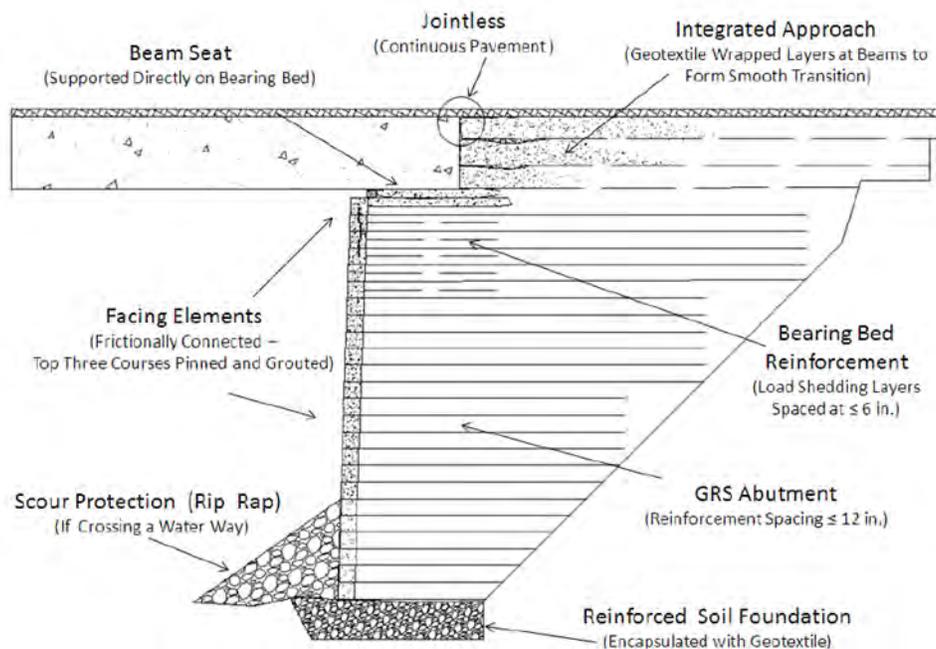
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## 1. Introduction

The Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) is a technology developed and promoted by the Federal Highway Administration (FHWA) to deliver accelerated bridge construction economically, primarily for relatively small bridges. A schematic of a typical GRS-IBS abutment is shown in Figure 1 (Adams et al., 2012). The technology harnesses the stiffness of GRS to eliminate the need for piling or other conventional foundation systems. Eliminating piling typically results in cost and schedule benefits. As shown in Figure 1, the reinforced soil provides a continuous foundation for both the superstructure and the integral approach. The integration reduces the likelihood of the “bump at the end of the bridge” that is often associated with pile foundations. Eliminating the bump is another benefit frequently cited by GRS-IBS proponents.



**Figure 1: Components of a GRS-IBS abutment from Adams et al. (2012).**

In 2013, three bridge replacement projects utilizing GRS-IBS were initiated in Missouri. One of the replacements, Rustic Road bridge in Boone County, was selected for instrumentation and performance monitoring. The bridge has several unique features, including a two-year return period for floodwaters to overtop the bridge deck and a 15-deg. skew. These features, along with the Rustic Road’s light traffic, made the bridge an appealing candidate for monitoring, which was conducted by the University of Missouri for approximately 1.5 years.

Chapter 2 of this report presents additional background information regarding GRS-IBS technology and the Rustic Road bridge, including project construction details. Details of the instrumentation selection and installation are presented in Chapter 3, and Chapter 4 presents the monitoring results. Finally, conclusions are presented in Chapter 5.

## 2. Background

This chapter includes an introduction to important technical concepts related to GRS-IBS as well as a description of the design and construction of the Rustic Road bridge replacement project. Both topics are relevant to the design and implementation of the monitoring program described in the rest of the report.

### 2.1 Technical Background

GRS was first implemented on a widespread basis in the 1970s for construction of slopes and retaining walls for Federal Lands Highways. In the past decade, FHWA has published several comprehensive research and guidance reports related to GRS, including recent guidelines for implementation of GRS for bridge applications as GRS-IBS. Summaries of the most relevant portions of the FHWA publications are presented in the sections below.

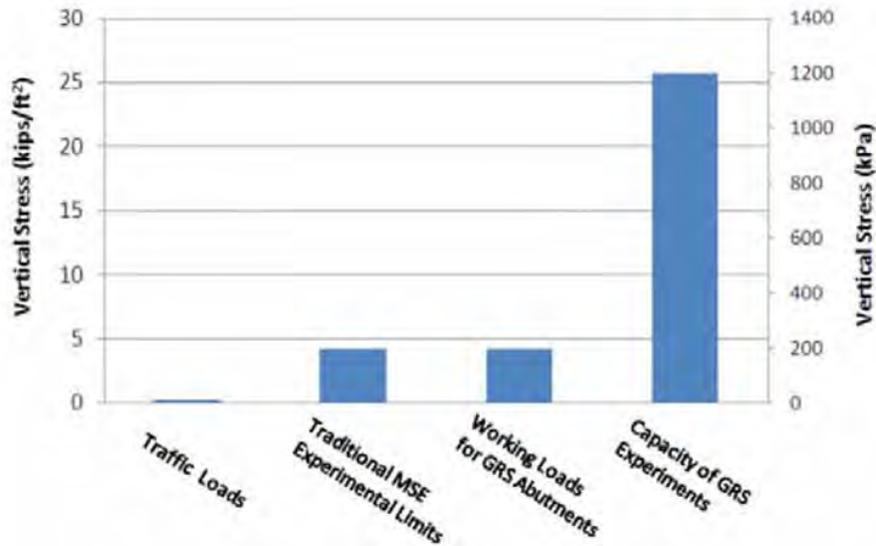
#### 2.1.1 GRS

GRS refers to the composite material consisting of compacted soil and closely spaced ( $\leq 12$  in. per FHWA) layers of geosynthetic reinforcement. FHWA's *Composite Behavior of Geosynthetic Reinforced Soil Mass* (Wu et al., 2013) explains that strength and stiffness of a soil mass are improved by reinforcement, which increases confinement of the soil while reducing lateral movement and dilation. As demonstrated by Figure 2, GRS is internally stable; the wall facing for GRS is not structural.



**Figure 2: Free-standing GRS structure (Adams et al., 2011).**

Wu et al. emphasize the distinction between GRS, which is a composite material, and mechanically stabilized earth (MSE), which also contains layers of compacted soil separated by reinforcement but does not behave like a composite material. Thus, while it is appropriate to consider individual tensile forces from the reinforcement for MSE walls, such an approach is not adequate for GRS because it neglects the effect of close reinforcement spacing on the behavior of the soil mass. As demonstrated in Figure 3, the composite behavior for closely spaced reinforcement has resulted in applied surcharges to GRS that are several times greater than those applied to MSE.



**Figure 3: Surcharges imposed on MSE and GRS (Adams et al., 2011).**

The difference in the role of reinforcement between MSE and GRS is reflected in the required tensile strength for each. For MSE, the required reinforcement strength,  $T_{req}$ , is directly proportional to the reinforcement spacing,  $S_v$ :

$$T_{req} = \sigma_h * S_v \quad \text{Eq. 1}$$

where  $\sigma_h$  is the lateral earth pressure at the reinforcement depth. For GRS, the relationship between the required reinforcement strength and reinforcement spacing is more complicated than for MSE because of the composite nature of GRS:

$$T_{req} = \left[ \frac{\sigma_h}{0.7 \left( \frac{S_v}{6d_{max}} \right)} \right] * S_v \quad \text{Eq. 2}$$

where  $d_{max}$  is the maximum grain size of the backfill. Equation 2 was developed by Wu et al. (2013) based on the results of analytical modeling and laboratory tests of full-scale physical models. Typically, a factor of safety (or reduction factors) would be applied when selecting the design reinforcement.

### 2.1.2 GRS-IBS

Early implementations of GRS primarily involved retaining walls and slopes, but its use for bridges via GRS-IBS (Figure 1) has accelerated since FHWA introduced GRS-IBS as an Every Day Counts initiative in 2011 (Adams et al., 2011). As part of the initiative, FHWA published the *GRS-IBS Interim Implementation Guide* (Adams et al., 2012) and the *GRS-IBS Synthesis Report* (Adams et al., 2011). The guide includes recommended material specifications and procedures for design and construction of GRS-IBS, as well as recommended inspection methods, QA/QC procedures, and maintenance procedures. The synthesis report documents technical background for GRS-IBS, including the research studies and case histories used to develop the implementation guide.

Chapter 3 of the guide presents information regarding materials for GRS-IBS walls, the most significant of which are shown in Figure 1. The facing elements for GRS-IBS are most frequently concrete masonry unit (CMU) blocks. CMU blocks have several advantages: they are relatively inexpensive, they provide a form for compaction of backfill material, and extending the geosynthetic between the rows of CMU blocks serves as a frictional connection. Selection of GRS backfill material is critical since the GRS is a structural component directly supporting the bridge load. The guide recommends either well-graded or open-graded

aggregate backfill, but notes that all GRS-IBS abutments at the time of publication (2012) used open-graded backfill because of its constructability and high hydraulic conductivity. The guide specifically recommends open-graded backfill for projects sites located in a flood zone. Rustic Road is such a site. The guide states GRS backfill must be properly compacted to a minimum of 95 percent of maximum dry density from a standard Proctor test (AASHTO T-99). The guide notes that many types of geosynthetic materials can satisfy strength requirements for most implementations of GRS, but all GRS-IBS abutments constructed at the time of publication had used a biaxial, woven polypropylene geotextile. Such geotextiles are typically selected because they are relatively inexpensive and easy to place.

A detailed GRS-IBS design procedure is presented in Chapter 4 of the guide, which begins with the outline of the procedure shown in Figure 4. The guide states that GRS has been shown to perform well “under certain extreme conditions,” but the guide limits its recommendations to GRS-IBS structures with heights not exceeding 30 ft and spans not exceeding 140 ft. The guide also emphasizes requirements for backfill compaction to 95% of maximum dry density and reinforcement spacing less than 12 in. in the introduction to the design guidance. The design procedure detailed in the guide and outlined in Figure 4 is similar to the procedure FHWA recommends for design of MSE walls, with a few important differences. One difference is the third step, which involves evaluating the feasibility of using GRS-IBS. The guide primarily discusses the importance of evaluating scour for GRS-IBS over water since GRS-IBS have no deep foundation elements. Another main difference is the load calculation. GRS-IBS is subjected to significant loading from the bridge dead and live loading; in a typical MSE abutment, the bridge loads are transferred to deep foundation elements rather than the MSE backfill. Finally, the internal stability analysis procedure is notably different for GRS-IBS since GRS backfill is a composite material (e.g., the differences between Equations 1 and 2 for required reinforcement strength).

Chapter 7 of the guide provides detailed procedures for construction of GRS-IBS. The introduction to the chapter emphasizes the feasibility of quickly constructing GRS-IBS since the construction is completed with “basic earthwork methods” and readily available materials. Most of the construction progress is completed with three relatively simple jobs: placement of wall face blocks, compaction of GRS backfill aggregate behind the blocks, and placement of reinforcement. The introduction also calls out four important details for successful GRS-IBS construction:

- A “level and even” bottom row of blocks, since each subsequent row of blocks and GRS course is built off the bottom row.
- Optimized crew size and equipment.
- Allowing the labor crew to become familiar with the construction procedure, specifically by having each member “do their part” in each of the three simple steps described above.
- Locating the excavator such that it can place backfill material without tracking.

The rest of the chapter provides specific details for the construction procedures, including site preparation/excavation, construction of the reinforced soil foundation below the GRS, placement and compaction of backfill, placement of reinforcement, alignment of the wall face, preparation of the beam seat, placement of the superstructure, and approach integration. Photographs of these tasks for the Rustic Road project are shown in the next section.

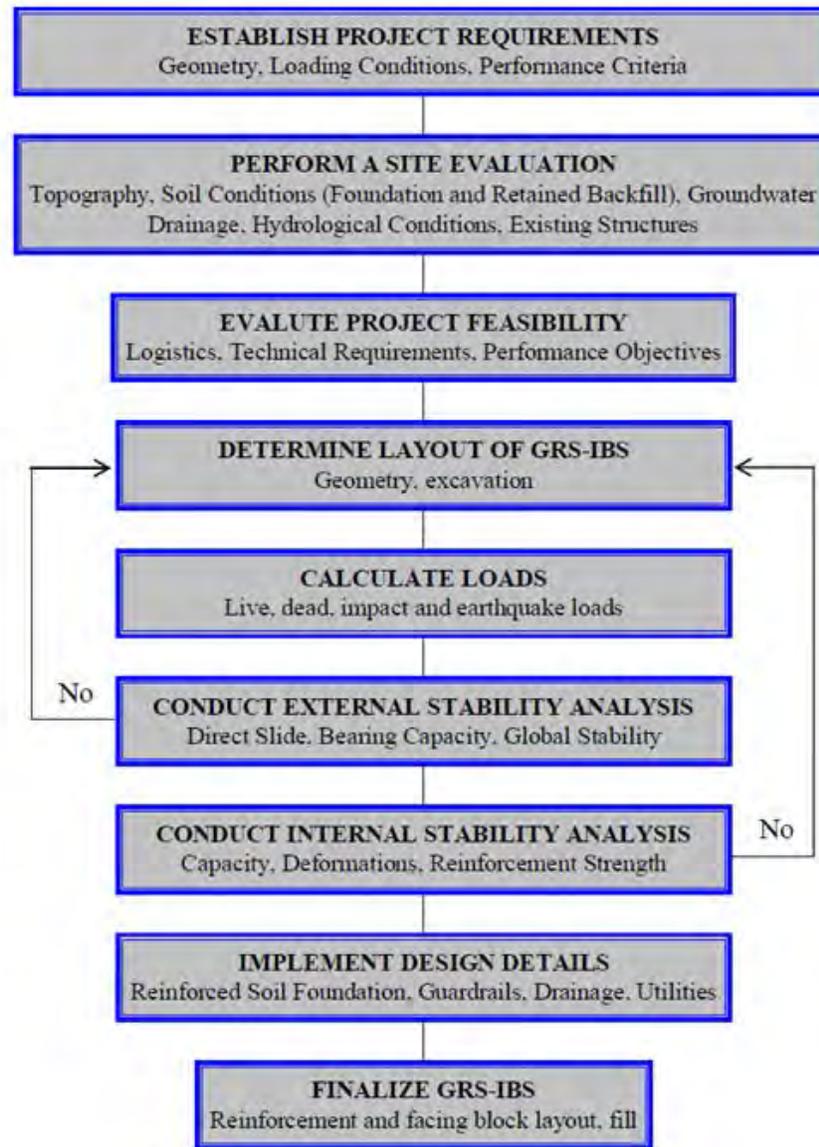


Figure 4: GRS-IBS design steps from FHWA *Interim Implementation Manual* (Adams et al., 2012).

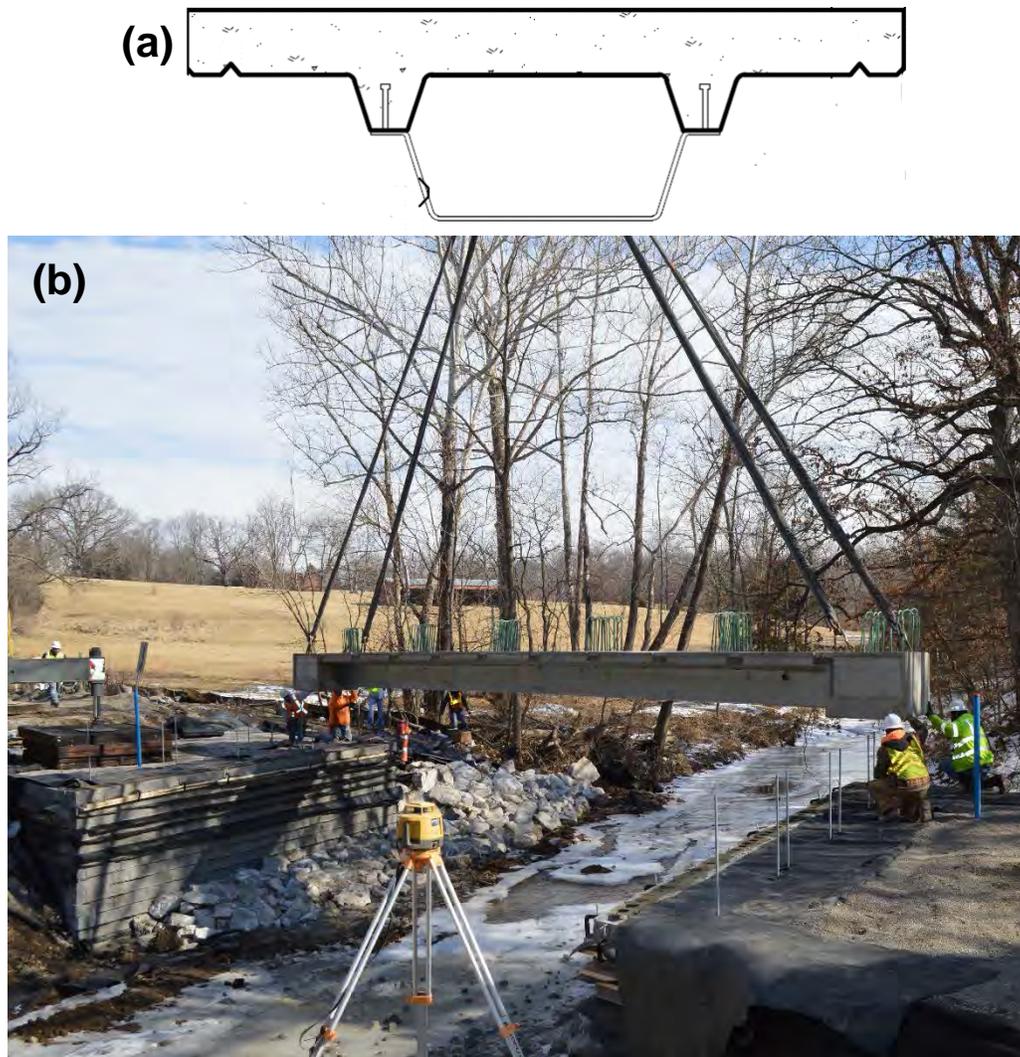
## 2.2 Rustic Road Bridge Replacement Project

Rustic Road is a low-volume road just east of Columbia, Missouri in Boone County. The road crosses the North Fork of Grindstone Creek to provide passage to approximately 10 residences before reaching a dead end. In 2013, deterioration of the original Rustic Road bridge (Figure 5) led to a bridge load rating that precluded fire trucks from crossing the bridge. The replacement project was identified as a candidate for GRS-IBS because the bridge is relatively short in span (50 ft) and height (14 ft), because of the need for rapid replacement since there are no detours for the roadway south of the bridge, and because relatively frequent flooding of the project site made the project an interesting GRS-IBS test case.



**Figure 5: Original Rustic Road bridge showing deterioration of the outer girder, including corrosion through an entire portion of the web near the abutment.**

Design of the bridge replacement was completed by Bartlett and West, Inc. of Jefferson City, Missouri. The design was completed largely in accordance with the FHWA *GRS-IBS Interim Implementation Manual* (Adams et al., 2012). Plans for the completed design are included in the appendix. GRS-IBS was not the only innovative initiative included for the Rustic Road bridge replacement. The superstructure consists of four tub girders with attached precast bridge deck sections as shown in Figure 6. The four pieces were fabricated off-site, and placement of the girders was completed during the course of one day. To counter buoyancy forces on the tub girders, each girder was anchored to plates embedded approximately 3 ft in the GRS via the bolts shown in Figure 6(b). Plates are shown in Figure 23. Vent holes were also included in the girders to prevent trapped air from forming between the tubs during a flood.



**Figure 6: Tub girders with precast bridge deck: (a) cross-section and (b) photograph of tub girder being lowered into place.**

The GRS backfill material for the Rustic Road GRS-IBS consists of open-graded aggregate meeting specifications for AASHTO No. 89 stone. The GRS reinforcement is a biaxial, woven polypropylene geotextile. In addition, needle-punched, nonwoven polypropylene geotextile was wrapped around each GRS layer just inside of the wall facing blocks. The separation geotextile was included to prevent loss of material in case of damage to the facing blocks. The facing blocks were 8-in. tall by 12-in. long by 8-in. wide split-face gray CMU blocks. Below grade (i.e. the first five courses of GRS), solid red CMU blocks of the same dimensions were used for scour resistance and detection (via any exposure during the life of the abutment).

Construction progress is documented in the photographs shown in Figure 7 through Figure 34. Details are described in the figure captions. Construction progress generally followed the sequence and procedures outlined in the *Interim Implementation Manual*, although several unique aspects of the project required deviations from the manual. Limestone bedrock was encountered at depths shallower than anticipated, requiring excavation via rock chipping hammer (Figure 7, Figure 8) to achieve adequate abutment embedment. In addition, persistent seepage entered the excavation for the north abutment from a permeable layer exposed by the excavation. A submersible pump was used to remove water from the excavation prior to compaction (Figure 15). The construction crew experienced difficulty with the facing blocks creeping outward during vibratory compaction. To reduce the displacement, the crew clamped

lumber to the reinforcement extending out in front of the wall from just beneath the course being compacted (Figure 20, Figure 21). Photographs of the bridge after construction are shown in Figure 33 and Figure 34. Additional photographs from construction are presented in Chapter 3 to detail the installation of instrumentation.



**Figure 7: Rock chipping hammer is used to excavate limestone bedrock below south abutment.**



**Figure 8: Rock chipping hammer and backhoe are used to excavate limestone bedrock below south abutment.**



Figure 9: Compaction of reinforced soil foundation for the south abutment.



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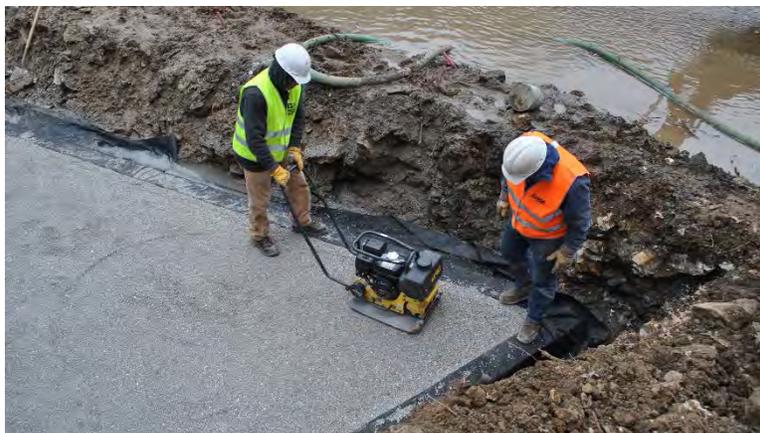


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**Figure 14: Completed third course of south abutment.**



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**Figure 18: Placing reinforcement around instrumentation (inclinometer and telltales) atop tenth course of GRS of north abutment. Instrumentation installation is presented in Chapter 3.**



**Figure 19: Completed tenth course of GRS of north abutment.**



**Figure 20: Leveling CMU blocks ahead of placing backfill. Boards clamped to reinforcement were used to prevent movement of the blocks for the previous course during compaction.**



**Figure 21: Securing boards to reinforcement to prevent movement of the CMU blocks during compaction.**



**Figure 22: Surveying and leveling to place anchor plates for girders. Anchors were necessary to provide resistance against buoyancy forces against the tub girders.**



**Figure 23: One anchor plate per girder was placed within backfill for each abutment. Plates were set approximately 3 ft below the top of abutment.**



**Figure 24: Preparation of the beam seat for the north abutment.**



**Figure 25: Placement of the reinforcement for the bottom layer of the beam seat for the north abutment. The beam seat consisted of two 4-in. thick layers of aggregate wrapped with reinforcement. Styrofoam was placed in the front of each layer, and a row of narrow CMU blocks was placed in the back of each layer.**



Figure 26: Compacting first layer of the beam seat for the north abutment.



Figure 27: Preparing second layer of beam seat for the north abutment.



Figure 28: Compacting second layer of beam seat.



**Figure 29: Grouting CMU block openings of the top layer of the abutment.**



**Figure 30: First girder is lowered into place. Crew ensured holes through girders were aligned with anchor bolts. Figure 6 is another photograph of lowering the first girder.**



**Figure 31: Placing third girder.**



**Figure 32: Crew places additional aggregate below first girder to achieve a level surface.**



**Figure 33: Rustic Road GRS-IBS from the southwest.**



**Figure 34: Rustic Road GRS-IBS from the northeast.**

### 3. Monitoring System Design and Installation

To monitor the performance of Rustic Road GRS-IBS, a system of instrumentation, land surveying, and visual observations was implemented. This chapter provides details regarding the design of the monitoring system, with each chapter section addressing a different component of the system: surveying to monitor external movement of the GRS-IBS, settlement plates and inclinometers to record displacement within the GRS-IBS abutments, earth pressure cells to measure total stresses within the abutment backfill, and piezometers to measure pore pressures within the abutment backfill. Results of the monitoring are presented in Chapter 4.

#### 3.1 External Movement and Scour

External movement is a critical indicator of the performance of any bridge system. External movement refers to displacement of the outside surfaces of the GRS-IBS, including settlement of the bridge or abutments and lateral displacement of the abutments (e.g. bulging). External movement of the GRS-IBS was primarily monitored via land surveying, which was performed by the City of Columbia survey crew. In addition, crack gages were installed on four CMU blocks that cracked shortly after completion of construction; the gages provide another indication of external movement. Visual observations during regular monitoring site visits provided another indication of any significant external movement. Visual observations also allowed for monitoring of other performance metrics such as the presence of scour, which is observed via significant displacement of rip rap or exposure of the red CMU blocks (Chapter 2).

Surveying was conducted by the City of Columbia survey crew on a quarterly basis, with an initial survey conducted upon completion of construction of the GRS-IBS as shown in Figure 35. The benchmark for all site surveys was a survey marker established in the limestone bedrock exposed in the bed of the creek below Rustic Road GRS-IBS (Figure 36). The survey crew used a total station device to perform the surveying. Twenty-eight points on the Rustic Road GRS-IBS were surveyed each quarter to monitor external movement. The points include 12 reflective markers on the face of each abutment as shown in Figure 37 as well as the four corners of the bridge. In addition, the survey crew surveyed the settlement plate devices described in the next section.

Shortly after construction completion, cracks developed at the top of all four abutment wing walls at the locations shown in Figure 38. The cracks are discussed in Chapter 4. Each crack was monitored using a crack gage as shown in Figure 39. Each crack gage has two plastic pieces, one with a 40 mm by 20 mm grid and the other with a crosshair. Epoxy was used to attach the pieces to opposite sides of the crack being monitored so that further opening of the crack would be indicated by movement of the crosshair with respect to the grid.



**Figure 35: City of Columbia survey crew performed initial survey after Rustic Road GRS-IBS was substantially complete but before it was open to traffic.**



Figure 36: Benchmark for surveying was established in limestone bedrock of creek running below Rustic Road GRS-IBS.



Figure 37: A grid of 12 reflective survey markers bolted to CMU blocks was installed on the face of each abutment: (a) Grid for north abutment with marker labels as established by survey crew and (b) close view of reflective marker. Labeling for south abutment markers is similar after substituting “S” for “N,” but the location of markers 1 and 3, 4 and 6, and 7 and 9 are reversed.

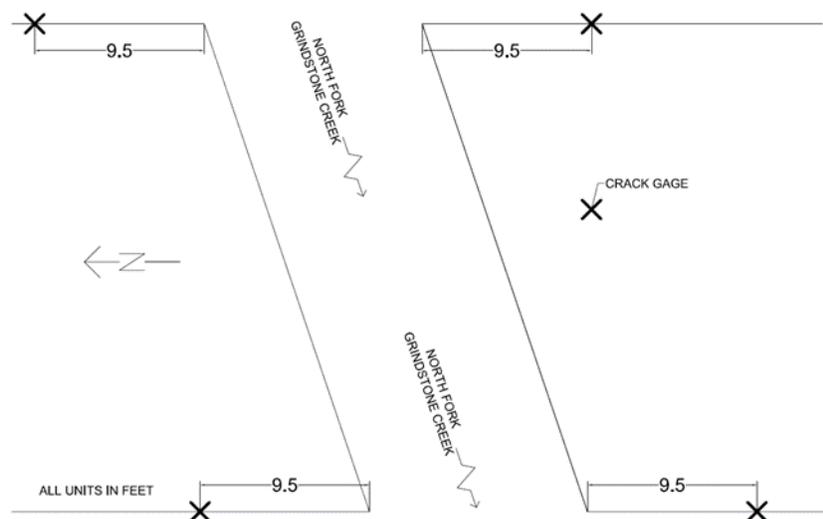
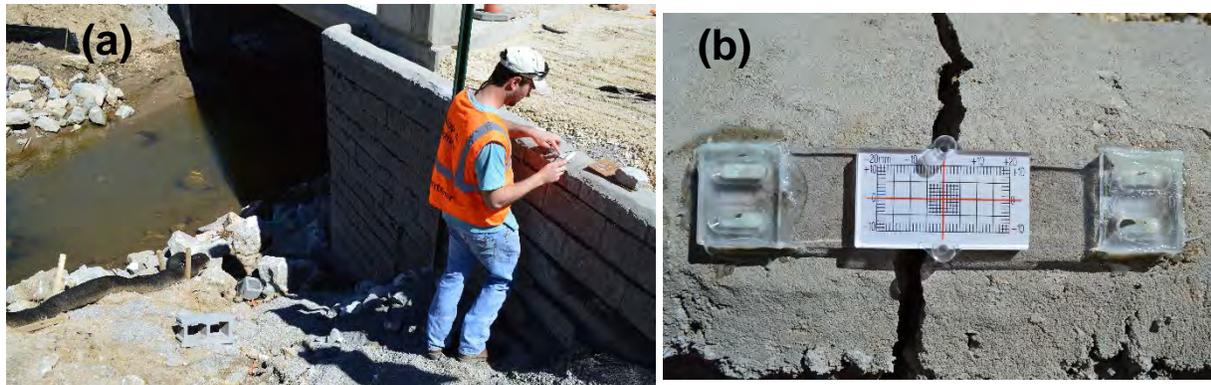


Figure 38: Crack gages were installed where cracks developed on all four wing walls (plan view).



**Figure 39: Crack gages were used to monitor cracks that developed shortly after completion of GRS-IBS: (a) installation of gage and (b) close view of crack gage.**

### 3.2 Internal Movement

Internal movement refers to displacement within the GRS abutment backfill. Internal movement is an important measure that can explain observed performance. For example, information regarding vertical displacement within the abutment backfill would help explain the origin of observed settlement at the surface of the abutment. For Rustic Road, vertical internal movement was monitored using three settlement plates installed in the north abutment, and lateral internal movement was monitored using an inclinometer, with one inclinometer casing installed through each abutment.

As shown in Figure 40, the settlement plate devices consist of 12-in. square, 0.25-in. thick steel plates that were embedded in the backfill, with threaded steel rods extending up from the plates to the top of the abutment. Coarse sand was placed between the bottom of the settlement plate and the coarse gravel GRS backfill to facilitate horizontal installation of the plates. The threaded rods extended up to the top of the abutment through loose PVC sleeves, which prevent friction between the rod and backfill (Figure 40b). As shown in Figure 41, the three settlement plates were installed above one another (i.e. along a vertical line) about 5 ft behind the north abutment wall, with the bottom plate about 2 ft above the reinforced soil foundation, the middle plate about 6 ft above the foundation (i.e. mid-height), and the top plate about 3 ft below the pavement. The upper two plates were slotted to allow the rod and pipe from the lower plate(s) to pass through the upper plates as shown in Figure 42. The pipes and rods from all three plates were housed in a common cast iron housing embedded in the pavement (Figure 43). In plan view, the settlement plates were located in the center of one driving lane to reduce the incidence of vehicle tires striking the housing. Settlement plate measurements were collected via the land survey performed by the City of Columbia survey crew as described in the previous section. The surveys were collected quarterly throughout the monitoring period.



**Figure 40: Installation of bottom settlement plate: (a) placement and leveling on thin layer of coarse sand and (b) loose PVC sleeve over steel rod prevents friction between rod and aggregate.**

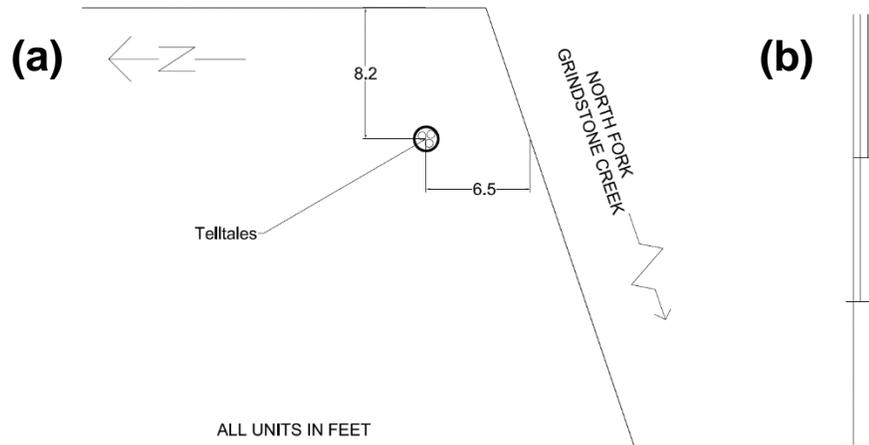


Figure 41: Settlement plate locations within the north abutment: (a) plan and (b) elevation.



Figure 42: Rods and PVC from bottom and middle two settlement plates extended up through slot in top settlement plate.

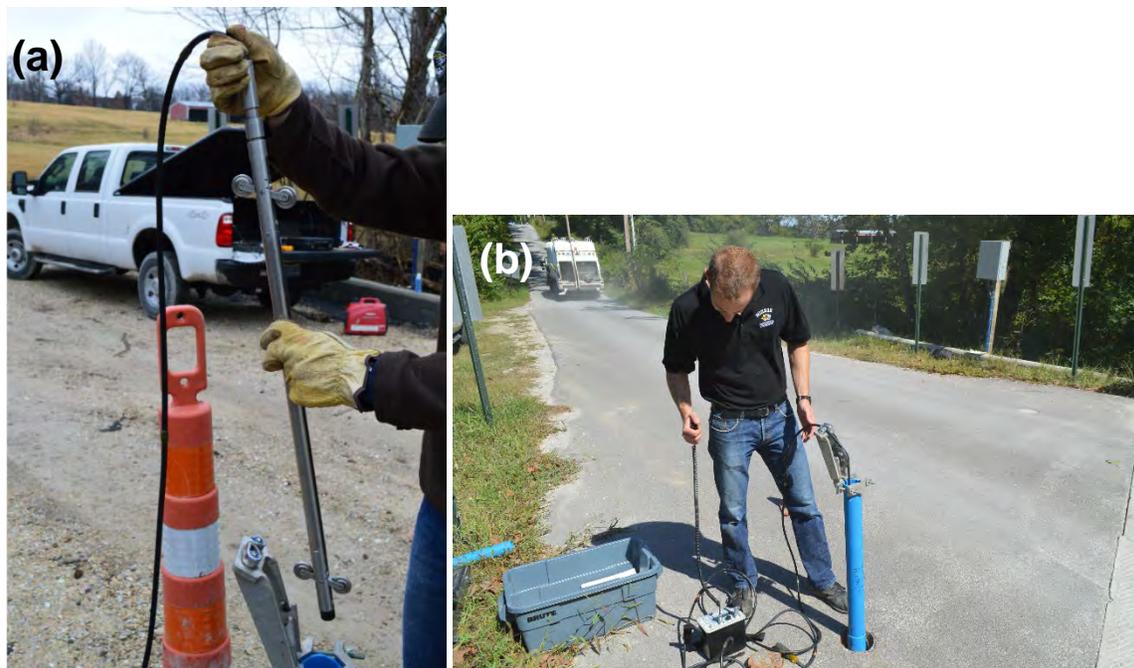


Figure 43: Settlement plate rods and PVC were extended to common cast iron housing embedded in the roadway surface: (a) housing with cover and (b) housing without cover.

An inclinometer system was used to measure lateral internal displacement of each GRS abutment. The inclinometer system consists of a proprietary plastic casing and an inclinometer probe. The casing has four machined grooves at 90-deg. angles running along its length. The casing is installed so that the bottom of the casing is fixed and the top is accessible at the ground surface. For the Rustic Road abutments, the bottom of each casing was fixed as shown in Figure 44: a 12-in. deep hole into limestone bedrock was established by chipping, the hole was filled with grout, and the casing was set in the hole. The probe has wheels that travel along the casing grooves. As the probe is lowered down and raised up the casing (Figure 45), the probe records measurements of angle with respect to gravity. Integration of the measurements results in an interpreted casing shape, and comparison of subsequent sets of readings produces change in casing shape, which is interpreted as lateral deflection of the abutment. Additional details of the inclinometer system are provided in the system instruction manual (Slope Indicator, 2011). Like the settlement plates, inclinometer casings were also located about 5 ft behind the abutment wall face and in the center of a driving lane (Figure 46), and the tops of the casings were located in cast iron housings. Inclinometer readings were collected by the University during each monitoring site visit. The visits occurred monthly for the first year of monitoring and then every other month for the following six months.



**Figure 44: Inclinometer casing installation: (a) chipping out 12-in. of limestone bedrock, (b) mixing grout within hole in bedrock, (c) casing is inserted in grouted hole, and (d) view of final installation. GRS abutment was built up around casing as shown in Chapter 2 photographs.**



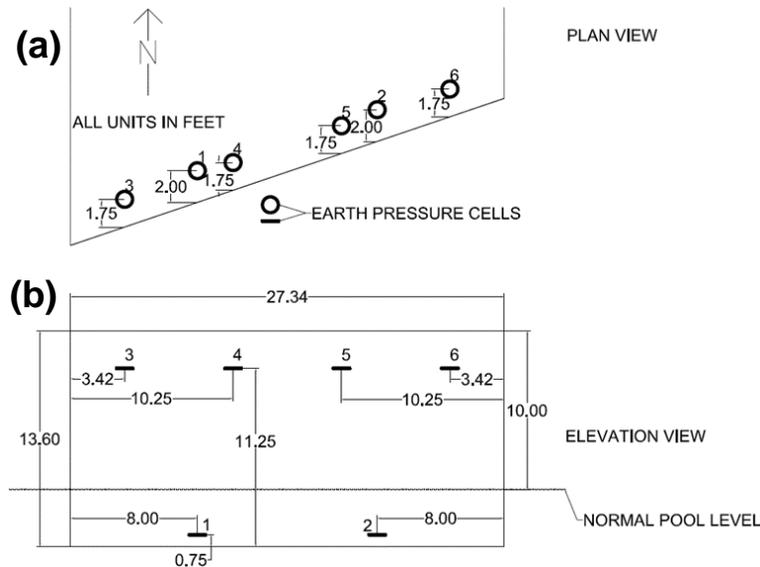
**Figure 45: Collection of inclinometer data: (a) inclinometer probe is inserted into casing and (b) probe measurements are recorded in 2-ft increments as the probe is lowered to the bottom of casing and then raised back up to the ground surface. A pulley assembly and casing extension were used to facilitate data collection.**



**Figure 46: Compacting north abutment backfill around settlement plate rods. Inclinometer casing is shown in background to right. GRS course being compacted is near the top of the abutment, level with the bottom of the bridge girders.**

### 3.3 Earth Pressure within GRS Backfill

Information regarding earth pressure, or stress, within the GRS backfill indicates how load is distributed within the abutment. GRS-IBS loading primarily comes from self-weight (i.e. weight of the GRS backfill) and the weight of the girders. Knowledge of the stress distribution within the abutment helps explain performance since deformations depend on stresses and vice versa. For Rustic Road, six earth pressure cells were installed in the north abutment backfill as shown in Figure 47. The two sensors installed near the bottom of the GRS-IBS backfill (EPC-1 and EPC-2) should measure the stress resulting from the total weight of the abutment and bridge girders. The four sensors installed in the bridge seat (EPC-3 through EPC-6) are intended to measure the load from the bridge girders. Of particular interest is the response of EPC-3 through EPC-6 during flood events that produce buoyancy forces on the bridge girders.



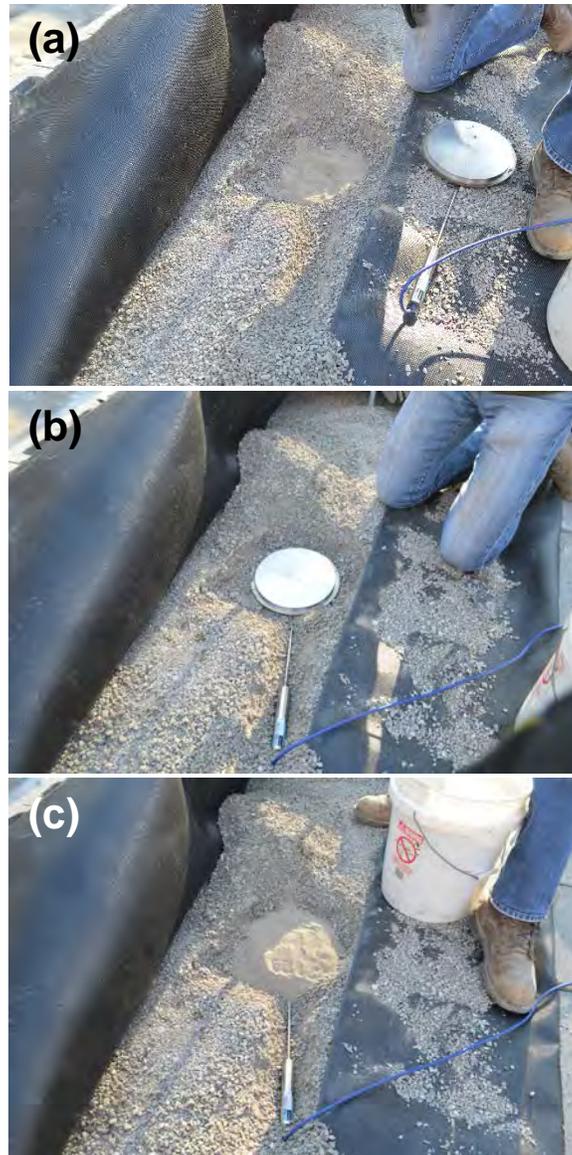
**Figure 47: Earth pressure cell locations: (a) plan and (b) elevation. Six cells were installed in the north abutment, two near the bottom and four near the top beneath the center of each girder.**

Vibrating-wire earth pressure cells were used. A photograph of one of the instruments is shown in Figure 48. The instruments consist of two circular stainless steel plates welded together with a thin space between the plates that is filled with hydraulic oil (Geokon, 2011). Theoretically, the pressure of the hydraulic fluid is equal to the pressure applied by the soil in contact with the plates. In practice, the pressure applied by the soil is typically somewhat different from the actual total stress within the soil because the stiffness of the pressure cells is not equal to the stiffness of the soil, resulting in a redistribution of load. If the cells are stiffer than the soil, the earth pressure indicated by the cells will be greater than the actual earth pressure (i.e. the earth pressure that would be experienced if the cells were not present). If the cells are less stiff than the soil, the earth pressure indicated by the cells will be less than the actual earth pressure. Note that the “earth pressure” measured by the instruments is equivalent to “total stress” in conventional geotechnical parlance. To determine effective stress, the pore pressure at the cell location must be subtracted from the total stress.



**Figure 48: Installation of EPC-1 near the bottom of the north abutment.**

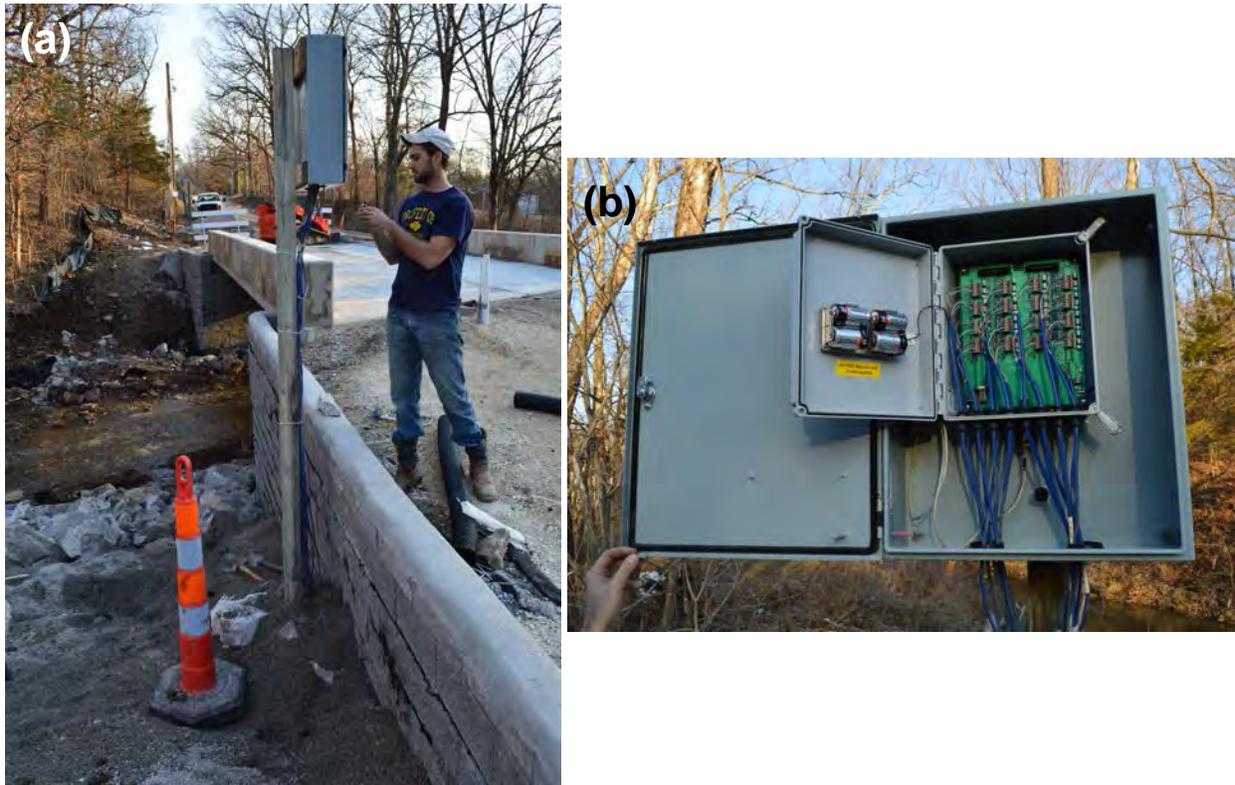
Earth pressure measurement difficulties associated with stiffness contrasts can be exacerbated by installation issues. The goal of the installation is to achieve a uniform stress distribution across the plates and above and below the cell. To achieve a uniform distribution, the instruments were installed horizontally and with a thin layer of fine sand above and below the cells as shown in Figure 49. The sand prevents the uneven distribution that would result from angular pieces of gravel directly in contact with the cells. The sand layers were relatively thin to prevent large changes in backfill density above and below the cell. Installation of EPC-3 through EPC-6 was complicated by GRS-IBS details for the beam seat, which include two 4-in. thick layers of backfill wrapped in geotextile as shown in the project plans included as an appendix to this report. EPC-3 through EPC-6 were installed in the bottom 4-in. thick layer (Figure 49).



**Figure 49: Installation of EPC-3 in bottom layer of bridge seat: (a) thin layer of fine sand is placed across bottom of hole for earth pressure cell, (b) earth pressure cell is placed atop fine sand, and (c) another layer of fine sand is placed atop earth pressure cell.**

For each earth pressure cell, a vibrating wire pressure transducer measures the pressure within the hydraulic fluid. In addition, a thermistor records temperature within the cell. Additional information regarding the earth pressure cells is included in the product instruction manual (Geokon, 2011).

Measurements from the cells were read and recorded by a data logger on site as shown in Figure 50. The logger was set to record measurements every 2 hours; the research team collected data during each site visit. The logger also recorded measurements from the pore pressure instruments described in the next section. Additional details regarding the logger are included in its instruction manual (Geokon, 2013b). The logger was housed in a National Electrical Manufacturer Association (NEMA) enclosure, which was installed on a post just east of the north abutment. The enclosure was installed so that the bottom of the enclosure was above the 100-year flood elevation for the project site.



**Figure 50: NEMA enclosure housing datalogger: (a) post installed just east of the north abutment, with bottom of enclosure above 100-year flood level and (b) front view of the NEMA enclosure and datalogger. Each blue cable carries signals from one vibrating wire device.**

### 3.4 Pore Pressure within GRS Backfill

One of the most important performance measures for any retaining wall system is how quickly the backfill drains. If the backfill is not freely draining, water pressure will develop on the face of the retaining wall, and pore pressures will reduce backfill shear strength. Measurement of pore pressures, and especially the response of pore pressure with time to water infiltration, is therefore an important indicator of abutment performance. Pore pressure measurement also provides information regarding the state of stress within the abutment backfill, complementing the earth pressure information described in the previous section and facilitating calculations of effective stress.

For Rustic Road, ten vibrating wire piezometers were installed to measure pore pressure within the north abutment backfill. The piezometers were distributed as shown in Figure 51 to obtain a representative sampling of the pore pressures throughout the abutment. A photograph of a piezometer during installation is shown in Figure 52. The piezometers were installed inside sand pockets with the GRS backfill to stabilize the pore pressure measurements.

The piezometer measuring device, a diaphragm that responds to changes in pore pressure within the backfill, is located inside the stainless steel housing (Geokon, 2013a). A filter stone is located on one end

of the housing. Signals from the vibrating wire element and an internal thermistor are transmitted via a cable exiting the opposite end of the housing. Measurements from the cells were read and recorded by the same data logger that was used to record earth pressure cell data (Figure 50). The logger was set to record measurements every 2 hours; the research team collected data during each site visit.

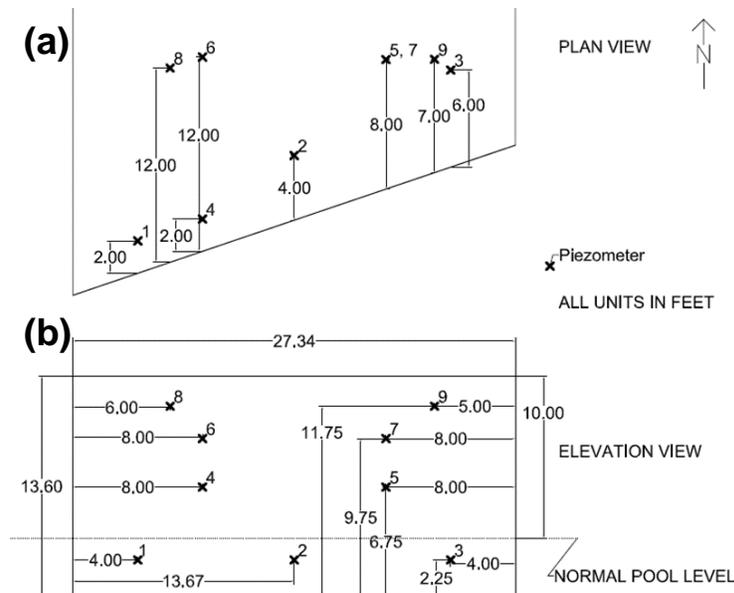


Figure 51: Piezometer locations within north abutment: (a) plan and (b) elevation.



Figure 52: Piezometer PZ-1: (a) installation in sand pocket within GRS backfill and (b) close view of vibrating wire piezometer.

### 3.5 Summary of Monitoring System

Table 1 is a summary of the monitoring system components used to measure the performance of Rustic Road GRS-IBS as described in this chapter. External movement, scour, internal movement, earth pressure, and pore pressure were measured by a combination of land surveying, visual observation, and electronic instrumentation. Monitoring was completed for a period of 19 months after the end of construction, with site visits every month during the first 12 months and every other month thereafter.

**Table 1: Summary of monitoring system performance metrics and corresponding monitoring system component details. Monitoring period was approximately 19 months.**

<b>Performance Metric</b>	<b>Monitoring System Component</b>	<b>Component Location(s)</b>	<b>Monitoring Frequency</b>
External movement	Land Surveying (by City of Columbia)	<ul style="list-style-type: none"> <li>• 12 reflective marker on face of each abutment (Figure 37)</li> <li>• 4 corners of bridge</li> </ul>	Quarterly throughout monitoring period.
	Crack gages	Top of each wing wall (Figure 38)	
	Visual observation	Entire project site	
Scour	Visual observation	<ul style="list-style-type: none"> <li>• Rip rap in creek bed and on side slopes</li> <li>• Red CMU blocks below grade on each wall face</li> </ul>	Monthly for first 12 months and every other month thereafter
Internal movement	Settlement plates (vertical displacement)	3 plates in a vertical line: bottom, middle, and top of the north abutment backfill (Figure 41)	Quarterly throughout monitoring period.
	Inclinometer (horizontal displacement)	One casing per abutment (Figure 46)	Monthly for first 12 months and every other month thereafter
Earth pressure	Vibrating wire earth pressure cells	As shown in Figure 47: <ul style="list-style-type: none"> <li>• Two cells near bottom of north abutment</li> <li>• 4 cells near top of north abutment (1 per girder)</li> </ul>	Data logger recorded measurements every 2 hours; data collected during every site visit
Pore pressure	Vibrating wire piezometers	10 piezometers distributed throughout the north abutment (Figure 51)	

## 4. Monitoring and Performance

Monitoring results are presented in this chapter. An overview of the monitoring activities is presented first, with subsequent sections each presenting results from a specific monitoring system component. The observed performance of Rustic Road GRS-IBS is summarized at the end of the chapter.

### 4.1 Summary of Monitoring Activities

Details of each monitoring system component were presented in Chapter 3 and summarized in Table 1. During the course of the 19-month monitoring period (March 2015 through September 2016), 15 site visits were conducted, one per month for the first year and every other month thereafter. For each site visit, the research team documented visual observations with notes and photographs, recorded crack gage data, performed inclinometer readings, and collected data that had been logged for the vibrating wire piezometer and earth pressure cells. In addition to research team site visits, the City of Columbia survey crew visited the site every three months during the monitoring period and surveyed the abutment face targets, corners of the bridge, and settlement plate rods.

### 4.2 Visual Observations and Crack Gages

Visual observations were documented with each site visit. Most of the observations were consistent with a bridge performing well in its early service life; “no apparent change since last site visit” was a common note. However, three sets of observations are noteworthy: (1) a high water event in early July 2015, (2) potential shifting of the scour protection at the downstream (west) corner of the south abutment, and (3) cracks that developed at the top of all four wing walls. Each set of observations is discussed below.

In late June and early July 2015, a series of rain events in Boone County led to a significant water level increase in the North Fork of Grindstone Creek as shown in Figure 53. During the site visit when the photograph was taken several days after rain had stopped, the creek level was observed near the top of the rip rap, approximately 3 ft above the normal level. The normal creek level is just above the reinforced soil foundation as shown in figures throughout Chapter 2. Water marks on the CMU blocks shown in Figure 53 indicate the creek level had been several feet higher than its level at the time of the photograph. The marks indicate the maximum water level was just below the abutment mid-height.



Figure 53: Rustic Road GRS-IBS during July 2015 high water event: (a) looking upstream from atop the bridge, showing creek high up its banks and (b) looking at north abutment.

During a site visit in August 2015, one month after the high water event, a gap was observed between the downstream (west) corner of the south abutment and the rip rap scour protection as shown in Figure 54(a). It is possible the gap formed in response to the high water event; it is also possible the gap was present from the first placement of the scour protection in May 2015 since that portion of the scour protection was underwater during June and July 2015 site visits. Regardless, observations during subsequent site visits as shown in Figure 54(b) and Figure 54(c) have not indicated any growth in the size of the gap nor any significant shifting in the rip rap.



**Figure 54: Photographs of downstream corner of south abutment: (a) August 31, 2015, (b) February 25, 2016, and (c) September 22, 2016. A gap between the abutment face and rip rap scour protection was first observed during the August 31, 2015 site visit, but no changes in the gap were observed throughout the course of subsequent monitoring.**

Shortly after construction, cracks were observed in CMU blocks at the top of each wing wall. All four cracks developed approximately 10 ft back from each abutment corner as shown previously in Figure 38, which indicates the location of crack gages that were installed to monitor the cracks. The cracks extend from the top of the wing wall down two or three rows of CMU blocks as shown in Figure 55. The location of the cracks corresponds to the back end of the beam seat as shown in Figure 56. It is possible the cracks are related to the stiffness contrast between the beam seat and the surrounding GRS backfill. It is also possible the cracks are a result of differential settlement associated with the front of the wall bearing directly on rock while the sloping backfill rests on soil, but the constructed cut slope was steeper than 1:1. As described in the rest of this chapter, the survey data indicate little external movement, and the settlement plate and inclinometer data indicate little internal movement. Crack gage data are shown in Figure 57. The data indicate little to no movement since the crack gages were placed: cracks on the east wing walls of both abutments have spread approximately 1 mm (0.04 in.), while the cracks on the west wing walls of both abutments have spread approximately 2.5 mm (0.1 in.)



**Figure 55: Crack on east wing of north abutment wall.**

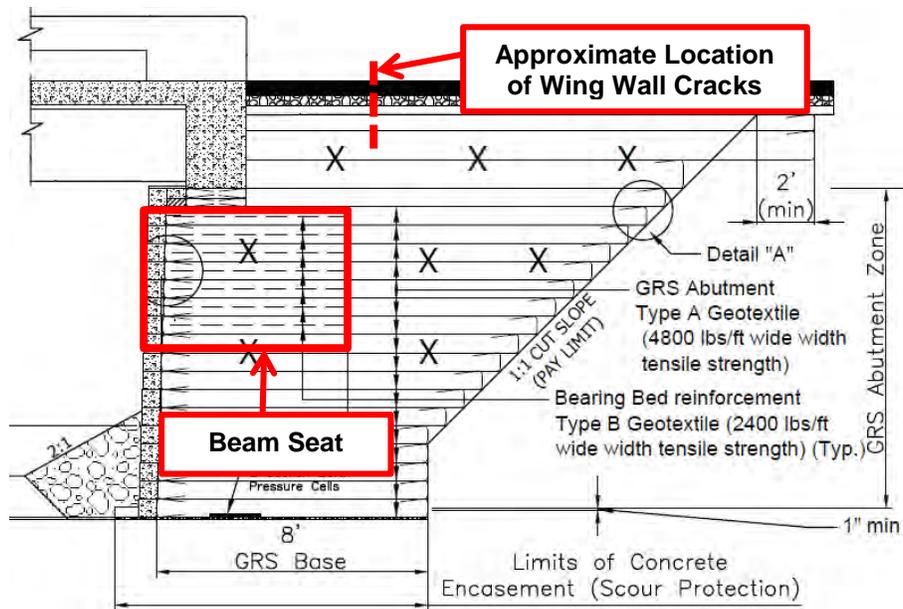


Figure 56: Crack location relative to bearing bed. Drawing is from Rustic Road plans included as an appendix to this report; annotations are original. As-constructed cut slope was steeper than 1:1.

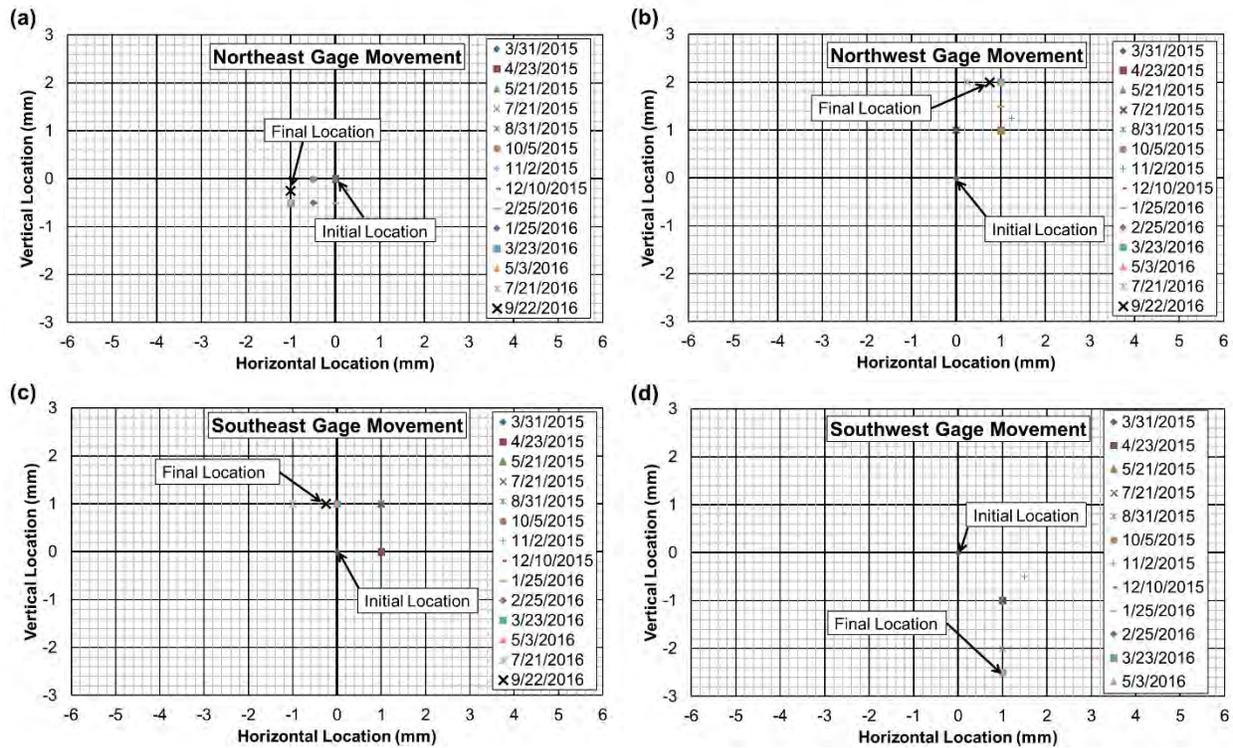


Figure 57: Crack gage data for each wing wall: (a) northeast, (b) northwest, (c) southeast, and (d) southwest. For each gage, initial (installed) and final (September 22, 2016) locations of the gage crosshair is shown.

### 4.3 Survey Results

Results of surveying are plotted in Figure 58 and Figure 59. Figure 58 shows settlement (vertical), with positive settlement indicating downward movement, and Figure 59 shows lateral (horizontal) movement. For each figure, there are eight plots, with all left-side plots representing north abutment data and all right-side plots representing south abutment data. For each figure, the top two plots represent data from survey points near the surface of the bridge, including the four corners of the bridge deck as well as the inclinometer casing. The second two plots represent data from the top row of survey markers installed on the wall face (Figure 37), the third two plots represent data from the middle rows of survey markers, and the bottom two plots represent data from the bottom row of markers.

Lateral movement values were calculated as the Pythagorean sum of the changes in northings and eastings between the two surveys:

$$\text{Lateral Movement} = \sqrt{(x - x_0)^2 + (y - y_0)^2}, \text{ where}$$

$x$  = northing

$x_0$  = initial northing

$y$  = easting

$y_0$  = initial easting

Internal movement measurements are included in the figures for the sake of comparison. Settlement data from settlement plate surveys are included in Figure 58 for the north abutment plots. Similarly, lateral movement data from inclinometer probe readings are included in Figure 59 to facilitate comparisons between survey data and inclinometer data. Settlement plate data is discussed further in Section 4.4 below, and inclinometer data is discussed further in Section 4.5 below.

The results of Figure 58 and Figure 59 indicate vertical and lateral movement of both abutment wall faces was negligible during the monitoring period. The results indicate up to 1.5 in. of vertical movement and up to 5 in. of lateral movement for points near the ground surface, but the ground surface data is highly variable and contradicted by results from internal movement devices (i.e. the top settlement plate and inclinometer probe results near the top of casing), which show negligible movement. It is reasonable to conclude Rustic Road GRS-IBS has not experienced significant external movement. The survey results are discussed in greater detail below.

For both abutments, settlement was less than 0.25 in. for all survey markers on the abutment faces while survey results for points near the ground surface indicate about 1 in. of settlement. However, the survey data for surface points varies considerably with time, especially compared to the survey markers on the face of the abutments. It is possible the surface of the bridge did indeed settle about 1 in., but it is also possible 1 in. is within the accuracy of the surface measurements since the bridge corners were not marked with permanent survey markers. The latter possibility is supported by the results of settlement plates, which settled only 0.25 in. If the surface had in fact settled 1 in., additional settlement would be expected for at least the top settlement plate.

A similar trend is noted in the lateral movement results: variable results at the ground surface indicating significantly more movement than was observed for the abutment face markers. For both abutments, lateral movement indicated for survey targets installed on the abutment faces was less than 0.5 in. for all surveys. However, for both abutments, survey results for points near the ground surface indicate as much as 5 in. of lateral displacement, although values were generally between 1.0 and 1.5 in. Variability of the surface data is considerable, and even greater than indicated by the top two plots of Figure 59 because of the definition of lateral movement provided above. For instance, the results for the northwest corner of the bridge indicate the corner moved about 1 in. northwest before moving back southeast, 5 in. past the point where it started. This is likely a result of the survey accuracy for surface points as discussed above, especially since the inclinometer probe data near the surface for both abutments indicated less than 0.2 in. of movement.

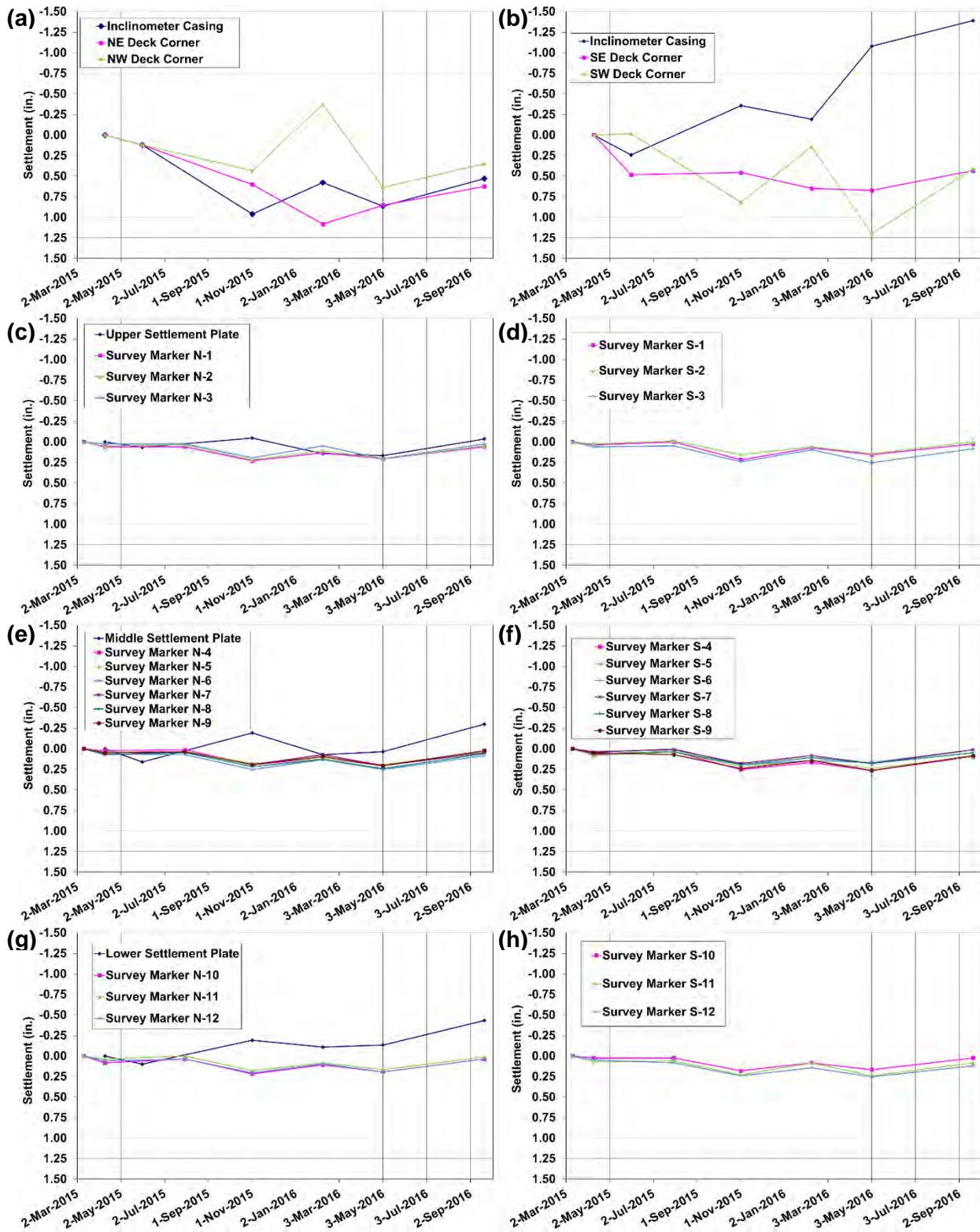


Figure 58: Settlement from survey data: (a) surface of north abutment, (b) surface of south abutment, (c) upper markers of north abutment, (d) upper markers of south abutment, (e) middle markers of north abutment, (f) middle markers of south abutment, (g) lower markers of north abutment, and (h) lower markers of south abutment. Positive settlement is movement downward.

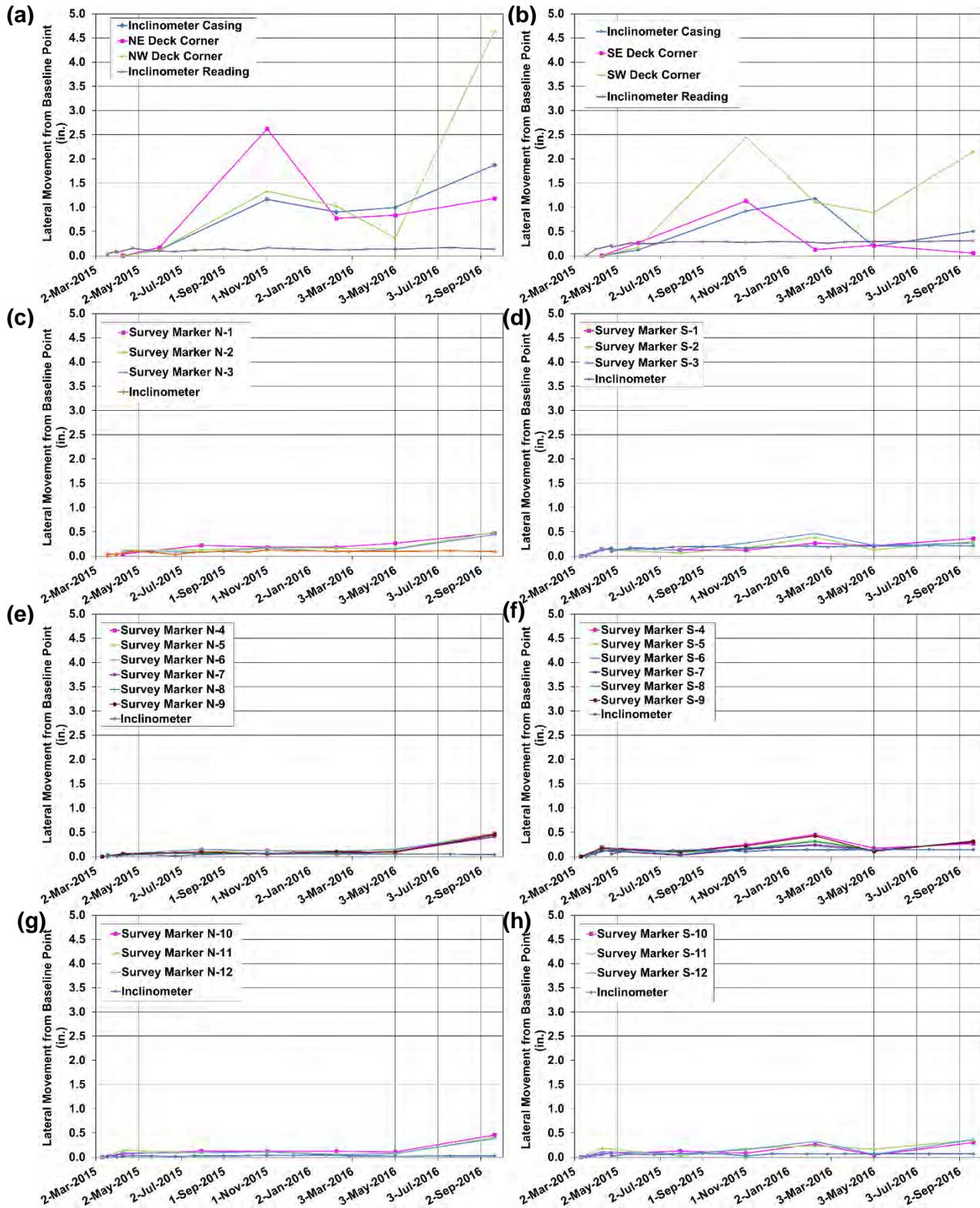


Figure 59: Lateral movement from survey data: (a) surface of north abutment, (b) surface of south abutment, (c) upper markers of north abutment, (d) upper markers of south abutment, (e) middle markers of north abutment, (f) middle markers of south abutment, (g) lower markers of north abutment, and (h) lower markers of south abutment.

#### 4.4 Settlement Plate Results

Figure 60 is a plot of settlement plate results based on surveying the settlement plate rods. The data fluctuate between 0.25 in. of settlement and 0.25 in. of upward movement. Based on these results, it is reasonable to conclude there is negligible settlement within the GRS backfill and 0.25 in. is the approximate accuracy of surveying the settlement plate rods. These conclusions are supported not only by the magnitude of settlement values, but also by the lack of any trend toward movement in one direction and by inconsistencies between the different plates. For instance, the middle plate indicating upward movement while the bottom plate indicates settlement, and the upper settlement plate moving less than either of the other plates.

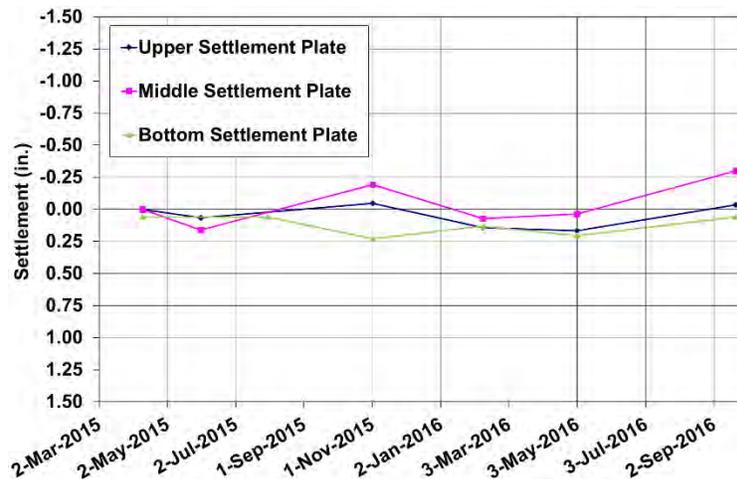


Figure 60: Settlement of settlement plates, as determined by survey results.

#### 4.5 Inclinator Results

Internal lateral deflection results interpreted from the inclinometer probe for the north and south abutments are shown in Figure 61 and Figure 62, respectively. (Information regarding the installation and operational theory of inclinometers was presented in Section 3.2.) The top two plots for each figure show the absolute shape of the inclinometer casing, with the left plot reflecting the shape in a plane perpendicular to the creek (i.e. parallel to the centerline of the roadway) and the right plots reflecting the shape in a plane parallel to the creek (i.e. perpendicular to the centerline of the roadway). The bottom two plots for each figure show the change in inclinometer casing shape relative to the initial reading, using the same directions as for the top two plots. Each data series in the bottom two plots therefore represents the difference between the corresponding data series in the top plot and the initial data series in the top plot.

The results of Figure 61 and Figure 62 indicate the lateral deflection that occurred during the monitoring period was negligible. For the north abutment, the installed casing shape was slightly curved, with the top of casing about 1 in. from vertical. The casing shape did not change significantly throughout the monitoring period; the greatest change in profile occurred at the top of the casing and was less than 0.2 in. Similar results were obtained for the south abutment. The installed casing shape was nearly vertical, with the top of casing less than 0.5 in from vertical. The greatest change in profile for the south abutment casing also occurred at the top of casing and was less than 0.4 in.

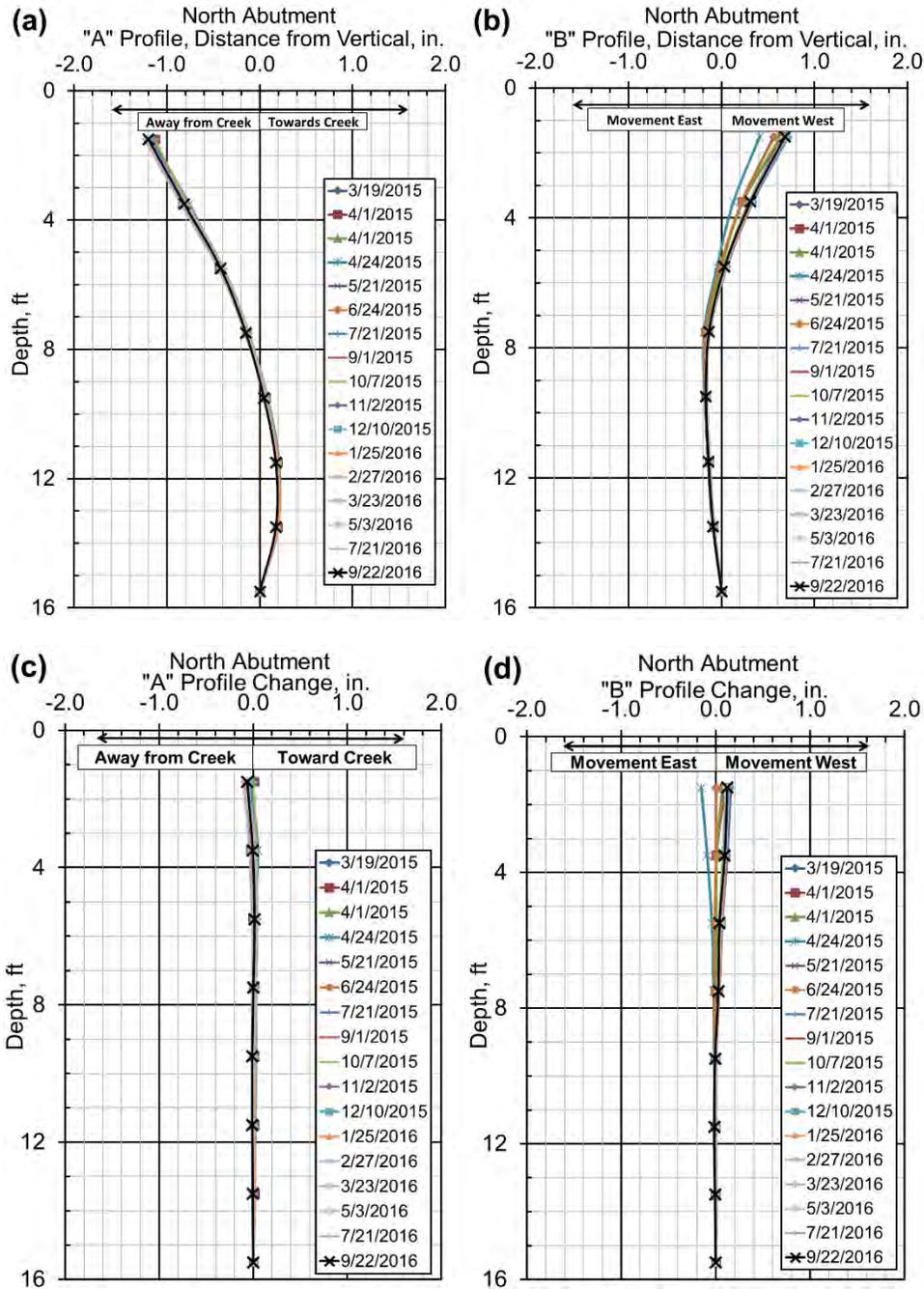
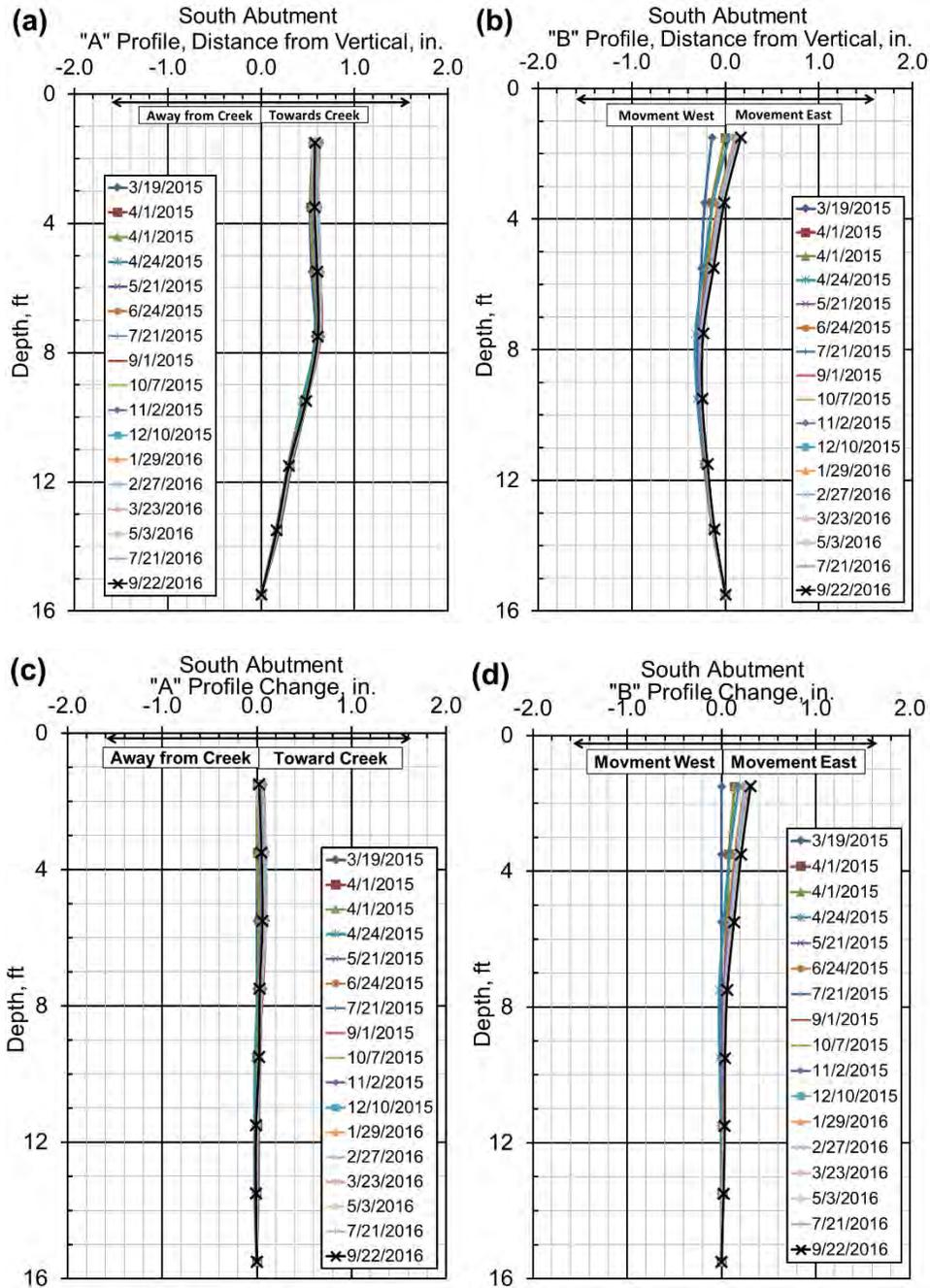


Figure 61: Inclinometer results for north abutment: (a) casing profile in direction toward creek, (b) casing profile in east-west direction, (c) change in casing profile in creek direction, and (d) change in casing profile in east-west direction. The data in plots (a) and (b) represent the absolute shape of the casing, and the curvature is dominated by the installed shape of the casing. The data in plots (c) and (d) represent the relative change in casing shape since installation and indicates negligible movement.

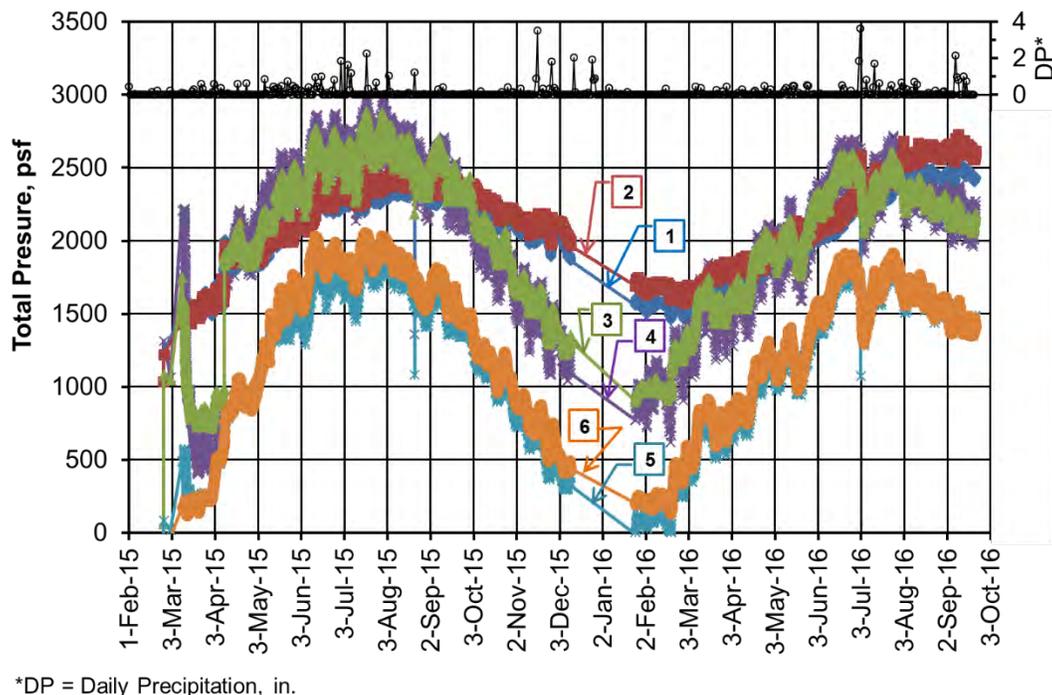


**Figure 62: Inclinometer results for south abutment: (a) casing profile in direction toward creek, (b) casing profile in east-west direction, (c) change in casing profile in creek direction, and (d) change in casing profile in east-west direction. The data in plots (a) and (b) represent the absolute shape of the casing, and the curvature is dominated by the installed shape of the casing. The data in plots (c) and (d) represent the relative change in casing shape since installation and indicates negligible movement.**

#### 4.6 Earth Pressure Results

Total stresses from the earth pressure cells throughout the monitoring period are plotted in Figure 63. Perhaps most evident from the time-series data is a cyclical trend for each earth pressure cell. The pressure cycle appears to have a time period of one year. The cause for the cyclical trend is most likely temperature, which follows a similar trend. Figure 64 shows the total stress measured by one of the instruments, EPC-5, versus the temperature measured by the thermistor in the same instrument. Indeed, there is a strong correlation between temperature and measured total stress. It is not clear whether the temperature effect is real, perhaps due to thermal expansion of the girders, which are constrained by the anchor bolts, or simply an internal effect associated with the earth pressure cell devices. Although a manufacturer-supplied temperature correction factor has been applied to the results shown in Figure 63, the factor only accounts for the effect of temperature on the cell itself, not the effect of temperature on the installed system that includes not only the cell but also the surrounding compacted soil (Geokon, 2011). In a study for the National Research Council of Canada, Daigle (2003) found that manufacturer temperature effect factors “largely underestimated the temperature effect.”

The effect of temperature can be removed from the results of Figure 63 by using the calculated slopes of the pressure-temperature lines (Figure 64) and the measured temperatures with time. Implementing such math assumes that the response shown in Figure 64 is strictly the effect of temperature on the installed earth pressure cell systems rather than any actual load increase due to temperature. One could argue such an assumption is unwise. Nevertheless, the results after removing the effect of temperature are presented in Figure 65. The results with (Figure 63) and without (Figure 65) temperature effects are useful for bounding the effect of temperature. It is also noteworthy that after removing temperature effects, the observed total stresses from all six cells are all relatively constant during the monitoring period, except for a gradual decrease in EPC-3 and EPC-4 during the last six months of monitoring.



**Figure 63: Earth pressures (total stresses) in north abutment from vibrating wire earth pressure cells. Cells EPC-1 and EPC-2 were installed near the bottom of the abutment backfill; each of the other cells was installed beneath one of the bridge girders (Figure 47). Daily precipitation records from nearby weather stations were averaged, and the result is shown atop the graph.**

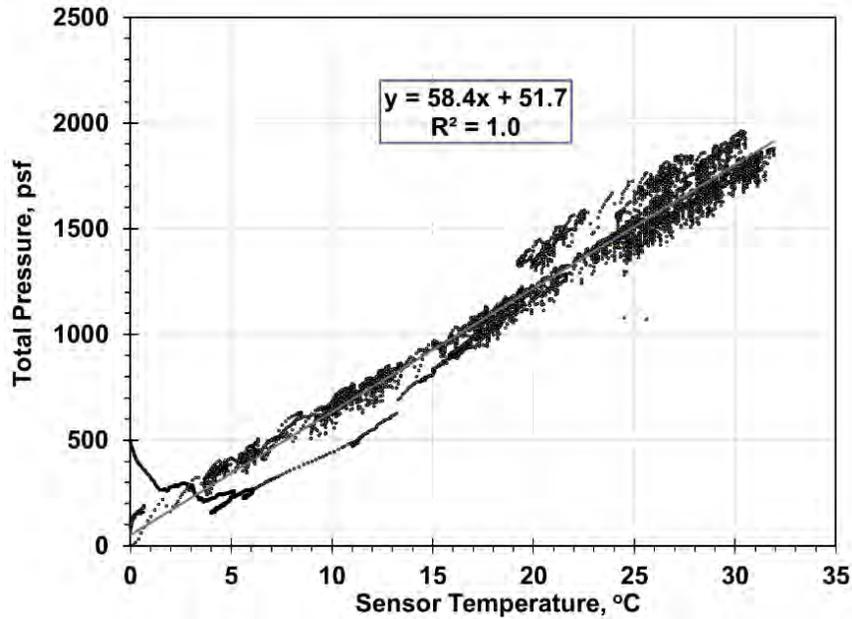
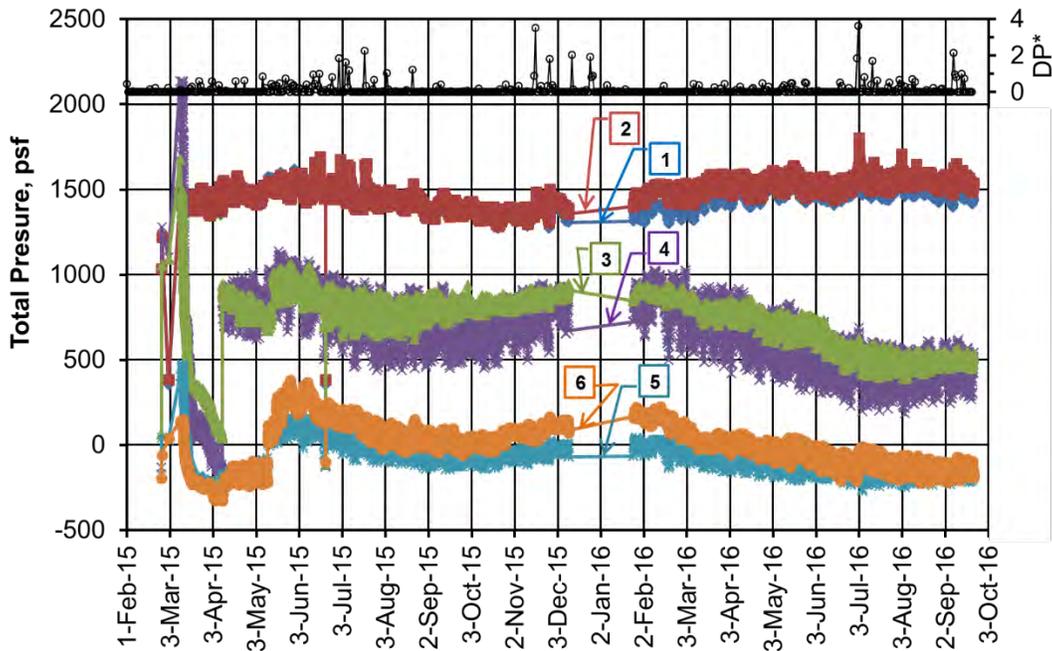


Figure 64: Earth pressure versus sensor temperature for EPC-5. Sensor temperature is measured by a thermistor housed inside the instrument.



\*DP = Daily Precipitation, in.

Figure 65: Earth pressure results after correcting for the effect of temperature using the observed pressure-temperature slopes like the one shown in Figure 64.

Other observations regarding the earth pressure data are noteworthy:

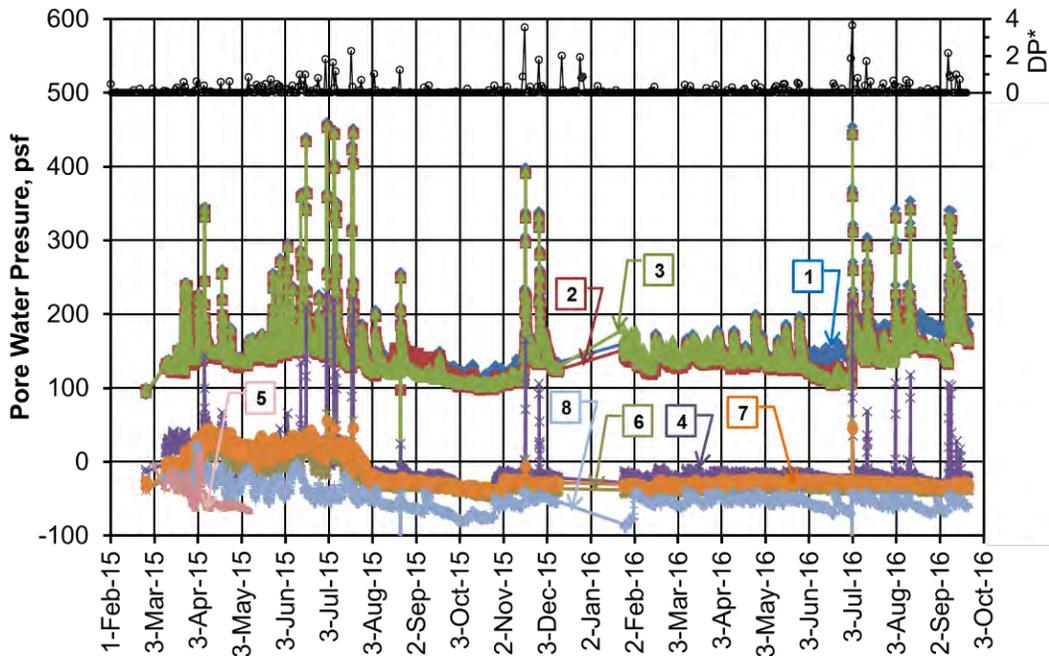
- Earth pressure measurements were not strongly influenced by precipitation events. As noted in the visual observations section above, the creek level was never high enough to result in buoyancy forces on the bridge girders.

- Measurements from EPC-3 and EPC-4 experienced two sudden increases and one sudden decrease in the first two months of operation. It is difficult to explain the cause of the sudden changes, but the net effect is likely less significant than one might assume based on inspection of the magnitude of the individual changes. The stresses in EPC-3 and EPC-4 are approximately 1000 psf greater than those in EPC-5 and EPC-6 both before and after the changes.
- EPC-3 through EPC-6 are each loaded by the weight of half of one of the bridge girders, which corresponds to a stress of approximately 1200 psf. After correcting for temperature effects, the observed stresses in EPC-3 through EPC-6 were all lower than the anticipated stress from the weight of the girders, although EPC-5 and EPC-6 measured stresses of approximately 800 psf.
- After accounting for the effect of temperature, the pressure recorded in EPC-1 and EPC-2 was greater than the pressure recorded in EPC-3 through EPC-6. This is consistent with the anticipated stress profile within the abutment: EPC-1 and EPC-2 should be subjected to loading from the weight of the girders as well as the weight of the overlying GRS abutment. Estimating the load from the girders to EPC-1 and EPC-2 is difficult because of the stress distribution within the GRS backfill. The stress due to the weight of backfill alone (ignoring girder weight) is approximately 1500 psf, assuming the backfill weighs 125 lb/ft<sup>3</sup>. EPC-1 and EPC-2 both measured approximately 1500 psf. Measurements for EPC-1 and EPC-2 are therefore lower than anticipated (since the girder weight would result in stresses greater than 1500 psf). This is similar to the observation of low measured stresses for EPC-3 through EPC-6.
- The cyclical pressure trend is less pronounced for EPC-1 and EPC-2 than for EPC-3 through EPC-6. This is consistent with the temperature effect hypothesis: EPC-1 and EPC-2 are at greater depths and therefore more insulated from surface temperature fluctuations.

#### 4.7 Piezometer Results

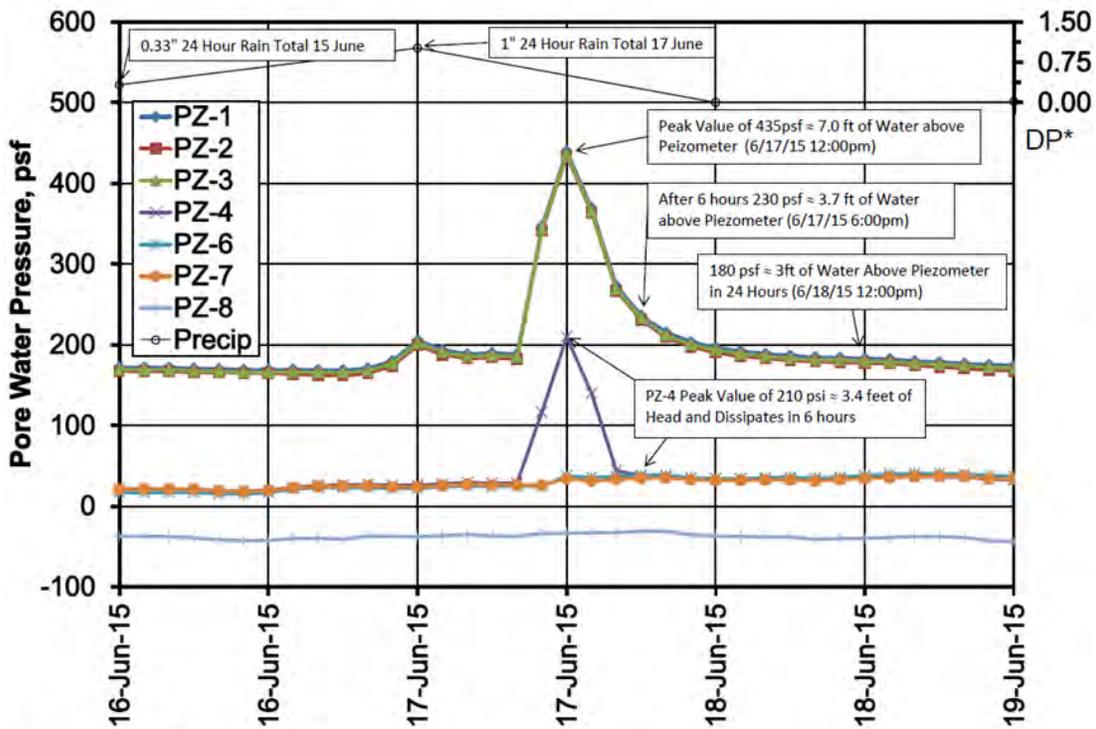
Pore water pressure results from the vibrating wire piezometers are plotted in Figure 66. The signal for one of the piezometers, PZ-5, became unstable about two months after the end of construction. The observed pore pressures are mostly consistent with time, but several locations show peaks that appear to be in response to precipitation events. The peaks dissipate quickly, which indicates the GRS backfill is freely draining, as designed. To examine the drainage more quickly, pore pressure data during one precipitation event is plotted in Figure 67. Indeed, the pore pressures generated in response to the precipitation event dissipate within six hours of the event.

The measured pore pressures appear to mostly be a function of the vertical location of the piezometer within the backfill. (Locations of the piezometers were shown in Figure 51.) PZ-1 through PZ-3 are located near the bottom of the abutment, approximately at the normal creek water elevation. These instruments typically recorded pore pressures of approximately 100 psf, a relatively small value corresponding to about 1.6 ft of water. The other piezometers were located above the normal creek water level and, as expected, recorded pore pressures around zero. The responses to rain events were also influenced by instrument height, with PZ-1 through PZ-3 showing strong responses to precipitation events, PZ-4 (6 ft above the bottom of the abutment) showing less strong and less frequent responses, and PZ-7 (9 ft above the bottom of the abutment) responding slightly to one precipitation event.



\*DP = Daily Precipitation, in.

Figure 66: Pore water pressures in north abutment from vibrating wire piezometers during monitoring period. The signal for PZ-5 became unstable approximately two months after the end of construction. For instrument locations, refer to Figure 51. Daily precipitation records from nearby weather stations were averaged, and the result is shown atop the graph.



\*DP = Daily Precipitation, in.

Figure 67: Close examination of change in pore pressures during one precipitation event. The signal for PZ-5 became unstable approximately two months after the end of construction.

#### **4.8 Summary of Observed Performance**

Monitoring activities during the 19-month monitoring period included visual observations and land surveying as well as measurements from settlement plates, inclinometers, earth pressure cells, and piezometers. Visual observations included documentation of a significant high water event, some potential shifting of scour protection that was deemed minor, and cracks that developed at the top of each wing wall. The cracks were monitored with crack gages, which all indicated very limited further movement after the initial observation. Land surveying results indicate external movement of the survey targets on the abutment wall faces was negligible during the monitoring period. Some movement was recorded for survey points on the surface of the bridge, but the movement values are likely a result of survey accuracy for the surface points, which were not established with permanent markers like the targets on the wall face. Internal movement of the GRS backfill was also negligible, as measured via settlement plates and inclinometers. The response of earth pressure cells appears to be dominated by temperature effects, but the measured pressures are otherwise reasonable and were not strongly influenced by precipitation events. Piezometers indicated pore pressures near the bottom of the GRS backfill spiked during precipitation events, but dissipated within six hours, indicating the backfill is freely draining as designed.

## 5. Conclusions

The predominant conclusion is that Rustic Road GRS-IBS is performing as intended: external and internal displacements are negligible, and the backfill is typically dry and drains freely after precipitation events. Cracking was observed atop the wing walls shortly after construction, but the cracks have not expanded in the ensuing 19 months. Other observations and recommendations from the monitoring of Rustic Road GRS-IBS include:

- Survey, settlement plate, and inclinometer measurements generally indicated negligible movement. The only exceptions are results from some surveys of some points near the surface of the bridge, but results for those points were highly variable and typically inconsistent with nearby measurements. Permanent survey targets should be used for all survey points to improve the accuracy and repeatability of survey results.
- The settlement plate design appears to have functioned as intended.
- Settlement plates and inclinometers are effective methods for measuring internal displacements when the devices are installed properly.
- Earth pressure cell measurements appear to have been strongly influenced by the temperature within the GRS backfill. After removing the effects of temperature, the measured stresses were relatively constant with time and relatively consistent with respect to location within the backfill; however, all of the measurements were somewhat lower than anticipated.
- Vibrating wire piezometers were an effective method of measuring pore pressures within the GRS backfill. Measurements indicated the backfill drains quickly in response to precipitation events.

## References

- Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W., and Hartmann, J. (2012). *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*. U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-HRT-11-026, Washington D.C., USA.
- Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W., and Hartmann, J. (2011). *Geosynthetic Reinforced Soil Integrated Bridge System, Synthesis Report*. U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-HRT-11-027, Washington D.C., USA.
- Daigle, L. and J.Q. Zhao (2003), *Assessing Temperature Effects on Earth Pressure Cells*, Report to the National Research Council of Canada Institute for Research in Construction, IRC-RR-131.
- Geokon (2013a), *Instruction Manual: Model 4580 Vibrating Wire Pressure Transducer*, Document Rev. N, June 2013.
- Geokon (2011), *Instruction Manual: Models 4800, 4810, 4815, 4820, and 4830 Vibrating Wire Earth Pressure Cells*, Document Rev. M, September 2011.
- Geokon (2013b), *Instruction Manual: Model LC-2x16 16 Channel Vibrating Wire Datalogger*, Document Rev. N, October 2013.
- Slope Indicator (2011), *Digitilt Inclinator Probe 50302599 Manual*, November 2011.
- Wu, J.T.H., T.Q. Pham, and M.T. Adams (2013), *Composite Behavior of Geosynthetic Reinforced Soil Mass*, U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-HRT-10-077, Washington D.C., USA.

## **Appendix – Bridge Plans for Rustic Road GRS-IBS**

# BOONE COUNTY, MISSOURI

## RUSTIC ROAD BRIDGE REPLACEMENT

### FEDERAL PROJECT NO. IBRD 9900(592)

BOONE COUNTY COMMISSION

Daniel K. Atwill

Janet M. Thompson

Karen Miller

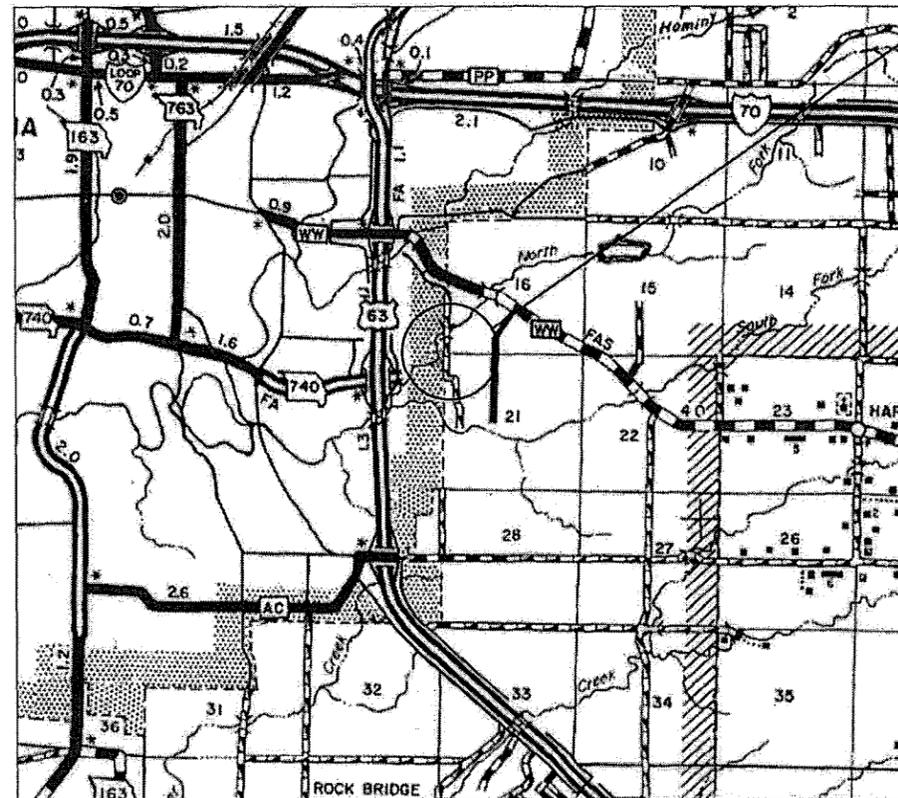
CHIEF ENGINEER OF RESOURCE MANAGEMENT

APPROVED:

*Derin Campbell*  
Derin Campbell, P.E.

7/23/14  
Date

LOCATION PLAN



PROJECT LOCATION

SECTION 16  
TOWNSHIP 48 NORTH  
RANGE 12 WEST



KEY MAP

#### DESIGN DESIGNATION

ADT <400  
CURRENT RUNNING SPEED 30 mph  
PROPOSED DESIGN SPEED 20 mph  
CART ROAD NUMBER 331  
FUNCTIONAL CLASSIFICATION Rural Local

#### LENGTH OF PROJECT

BEGIN STATION 10+65 ±  
END STATION 13+00 ±  
PROJECT LENGTH 0.045 miles

#### LEGEND

(USED IN PLANS)

LOCATION SURVEY MARKER	
UTILITIES	
UTILITY POLE	
UNDERGROUND TELEPHONE	
OVERHEAD POWER	
WATER	
WATER VALVE	
GROUND MOUNTED SIGN	
TELEPHONE PEDESTAL	
FENCE	
BENCHMARK	

#### UTILITIES

PUBLIC WATER SUPPLY DISTRICT NO. 9  
OF BOONE COUNTY  
391 NORTH RANGELINE ROAD  
COLUMBIA, MO 65201  
PH. 573-474-9521

COLUMBIA POWER AND LIGHT  
701 E. BROADWAY  
COLUMBIA, MO 65201

BOONE COUNTY SEWER  
1314 NORTH SEVENTH STREET  
COLUMBIA, MO 65201  
573-443-2774

CENTURY LINK  
625 CHERRY STREET  
COLUMBIA, MO 65201  
PH. 573-886-3336



#### INDEX OF SHEETS

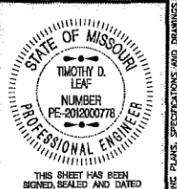
DESCRIPTION	SHEET NUMBER
TITLE SHEET	1
GENERAL NOTES AND QUANTITIES	2
ROADWAY DRAWINGS	
TYPICAL SECTIONS	3
RIGHT OF WAY AND EASEMENT PLAN	4
MAINTENANCE OF EXISTING TEMPORARY BYPASS PLAN	5
ROADWAY PLAN--PROFILE	6
ROADWAY PLAN AT BRIDGE	7
TRAFFIC CONTROL	8
EROSION CONTROL PLAN	9
CROSS SECTIONS	10-13
DRAWINGS FOR BRIDGE NO. 33100041	
ELEVATION AND PLAN	14
GEOSYNTHETIC REINFORCED SOIL SYSTEM	15
NORTH ABUTMENT PLAN AND ELEVATION	16
SOUTH ABUTMENT PLAN AND ELEVATION	17
DETAILS OF END BENTS AND DEADMAN ANCHORAGE	18
BRIDGE TYPICAL SECTIONS	19
CORRAL RAIL	20
GEOTECHNICAL DATA	
BORING LOG SHEET NOS. 1-2	21
BORING LOG SHEET NOS. 3-4	22

**BARTLETT & WEST**

1710 ROUTE 100, SUITE 100, JEFFERSON CITY, MO 65108-4000  
PHONE: 573-635-1111 FAX: 573-635-1112  
MISSOURI PROFESSIONAL ENGINEER LICENSE NO. 000187-ENGINEERING  
WWW.BARTLETTWEST.COM

TITLE SHEET

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(592)  
BOONE COUNTY, MISSOURI



THIS SHEET HAS BEEN  
SIGNED SEALED AND DATED  
ELECTRONICALLY.

STD 616.10	DESIGNED BY: TDL
STD 617.20	DRAWN BY: MSS
STD 706.35	APPROVED BY: RAG
STD 725.00	DESIGN PROJ: 16137.110
STD 806.10	SCALE: AS NOTED
STD 903.03	DATE: JULY/2014
	DRAWING NO: NONE
	SHEET NO: 1 of 22

FINAL PLANS  
JULY, 2014

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 Last edit on: 09/09/2014

Estimated Quantities (Roadway)			Final Quantities
** Mobilization	Lump Sum	1	
Contractor Furnished Surveying and Staking	Lump Sum	1	
Removal of Improvements	Lump Sum	1	
Clearing and Grubbing	Lump Sum	1	
Maintenance of Temporary Bypass	Lump Sum	1	
Removal of Temporary Bypass	Lump Sum	1	
Construction Signs	Square Foot	181.5	
Type III Movable Barricade	Each	2	
Type III Movable Barricade w/ Light	Each	2	
Type 5 Aggregate for Base 4" Thick	Square Yard	233	
8" Thick Asphalt Pavement	Square Yard	233	
4" Thick Gravel Pavement	Square Yard	100	
Silt Fence	Linear Foot	383	
Type I Ditch Check	Each	7	
Type II Ditch Check	Each	5	
Sediment Removal	Cubic Yard	10	
Perm Erosion Control Geotextile (Rip Rap)	Square Yards	214	
* Furnishing Type 2 Rock Blanket	Cubic Yard	72	
* Placing Type 2 Rock Blanket	Cubic Yard	72	
Type III Object Markers	Each	12	
*** Restoration	Lump Sum	1	

Estimated Quantities (Bridge)		
Removal of Bridges (3310004)	Lump Sum	1
Class 2 Excavation in Rock	Cubic Yard	34
Geosynthetic Reinforced Soil System (GRS)	Lump Sum	2
**** Pre-Engineered Superstructure	Square Foot	1284
Corral Rail	Linear Foot	107

GRS Abutment Open-Graded Backfill (AASHTO No. 89)		
	U.S. Sieve Size	Percent Passing
Gradation (AASHTO M-43)	1/2 inch	100
	3/8 inch	90-100
	No. 4	20-55
	No. 8	5-30
	No. 16	0-10
	No. 50	0-5
Plasticity Index (PI) (AASHTO T-90)	PI ≤ 6	
Soundness (AASHTO T-104)	The backfill shall be substantially free of shale or other poor durability particles. The material shall have a magnesium sulfate loss of less than 30 percent after four cycles (or a sodium value less than 15 percent after five cycles).	
Min. Internal Friction Angle	42°	

**General Notes:**

**Design Specifications:**  
 2002 - AASHTO 17th Edition (Seismic) Load Factor Design  
 Seismic Performance Category A  
 2011 - Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide FHWA-HRT-4-026.

**For Bridge Rating only:**  
 2012 - AASHTO LRFD 6th Edition and 2013 Interims  
 Load and Resistance Factor Design

**Design Loading:**  
 HS-20-44 (LFD Superstructure, LFD Substructure)  
 35#/Sq. Ft. Future Wearing Surface  
 Earth 120 #/Cu. Ft., Equivalent Fluid Pressure 45#/Cu. Ft.

**Design Unit Stresses:**  
 Class B Concrete (Substructure)  $f'_c = 3,000$  psi  
 Class B-1 Concrete (Corral Rail)  $f'_c = 4,000$  psi  
 Class A-1 Concrete (Superstructure, except Corral Rail)  $f'_c = 5,000$  psi  
 Reinforcing Steel (Grade 60)  $f_y = 60,000$  psi  
 Structural Steel (ASTM A709 Grade 36)  $f_y = 50,000$  psi

**Geotextile Fabric:**  
 - Minimum Tensile Strength = 4800 lb/ft  
 - Tensile Strength @ 2% Strain (see JSP's) = 960 lb/ft

**GRS Backfill Material:**  
 AASHTO No. 89 Clean, Crushed Angular Stone (See Table Below)

**Block Wall:**  
 For Details of blocks not shown in plans, see Job Special Provisions.  
 CMU= Concrete Masonry Unit

**Excavation:**  
 Comply with Occupational Safety and Health Administration (OSHA) for all excavations.

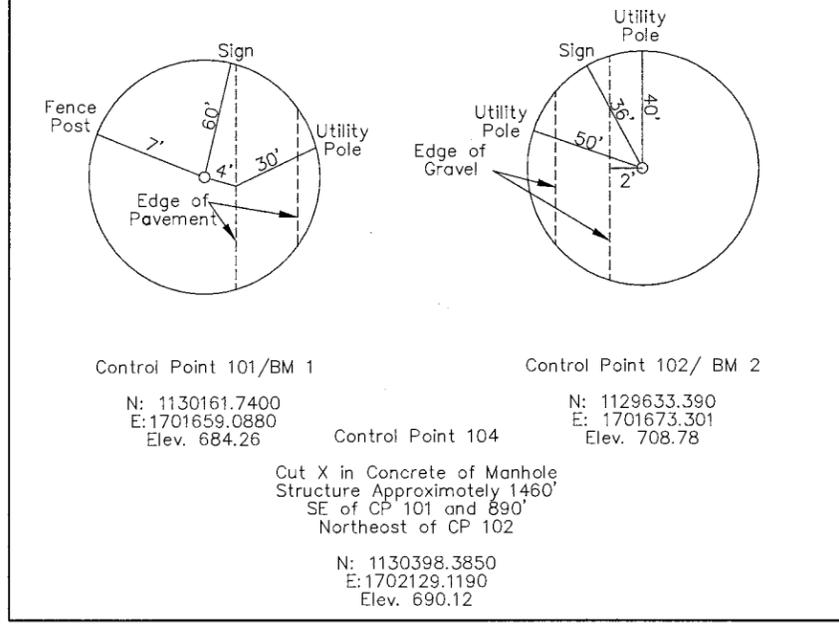
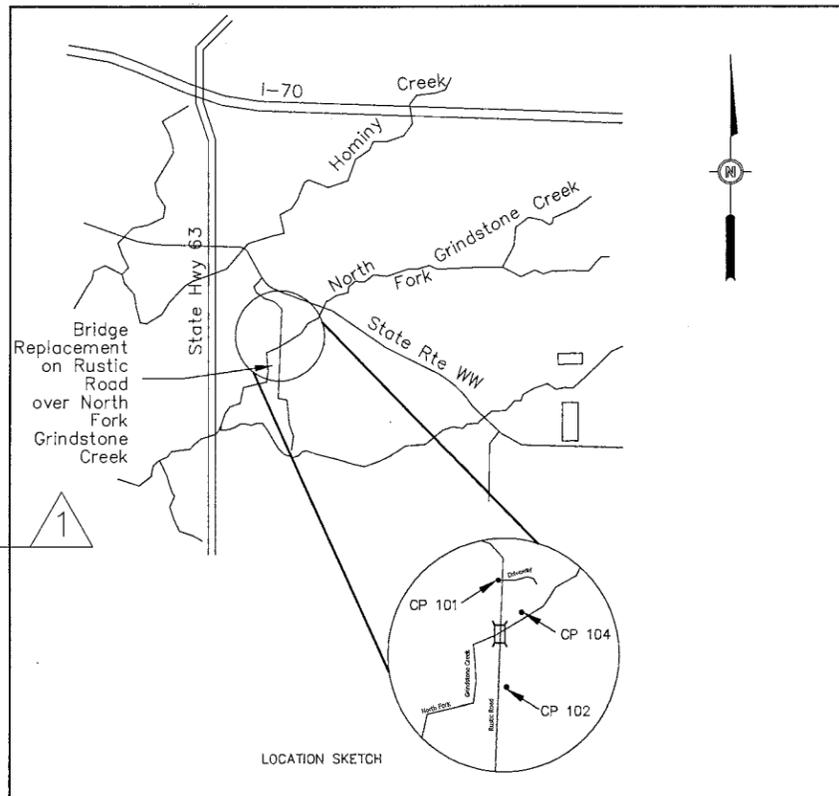
**Joint Filler:**  
 All joint filler shall be in accordance with Sec 1057 for preformed sponge rubber expansion and partition joint filler, except as noted.

**Reinforcing Steel:**  
 Minimum clearance to reinforcing steel shall be 1 1/2", unless otherwise shown.

**Traffic Handling:**  
 Close road of bridge during construction and maintain temporary bypass. See Traffic Control Plan on sheet no. 8.

**Miscellaneous:**  
 "Sec" refers to the sections in the Missouri Standard and supplemental Specifications unless specified otherwise.

High strength bolts, nuts and washers may be sampled for quality assurance as specified in Sec 106.



HYDROLOGIC DATA	
Drainage Area = 6.45 Square Miles	
BACKWATER/BASE FLOOD DATA (100 YR)	
High Water Elev. = 688.53	
Design Discharge = 6,400 cfs	
Estimated Backwater = 0.84 ft.	
Ave Velocity thru Opening = 7.64 ft/s	
FREEBOARD	
Design Frequency = 25 (year)	
Design Discharge = 54,400 cfs	
Freeboard = -2.99 ft.	
Design High Water (DHW) Elev. = 686.91	
ROADWAY OVERTOPPING	
Design Elev.(1' Below Shoulder) = 685.87	
Design Discharge = 2000 cfs	
Design Frequency > 4 (year)	

**Notes:**

All construction materials and methods shall comply with the latest edition of the Missouri Standard Specifications for Highway Construction and the Missouri Standard Plans for Highway Construction unless specified otherwise.

The Contractor shall maintain proper drainage and erosion control at all times during construction.

The locations of existing utilities are shown for informational purposes only and are not guaranteed to be accurate or complete. It is the responsibility of the Contractor to contact all necessary utility companies and obtain utility staking prior to the start of construction.

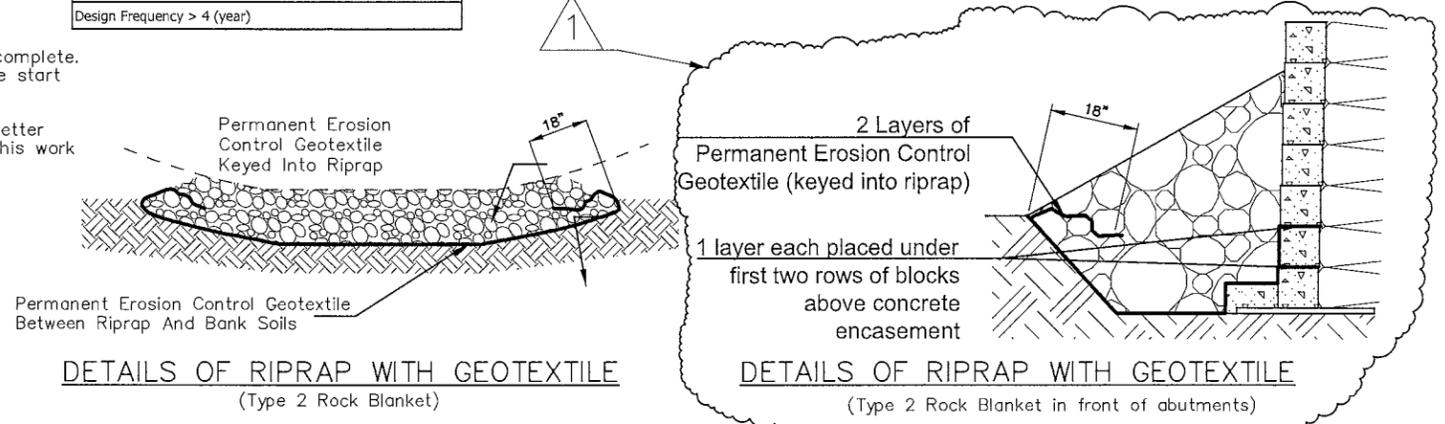
Contractor shall repair or replace any fencing or gates removed or damaged during construction activities to equal or better than existing condition. Work shall be done to the approval of the affected land owner and the Engineer. Payment for this work shall be included in the pay item for "Removal of Improvements".

(\*) Materials, construction requirements and payment for both Furnishing and Placing Type 2 Rock Blanket shall be in accordance with Sec 611 and the Job Special Provisions.

(\*\*) Mobilization will include demobilization and any expenses required for coordination with utilities.

(\*\*\*) Restoration shall conform to the Job Special Provisions.

(\*\*\*\*) Square Feet of Bridge quantity shall not exceed plan quantity.



Note: This Drawing is Not to Scale. Follow Dimensions

**BARTLETT & WEST**  
 1718 SOUTHMOORE DR. SUITE 100 - JEFFERSON CITY, MO 65108-4000  
 PHONE 872.824.3101 FAX 872.824.3102  
 MISSOURI CERTIFICATE NO. 000187 - ENGINEERING  
 100.8/29/14 Updated geotextile properties and location details for backfill.

**GENERAL NOTES AND QUANTITIES**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI

STATE OF MISSOURI  
 TIMOTHY D. LEAF  
 NUMBER  
 PE-2012000778  
 PROFESSIONAL ENGINEER  
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 SIGNED, SEALED AND DATED  
 ELECTRONICALLY.

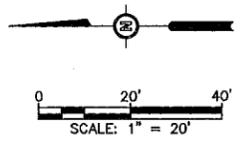
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	GRAVEL
	PERM. UTIL. ESMT.
	TEMP. CONST. ESMT.



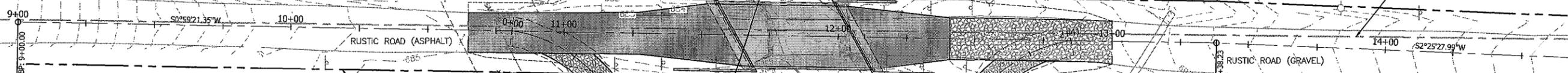
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1	65.74	6258	2160	65.74
2	77.12	6916	-	77.12
3	5.88	2189	-	5.88

OWNERS: PETER A. ANSBACHER, TRUSTEE OF PETER A. ANSBACHER TRUST NO. 1 DEED IN BOOK 832 PAGE 8

ANSBACHER TRACT  
①

OWNERS: PETER A. ANSBACHER, TRUSTEE OF PETER A. ANSBACHER TRUST NO. 1 DEED IN BOOK 832 PAGE 8

ANSBACHER TRACT  
①



PITTENGER FAMILY TRUST TRACT  
②

OWNERS: DONALD E. AND SABRA G. PITTENGER, PITTENGER FAMILY TRUST DEED IN BOOK 2602 PAGE 145

CURBY TRACT  
③

LOT 1  
MAPLE HEIGHTS SUBDIVISION  
PLAT BOOK 19 PAGE 7  
OWNERS: JON K. AND VICKI M. CURBY  
DEED IN BOOK 1542 PAGE 580

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**RIGHT OF WAY & EASEMENT PLAN**

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(692)  
BOONE COUNTY, MISSOURI



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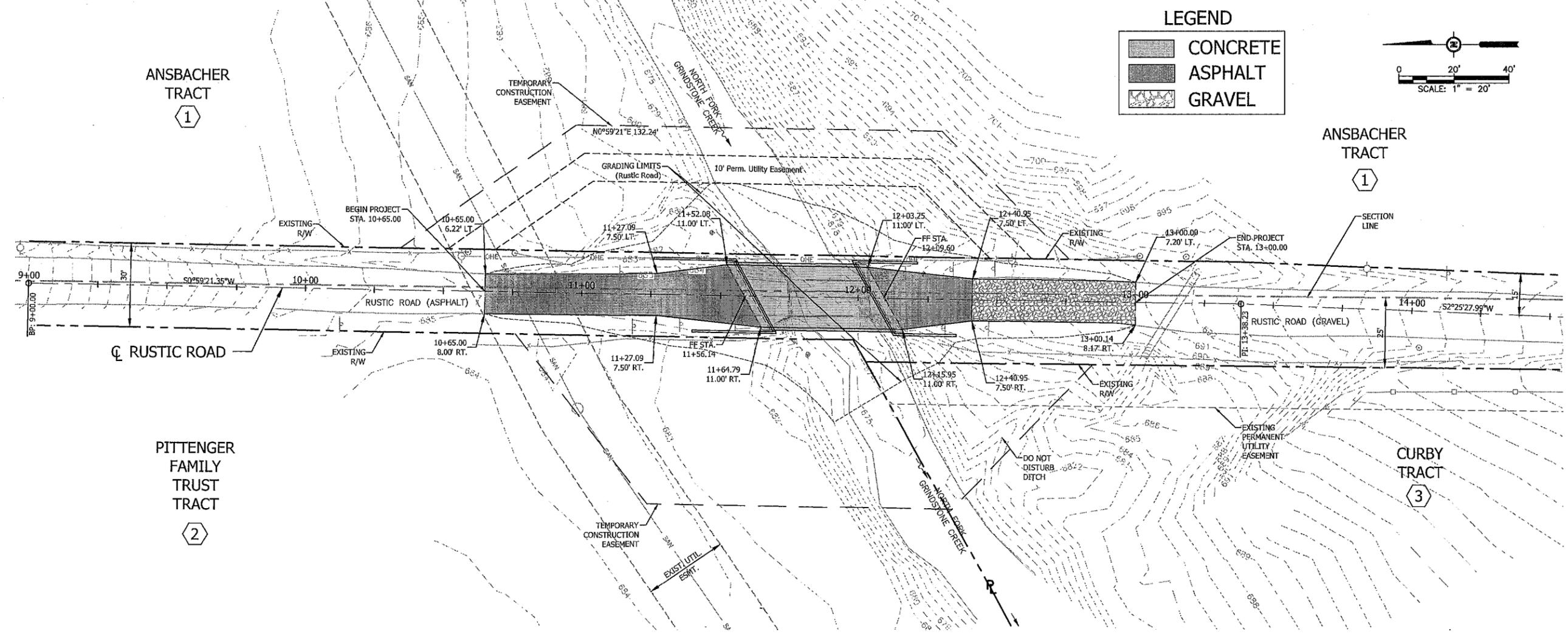
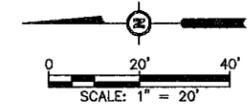
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DRAWN BY:	MSS
APPROVED BY:	RAG
DESIGN PROJ.:	16137.110
SCALE:	AS NOTED
DATE:	JULY/2014
DRAWING NO.:	NONE
SHEET NO.:	4 of 22

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LEGEND

CONCRETE  
 ASPHALT  
 GRAVEL



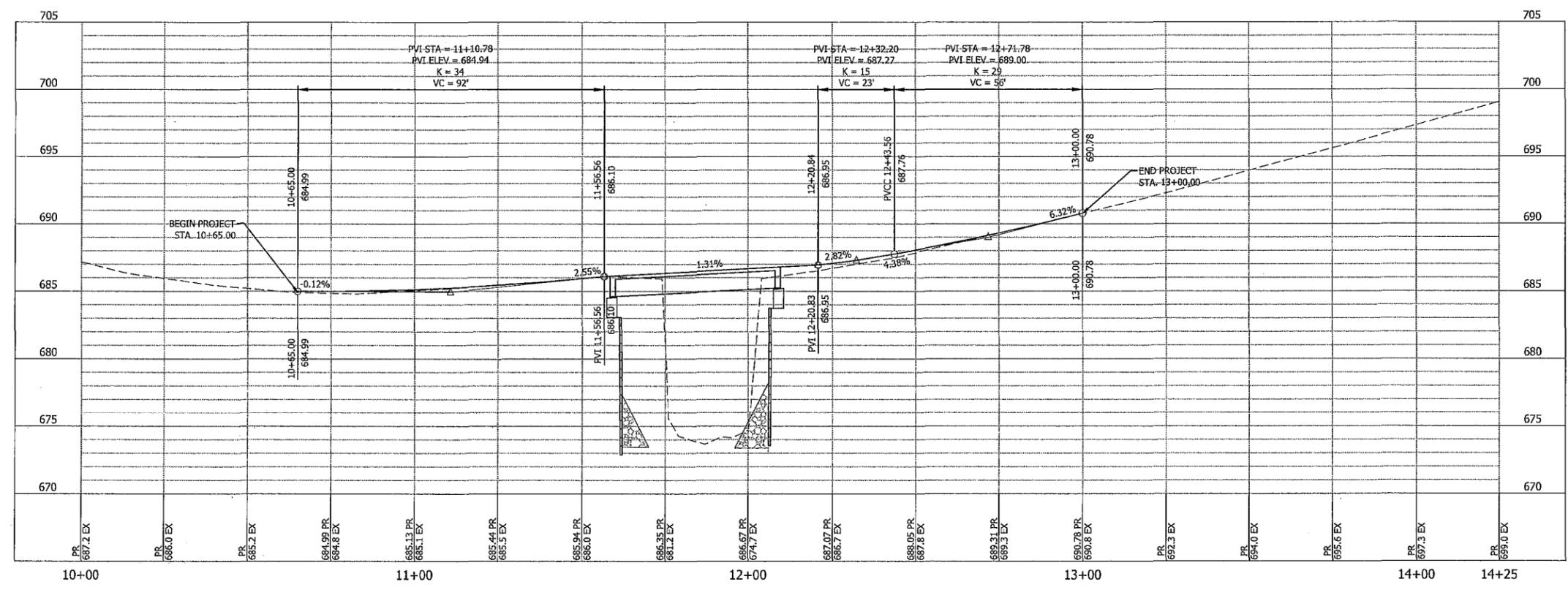
ANSBACHER TRACT  
 1

ANSBACHER TRACT  
 1

PITTENGER FAMILY TRUST TRACT  
 2

CURBY TRACT  
 3

Notes:  
 For limits of temporary construction easements see sheet no. 4.  
 The note "Do Not Disturb Ditch" does not apply to the required erosion control measures.



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**FINAL ROADWAY PLAN-PROFILE**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI

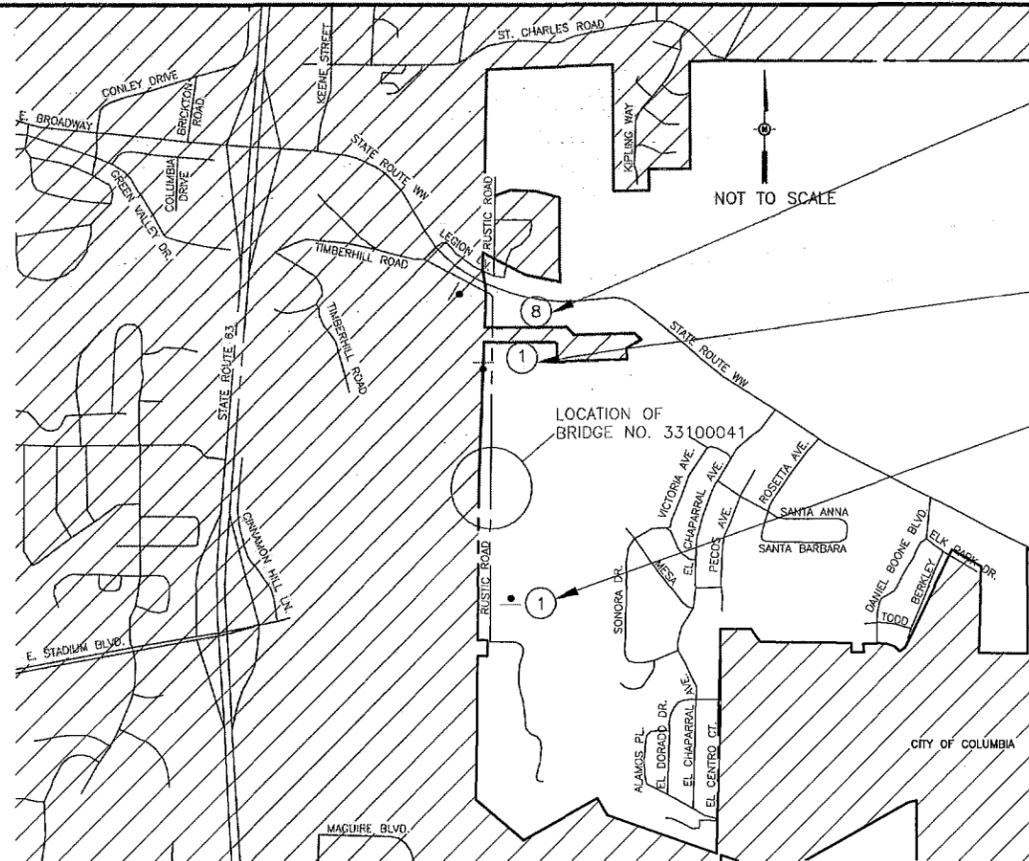
STATE OF MISSOURI  
 TIMOTHY D. LEW  
 NUMBER PE-20200778  
 PROFESSIONAL ENGINEER

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 SEALED DATE: 07/22/14  
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 DRAWN BY: MSS  
 APPROVED BY: RAG  
 DESIGN PROJ: 16137.110  
 DATE: JULY/2014  
 DRAWING NO: NONE  
 SHEET NO: 6 of 22

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 Last edit on:



Drawing name: W:\Proj\16000\16137\110\AutoCad\Plan Set\Traffic-Control.dwg Layout name: Traffic Control Signs Plotted on: Jul 22, 2014 - 1:43pm  
Last edit on: EDITDATE



**BRIDGE  
CLOSED AT  
NORTH FORK  
GRINDSTONE CREEK**  
60" X 30"  
(Black lettering with  
Orange background)  
8

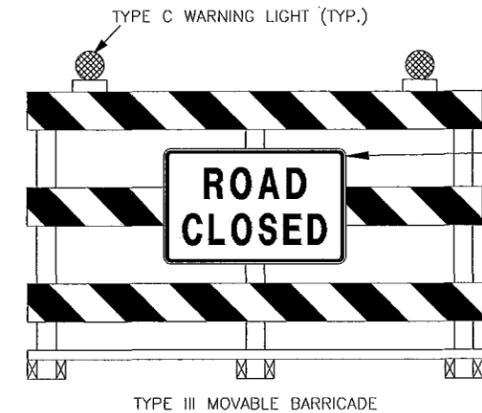
- 1 ROAD WORK AHEAD  
W020-1  
48" X 48"
- 2 SPEED LIMIT 25  
R2-1  
24" X 30"
- 5 SPEED LIMIT 35  
R2-1  
24" X 30"
- 9 YIELD  
R1-2  
36" X 36" X 36"

- 10 ROAD NARROWS  
W5-1  
48" X 48"
- 11 ONE LANE ROAD 200 FT  
W20-4  
48" X 48"

**TO  
ONCOMING  
TRAFFIC**  
R1-2aP  
36" X 30"  
9

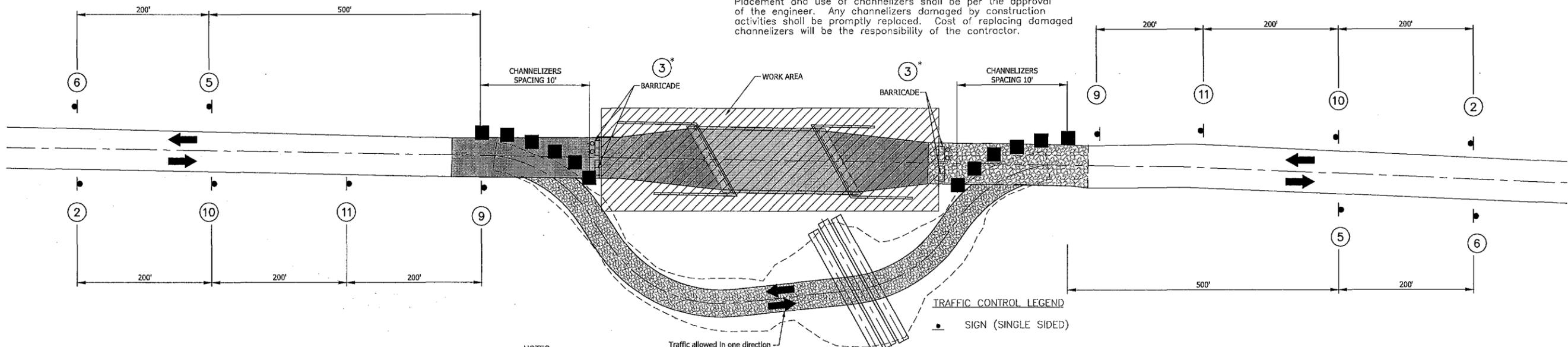
**END  
ROAD WORK**  
G20-2  
36" X 18"  
6

**ROAD  
CLOSED**  
R11-2  
48" X 30"  
3\*



\* ONE SIGN REQUIRED PER SIDE OF CLOSURE ONLY

Notes:  
Contractor shall comply with the Manual on Uniform Traffic Control Devices (MUTCD) for the traffic control of this project.  
Cover all conflicting signs.  
Use a minimum sign spacing of 200' unless specified otherwise or as directed by the Engineer.  
See Standards No. 616.10 and 903.03 of the Missouri Standard Plans for Highway Construction for details not shown.  
Contractor will be responsible for eliminating conflicts caused by opposing traffic on the temporary bypass.  
Placement and use of channelizers shall be per the approval of the engineer. Any channelizers damaged by construction activities shall be promptly replaced. Cost of replacing damaged channelizers will be the responsibility of the contractor.



NOTES:  
DIMENSIONS IN FEET UNLESS OTHERWISE NOTED.  
(1) POSTED SPEED LIMIT PRIOR TO ROAD WORK.  
(2) SPACINGS MAY BE ADJUSTED AS NECESSARY TO MEET FIELD CONDITIONS.

- TRAFFIC CONTROL LEGEND
- SIGN (SINGLE SIDED)
  - ▬ TYPE III MOVABLE BARRICADE
  - ⊙ TYPE C WARNING LIGHT
  - CHANNELIZERS

Note: Drawing is not to scale. Follow dimensions

BARTLETT & WEST

TRAFFIC CONTROL

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(592)  
BOONE COUNTY, MISSOURI

STATE OF MISSOURI  
TIMOTHY D. LEAF  
NUMBER  
PE-201200778  
PROFESSIONAL ENGINEER

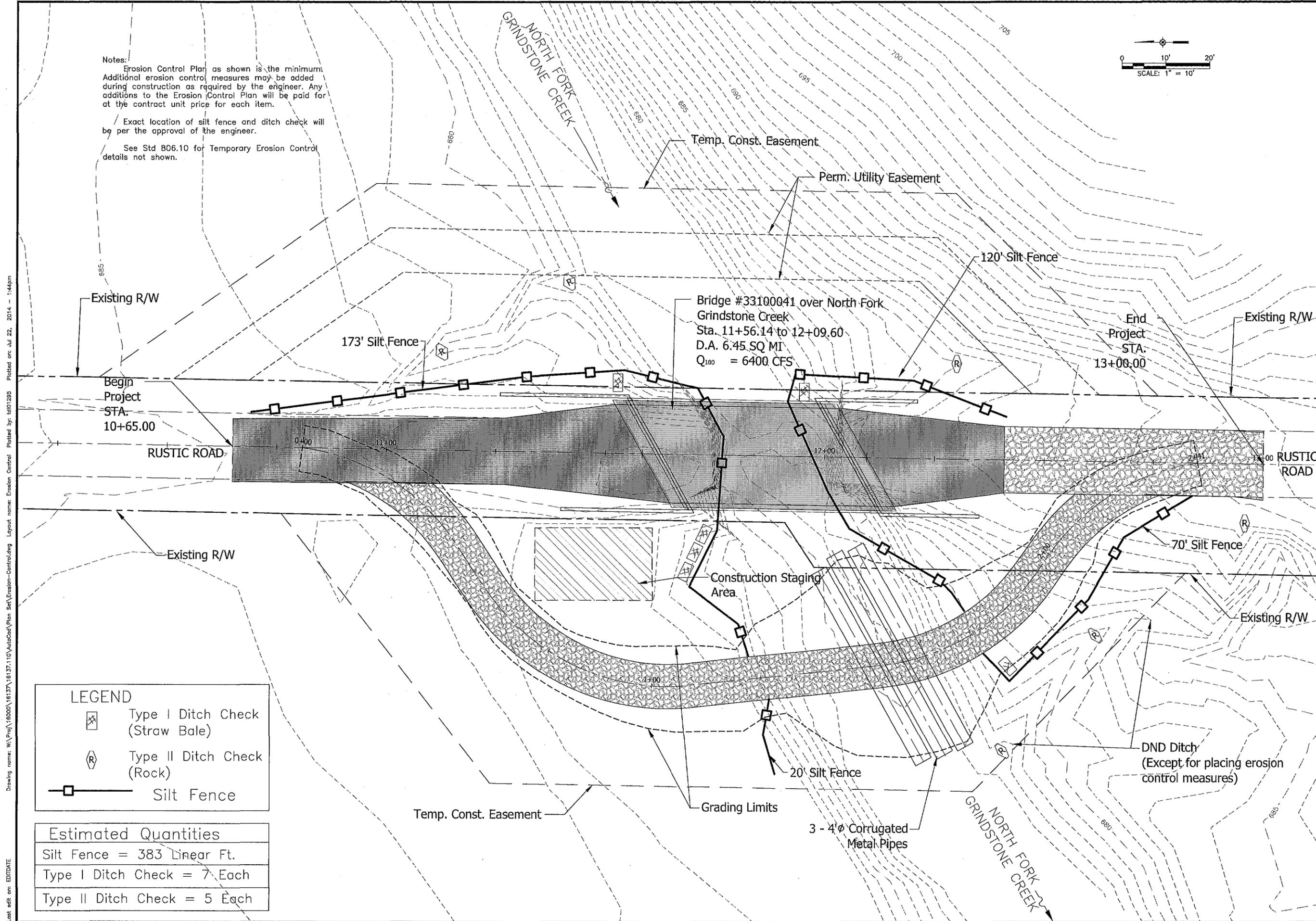
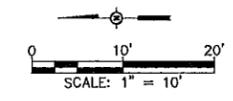
THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

SEALED DATE:	07/22/14
DESIGNED:	TDL
DRAWN BY:	MSS
APPROVED BY:	RAG
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SCALE:	AS NOTED
DATE:	JULY/2014
DRAWING NO.:	NONE
SHEET NO.:	8 of 22

Notes:  
 Erosion Control Plan as shown is the minimum. Additional erosion control measures may be added during construction as required by the engineer. Any additions to the Erosion Control Plan will be paid for at the contract unit price for each item.

Exact location of silt fence and ditch check will be per the approval of the engineer.

See Std 806.10 for Temporary Erosion Control details not shown.



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 Layout name: Erosion Control  
 Plotted on: Jul 22, 2014 - 1:44pm  
 Plotted by: f4l01295  
 Last edit on: EDDDATE

LEGEND	
	Type I Ditch Check (Straw Bale)
	Type II Ditch Check (Rock)
	Silt Fence

Estimated Quantities	
Silt Fence	= 383 Linear Ft.
Type I Ditch Check	= 7 Each
Type II Ditch Check	= 5 Each

NO.	DATE	DESCRIPTION

**BARTLETT & WEST**

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 PHONE: 970-536-3100, FAX: 970-536-3700  
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**EROSION CONTROL PLAN**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI

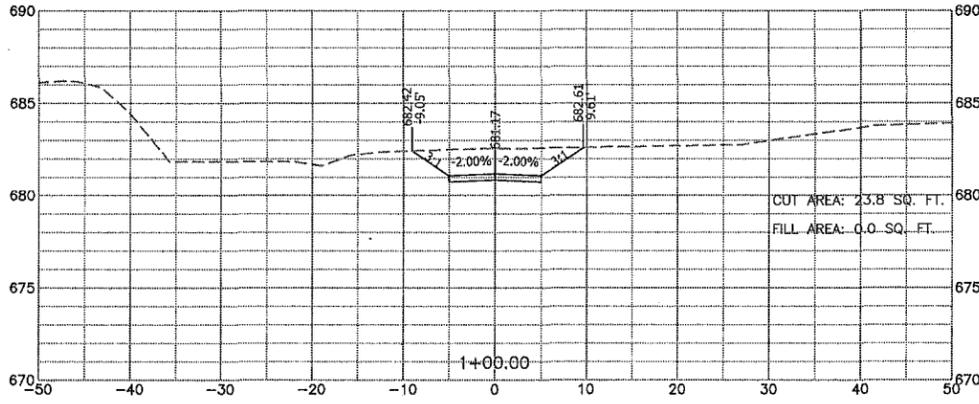
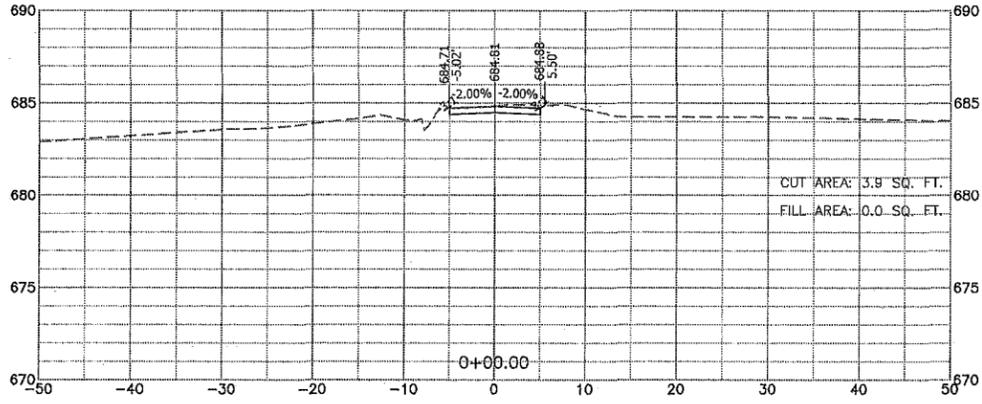
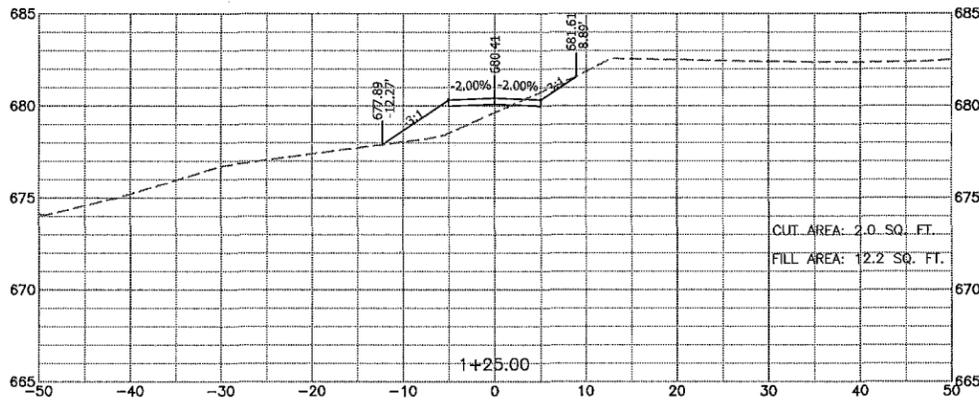
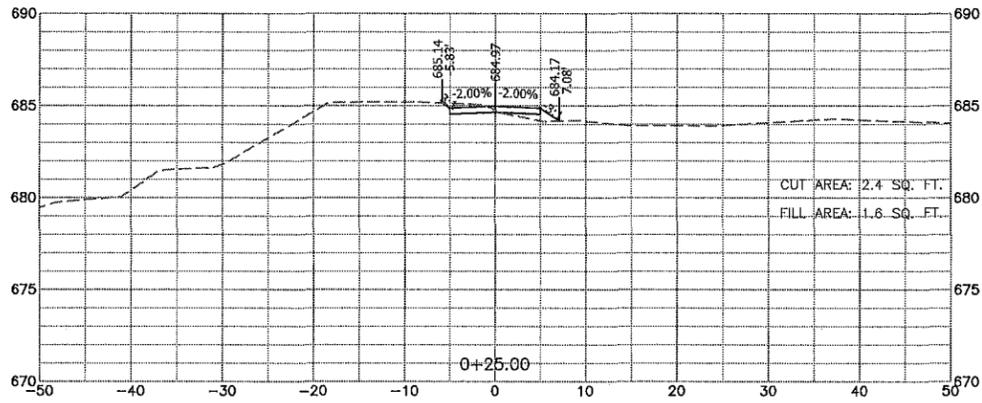
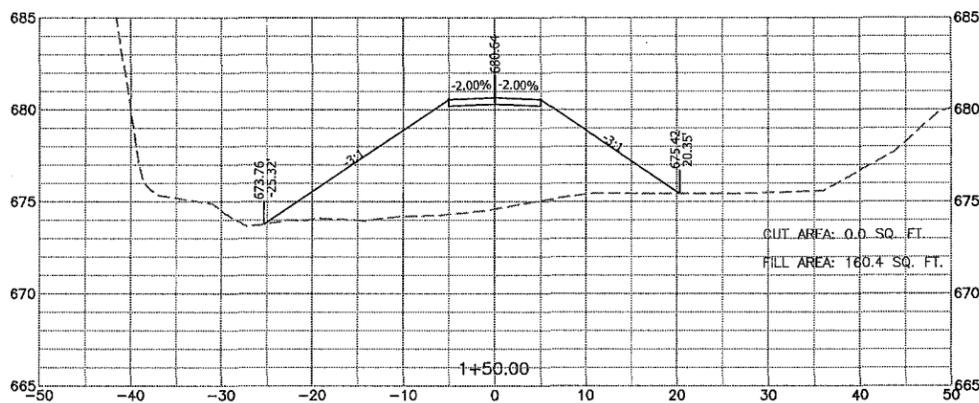
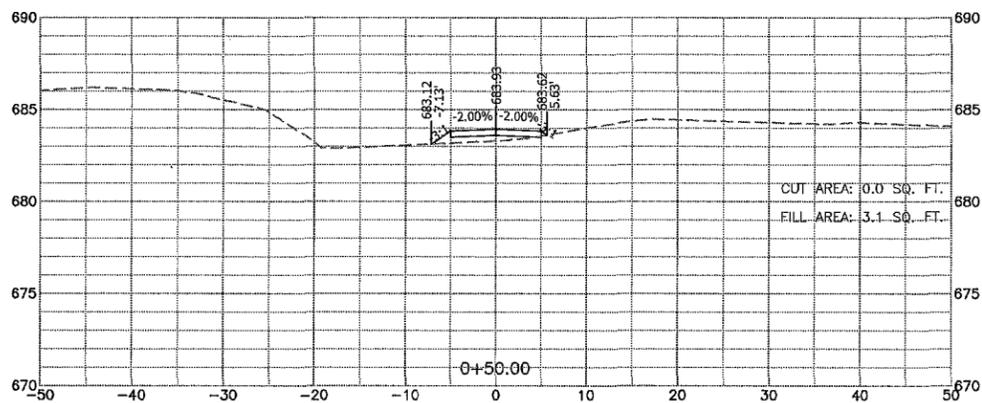
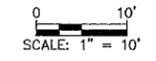
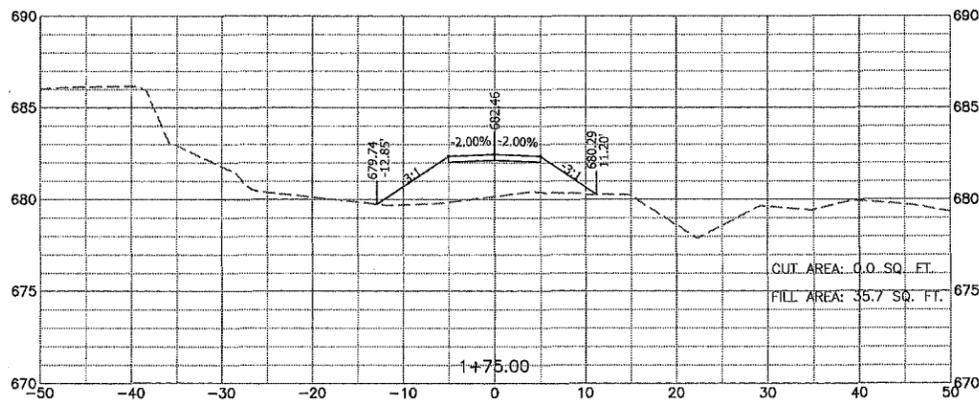
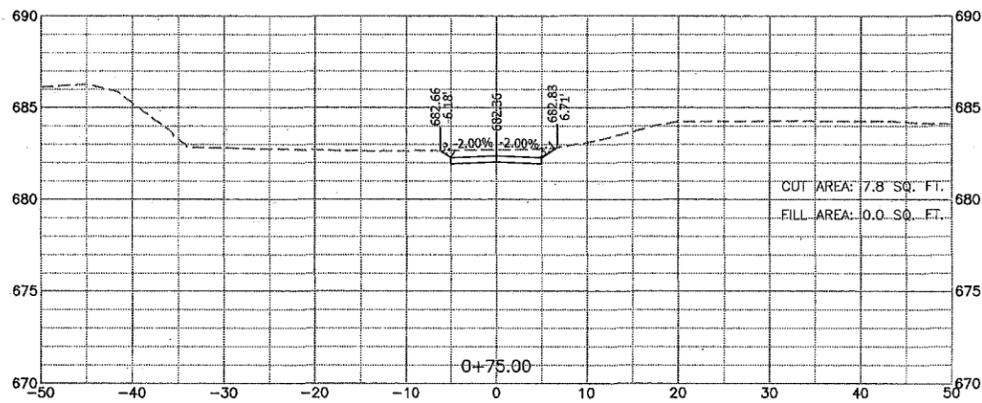
STATE OF MISSOURI  
 TIMOTHY D. LEAF  
 NUMBER PE-2012000778  
 PROFESSIONAL ENGINEER

THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

SEALED DATE:	07/22/14
DESIGNED BY:	TDL
DRAWN BY:	MSS
APPROVED BY:	RAG
PROJ.:	16137.110
SCALE:	AS NOTED
DATE:	JULY/2014
DRAWING NO.:	NONE
SHEET NO.:	9 of 22

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 Layout name: Layout1 (3)  
 Plotted by: tdl1285  
 Plotted on: Jul 22, 2014 - 1:44pm  
 Last edit on: 07/09/14



NOTE:  
 SEE JOB SPECIAL PROVISIONS FOR DETAILS OF REMOVAL OF  
 TEMPORARY BYPASS AND REQUIREMENTS TO RESTORE TOPOGRAPHY  
 TO EXISTING CONDITIONS.

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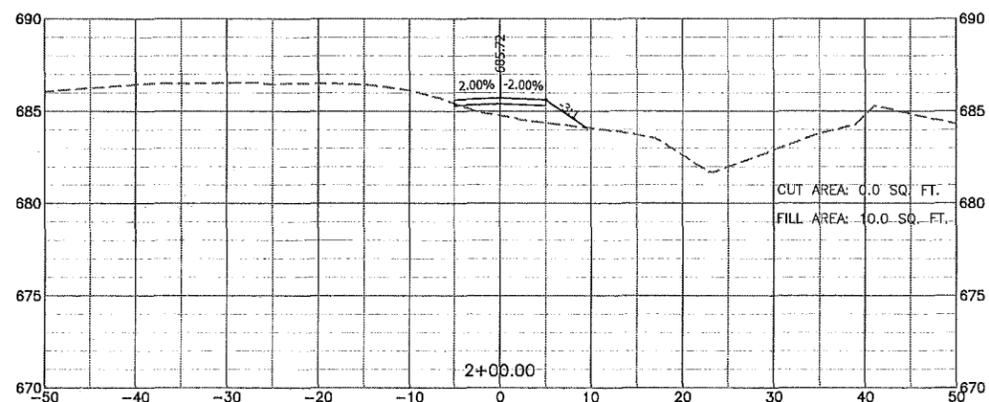
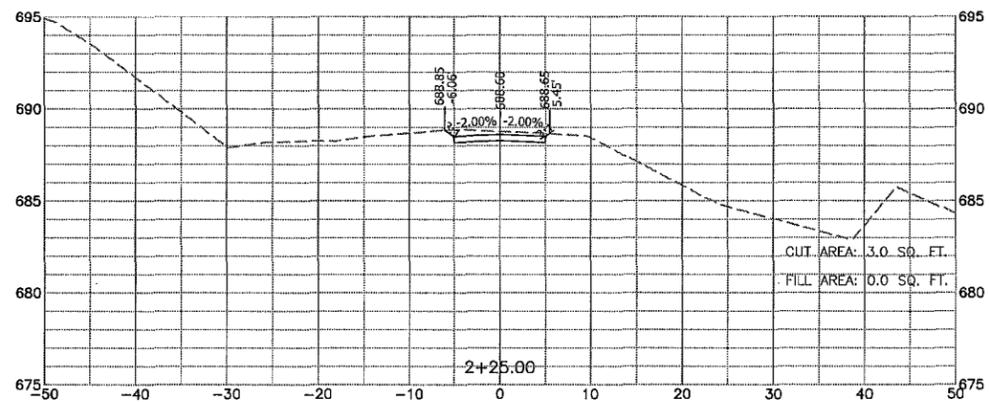
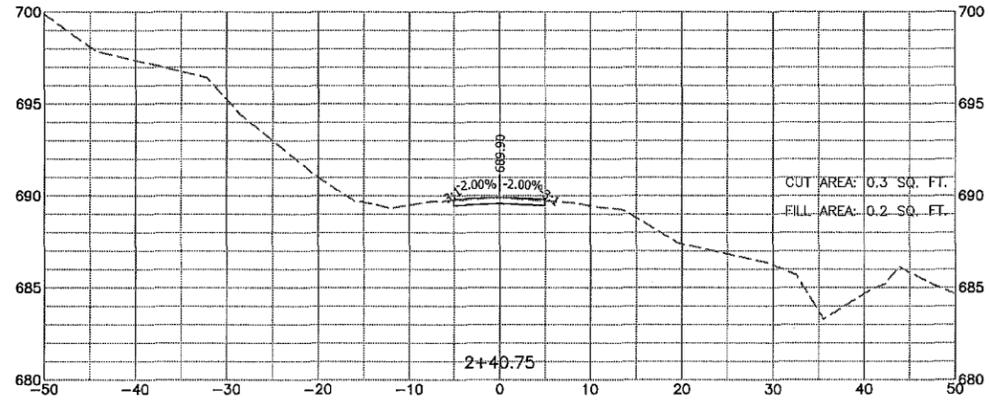
**CROSS SECTIONS  
 TEMPORARY BYPASS**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(692)  
 BOONE COUNTY, MISSOURI

STATE OF MISSOURI  
 TIMOTHY D. LEAF  
 NUMBER  
 PE-2012000778  
 PROFESSIONAL ENGINEER  
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NOTE:

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**CROSS SECTIONS  
 TEMPORARY BYPASS**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI



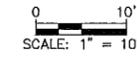
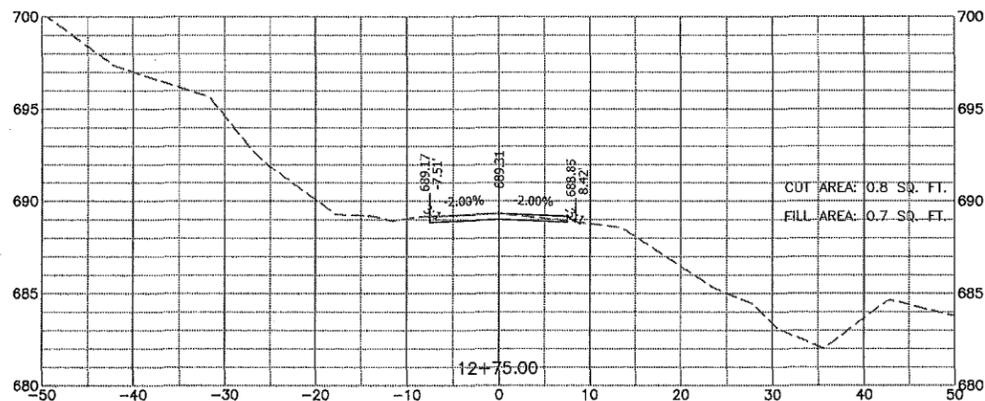
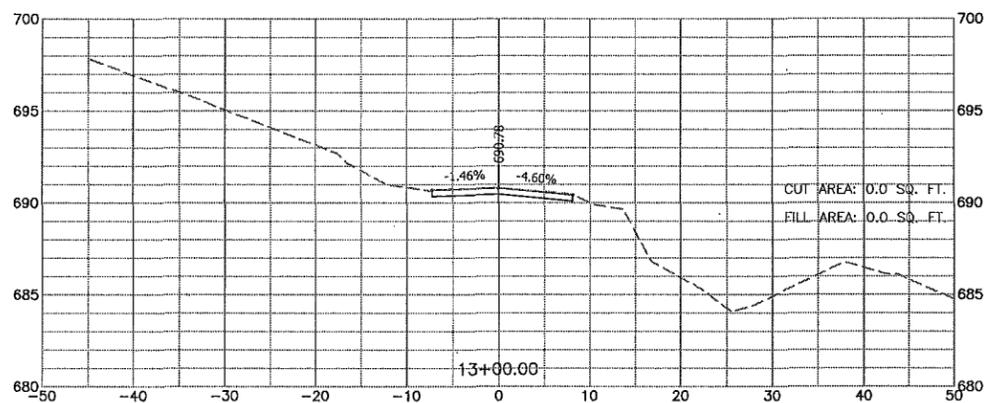
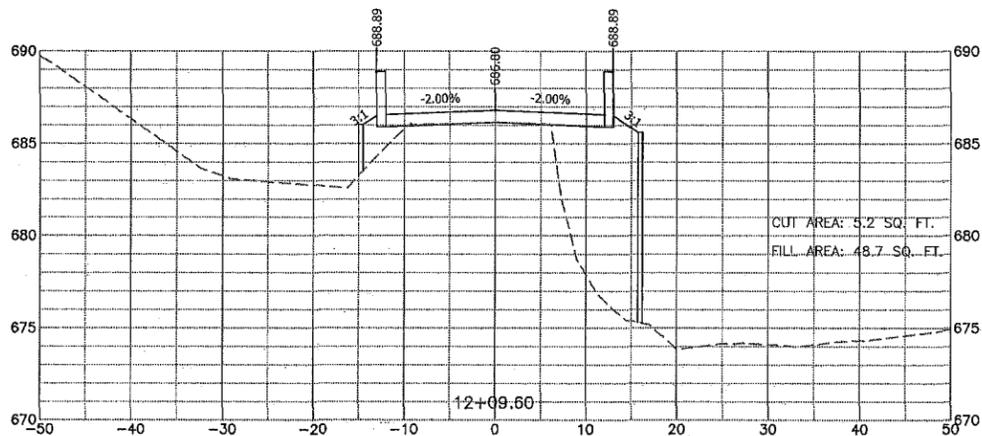
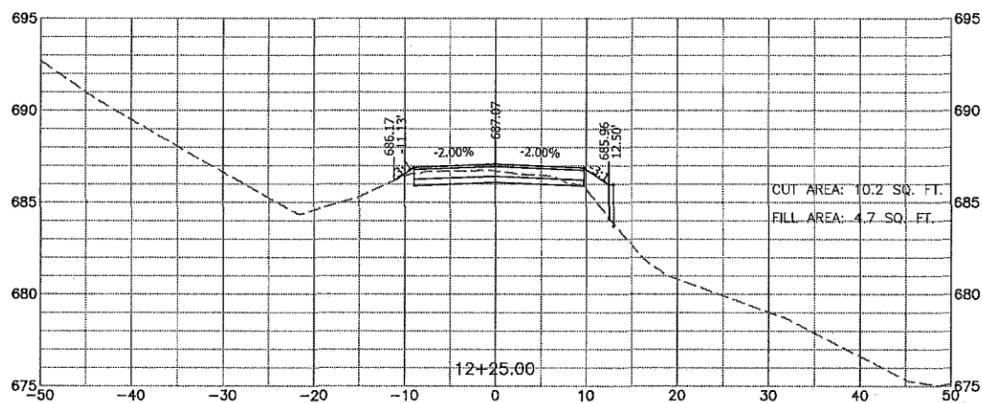
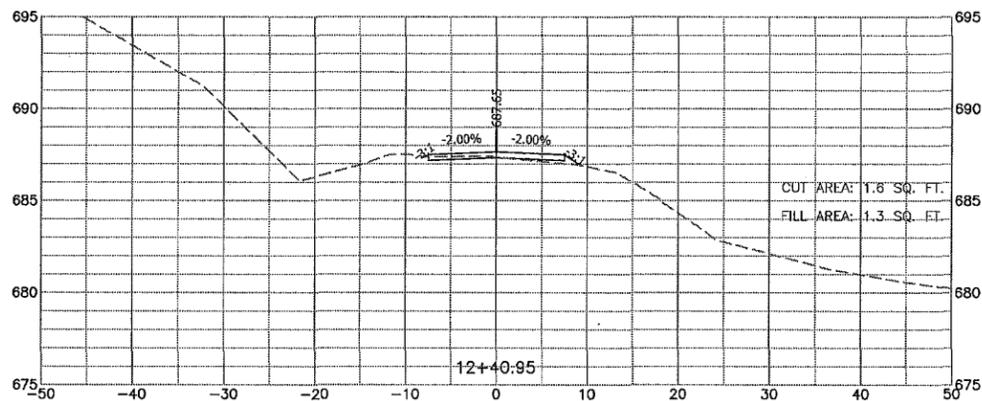
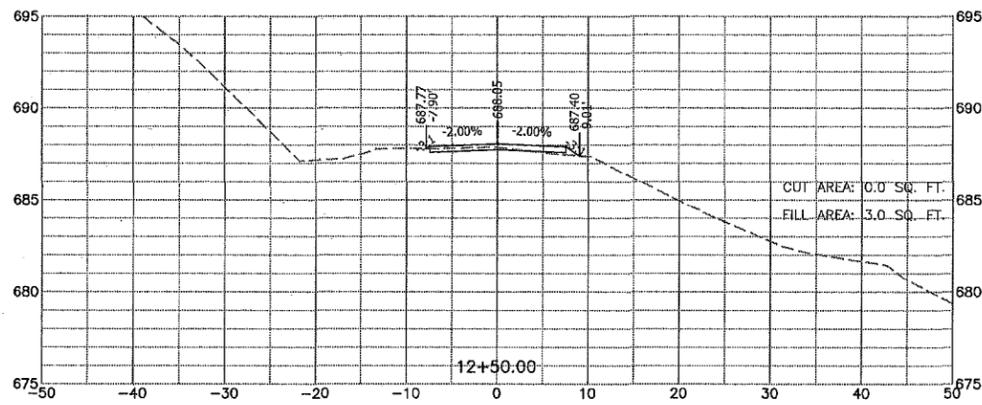
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SEALED DATE: 07/22/14  
 DESIGNED BY: TDL  
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 SCALE: AS NOTED  
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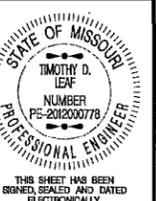
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**CROSS SECTIONS  
RUSTIC ROAD**  
RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(692)  
BOONE COUNTY, MISSOURI



THIS SHEET HAS BEEN  
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SEALED DATE: 07/22/14  
DESIGNED BY: TDL  
DRAWN BY: MSS  
APPROVED BY: RAG  
DESIGN PROJ: 16137.110  
SCALE: AS NOTED  
DATE: JULY/2014  
DRAWING NO: NONE  
SHEET NO: 13 of 22

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NOTES:  
For General Notes, Final Quantities, and Location Sketch, See sheet No. 2

Outline of existing bridge 3310004 is indicated by light Dashed Lines. Heavy lines indicate new work.

\* Concrete encasement as shown is a cutaway view in order to show the level course. Concrete encasement, as constructed shall completely enclose the level course.

**INSTRUMENTATION NOTES:**  
The University of Missouri will provide the contractor with a detailed instrumentation plan.

University of Missouri to furnish the following items:

- Telltales (North Bent Only) - there are a total of 3 telltales, 1 installed at the footing, 1 at the lower 1/2 point of the abutment and 1 at the upper 1/2 point of the abutment. Telltales will be installed in-line vertically through a common casing. For locations of telltales see Sheet No. 18.

- Earth pressure cells (North Bent only) - All cells to be installed with a thin layer of fine aggregate above and below the pressure cells. For pressure cells below the superstructure backwall, cells to be installed in the Beam Seat aggregate approximately 1" below the backwall. For locations of Earth pressure cell see Sheets 15 and 18.

- Tensiometers (North Bent only) - tensiometers shall be placed as shown on the plans. For locations of tensiometers see Sheet No. 15.

The above items will have cables which will run through an instrumentation trench to the NEMA box. Details of the trench will be provided in the instrumentation plan.

- Inclinator and SAA casings (both End Bents) - Contractor is to install the casings prior to construction of the substructure. Casings should be embedded in grout 1 foot into rock. Casings will be placed outside of the pavement, if possible.

Contractor to furnish the following items:

- Type 3 NEMA box - the minimum dimensions of the NEMA box are 24" x 20" x 6" (HxWxD). No direct payment will be made for the NEMA box or associated items. Costs shall be subsidiary to the project.

- Survey Markers (both End Bents) - use angled retroreflective survey markers (targets) (approximately 120 mm wide by 75 mm tall) placed as shown on the plans. No direct payment will be made for survey markers. Costs shall be subsidiary to the project. For locations of survey markers see Sheets 16-17.

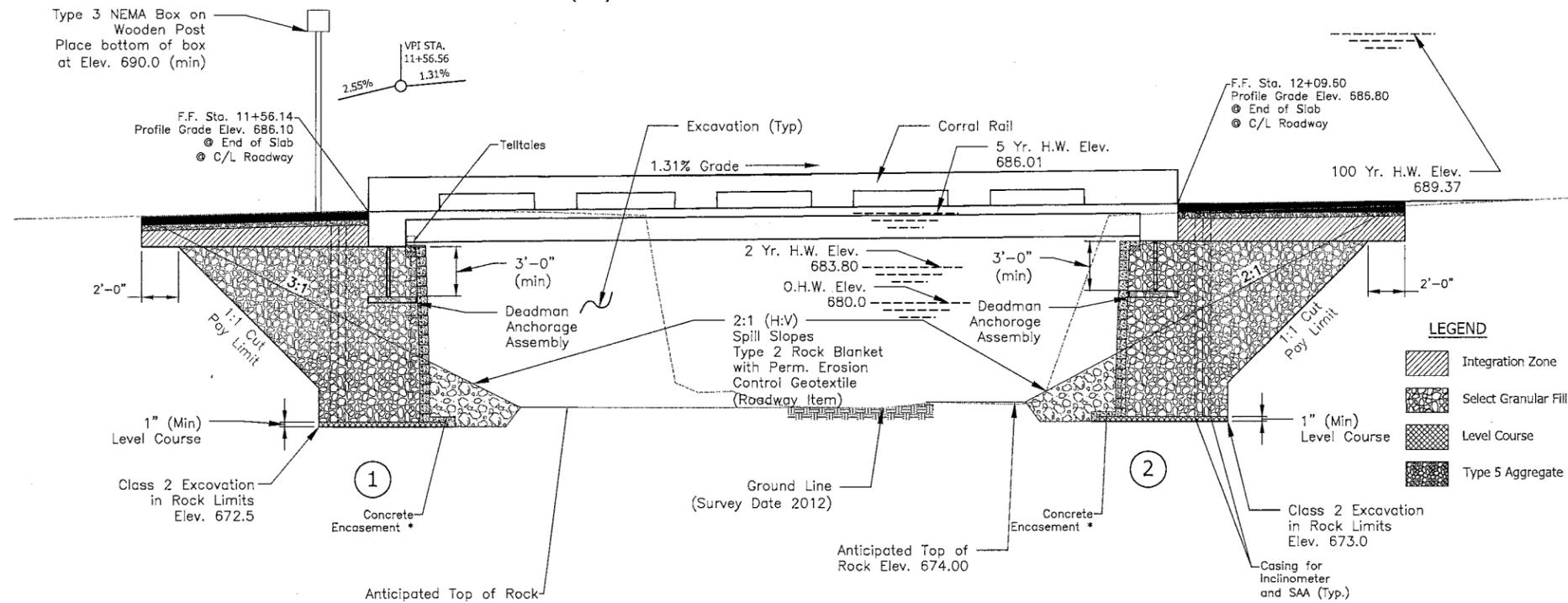
- Inclometers will require holes to be drilled into the rock. Holes will be approximately 2" in diameter and the depth will be as directed by the University.

The items listed under Instrumentation Notes will be installed in coordination between the University and the contractor.

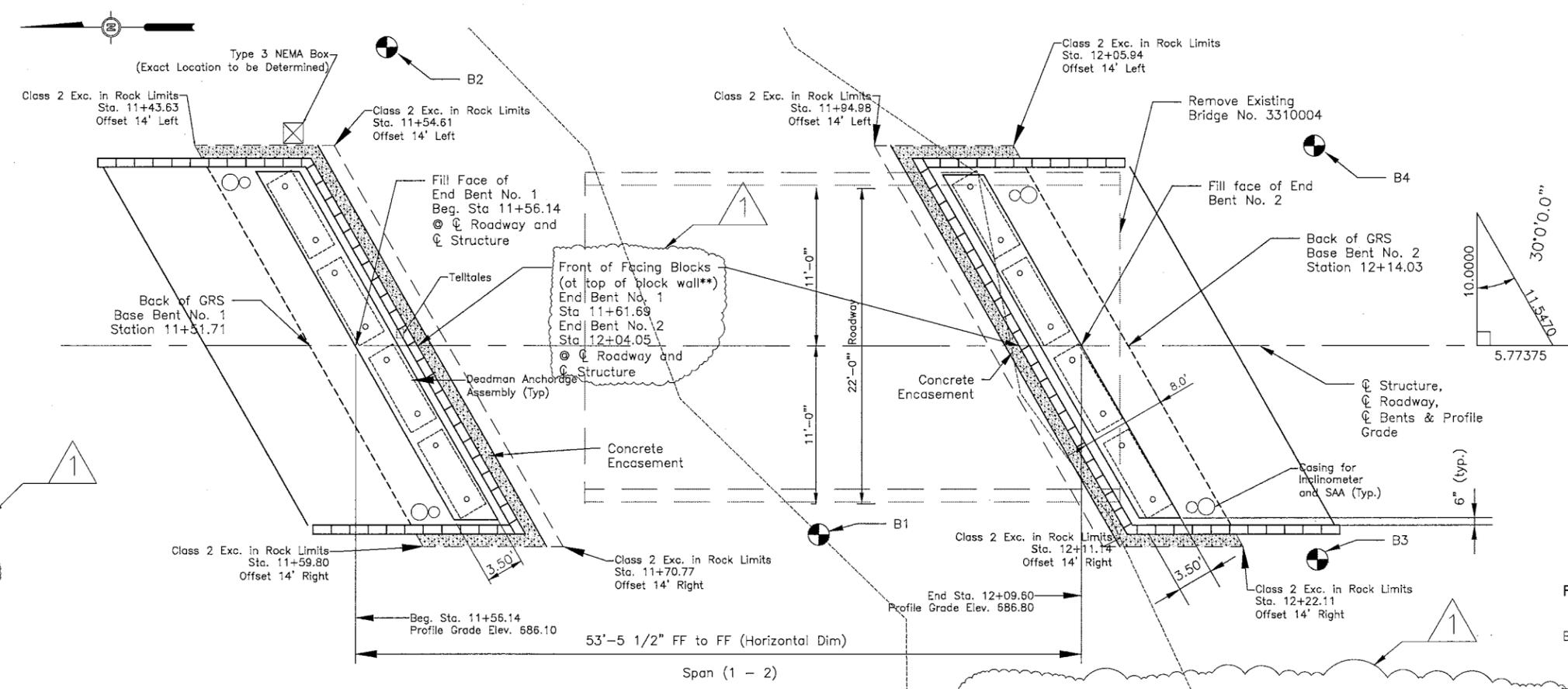
See Job Special Provisions for additional information associated with instrumentation.

# BRIDGE - RUSTIC ROAD OVER NORTH FORK GRINDSTONE CREEK

(50') STEEL TUB WITH PRECAST SLAB GIRDER SPAN



GENERAL ELEVATION



PLAN

Note: This Drawing is Not to Scale. Follow Dimensions

"⊙" Indicates location of borings.  
Notice and Disclaimer Regarding Boring Log Data

The locations of all subsurface borings for this structure are shown on the bridge plan sheet for this structure. Boring data for the numbered locations is shown on Sheet Nos. 21-22. The boring data for all locations indicated, as well as any other boring logs or other factual records of subsurface data and investigations performed for the design of the project, will be available from the Engineer upon written request. No greater significance or weight should be given to the boring data depicted on the plan sheet than is subsurface data available elsewhere.

The Engineer does not represent or warrant that any such boring data accurately depicts the conditions to be encountered in constructing this project. A contractor assumes all risks it may encounter in basing its bid prices, time or schedule of performance on the boring data depicted here or those available from the Engineer, or on any other documentation not expressly warranted, which the contractor may obtain.

**BARTLETT & WEST**  
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PHONE 573.633.1311 - FAX 573.633.7904  
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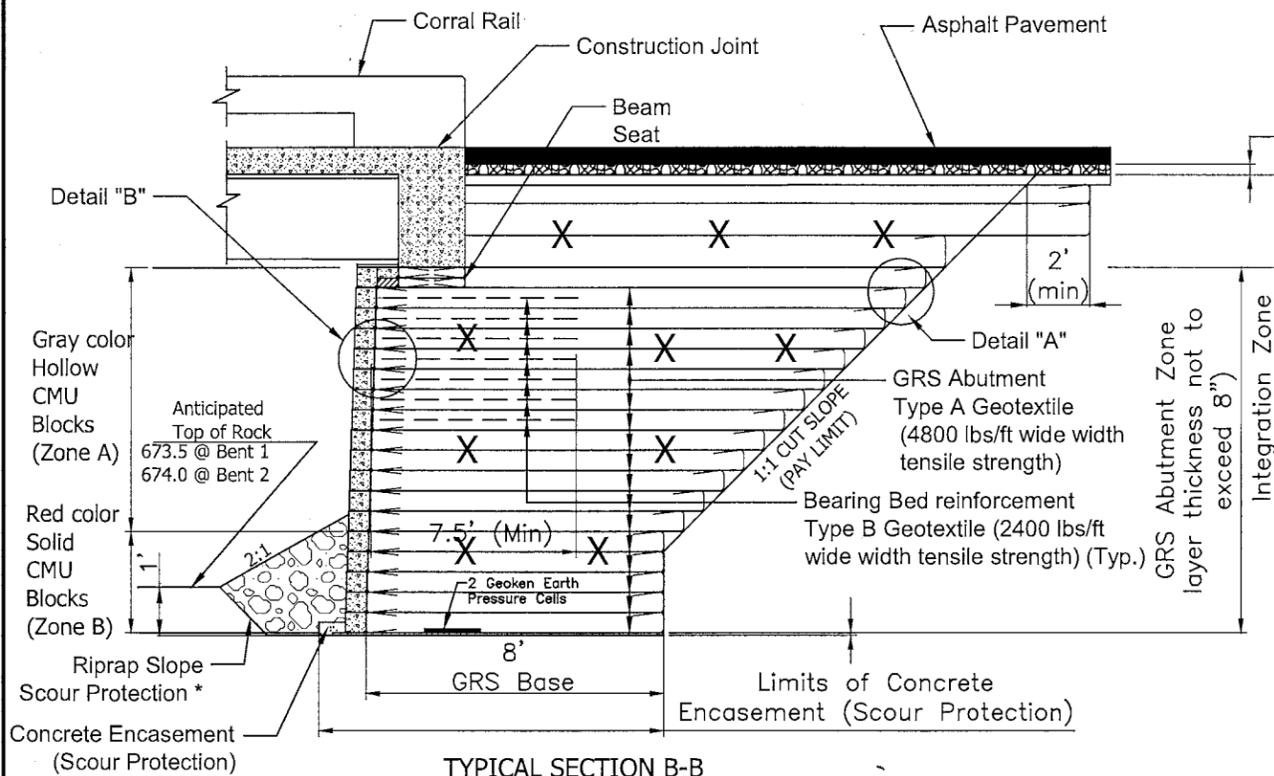
**ELEVATION AND PLAN**  
RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(592)  
BOONE COUNTY, MISSOURI

STATE OF MISSOURI  
TIMOTHY D. LEK  
NUMBER PE-201200778  
PROFESSIONAL ENGINEER  
THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

SEAL DATE:	08/29/14
DESIGNED BY:	TDL
DRAWN BY:	MSS
APPROVED BY:	RAG
DESIGN PROJ.:	16137.110
SCALE:	AS NOTED
DATE:	AUGUST/2014
DRAWING NO.:	NONE
SHEET NO.:	14 of 22

Drawing name: W:\Proj\16000\16137\110\AutoCAD\Plan\_S&I\Bridge\_Drawings.dwg Layout name: ELEV&PLAN Plotted on: Aug 29, 2014 1:58pm

Drawing name: W:\Projects\16000\16137\110\AutoCad\Plan Set\GRS\_System.dwg  
 Layout name: Mech Fill  
 Plotted by: TDL01285  
 Plotted on: Aug 29, 2014 - 2:00pm  
 Last edit on: 08/29/14



**TYPICAL SECTION B-B**  
(SEE SHEETS 16 & 17 FOR LOCATION OF SECTION B-B)

\* Geotextile in front of block wall, embedded in the riprap, shall be doubled up. Extend riprap geotextile between the two bottom layers of facing blocks above the concrete encasement to provide a frictional connection. See sheet no. 2 for details of geotextile and riprap not shown.

Item		Bent 1	Bent 2
Excavation	Cubic Yard	500	260
Type B Concrete Encasement (Scour Protection)	Cubic Yard	2	2
Fine Aggregate Level Course (1" Thick Minimum)	Square Yard	32	32
Select Granular Fill	Cubic Yard	170	170
Type A Geosynthetic Reinforcement	Square Yard	1800	1800
Type B Geosynthetic Reinforcement	Square Yard	145	145
Separation Geotextile Fabric	Square Yard	580	580
Area of Facing Blocks (Zones A and B)	Square Feet	1070	965
Reinforcing Steel	Pounds	123	123
Deadman Anchorage System	Pounds	785	785

Notes:  
The table of Estimated Quantities for Geosynthetic Reinforced Soil System represents the quantities used in preparing the cost estimate and are for information only. Payment for all materials and labor to construct the GRS System at the end bents, including the items in the above table, will be considered completely covered by the lump sum price for Geosynthetic Reinforced Soil System. Variations may be encountered in the estimated quantities but the variations cannot be used for an adjustment in the contract unit prices.

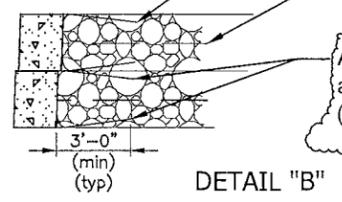
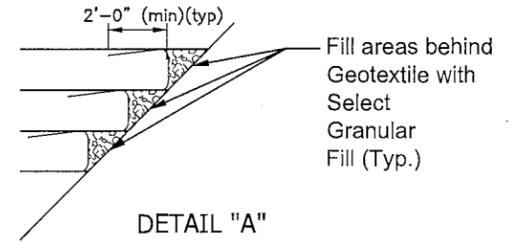
Work this sheet with sheets 14, 16, 17 & 18.

Type A Geosynthetic Reinforcement may be substituted for Type B Geosynthetic Reinforcement. No adjustments will be made to the contract price.

For properties of Select Granular Fill, Block Wall (inc. Zones A & B) and Geosynthetic Reinforcement and Separation Geotextile, see the Job Special Provisions.

For Details and Elevations of Beam Seat see Sheet No. 18.

For Details of the installation of Instrumentation Devices see Sheet No. 14 and the Job Special Provisions.

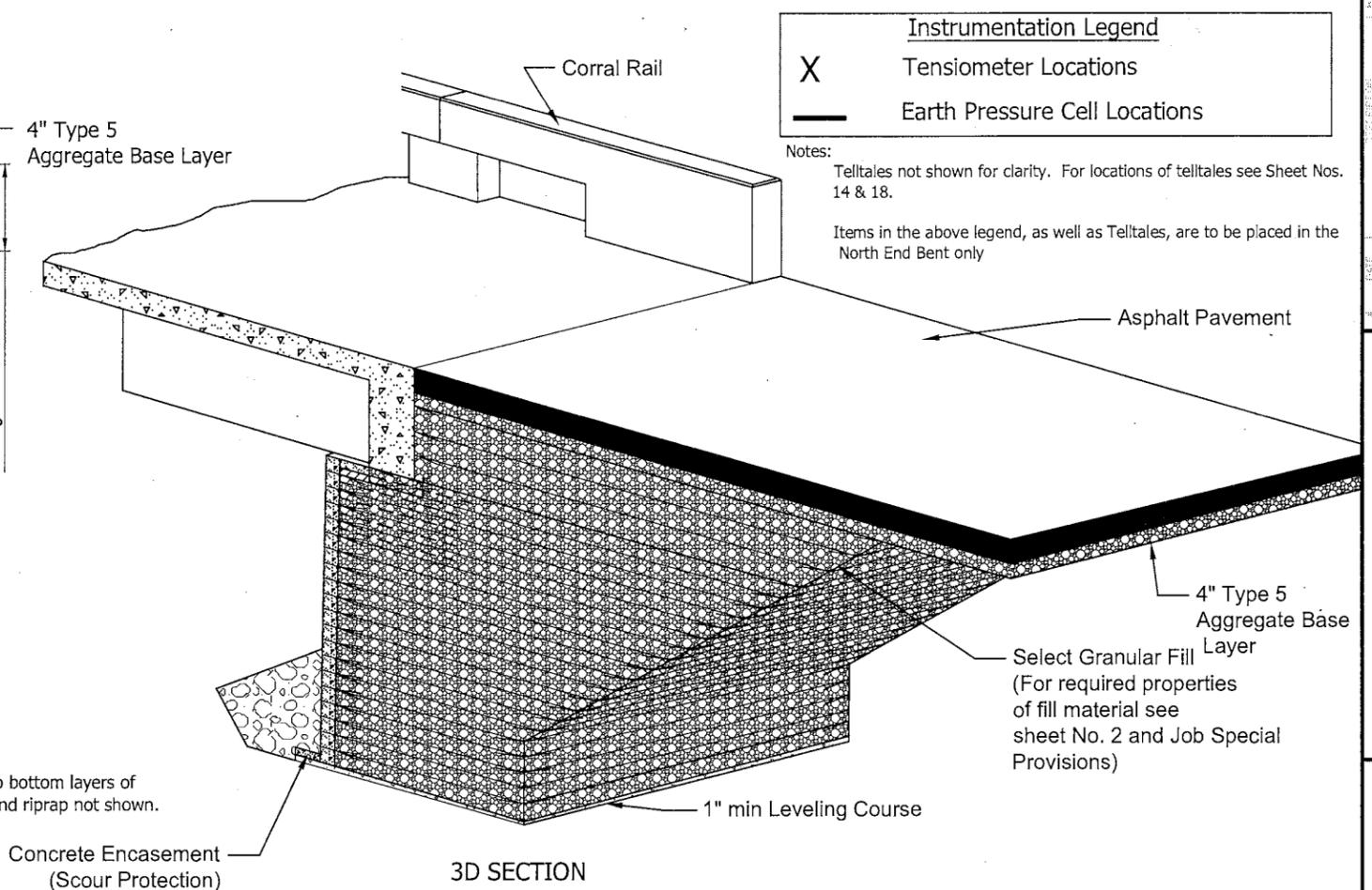


Wrapped sections shall be a separation geotextile material in accordance with the Job Special Provisions and Sec. 1011.3.4.

Type B Geosynthetic placed to back of facing blocks

Add Separation Geotextile and wrap behind the CMU blocks as shown (Front Face of Wall only (typ.))

Note: Drawing is not to scale. Follow dimensions



**Instrumentation Legend**

X Tensiometer Locations

— Earth Pressure Cell Locations

Notes:  
Telltales not shown for clarity. For locations of telltales see Sheet Nos. 14 & 18.  
Items in the above legend, as well as Telltales, are to be placed in the North End Bent only

Construction Notes (for further details and clarification see the Job Special Provisions):

**Leveling Course** - Setting the first course of the facing block level and to grade is critical in maintaining wall alignment for the entire height of the abutment. Therefore a leveling course of fine aggregate shall be placed to provide a suitable surface for placing the lowest layer of CMU blocks. The leveling course shall be kept as close as possible to the minimum thickness stated on the plans. Leveling course for facing blocks shall be 6 inches wider than the facing blocks on each side, minimum.

**Concrete encasement** - This item is to be constructed 4" thick minimum and shall extend to the edge of the rock excavation adjacent to the wings and be a minimum of 3" beyond the edge of the level course elsewhere. Concrete encasement shall be placed to the limits shown above and as shown on sheet no. 14.

**Block Wall** - Each layer of the block wall shall be constructed entirely before beginning the next layer. A running bond pattern shall be maintained between layers of blocks. Geotextile placed under the Rip-Rap shall be anchored to the lowest two layers of blocks above the concrete encasement. Check the wall for plumbness a minimum of every three layers of blocks and correct any deviations greater than 1/4".

Pin and grout corner blocks for the entire height of the wall. See FHWA GRS-IBS Implementation Guide Section 7.7.6 for example of how to cut blocks at the corners of the wall.

**Select Granular Fill (AASHTO 89 Stone)** - The stone backfill shall be placed behind each layer of CMU blocks in a lift thickness not to exceed the CMU block height in Zones A & B and not to exceed 12" in the Integration Zone. Placement of the aggregate shall be from the wall face backward to prevent the formation of wrinkles in the geotextile. The backfill shall be compacted in accordance with the Job Special Provisions.

At the end of a day's operations, slope the last lift of backfill away from the wall face to direct surface runoff away from the wall. Do not allow surface runoff from adjacent areas to enter the wall construction area.

**Geosynthetic Reinforcement** - The geosynthetic layers shall extend between the layers of CMU blocks to provide a frictional connection. Pull the geosynthetic taut prior to backfilling to remove wrinkles. To limit construction damage to the geosynthetic, construction equipment shall not drive directly over the geosynthetic. An aggregate thickness of 6" is sufficient to prevent equipment from damaging the geosynthetic. No lapping of fabric shall be permitted along the face. Where lapped elsewhere, a 1/4" thickness of stone shall be spread between pieces of fabric. See Job Special Provisions for other details related to placing Geosynthetic Reinforcement.

**Beam Seat Construction** - Beam Seat shall be constructed as described in Section 7.8.1 of the FHWA GRS-IBS Implementation Guide. For Beam Seat elevations see sheet no. 18. Thickness of the beam seat zone is approximately 8" and consists of a minimum of two 4" lifts of wrapped-face GRS. Place precut 4" thick Closed Cell Foam on the top of the bearing bed reinforcement, butted against the back side of the CMU Facing Block. Set half-height CMU blocks on top of the Closed Cell Foam. Closed Cell Foam and Half-height CMU blocks are incidental to the Pay Item "Geosynthetic Reinforced Soil System (GRS)". Wrap 4" lifts across the beam seat. Before folding the final wrap, it may be necessary to grade the surface aggregate of the beam seat slightly high, to about 1/2", to aid in seating the footing and to maximize contact with the bearing area.

**Integration Zone Placement** - Following the placement of the superstructure beams, Separation Geotextile layers are to be placed along the back of the superstructure backwall, built in maximum lift heights of 6" (maximum vertical spacing of reinforcement ≤ 12"). The top of the final wrap should allow at least 2" of aggregate base cover over the geotextile to protect it from hot mix asphalt.

**BARTLETT & WEST**

**GEOSYNTHETIC REINFORCED SOIL SYSTEM**

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900/592  
BOONE COUNTY, MISSOURI

STATE OF MISSOURI  
TIMOTHY D. LEIF  
REGISTERED PROFESSIONAL ENGINEER  
NUMBER PE-201200778

THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

SEAL DATE: 08/29/14  
DESIGNED BY: TDL  
DRAWN BY: MSS  
APPROVED BY: RAG  
DESIGN PROJ: 16137.110  
SCALE: AS NOTED  
DATE: AUGUST/2014  
DRAWING NO: NONE  
SHEET NO: 15 of 22

1718 ROUTHROBE DR., SUITE 100 - JEFFERSON CITY, MO 65106-4000  
 PHONE: 877-884-1811 FAX: 877-884-7704  
 MISSOURI CERTIFICATE NO. 0000000107 - ENGINEERING  
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 8/29/14 Revised notes quantities pertaining to separation geotextile  
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\* For Beam Seat Elevations see Sheet No. 18

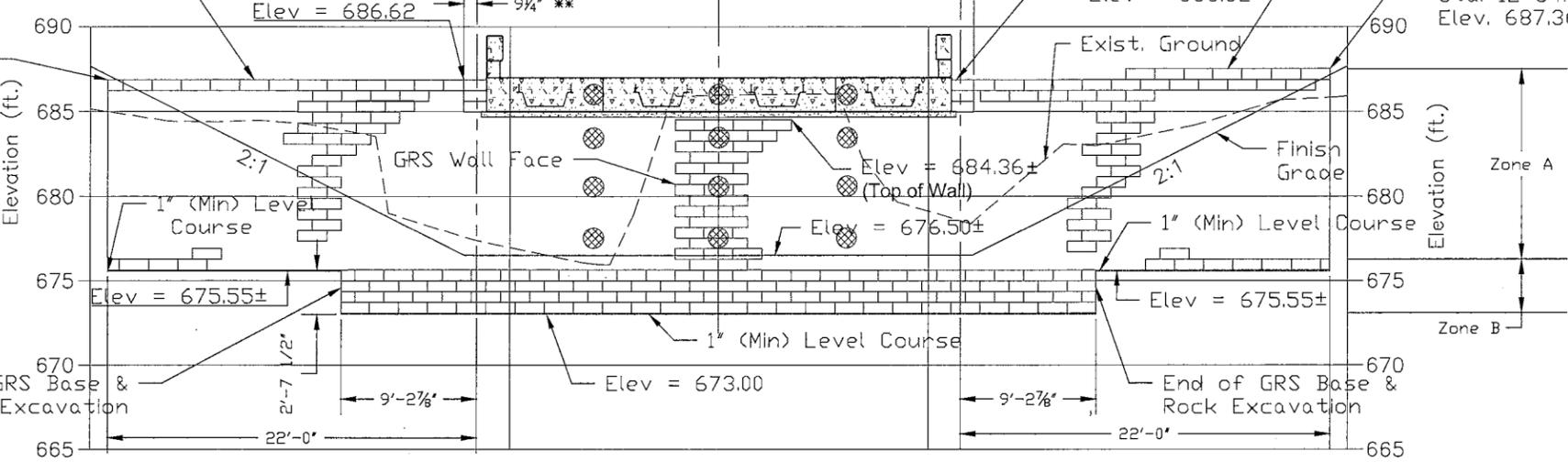
\*\* See Detail "A"

Construct a grout cap 1" thickness (min) on top of wings and slope to drain (Typ.)

End of Wing Sta. 12+19.57 Elev. 686.92±

Elev = 675.55±

End of GRS Base & Rock Excavation



ELEVATION VIEW - SOUTH ABUTMENT (Looking @ Front Face of Wall)

Notes: For Details pertaining to Zones A & B, see Job Special Provisions.

For additional construction notes associated with the block wall see the Job Special Provisions.

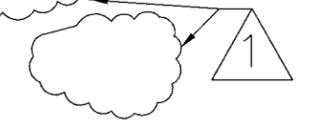
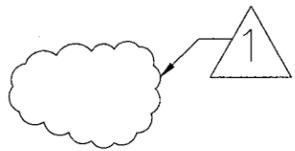
Notes: Work this sheet with sheets 15 & 18. For details of grout cap not shown, see sheet no. 18.

For details of Zones A and B and additional block wall details see Job Special Provisions. Top of Wall elevations shown do not include the grout cap along the front face or on the wings.

Bottom of Wing Wall may be stepped to avoid unnecessary rock excavation. A 1" min level course shall be constructed before placing blocks in the stepped sections. Stepped sections, if necessary, shall meet the approval of the Engineer.

For details of corner blocks see Sheet No. 16.

\*\*\* Trim or sawcut these blocks to provide a custom fit to the superstructure to prevent loss of fill material. If the gap between the superstructure and these facing blocks is difficult to fill with small sections of block a non-shrink grout shall be used to close the space per the approval of the Engineer.



Turning Point (at Top of Wall) Sta. 11+97.57 Offset = 12.5' LT

End Wall Sta. 12+19.57 Offset = 12.5' LT

Inside Face of CMU Blocks

Approximate GRS Area at Top of Wall

FF Sta. 12+09.60

€ Roadway

Inside Face of CMU Blocks

EOP

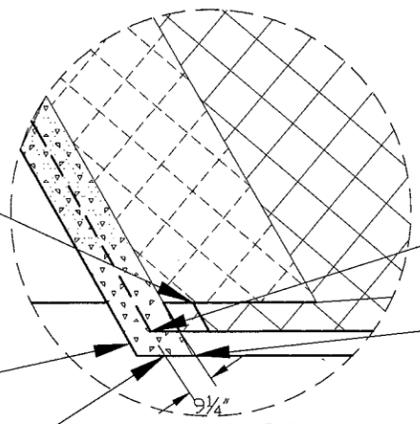
End of Wall Sta. 12+34.01 Offset = 12.5' RT

Turning Point (at Top of Wall) Sta. 12+12.01 Offset = 12.5' RT

PLAN VIEW - SOUTH ABUTMENT

Concrete Encasement at the Bottom of the Wall and Wall Taper are Not Shown for Clarity

Note: This drawing is not to scale. Follow dimensions.



Detail "A" Grout Cap on Wings not shown for clarity.

Front of Wall  
Outside Edge of Wingwall

Extend grout cap to outside edge of wingwall and slope to drain towards stream (Typ.)

Drawing name: W:\Proj\16000\16137\110\AutoCad\Plan Set\Abutments\_GRS\_Walls.dwg Layout name: Layout1 Plotted on: Aug 23, 2014 - 2:02pm

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**SOUTH ABUTMENT PLAN & ELEVATION**

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(592)  
BOONE COUNTY, MISSOURI

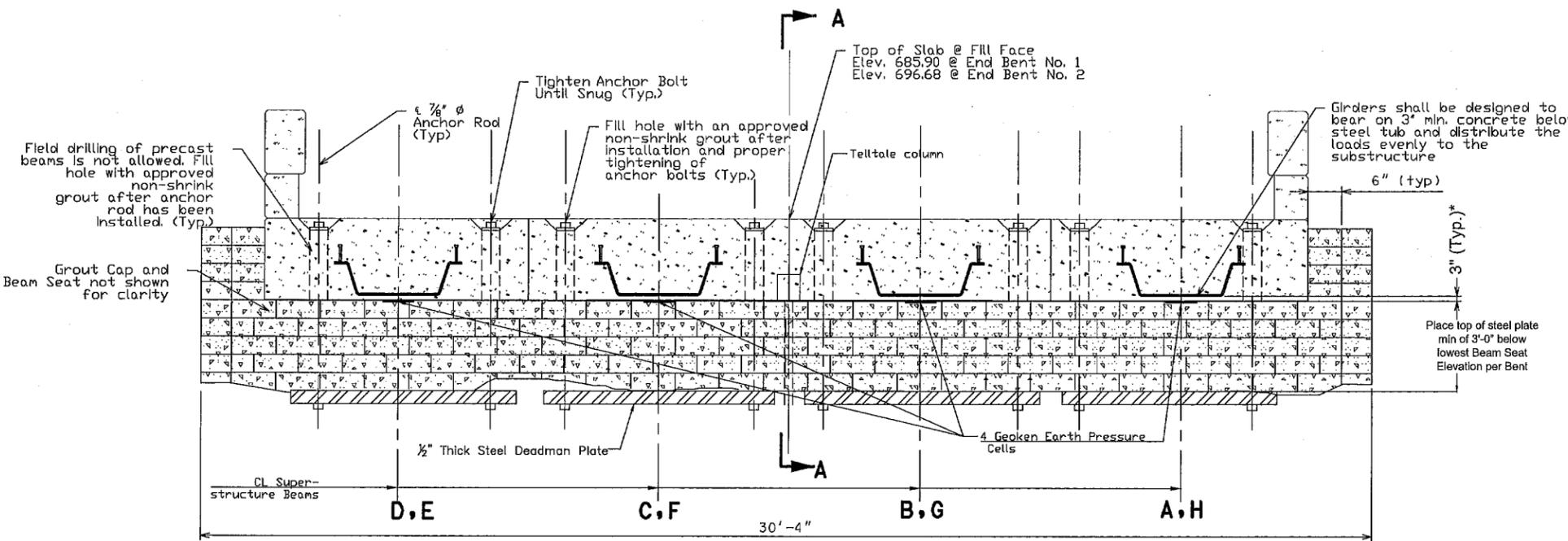
STATE OF MISSOURI  
TIMOTHY D. LEAF  
NUMBER PE-2012000778  
PROFESSIONAL ENGINEER

THIS SHEET HAS BEEN SIGNED, SEALED AND DATED ELECTRONICALLY.

SEALED DATE:	08/29/14
DESIGNED BY:	TDL
DRAWN BY:	MSS
APPROVED BY:	RAG
DESIGN PROJ.:	16137.110
SCALE:	AS NOTED
DATE:	AUGUST/2014
DRAWING NO.:	NONE
SHEET NO.:	17 of 22

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 Layout name: Layout1  
 Plotted on: Jul 22, 2014 1:46pm  
 Plot scale: 1:1  
 Lant set: 00/00/00



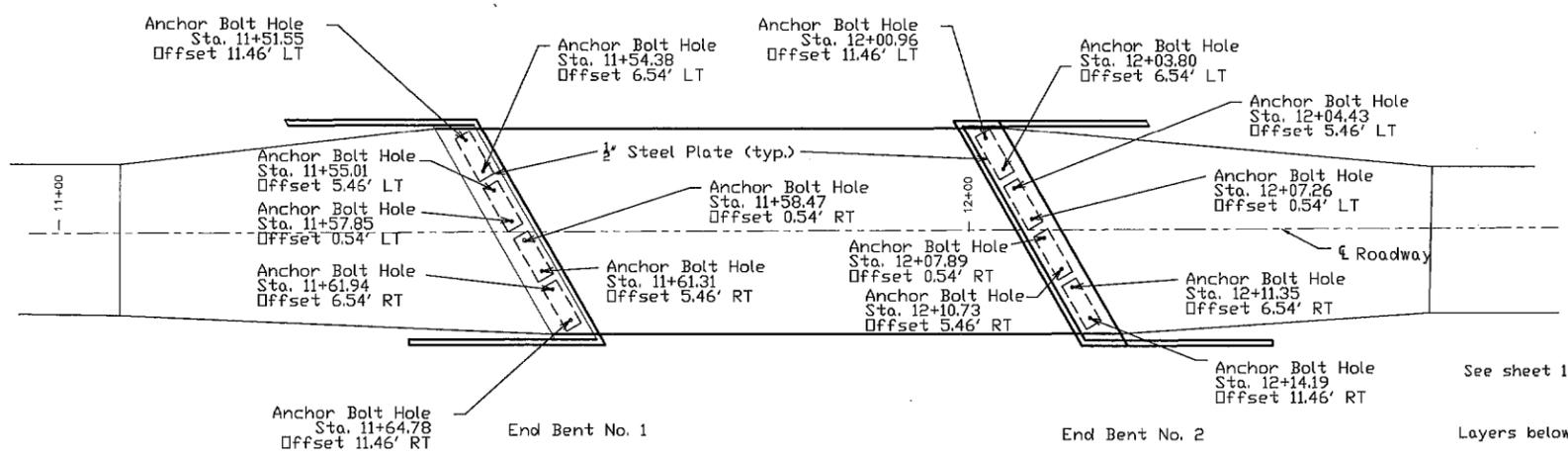
**SECTION NEAR END BENT**  
 Bent No. 1 Looking Back Station  
 Bent No. 2 Looking Ahead Station

CL BEAM SEAT ELEVATIONS FOR BENT 1	
Elevation A	683.46
Elevation B	683.55
Elevation C	683.59
Elevation D	683.64

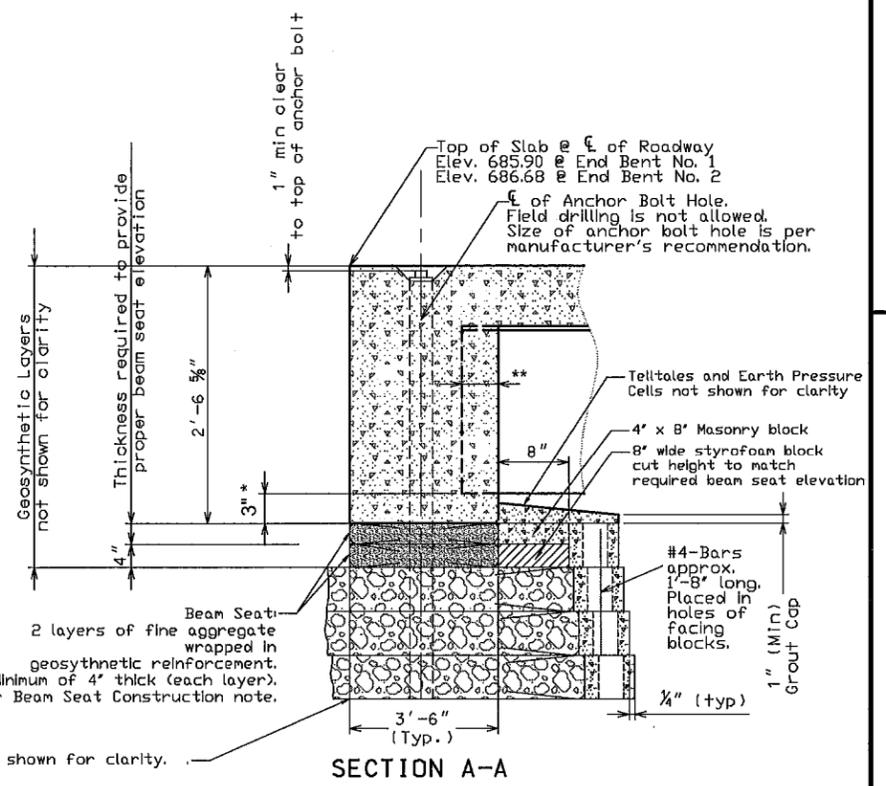
CL BEAM SEAT ELEVATIONS FOR BENT 2	
Elevation E	684.15
Elevation F	684.20
Elevation G	684.24
Elevation H	684.28

\* Concrete below the steel tub is necessary to provide a uniform bearing area to distribute loads from the superstructure to the substructure. This concrete shall be reinforced in order to prevent cracking. Reinforcement is to be designed by the manufacturer. Distance between the bottom of the steel tub and the bottom of the concrete backwall may be adjusted if deemed necessary by the manufacturer. If this dimension is altered, then the Beam Seat Elevations shall be adjusted accordingly.

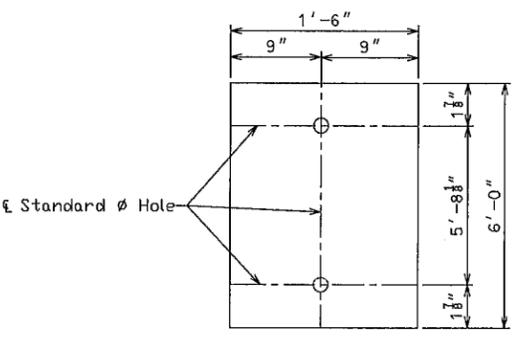
\*\* Embedment of steel tub into concrete backwall is per the manufacturer.



**PLAN SHOWING ANCHOR BOLT LOCATIONS**



**SECTION A-A**



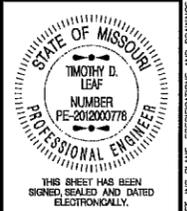
**1/2" Steel Deadman Plate Dimensions**

**Notes:**

- Work this sheet with sheets 14, 15, 16 & 17.
- Anchor rod nuts to be placed on the bottom of the steel deadman plates to be shop welded.
- All anchor rods, nuts and deadman plates shall be fabricated from ASTM A709 Grade 36 Steel.
- Anchor rods and associated hardware shall be galvanize according to ASTM A123.
- 1/2" steel deadman plates shall be cleaned and receive a heavy coating of an approved bituminous paint prior to final placement.
- Anchor bolt holes in the precast beams shall be oversized per the manufacturer's recommendations to allow for clearance of the anchor bolts, taking into account the high probability of field variations of the anchor bolt locations.
- Payment for any material and labor required to construct the Deadman Anchorage System, including the anchor rods, will be considered completely covered by the lump sum price for Geosynthetic Reinforced Soil System (GRS).
- For estimated quantities of Deadman Anchorage System see Sheet No. 15.
- Holes may be cut in the geosynthetic reinforcement material to facilitate placement of anchor rods. Only remove material necessary to fit geosynthetic reinforcement around anchor rods. No slits are to be cut in the geosynthetic reinforcement. Follow the manufacturer's recommendations for approved methods.
- Fill top 3 rows of blocks along front face of abutments with an approved non-shrink grout after placing the #4 bars. Continue placing grout for the 1" minimum thickness grout cap on top of blocks as shown, without allowing a cold joint to form between grout cap and grout applied to fill the blocks.
- Fill top 3 rows of blocks along wings of abutments and top 3 rows of blocks adjacent to the superstructure with an approved non-shrink grout after placing the #4 bars. After grout has cured place the capstones on top of the wings as shown in the details on Sheets 16 & 17.
- For details of instrumentation devices not shown, see Sheet No. 14 and the Job Special Provisions.

Note: Drawing is not to scale. Follow dimensions

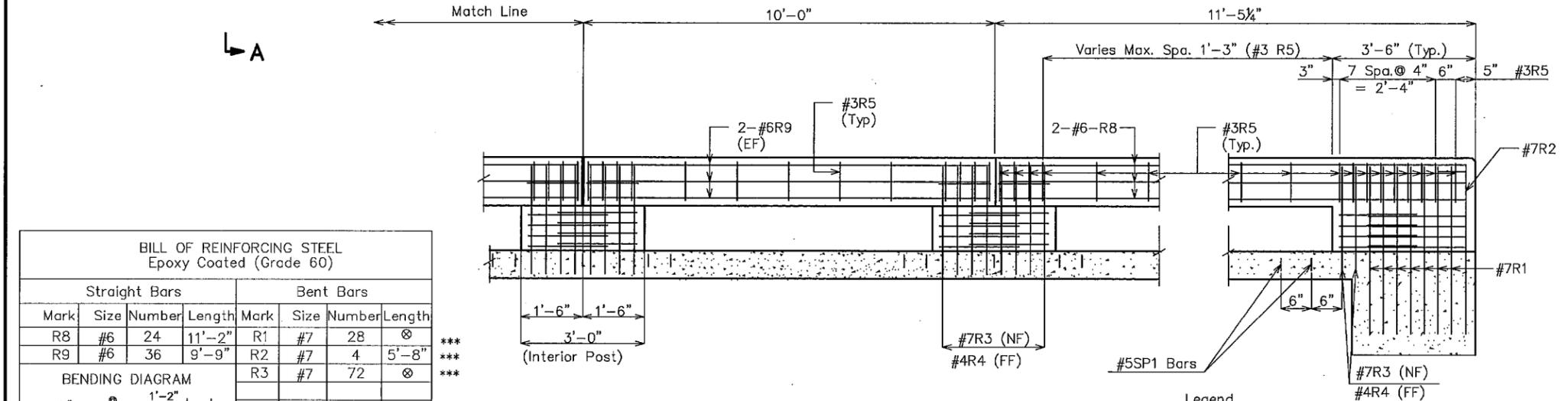
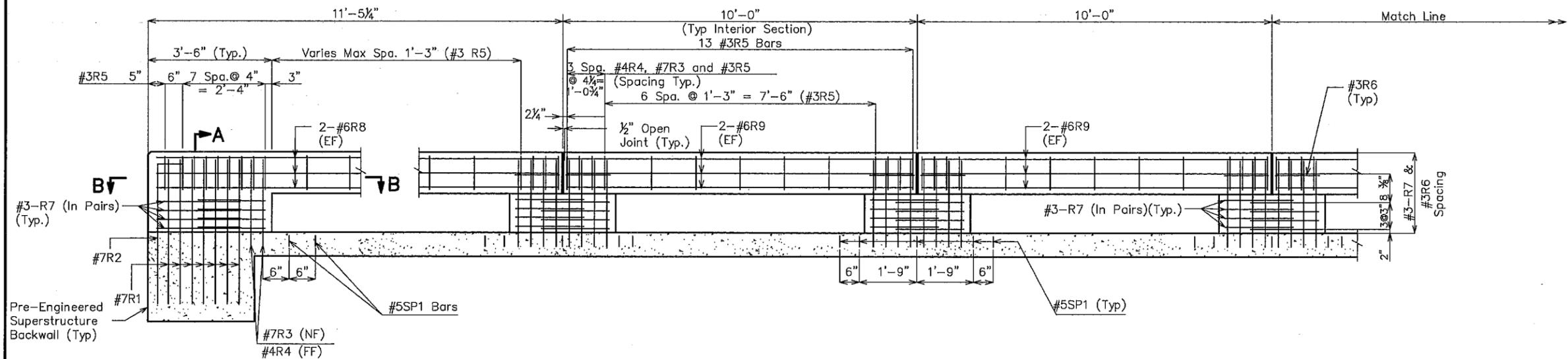
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**DETAILS OF END BENTS AND DEADMAN ANCHORAGE**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI



SEALED DATE:	07/22/14
DESIGNED BY:	TDL
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APPROVED BY:	RAG
DESIGN PROJ.:	16137.110
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DATE:	JULY/2014
DRAWING NO.:	NONE
SHEET NO.:	18 of 22



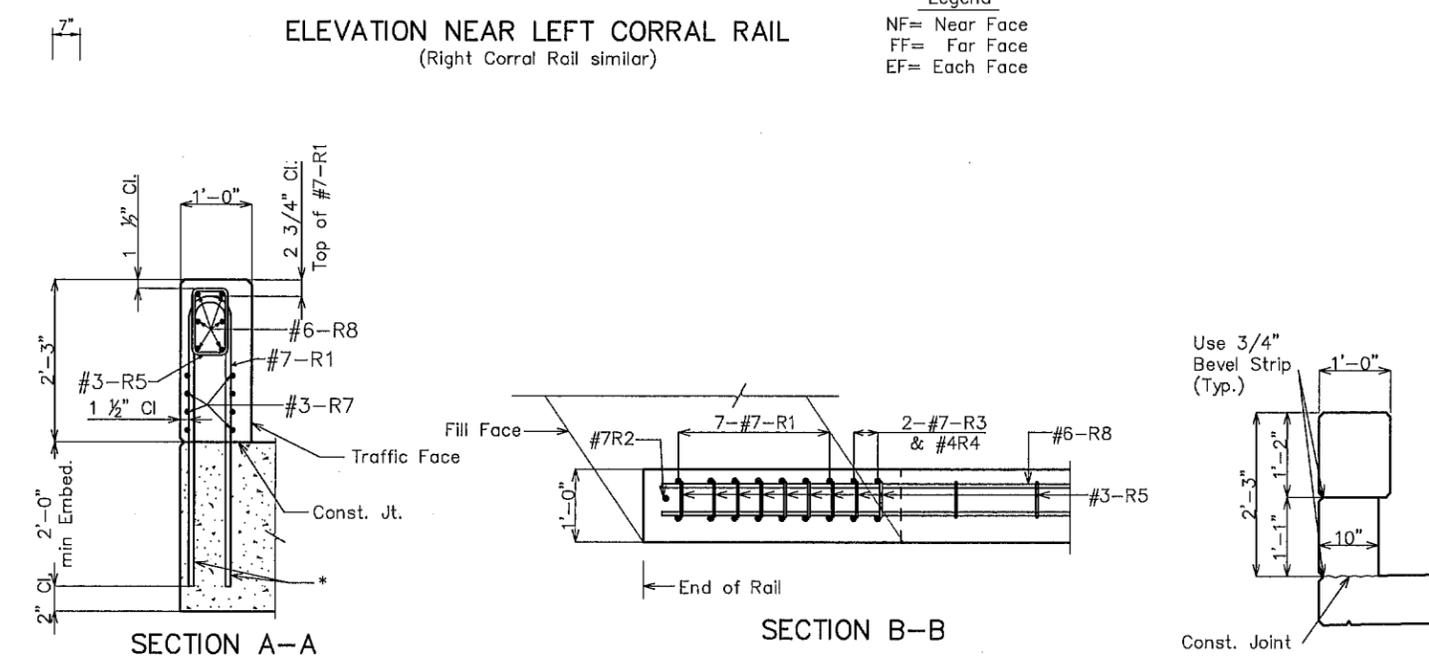
Drawing name: W:\Proj\16000\16137\16137.110\Misc\Corral\_Rail.dwg Layout name: Corral Rail Plotted on: Jul 22, 2014 1:46pm  
 Last self on: 00/00/00



**BILL OF REINFORCING STEEL**  
Epoxy Coated (Grade 60)

Straight Bars				Bent Bars			
Mark	Size	Number	Length	Mark	Size	Number	Length
R8	#6	24	11'-2"	R1	#7	28	⊗
R9	#6	36	9'-9"	R2	#7	4	5'-8"
				R3	#7	72	⊗
				SP1	#5	48	⊗
				R4	#4	72	⊗
				R5	#3	154	4'-4"
				R6	#3	16	4'-6"
				R7	#3	96	4'-8"

**BENDING DIAGRAM**



**Notes:**

Concrete in the Corral Rails shall be Class B-1.

All reinforcement in Corral Rail shall be epoxy coated in accordance with Sec 710.

Payment for all concrete and reinforcement, complete-in-place, except for reinforcement that will be embedded in the precast beams, will be considered completely covered by the contract unit price for "Corral Rail" per linear foot.

Measurement of Corral Rail is to the nearest linear foot measured along the outside top of slab from Fill Face End Bent No. 1 to Fill Face End Bent No. 2.

Concrete traffic barrier delineators shall be placed on top of the Corral Rail as shown on Missouri Standard Plans 617.10 and in accordance with Sec 617. Concrete traffic barrier delineators will be considered completely covered by the contract unit price for "Corral Rail".

\*#7R1 & #7R2 Bars may be hooked on low end to provide the equivalent of 2'-0" min embedment of bars into precast beam.

\*\*The hook may be canted to provide clearance and/or fit between reinforcing.

\*\*\*Reinforcing is to be designed and provided by the precast girder manufacturer.

⊗ To be determined by manufacturer  
 ⊗ Bend this leg to match the slope of the roadway

**END POST DETAILS**

**TYPICAL INTERIOR POST SECTIONS**

Notes This Drawing is Not to Scale. Follow Dimensions



**CORRAL RAIL**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI



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 DESIGN PROJ: 16137.110  
 SCALE: AS NOTED  
 DATE: JULY/2014  
 DRAWING NO: NONE  
 SHEET NO: 20 of 22

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**BORING LOG NO. B-1**

Page 1 of 1

**PROJECT:** Rustic Road Bridge **CLIENT:** Bartlett & West Inc.  
**SITE:** Rustic Road at North Fork Grindstone Creek  
 Columbia, Missouri

DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (in.)	FIELD TEST RESULTS	SAMPLE NUMBER	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	ATTERBERG LIMITS	PERCENT FINES
0.0								LL-PL-PI	
1.5			8	8-1-1 N=2	1		10		
3.4			0	50/0" N=50/0"	2				

Stratification lines are approximate. In-situ, the transition may be gradual. 2 feet at completion of drilling. Hammer Type: Automatic SPT Hammer

Advancement Method: 4" Continuous flight auger  Abandonment Method: Boring backfilled with soil cuttings upon completion.	See Exhibit A-8 for description of field procedures  See Appendix B for description of laboratory procedures and additional data (if any).  See Appendix C for explanation of symbols and abbreviations. Elevations were measured in the field using an engineer's level and grade rod.	Notes:  Boring Started: 1/22/2013 Drill Rig: CME-45 Project No.: 09125182	Boring Completed: 1/22/2013 Driller: S. Becker Exhibit: A-4
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THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL\_09125182.GPJ TERRACON2012.GDT 1/30/13

Drawing name: W:\Proj\16000\16137\110\AutoCAD\Plan\_Set\Gratech\_Boring.dwg Layout name: Gratech-BORE Plotted by: ttd01295 Plotted on: Jul 22, 2014 1:47pm

**BORING LOG NO. B-2**

Page 1 of 1

**PROJECT:** Rustic Road Bridge **CLIENT:** Bartlett & West Inc.  
**SITE:** Rustic Road at North Fork Grindstone Creek  
 Columbia, Missouri

DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (in.)	FIELD TEST RESULTS	SAMPLE NUMBER	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	ATTERBERG LIMITS	PERCENT FINES
0.0								LL-PL-PI	
0.3		TOPSOIL, 3"							
1.5		SANDY LEAN CLAY (CL), brown, medium stiff							
3.6			10	2-2-2 N=4 2000 (HP)	1		22		62
5.0			8	7-4-7 N=11	2		21		
6.0		SAND (SW), with lean clay and gravel, brown and dark brown, medium dense							
6.5		LIMESTONE, light gray, slightly weathered, hard -- Practical auger refusal at 6.5 feet							
11.5			60	RQD = 68%	R1		4100 3800		

Stratification lines are approximate. In-situ, the transition may be gradual. Hammer Type: Automatic SPT Hammer

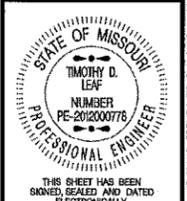
Advancement Method: 0' to 6.5': 4" Continuous flight auger 6.5' to 11.5': NQ2 diamond bit core  Abandonment Method: Boring backfilled with soil cuttings upon completion.	See Exhibit A-8 for description of field procedures  See Appendix B for description of laboratory procedures and additional data (if any).  See Appendix C for explanation of symbols and abbreviations. Elevations were measured in the field using an engineer's level and grade rod.	Notes:  Boring Started: 1/22/2013 Drill Rig: CME-45 Project No.: 09125182	Boring Completed: 1/22/2013 Driller: S. Becker Exhibit: A-5
--	--	---	---



THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL\_09125182.GPJ TERRACON2012.GDT 1/30/13



**GEOTECHNICAL DATA - BORING LOG**  
**SHEET NOS. 1-2**  
 RUSTIC ROAD BRIDGE REPLACEMENT  
 FEDERAL PROJECT NO. IBRD 9900(592)  
 BOONE COUNTY, MISSOURI

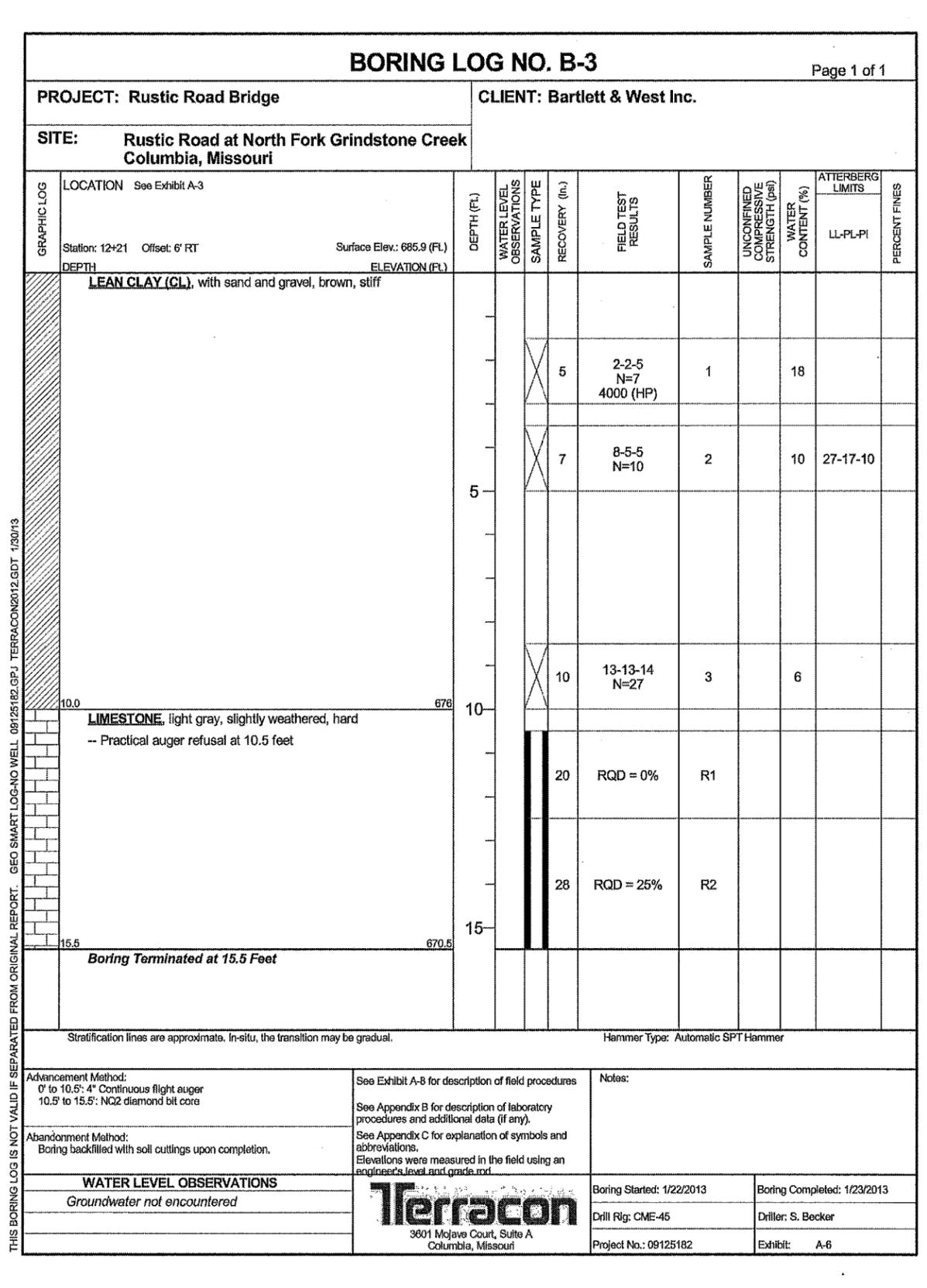
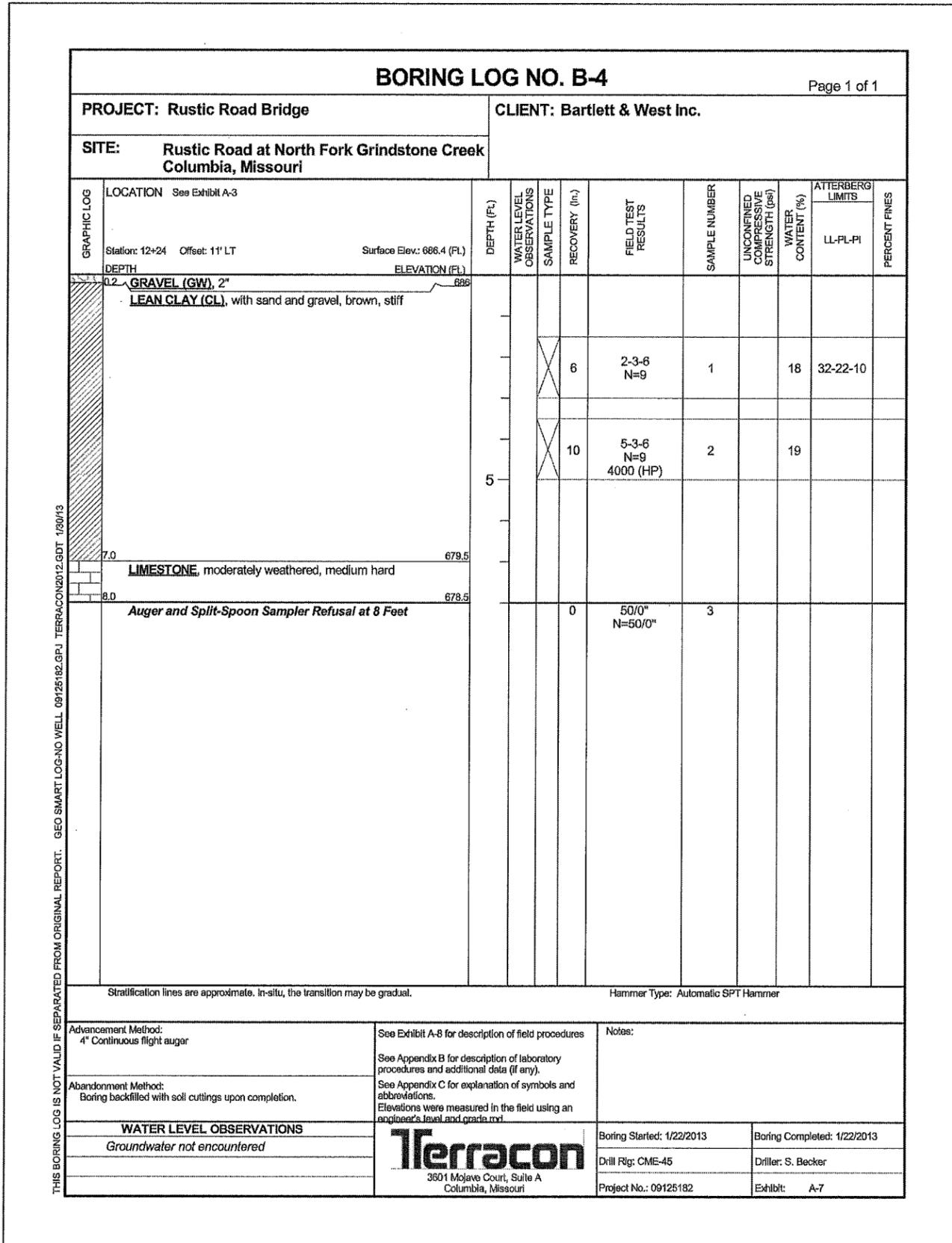


SEALING DATE: 07/22/14  
 DESIGNED BY: TDL  
 DRAWN BY: MSS  
 APPROVED BY: RAG  
 DESIGN PROJ: 16137.110  
 SCALE: AS NOTED  
 DATE: JULY/2014  
 DRAWING NO: NONE  
 SHEET NO: 21 of 22

Note: Drawing is not to scale. Follow dimensions

Drawing name: W:\Proj\18000\16137\16137-110\AutoCAD\Plan Set\Geotech\Boring.dwg Layout name: Geotech1 (2) Plotted by: tel01295 Plotted on: Jul 22, 2014 - 1:47pm

Last edit on: 07/09/09



Note: Drawing is not to scale. Follow dimensions



**GEOTECHNICAL DATA - BORING LOG  
SHEET NOS. 3-4**

RUSTIC ROAD BRIDGE REPLACEMENT  
FEDERAL PROJECT NO. IBRD 9900(592)  
BOONE COUNTY, MISSOURI



SEAL DATE: 07/22/14  
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 SCALE: AS NOTED  
 DATE: JULY/2014  
 DRAWING NO: NONE  
 SHEET NO: 22 of 22

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