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PERFORMANCE MONITORING AND ANALYSIS OF GEOSYNTHETIC REINFORCED SOIL INTEGRATED BRIDGE SYSTEMS (GRS-IBS) IN OKLAHOMA

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PERFORMANCE MONITORING AND ANALYSIS OF GEOSYNTHETIC REINFORCED SOIL INTEGRATED BRIDGE SYSTEMS (GRS-IBS) IN OKLAHOMA

A THESIS APPROVED FOR THE
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Dedicated to my professors, my family, my wife and to the very life, that never stops showing me the right path.

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Abstract

In 2013, the University of Oklahoma, together with the Oklahoma Department of Transportation (ODOT) and the Oklahoma Bureau of Indian Affairs (BIA), started a feasibility study of GRS-IBS, which had been promoted by the FHWA as a cost-effective solution to repair and/or rebuild bridges with spans that were primarily shorter than 25 m (80 ft). This technology also eliminates the "bump at the end of the bridge", which reduces the maintenance/repair cost of bridges without an integrated approach roadway. The purpose of this study was to continue the work by Hatami et al. (2016) and Ngo (2016) by performing the following tasks: (1) continuing the survey of documented GRS-IBS in the U.S., (2) performance (i.e. settlement) monitoring of six bridges (i.e. four GRS-IBS and two conventional) that were built in Kay County, OK within a one-mile segment of 44th Street near Blackwell, OK (Hatami et al., 2016; Ngo, 2016); (3) developing a numerical model for the analysis of GRS-IBS systems; and (4) developing the framework for an interactive online database for all of the documented GRS-IBS projects surveyed in this study (some 144 projects). The database of documented GRS-IBS projects in the U.S. (with the ancillary online website upon completion) together with the numerical simulation tool is helpful to ODOT and other departments of transportation in examining the costs and benefits of GRS-IBS as a potential solution for future bridge construction projects in Oklahoma and other states. Currently, GRS-IBS has been proven to be a cost-effective solution for bridge spans less than 25 m (80 ft) on county and local roads only. However, it is expected that through further development of this technology, and continued reports of its successful performance, its use and acceptance across the country will become more widespread.

Chapter 1. Introduction

1.1 Background

According to the 2015 National Bridge Inventory (FHWA 2016), 58,495 bridges are classified as structurally deficient. This number represents 9.6% of the total national inventory. The state of Oklahoma has been classified for three consecutive years (2013-2015) as the third state with the most structurally deficient bridges. From a total of 23,049 bridges reported for this state, 3,776 are classified as deficient. This number denotes 16.4%. Additionally, many existing bridges in the U.S. are not only structurally deficient but functionally obsolete (ASCE 2013). Another important issue, even with bridges that are currently in service, is the formation of a bump at the transition to the roadway, which stems from differential settlements between the abutments and the approach embankment. As early as 1997, state Departments of Transportation (DOTs) across the United States have collectively been contributing more than one hundred million dollars in research annually to try to resolve this problem on 150,000 bridges across the nation (Briaud et al 1997). In 2009, after the 2008 economic crisis, GRS-IBS became part of a program launched by the Federal Highway Administration (FHWA) in cooperation with the American Association of State Highway and Transportation Officials (AASHTO) called Every Day Counts (EDC); whose main initiative is to speed up the delivery of highway projects, and encourage creative and innovative solutions to address the challenge of dealing with very limited budgets. Geosynthetic Reinforced Soil – Integrated Bridges (GRS-IBS) emerged among the different successful technologies recognized in EDC-1, EDC-2 and EDC-3 because the technology provided an efficient, innovative, and economic solution to the reconstruction of many short, single-span bridges in lessened time (Adams et al 2011; Adams et al. 2012).

1.2 Need for the Study

Finding long-lasting, stable, cost-effective, and efficient solutions to transportation-related problems is a priority for any of the federal agencies working with roads and highways, especially in periods of more limited budgets. Considering this, GRS-IBS technology has proven to be an effective approach for constructing new bridges or replacing deficient ones located on local and rural roads. Specifically, in the state of Oklahoma, where rural roadways compose more than 50% of the state roads, GRS-IBS provide an achievable solution to the budgetary issues associated with building new bridges or replacing old ones in many county and local roadways. Therefore, from the investigation point of view, there is a need to cover more extensively the advantages, challenges, and fundamentals of GRS-IBS technology. Hence, this study emerged as a need for a better comprehension of this technique based on three main reasons: First, to determine the feasibility of GRS-IBS bridges in Oklahoma, a comprehensive study of all the GRS-IBS bridges successfully built in the U.S. needs to be performed, and a database with the bridge locations and characteristics need to be constructed in order to gather all the information available from them. It is necessary that this database can be easily accessed and maintained in a web-page environment; Secondly, as GRS-IBS has already been used in several small bridges in Oklahoma, there is a need to monitor and document these projects over time to determine the feasibility of this technology in future projects. Furthermore, there is a need to determine if the current FHWA guidelines are adequate, or need to be adapted when implementing GRS-IBS technology for building future bridges in Oklahoma; Finally, a numerical model was developed to analyze the influences of selected design factors on the performance of GRS-IBS bridge systems.

1.3 Study Objectives and Tasks

The main objective of this thesis was to study the feasibility and cost efficiency of GRS-IBS projects, relative to conventional deep foundation bridges on selected county roads in Oklahoma. The scope of this study primarily includes a side-by-side comparison of four (4) GRS-IBS projects with two (2) conventional bridges in close vicinity of one another in Kay County, OK, which provided a unique field study within the United States. To achieve this purpose, the following tasks were defined and performed:

- Gathered and included information on 15 additional documented cases from across the U.S. in the database of GRS-IBS projects that had been developed in collaboration with Ngo (2016).
- Collected all the available information on the GRS-IBS bridges in Kay County and Lincoln County, including design plans, materials, geotechnical reports, construction periods, cost, construction, and performance monitoring, through local users and engineers' feedback. Two methods were used to obtain this information: (1) Direct contact with Mr. Tom Simpson, P.E., at the Bureau of Indian Affairs (BIA) in Anadarko, Oklahoma, and (2)

on-site documentation and performance monitoring, before, during and after the construction.

- Carried out a monitoring program to measure and document the serviceability performance of the four (4) GRS-IBS and two (2) conventional bridges in Kay County, OK by periodically visiting the sites and surveying the bridges.
- Developed a numerical model that upon further development and validation can be used as a design tool for GRS-IBS bridges based on bridge geometry and initial designing parameters.

1.4 Thesis Layout

This thesis is comprised of six chapters. Chapter 1 includes the background, need for the study, study objectives and the layout of the thesis.

Chapter 2 provides an introduction and background on the GRS-IBS technology, including its design requirements, FHWA guidelines (Adams et al. 2012), advantages, limitations, and design requirements. Selected case studies are provided for a more extensive explanation of the performance of the GRS-IBS systems.

Chapter 3 includes details on the development of a database and an ancillary webpage, which contain information on a total of 144 documented GRS-IBS from across the U.S., including cost, facing type, superstructure, average daily traffic, and performance monitoring measures.

Chapter 4 discusses monitoring of two (2) conventional and four (4) GRS-IBS bridges in Kay County, OK, as well as brief documentation of one (1) GRS-IBS bridge in Lincoln County, OK.

Chapter 5 provides a detailed description of the numerical model developed using the Fast Lagrangian Analysis of Continua (FLAC) Software (Itasca, 2011), and the results of a parametric study.

Finally, Chapter 6 summarizes the findings and conclusions of the previous five chapters and provides recommendations for future work according to this study.

Chapter 2. Literature Review

This chapter consists of a general explanation of GRS-IBS technology. It covers a literature review that includes a description of the method, its benefits, basic elements, and requirements. It concludes by presenting several case studies of GRS-IBS projects in the U.S.

2.1 Background on GRS-IBS

In ancient times, constructors used straw and plant matter to improve soil's tensile strength (Adams et al. 2012). This technique evolved over the years and brought about mechanically stabilized earth walls (MSE) in the 1960s and geosynthetic reinforced soil abutments (GRS) in the 1980s. Although both technologies use reinforcement for cost reduction and improve the tension within soil structures, MSE is affected by steel corrosion over time. Geosynthetic Reinforced Soil-integrates Bridge System (GRS-IBS) is a fast, cost-effective method of bridge support that merges the roadway into the superstructure to eliminate joints between the bridge and the approach slab. This method of accelerated bridge construction (ABC) involves the following three basic components: (1) modular facing, (2) compacted granular fill, and (3) tightly-spaced geosynthetic reinforcement layers. In addition: GRS reduces lateral deflection, limits dilation, and enhances confinement. GRS-IBS emerged recently as a cost and time saving bridge construction technique. It was selected as part of the FHWA EDC-1, EDC-2, and EDC-3 initiatives (**Figure 1**; Alzamora et al. 2015). Additionally, it has been used in the construction or reconstruction of over 250 bridges across the U.S.

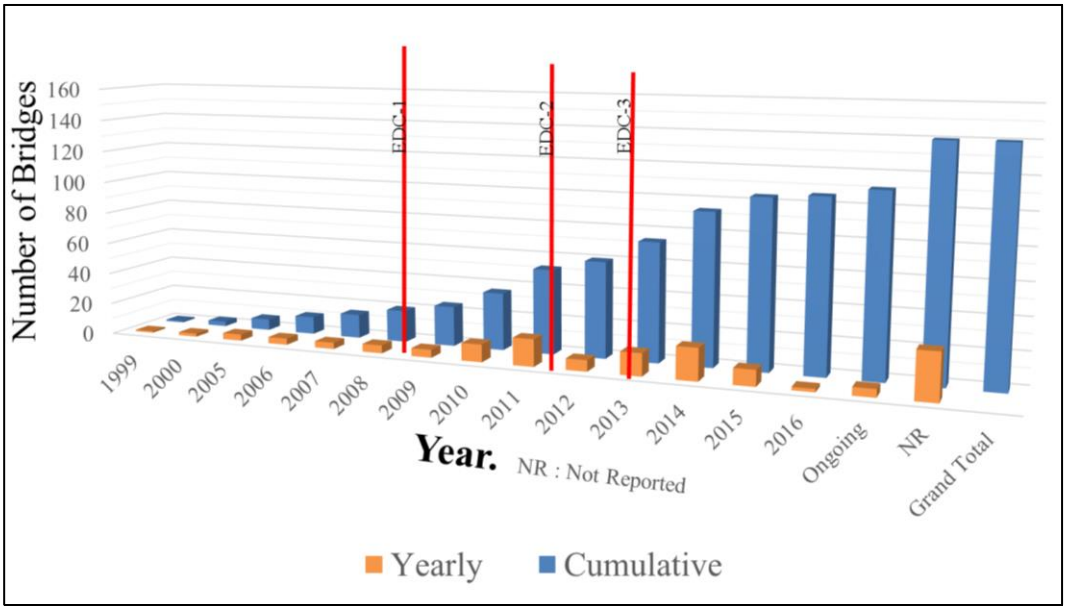


Figure 1: Yearly and cumulative number of GRS-IBS bridges constructed vs year

2.2 GRS-IBS Adams et al. (2012) guidelines

GRS-IBS has three main components **(Figure 2**): (1) the integrated approach, which incorporates the approach section of the roadway with the bridge superstructure, in order to generate a joint-less transition between the bridge and the roadway; (2) the abutment, which is comprised by modular facing elements, compacted granular fill and a set of tightly-spaced geosynthetic reinforcement layers which will be described in the following sections; and (3) the reinforced soil foundation (RSF). Additionally, Adams et al. (2012) recommends that GRS-IBS bridges should be designed using the AASHTO LRFD method (AASHTO 2014). The dead loads on the GRS abutment include the weights of the bridge superstructure, roadway pavement, and the integrated approach. Surcharge load includes the structural backfill of the road base. The traffic and truck loads are included as a live load on the approach pavement and the bridge superstructure.

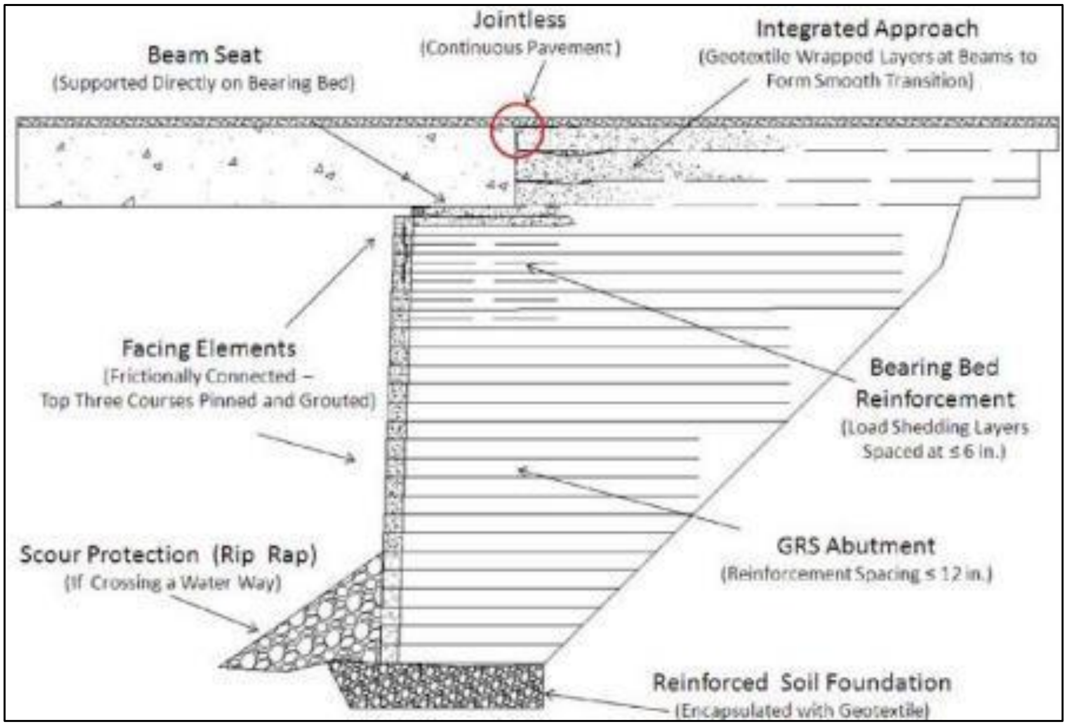


Figure 2: Typical GRS-IBS cross section (Adams et al. 2012)

The backfill of the abutments is design to support traffic load in which backfill contributes significantly. Open or well-graded aggregate, (**Figure 3**) or a combination of both can be used as backfill material. Adams et al. (2012) recommends that it is better if the aggregate is angular, which will provide greater shear strength, and that its gradation should allow for optimum compaction, workability, and drainage. Additionally, either aggregate used (**Figure 4**) should be compacted to achieve a minimum of 95% of maximum dry unit weight based on the AASHTO T-99 (Standard Proctor) procedure. It is common practice to use open-graded type rather than well-graded type for the RSF and abutment backfill.

Figure 3: (a) well-graded 21-A gravel; (b) Open-graded AASHTO No. 89 gravel (Adams et al. 2012)

	U.S. Sieve Size	Percent Passing
Gradation (VDOT 21-A)	2 inch	100
	1 inch	94–100
	3/8 inch	63–72
	No. 10	32–41
	No. 40	14–24
	No. 200	6–12
Plasticity Index (PI) (AASSHTO 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).	

	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).	

Figure 4: Requirements for GRS abutment backfill: (a) Well-graded (VDOT 21-A); (b) Open-graded (AASHTO No. 89)

The abutment facing element serves two purposes: provide a formwork for backfill compaction and also serves as facade that protects the granular fill from outside weathering. The split face concrete masonry unit (CMU), with nominal dimensions of 203.2 mm × 203.2 mm × 406.4 mm (8 in. × 8 in. × 16 in.), is the most commonly used facing element for the GRS abutment (**Figure 5**). This facing element is required to have a minimum compressive strength of 27,579 KPa (4,000 psi) and a water absorption limit of 5%. Finally, for proper construction of the abutment, it is necessary that the first row of facing blocks is properly aligned and that the backfill directly behind the facing is adequately compacted.

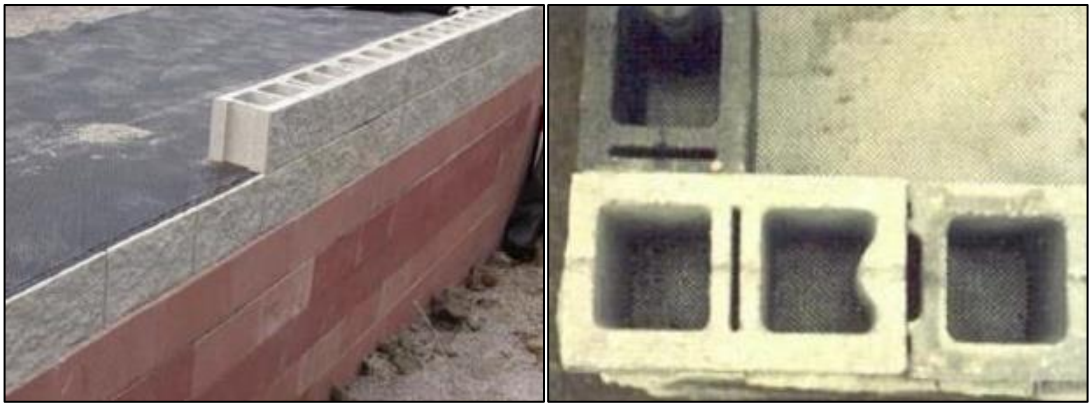


Figure 5: Split-Face CMU hollow blocks for GRS abutment systems (Adams et al. 2012)

Adams et al. (2012) recommends the use of geotextiles for the integrated approach, and geotextiles or a geogrid for the GRS abutments. The reinforcement must have a minimum ultimate strength of 70KN/m (4,800 lb/ft.) Several reported projects have been built using biaxially woven polypropylene geotextiles for the reinforcement. Additionally, Alzamora (2014) suggested the following construction guidelines that contractors should be aware of when placing geosynthetic reinforcement: (1) It must be rolled out with strong direction perpendicular to the abutment face, (2) wrinkles must be removed and material cannot overlap, especially at the facing, (3) it must be extended to connecting devices inside the facing, or to a minimum of 75% of the block width, (4) it must not be overlapped or tied, and (5) it must be trimmed at the facing of the blocks.

2.3 Advantages and Limitations

GRS-IBS technology has several major advantages over conventional construction techniques including cost, efficiency, and environmental benefits. The following features summarizes its advantages: (1) it's an environmentally friendly technique with minimal impact on the environment, (2) it eliminates the need for installing deep foundations or cast-in-place (CIP) concrete, (3) the cost savings can potentially be between 25% and 60% compared to that of conventional bridges, (4) reduced construction time, (5) uses readily available materials and equipment, (6) eliminates the "bump" at the end of the bridge, creating a smoother and safer transition from the bridge to the road, (7) improved durability, (8) flexible design that can be easily field-modified for unforeseen site conditions, and (9) improved seismic performance (Adams et al. 2012; Alzamora 2015), among others. These benefits make this technology a feasible alternative to the conventional methods of constructing new bridges, or replacing old ones. However, **Table 1** shows limitations presented by Adams et al. (2012)

Table 1: Summary of GRS-IBS guidelines form Adams et al. (2012)

Specification	Recommendation	Reference
Abutment height	*Less than 30 ft.	Adams et al. 2012, p.25
Maximum span length	*140 ft.	Adams et al. 2012, p.25
Reinforcement spacing	*Less than 12 in.	Adams et al. 2012, p.27
Facing elements	*A common option includes (8 in. × 8 in. × 16 in.) Concrete Masonry Units (CMU) with a minimum compressive strength of 4,000 psi and water absorption limit of 5%	Adams et al. 2012, p.16
GRS abutment backfill	*Friction angle (∅) should be no less than 38^{o} *Maximum aggregate size between 0.5 in and 2 in with fines content less than 12%	Adams et al. 2012, p.18
Geosynthetic	*Should be at least 4,800 lb/ft for GRS load-bearing	Adams et al.

Specification	Recommendation	Reference
ultimate tensile strength	applications	2012, p.21
Design code	*All federal-aid funded projects should be designed using the AASHTO Load and Resistance Factor Design (LRFD) method	Adams et al. 2012, p.27
Depth of excavation for the RSF (DRSF)	*Should equal one-quarter the total width of the GRS abutment base including the block face *Additional excavation may be necessary depending on the soil condition (e.g., compressible soils) and should be determined by the engineer	Adams et al. 2012, p.27
Potential scour	*FHWA Hydraulic Engineering Circulars, (HEC 23): recommended for smaller, more culvert-like structures (flow length through structure is longer than structure width) *FHWA Hydraulic Engineering Circulars, (HEC 18 and 20): Recommended for larger, more bridge-like structures (opening length is greater than the flow length through the structure)	Adams et al. 2012, p.32
Seismic design	*GRS abutments are expected to perform well in medium earthquakes.	Adams et al. 2012, p.73

2.4 Design Requirements

Figure 6 shows the recommended design steps by Adams et al. (2012).

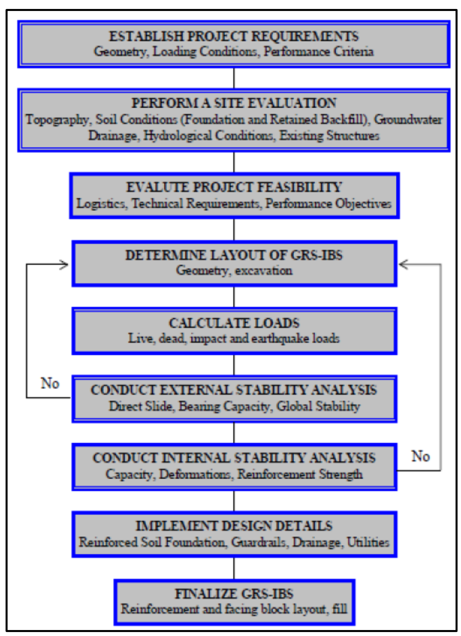
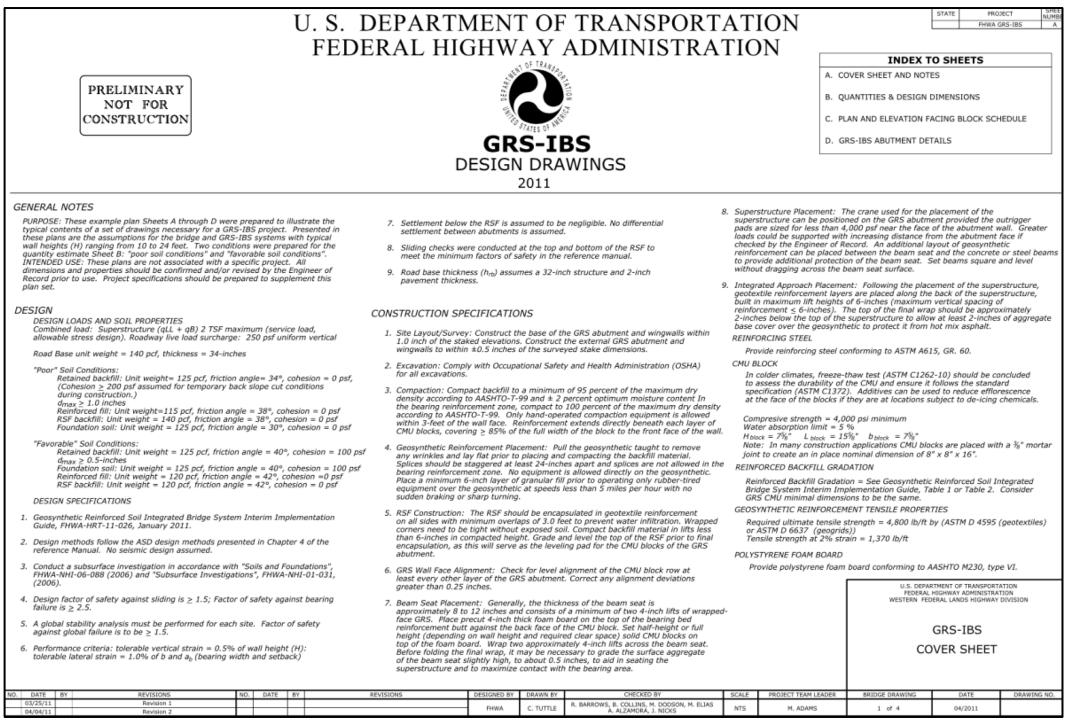
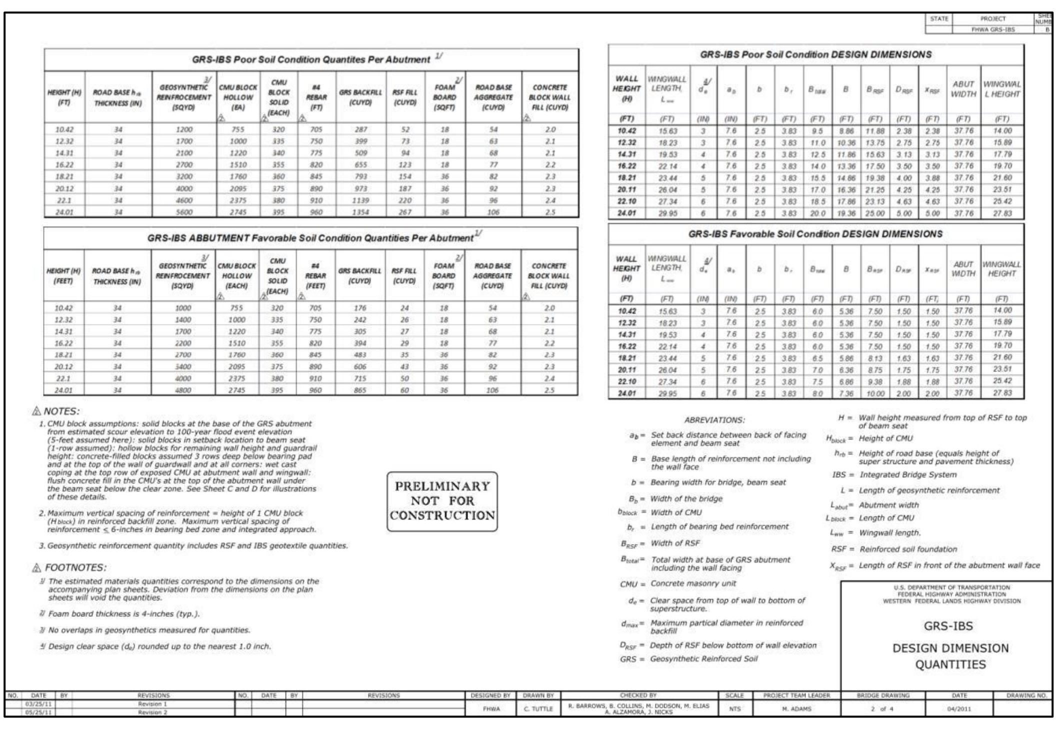


Figure 6: Flowchart of FHWA recommended design steps for GRS-IBS (Adams et al. 2012)

Figure 7 shows a summary of standard plans reported by Adams et al. (2012).





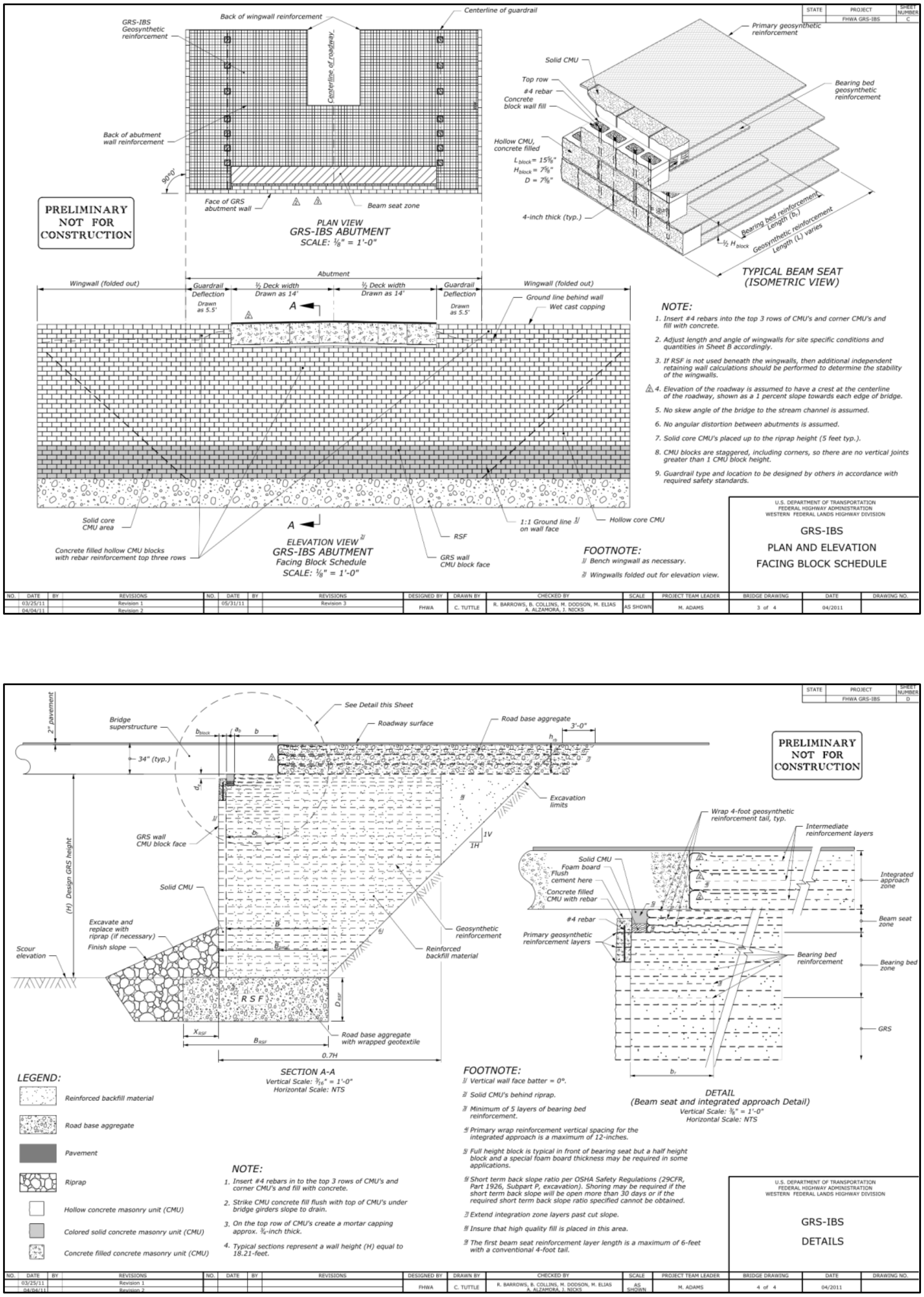


Figure 7: GRS-IBS standard plans (Adams et al. 2012)

2.5 Case Studies

This research project started in summer of 2013. The majority of results and case studies where published by Ngo (2016), Hatami et al. (2015), and Hatami et al. (2016). The author of this document contributed to the initial database by optimizing and maintaining it since January 2015, when joining the research group. Additionally, the author started the development of an online tool which will be explained in Section *3.1 – Webpage-based Database of GRS-IBS Projects in the U.S.* To date, we identified three (3) ongoing and 144 completed GRS-IBS bridges in 39 different states, Puerto Rico, and the District of Columbia (**Figure 8**).

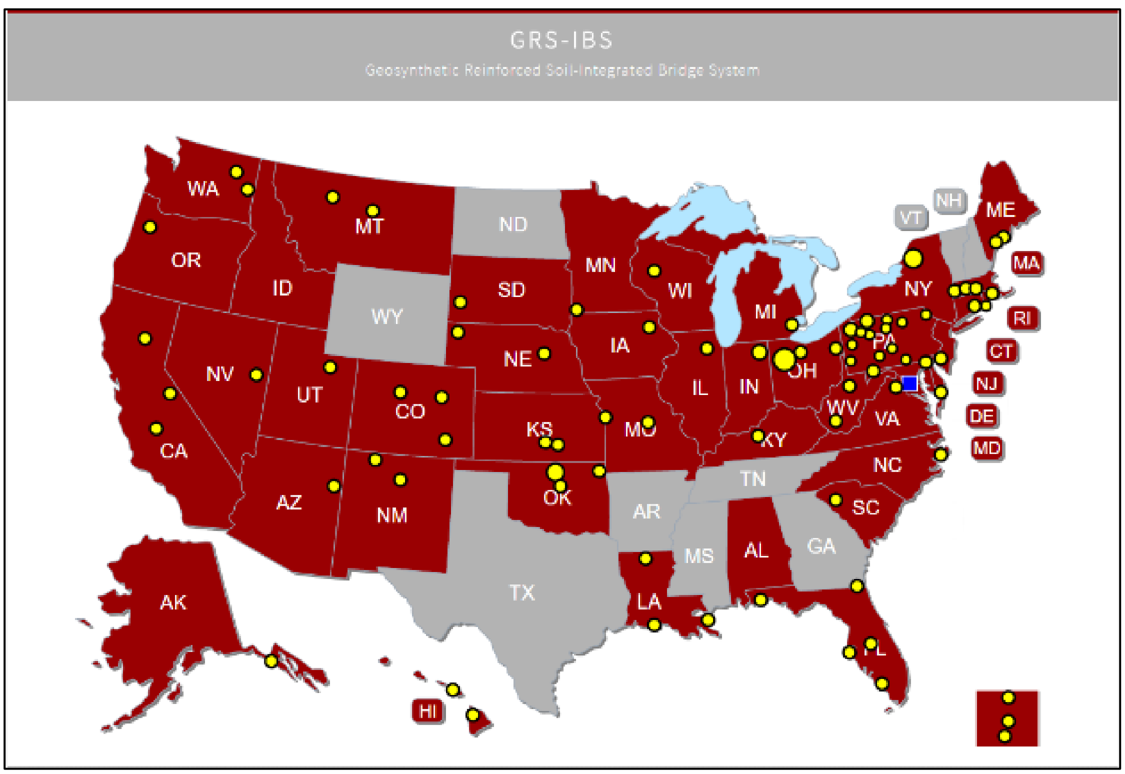


Figure 8. Completed and ongoing GRS-IBS projects across the U.S

Based on the results of the literature review, the following main parameters of the bridges where selected as the primary criterion to be reported, analyzed and compared: (1) Identification, (2) State, (3) Region, (4) County, (5) Bridge Name, (6) Geometry (e.g. length of spans, abutment height, width, and area), (7) Cost, (8) Completion year, (9) Superstructure and facing type, and (10) Reference and/or source of the information. This information will be subsequently used to identify trends, good practices, and improvement potentials that will help in the design and construction process of new bridges in Oklahoma and other states. This study presents the general information of some highlighted GRS-IBS cases, in addition to the cases reported by Ngo (2016). Additionally, this study includes the most used surveying techniques and performance monitoring information of some surveyed cases across U.S

2.5.1 BR 1-366 Bridge and BR 3-140, Delaware

Delaware reported the construction of two (2) GRS-IBS bridges: (1) BR 3-140 Bridge in Sussex County (**Figure 9**), and (2) BR 1-366 Bridge in Newcastle County (**Figure 10**). BR 1-366 Bridge was constructed in 2013 to replace an existing 77-year-old bridge, with an average daily traffic (ADT) value of 2,094. BR 3-140 in Sussex County was completed on April 11, 2014; a 13-year-old bridge, which was damaged, as result of the culverts being plugged by a large quantity of debris and branches after Hurricane Sandy in 2012.

The Department of transportation of Delaware (DelDOT) received a grant from the Federal Highway Administration (FHWA) for $300,00 to replace BR 1-366. With total costs of $737,090 (Benton 2014) and $419,634 (Walls 2014) for BR 1-366 and BR 3-140 respectively, bridges in Delaware cost less than half that of traditional bridges. Additionally, the bridges were constructed by Mumford and Miler Concrete, Inc. and George & Lynch, Inc. under DelDot supervision, providing local employment and experience (Benton 2014). **Table 2** shows selected data on the existing GRS-IBS bridges in Delaware.

Table 2: Summary table design features of GRS-IBS projects in Delaware

Bridge	Span m (ft)	Abutment Height m (ft)	Bridge Width m (ft)	Cost ($)	Completion Year	AADT	Superstructure	Contractor
BR 3-140	12.19 (40)	3.96 (13.0)	7.32 (24)	$419,634	2014	125 in 2012	Precast concrete beam	George & Lynch, Inc.
BR 1-366	11.28 (37)	4.75 (15.6)	12.19 (40)	$737,090	2013	2094 in 2010	Adjacent precast concrete frame boxes	Mumford & Miller Concrete Inc.

(a) (b)

(c)

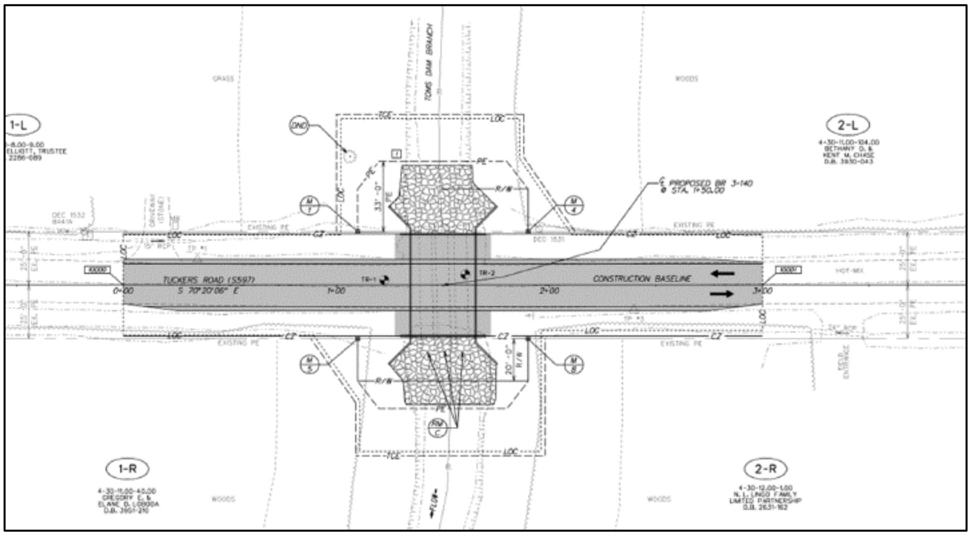
Figure 9: BR 3-140 project in Sussex County, Delaware: (a) Original structure; (b) Original structure after Hurricane Sandy in 2012; (c) New GRS-IBS bridge (Walls 2014)

(a)

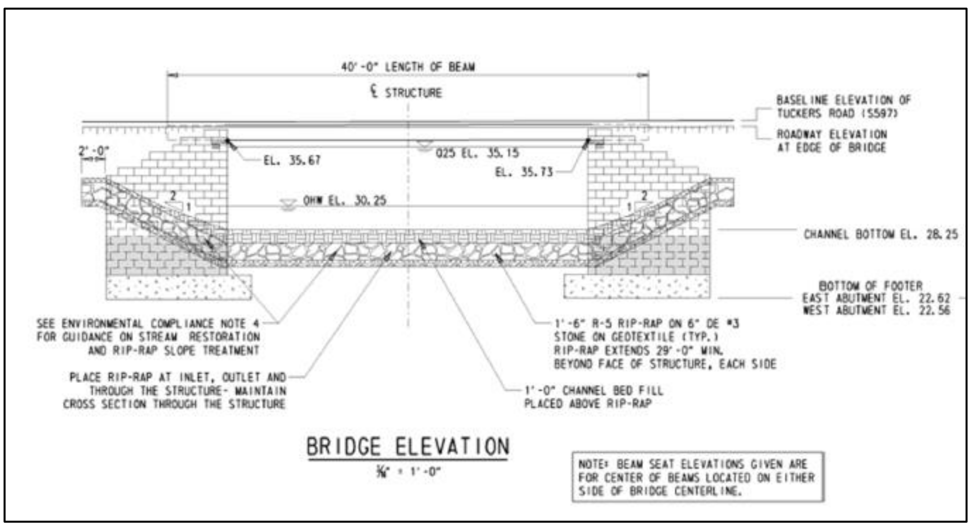
(b)

Figure 10: BR 1-366 project in Newcastle County, Delaware: (a) Previous bridge; (b) New GRS-IBS Bridge (Benton 2014)

Figure 11 through **Figure 12** show the plan and elevation views of BR 3-140 and BR 1-366.

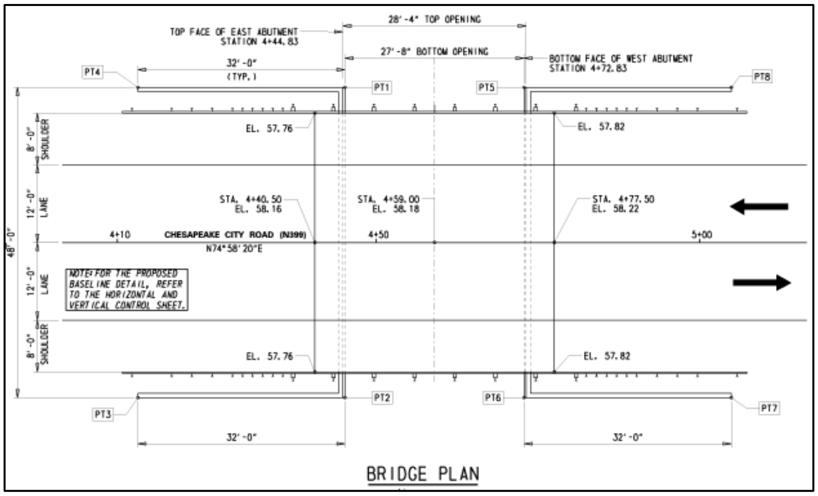


(a)



(b)

Figure 11: Design drawings of the BR 3-140 Bridge (Walls 2014)



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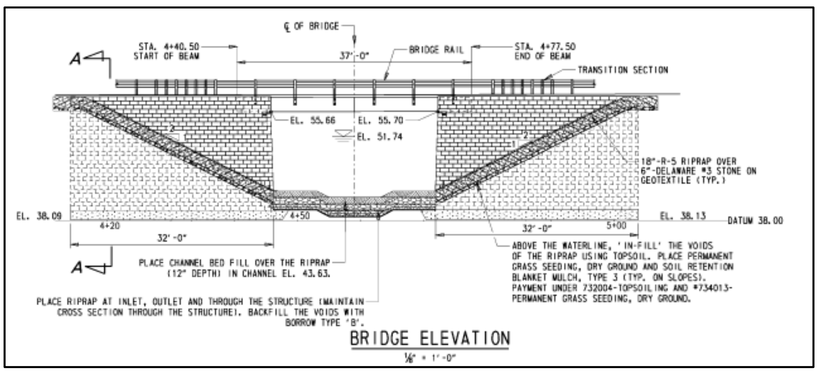

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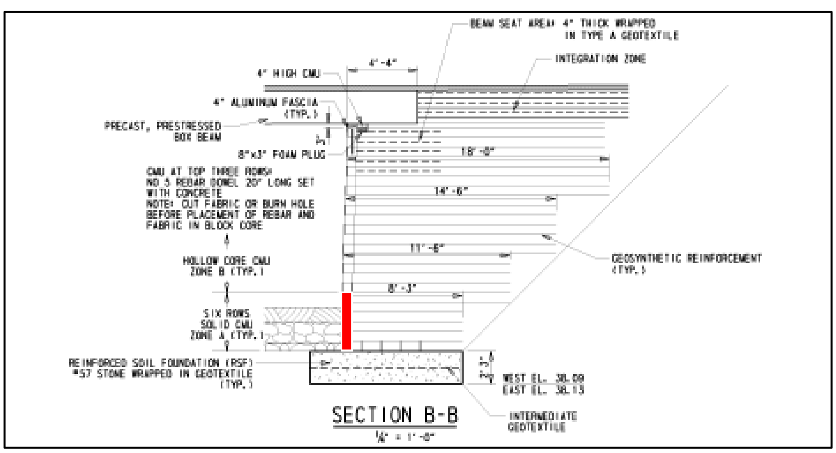


(c)

Figure 12: Design drawings of the BR 1-366 Bridge: (a) Bridge plan; (b) Bridge elevation (Talebi et al. 2014); (c) Cross section of BR 1-366 Bridge abutment (Benton 2014)

Information about loading on the GRS abutment of BR 1-366 is schematically shown in **Figure 13** (Talebi et al. 2014). Individual Design loads are given in **Table 3**. Magnitudes of total dead load and live load on the bridge abutments are 84.00 kPa (1.77 ksf) and (112.00 kPa) 2.35 ksf, respectively (**Table 4**).

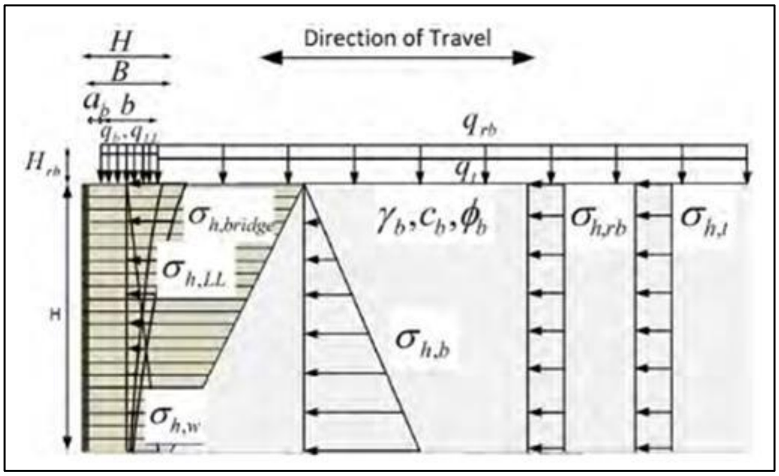


Figure 13: Lateral and vertical loads on a GRS abutment (Talebi et al. 2014)

Table 3: 1-366 Bridge design loads (Talebi et al. 2014)

Loading	Notation	Value
Bridge DL KPa (ksf)	qb	73.20 (1.53)
Bridge LL KPa (ksf)	qLL	97.00 (2.04)
Roadway LL KPa (ksf)	qt	14.84 (0.31)
Road base DL KPa (ksf)	qrb	11.49 (0.24)
Weight of GRS abutment KN/m (klf)	W	2.15 (147.51)
Weight of reinforce soil foundation KN/m (klf)	WRSF	0.39 (26.85)
Lateral load (Retained backfill) KN/m (klf)	Fb	0.71 (48.61)
Lateral load (qrb effect) KN/m (klf)	Frb	0.17 (11.87)
Lateral load (qt effect) KN/m (klf)	Ft	0.22 (15.19)

Table 4: Loading data for BR 1-366 Bridge in Delaware

Bridge	LL KPa (ksf)	DL KPa (ksf)
BR 1-366	112.51 (2.35)	1.77

The bridge superstructures are precast concrete beams and adjacent precast concrete frame boxes respectively. As recommended by Adams et al. (2015), both solid and split-face, hollow 203.2 mm × 203.2 mm × 406.4 mm (8 in. × 8 in. × 16 in.) CMU blocks were used to protect the GRS structure. Also, the non-biodegradable TerraTex HPG-57 with an ultimate strength of 70kN/m (4,800 lb/ft) was used in the both projects.

Benton (2014) reported that No. 89 stone was specified. However, an open graded material with a select No. 8 stone was used for the GRS abutments, due to local availability. Crushed stone No. 8 with 3/8 in. to ½ in. aggregate size was used in the BR 1-366 project.

High performance geotextile Terra Tex HPG-57 was used in the BR 1-366 project, and in the BR 3-140 project. Both products are biaxial woven polypropylene geotextiles produced by Hanes Geo Components. They are also non-biodegradable with a minimum ultimate strength of 70 KN/m (4,800 lb/ft) (**Table 5**).

Table 5: TerraTex High Performance Geotextile (HPG-57) properties used in the 1-366 project (Hanes Geo Components)

PROPERTY	TEST METHOD	ENGLISH	METRIC
Wide Width Tensile (Ultimate)	ASTM D4595	**4,800 x 4,800** lbs/ft	**70.0 x 70.0** kN/m
Wide Width Tensile (2% Strain)	ASTM D4595	**960 x 1,320** lbs/ft	**14.0 x 19.3** kN/m
Wide Width Tensile (5% Strain)	ASTM D4595	**2,400 x 2,700** lbs/ft	**35.0 x 39.4** kN/m
Permittivity [1]	ASTM D4491	**0.400** sec^{-1}	**0.400** sec^{-1}
Water Flow Rate [1]	ASTM D4491	**30** gpm/ft^2	**1,222** Lpm/m^2
AOS [1,2]	ASTM D4751	**30** US Std. Sieve	**0.600** mm
UV Resistance	ASTM D4355	**80** % @ 500 hrs	**80** % @ 500 hrs

Table 6: Summary table of materials used in GRS-IBS projects in Delaware

Bridge/FHWA	Facing blocks	Backfill Materials			Geosynthetic	Geosynthetic Ultimate Strength-Tf KN/m (lb/ft)
		Materials	c'	ϕ'		
FHWA Guidelines	Concrete Masonry Unit (CMU)	Well graded or open graded	0	$\Phi \geq 38^{o}$, well/open graded	*Either geogrid or geosynthetic *Most common is Biaxial Woven Polypropylene (PP) geotextile	≥ 70 KN/m (4,800)
BR 1-366	CMU red solid core and split-face hollow core blocks	No. 8 Stone	0	40^{o}	*HPG-57 Geotextile	70 KN/m (4,800)
BR 3-140	Red solid core and voided CMUs	N/A	0	N/A	*Biaxial woven polypropylene Geotextile	70 KN/m (4,800)

The geotechnical tests were reported by Talebi et al. (2014). Two boring logs were taken from each side of each abutment of the BR 1-366 bridge, which showed high blow counts. A total of six consolidation tests, two unconfined compression tests, 41 soil classification tests, and four UU triaxial shear tests were carried out to determine the properties of the subgrade soil. The test results identified the foundation soil as consisting of stiff clays, and medium to dense sands which are suitable for shallow foundation design. **Figure 14** shows soil properties from one of the boreholes.

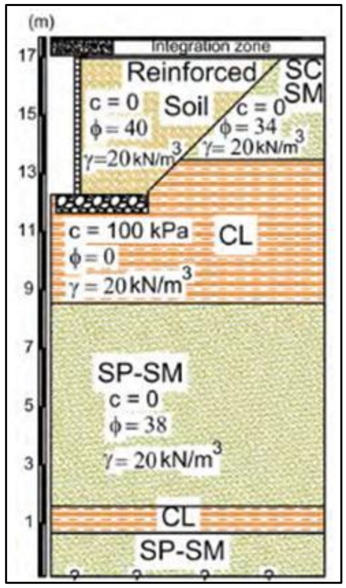


Figure 14: Soil properties and stratigraphy from one of the boreholes at the BR 1-366 bridge site (Talebi et al. 2014)

The use of GRS as abutment requires proper hydraulic design. This includes the evaluation of long term aggradation and degradation, scour vulnerability, potential for lateral migration of stream and the calculation of contraction and abutment scour. The predicted depth of scour must be less than the permanent design value to insure the safety and durability of the bridge abutment. Both bridges BR 1-366 and 3-140 involve water crossings. Therefore, riprap was used as a scour countermeasure (**Table 7**).

Table 7: Summary table for hydraulic data in Delaware

Bridge	Scour (ft)	Service Under Bridge
FHWA Guidelines	*Low scour potential, scour countermeasures: riprap aprons, gabion mattresses, and articulated concrete blocks	Either waterway or roadway
BR 1-366	*A riprap slope was placed against the facing block to armor the face of the wall against scour	Waterway
BR 3-140	*Riprap	Waterway

Finally, Talebi et al. (2014) reported excavation and construction procedures in the BR 1-366 project (**Figure 15**). Construction of the east abutment started on March 22, 2013 and was completed in 14 days. Construction of the west abutment started on April 3, 2013 and was completed in 20 days. The entire project was completed in approximately seven (7) weeks, which was comparatively quick for a first-time experience. The construction crew did not include more than five (5) individuals: three (3) to four (4) laborers and one equipment operator. The equipment operator handled the bulldozer to excavate the soil in preparation for the RSF and the GRS abutment, and placement of the GRS fill.

(a)

(b)

Figure 15: Construction of BR 1-366: (a) East abutment; (b) west abutment (Talebi et al. 2014)

2.5.2 Palestine Road, Kentucky

During 2014, one GRS-IBS bridge was built in Kentucky to replace an older bridge that suffered from frequent flooding and was required to be closed regularly, in order to clear the resulting debris from the 18 in pipe culvert. The solution to these issues was to build a cost-effective bridge on a rural secondary road. The new bridge was built on Palestine road, north of Campbellsville in Taylor County, Kentucky.

Construction of the new bridge lasted less than three weeks, considering there was a one week delay for late material delivery, and a lack of details on the construction of the substructure. The materials used for the superstructure, were salvaged box beams left from another project, and for the facing: conventional CMU blocks were used. The bridge's design features are shown in **Table 8.**

Table 8: Summary table design features of GRS-IBS project in Kentucky

Bridge	Span m (ft)	Width m (ft)	Skew (deg)	Cost ($)	Completion Year	Superstructure	Type of environment
FHWA Guidelines	≤42.7 (140)	-	-	-	-	Concrete or Steel	Urban or Rural
Palestine Road	16.8 (55.0)	7.6 (25.0)	5	114,000	2015	Salvaged box beams	Rural

The bridge settlement was monitored by the staff, and was found to be negligible. It was determined that the GRS-IBS alternative saved 20% in cost when compared to a conventional solution. **Figure 16** shows the stages of construction of the GRS-IBS. It is important to note that it is essential to have adequate compaction in both the RSF and the GRS abutment, in order to meet strength and performance requirements.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

Figure 16: GRS-IBS construction in Palestine road north of Campbellsville in Taylor County, Kentucky: (a) Stage 1:Demolition of old bridge; (b) Stage 2: Placement of first row of CMU blocks; (c) Stage 3: Construction of GRS abutment; (d) Stage 4: Preparation for placement of bridge deck; (e) Stage 5: Placement of bridge deck (f) Stage 6: Finalizing the integrated approach and the roadway surface; (g) New GRS-IBS bridge (Sweger 2014)

2.5.3 Cecil Creek, Big Lake, Cut off Creek, Louisiana

The state of Louisiana has 6 GRS-IBS bridges. This makes it the lead state in number of GRS-IBS bridges for the South-East portion of the country. Out of those six, one was built to handle higher traffic in Saint Bernard County, two bridges were built to replace an old timber trestle bridge in Vermilion County and three bridges in Union County used to replace an old rail car bridge. (**Figure 17**)

These bridges comply with the span recommendations by Adams et al. (2012) for GRS-IBS bridges. The longest bridge is the one in Saint Bernard County with a span of 110 ft. The shortest one is in Vermilion County with a span of 9.1 m (30.0 ft). and finally, the three bridges in Union County have a span of 23.2 m (76.0 ft.)

The recommendations by Adams et al. (2012) for abutment height in GRS-IBS are 4.6 m (15.0 ft.), with a maximum of 9.1 m (30 ft.), while still being verifiably safe. The abutment heights reported for the bridges in Vermilion County, were 10 ft. for the Creek Bridge and 4.05 m (13.3 ft.) for the one at Maree Michel Bridge. The three GRS-IBS bridges in Union County saved 40% in construction expenses compared to pile-supported bridges (Meunier 2013).

All of these bridges built in the state of Louisiana are credited with being among the most efficient and cost-effective bridges of their size; proving that GRS-IBS technology is a great match for the state needs. The selected data on the existing GRS-IBS bridges in Louisiana is shown in **Table 9**.

(a)

(b)

Figure 17: Example GRS-IBS projects, Louisiana (Meunier 2013)

Table 9: Summary table of GRS-IBS bridges design features in Louisiana

Bridge	Span m (ft)	Abutment Height m (ft)	Cost ($)	Completion Year	Super structure	LL KN (kips)	DL KN (kips)	Geosynthetic
FHWA Guidelines	42.7 (140)	≤ 9.5 (30.0)	-	-	Concrete or Steel	-	-	Either geogrid or geosynthetic. Most common is Biaxial Woven Polypropylene (PP) geotextile in the abutment
Cecil Creek	23.1 (76)	N/A	40% less than pile-supported	2005	N/A	N/A	N/A	N/A
Big Lake	23.1 (76)	N/A	40% less than pile-supported	2005	N/A	N/A	N/A	N/A
Cut off Creek	23.1 (76)	N/A	40% less than pile-supported	2005	N/A	N/A	N/A	N/A
Yscloskey	33.5 (110)	N/A	N/A	2005	N/A	N/A	N/A	BX 1200 geogrid placed every 20 cm (8 in.) to 24 cm (10 in.) within reinforced stone footing
Creek	9.1 (30)	3.1 (10.0)	N/A	2005	Slab span bridge	1,054 (237) at beam seat	845 (190) at beam seat	N/A
Maree Michel	16.7 (65)	4.1 (13.3)	N/A	2005	steel girder	1,423 (320) at beam seat	1,254 (282) at beam seat	N/A

2.5.4 CR12, CR24,CR35 and others , New York

St. Lawrence County in New York, is the leading county in the nation with regards to bridge technology. In 2013, the County reported to have 356 bridges, with 84 of them being deficient and in need of care. Of these deficient bridges, 26 were in critical condition and needed to be urgently replaced. The county put these on the priority replacement list and left the remaining 58 bridges in the corrective maintenance list (Bogart 2013). Two main reasons were considered for using GRS-IBS technology to build bridges in St Lawrence County: (1) The speed of building GRS-IBS bridges is faster than that of conventional bridges, and, (2) GRS-IBS technology has a lower construction cost than conventional bridges. GRS-IBS technology was chosen, since it represented the perfect solution when considering the large number of bridges St. Lawrence County needed to replace and the county's budget deficit.

All the GRS-IBS bridges built in St. Lawrence County were single span structures that were shorter than 43 m (140 ft.) of span, making them compliant with the recommendations by Adams et al. (2012). Several conditions were considered when establishing the span of each of these bridges such as hydraulic opening, setback for new abutment location, and bearing area. Of these bridges, the longest one was at Trout Brook with a 28.6 m (94-ft.) span, and the shortest was the CR12 project. Additionally, all the GRS-IBS bridges in this county fell within the advised abutment height by Adams et al. (2012) of 9.5 m (30 ft.) A good example is the CR12 project with an abutment height of 4.6 m (15 ft.) **Figure 18** shows examples of the GRS-IBS bridges built in St. Lawrence County.

(a)

(b)

(c)

(d)

(e)

(f)

(g) (h)

(i)

Figure 18: Example GRS-IBS projects in St. Lawrence County, NY: (a) CR 12 Project; (b) CR 24 or Leonard Brook; (c) CR 31 or Brandy Brook; (d) CR 35 or Trout Brook; (e) CR 38 or Plum Brook; (f) CR 25 or Little River; (g) CR 40 or Hutchins Creek; (h) River Road, and (i) Fraser Road or Oswegatchie River (Bogart 2013)

The loading information of each of the bridges in St. Lawrence County was determined by the amount of traffic on the bridge. Per Adams et al. (2012), GRS-IBS bridges should be applied in low volume road construction. In the case of the bridges built in the state of New York, Hutchins Creek bridge presented the greatest average daily traffic with 2,334 vehicles per day. Most bridges in St. Lawrence county (13 of them), had an average daily traffic (ADT) of less than 1,000 vehicles per day.

Regarding bridge widths, the widest were the CR12 and Leonard Brook projects with 10 m (33 ft.), and the narrowest was at Oswegatchie River. Note that Adams et al. (2012) does not specify any recommendation on the GRS-IBS bridge's width. Generally, GRS-IBS bridges cost between 30 % and 50% less than traditional bridges. In the state of New York, the most expensive GRS-IBS bridge was built in Chippewa Creek at a cost of $373,000. The rest of the GRS-IBS bridges range from $165,000 to $320,000 in cost.

Most of the GRS-IBS bridges in St. Lawrence County were constructed using 203 mm × 203 mm × 406 mm (8 in. × 8 in. × 16 in.) CMU blocks as facing wall with existing concrete abutment and for those with water crossings there was rip rap on the sides (Bogart 2013).

Bogart (2013) reported four main lessons from the construction of these bridges: (1) In order to stay behind the existing abutment, the span must be increased and the additional costs will be compensated by the lack of a cofferdam, (2) RSF can be usually installed in dry conditions without constructing a cofferdam, (3) Environmental permitting is shortened to 10 days, and (3) Construction in water increases the cost.

NYDOT concluded that overall, GRS-IBS bridges provided better performance than conventional pile-cap foundation systems at a lower cost. For St. Lawrence County, 50% of the bridges could be replaced using this technology, saving around 50% of the costs. NYDOT recommended to keep the partial concrete abutment in place and have a dry construction zone for the reinforced soil foundation, in order to have cheaper and faster results. **Table 10** shows a summary of design features for 13 GRS-IBS bridges in St Lawrence County, NY

Table 10: Summary table design features of GRS-IBS projects in St. Lawrence County, New York.

Bridge	Span m (ft)	Bridge Width m (ft)	Cost ($)	Completion Year	Superstructure	ADT	Facing blocks	Scour	Service Under Bridge
FHWA Guidelines	≤43.6 (140)	-	-	-	Concrete or Steel	Typical for low volume road	Concrete Masonry Unit (CMU)	Low scour potential, scour countermeasures: riprap aprons, gabion mattresses, and articulating concrete blocks	Waterway or Roadway
CR12 Project	12.4 (40.5)	10.6 (33)	$240,000	2011	N/A	N/A	N/A	N/A	Waterway
CR12 o. Malterna Creek	12.4 (40.5)	N/A	N/A	2009	N/A	N/A	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	N/A
CR24 o. Leonard Brook	14.3 (47)	10.6 (33)	$232,122	2010	Prestressed concrete, Box Beam or Girders - Multiple	580 (2011)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway
CR35 o. Trout Brook	20.1 (67)	10.6 (33)	$310,000	2010	Prestressed concrete, Box Beam or Girders - Multiple	1369 (2001)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway
CR31 o. Brandy Brook	17 (56)	6.1 (20)	Material cost $165,000	2010	Prestressed concrete, Box Beam or Girders - Multiple	940 (2010)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway

(cont'd)

Bridge	Span m (ft)	Bridge Width m (ft)	Cost ($)	Completion Year	Superstructure	ADT	Facing blocks	Scour	Service Under Bridge
CR38 o. Plum Brook	19.5 (63.5)	9.7 (32.0)	Material cost $175,000	2010	Prestressed concrete, Box Beam or Girders - Multiple	900 (2010)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway
CR60 o. Little River	21.3 (70.0)	9.1 (30)	N/A	2011	Prestressed concrete, Box Beam or Girders - Multiple	560 (2009)	N/A	N/A	Waterway
CR27 o. N. Br. Grasse River	21.9 (72.0)	N/A	$320,764	2011	N/A	N/A	N/A	N/A	N/A
Fraser Road o. Oswegatchie River	25.9 (85.0)	3.7 (12)	N/A	2011	Prestressed concrete, Box Beam or Girders - Multiple	10 (2011)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway
CR25 o. Little River	26.2 (88.0)	N/A	N/A	2011	N/A	N/A	N/A	N/A	N/A
CR40 o. Hutchins Creek	16.5 (54.0)	(9.7) 32	$252,362	2011	Prestressed concrete, Box Beam or Girders - Multiple	2334 (2001)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	N/A	Waterway
CR3 o. Chippewa Creek	25.9 (95.0)	N/A	$373,110	2011	N/A	N/A	N/A	N/A	N/A
River Road O. Trout Brook	28.6 (94.0)	(7.3) 24.0	N/A	2011	Prestressed concrete, Box Beam or Girders - Multiple	630 (2010)	203 mm x 203 mm x 406 mm CMU and existing concrete abutment	Existing concrete abutment and rip rap on the sides	Waterway

2.5.5 Ohkay Owingeh Tribe, New Mexico

The first GRS-IBS bridge in New Mexico was built 48 km (30 miles) north of Santa Fe by the Ohkay Owingeh Tribe using local labor. This bridge replaced the one shown in **Figure 19,** which suffered from frequent flooding given its skewed angle. Additionally, the sufficiency rate of the bridge was assessed as "fair" with 65.8. For this reason, the local tribal council designed a long-term resolution with a 100-year flooding plan. The project site was located on a major drainage basin close to Arroyo de Chingague. The design used a single span to reduce the amounts of potential sediment deposits. Three main observations can be made considering this project: (1) With proper training, unexperienced crews can complete a GRS-IBS project in a timely and cost efficient manner; (2) Even in dry areas, scour analysis is an important design consideration for GRS-IBS projects; and (3) in cases involving turbid rivers sediment deposits shall be also considered. **Table 11** shows a summary for the GRS-IBS design in New Mexico.

Table 11: Summary table of GRS-IBS projects design features in New Mexico

Bridge	Span m (ft)	Bridge Width m (ft)	Abutment Height m (ft)	Completion year	Type of environment
FHWA Guidelines	≤ 42.67 (140)	-	≤9.1 (30)	-	Urban or Rural
White Swan	19.8 (65.0)	8.5 (28.0)	3.1 (10.0)	2015	Rural

(a)

(b)

Figure 19: White Swan GRS-IBS Bridge in Ohkay Owingeh Pueblo, New Mexico: (a) Aerial views of the location; (b) Existing bridge prior to the construction (Albert 2015, Peters 2015)

The approximate construction time of the GRS-IBS project in New Mexico was 5 weeks, plus a 5-week delay in the delivery of the precast concrete beams for the bridge superstructure. **Figure 20** shows the construction of the GRS-IBS bridge and **Figure 21** shows it completed.

(a)

(b)

(c)

Figure 20: White Swan Bridge during construction in Ohkay Owingeh Pueblo, New Mexico: (a) Abutment construction; (b), (c) Superstructure installation (Albert 2015, Meyer 2015)

Figure 21: Completed White Swan Bridge in Ohkay Owingeh Pueblo, New Mexico (Albert 2015)

Table 12 shows that this project saved 58% in cost and 44% in construction time compared to conventional designs. This bridge is also included in the survey of GRS-IBS projects in the U.S. (Hatami et al. 2016) since it represents another good example of the advantages of using this technology when there are time and cost constraints.

Table 12: White Swan GRS-IBS Bridge in Ohkay Owingeh Pueblo, New Mexico (Albert 2015): (a) Summary information; (b) Estimated cost savings as compared to conventional design; (c) Actual monetary savings relative to the allocated budget; (d) Labor cost savings; (e) Time savings

County	Bridge	Construction Time	Surface Road	Superstructure	Scour Countermeasure	Ser Under Bridge
Rio Arriba	White Swan Bridge	75 days	Asphalt	9 precast concrete beams	Riprap	Waterway

(a)

Conventional design (engineer estimation)	$1,000,000
GRS-IBS design	$419,331.26
Monetary savings	$580,668.74 (58% Saving)

(b)

Accelerated Innovation Deployment (AID)	$200,000
FHWA ridge Replacement Program	$284,706
Total project cost	$419,331.26
Project balance	65,374.74 (13% under budget)

(c)

Estimated contractual cost for outside contractor	$105,000
Local road crew actual labor cost (4 crews)	$52,103
Total labor cost savings	$52,897 (50.4% Saving)

(d)

Conventional design (Engineer estimation)	4 ½ Months
GRS-IBS design	2 ½ Months
Time savings	2 Months (44% Saving)

(e)

2.6 Performance Monitoring techniques

A review of the monitoring techniques related to GRS-IBS projects is given in this section. The survey techniques include measuring vertical and lateral deformations, bridge settlements, thermal movements, and stress distributions during the service life of the GRS-IBS bridges. They also include the descriptions, applications, advantages, precision and installation considerations of the instruments, and monitoring techniques reported in the literature. The vertical and lateral deformations are especially important because they are related to the serviceability of the bridge's structure. Since the GRS abutment backfill is composed of granular material, a significant part of the vertical deformation (settlement) typically occurs immediately after the superstructure is placed on the abutment. Lateral deformations can be estimated using the profile of vertical settlements (Adams et al. 2012) and/or using measured reinforcement strains due to vertical load. Additionally, **Table 13** shows the updated information of the different monitoring instruments used in GRS-IBS projects in this study. Total Station Theodolite (TST) is the most widely used technique due to its ease of implementation and low cost.

Table 13: Summary of different monitoring instruments used in GRS-IBS projects surveyed in this study (Hatami et al. 2015; Ngo 2016)

Instrumentation Type	No. Bridges Installation
Total Station Theodolite	16
Pressure Cells	9
Inclinometers	5
Piezometers	4
Extensomers	3
Strain Gauges	3
Settlement Plates	2
ShapeAccelArrays (SAA)	2
Tensiometers	1
Fiber Optic Sensors	1
Telltales	1
Volumetric Water Sensors	1
Thermistors	1

2.6.1 Total Stations theodolite

TST's are used to record the bridge's settlement and movements of the GRS abutments. In a few pilot GRS-IBS projects documented by Adams et al. (2012), the total settlements and deformations of the abutment facing wall and the superstructure have been measured using an electronic distance measurement (EDM) survey or a standard survey level and rod system. The precision of both surveying methods is on the order of ± 0.0015 m (0.005 ft.) According to Adams et al. (2012), "the difference between the settlement measured on the abutment facing wall and the superstructure is the vertical deformation within the GRS mass alone due to the bridge load". The angular distortion and differential settlement can be evaluated by measuring the bridge settlement at four corners of the bridge with a survey level. **Figure 22** shows the layout of how a standard survey is carried out.

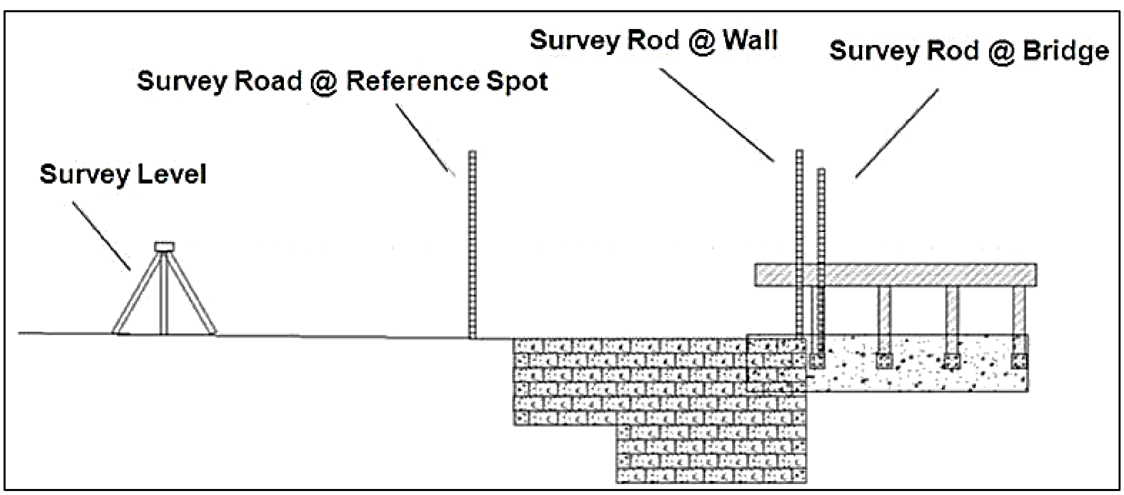


Figure 22: Standard survey level method to measure superstructure and wall settlement (Adams et al. 2012)

2.6.2 Inclinometers

Inclinometers are used in geotechnical engineering for performance monitoring of slopes and earthwork structures. They are also used in combination with micro-electro-mechanical systems (MEMs) to monitor wall deformations, ground movements and gradual landslides through real-time, remote-sensing means or typically more affordable onsite readout approaches. There are two types of inclinometers: Slope Inclinometer Arrays, commonly known as In-Place Inclinometers (IPI), and Manual Slope Inclinometers (Abdoun et al. 2008).

In the case of IPI, a series of inclinometers is continually kept inside the casing in each borehole, which makes it possible to provide remote, continuous and real-time sensing (**Figure 23a**). This type of inclinometer is ideal for early failure warning with a resolution of about ± 0.01 mm/m (0.012 in/ft; HMA 2014), but at a comparatively high total cost because the inclinometers have to stay at a fixed location, and therefore cannot be shared across different boreholes.

Manual Slope Inclinometers are used across several boreholes (**Figure 17b**). It is typically used in vertical boreholes with a flexible, grooved casing to guide the inclinometer into the depth of the fill (Abdoun et al. 2008). The cost can be controlled by reducing the number of inclinometers with accuracy of ±2 mm per 25 m (± 0.0026 in. per 25 ft; HMA 2014). However, this system increases the amount of work necessary to complete the monitoring task. It requires at least one operator to access the site, temporarily install the inclinometer inside a borehole and take measurements. This procedure should be repeated at every borehole (RST 2014). Furthermore, due to the periodic but discontinuous nature of taking measurements, important events and information may be missed in the periods between the measurements (Abdoun et al. 2008).

(a)

(b)

Figure 23: (a) In-Place Inclinometer (IPI); (b) Inclinometer Probe System (Durham 2014)

In GRS-IBS applications, inclinometers are used to monitor lateral ground movements during construction and subsequent bridge operation. In the survey of related GRS-IBS projects across the U.S., the following five (5) projects in three (3) states were found to have reported using inclinometers to monitor abutment deformations:

- BR 1-366 Bridge in New Castle County, DE (**Figure 24**) four (4) In-Place Inclinometers (IPI) were installed in the clay foundation layer under the west abutment (Talebi et at. 2014)
- 250th Street Bridge in Buchanan County, IA **(Figure 25**) inclinometers were installed inside 3.34 in. diameter casings
- Cecil Creek Bridge in Union Parish, LA
- Big Lake Bridge in Union Parish, LA
- Cutoff Creek Bridge in Union Parish, LA (Meunier 2013)

The 8.48 cm (3.34 in.) diameter inclinometer casing has been reported to be suitable for landslide and long-term monitoring applications (Durham 2013). Additionally, (Vennapusa 2012) listed the steps for the inclinometer installation: (1) Drill a borehole in the abutment, (2) Fill the inclinometer casing with water to overcome buoyancy effects in the hole due to groundwater, (3) Insert the casing into the borehole (**Figure 25**), and (4) Fill the cavity around the casing with sand and cement grout up to the top foot of the cavity (**Figure 25**).

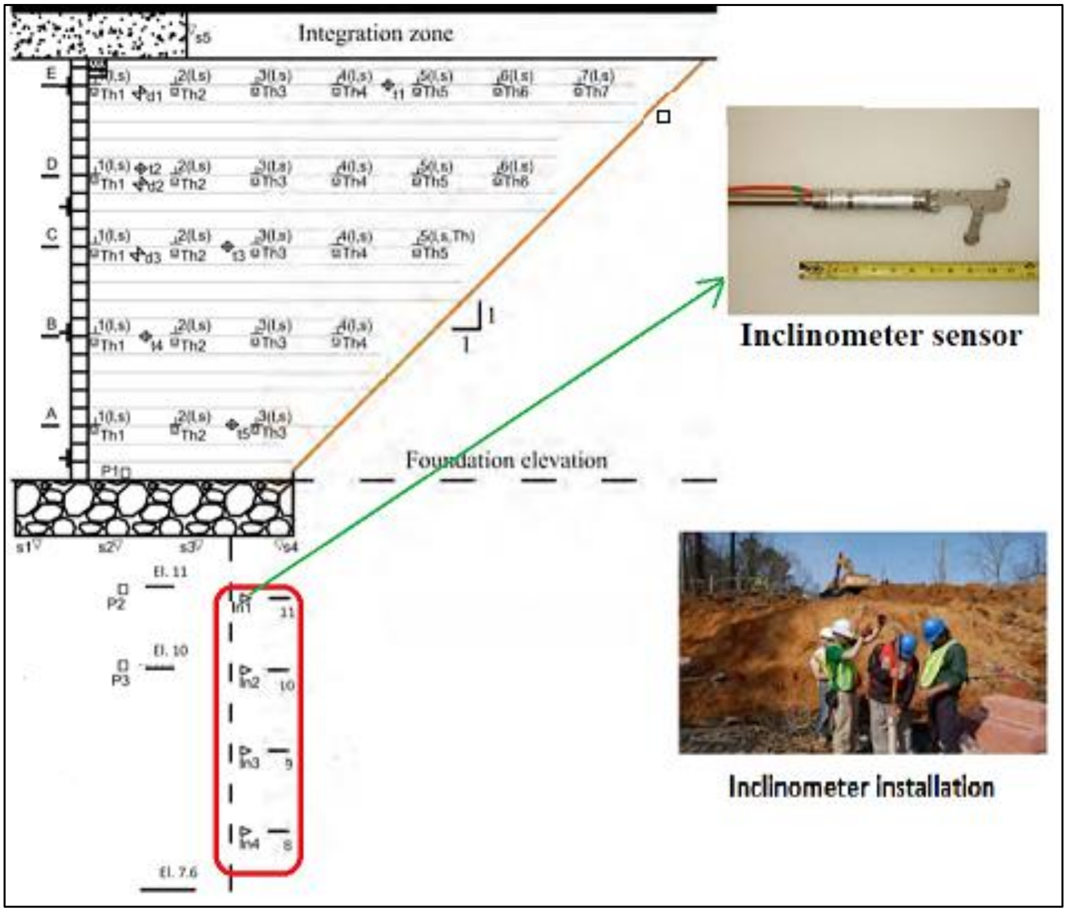


Figure 24: Inclinometers used in the BR 1-366 Bridge in Delaware (Talebi et al. 2014)

Figure 25: Three 5-foot inclinometer casings snapped together during installation on a bridge abutment in Delaware (Vennapusa 2012)

2.7 Performance monitoring Case Studies

This section presents performance monitoring in several states.

2.7.1 BR 1-366, Delaware

Table 14 and **Figure 26** show the Instrumentation plan to monitor the long-term performance of the 1-366 Bridge. Instrumentation types, quantities and locations, together with a schematic of an instrumented cross section are shown below.

Table 14: Sensor types, locations, and quantities used in 1-366 GRS-IBS abutments (Talebi et al. 2014)

Instrument types	**Location**	**No.**
Inclinometer sensors	Foundation, West Abutment	4
Piezometers	Foundation, West Abutment	3
Pressure cells	Foundation, GRS Abutment, between Bridge and Integrated Zone, West Abutment	8
Strain gauges	West GRS Abutment, East GRS Abutment and beneath the Bridge	110
Thermistors	West GRS Abutment and East GRS Abutment	50
Volumetric water content sensors	West GRS Abutment	5
Surveying points	West and East Facing Walls	40

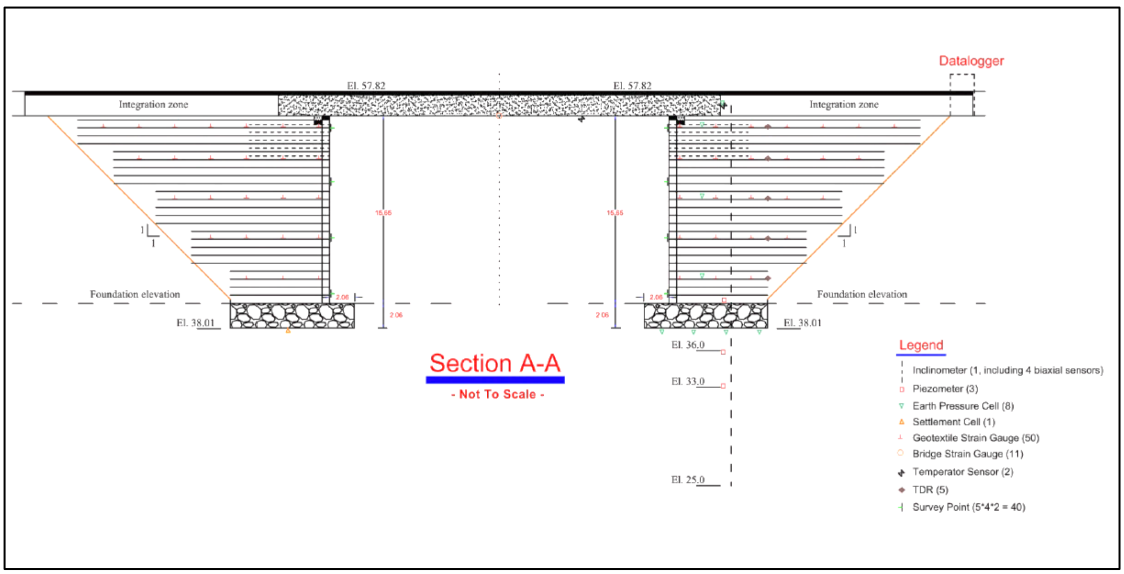


Figure 26: Instrumented plan for the 1-366 GRS-IBS bridge abutments in Delaware (Talebi et al.2014)

Figure 27 shows the locations of the twenty survey points that Talebi et al. (2014) placed on each abutment, which were spread out at the top, middle, and bottom of the facing wall to measure facing wall deflection during service. However, details such as the precision of the survey equipment or its type (e.g. EDM vs. standard surveying level) were not specified.

Figure 27: Survey points on the abutment of 1-366 Bridge in Delaware (Talebi et al.2014)

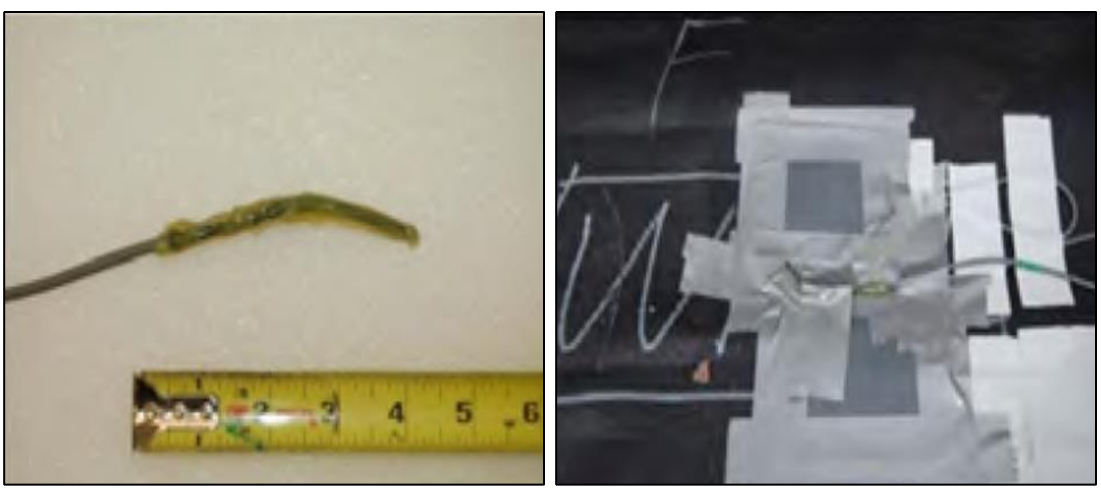
Figure 28 shows the 1-366 bridge abutment sensors and their installation at the site. Talebi et al. (2014) reported that four inclinometer sensors and three vibrating wire piezometers were installed to measure displacements and pore water pressure in the clay foundation layer during construction and service periods. Eight vibrating wire pressure cells were placed in various locations (i.e. one between the superstructure and the integration approach, three inside the west abutment, and four under the foundation) to monitor static and instantaneous pressure fluctuations. Strains in the HPG-57 geotextile reinforcement were measured using 50 strain gauges mounted on the geotextile. The strain gauges and pressure cells were covered with sand to protect them against the overlying No. 8 stone backfill material. Twenty-five YSI 55000 thermistors were wired and waterproof sealed in the University of Delaware laboratory prior to field installation, and were placed between the strain gauges to detect temperature and its effect on the measured strains in the woven geotextile. Finally, five MAS-1 volumetric moisture content sensors were placed in the west side abutment to check the moisture content and its effect on strains in the geotextile.

(a)

(b)

(c)

(d)



(e)

(f)

Figure 28: (a) Inclinometers and their site installation at the site in Delaware; (b)Piezometers and their site installation; (c) Pressure cells and their site installation; (d)Strain gauges and their site installation; (e) Thermistors and their site installation; (f)Volumetric moisture content sensors (Talebi et al. 2014)

2.7.2 Cecil Creek, Big Lake, Cut off Creek, Louisiana

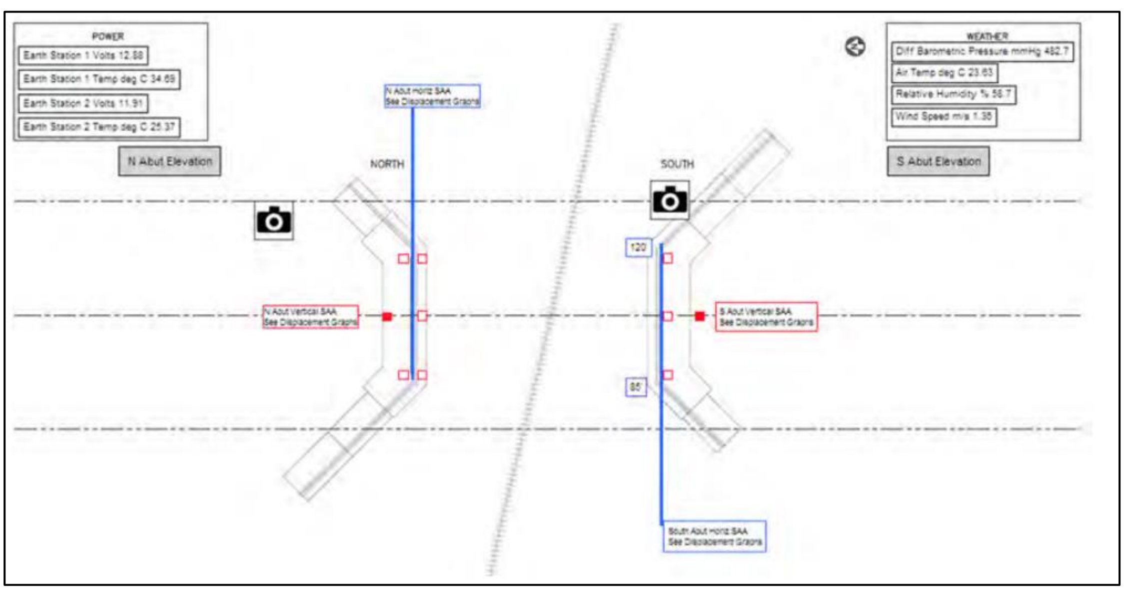
Meunier (2013) reported that the selected GRS-IBS projects in Union County were monitored using inclinometers and extensometers with a total settlement of less than 3.81 cm (1.5 in.) and differential settlement of less than 0.9 in. (**Table 15**)

Table 15: Summary of instrumentation and monitoring of settlements in the selected GRS-IBS projects in Louisiana

Bridge	Instrument	Settlement cm (in)	Differential Settlement cm (in)
Cecil Creek	Inclinometers and extensometers	1.98 (0.78)	0.6 (0.27)
		2.67 (1.05)	
Big Lake 2	Inclinometers and extensometers	1.65 (0.69)	2.29 (0.9)
		0.91 (0.36)	
Cut off Creek	Inclinometers and extensometers	0.1 (0.06)	1.52 (0.6)
		0.7 (0.03)	

2.7.3 CR-55 Bridge, Minnesota

The instrumented monitoring for this project was set for three (3) years. **Figure 29** shows the plan view and elevation view of GRS-IBS instrumentation in Rock County Minnesota (**Figure 30**; Budge et al. 2014). **Table 16** shows the summary table for the CR-55 Bridge design features in Minnesota. The monitoring techniques for the bridge abutment are shown in **Table 17** (Budge et al. 2014). Two horizontal Shape Accel Array (SAA) systems were installed at the base of the fill material to measure the vertical position change with respect to a fixed end. Forty-two optical prisms were installed on the facing wall to detect the lateral and vertical movement of the abutment. Two vertical Shape Accel Array systems were placed on the abutments to check movement of the facing wall. Vibrating-wire (VW) earth pressure cells (EPC) were mounted at the base of the abutment to check backfill pressures acting on the foundation soils. Since this area has been experiencing large temperature variations, a weather station was installed to measure the temperature and solar radiation.



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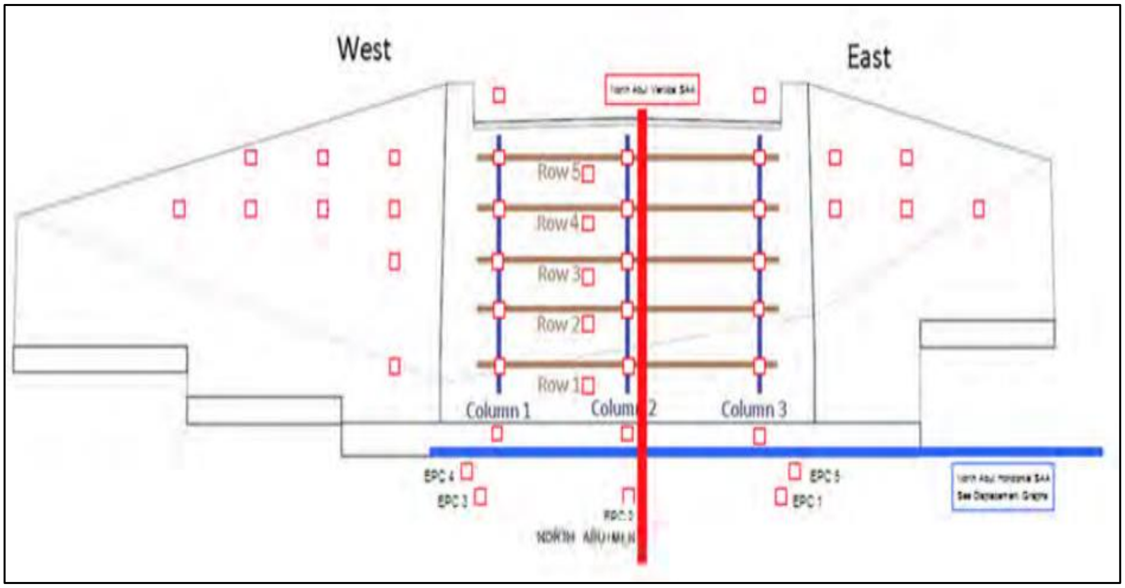

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Figure 29: Plan view of GRS-IBS project in Rock County Minnesota; (b) Elevation view of GRS-IBS project in Rock County Minnesota (Budge et al. 2014)

Table 16: Summary table for the CR-55 Bridge design features in Minnesota

Bridge	Span m (ft)	Abutment Height m (ft)	Completion Year	Superstructure	AADT
FHWA Guidelines	≤ 43 (≤140)	9.14 (≤ 30.00)	-	Concrete or Steel	-
CR-55 over MN Southern Railway	23.62 (77.50)	6.92 (22.70)	2013	Concrete adjacent boxes	135

Table 17: Instrumentation monitoring for the CR 55 Bridge in Minnesota

Bridge	Instrument
CR-55 over MN Southern Railway	Horizontal Shape Accel Array (SAA), Vibrating-wire (VW), Earth pressure cells (EPC), optical prism, weather station

6

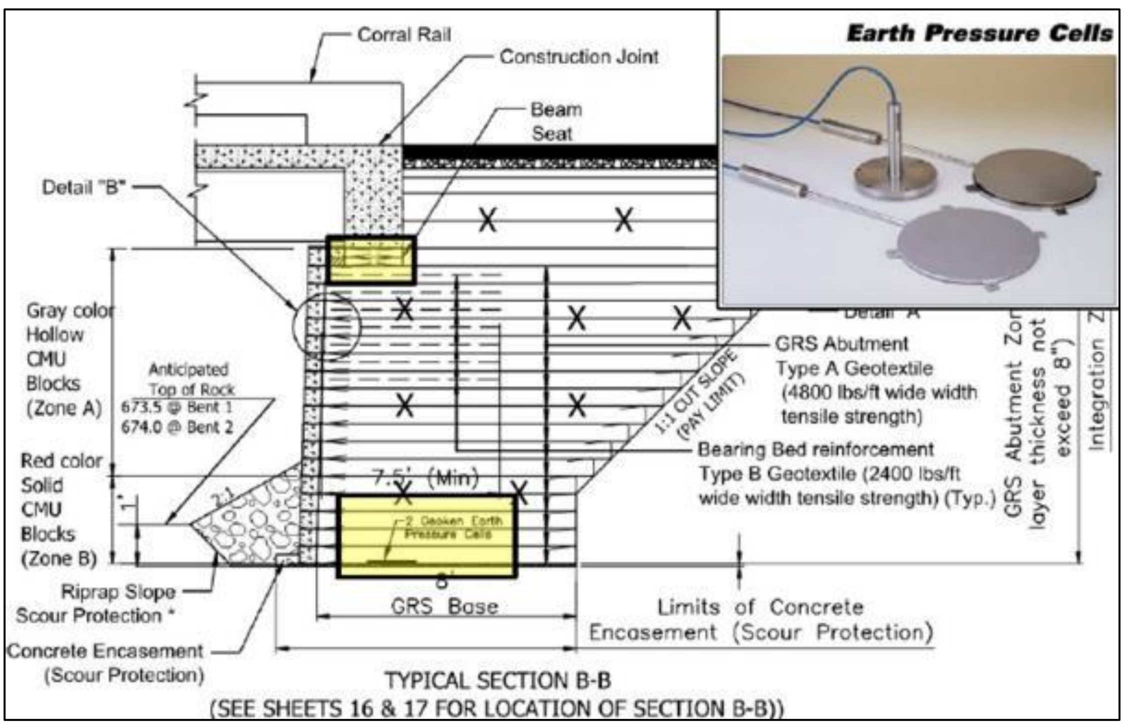
Figure 30: North abutment construction photographs for the CR-55 Bridge in Minnesota (Budge et al. 2014)

2.7.4 Rustic Road Bridge, Missouri

The Rustic Road Bridge was instrumented for performance monitoring purposes as shown in **Figure 31**. In addition to surveying, instrumentation was used to monitor the bridge performance including telltales, tensitometers, earth pressure cells, inclinometers, and Shape Accel Arrays (SAA). **Table 18** shows the summary table for the design features in Rustic Road Bridge.

Table 18: Summary of GRS-IBS bridges in Missouri

County	Bridge	Span m (ft)	Abutment Height (ft)	Bridge Width (ft)	Skew (Degrees)	Cost ($)	Completion Year
Boone	Route B Bridge	19.81 (65.00)	5.79 (19.00)	9.75 (32.00)	31.5	$514,000	2014



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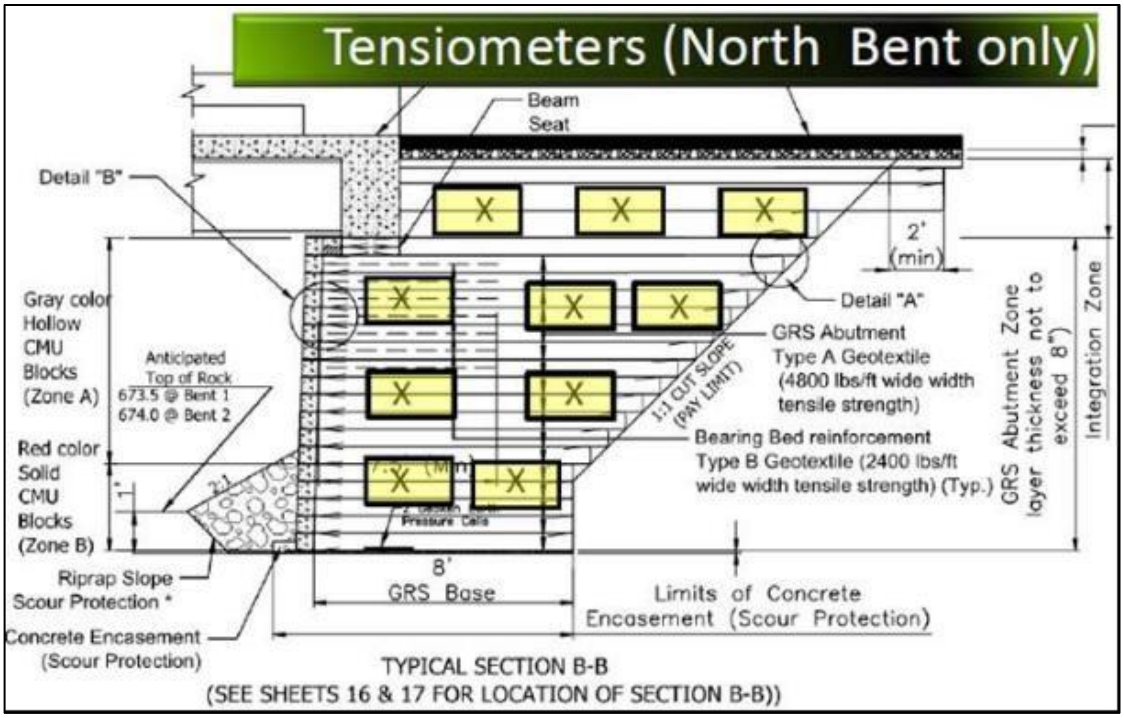

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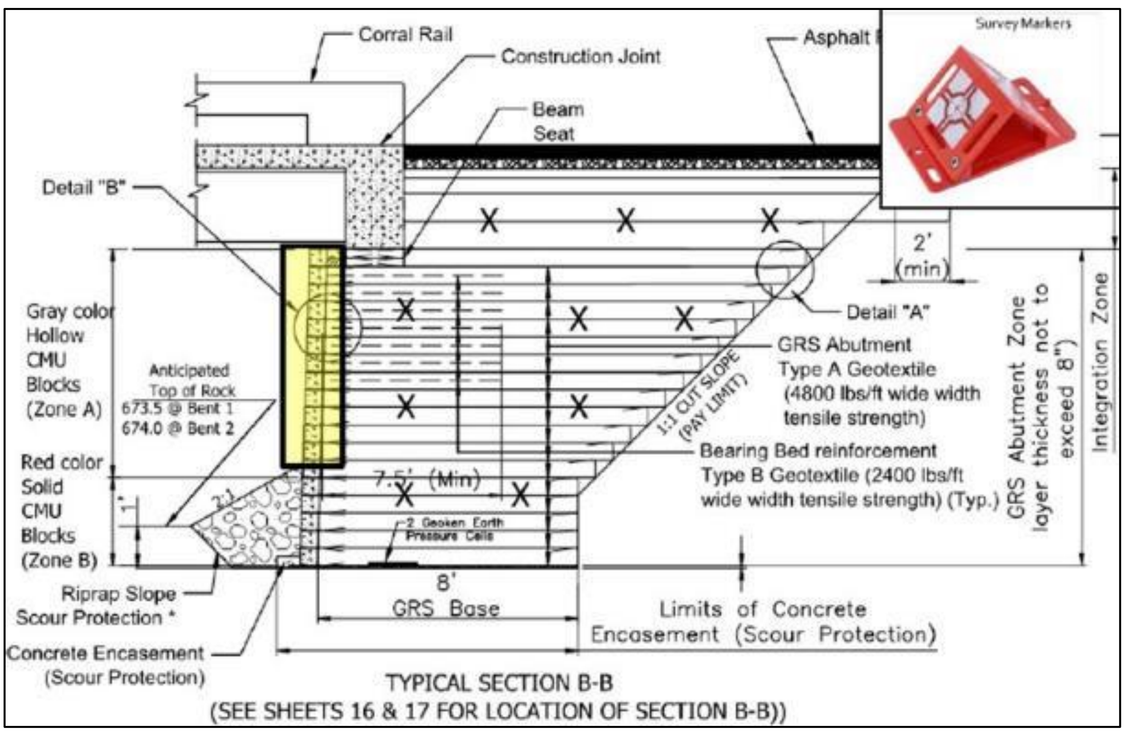


(c)

Figure 31: Instrumentation plans for the Rustic Road Bridge in Boone County, Missouri (a) Earth pressure cells; (b) Tensitometers; (c) Reflective targets

2.7.5 *Mount Pleasant, Pennsylvania*

Mount Pleasant Road Bridge was monitored for a duration of one year using survey techniques (unclassified standard level survey or EDM survey).Albert (2011) reported that 17 survey points were monitored and the vertical settlement of bridge was recorded as 0.25 mm (0.01 in.) , which indicates the bridge performed excellent and the "bump at the end of the bridge" is inexistent (**Table 20** and **Figure 32**). **Table 19** shows the performance monitoring for Mount Pleasant Road Bridge in Clearfield County.

Table 19: Performance monitoring for Mount Pleasant Road Bridge in Clearfield County, Pennsylvania

Bridge	Survey Technique	Monitoring Period	Settlement mm (in)
Mount Pleasant Road Bridge	Survey	1 year	0.25 (0.01)

Table 20: Summary of Clearfield County GRS bridge vertical settlement (Albert 2011)

Point	Elevation (ft)					Description
	10/20/2011	12/7/2011	2/3/2012	4/13/2012	7/23/2012	
A	993.85	993.85	993.81	993.82	993.83	NE Top of GRS Block Row
B		996.09	996.09	996.08	996.08	Ne Top of Cap
C		994.16	994.16	994.16	994.15	NE Top of Sill Plate
D	993.82	993.81	993.77	993.78	993.82	SE Top of GRS Block Row
E		996.07	996.03	996.06	996.06	SE Top of Cap
F		994.14	994.15	994.15	994.14	SE Top of Sill Plate
G	993.87	993.87	993.83	993.84	993.86	SW Top of GRS Block Row
H		996.11	996.09	996.10	996.09	SW Top of Cap
I		994.20	994.20	994.21	994.19	SW Top of Sill Plate
J	993.88	993.89	993.85	993.87	993.87	NW Top of GRS Block Row
K		996.12	996.11	996.11	996.12	NW Top of Cap
L		994.21	994.21	994.20	994.19	NW Top of Sill Plate
M		996.38	996.36	996.36	996.35	NW Top of Deck
N		996.35	996.32	996.32	996.32	NE Top of Deck
O		996.34	996.32	996.31	996.32	SE Top of Deck
P		996.36	996.36	996.36	996.36	SW Top of Deck
Q		996.41	996.41	996.41	996.40	Center of Deck

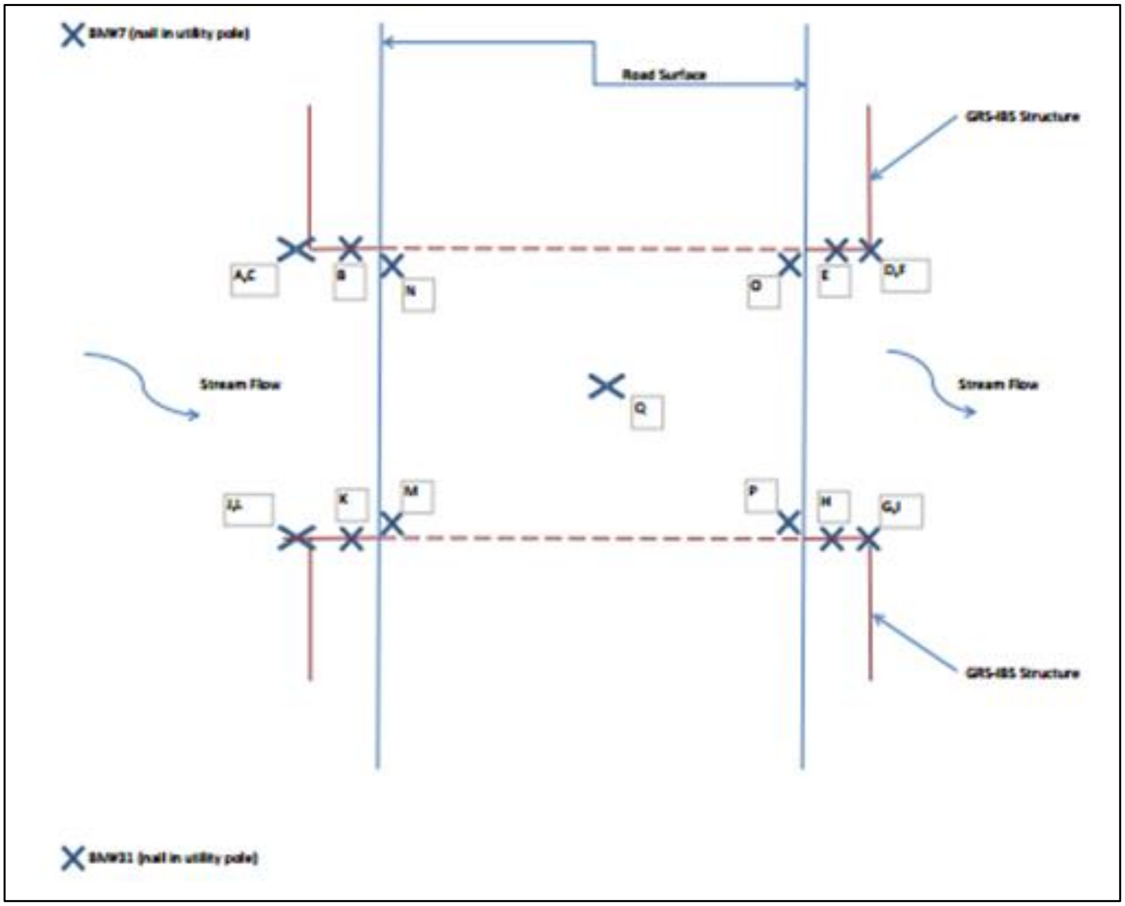


Figure 32: Surveying points for Mount Pleasant Road GRS bridge (Albert 2011)

2.7.6 STH 40 Bloomer Bridge, Wisconsin

Garnier-Villarreal et al. (2014) reported that for the STH 40 Bloomer Bridge the monitoring program included: (1) Foundation, (2) Abutment walls, (3) Deck, (4) Roads, and (5) creek erosion. Standard survey level and rod system with a precision range of ±3.1 mm (0.12 in.) was used to monitor the deformation of STH 40 Bloomer Bridge caused by environmental and service loads. **Figure 33** shows a sketch of the surveying layout with a photograph of the survey target as shown in **Figure 34**.

The setup included a reference point, two surveying positions and several survey points. The elevations of target points on the bridge abutment were determined using the distances and angles between any given two points. Initially, the benchmark at the reference point by the power line pole (denoted as REF in **Figure 35**) was the only point with a known elevation, from which the elevation of surveying position 1 (POS 1 in **Figure 35**) was determined. The elevation of surveying position 2 (POS 2 in **Figure 35**) was then determined by referencing POS 1. The elevation of each surveying point, identified by reflective targets and settlement plates, was then determined from either POS 1 or POS 2, depending on the visibility from the surveying positions.

This procedure of elevation determination needs to be carried out periodically to monitor the elevation changes of each surveying point. Elevation changes indicate vertical displacement, which is determined by computing the difference between the latest and the initial elevation data. **Figure 36** shows the nomenclature and description of survey points for the STH 40 over Hey Creek Bridge in Wisconsin. Preliminary results show that the bridge has performed as expected, showing no differential settlement and therefore proving a simple, cost-effective solution for replacing deficient bridges. **Table 21** shows the summary of monitoring of settlements in STH 40 over Hey creek project.

Table 21: Summary of monitoring of settlements in STH 40 over Hey creek project

Bridge	Survey		Monitoring Period (yrs)
	Technique	Precision m (in)	
STH 40 Bloomer over Hay Creek	Conventional Survey	0.12	2

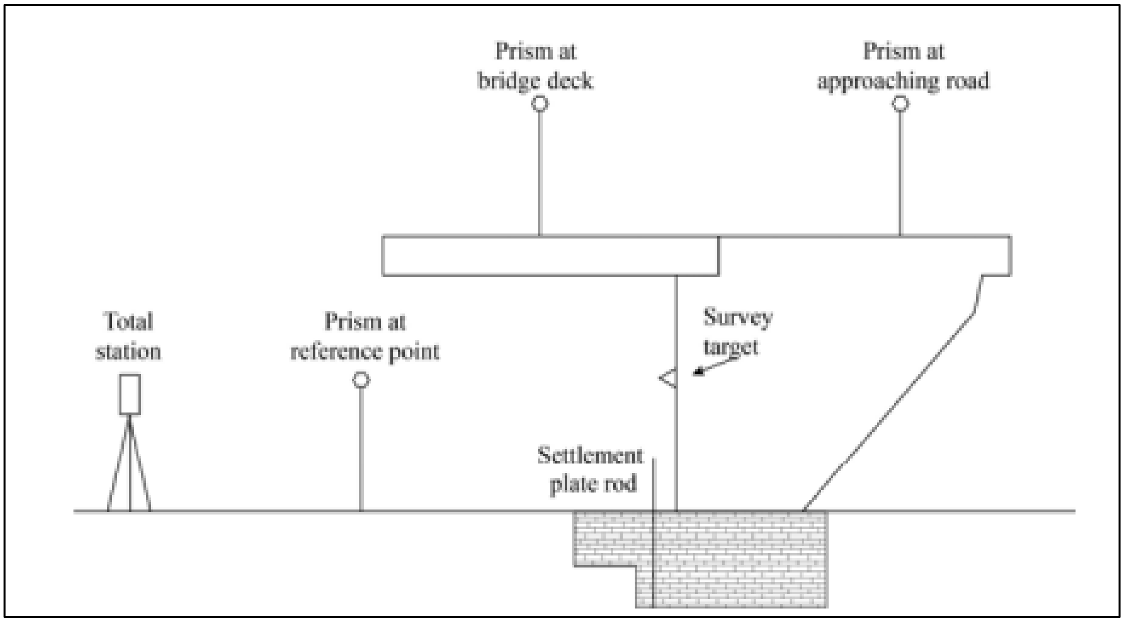


Figure 33: Surveying set up use to measure the deformation of the GRS bridge in Wisconsin (Garnier-Villarreal et al. 2014)

Figure 34: Surveying reflective target installed on abutment wall for STH 40 Bloomer Bridge (Garnier-Villarreal et al. 2014)

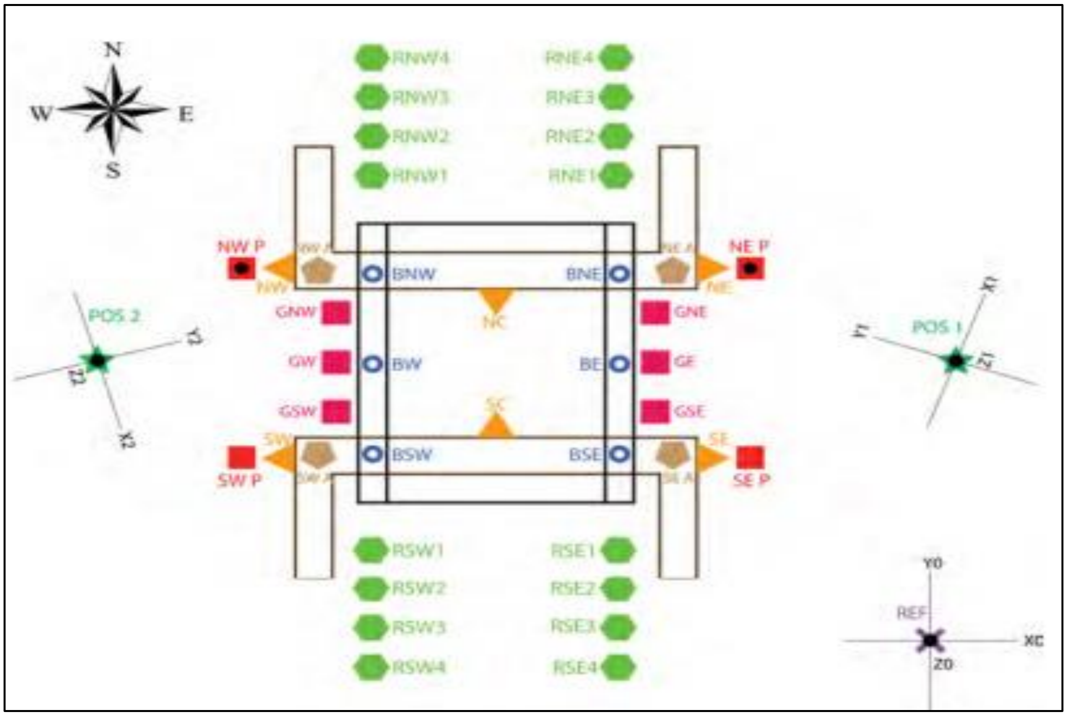


Figure 35: Locations of survey points on the STH 40 over Hey Creek Bridge in Wisconsin (Garnier-Villarreal et al. 2014)

Surveying point	Description
NC/NE/NW	Surveying targets on north abutment wall: central, east and west locations
SC/SE/SW	Surveying targets on south abutment wall: central, east and west locations
BE/BNE/BSE	Surveying points on bridge deck: east, northeast and southeast locations
BW/BNW/BSW	Surveying points on bridge deck: west, northwest and southwest locations
NE P/NW P/SE P/SW P	Settlement plates in foundation soil: northeast, northwest, southeast and southwest locations
RNE/RNW/RSE/RSW	Approaching road points: northeast, northwest, southeast and southwest locations
NE A/NW A/SE A/SW A	Points on top of the abutment walls' corners: northeast, northwest, southeast, and southwest locations
GNE/GE/GSE/GNW/NW/GSW	Creek surveying points: northeast, east, southeast, northwest, west and southwest locations

Figure 36: Nomenclature and description of survey points for the STH 40 over Hey Creek Bridge in Wisconsin (Garnier-Villarreal et al. 2014)

Chapter 3. Analysis of Data on Surveyed GRS-IBS Projects

Based on the literature review performed by Hatami et al. (2015), Hatami et al. (2016), Ngo (2016), and the author across the United States: a summary, development of a web page, and analysis factors such as cost facing type, traffic volume, and performance monitoring methods are presented in this chapter.

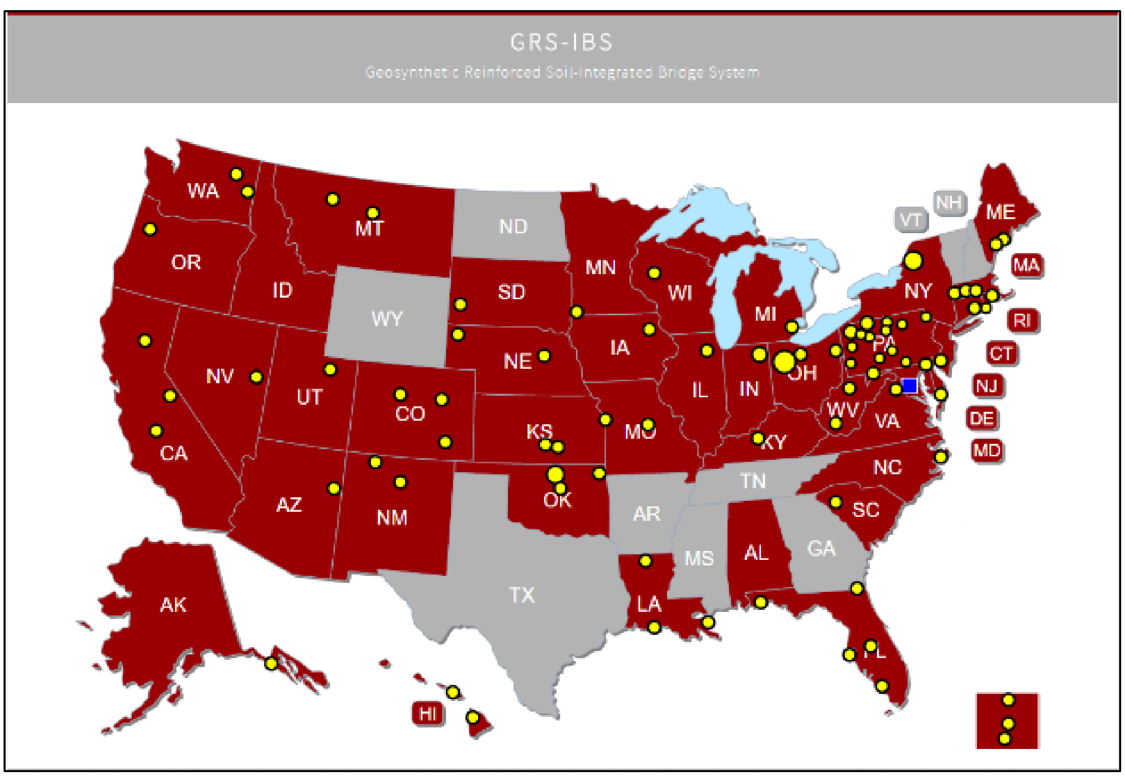
3.1 Webpage-based Database of GRS-IBS Projects in the U.S.

The development of a web page containing the surveyed case studies started in 2016. The objective was to produce a tool that would give designers and constructors access to information about the good practices and improvement potentials of the documented cases across United States.

The first stage was maintaining the database created by Hatami et al. (2015), Ngo (2016) and the author (i.e. checking for data errors, erasing duplicated fields, etc.). Additionally, while performing this task, the author added case studies to the database. This database was maintained in Microsoft Acces ® and exported to Microsoft Excel ®.

Initially, the queries were based on the following fields (Ngo 2016): Span length, abutment height, superstructure width, cost, completion year, construction time, surface road type, superstructure material, instrument monitoring, reported deformations, facing blocks material, geosynthetic type and specification, construction technique, geotechnical data, hydraulic data and some remarks.

However, based on the inconsistent information from the available sources, the author decided to discard certain fields and instead develop the web page with the following fields: ID, county, span length, abutment height, width, area, cost, completion year, facing type and source reference. **Figure 37** shows example screenshots of this developing website, which will have the following features: (1) A front page, which contains an interactive map of the United States, with participating and nonparticipating states in the FHWA GRS-IBS program differentiated using crimson and gray colors (A user can access documented GRS-IBS projects in a crimson-colored state from the database by clicking on the yellow drop pins that indicate the locations of existing bridges in that state (**Figure 37a**)); and (2) subsequent state-specific webpages, which describe technical specifications, features, and lessons learned references and pictures of documented bridges (**Figure 37b**).



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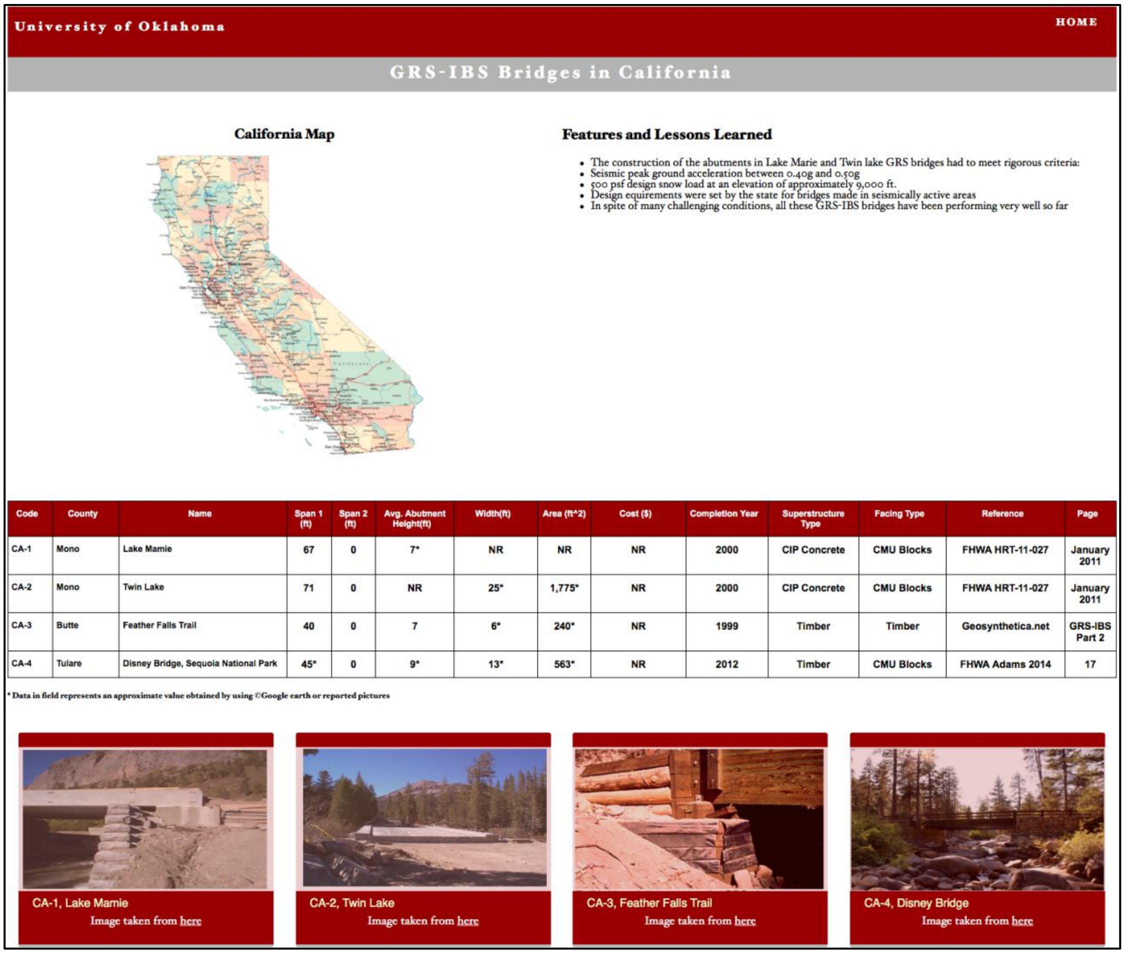


Code	County	Name	Span 1 (ft)	Span 2 (ft)	Avg. Abutment Height(ft)	Width(ft)	Area (ft^2)	Cost ($)	Completion Year	Superstructure Type	Facing Type	Reference	Page
CA-1	Mono	Lake Mamie	67	0	7*	NR	NR	NR	2000	CIP Concrete	CMU Blocks	FHWA HRT-11-027	January 2011
CA-2	Mono	Twin Lake	71	0	NR	25*	1,775*	NR	2000	CIP Concrete	CMU Blocks	FHWA HRT-11-027	January 2011
CA-3	Butte	Feather Falls Trail	40	0	7	6*	240*	NR	1999	Timber	Timber	Geosynthetica.net	GRS-IBS Part 2
CA-4	Tulare	Disney Bridge, Sequoia National Park	45*	0	9*	13*	563*	NR	2012	Timber	CMU Blocks	FHWA Adams 2014	17

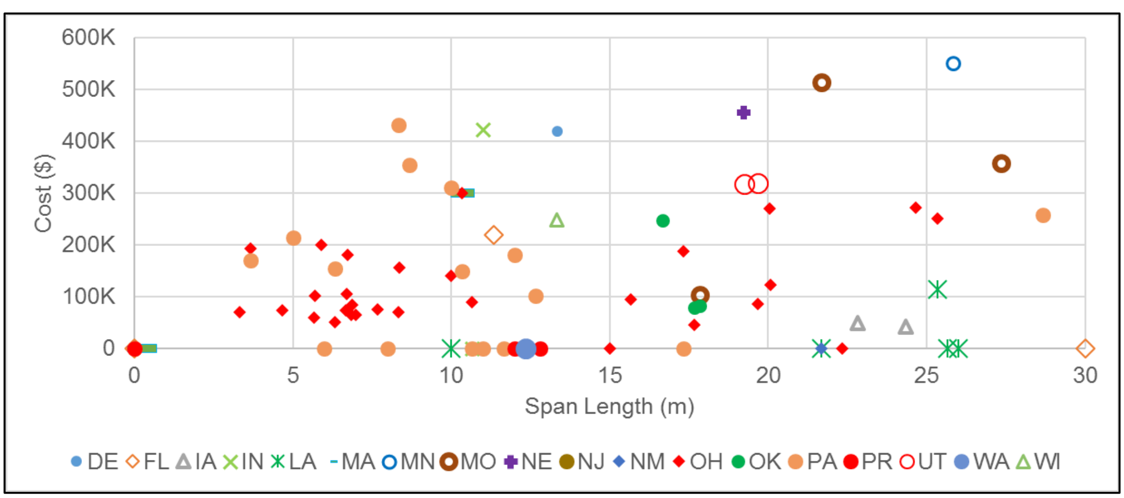
* Data in field represents an approximate value obtained by using ©Google earth or reported pictures

(b)

Figure 37: Screenshots of GRS-IBS webpage (under construction): (a) Index U.S. map highlighting states with GRS-IBS projects; (b) Example GRS-IBS bridges in California

3.2 Cost

Ngo (2016) reported that the savings resulting from selected GRS-IBS range from 16% to 63% less expensive than conventional alternatives. In addition, the abutment cost per square meter is $1,026 vs $2,239 (square foot is $95.5 vs $208.5) for traditional abutments in Pennsylvania. **Figure 38** shows that the cost increases by approximately $25,000 for every 3 m (10 ft.) span length, with some exceptions around span lengths of 10 m -12 m (30 ft.-40 ft.). Likewise, **Table 22** shows that the linear regression of cost vs span length has an R Squared of 0.42, which is similar to the one reported by Ngo (2016) of 0.53 in for the state of Pennsylvania. Also, **Figure 39** shows that most of the documented bridges have a span length between 0 to 10 m (0 to 30 ft.) and cost between $0 and 100,000. **Figure 40** shows no clear correlation between the abutment height and cost; as the total cost is driven by various factors, such as span length, superstructure type, transportation, and labor. However, **Figure 41** shows that most of the documented bridges have an abutment shorter than 4 m (13.09 ft.). The aforementioned factors ratify the feasibility of this technology for bridges within the span length, abutment height and budged.



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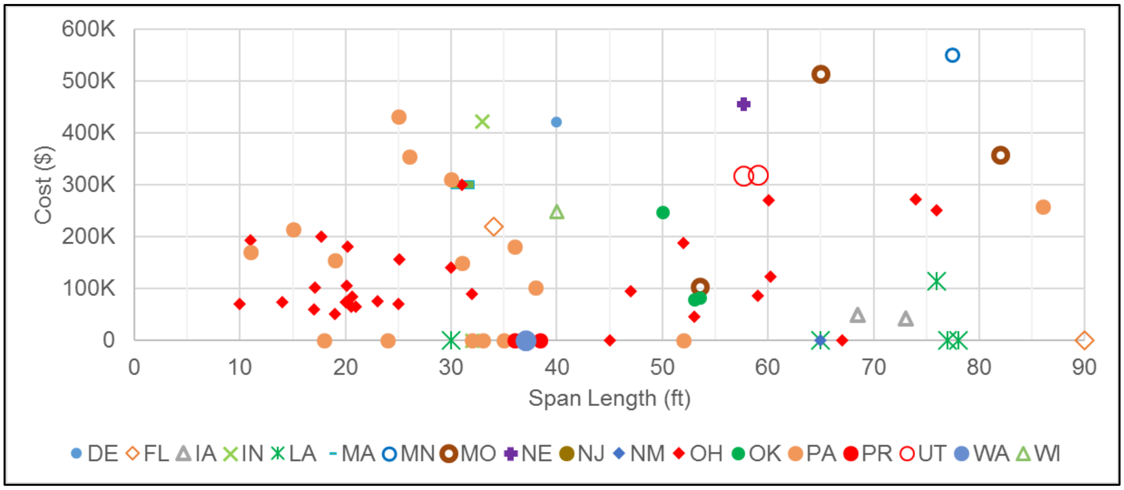
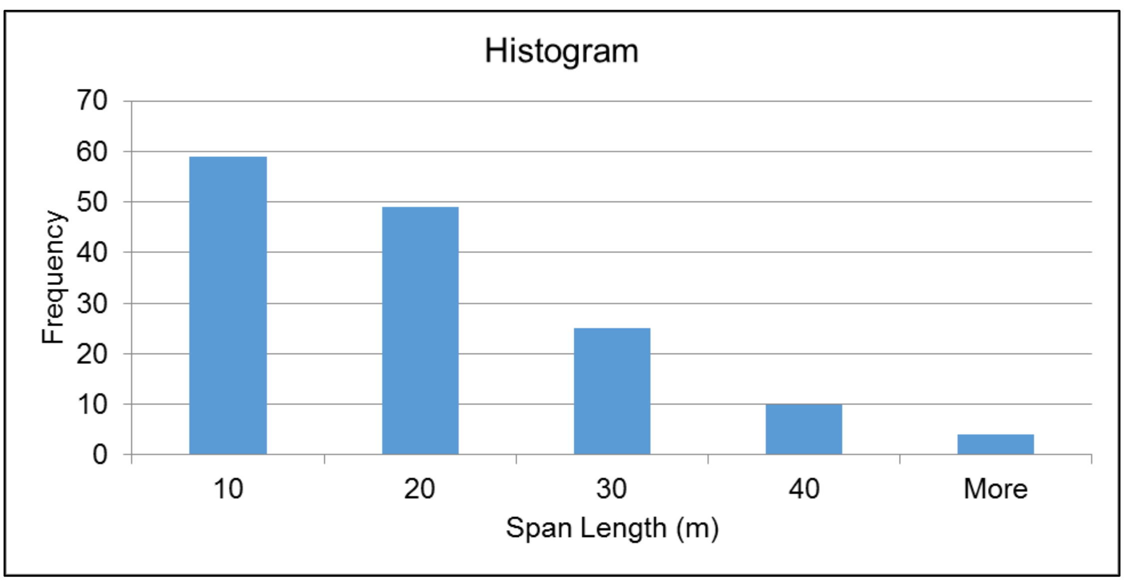


(b)

Figure 38: Cost vs Span Length (Cost < $600,000): (a) SI units; (b) Imperial units

Table 22. Cost vs Span Length linear regression

Linear regression output			
Regression Statistics			*Coefficients*
Multiple R	0.649852205	Intercept	1,070.93
R Square	0.422307888	X Variable 1	7,702.58
Adjusted R Square	0.418323804		
		Cost =	
Standard Error	319936.8641	$1,070 + length(m) * 7,702 ($/m)	
Observations	147		



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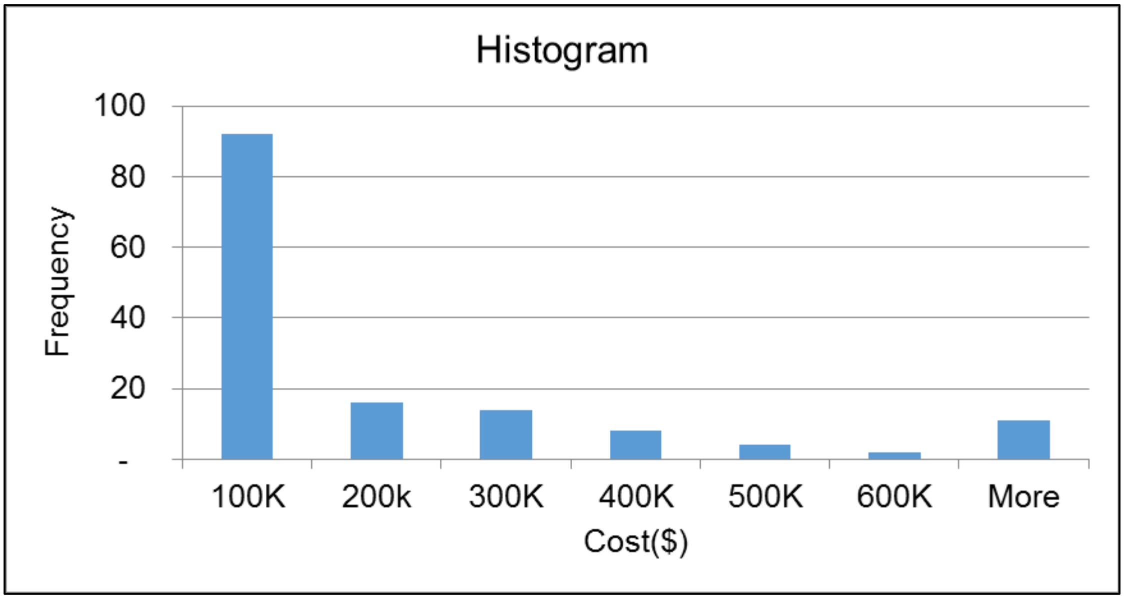
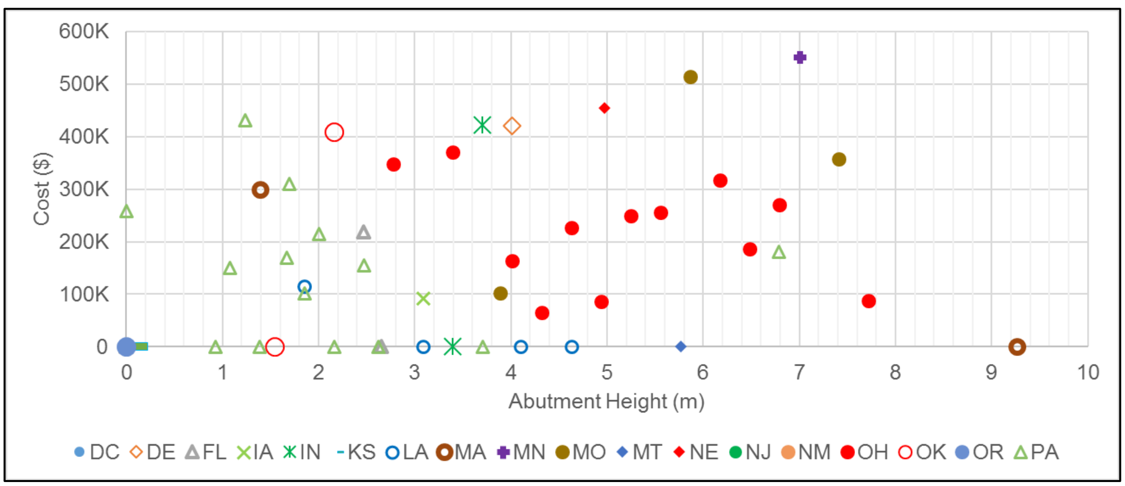


(b)

Figure 39: Histogram: (a) Span length (m); (b) Cost ($)



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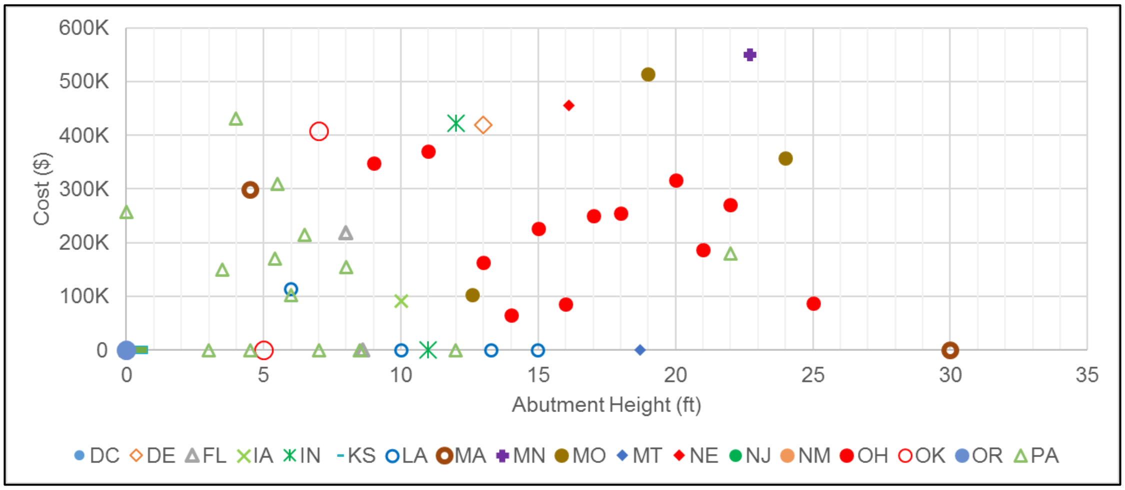


(b)

Figure 40: Cost vs Abutment Height: (a) SI units; (b) Imperial units

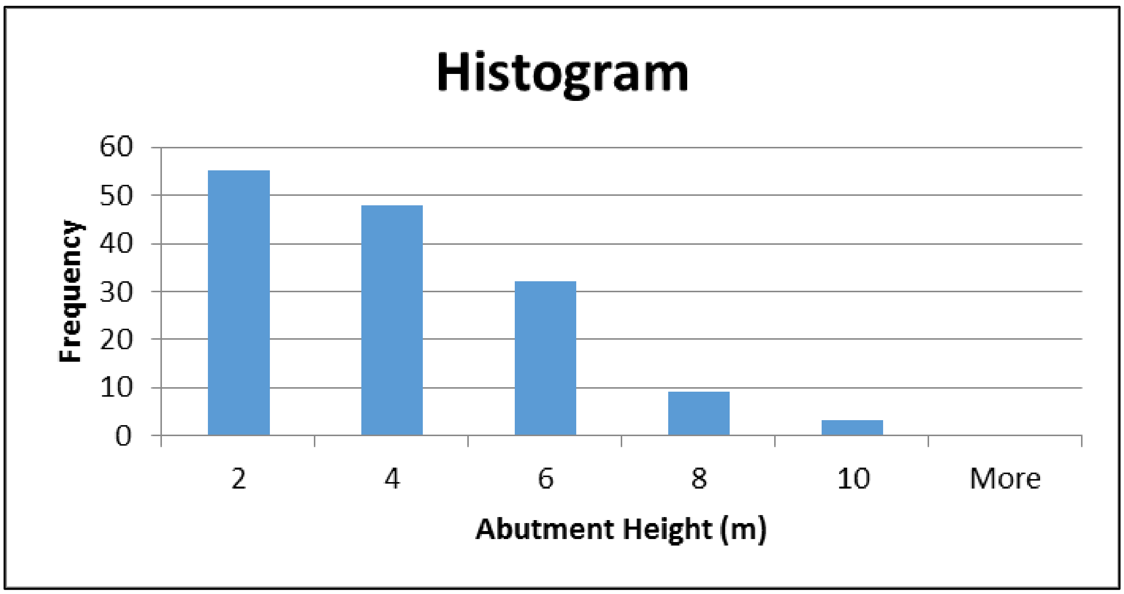


Figure 41: Histogram abutment height (m)

3.3 Facing Type and Superstructure Type

As explained in section 2.2, the main purpose of the facing type is to protect the backfill from weathering, and serve as a façade which facilitates backfill compaction. **Table 23** and **Figure 42** show the updated information presented by Hatami et al. (2015), Hatami et al. (2016) and Ngo (2016).

Out of 144 surveyed projects, 33% didn't report the facing information. 52% were built with CMU due to their low cost and installation ease. Adams et al. (2012) presents CMU blocks as the standard facing type. However, this facing type is only suitable for GRS abutments built in zones with stream velocities less than 7 fps. If the stream velocity is within 7fps to 10fps, then the blocks must be reinforced with rebar and grout.

Sheet piling was found in 6% percent of the projects surveyed. Within that percentage, Kay County represent 25% of it. Two GRS-IBS bridges where built with this facing type. The main advantage of it is the ease of installation with a track hoe and/or excavator, which reduces the construction time as reported in Kay County. However, it is worth noting that sheet piles shall have perforated holes in order to account for drainage in situations such as the flash flood event in Kay County (Ngo 2016).

Large precast block were found to be the third most common facing type, comprising only 3% of the total surveyed GRS-IBS. Large precast blocks are the facing type used when stream velocities are faster than 10 fps.

Table 23: Updated GRS-IBS facing types (Hatami et al. 2015; Ngo 2016)

Facing Wall Type	Nominal Dimension	Number of Bridges	Percentage
CMU	203mm × 203 mm × 406 mm (8 in. × 8 in. × 16 in)	75	52%
Steel piling	N/A	8	6%
Large precast block	457 mm × 1168 mm × 711 mm (18 in. × 46 in. × 28 in.) and 406 mm × 1219 mm × 610 mm (16 in. × 48 in. × 24 in.)	4	3%
Treated timber panels	152 mm × 152 mm (6 in. × 6 in.)	3	2%
Cellular Confined System (CSS)	152 mm (6 in. tall)	2	1%
Flexible geosynthetic wrapped facing	Each Reinforced Spacing	1	1%
Pre-cast panels	203 mm (8 in.) thick	1	1%
Segmental Retaining Wall (SRW)	N/A	1	1%
Redi-precast modular blocks	152 mm (6 in. tall)	1	1%
Not reported	Not reported	48	33%

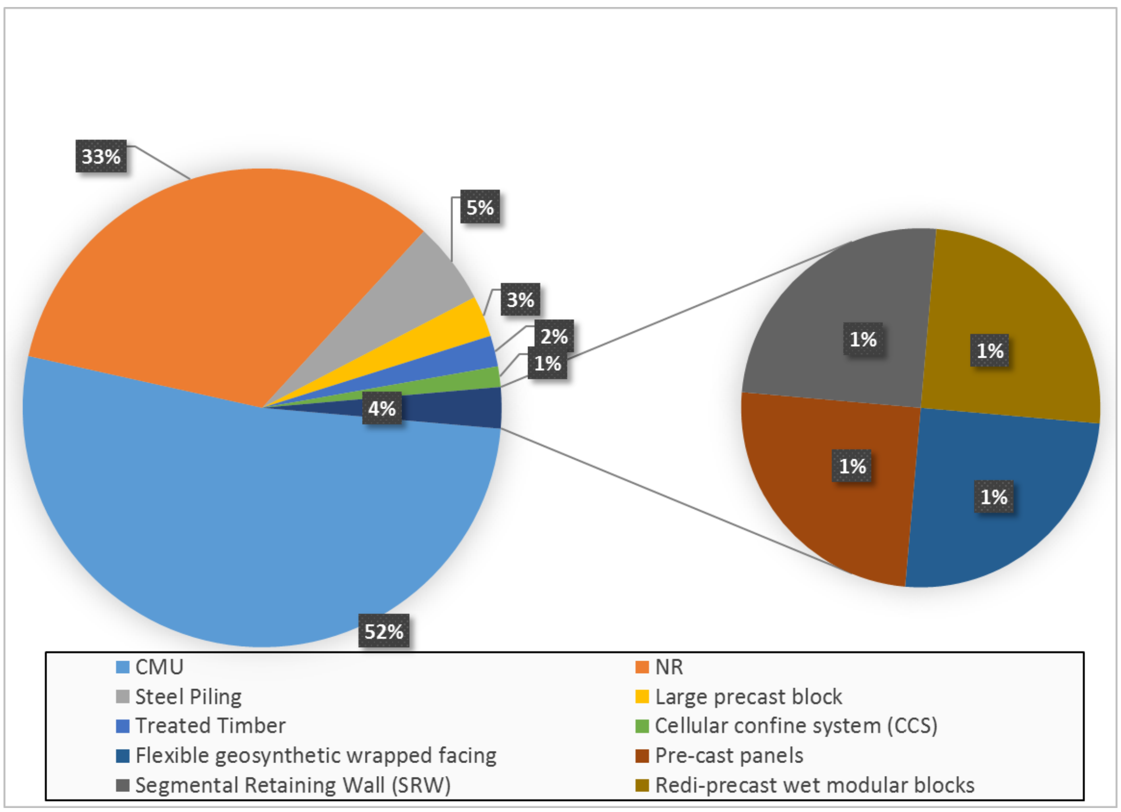


Figure 42: Pie chart distribution of facing types reported in the surveyed GRS-IBS projects (From a total of 144 projects)

Out of 144 surveyed projects, 46% didn't report the superstructure information. 58% of the reported cases used cast in place (CIP) concrete as their superstructure type. One of the main reasons of this is due to the knowledge of the constructability method, and sometimes the reduced cost compared to other superstructure types. However, this superstructure type will take more construction time than precast concrete slabs, which were reported as the second most used superstructure method.

Precast concrete slabs were found in 23% percent of the projects surveyed. Their main advantage is the speed with which they can be installed. However, for bridges with long span lengths, the precast slabs must be installed with cranes, which in-turn increases the cost. As reported in Kay County, the bridge that was constructed with precast slabs cost 25% more than a CIP concrete superstructure bridge. It is worth noting that one of the main advantages of precast slabs is that the bridge can be opened for use right after the installation; which doesn't occurs with the CIP concrete, that needs a minimum time of 28 days to gain the service strength.

Other types of superstructures: such as prestressed, and precast-prestressed timber, were reported on 19% of the remaining bridges. **Table 24** and **Figure 43** show the quantity of superstructure types reported.

Table 24: GRS-IBS superstructure types

Superstructure Type	Number of Bridges	Percentage
CIP Concrete	45	58%
Precast Concrete	18	23%
Timber	6	8%
Prestressed Concrete	2	3%
Precast, Prestressed Concrete	2	3%
Other	4	5%

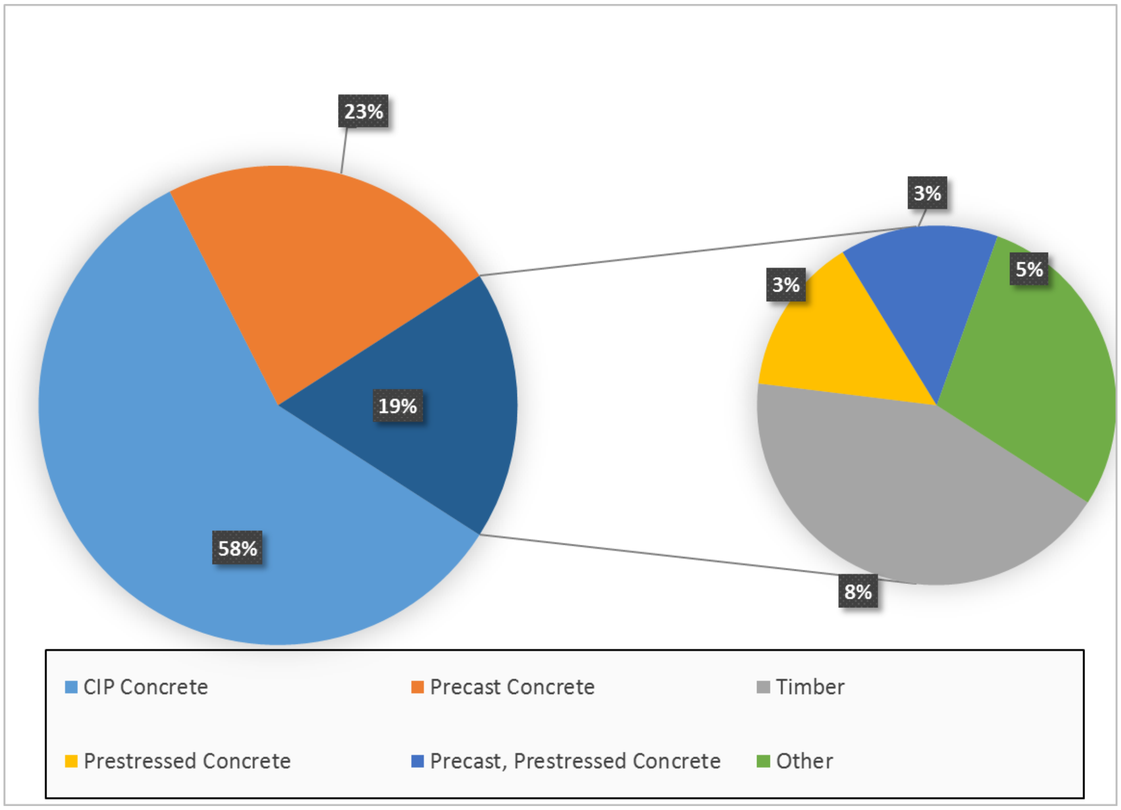


Figure 43: Pie chart distribution of facing types reported in the surveyed GRS-IBS projects (From a total of 77 projects)

3.4 Traffic Volume AADT

Out of 144 projects, 49% didn’t report the traffic volume. However, based on the geographical location during the development of the web page, the author could establish if the GRS-IBS bridge was a low volume road (specifically a local rural road). Per the FHWA 2013 report, low volume roads are defined as those with an AADT<400 in rural areas or an AADT<700 in urban areas. **Table 25** and **Figure 44** show the updated histograms reported by Ngo (2016) and Hatami et al. (2016). This figure shows that out of 85 bridges, 71 were built on low-volume roads. This confirms the recommendation provided by Adams et al. (2012), which states that GRS-IBS shall be primarily built for low-volume roads.

Table 25: Statistics on AADT values GRS-IBS project across the U.S.

	Functional Classification	AADT	Reported Bridges
Rural roads	Local	15-400	47
	Minor Collector	150-1110	8
	Major Collector	300-2600	2
	Principal Arterial (interstate)	12000-34000	2
Urban roads	Collector	1100-6300	1
	Principal Arterial (interstate)	34500-129000	2
Not Reported	NR 1	<400	16
	NR 2	400-1110	5
	NR 3	>1110	2
		Total Bridges	85

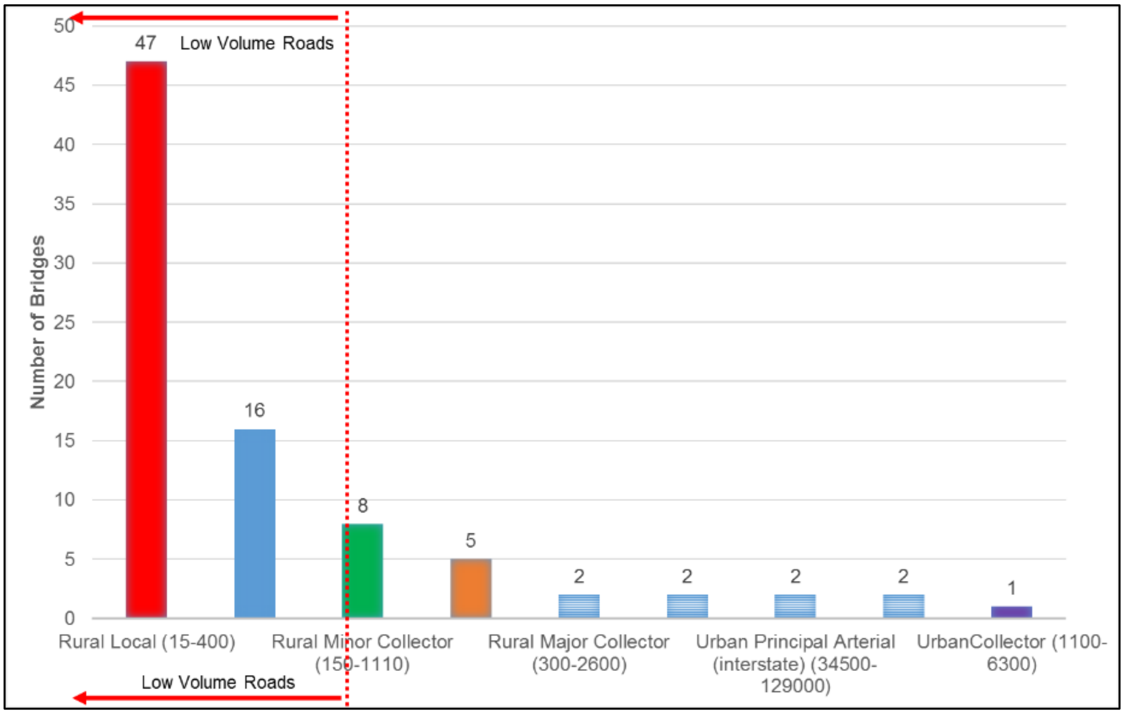


Figure 44: Classification of reported roads according to their AADT value

3.5 Performance Monitoring

Vertical and lateral deformations are essential because they are related to the serviceability of the bridge. Because GRS-IBS is still considered a new technology, the monitoring of different bridges shows the technical benefits of this technology. Also, lateral deformation can be correlated to vertical deformation as reported by Adams et al. (2012). Out of 144 projects: 68% didn't report the monitoring technique, 35% of the reported cases used a TST, 20% used piezometers, and 11% used inclinometers. Ngo (2016) and Hatami et al. (2016) reported that the maximum deformation was recorded in Tiffin river bridge, OH (5.33 cm~ 2.1 in.) which is almost 4 times bigger than the recorded deformation in Kay County, OK (1.2cm ~0.5 in.). An update of the survey of instruments that were reported by Ngo (2014), Hatami et al. (2015) and Hatami et al. (2016) for the GRS projects across the U.S. is summarized in **Table 26**.

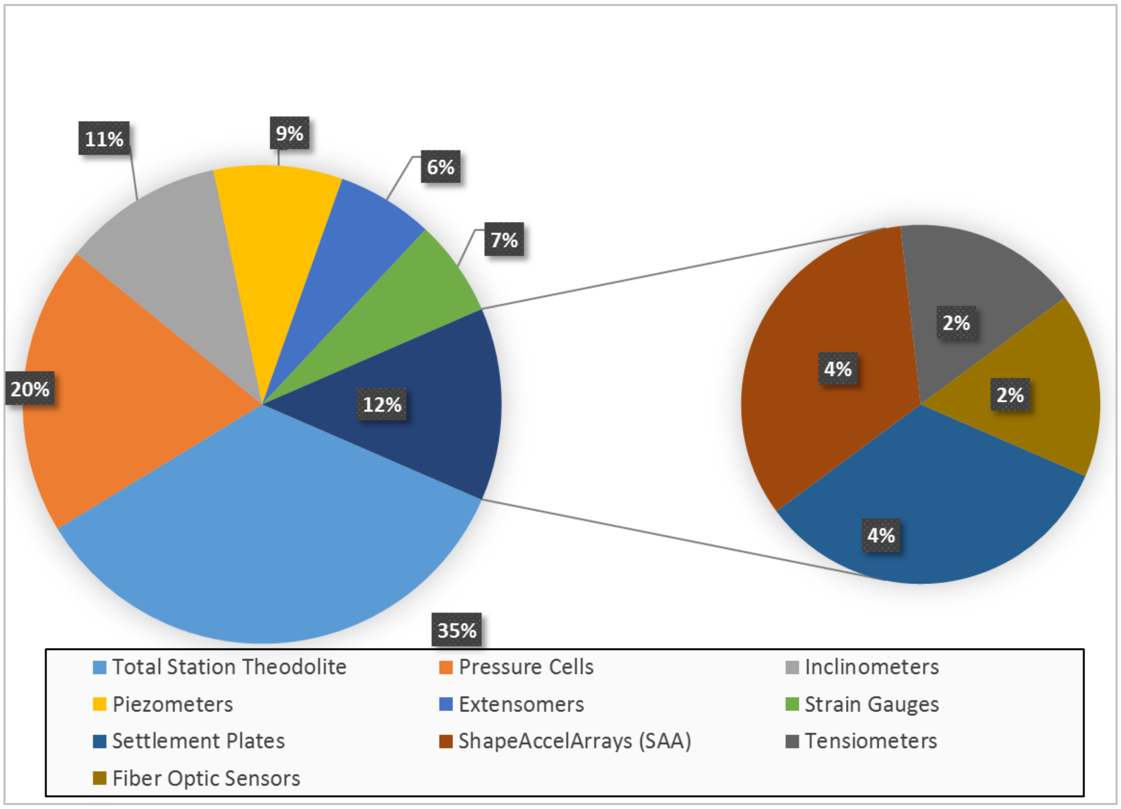


Figure 45: Pie chart distribution of monitoring types reported in the surveyed GRS-IBS projects (From a total of 46 projects)

Table 26: Selected GRS-IBS bridges with a reported performance monitoring program (Hatami et al. 2015; Hatami et al. 2016; Ngo 2016)

State	Bridge	Instrumentation Type	Survey Period	Bridge Settlement	Lateral Def.
DE	BR 1-3661	Surveying, inclinometer sensors, piezometers, pressure cells, strain gauges, thermistors, volumetric water content sensors	N/A	N/A	N/A
HI	Kauaula Stream Bridge	Surveying	12 months	2.16 cm (0.85 in.)	2.54 cm (1 in.)
IA	250th Street	Inclinometers, piezometers, semiconductor and vibrating wire earth pressure cells	12 months	1.27cm (0.5 in.)	1.02 cm (0.4 in.)
	Olympic Avenue Bridge	Surveying	14 months	1.78 cm (0.7 in.)	0 cm (0 in.)
LA	Cecil Creek	Inclinometers and extensomers	5 months	3.00 cm (1.18 in.)	N/A
	Big lake	inclinometers and extensomers	5 months	0.89 cm (0.35 in.)	N/A
	Cut off Creek	inclinometers and extensomers	5 months	2.39 cm (0.94 in.)	N/A

State	Bridge	Instrumentation Type	Survey Period	Bridge Settlement	Lateral Def.
MA	SR 7A over Housatonic RR	Pressure Cell, Inclinometer	N/A	N/A	N/A
MN	CR 55 over MN Southern Railway	Horizontal and Vertical ShapeAccelArray (SAA), Vibrating-wire (VW) Earth pressure cells (EPC), optical prism, weather station	10 months	4.32 cm (1.7 in.)	4.83 cm (1.9 in.)
MO	Rustic Road Bridge	Surveying on facing wall, Earth Pressure Cell, Tensiometer, Telltale, inclinometer, SAA	N/A	N/A	N/A
MT	US HGW 89 south of Dupuyer	Surveying	19 months	0.91 cm (0.36 in.)	N/A
NC	East Canal Bridge	Standpipe piezometers and settlement plates	N/A	1.52 cm (0.6 in.)	N/A
	Mattamuskeet National Wildlife Refuge - Central Canal Bridge	Standpipe piezometers and settlement plates	N/A	1.22 cm (0.48 in.)	N/A
OH	Bowman Road	EDM and total station, Earth pressure cells, strain gauge	20 months	2.16 cm (0.84 in.)	0.51mm (0.0)
	Vine street	Surveying	40 months	1.07 cm (0.42 in.)	3.3 cm (0.1 in.)
	Glenberg road	Surveying	43 months	3.25 cm (1.28 in.)	0.32 in
	Tiffin River	EDM and total station, Vibrating wire earth pressure cell	18 months	5.33 cm (2.1 in.)	0.04 in
	Huber road	Surveying	40 months	1.27 mm (0.05 in.)	2.0 mm (0.0 in.)
OK	Bridge 2	Surveying	18 months	1.27 cm (0.5 in.)	N/A
	Bridge 3	Surveying	18 months	1.27 cm (0.5 in.)	N/A
	Bridge 4	Surveying	18 months	1.27 cm (0.5 in.)	N/A
	Bridge 5	Surveying	18 months	1.27 cm (0.5 in.)	N/A
PA	Mount Pleasant Road Bridge	Surveying	7 months	0.91cm (0.36 in.)	N/A
PR	1121 Bridge (West Bound)	Pressure Cells and geosynthetic fiber-optic sensors	NR	NR	N/A
WI	STH 40 Bloomer over Hay creek	Surveying	10 months	1.47 cm (0.58 in.)	N/A

3.6 Reported Problems and Lessons Learned in Different States

Table 27 shows an update of the summary of lessons learned on different aspects of GRS-IBS projects reported by Hatami et al. (2015), Hatami et al. (2016) and by Ngo (2015). Also, **Figure 46** shows some examples of the implementation of lessons learned and improvement potentials in the web page.

Table 27: Reported problems and lessons learned in GRS-IBS construction across the U.S. (Hatami et al. 2015; Hatami et al. 2016; Ngo 2016)

	State	Lessons Learned/Issues Found
Knowledge	DE	* Inspectors need to understand how the GRS-IBS work
Attitude	OH ME NY	* The most vital lesson was a readiness to try it with an open mind * Staff attitude towards new technology is crucial * NYDOT 50% of the county bridges are suitable to be replaced using GRS-IBS technology
Experience	DE MT NC OH	* Allow for learning curve, so the second abutment will be much better than the first * The contractor needs to provide proper training to their project managers and workers on basic elements of assembling this type of bridge support * Highly dependent on contractor's QA/QC; otherwise could become distorted during construction * Taking advantage of others' experiences is crucial
Cost and Time	NY	* Construction would be more expensive in water
	MA	* 49% cost saving as compared to micropile foundations
	NM	*58% savings in the cost project *44% savings in its construction time relative to a conventional design
	NY	*Savings in materials, labor and equipment *Consistent cost saving of 50% as compared to other methods
Design	CA	*Seismic rigorous criteria met including peak ground accelerations between 0.40g and 0.5g
	MO OH CO	* Check buoyancy and consider anchorage * GRS-IBS design is about getting comfortable that it acts as a composite material
	ME	*GRS-IBS can have more than one span (e.g. 3 spans) and reinforced piers.
Equipment	MO	* Big roller compactor next to blocks was not a concern
Geosynthetics	MO	* Geogrid orientation and placement are key * Additional geotextile behind facing blocks
Backfill Materials	MO	* Using an open graded granular backfill increases production and can reduce testing requirements * Material availability

	State	Lessons Learned/Issues Found
	MT	* One fill layer was overly saturated and had to be removed and replaced with new backfill * Excessive water in the backfill during compaction should be avoided
	PR	* Only open-graded material is permitted and it is easier to source in Puerto Rico making it faster to place and compact * The compaction process can affect the alignment of the hollow blocks on the well-graded materials because a 95% compaction is required. In this case, the loose materials caused increased forces on the blocks, which made them outward
Spacing	IA	* Ultimate Tensile Strength of geosynthetics ≥ 4,800 lbs/ft and good permeability (30gal/min/ft^2) is required
Foundation	IA	* Avoid the excavation at the toe of slopes because of its instability. Any excavation at the toe of slope must be done before constructing the fill layer * Subsurface soil information before bridge construction is important
	NY	*Having a dry construction zone for the reinforced soil foundation yield cheaper and faster results
Bearing Capacity	IA	* Evaluate the bearing capacity in full-scale field testing to failure to determine the ultimate bearing capacities with different backfill and geosynthetic materials
	NC	*Soft clays in the subsurface profile. Thus, preloaded for settlement prevention
Facing Block	MO MT PR	* Hollow facing blocks were pushed outward during compaction
	DE	* East abutment with broken blocks * 3/4 in. wide joint gap in 2nd row from top * If the edges are too smooth, the blocks slide easily; thus, a batter is necessary to allow movement * First course of block is vital. Must be straight, level and plumb
	MO	* Wet cast block is more durable * Dry cast CMU block does not meet freeze-thaw requirement *Large block are less uniform in size than regular CMU
	PR	* Solid blocks with a minimum weight of 66 pounds (30 kilograms) for the facing of the abutments. Lighter (hollow) CMU (~45 lbs) will be easily pushed out during the compaction.
	OK	* The abutments' leaning profiles and some gaps in the facing blocks
	MT	* A frontal gap was created at the abutment corner radius caused by rectangular shape of CMU block * Grout patching of the gaps between the blocks is substandard
	NC	*Cellular confinement system (CSS) used as facing wall and scour countermeasure.
	ME	*RediRock large wet cast blocks outperform in tidal and marine environment
Bidding	OH MO	* Good education prior to bidding is essential
	MO	* Allow flexibility in the construction timeframe

	State	Lessons Learned/Issues Found
Performance monitoring	IA	* Must evaluate long-term performance of GRS abutment with different facing elements (sheet piles, CMUs, and timber-faced wall) * Must evaluate long-term performance of GRS abutment with different granular fills (sheet piles, CMUs, and timber-faced wall) * Must evaluate long-term performance of GRS abutment with different geosynthetics (sheet piles, CMUs, and timber-faced wall) * TST most efficient monitoring technique
	OH	* More than 20 bridges using GRS abutments have performed well
.Scour Countermeasures	MD	* A 1.17 m (46 in.) thick bed of Class III riprap underlaing by geotextile fabric was placed across the complete stream channel

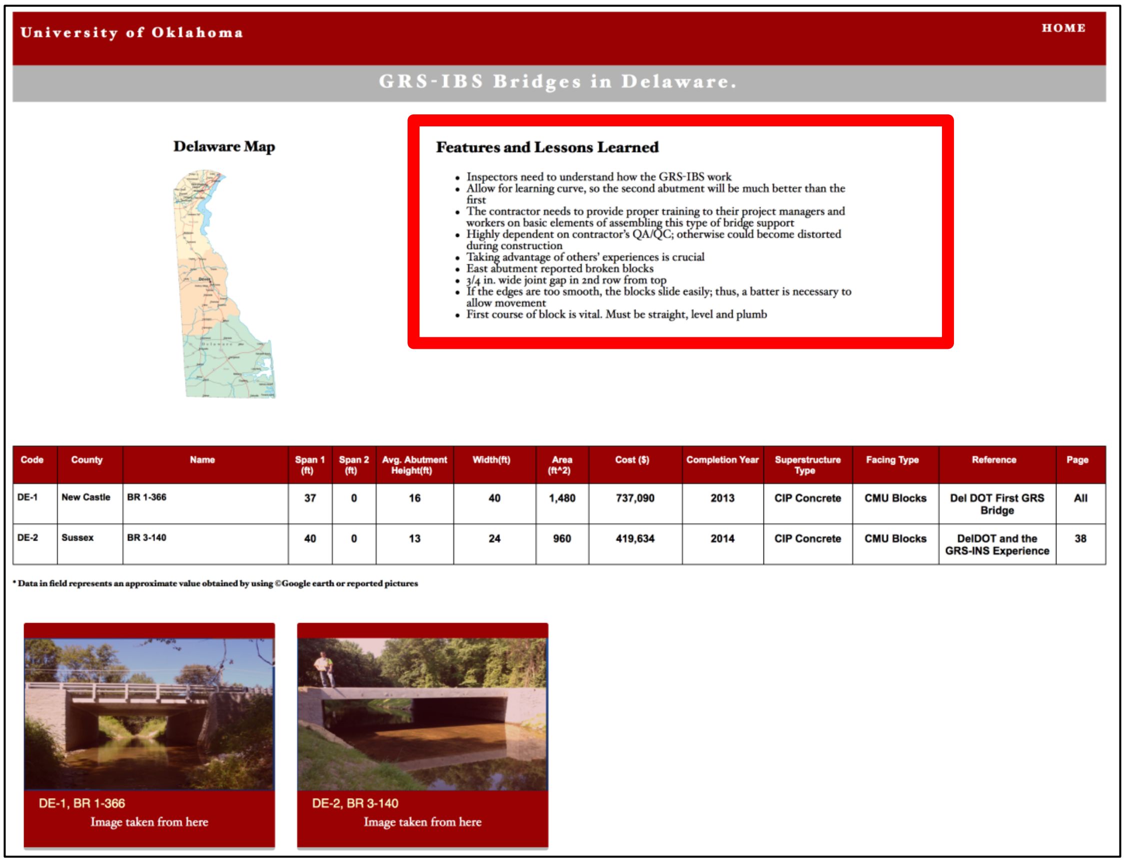


Figure 46: Screenshots of GRS-IBS webpage (under construction): Example features and Lessons Learned GRS-IBS bridges in Delaware

3.7 Conclusions and Recommendations from Experiences in Different States

Out of the 144 projects surveyed during this study, the following conclusions and recommendations were identified as the principal trends throughout the country: (1) GRS-IBS provide cost savings up to 68%; (2) the bridges can be completed in half the time of conventional bridges with the same span length; (3) severe weather will not affect the construction process; (4) construction crews with only 4 to 6 non-skilled workers can easily complete the bridges; (5) the foundation can be placed on any type of soil condition; (6) settlement reported is less than 4 cm (1.57 in.) which eliminated the "bump" at the end of the bridge; (7) most of the bridges are built to have a span length and abutment height shorter than 30 m and 18 m respectively; (9) 83% of the bridges were built in low volume roads; (10) proper compaction and placement of the geotextile will avoid facing deformation; (11) it is important to evaluate the abutment behavior with different backfill materials, reinforcement types and facing elements; (12) states like Ohio, Pennsylvania and New York reported that GRS-IBS saved up to 50% compared to conventional bridges. Thus, with the same budget they could build twice the amount of conventional bridges; (13) it is better using solid CMU blocks rather than hollow CMU blocks because the latter will be easily pushed out during compaction; (15) GRS-IBS can be used in all the possible environments with span lengths longer than 43 m (140 ft.); and (16) GRS-IBS was reported to be easy to build and to maintain. **Table 28** shows the updated comparison of recommended GRS-IBS specifications per the FHWA guidelines (Adams et al. 2012), with those reported in completed projects across the United States reported by Hatami et al. (2015), Hatami et al. (2016) and Ngo (2016).

Table 28: Comparison of recommended GRS-IBS specifications per the FHWA guidelines (Adams et al. 2012) with those reported in constructed projects across the United States (Hatami et al. 2015; Hatami et al. 2016; Ngo 2016)

Design Matrix	FHWA Recommendations	Reported GRS-IBS projects in the U.S.
Span length	Max Span < 42.7m (140 ft.)	134 bridges reported with span length. Among those: *44% (59 bridges) shorter than 9 m (30 ft.) *56% (75 bridges) longer than 9 m (30 ft.) *27% (37 bridges) longer than 18 m (60 ft.) *2% (3 bridges) longer than 42 m (140 ft.)
	Single span bridge	106 bridges reported number of spans Among those: *94% (102 bridges) is single span bridges Except: *ME (2 spans) *CO (2 side-by-side bridges each with 3 spans)
Abutment height	<9.14 m (30 ft.)	134 bridges reported with abutment height. Among those: *32% (43 bridges) greater than 4.5m (15 ft.) *1% (2 bridge) greater than 9.0 m (30 ft.)
Facing elements	CMU 203mm × 203 mm × 406 mm (8 in. × 8 in. × 16 in) block with: *Minimum compressive strength of 27,580 KPa (4,000 psi) *Water absorption limit of 5%	*Majority are CMU 203mm × 203 mm × 406 mm (8 in. × 8 in. × 16 in.) *Large wet cast concrete 457 mm × 1168 mm × 711 mm (18 in. × 46 in. × 28 in.) and 406 mm × 1219 mm × 610 mm (16 in. × 48 in. × 24 in.) *Sheet piling panel *Cellular confinement system *15.2 cm × 15.2 cm (6 in. × 6 in.) treated timber
GRS abutment backfill	Well/Open graded or with: *Max aggregate size ranges from1.27cm to 5.08 cm (0.5 in. to 2 in.) *Fines content < 12% *(well-graded) and < 6% *(open-graded), Φ' > 38o	Each one meets this requirement except NC with Φ' = 34o

Design Matrix	FHWA Recommendations	Reported GRS-IBS projects in the U.S.
Geosynthetic	Geogrid or geotextile in abutment but must use geotextile in RSF and approach roadway	69 bridges reported with geosynthetic type. Among those: *82% (57 bridges) geotextile *18% (12 bridges) geogrid.
	Geosynthetic Ultimate Strength ≥ 70KN/m (4,800 lb/ft) for GRS load-bearing application with minimum FSbearing = 3.5	Almost all geotextiles meet this requirement. Except: *Iowa (17.5 KN/m (1200 lbs/ft) Lower FSbearing = 1.8 to 2.6)
Spacing of the reinforcement	*≤ 30.5 cm (12 in.) for primary reinforcement *10.2 cm (4 in.) for secondary in the top 5 layers of the GRS abutment bearing beds for CMU 203mm × 203 mm × 406 mm (8 in. × 8 in. × 16 in)blocks	Each one meets this requirement Typically: *20.32 cm (8 in.) spacing for primary reinforcement *10.16 cm (4 in.) spacing for secondary reinforcement due to CMU 20.32 cm (8 in.) height
Thickness of RSF	61 cm (24 in.) or 0.25B	Each one meets this requirement
AADT	Low volume local road < 400 (rural) or < 700 (urban) - (FHWA 2013)	Most of the DOT’s use GRS-IBS in low volume roads, some use in heave traffic
Performance monitoring	Standard survey level and rod system or EDM survey	Typical surveying. Others are inclinometer, extensometer, strain gauge, earth pressure cell, piezometer, settlement plate, weather station, Tensiometer, ShapeAccelArray, and thermistor
Scour countermeasure	*Riprap aprons *Gabion mattresses *Articulating concrete blocks	Most of the project used: *Riprap *Cellular confinement system *Sheet piling
Service under bridge	*Bridge crossing driveway is more advisable. *When crossing waterway, precaution should be taken regarding: *Stream instability *Scour *Adverse flow conditions	133 bridges reported with the service under bridge Among those: *90% (119 bridges) over waterways only *7% (9 bridges) over driveways only including 2 interstate highways *2% (3 bridges) over railroads only *1% (2 bridges) in Colorado over both railroad and driveway

Design Matrix	FHWA Recommendations	Reported GRS-IBS projects in the U.S.
Construction days		43 bridges reported with construction days Among those: *37% (16 bridges) under 30 days *65% (29 bridges) under 60 days *4% (2 bridges) in Colorado have taken more than 120 days due to its complex 3-span design

Chapter 4. GRS-IBS Bridges in Ottawa County, Lincoln County, and Kay County, OK

This chapter provides detailed information and discussion on eight low-volume road bridges (i.e six GRS-IBS bridges and comparable conventional bridges) that were constructed in Ottawa County, Lincoln County and Kay County, OK. The information presented includes: (1) General information for the bridge built in Ottawa County, (2) Background, construction and cost analysis for the bridge built in Lincoln County, and (3) A brief review and update of the information presented by Ngo (2016) and Hatami et al. (2016) in addition to geotechnical data, hydraulic considerations, and construction phase for the four GRS-IBS bridges in Kay County. The chapter continues with the performance monitoring of two conventional bridges and four GRS-IBS bridges in Kay County which includes : (1) Weather Data, (2) Local Seismicity, (3) Traffic Count, (4) Alternative monitoring systems, (5) Surveying Methodology, and (6) Surveying results for six bridges. The chapter concludes with the lessons learned about the reported bridges to date.

4.1 General information of one GRS-IBS bridge in Ottawa County, Oklahoma

In 2013, the first GRS-IBS bridge was constructed in Oklahoma (**Figure 47**). This bridge was built using county staff under the direction of Mr. Russell Earl. Since this bridge was the first GRS-IBS bridge in Oklahoma, there were some issues related to the abutments leaning profiles in the facing blocks (**Figure 48**). However, this issued appeared to be primarily cosmetic and the bridge to date has been reported with no sign of settlement. Also, Sheffert (2013) reported that the learning curve was very steep and the bridge serve as significant learning experience for the bridges built in 2014 in Kay County, OK. Finally, is worth noting that the bridge was built over a county road and therefore, no scour countermeasure was necessary for the abutments.

Figure 47: First GRS-IBS Bridge in Ottawa County, Oklahoma, built in 2013

(Photographs courtesy of C. Westlund, PE)

Figure 48: Issues observed in the first GRS-IBS bridge in Ottawa County, OK, including leaning wing walls, gaps in facing blocks and exposed geotextile reinforcement. (Photographs courtesy of C. Westlund, PE)

4.2 General information of one GRS-IBS bridge in Lincoln County, Oklahoma.

The Bureau of Internal Affairs (BIA) and the Kickapoo tribe in Oklahoma initiated the bidding process for a GRS-IBS bridge in Lincoln County (Yates Bridge) in May of 2016. **Figure 49** shows the location of the new GRS-IBS bridge. Technical details as the layout of the bridge at the site, and the specifications and sources of the construction materials were discussed in a preconstruction meeting on June 9, 2016 at the bridge site by the parties involved, i.e. a design engineer from the EST, Inc., two representatives from River Ridge Construction, LLC, two Lincoln County District Engineers and one delegate from the BIA office. Completed in July 2016, the Yates Bridge with 5.8 m (17 ft.) -high abutments at the beam seat was more challenging to build because of its height and the fairly steep back-slope. Also, is a significant GRS-IBS project in Oklahoma (**Figure 50**) in comparison to the GRS-IBS bridges in Kay County were only approximately 2.13 m (7 ft.) high at the beam seat.

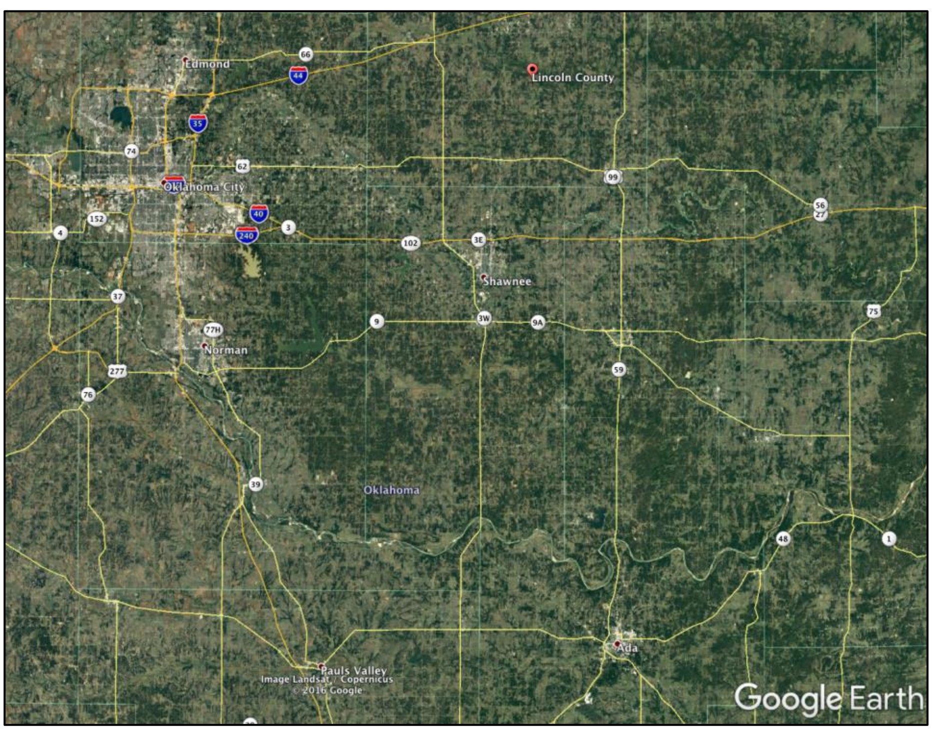


Figure 49: Lincoln County, OK location

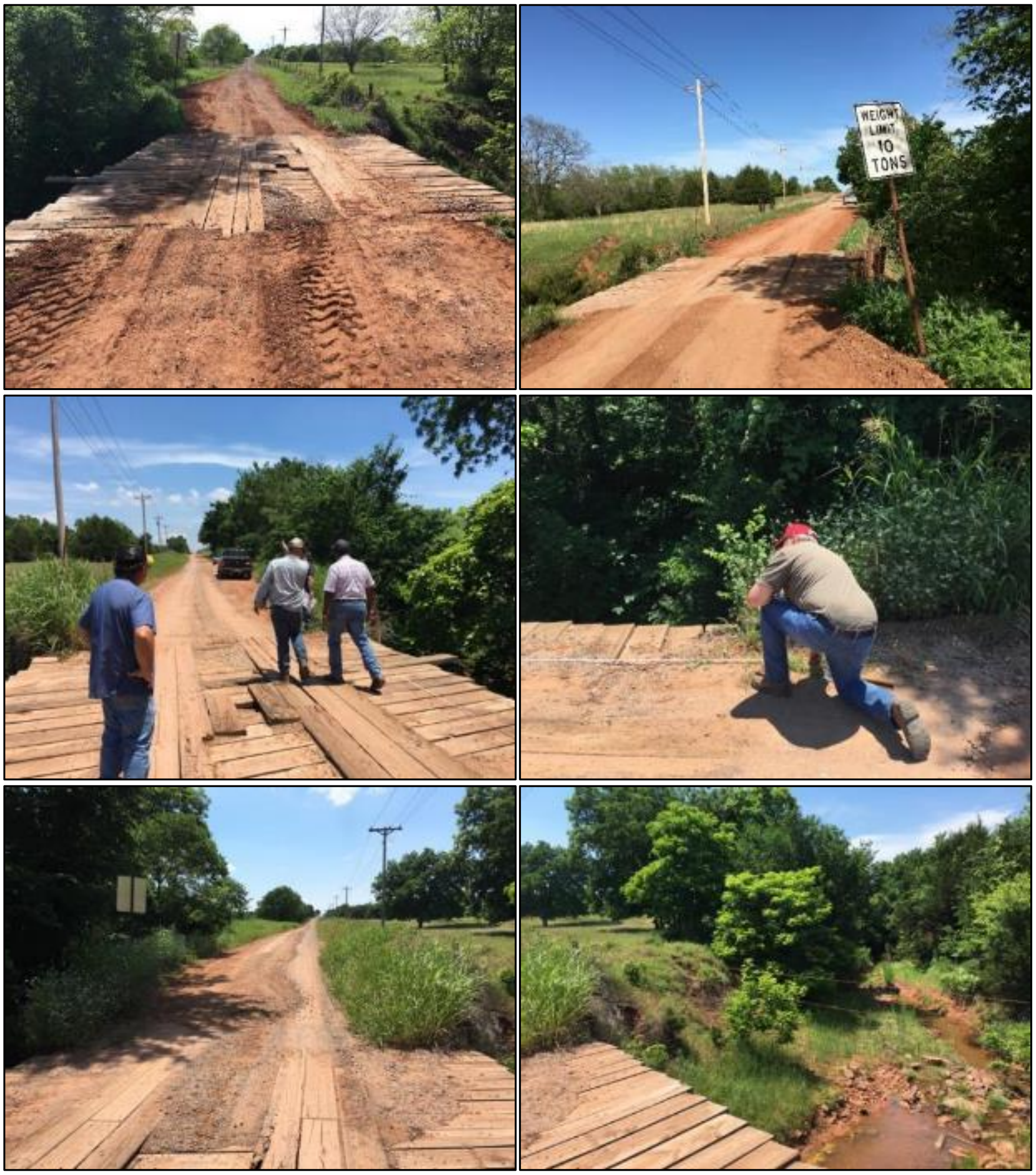


Figure 50: On-site preconstruction meeting on June 9, 2016 at Yates Bridge in Lincoln County

4.3 Construction of one GRS-IBS bridge in Lincoln County, Oklahoma.

In early June 2016, the construction of the bridge started. Pictures of the site after demolition of the old bridge and excavation for the abutments of a new GRS bridge are shown in **Figure 51**. **Figure 52** shows construction of east abutment including its reinforced soil foundation (RSF). The construction challenges included the increase of the stability of the back-slope in a sandy loam soil and so that the crew could access the bottom of the abutment with the track-hoe. Thus, it was decided to use a much milder 2:1 slope instead of the 1:1 slope shown in Adams et al. (2012) standard details. Additionally, the first layer of reinforcement underneath the GRS abutment was overlain with a new layer of geotextile due to its contamination with silt deposits because of flooding of the site for two days (**Figure 53**). **Figure 54** shows construction of the concrete deck and **Figure 55** shows design drawings used in the project.

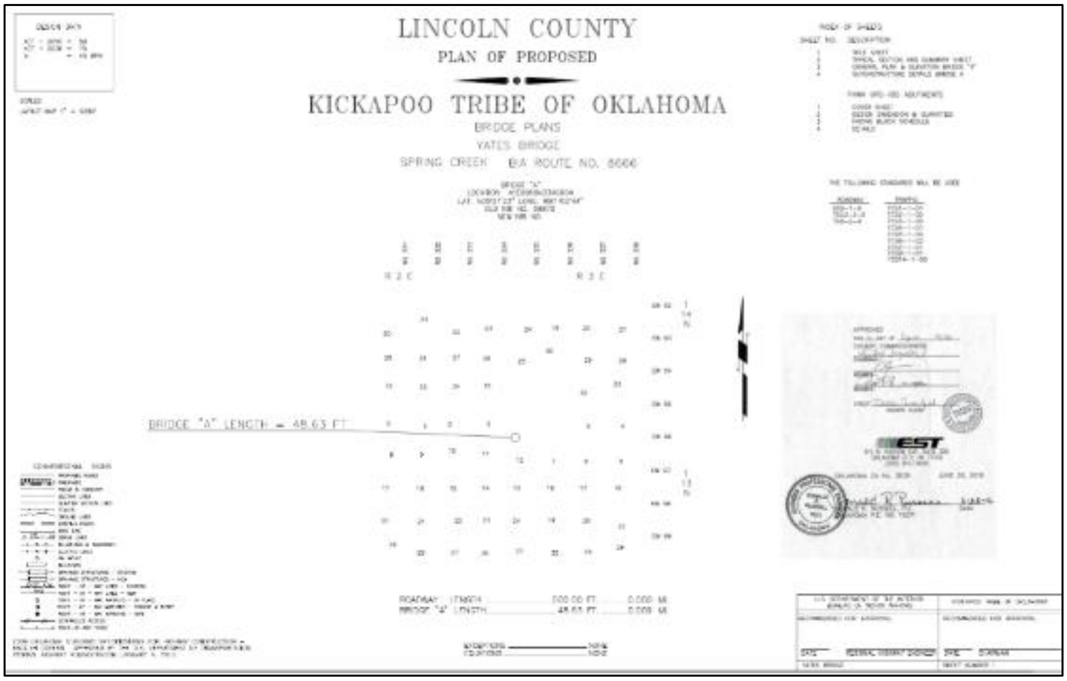
Figure 51: Yates Bridge site after demolition of old bridge and excavation for new GRS abutments

Figure 52: Placement of the first row of CMU blocks (Yates Bridge, 7/11/2016)

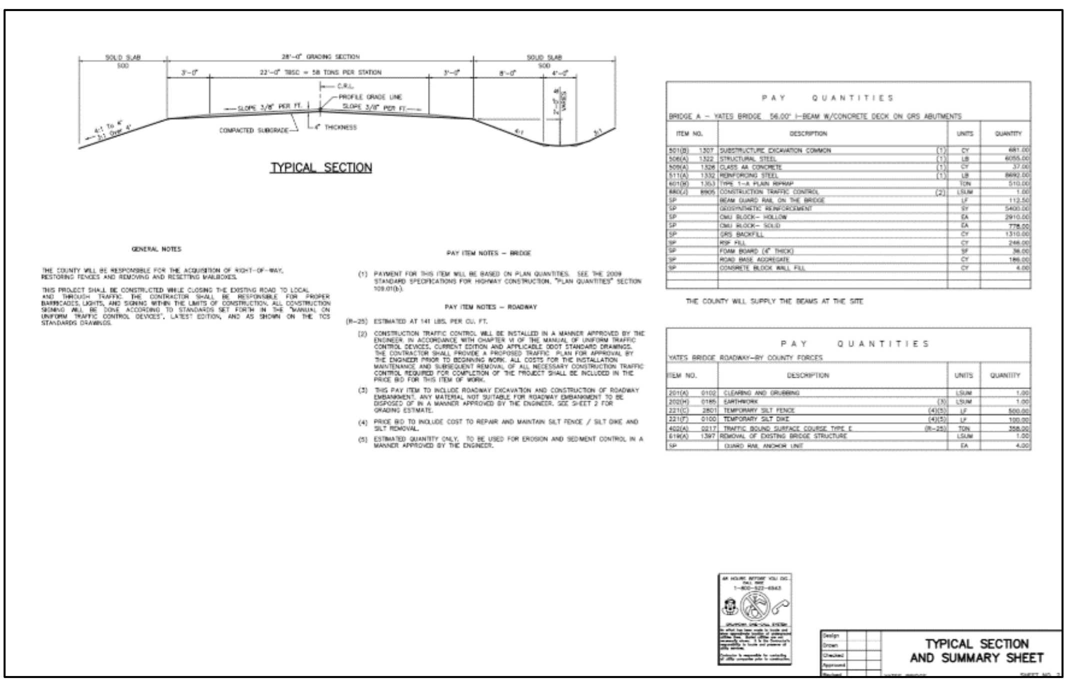
Figure 53: Pumping of flood water out of the GRS abutment site, and placement of a new reinforcement layer on the top of the RSF

Figure 54: Construction of concrete deck at Yates Bridge, Lincoln County, OK

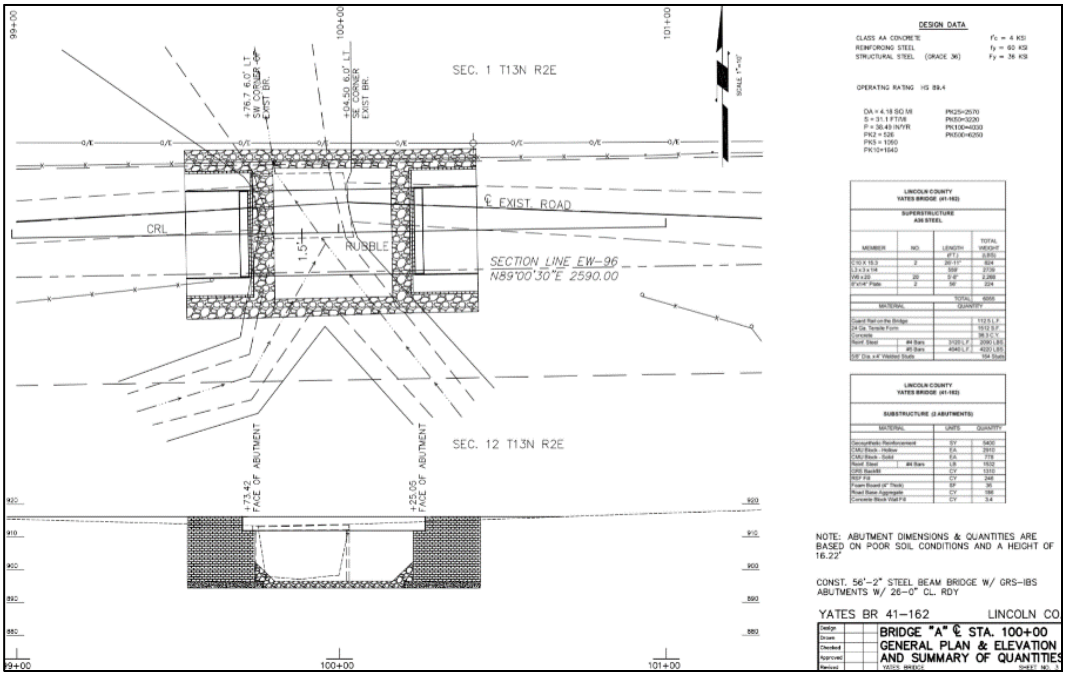
(Photograph courtesy of Mr. Tom Simpson, PE)


LINCOLN COUNTY
PLAN OF PROPOSED
KICKAPOO TRIBE OF OKLAHOMA
BRIDGE PLANS
YATES BRIDGE
SPRING CREEK BIA ROUTE NO. 6666

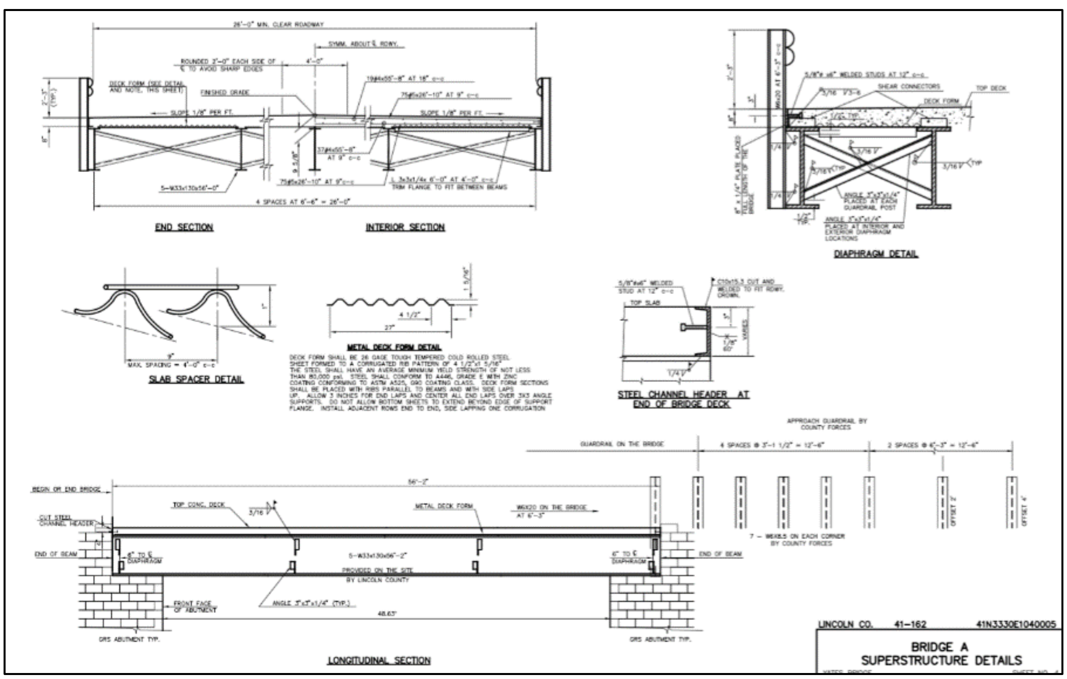
(a)


TYPICAL SECTION
GENERAL NOTES
PAY ITEM NOTES – BRIDGE
PAY ITEM NOTES – ROADWAY
PAY QUANTITIES
THE COUNTY WILL SUPPLY THE BEAMS AT THE SITE
PAY QUANTITIES
TYPICAL SECTION
AND SUMMARY SHEET

(b)


SEC. 1 T13N R2E
EXIST. ROAD
CRL
RUBBLE
SECTION LINE EW-96
N89°00'30"E 2590.00
SEC. 12 T13N R2E
NOTE: ABUTMENT DIMENSIONS & QUANTITIES ARE BASED ON POOR SOIL CONDITIONS AND A HEIGHT OF 16.22'
CONST. 56'-2" STEEL BEAM BRIDGE W/ GRS-IBS ABUTMENTS W/ 26'-0" CL. RDY
YATES BR 41-162
LINCOLN CO.
BRIDGE "A" ℄ STA. 100+00
GENERAL PLAN & ELEVATION
AND SUMMARY OF QUANTITIES

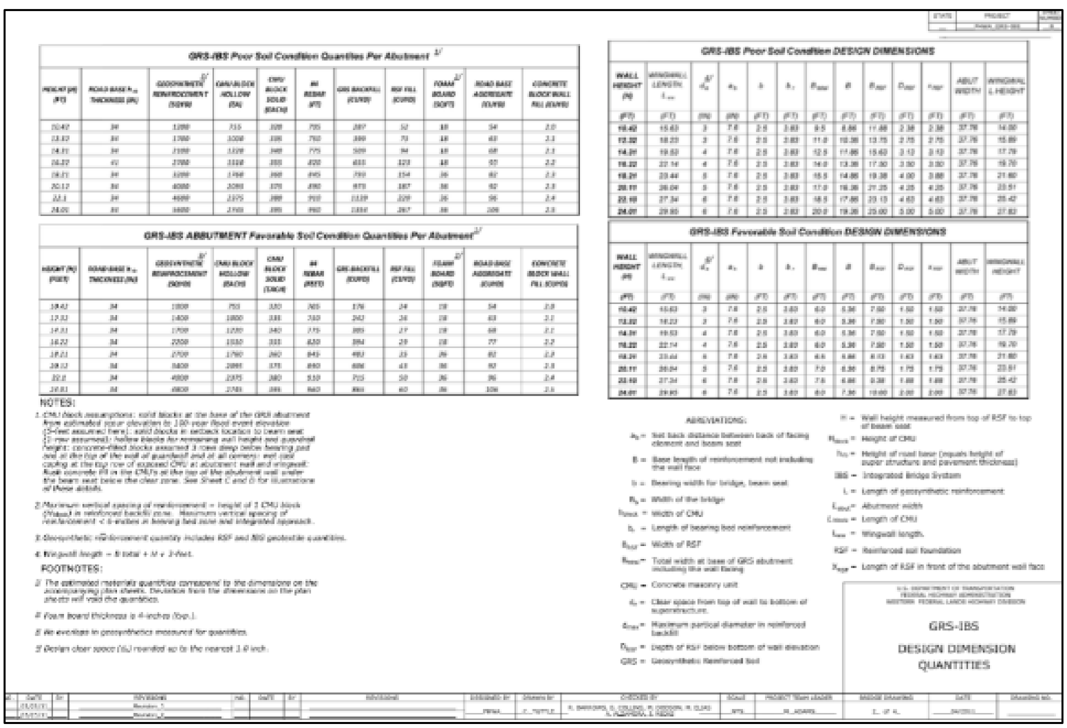
(c)


END SECTION
INTERIOR SECTION
DIAPHRAGM DETAIL
SLAB SPACER DETAIL
METAL DECK FORM DETAIL
STEEL CHANNEL HEADER AT END OF BRIDGE DECK
LONGITUDINAL SECTION
LINCOLN CO.
41-162
BRIDGE A
SUPERSTRUCTURE DETAILS

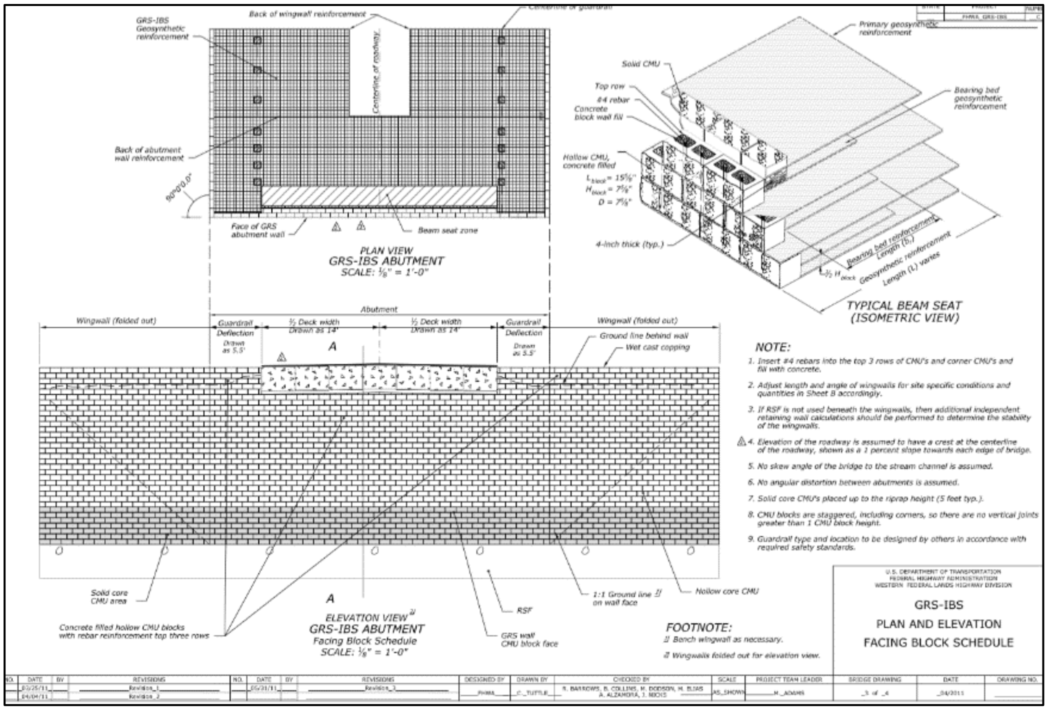
(d)


U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
GRS-IBS
DESIGN DRAWINGS
2011
INDEX TO SHEETS
A. COVER SHEET AND NOTES
B. QUANTITIES & DESIGN DIMENSIONS
C. PLAN AND ELEVATION FACING BLOCK SCHEDULE
D. GRS-IBS ABUTMENT DETAILS
GENERAL NOTES
DESIGN
CONSTRUCTION SPECIFICATIONS
GRS-IBS
COVER SHEET

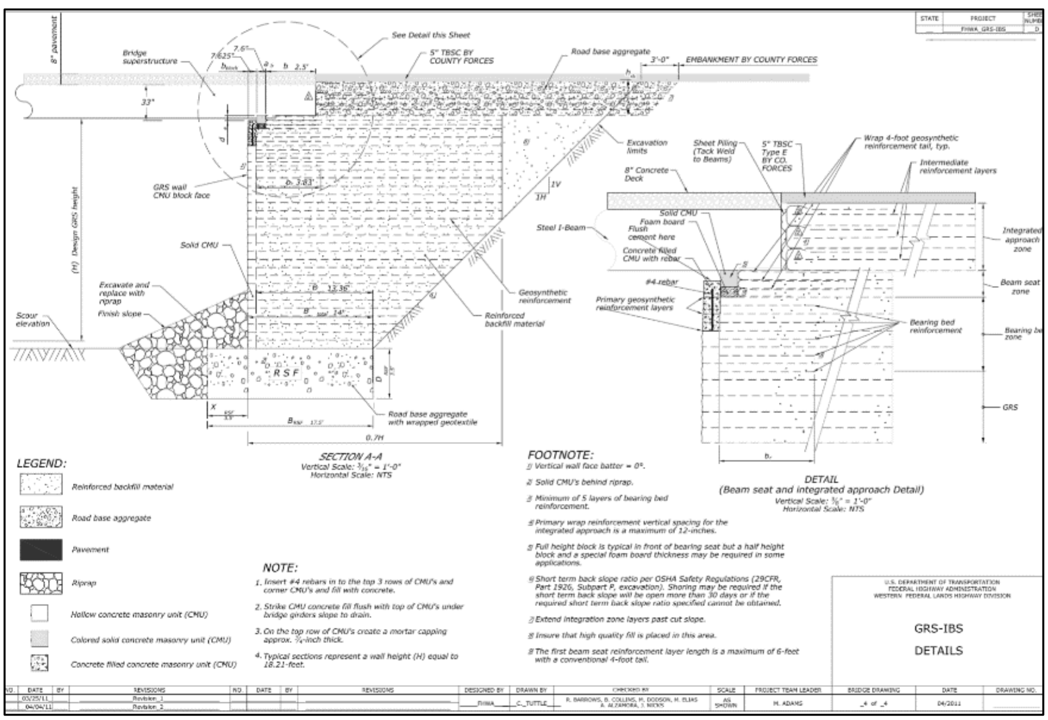
(e)


GRS-IBS Poor Soil Condition Quantities Per Abutment
GRS-IBS Poor Soil Condition DESIGN DIMENSIONS
GRS-IBS ABUTMENT Favorable Soil Condition Quantities Per Abutment
GRS-IBS Favorable Soil Condition DESIGN DIMENSIONS
NOTES:
FOOTNOTES:
ABBREVIATIONS:
GRS-IBS
DESIGN DIMENSION
QUANTITIES

(f)



(g)

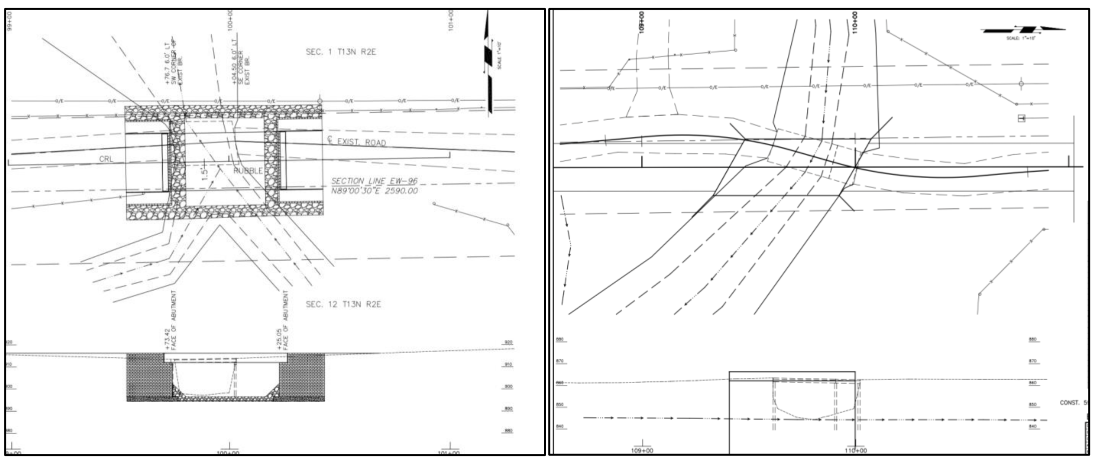


(h)

Figure 55: GRS-IBS Yates Bridge over Spring Creek in Lincoln County, OK; (a) Title sheet; (b) Typical sections; (c) General plan & elevation; (d) Superstructure details; (e) Cover sheet; (f) Design dimensions and quantities; (g) Plan and elevation, facing block schedule; (h) GRS-IBS details

4.4 Cost analysis of one GRS-IBS bridge in Lincoln County, Oklahoma.

In this section cost details are provided for the Yates Bridge over Spring Creek in Lincoln County (GRS-IBS) in comparison with the projected costs of a comparable bridge (i.e Guilliam Bridge) with conventional abutments based in the information obtained from Mr. Tom Simpson of the BIA. The GRS-IBS abutment was the only construction alternative considered for this project. However, the bidding results of October 19,2015 for a pile-supported abutment bridge with comparable bridge dimensions were also obtained. **Figure 56** shows selected information on the two bridges including some design specifications and bill of quantities.



(a)

DESIGN DATA

CLASS AA CONCRETE — f'c = 4 KSI
REINFORCING STEEL — fy = 60 KSI
STRUCTURAL STEEL (GRADE 36) — Fy = 36 KSI

OPERATING RATING HS 89.4

DA = 4.18 SQ MI
S = 31.1 FT/MI
P = 38.49 IN/YR
PK2 = 526
PK5 = 1090
PK10=1640
PK25=2570
PK50=3220
PK100=4030
PK500=6250

LINCOLN COUNTY
YATES BRIDGE (41-162)

SUPERSTRUCTURE
A36 STEEL

MEMBER	NO.	LENGTH (FT.)	TOTAL WEIGHT (LBS)
C10 X 15.3	2	26'-11"	824
L3 x 3 x 1/4		559'	2739
W6 x 20	20	5'-8"	2,268
8"x1/4" Plate	2	56'	224
		TOTAL	6055

MATERIAL		QUANTITY	
Guard Rail on the Bridge			112.5 L.F.
24 Ga. Tensile Form			1512 S.F.
Concrete			36.3 C.Y.
Reinf. Steel	#4 Bars	3120 L.F.	2090 LBS.
	#5 Bars	4040 L.F.	4220 LBS.
5/8" Dia. x 4" Welded Studs			164 Studs

LINCOLN COUNTY
YATES BRIDGE (41-162)

SUBSTRUCTURE (2 ABUTMENTS)

MATERIAL		UNITS	QUANTITY
Geosynthetic Reinforcement		SY	5400
CMU Block - Hollow		EA	2910
CMU Block - Solid		EA	778
Reinf. Steel	#4 Bars	LB	1532
GRS Backfill		CY	1310
RSF Fill		CY	246
Foam Board (4" Thick)		SF	36
Road Base Aggregate		CY	186
Concrete Block Wall Fill		CY	3.4

NOTE: ABUTMENT DIMENSIONS & QUANTITIES ARE BASED ON POOR SOIL CONDITIONS AND A HEIGHT OF 16.22'

CONST. 56'–2" STEEL BEAM BRIDGE W/ GRS–IBS ABUTMENTS W/ 26–0" CL. RDY

YATES BR 41–162 LINCOLN CO

DESIGN DATA

CLASS AA CONCRETE — f'c = 4 KSI
REINFORCING STEEL — fy = 60 KSI
STRUCTURAL STEEL (GRADE 36) — Fy = 36 KSI

MAXIMUM FACTORED PILE LOAD –
ABUTMENTS: 19.3 TONS / PILE

DA = 2.33 SQ MI
S = 48.2 FT/MI
P = 38.75 IN/YR
PK2 = 389
PK5 = 813
PK10=1240
PK25=1950
PK50=2420
PK100=3040
PK500=4700

LINCOLN COUNTY
BRIDGE A (41–280)
59.00' Long

A36 STEEL

MEMBER	NO.	LENGTH (FT.)	TOTAL WEIGHT (LBS)
HP12x53	2	33'–0"	3,498
C10x15.3	2	30'–0"	918
C8x11.5		248'	2,852
L4x3x1/4		100'	580
L3x3x1/4		291'	1,426
W6x20	20	5'–8"	2268
1/2" Plate	36	5 3/4" X 10 7/8"	352
8"x1/4" Plate	2	59'	236
		TOTAL	12,130

MATERIAL		QUANTITY	
Guard Rail			225 L.F.
Driven Guard Rail Posts (W6x8.5)			28 Ea.
Type 1 End Section (Elephant Ear)			4 Ea.
24 Ga. Tensile Form			1512 S.F.
Piles HP 10x42			240 L.F.
Piles HP 12x53			480 L.F.
12 Ga. Sheet Piling			2,400 S.F.
Concrete			38.2 CY
Reinf. Steel	#4 Bars	3480 L.F.	2325 LBS.
	#5 Bars	4240 L.F.	4420 LBS.
5/8" Dia. x 4" Welded Studs			170 Studs

59'–0" STEEL BEAM BRIDGE W/ 26–0" CL. RDY

BR 41–280 LINCOLN CO

(b)

Figure 56: Side-by-side comparison between GRS-IBS Yates Bridge over Spring Creek (Left) and pile-supported Guilliam Bridge over Kickapoo Creek (Right) in Lincoln County, OK: (a) Plan and elevation views; (b) Design data and bill of quantities

Bid tables for both bridges are shown in **Figure 57**, which include quote comparisons between different contractors and engineer's estimate. Also, this data indicates that GRS abutment and the conventional abutment were comparable with respect to factors such as their width, height, span length, ADT and superstructure. Contractors' quotes on the two projects vary over a rather wide range (nearly by a factor of two). However, after ignoring the highest unit prices, it can be observed that the unit cost of the superstructure ($/ft^2) for the conventional bridge is higher than that of the GRS-IBS bridge (i.e. $39.03/ft^2 vs. $31.27/ft^2). However, this difference seems to be counterbalanced by a slightly higher cost of the GRS abutment relative to the pile support. Even though these prices are only estimates, the data in **Figure 57** indicate that the cost-effectiveness of GRS-IBS relative to conventional bridges could be significantly compromised for taller abutments. In addition, since the back-slope of the GRS abutments had to be changed from 1:1 to a milder 2:1 for stability, there were additional quantities of excavation and aggregate that were used to build the abutments. Per the latest estimate, the actual cost of the bridge was approximately $170,000 (Simpson, T. 2016).

Bid Tabulation

Lincoln County

Bid No. 16-01
Yates Bridge w/ GRS Abutments
Bids Received May 2, 2016

EST, Inc.
615 N. Hudson, 3rd Floor
Oklahoma City, OK 73012
(405)815-3600

BR A - YATES BRIDGE 56' I-BEAM W/ CONCRETE DECK ON GRS ABUTMENTS

ITEM NO.		DESCRIPTION	UNITS	QUANTITY	Engineer's Estimate		Contractor A		Contractor B	
501(B)	1307	SUBSTRUCURE EXCAVATION COMMON	CY	681.00	20.00	$ 13,620.00	12.00	$ 8,172.00	8.00	$ 5,448.00
506(A)	1322	STRUCTURAL STEEL	LB	6055.00	2.50	$ 15,137.50	5.22	$ 31,607.10	6.00	$ 36,330.00
509(A)	1326	CLASS AA CONC.	CY	37.00	350.00	$ 12,950.00	229.00	$ 8,473.00	400.00	$ 14,800.00
511(A)	1332	REINFORCING STEEL	LB	8692.00	1.00	$ 8,692.00	0.60	$ 5,215.20	1.20	$ 10,430.40
601(B)	1353	TYPE 1-A PLAIN RIPRAP	TON	510.00	35.00	$ 17,850.00	44.25	$ 22,567.50	55.00	$ 28,050.00
880(J)	8905	CONSTRUCTION TRAFFIC CONTROL	LSUM	1.00	2000.00	$ 2,000.00	2500.00	$ 2,500.00	2000.00	$ 2,000.00
SP	0	BEAM GUARD RAIL ON THE BRIDGE	LF	125.00	11.00	$ 1,375.00	30.00	$ 3,750.00	50.00	$ 6,250.00
SP	0	GEOSYNTHETIC REINFORCEMENT	SY	5400.00	3.25	$ 17,550.00	2.75	$ 14,850.00	6.00	$ 32,400.00
SP	0	CMU BLOCK - HOLLOW	EA	2910.00	2.80	$ 8,148.00	4.00	$ 11,640.00	19.00	$ 55,290.00
SP	0	CMU BLOCK - SOLID	EA	778.00	3.00	$ 2,334.00	2.40	$ 1,867.20	22.00	$ 17,116.00
SP	0	GRS BACKFILL	CY	1310.00	30.00	$ 39,300.00	24.45	$ 32,029.50	95.00	$ 124,450.00
SP	0	RSF FILL	CY	246.00	30.00	$ 7,380.00	30.00	$ 7,380.00	100.00	$ 24,600.00
SP	0	FOAM BOARD (4" THICK)	SF	36.00	2.00	$ 72.00	10.00	$ 360.00	50.00	$ 1,800.00
SP	0	ROAD BASE AGGREGATE	CY	186.00	30.00	$ 5,580.00	28.00	$ 5,208.00	100.00	$ 18,600.00
SP	0	CONCRETE BLOCK WALL FILL	CY	4.00	220.00	$ 880.00	150.00	$ 600.00	900.00	$ 3,600.00
SP	0	12 GA. SHEET PILING	SF	1998.00	10.00	$ 19,980.00	0.45	$ 899.10	6.00	$ 11,988.00
				TOTAL		$ 172,848.50		$ 157,118.60		$ 393,152.40

ANALYSIS

Abutment type	GRS-IBS
Bridge Name	Yates
Height	16.22 ft
Span Legth	56.3 ft
Width	26 ft
Superstructure type	Steel Beam
ADT-2015	50
ADT-2035	75
V	45

Conclusions		
Area of Superstructure	1,463.80	ft^2
Cost of superstructure per ft^2	31.27	$/ft^2
Cost of abutment per LF of height	7,376.68	$/LF
Cost of abutment per LF of height	-	$/LF

					Superstructure		Abutment	
Item No		Description	Price	Units	Qty	Total	Abutment	Total
501(B)	1307	Substructure Excavation	$ 16.00	CY		$ -	681	$ 10,896.00
506(A)	1322	Structural Steel	$ 3.86	LB	6,055.0	$ 23,372		$ -
509(A)	1326	Class AA Concrete	$ 289.50	CY	37.0	$ 10,712		$ -
511(A)	1332	Reinforcing Steel	$ 0.80	LB	8,692.0	$ 6,954		$ -
601(B)	1353	Type 1-A Plain Riprap	$ 39.63	TON		$ -	510	$ 20,208.75
880(J)	8905	Construction Traffic Control	$ 2,166.67	LSUM	1.0	$ 2,167		$ -
SP	0	Beam Guard Rail	$ 20.50	LF	125.0	$ 2,563		$ -
SP	0	Geosynthetic Reinforcement	$ 3.00	SY		$ -	5,400	$ 16,200.00
SP	0	CMU Block- Hollow	$ 3.40	EA		$ -	2,910.0	$ 9,894
SP	0	CMU Block - Solid	$ 2.70	EA		$ -	778.0	$ 2,101
SP	0	GRS Backfill	$ 27.23	CY		$ -	1,310.0	$ 35,665
SP	0	RSF Fill	$ 30.00	CY		$ -	246.0	$ 7,380
SP	0	Foam Board (4"Thick)	$ 6.00	SF		$ -	36.0	$ 216
SP	0	Road Base Aggregate	$ 29.00	CY		$ -	186.0	$ 5,394.00
SP	0	Concrete Block Wall Fill	$ 185.00	CY		$ -	4.0	$ 740.00
SP	0	12 GA Sheet piling	$ 5.48	SF		$ -	1,998.0	$ 10,955.70
					Superstructure	$ 45,766.57	Abutment	$ 119,649.80

Information From Mr Simpson's e-mails **Image 1**				
Total Qty	Engineer	Contractor A	Contractor B	Contractor C
681.0	$ 20.0	$ 12.0	*8	
6,055.0	$ 2.5	$ 5.2	*6	
37.0	$ 350.0	$ 229.0	*400	
8,692.0	$ 1.0	$ 0.6	*1.2	
510.0	$ 35.0	$ 44.3	*55	
1.0	$ 2,000.0	$ 2,500.0	$ 2,000.0	
125.0	$ 11.0	$ 30.0	*50	
5,400.0	$ 3.3	$ 2.8	*6	
2,910.0	$ 2.8	$ 4.0	*19	
778.0	$ 3.0	$ 2.4	*22	
1,310.0	$ 30.0	$ 24.5	*95	
246.0	$ 30.0	$ 30.0	*100	
36.0	$ 2.0	$ 10.0	*50	
186.0	$ 30.0	$ 28.0	*100	
4.0	$ 220.0	$ 150.0	*900	
1,998.0	$ 10.0	$ 0.5	$ 6.0	
		*Out of the analysis, prices to high		
Total price	$ 172,849	$ 157,119	*	

(a)

Bid Tabulation

Lincoln County

Bid No. 15-13
Guilliam Bridge
Bids Received October 19, 2015

EST, Inc.
615 N. Hudson, 3rd Floor
Oklahoma City, OK 73012
(405)815-3600

ITEM NO.		DESCRIPTION	UNITS	QUANTITY	Engineer's Estimate		Contractor A		Contractor B		Contractor C	
506(A)	1322	STRUCTURAL STEEL	LB	15008.00	1.20 $	18,009.60	2.50 $	37,520.00	5.50 $	82,544.00	9.50 $	142,576.00
509(A)	1326	CLASS AA CONC.	CY	38.20	400.00 $	15,280.00	320.00 $	12,224.00	825.00 $	31,515.00	900.00 $	34,380.00
511(A)	1332	REINFORCING STEEL	LB	6630.00	1.20 $	7,956.00	0.86 $	5,701.80	1.50 $	9,945.00	2.50 $	16,575.00
514(A)	6010	PILES, FURNISHED (HP 10x42)	LF	240.00	21.00 $	5,040.00	34.00 $	8,160.00	35.00 $	8,400.00	85.00 $	20,400.00
514(A)	6011	PILES, FURNISHED (HP 12x53)	LF	480.00	26.00 $	12,480.00	40.00 $	19,200.00	42.00 $	20,160.00	95.00 $	45,600.00
514(B)	6292	PILES, DRIVEN (HP 10x42)	LF	240.00	8.00 $	1,920.00	10.00 $	2,400.00	15.00 $	3,600.00	50.00 $	12,000.00
514(B)	6294	PILES, DRIVEN (HP 12x53)	LF	480.00	10.00 $	4,800.00	10.00 $	4,800.00	18.00 $	8,640.00	50.00 $	24,000.00
SP	0	24 GA. TENSINLE FORM	SF	1755.00	5.00 $	8,775.00	4.72 $	8,283.60	9.20 $	16,146.00	13.00 $	22,815.00
SP	0	12 GA. SHEET PILING	SF	2190.00	10.00 $	21,900.00	9.23 $	20,213.70	20.00 $	43,800.00	35.00 $	76,650.00
SP	0	BEAM GUARD RAIL ON THE BRIDGE	LF	125.00	25.00 $	3,125.00 *	11.00 $	1,375.00	35.00 $	4,375.00	75.00 $	9,375.00
880(J)	8905	CONSTRUCTION TRAFFIC CONTROL	LSUM	1.00	2000.00 $	2,000.00	2000.00 $	2,000.00	4000.00 $	4,000.00	11227.00 $	11,227.00
			TOTAL SURFACING		$	101,285.60 *	$	121,878.10	$	233,125.00	$	415,598.00

* Corrected

Abutment type	Pile Abutment
Bridge Name	Guilliam
Height	17 ft
Span Legth	59 ft
Width	26 ft
Superstructure type	Steel Beam
ADT-2015	100
ADT-2035	150
V	45 MPH

Conclusions		
Area of Superstructure	1,534.00	ft^2
Cost of superstructure per ft^2	39.03	$/ft^2
Cost of abutment per LF of height	3,775.65	$/LF
Cost of abutment per LF of height	2,848.59	$/LF

ANALYSIS

Item No		Description	Price	Units	Superstructure Qty	Superstructure Total	Abutment	Abutment Total
506(A)	1322	Structural Steel	$ 3.07	LB	8,000.0	$ 24,533	7,008	$ 21,491.20
509(A)	1326	Class AA Concrete	$ 515.00	CY	38.2	$ 19,673	-	$ -
511(A)	1332	Reinforcing Steel	$ 1.52	LB	6,630.0	$ 10,044	-	$ -
514(A)	6010	Piles, Furnished (HP 10 x 42)	$ 30.00	LF		$ -	240.0	$ 7,200
514(A)	6011	Piles, Furnished (HP 12 x 53)	$ 36.00	LF		$ -	480.0	$ 17,280
514(B)	6292	Piles, Driven (HP 10 x 42)	$ 11.00	LF		$ -	240.0	$ 2,640
514(B)	6294	Piles, driven (HP 12 x 53)	$ 12.67	LF		$ -	480.0	$ 6,080
SP	0	24 GA Tensile form	$ 6.31	SF		$ -	1,755.0	$ 11,068
SP	0	12 GA Sheet Piling	$ 13.08	SF		$ -	2,190.0	$ 28,638
SP	0	Beam Guard Rail	$ 23.67	LF	125.0	$ 2,958		$ -
880(J)	8905	Construction Traffic control	$ 2,666.67	LSUM	1.0	$ 2,667		$ -

Superstructure	$ 59,875.78	Abutment cost with furnished Piles	$ 64,186
		Abutment cost with driven Piles	$ 48,426

Information from bidding results				
Total Qty	Engineer	Contractor A	Contractor B	Contractor C
15,008.0	$ 1.2	$ 2.5	$ 5.5	*9.5
38.2	$ 400.0	$ 320.0	$ 825.0	*900
6,630.0	$ 1.2	$ 0.9	$ 1.5	$ 2.5
240.0	$ 21.0	$ 34.0	$ 35.0	*85
480.0	$ 26.0	$ 40.0	$ 42.0	*95
240.0	$ 8.0	$ 10.0	$ 15.0	*50
480.0	$ 10.0	$ 10.0	$ 18.0	*50
1,755.0	$ 5.0	$ 4.7	$ 9.2	*13
2,190.0	$ 10.0	$ 9.2	$ 20.0	*35
125.0	$ 25.0	$ 11.0	$ 35.0	*75
1.0	$ 2,000.0	$ 2,000.0	$ 4,000.0	*11227
		*Out of the analysis, prices to high		
Total price	$ 101,286	$ 121,878	$ 233,125	*

(b)

Figure 57: Bid tables for comparable bridge projects in Lincoln County, OK: (a) GRS-IBS Yates Bridge over Spring Creek; (b) Pile-supported Guilliam Bridge over Kickapoo Creek

4.5 General Information of the studied bridges in Kay County, Oklahoma

During the period between April 2014 and February 2015, four (4) GRS-IBS bridges and two (2) conventional bridges were constructed to replace six (6) bridges over Dry Creek near Blackwell, in Kay County, OK (**Figure 58**). All of them were built within a one-mile segment of the 44th St., which provided a unique opportunity for a side-by-side comparison as reported by Ngo (2016). Their performance was compared with one another for essentially the same geotechnical, climatic, traffic and construction conditions. In addition to the aforementioned conditions, this section reports the weather data, local seismicity and traffic count. The bridges are numbered as shown in **Table 29** and **Figure 59** for ease of reference in this study.

Figure 58: General location of Kaw Nation bridge project (44th Street) in Kay County, Oklahoma

Table 29: Structural types of decks and abutments used in Kay County bridges (Hatami et al. 2016)

Bridge	Abutment Types	Superstructure (Planned)	Superstructure (As built)
1	Driven H-piles	Steel girder	Steel girders and tied rebar cast-in-place concrete deck
2	GRS	Steel girder	
3		Box beams	
4		Precast slab span	
5		Girder/ slabs	Steel girder and precast concrete deck slab
6	Driven H-piles	Girder/ slabs	

Figure 59: Locations of GRS-IBS bridges in Kay County

Figure 60 through **Figure 63** show side-by-side comparisons between the old and new (GRS-IBS) bridges (Bridges Nos. 2 through 5) in Kay County.

(a)

(b)

Figure 60: (a) Old and (b) New Bridge No. 2 (Photographs Courtesy of Mr. Tom Simpson, PE)

(a)

(b)

Figure 61: (a) Old and (b) New Bridge No. 3 (Photographs Courtesy of Mr. Tom Simpson, PE)

(a)

(b)

Figure 62: (a) Old and (b) New Bridge No. 4 (Photographs Courtesy of Mr. Tom Simpson, PE)

(a)

(b)

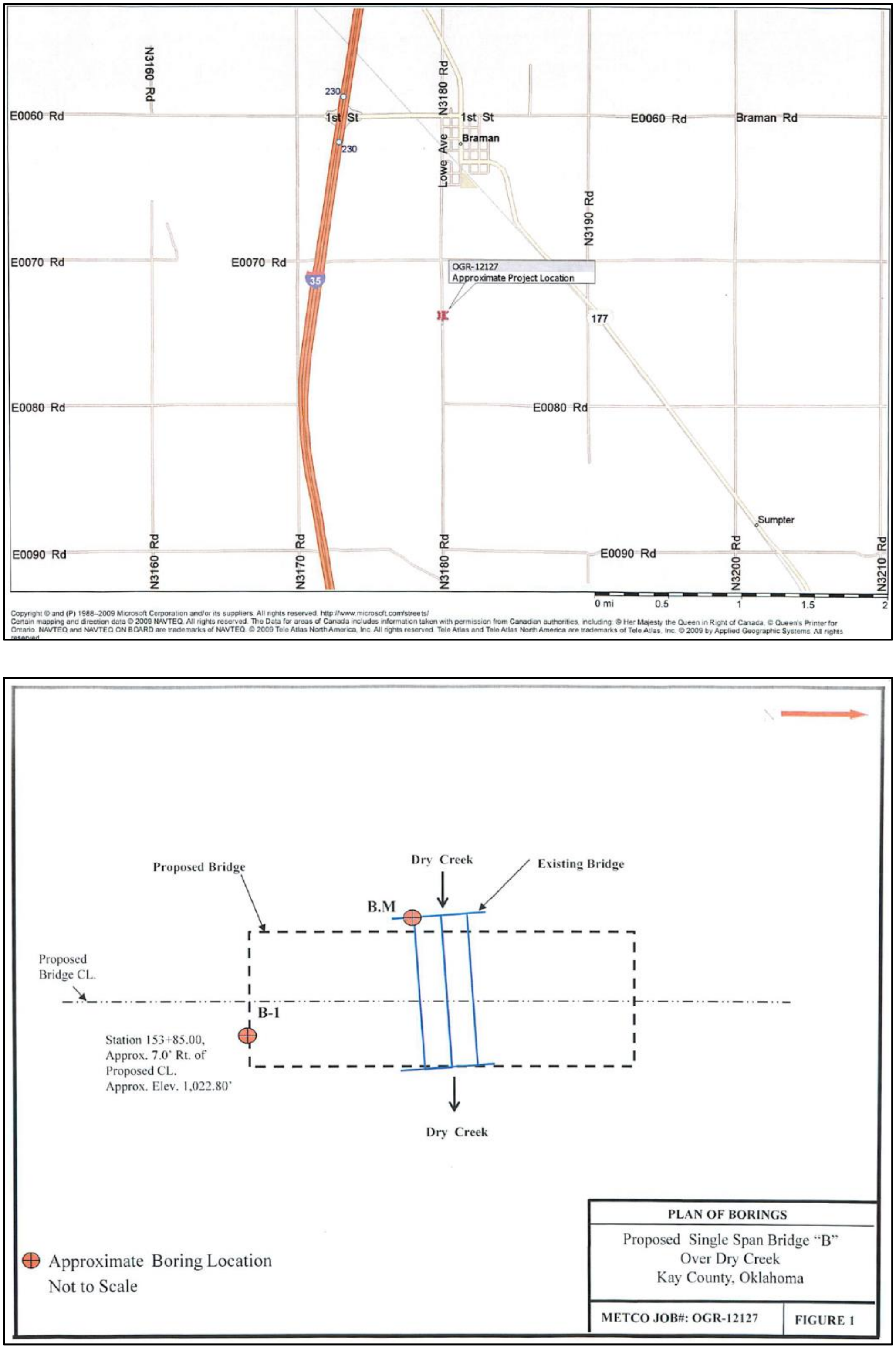
Figure 63: (a) Old and (b) New Bridge No. 5 (Photographs Courtesy of Mr. Tom Simpson, PE)

This technology provides economical and reliable solutions to replace many structurally deficient and functionally obsolete bridges on county roads across the states.

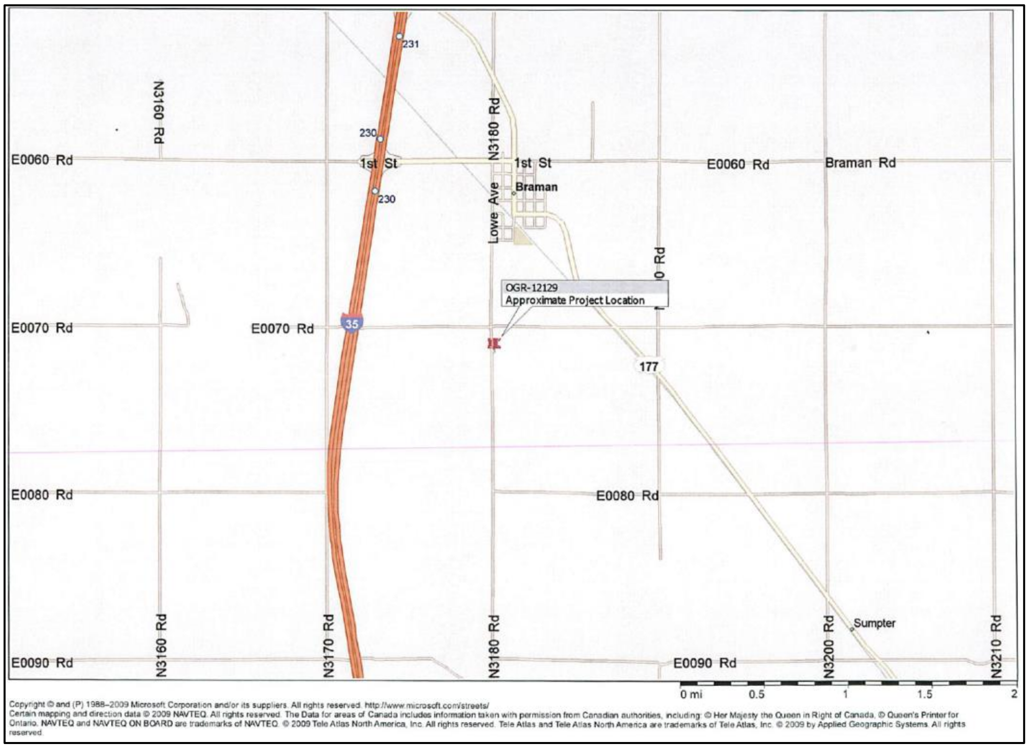
4.6 Geotechnical Data of Kay County, Oklahoma

Geotechnical information of the geotechnical report No. OGR-12126/27/28/29 by METCO (METCO 2012) is given in this section. One soil boring was drilled at the location of each replacement bridge using a truck-mounted hollow-stem drill rig. METCO reported 4 borings named B, C, D and E that corresponded to bridge 2,3,4, and 5 respectively (**Figure 59**). The borings, that contain essentially similar data and analysis, were drilled to an approximate depth of 18.3 m (60 ft.)

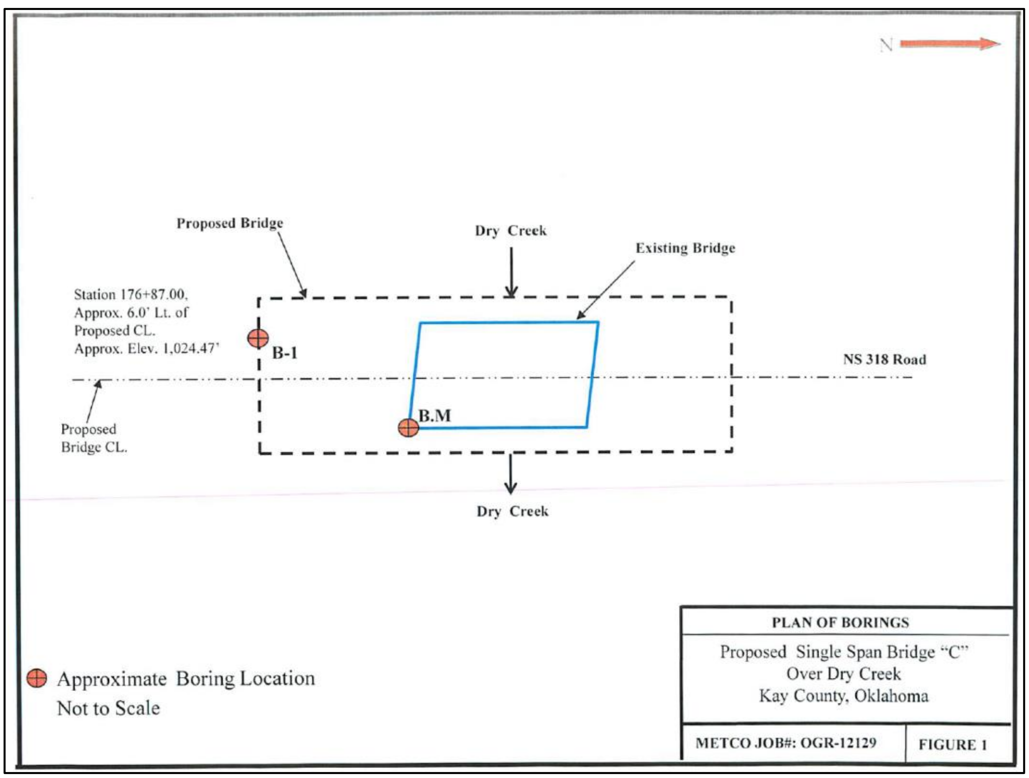
Apart from four (4) in. of gravel and topsoil, the boring generally encountered lean clays to an approximate depth of 11.9 ~ 13.4 m (39~44 ft.) below existing grade, underlain by soft to moderately hard sandy weathered shale of approximately 18.3 m 60.0 ft. Standard penetration resistance (N-Value; ASTM D1586) recorded in the soils ranged between weight-of-hammer (soft consistency) and 85 blows per foot of penetration (stiff soil). Texas cone penetration test results (in general conformance with ASTM D3431) in the sandy weathered shale bedrocks ranged from 100 blows/5 inches of penetration to 100 blows/2 inches of penetration indicating soft to moderately hard rock. Groundwater was encountered at approximately 3.7m (12 ft.) to 4.0 m (13 ft.) below existing grade. **Figure 64** shows the boring locations. In addition, **Figure 65** shows the fence diagram for all the bridges.


E0060 Rd
E0070 Rd
E0080 Rd
E0090 Rd
N3160 Rd
N3170 Rd
N3180 Rd
N3190 Rd
N3200 Rd
N3210 Rd
1st St
Braman
Lowe Ave
Braman Rd
230
35
177
Sumpter
OGR-12127
Approximate Project Location
0 mi
0.5
1
1.5
2
Proposed Bridge
Dry Creek
Existing Bridge
B.M
Proposed
Bridge CL.
B-1
Station 153+85.00,
Approx. 7.0' Rt. of
Proposed CL.
Approx. Elev. 1,022.80'
Dry Creek
PLAN OF BORINGS
Proposed Single Span Bridge "B"
Over Dry Creek
Kay County, Oklahoma
METCO JOB#: OGR-12127
FIGURE 1
Approximate Boring Location
Not to Scale

(a)

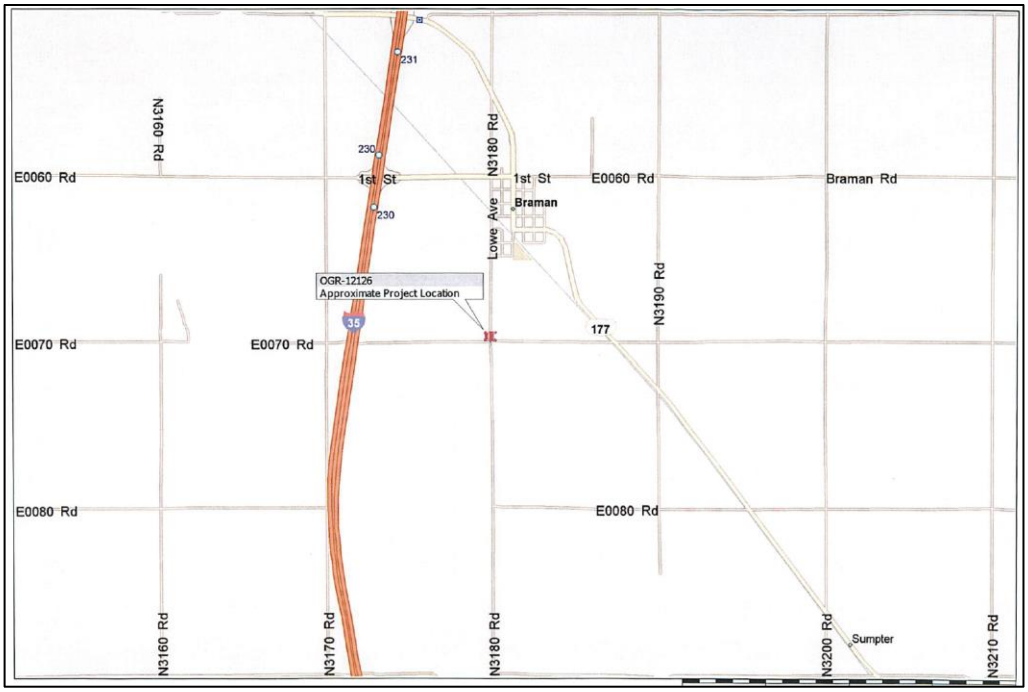


231
230
230
N3160 Rd
N3180 Rd
E0060 Rd
1st St
1st St
Braman
E0060 Rd
Braman Rd
Lowe Ave
OGR-12129
Approximate Project Location
E0070 Rd
E0070 Rd
35
177
E0080 Rd
E0080 Rd
Sumpter
N3160 Rd
N3170 Rd
N3180 Rd
N3200 Rd
N3210 Rd
E0090 Rd
E0090 Rd
0 mi 0.5 1 1.5 2

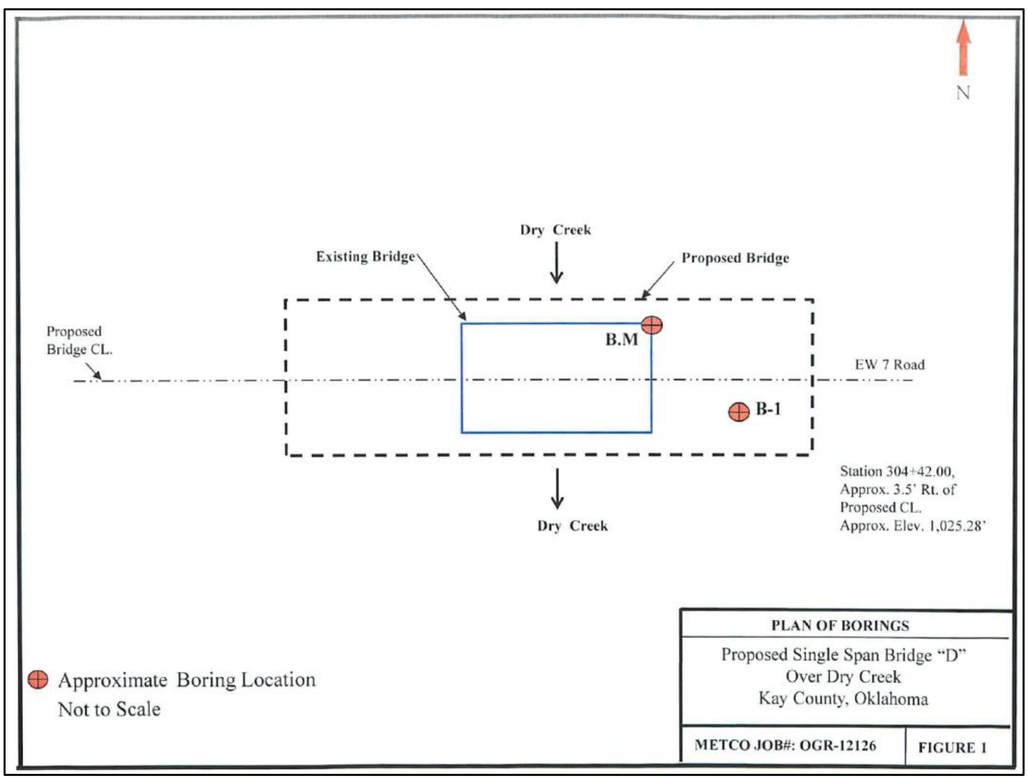


N
Proposed Bridge
Dry Creek
Existing Bridge
Station 176+87.00,
Approx. 6.0' Lt. of
Proposed CL.
Approx. Elev. 1,024.47'
B-1
NS 318 Road
B.M
Proposed
Bridge CL.
Dry Creek
Approximate Boring Location
Not to Scale
PLAN OF BORINGS
Proposed Single Span Bridge "C"
Over Dry Creek
Kay County, Oklahoma
METCO JOB#: OGR-12129
FIGURE 1

(b)

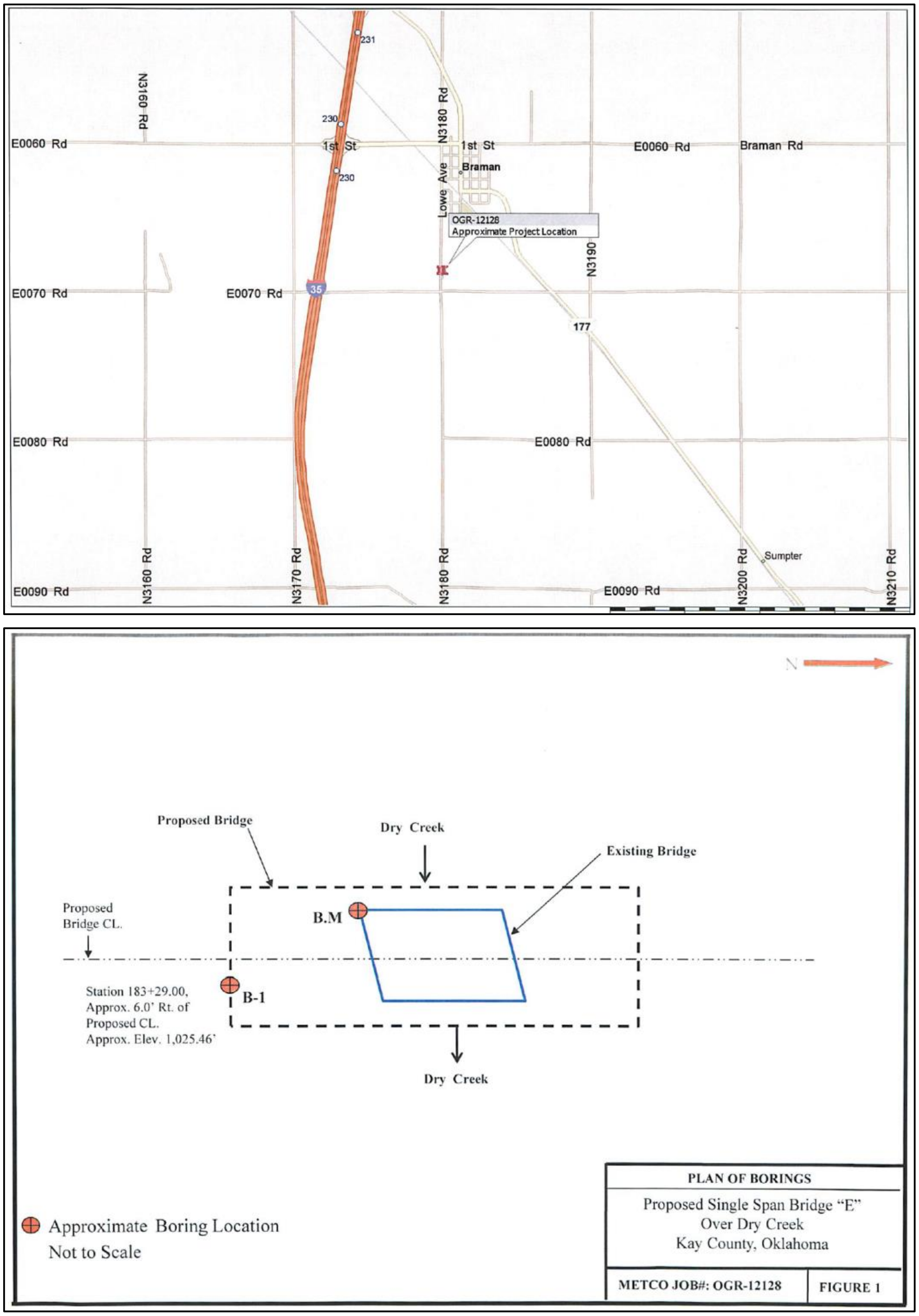


E0060 Rd
1st St
N3180 Rd
1st St
E0060 Rd
Braman Rd
Braman
Lowe Ave
N3160 Rd
OGR-12126
Approximate Project Location
E0070 Rd
E0070 Rd
177
N3190 Rd
E0080 Rd
E0080 Rd
N3160 Rd
N3170 Rd
N3180 Rd
N3200 Rd
Sumpter
N3210 Rd



N
Dry Creek
Existing Bridge
Proposed Bridge
Proposed Bridge CL.
B.M
EW 7 Road
B-1
Station 304+42.00, Approx. 3.5' Rt. of Proposed CL. Approx. Elev. 1,025.28'
Dry Creek
Approximate Boring Location
Not to Scale
PLAN OF BORINGS
Proposed Single Span Bridge "D"
Over Dry Creek
Kay County, Oklahoma
METCO JOB#: OGR-12126
FIGURE 1

(c)



(d)

Figure 64: Geographical location of single-span: (a) Bridge 2-‘B’; (b) Bridge 3-‘C’; (c) Bridge4-‘D’; (D) Bridge 5-‘E’; over Dry Creek in Kay County, OK (METCO 2012)

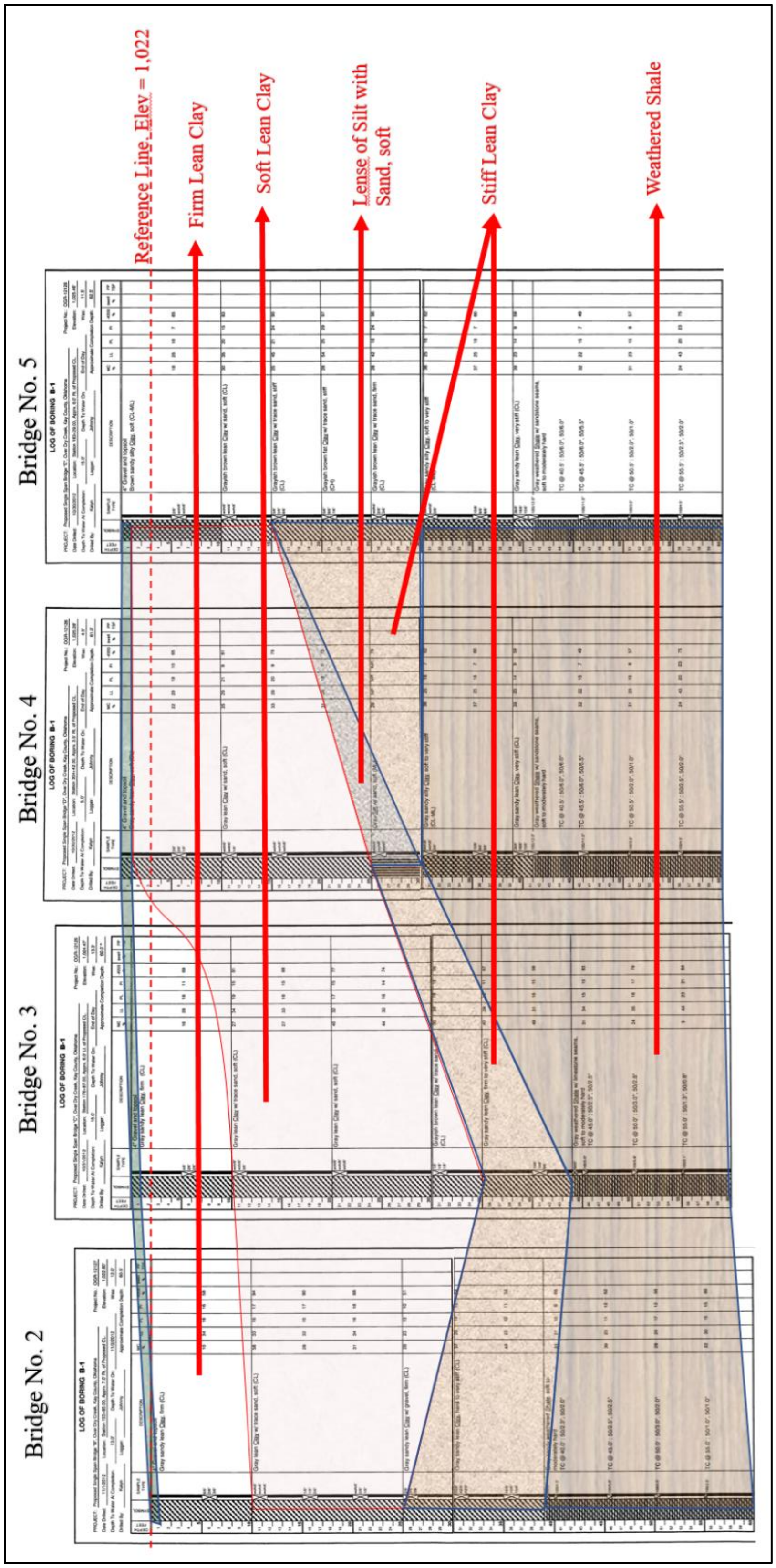


Figure 65: Fences of borings Based on geotechnical engineering report by METCO, 2012)

In addition to the geotechnical report soil samples were obtained from a borehole at a depth of 4.27 m (14 ft.) near Bridge No. 5 using a hand auger in February 2015. **Figure 66** shows the soil samples with the tare numbers indicating their original depth in ft. Results of a visual classification according to ASTM D2488 are shown in **Figure 67**. The classification included the samples moisture condition, color and consistency. The samples gravimetric water contents were then determined using the oven drying method (ASTM D2216) with the results as shown in **Figure 68**. Hand calculations indicated that a gravimetric water content value of 30% corresponds to full saturation. Therefore, it was concluded that the soil below the depth of 1.83 m (6 ft.) from Bridge No. 5 abutment toe was saturated when the soil samples were obtained from the site. However, due to capillary rise the phreatic surface was expected to be deeper than 1.83 m (6 ft.)

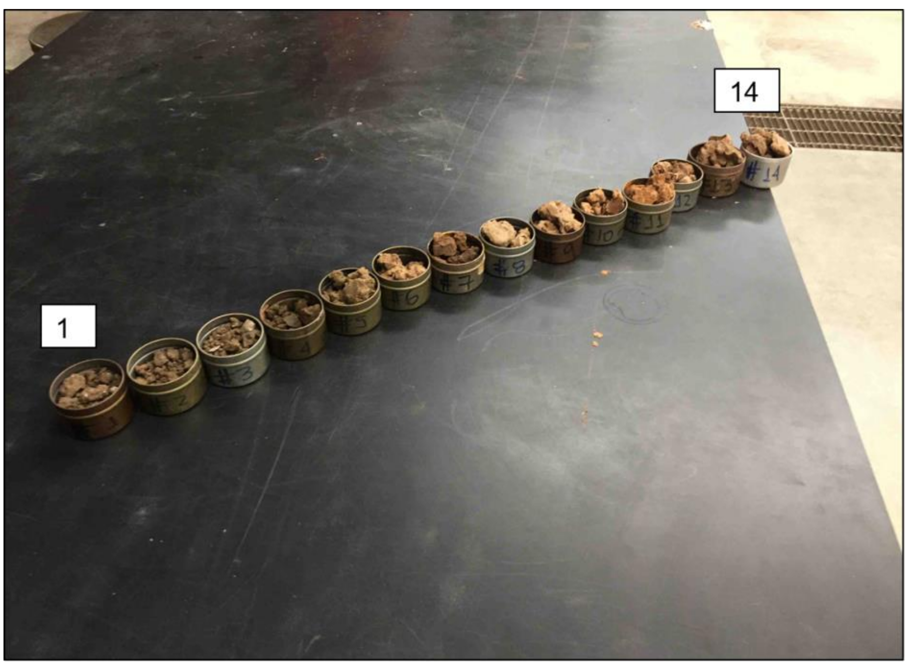


Figure 66: Hand auger samples taken from near an abutment of Bridge No. 5 down to a depth of 4.27m. Numbers indicate the sampling depth in ft. (e.g. Sample No. 14 was taken from the last foot of the borehole between 3.96 m and 4.27 m below ground surface).

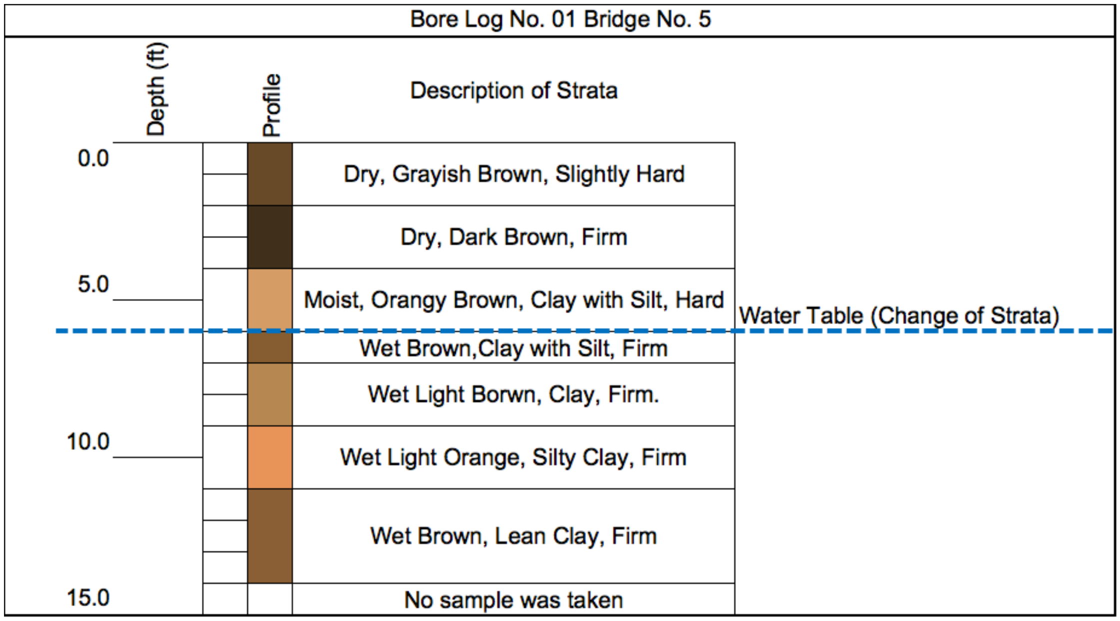


Figure 67: Visual classification of soil samples taken from a borehole at the location of Bridge No. 5 following ASTM D2488 test protocol

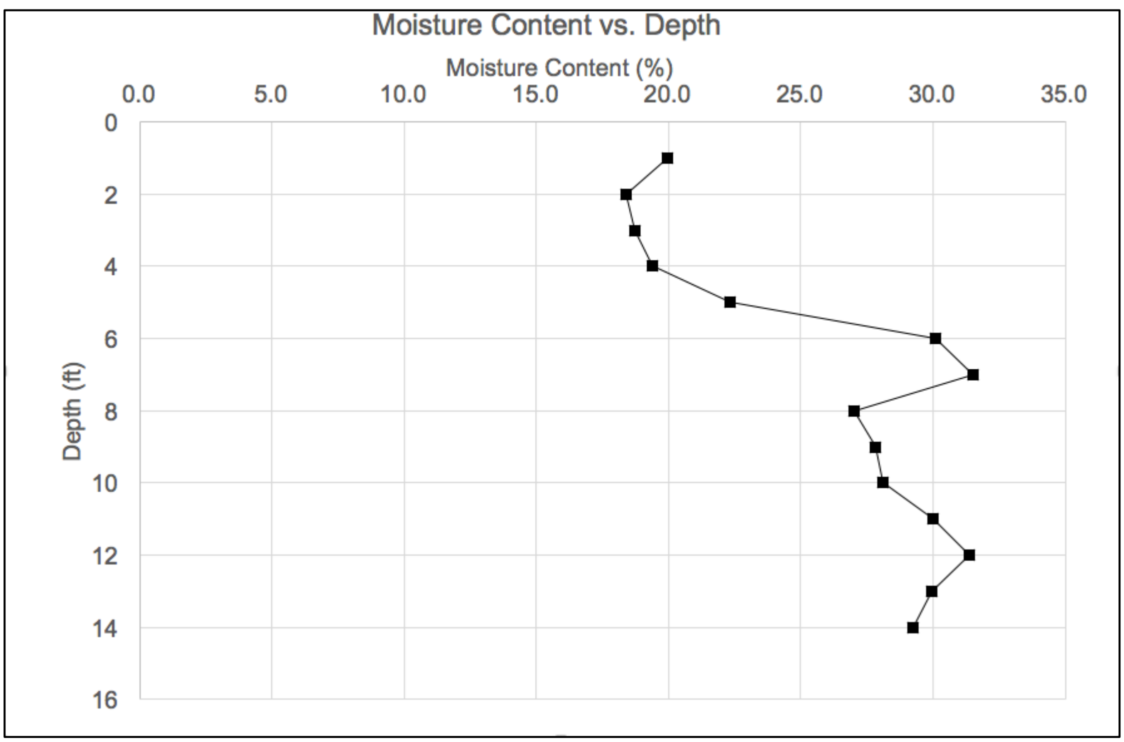


Figure 68: Gravimetric water content results for the soil samples from a borehole at the site of Bridge No. 5

4.7 Hydraulic Considerations of four GRS-IBS bridges in Kay County, Oklahoma

According to Simpson (2015), the maximum water velocities for the GRS projects over Dry Creek in Kay County are within the range between 0.5 m/sec and 0.76 m/sec (1.5 and 2.5 ft/sec), which indicates that these structures are in a flood plain rather than in a high run-off area. Therefore, scouring is not a real concern for these abutment structures.

Nevertheless, the lower part of the GRS facings was covered with riprap as added precaution for their stability (**Table 30**). Also, the reinforced soil foundation (RSF) of each GRS-IBS was conservatively placed 30 in. below the scour depth determined by the hydraulic engineer.

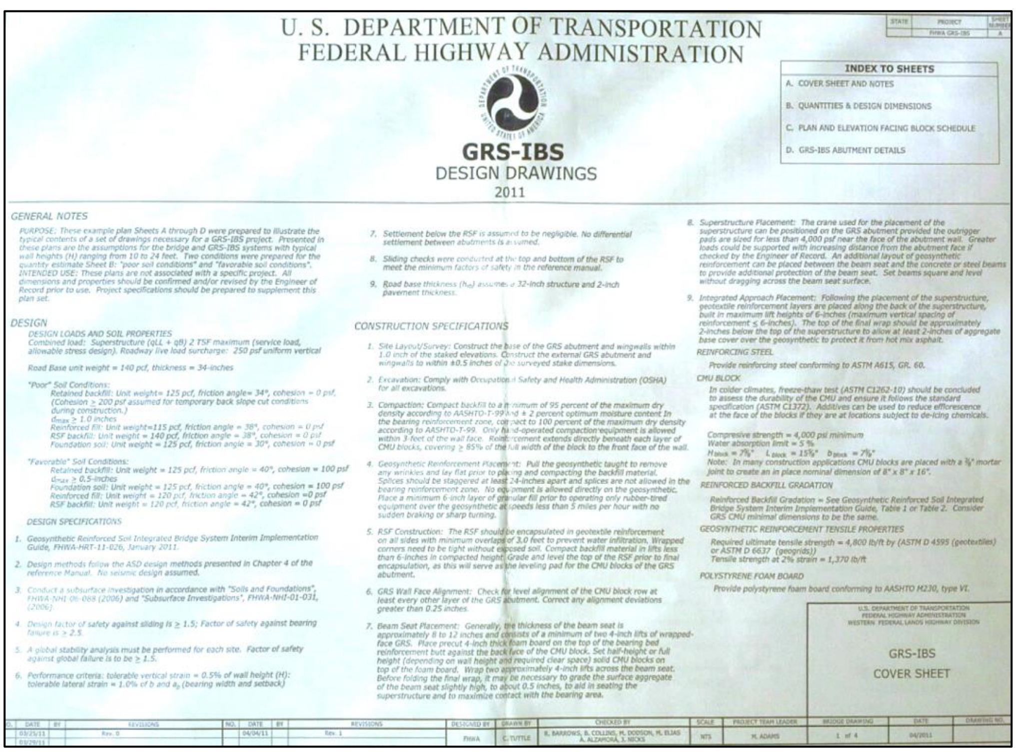
Is worth noting that on May 23th, 2015 all six bridges experienced record-breaking rainfalls and flash flooding (Ngo 2015; Hatami et al. 2016). Bridges 5 and 6 were submerged by approximately 0.3m (1ft).

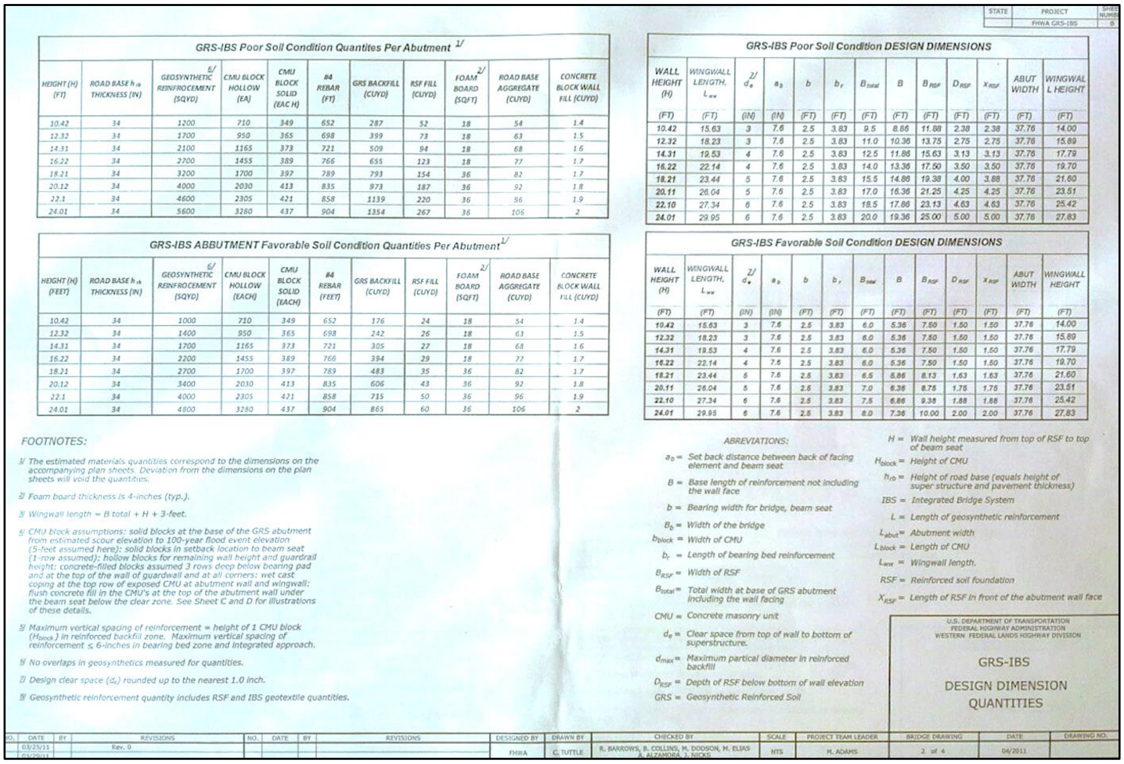
Table 30: Scour countermeasure information on GRS-IBS projects in Oklahoma

<table>
<tr><th colspan="2">Bridge</th><th>Scour Countermeasure</th><th>Service Under Bridge</th></tr>
<tr><td colspan="2">FHWA Guidelines</td><td>*Gabion mattresses
*Riprap aprons
*Articulated concrete blocks</td><td>Waterway or Roadway</td></tr>
<tr><td rowspan="4">Kay County</td><td>Bridge – 1</td><td rowspan="4">Riprap</td><td rowspan="4">Waterway</td></tr>
<tr><td>Bridge – 2</td></tr>
<tr><td>Bridge – 3</td></tr>
<tr><td>Bridge – 4</td></tr>
<tr><td colspan="2">Lincoln County
Yates Bridge</td><td>Rip Rap</td><td>Waterway</td></tr>
<tr><td colspan="2">Ottawa County Bridge</td><td>None</td><td>Roadway</td></tr>
</table>

4.8 Construction Phase of two conventional bridges and four GRS-IBS bridges in Kay County, Oklahoma

The GRS-IBS bridges in Kay County were constructed in overall conformance with the FHWA standard drawings (Adams et al. 2011). However, some adjustments were made in the construction of some of the bridges depending on the conditions of the site and availability of materials, among other factors. For instance, **Figure 69** shows the as-built drawings for one of the GRS-IBS bridges with the deviations from the original plan marked in red. Example changes include the facing type (e.g. hollow CMU filled with aggregate and use of a filled steel channel instead of Styrofoam panels as a seating pad underneath the superstructure beams).Photographs of all six (6) bridges in this study are shown in **Figure 70** through **Figure 74**. Also **Figure 75** depicts the GRS-IBS Bridge No. 3. **Figure 76** shows a pictorial account of the construction of Bridge No. 5 including its abutments and superstructure.





STATE | PROJECT FHWA GRS-IBS | SHEET NUMBER B

GRS-IBS Poor Soil Condition Quantites Per Abutment 1/

HEIGHT (H) (FT)	ROAD BASE h_{rb} THICKNESS (IN)	GEOSYNTHETIC REINFROCEMENT 6/ (SQYD)	CMU BLOCK HOLLOW (EA)	CMU BLOCK SOLID (EAC H)	#4 REBAR (FT)	GRS BACKFILL (CUYD)	RSF FILL (CUYD)	FOAM BOARD 2/ (SQFT)	ROAD BASE AGGREGATE (CUYD)	CONCRETE BLOCK WALL FILL (CUYD)
10.42	34	1200	710	349	652	287	52	18	54	1.4
12.32	34	1700	950	365	698	399	73	18	63	1.5
14.31	34	2100	1165	373	721	509	94	18	68	1.6
16.22	34	2700	1455	389	766	655	123	18	77	1.7
18.21	34	3200	1700	397	789	793	154	36	82	1.7
20.12	34	4000	2030	413	835	973	187	36	92	1.8
22.1	34	4600	2305	421	858	1139	220	36	96	1.9
24.01	34	5600	3280	437	904	1354	267	36	106	2

GRS-IBS ABUTMENT Favorable Soil Condition Quantities Per Abutment 1/

HEIGHT (H) (FEET)	ROAD BASE h_{rb} THICKNESS (IN)	GEOSYNTHETIC REINFROCEMENT 6/ (SQYD)	CMU BLOCK HOLLOW (EACH)	CMU BLOCK SOLID (EACH)	#4 REBAR (FEET)	GRS BACKFILL (CUYD)	RSF FILL (CUYD)	FOAM BOARD 2/ (SQFT)	ROAD BASE AGGREGATE (CUYD)	CONCRETE BLOCK WALL FILL (CUYD)
10.42	34	1000	710	349	652	176	24	18	54	1.4
12.32	34	1400	950	365	698	242	26	18	63	1.5
14.31	34	1700	1165	373	721	305	27	18	68	1.6
16.22	34	2200	1455	389	766	394	29	18	77	1.7
18.21	34	2700	1700	397	789	483	35	36	82	1.7
20.12	34	3400	2030	413	835	606	43	36	92	1.8
22.1	34	4000	2305	421	858	715	50	36	96	1.9
24.01	34	4800	3280	437	904	865	60	36	106	2

GRS-IBS Poor Soil Condition DESIGN DIMENSIONS

WALL HEIGHT (H)	WINGWALL LENGTH, L_{ww}	7/ d_e	a_b	b	b_r	B_{total}	B	B_{RSF}	D_{RSF}	X_{RSF}	ABUT WIDTH	WINGWALL HEIGHT
(FT)	(FT)	(IN)	(IN)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)
10.42	15.63	3	7.6	2.5	3.83	9.5	8.86	11.88	2.38	2.38	37.76	14.00
12.32	18.23	3	7.6	2.5	3.83	11.0	10.36	13.75	2.75	2.75	37.76	15.89
14.31	19.53	4	7.6	2.5	3.83	12.5	11.86	15.63	3.13	3.13	37.76	17.79
16.22	22.14	4	7.6	2.5	3.83	14.0	13.36	17.50	3.50	3.50	37.76	19.70
18.21	23.44	5	7.6	2.5	3.83	15.5	14.86	19.38	4.00	3.88	37.76	21.60
20.11	26.04	5	7.6	2.5	3.83	17.0	16.36	21.25	4.25	4.25	37.76	23.51
22.10	27.34	6	7.6	2.5	3.83	18.5	17.86	23.13	4.63	4.63	37.76	25.42
24.01	29.95	6	7.6	2.5	3.83	20.0	19.36	25.00	5.00	5.00	37.76	27.83

GRS-IBS Favorable Soil Condition DESIGN DIMENSIONS

WALL HEIGHT (H)	WINGWALL LENGTH, L_{ww}	7/ d_e	a_b	b	b_r	B_{total}	B	B_{RSF}	D_{RSF}	X_{RSF}	ABUT WIDTH	WINGWALL HEIGHT
(FT)	(FT)	(IN)	(IN)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)
10.42	15.63	3	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	14.00
12.32	18.23	3	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	15.89
14.31	19.53	4	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	17.79
16.22	22.14	4	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	19.70
18.21	23.44	5	7.6	2.5	3.83	6.5	5.86	8.13	1.63	1.63	37.76	21.60
20.11	26.04	5	7.6	2.5	3.83	7.0	6.36	8.75	1.75	1.75	37.76	23.51
22.10	27.34	6	7.6	2.5	3.83	7.5	6.86	9.38	1.88	1.88	37.76	25.42
24.01	29.95	6	7.6	2.5	3.83	8.0	7.36	10.00	2.00	2.00	37.76	27.83

FOOTNOTES:

1/ The estimated materials quantities correspond to the dimensions on the accompanying plan sheets. Deviation from the dimensions on the plan sheets will void the quantities.

2/ Foam board thickness is 4-inches (typ.).

3/ Wingwall length = B total + H + 3-feet.

5/ CMU block assumptions: solid blocks at the base of the GRS abutment from estimated scour elevation to 100-year flood event elevation (5-feet assumed here): solid blocks in setback location to beam seat (1-row assumed): hollow blocks for remaining wall height and guardrail height: concrete-filled blocks assumed 3 rows deep below bearing pad and at the top of the wall of guardwall and at all corners: wet cast coping at the top row of exposed CMU at abutment wall and wingwall: flush concrete fill in the CMU's at the top of the abutment wall under the beam seat below the clear zone. See Sheet C and D for illustrations of these details.

5/ Maximum vertical spacing of reinforcement = height of 1 CMU block (H_{block}) in reinforced backfill zone. Maximum vertical spacing of reinforcement ≤ 6-inches in bearing bed zone and integrated approach.

6/ No overlaps in geosynthetics measured for quantities.

7/ Design clear space (d_e) rounded up to the nearest 1.0 inch.

8/ Geosynthetic reinforcement quantity includes RSF and IBS geotextile quantities.

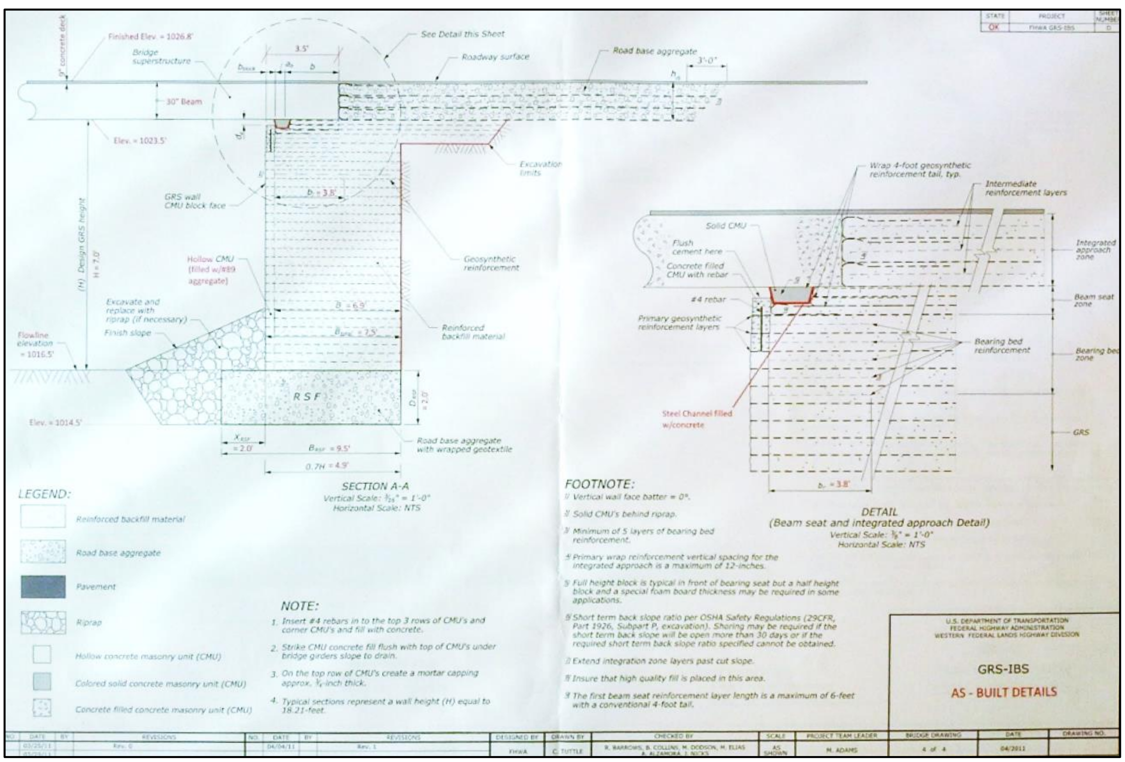
ABREVIATIONS:

a_b = Set back distance between back of facing element and beam seat
B = Base length of reinforcement not including the wall face
b = Bearing width for bridge, beam seat
B_b = Width of the bridge
b_{block} = Width of CMU
b_r = Length of bearing bed reinforcement
B_{RSF} = Width of RSF
B_{total} = Total width at base of GRS abutment including the wall facing
CMU = Concrete masonry unit
d_e = Clear space from top of wall to bottom of superstructure.
d_{max} = Maximum partical diameter in reinforced backfill
D_{RSF} = Depth of RSF below bottom of wall elevation
GRS = Geosynthetic Reinforced Soil
H = Wall height measured from top of RSF to top of beam seat
H_{block} = Height of CMU
h_{rb} = Height of road base (equals height of super structure and pavement thickness)
IBS = Integrated Bridge System
L = Length of geosynthetic reinforcement
L_{abut} = Abutment width
L_{block} = Length of CMU
L_{ww} = Wingwall length.
RSF = Reinforced soil foundation
X_{RSF} = Length of RSF in front of the abutment wall face

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION

GRS-IBS
DESIGN DIMENSION QUANTITIES

NO.	DATE	BY	REVISIONS	NO.	DATE	BY	REVISIONS	DESIGNED BY	DRAWN BY	CHECKED BY	SCALE	PROJECT TEAM LEADER	BRIDGE DRAWING	DATE	DRAWING NO.
	03/25/11		Rev. 0					FHWA	C. TUTTLE	R. BARROWS, B. COLLINS, M. DODSON, M. ELIAS A. ALZAMORA, J. NICKS	NTS	M. ADAMS	2 of 4	04/2011	
	03/29/11														



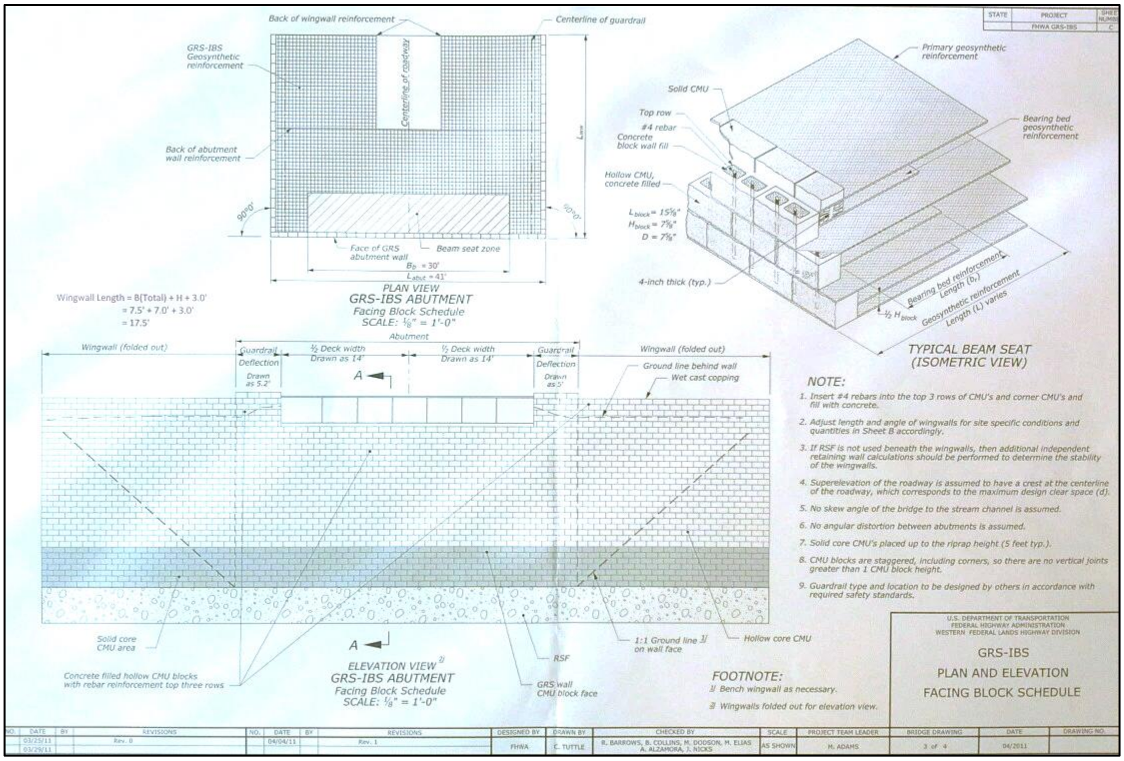


Figure 69: As-built drawings for GRS-IBS Bridge No. 2 in Kay County

(Courtesy of Mr. Tom Simpson, PE)

Figure 70: Bridge No. 1 - conventional abutment support with H-Piles driven to bedrock

Figure 71: Bridge No. 2 - GRS-IBS with an 18 in.-wide concrete-filled steel channel as the bearing pad for the steel girders

Figure 72: Bridge No. 3 - GRS-IBS under construction with sheet pile facing

Figure 73: Bridge No. 5 - GRS-IBS with concrete block facing (geotextile reinforcement needs to be trimmed)

Figure 74: Bridge No. 6 - Bridge on conventional pile support under construction

Figure 75: A GRS-IBS (Bridge No. 3) under construction near Blackwell in Kay County with sheet piling for the abutment facing

Figure 76: Different stages of GRS-IBS construction near Blackwell in Kay County, OK (photographs courtesy of Mr. Tom Simpson, PE)

Additional photographs of completed Bridges No. 3 (GRS-IBS with sheet pile abutment facing), Bridge No. 5 (GRS-IBS with CMU blocks facing) and No. 6 (Conventional bridge abutments on H-piles driven to the bedrock but with sheet pile facing) are shown in **Figure 77** through **Figure 80**, respectively. The 22^{o} skew in Bridge No. 3 is indicated with a red circle in **Figure 77**. In Bridge No. 5 (**Figure 79**), box beams were originally planned for the superstructure in the design. However, 7-inch concrete beams overlaid with steel girders from a 50-year-old bridge from the I-40 Crosstown project in Oklahoma City were ultimately used in its construction.

Figure 77: Completed Bridge No. 3 (GRS-IBS) with sheet pile facing (red circle indicates a 22 skew in the alignment of the bridge superstructure)

Figure 78: Completed Bridge No. 5 bridge (GRS-IBS) with sheet pile abutment facing

Figure 79: Completed Bridge No. 5 bridge (GRS-IBS) with sheet pile abutment facing

Figure 80: Completed Bridge No. 6 (conventional abutments on H-Piles driven to bedrock but with sheet pile abutment facing)

4.9 Cost and Summary data of GRS-IBS bridge in Kay County, OK

Table 31 and **Table 32** include updated data on the GRS-IBS and conventional bridges reported by Ngo (2016) and Hatami et al. (2016) that are the focus of this study. Kay County reported savings of up to $40,000 dollars in this very first major experience with GRS-IBS construction and they expect greater time and monetary savings as more experience is gained in the future projects.

Table 31: Updated information on the 6 bridges in Kay County, OK

(includes information from Mr. Tom Simpson, PE)

Bridge	Span Length m (ft)	Abutment Height m (ft)	Bridge Width m (ft)	Abutment Cost	Total Cost	Construction Time (days)	Completion Year
Conventional Bridge 1	15.3 (50.0)	2.2 (7.0)	9.2 (30)	$ 60,000	$ 105,000	30 - 40	2014
GRS-IBS Bridge 2				$ 31,000	$ 79,000	30	2014
GRS-IBS Bridge 3				$ 35,000	$ 82,000	30	2015
GRS-IBS Bridge 4				$ 35,000	$ 82,000	30	2015
GRS-IBS Bridge 5				$ 31,000	$ 142,000	21	2014
Conventional Bridge 6				$ 60,000	$ 165,000	24	2014

Table 32: Summary data on the six bridges in Kay County, OK

<table>
<tr><th>Bridge</th><th>Facing element</th><th>GRS fill</th><th>GRS reinforcement</th><th>Foundation type</th><th>Scour protection</th></tr>
<tr><td>Conventional Bridge 1</td><td>Sheet piling</td><td>N/A</td><td>N/A</td><td>H-Piles driven to bedrock</td><td>No rip-rap</td></tr>
<tr><td>GRS-IBS Bridge 2</td><td>CMU</td><td rowspan="4">No. 89 stone in abutment, No. 57 gravel in road base and RSF</td><td rowspan="4">TerraTex HPG-57 woven geotextile</td><td rowspan="4">RSF</td><td>Rip-rap</td></tr>
<tr><td>GRS-IBS Bridge 3</td><td>5-meters (15-foot)-high sheet piling</td><td rowspan="2">No rip-rap</td></tr>
<tr><td>GRS-IBS Bridge 4</td><td>5 meters (15-foot)-high sheet piling</td></tr>
<tr><td>GRS-IBS Bridge 5</td><td>CMU</td><td>Rip-rap</td></tr>
<tr><td>Conventional Bridge 6</td><td>Sheet piling</td><td>N/A</td><td>N/A</td><td>H-Piles driven</td><td>No Riprap</td></tr>
</table>

4.10 Performance Monitoring of GRS-IBS Bridges and Comparable Conventional Bridges in Kay County, OK

4.10.1 Weather Data

Performance monitoring started with the recollection of weather data for the site of GRS-IBS bridges in Kay County, OK. The data was obtained from Daymet and Mesonet which provided historical peak weather data such as maximum and minimum temperatures, precipitation record between 1985 and 2016. The data in **Table 33** indicate that the site of the GRS-IBS projects in Kay County is subjected to significant temperature fluctuations. For instance, in 2011 the temperature varied between -21^{o}F and 110^{o}F. Also, the peak precipitation was recorded as 10.73 in. in 2015, which resulted in flooding in the area reported by Ngo (2016) and Hatami et al. (2016)

Table 33: Historical weather data for the geographical location of GRS-IBS bridges near Blackwell in Kay County during the period between 1985 and 2015

Year	Temp. Max (ºC)	Temp. Max (ºF)	Temp. Min (ºC)	Temp. Min (ºF)	Max Precip. (mm)	Max Precip (in)
1985	41	105.8	-19	-2.2	48	1.89
1986	43.5	110.3	-14	6.8	120	4.72
1987	39.5	103.1	-15.5	4.1	76	2.99
1988	40.5	104.9	-24	-11.2	56	2.20
1989	39	102.2	-24.5	-12.1	69	2.72
1990	41.5	106.7	-18.5	-1.3	37	1.46
1991	40.5	104.9	-13	8.6	37	1.46
1992	37	98.6	-10.5	13.1	54	2.13
1993	40	104	-16	3.2	50	1.97
1994	39.5	103.1	-15	5	73	2.87
1995	41	105.8	-14.5	5.9	55	2.17
1996	43	109.4	-21	-5.8	86	3.39
1997	36.5	97.7	-17	1.4	90	3.54
1998	41	105.8	-16	3.2	98	3.86
1999	40	104	-14	6.8	53	2.09
2000	42	107.6	-17.5	0.5	44	1.73
2001	41	105.8	-14	6.8	47	1.85
2002	38	100.4	-17.5	0.5	60	2.36
2003	40.5	104.9	-16.5	2.3	64	2.52
2004	37.5	99.5	-16	3.2	47	1.85
2005	38.5	101.3	-18	-0.4	76	2.99
2006	42	107.6	-15.5	4.1	66	2.60
2007	39.5	103.1	-15.5	4.1	59	2.32
2008	39.5	103.1	-15.5	4.1	83	3.27
2009	39.5	103.1	-15	5	106	4.17
2010	41	105.8	-17.5	0.5	188	7.41
2011	43.5	110.3	-29.5	-21.1	119	4.68
2012	43.5	110.3	-12	10.4	66	2.61
2013	39.5	103.1	-16	3.2	157	6.18
2014	40	104	-19	-2	208	8.17
2015	41	105	-14	6	273	10.73
2016	40	104	-11	12	212	8.33

4.10.2 Local Seismicity

During the past two years, seismic activity has been increasing in including frequent earthquakes with magnitudes smaller than M = 4.5 (USGS 2016) on the west side of Kay County (**Figure 81** and **Figure 82**). Nevertheless, current FHWA guidelines (Adams et al. 2012) suggest that GRS-IBS should withstand significant seismic loads (reportedly, as high as 1g ground acceleration or M = 6), which is significantly greater than what has so far been recorded in Oklahoma. Therefore, GRS-IBS abutments can provide viable solutions to replace deficient bridges or construction of new bridges in Oklahoma.

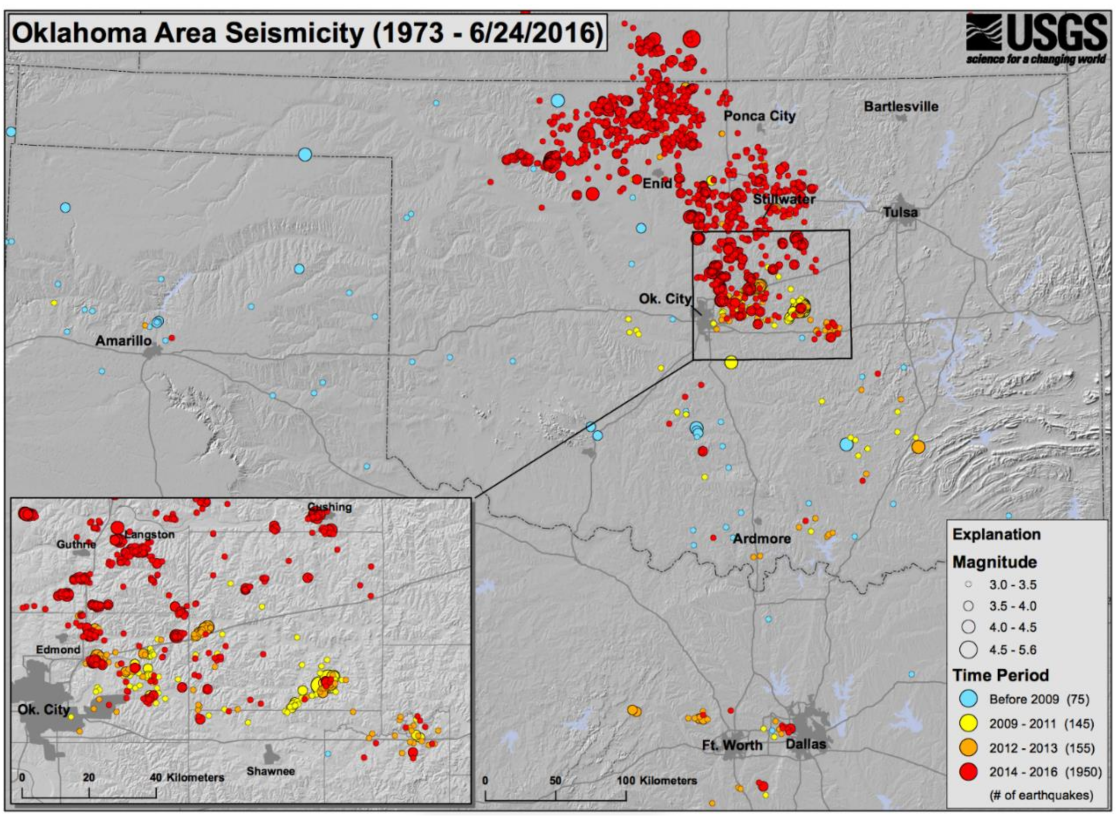


Figure 81: Oklahoma seismicity data from 1973 to June 24, 2016 (USGS 2016)

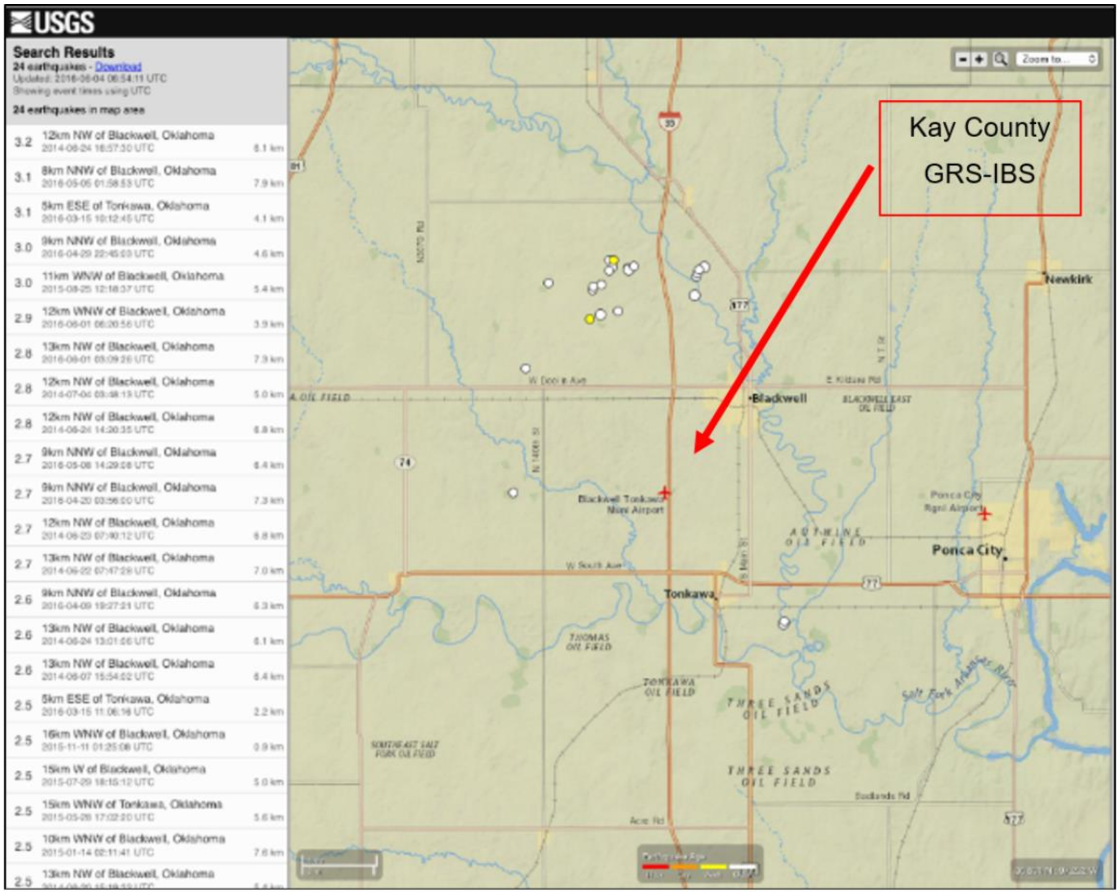
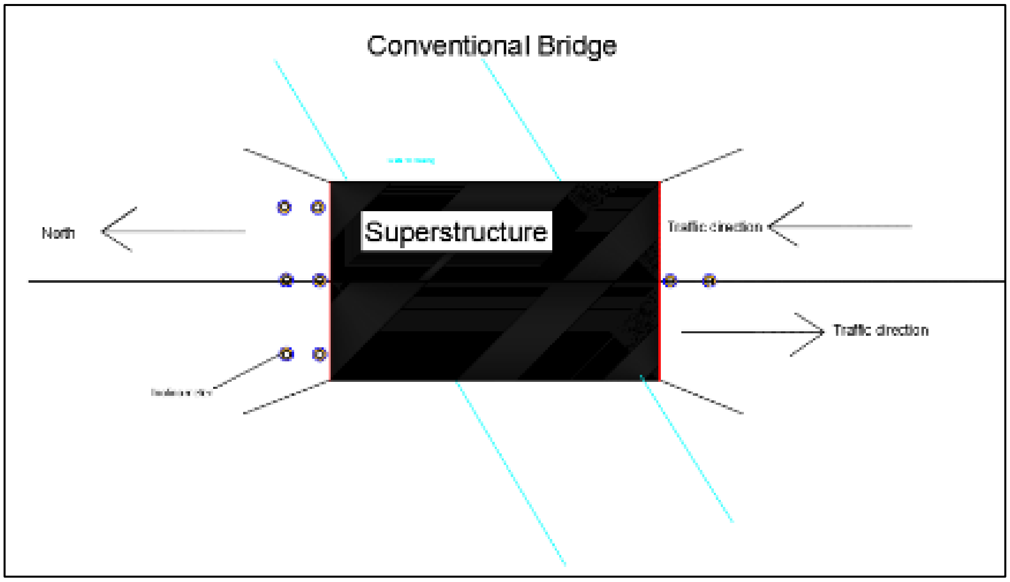


Figure 82: Seismicity data for Kay County during the period beween 2014 and May 31, 2016 (USGS 2016)

4.10.3 Alternative Monitoring Systems Examined for Kay County Bridges

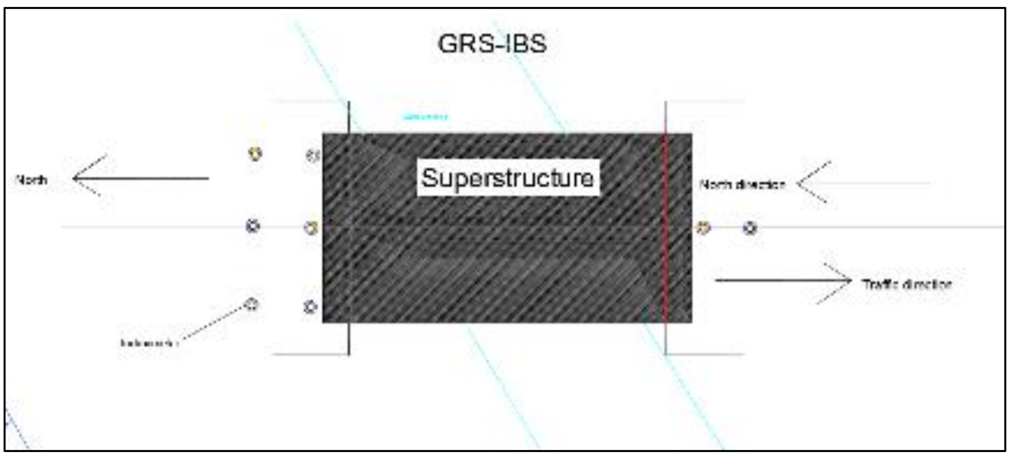
At the begging of this study, the possibility of using inclinometers in the bridge abutments was considered as a monitoring technique (**Figure 83**, **Figure 84 and Table 34**). Inclinometers together with Micro-electro-mechanical systems (MEMs) provide real-time and remote-sensing capabilities to monitor wall deformations, ground movements, and slope movements. The bedrock at the site of the bridges in Kay County is approximately 12.2 m 40 ft. deep and the native soil at the sites of these bridges is mostly soft lean clay (METCO 2012).

However, based on the literature review results, the feedback provided by the Oklahoma Department of Transportation, the Bureau of Internal Affairs, and given the depth of the borehole needed for the inclinometers to reach the bedrock to provide reliable measurements and the comparatively short height of the GRS abutments, it was determined that using inclinometers would not be the best monitoring option for the Kay County projects and it was decided that the most practical and economic method survey method to use was the Total Station Theodolite (TST).



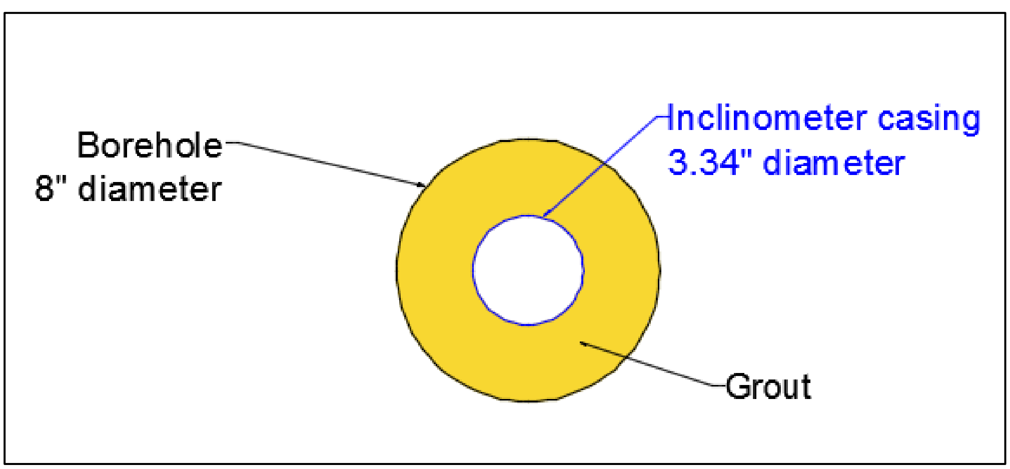
Conventional Bridge
Superstructure
North
Traffic direction
Traffic direction

(a)



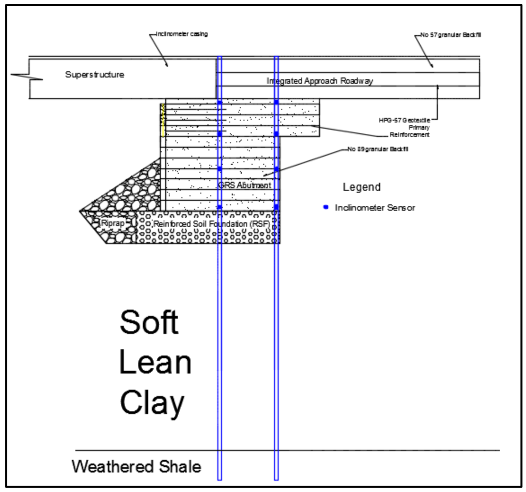
GRS-IBS
Superstructure
North
Traffic direction

(b)



Borehole
8" diameter
Inclinometer casing
3.34" diameter
Grout

(c)



(d)

Figure 83: Initial inclinometer plans for conventional and GRS-IBS bridges: (a) – (d)

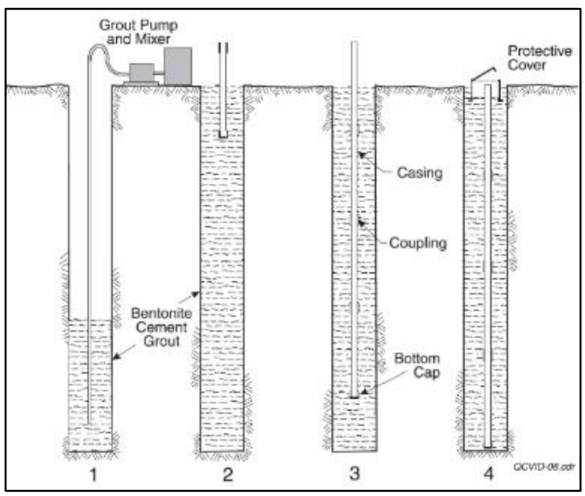


Figure 84: Recommended grout mix for the inclinometer boreholes and installation process (RST 1997)

Table 34: Inclinometer grout

Bentonite- Cement Grout		
Materials	Weight kg (lb)	Percent
Portland Cement	42.6 (94)	15%
Bentonite*	17.7 (39)	6%
Water	75 gallons ~ 283 kg (625 lb)	79%
*Mix bentonite with water first, then add the cement		

4.10.4 Survey with Total Station Theodolite (TST)

As reported by Ngo (2016), a three-week TST training program was offered by Dr. Russel Dutnell on the University of Oklahoma campus. The training included a closed-loop survey near the CEC engineering building. The TST used was a Topcon GTS-211D model shown in **Figure 85a.** Also, the accuracy of each mode of operation is given in **Table 35**. *Fine Mode* with the accuracy of 1 mm (0.003 ft.) was used to survey the GRS-IBS abutments and comparable conventional bridges in Kay County.

Subsequent to the survey training period, a pilot surveying set was performed to Bridge No. 2 in Kay County, OK. One provisional benchmark and control point were installed on the shoulders of the road outside of the approach embankment. A GPS Garmin -72H was used initially to obtain the coordinates of the benchmark and the control points (**Figure 85b**). However, the accuracy of this GPS device is approximately 5 ft. Therefore, the coordinates determined in this preliminary stage (**Table 36**) were used only to help locate the reference points in the subsequent visits to the site. After the training, provisional survey points listed in **Table 37** were monitored as a pilot test (**Figure 86**).

Table 35: Topcon GTS-211D measurement modes

Mode	Brief Description	Accuracy (ft)
Fine Mode	This is the normal mode. Measurement time is 2.5 seconds	1 mm (0.003)
Tracking mode	Useful when tracking the prism in motion. easurement time is 0.3 seconds	1 cm (0.03)
Coarse mode	Can be used for stake out. Measurement time is 0.5 seconds	1 cm (0.03)

Figure 85: (a) Total Station Topcon GTS-211D used in this study to survey and monitor the deformations of GRS-IBS and conventional bridges; (b) GPS Garmin - 72H

Table 36: Coordinates of the benchmark and control points as obtained using the Garmin GPS

Point	North Coordinates	East Coordinates
Benchmark (Total Station)	36° 54.341’	97° 20.214’
Control point (Back Sight)	36° 54.338’	97° 20.203’

Table 37: Survey points initially considered for Bridges Nos.1 & 2

Bridge No.1	Geographical Coordinates	
Control Point	36°54'13.15 N	97°20'12.58"W
S1	36°54'13.53"N	97°20'12.72"W
S2	36°54'13.54"N	97°20'12.40"W
S3	36°54'13.08"N	97°20'12.75"W
S4	36°54'13.07"N	97°20'12.41"W
Benchmark	36°54'12.78"N	97°20'12.28"W

(cont’d)

Bridge No.2	Geographical Coordinates	
Control Point	36°54'21.22"N	97°20'12.67"W
S1	36°54'21.65"N	97°20'12.86"W
S2	36°54'21.66"N	97°20'12.50"W
S3	36°54'21.11"N	97°20'12.78"W
S4	36°54'21.11"N	97°20'12.50"W
Benchmark	36°54'20.89"N	97°20'12.43"W

Figure 86: Bridges Nos. 1 & 2 with pilot survey points

After obtaining the coordinates of the benchmark and the control points, the following conventions were assigned: N (Northing), E (Easting) and Z (Z-value; i.e. collectively, NEZ) coordinates to the benchmark and the back-sight control points. **Table 36 and Table 38** list the coordinates obtained from the first monitoring attempt for Bridge No. 2. The data related to the NEZ coordinates, was input in the software program TopoCal (http://www.topocal.com/), which generated contour lines that allowed visual comparisons among different coordinates that were taken over the monitoring period. **Figure 87** shows an example TopoCal model of the GRS Bridge No. 2 in which the superstructure, the benchmark and the control point are shown. **Figure 88** shows a close-up view of the contour lines for the superstructure and the GRS abutment.

Table 38: Survey coordinates on Bridge No. 2 (in meters)

	Designation of the point in TopoCal model	East (X)	North (Y)	Elevation (Z)
Southern boundary of superstructure.	Benchmark (Total Station)	1000.000	1000.000	1000.000
	Control Point (Back Sight)	1000.016	1022.191	1000.227
	1	978.321	1001.214	1000.726
	2	978.004	1002.320	1000.756
	3	977.851	1003.068	1000.765
	4	977.537	1004.149	1000.784
	5	977.310	1005.101	1000.785
	6	977.062	1005.946	1000.782
	7	976.827	1006.756	1000.782
	8	976.642	1007.457	1000.781
	9	975.967	1009.824	1000.742
Middle of superstructure.	10	970.883	999.253	1000.719
	11	970.398	1000.968	1000.753
	12	969.873	1002.905	1000.776
	13	969.328	1005.284	1000.761
	14	968.707	1007.769	1000.710
Northern boundary of superstructure.	15	959.459	1005.192	1000.709
	16	960.434	1002.227	1000.752
	17	960.704	1000.739	1000.760
	18	961.265	998.601	1000.750
	19	961.774	996.680	1000.732

Survey coordinates on Bridge No. 2 (Cont'd)

	Designation of the point in TopoCal model	East (x)	North (Y)	Elevation (Z)
South Abutment		978.557	1001.334	1000.695
		977.959	1003.088	1000.747
		977.442	1005.133	1000.755
		976.775	1007.565	1000.769
		976.229	1009.889	1000.722
		978.662	1003.265	1000.720
		978.363	1005.421	1000.742
		977.608	1007.715	1000.744
	S-ABT 1	979.913	1003.587	1000.708
		979.132	1005.694	1000.730
	S- ABT 2	978.440	1007.988	1000.748
North Abutment		961.553	996.586	1000.731
		961.103	998.567	1000.758
		960.570	1000.679	1000.749
		960.149	1002.104	1000.736
		959.667	1003.610	1000.716
		960.464	998.373	1000.746
		959.941	1000.489	1000.754
		959.533	1002.127	1000.744
	N-ABT 1	959.428	998.061	1000.746
		958.835	1000.220	1000.749
	N - ABT 2	958.403	1001.853	1000.764

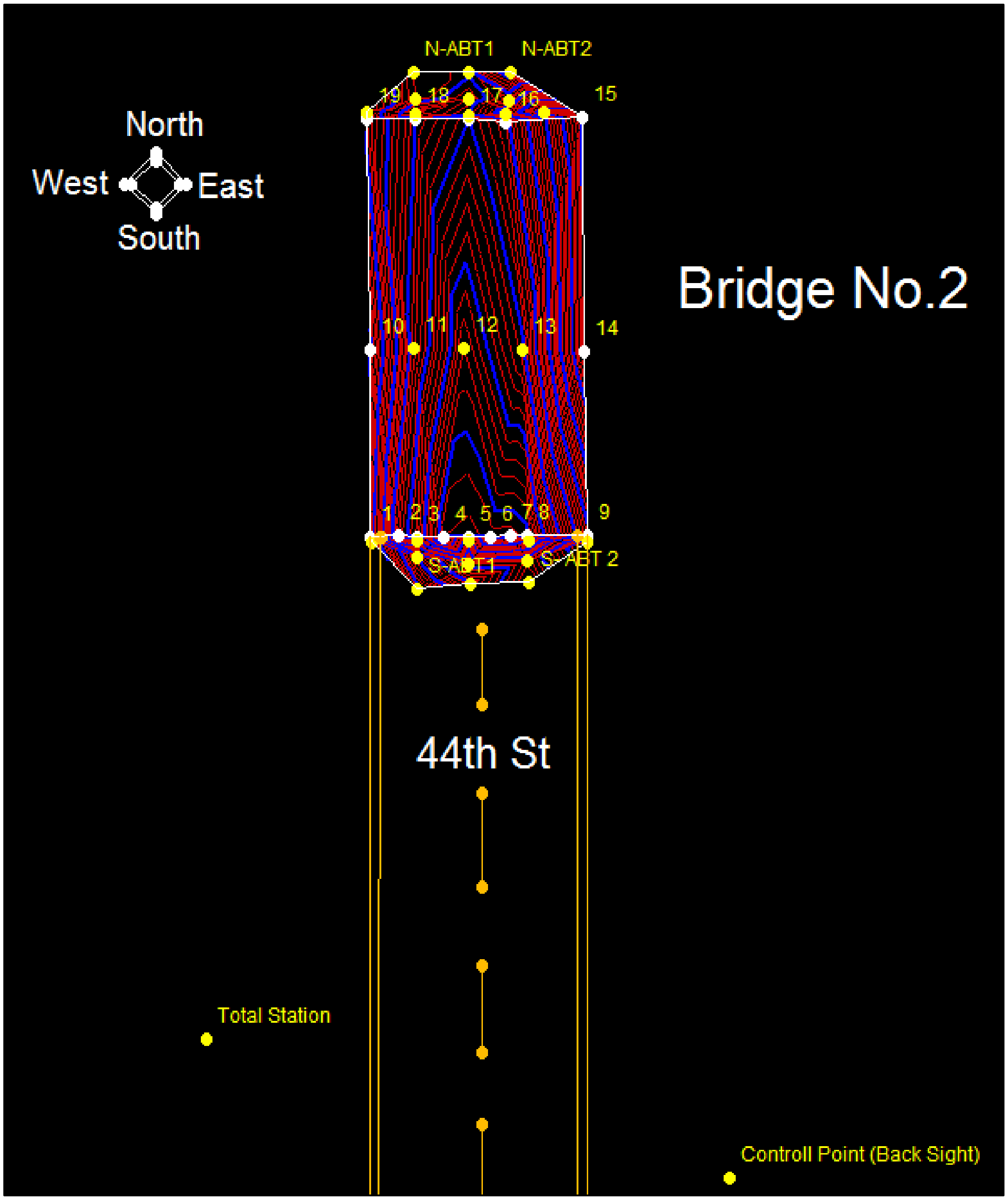


Figure 87: TopoCal contour lines for Bridge No. 2

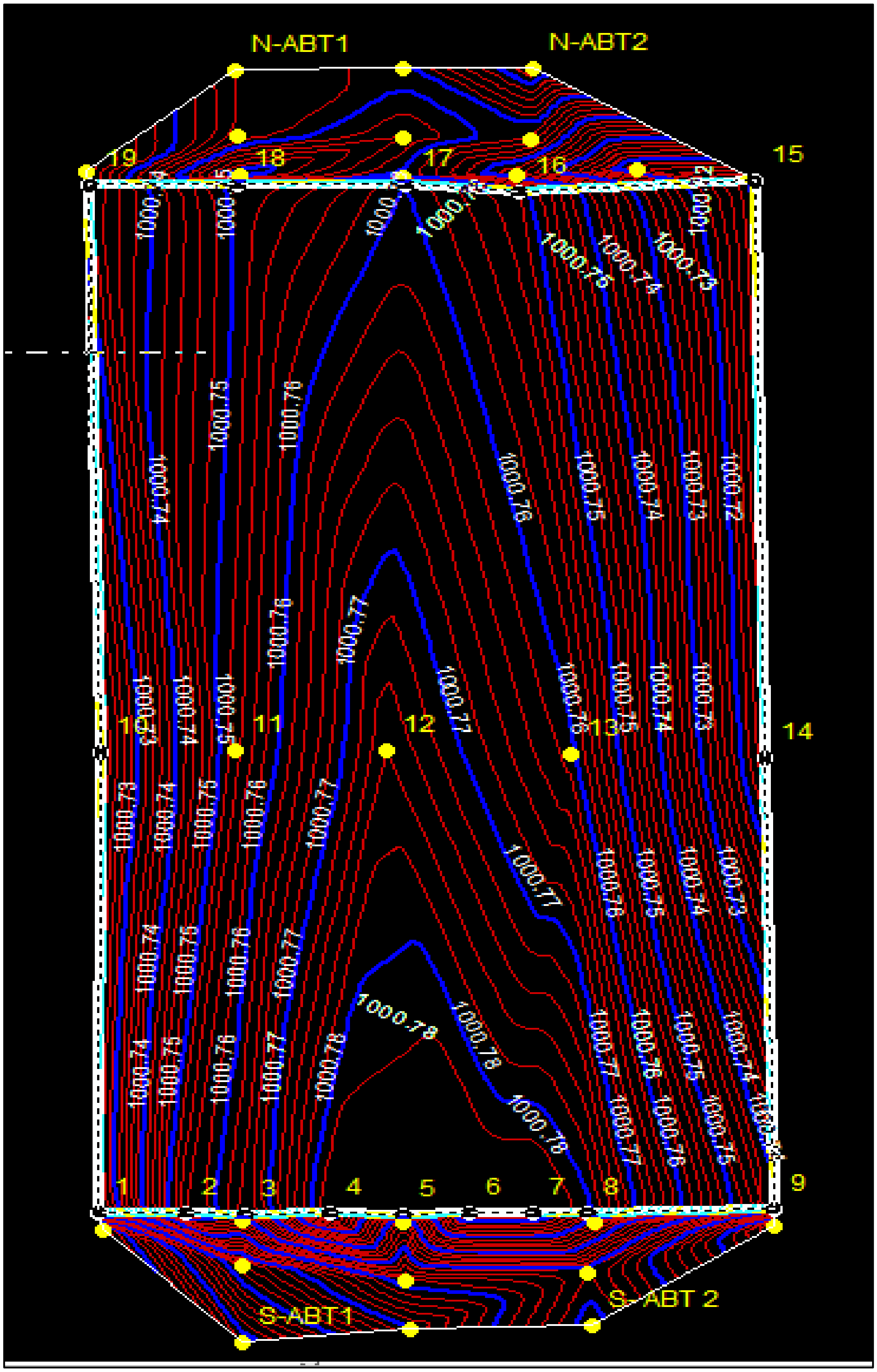


Figure 88: Higher resolution TopoCal contour lines for Bridge No. 2

(Blue line interval=10 mm, Red line interval = 2 mm)

4.10.5 *Benchmark installation and control Points*

Ngo (2016) and Hatami et al. (2016) reported the benchmark installation process which can be summarized by the following steps: (1) A 53-cm deep (21-inch deep), 20 cm (8 in.) diameter hole was dug at a higher terrain location near the bridges, (2) A 76 cm-long (30 inch-long), ½ in-dia. rebar was placed in the hole, (3) Water was carefully added to concrete mix to obtain a desired strength of > 13,800 KPa (2,000 psi) for the benchmark concrete cylinder in the ground, (4) Concrete was poured in the hole around the benchmark rebar and it was tamped to expel the air bubbles, and (5) The top surface of the concrete was leveled and completed with a benchmark cap. **Table 39** shows the coordinates of installed benchmarks and control points used to survey the Kay County bridges in this study. The locations of these points were chosen so that they fall within 10 m (33 ft.) from the centerline of the road on either side because the right-of-way on most county roads is 20 m (66 ft.) wide (Simpson 2015). Also, **Figure 89** depicts the locations of the installed benchmarks which are labeled as BMXY for ease of reference where:

X = Bridge designation number ranging between 1 (southern) and 6 (northern)

Y = Benchmark designation number ranging between 1 (eastern) and 3 (western)

Table 39: Coordinates of benchmark used to survey Kay County bridges

	Bridge No. 1	
Type	North (Lat.)	East (Long.)
BM11	36°54.206'N	97°20.219'W
BM12	36°54.343'N	97°20.203'W

	Bridge No. 2	
Type	North (Lat.)	East (Long.)
BM21	36°54.374'N	97°20.224'W
BM22	36°54.382'N	97°20.218'W
BM23	36°54.343'N	97°20.206'W

	Bridge No. 3 and 4	
Type	North (Lat.)	East (Long.)
BM31	36°54.754'N	97°20.249'W
BM32	36°54.770'N	97°20.227'W
BM33	36°54.766'N	97°20.223'W

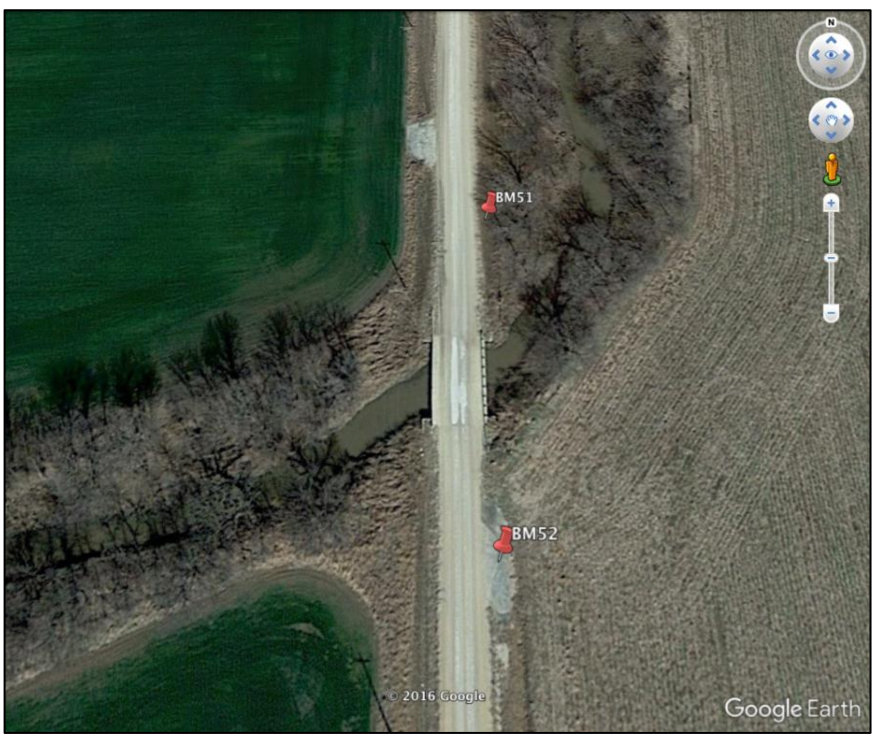
	Bridge No. 5	
Type	North (Lat.)	East (Long.)
BM51	36°54.862'N	97°20.211'W
BM52	36°54.817'N	97°20.209'W

	Bridge No. 6	
Type	North (Lat.)	East (Long.)
BM61	36°54.977'N	97°20.208'W
BM62	36°54.928'N	97°20.210'W

(a)

(b)


BM31
BM32
Fork Rd
BM33
N 44th St
© 2016 Google
Google Earth
(c)


BM51
BM52
© 2016 Google
Google Earth
(d)

(e)

Figure 89: Locations of the benchmarks: (a) Bridge No. 1; (b) Bridge No. 2; (c) Bridge No.3 and Bridge No. 4; (d) Bridge No. 5; (e) Bridge No. 6 (Google Earth 2016)

Similarly, for bridges 1,3,4,5 and 6 on the 44th street (i.e. with the exception of Bridge No. 4 which has an E-W alignment) the control points for surveying are labeled as SSXY, CCXY, NNXY where:

SS = Transverse South Axis (**Figure 90a**)

CC =Transverse Center Axis (**Figure 90b**)

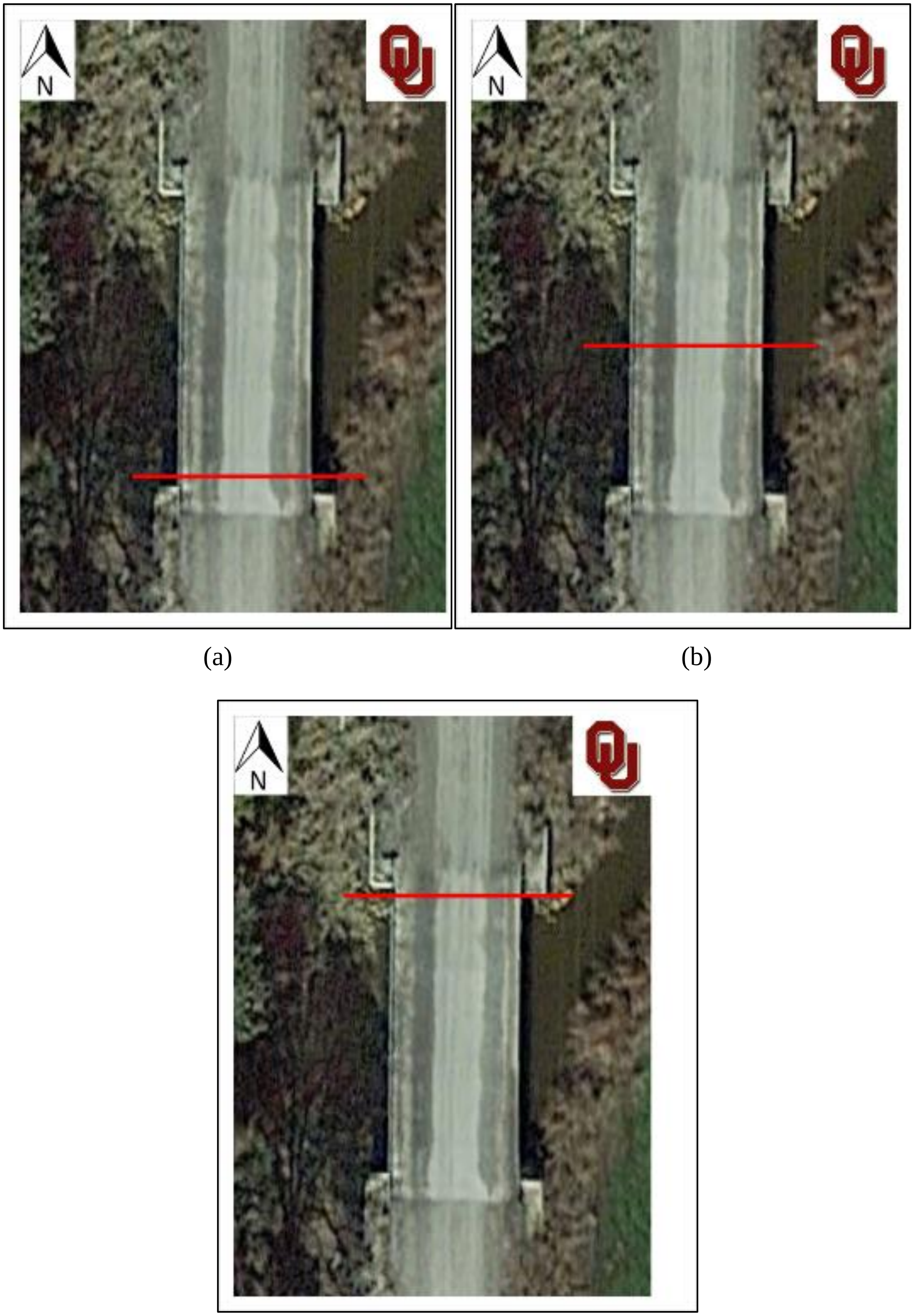
NN = Transverse North Axis (**Figure 90c**)

Xn = Bridge designation No. ranging between 1 and 6

Yn = Benchmark designation No. ranging between 1 (western) and 9 (eastern)

Note: Bridge No. 2 has Center South (CS) and Center North Axis(CN) instead of only having one center CC axis.

Figure 91 shows the marked control points for all bridges.



(a) (b)

(c)

Figure 90: Transverse axis for bridges 1,3,4,5 and 6 :(a) South Axis-SS; (b) Center Axis- CC; (c) North Axis -NN


(a)


(b)

(c)

(d)

(e)

(f)

Figure 91: Marked control points: (a) Bridge No. 1; (b) Bridge No. 2; (c) Bridge No.3; (d) Bridge No.4; (e) Bridge No. 5;(f) Bridge No.6

4.11 Survey Results for GRS-IBS bridges and Comparable Conventional Bridges in Kay County, OK.

4.11.1 Bridge No. 1

During one visit to the bridge on August 30, 2015, one set of settlement control points was surveyed. **Table 40** and **Figure 92** show the baseline results for Bridge No. 1.

Table 40: Coordinates of surveyed points on Bridge No. 1

(South Axis)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS11	998.535	1054.600	1.50	46.71
SS12	997.328	1054.253	2.76	48.11
SS13	996.346	1053.964	3.78	49.42
SS14	995.400	1053.701	4.76	50.22
SS15	994.301	1053.400	5.90	50.82
SS16	993.348	1053.128	6.89	50.72
SS17	992.339	1052.846	7.94	49.42
SS18	991.094	1052.493	9.23	48.41
SS19	990.119	1052.284	10.23	47.11

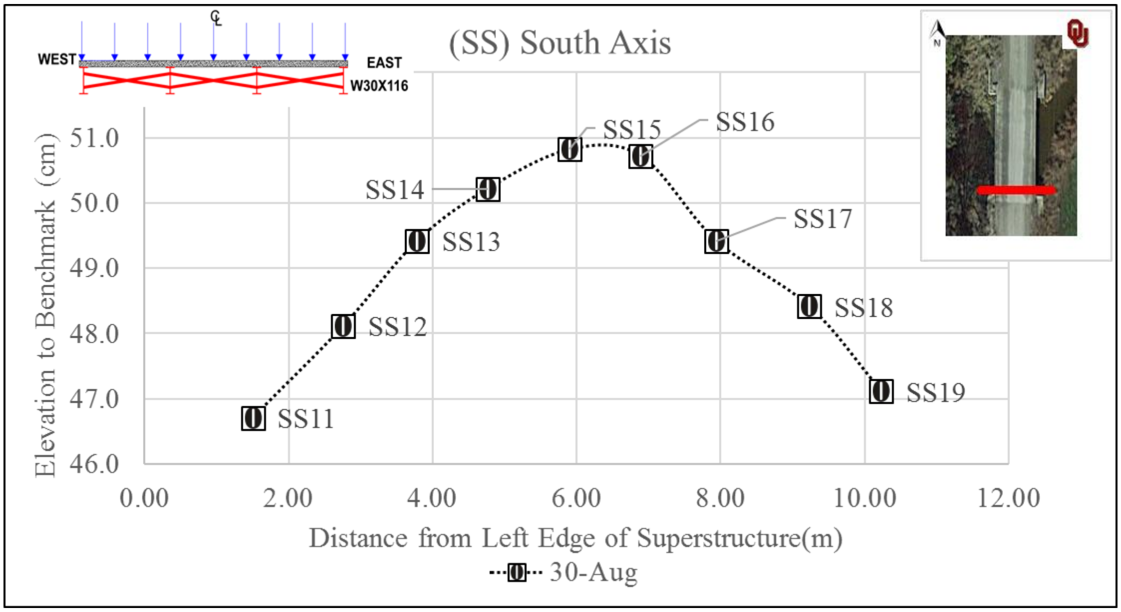
(Center Axis)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC11	1002.488	1040.331	1.50	46.41
CC12	1001.439	1040.007	2.60	47.81
CC13	1000.335	1039.720	3.74	48.31
CC14	999.250	1039.385	4.87	49.32
CC15	998.173	1039.093	5.99	50.42
CC16	997.247	1038.817	6.96	49.52
CC17	996.126	1038.495	8.12	48.41
CC18	995.116	1038.201	9.17	47.71
CC19	994.038	1037.876	10.30	46.31

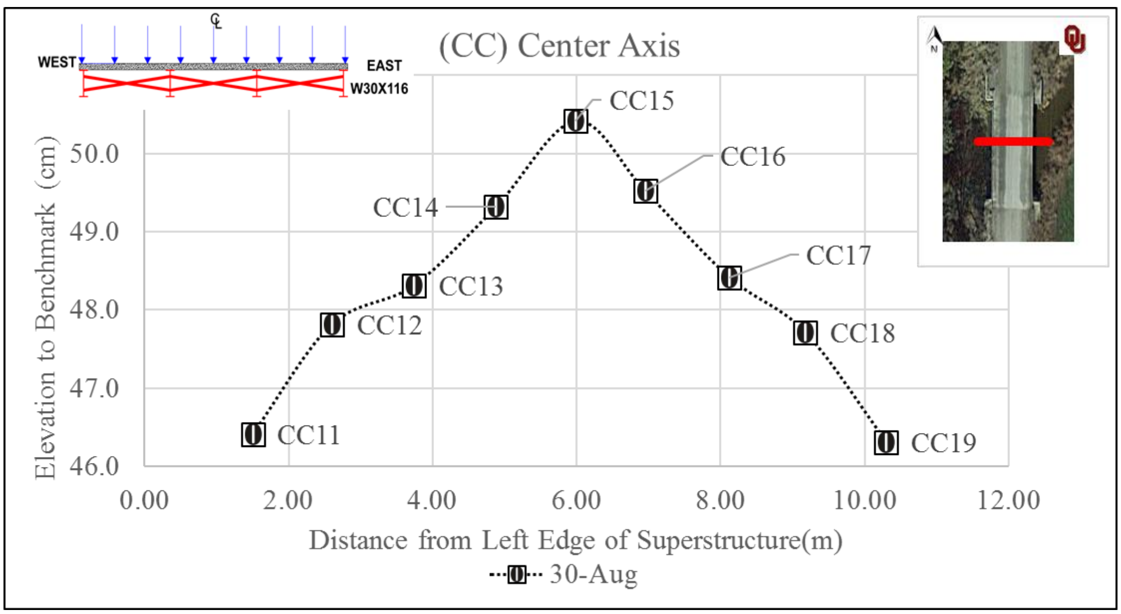
Note: Coordinates in m. and Elevation in cm.

(North Axis)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN11	1000.430	1047.378	1.50	44.91
NN12	999.333	1047.106	2.63	46.91
NN13	998.263	1046.779	3.75	48.11
NN14	997.191	1046.545	4.85	49.52
NN15	996.237	1046.236	5.85	49.32
NN16	995.259	1046.000	6.86	49.22
NN17	994.242	1045.705	7.91	48.92
NN18	993.165	1045.457	9.02	47.31
NN19	992.102	1045.158	10.12	45.91



(SS) South Axis
WEST
EAST
W30X116
SS11
SS12
SS13
SS14
SS15
SS16
SS17
SS18
SS19
Elevation to Benchmark (cm)
46.0
47.0
48.0
49.0
50.0
51.0
0.00
2.00
4.00
6.00
8.00
10.00
12.00
Distance from Left Edge of Superstructure(m)
30-Aug



(CC) Center Axis
WEST
EAST
W30X116
CC11
CC12
CC13
CC14
CC15
CC16
CC17
CC18
CC19
Elevation to Benchmark (cm)
46.0
47.0
48.0
49.0
50.0
0.00
2.00
4.00
6.00
8.00
10.00
12.00
Distance from Left Edge of Superstructure(m)
30-Aug

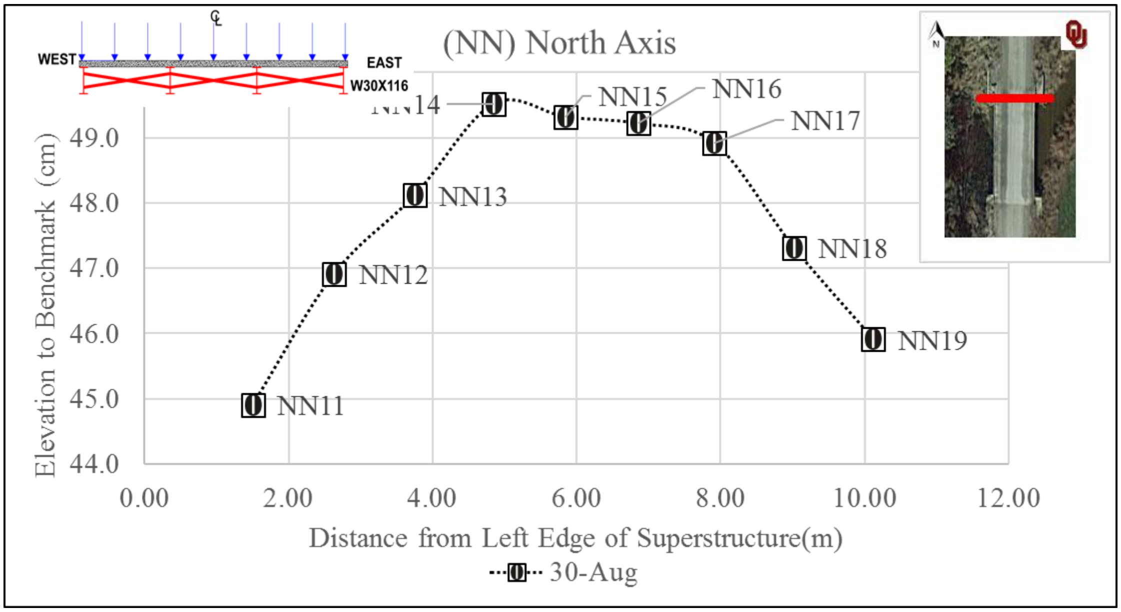


Figure 92: Coordinates of surveyed points on south, center and north axis of Bridge No.1 relative to BM11

4.11.2 Bridge No. 2

In seven (7) separate visits to the bridge sites during the period between May 2015 and January 2017 (specifically, on 05/18/15, 05/31/15, 06/08/15, 07/17/15, 08/30/15, 20/11/16 and 01/20/2017), seven (7) slightly different sets of settlement control points were surveyed for Bridge No. 2, which is the first bridge for which the survey points were set up. **Table 41** through **Table 44** and **Figure 93** through **Figure 96** show the survey results for Bridge No. 2 from these seven (7) separate visits. The results show that the accuracy of the elevation measurements is less than 10 mm (0.03 ft.). Aggregated survey data on the movements of the deck for Bridge No. 2 are shown in **Figure 93** through **Figure 96**, which indicate possible movements beyond the expected random variations in the survey data from different visits. To investigate the actual movements of the bridge deck more accurately, some measurements for each survey point was isolated and plotted separately as per the examples shown in **Figure 97**. The diagram on the left of each plot (**Figure 97**) shows the corresponding cross section of the bridge indicating the locations of its girders and survey points. The survey point specific to the data presented in each graph is shown with a larger arrow on the diagram. Different seasons during the surveying period are also marked on the graph for future analysis of any possible effects.

These results suggest that the bridge deck has undergone a seemingly consistent and predominantly upward movement between 6.35 mm to 12.7 mm (¼ and ½ in.) during this monitoring period. In addition, further analysis of our survey data was carried out for Bridge No. 2. **Figure 98** shows three-dimensional contour plots of the changes in the bridge deck elevation as seen from the south GRS abutment over the period of six and a half months between May and November 2015. The contours are plotted using the survey data from four (4) transverse axes (SS, CS, CN and NN), which are set up on the bridge as described in previous sections. An inset diagram at the bottom of each figure shows the corresponding cross section of the bridge indicating the locations of its girders and survey points (shown with downward arrows).

The results suggest that the bridge deck has undergone some differential settlement between the girders during this monitoring period. Also, an overall (albeit small) upward trend in the measured movements of the bridge deck might be due to a slight settlement of the benchmarks in the vicinity of the bridge as opposed to an actual heaving deformation of the abutments. However, the magnitudes of movements so far are within 5-15 mm, which are considered within the accuracy of the survey method used on these bridges. Further monitoring of the bridge movement in long term should help determine the validity and accuracy of this movement and its possible cause. Nevertheless, the survey results for Bridge No. 2 from the seven separate visits shown in **Figure 98** indicate that over the period of two years, there have not been significant deformations in the GRS bridge abutments in spite of severe weather conditions, and record rainfall and flooding in Spring 2015 (Ngo 2016; Hatami et al. 2016). Besides the survey results which point to fairly insignificant movements, the bridge has not shown any visible signs of serviceability or aesthetics-related problems since its construction in April 2014 either.

Table 41: Coordinates of surveyed points on south axis of Bridge No. 2

(1st set)

8-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3152.179	3383.294	164.47	2.58
SS22	3149.912	3380.666	164.64	2.69
SS23	3147.574	3378.363	165.14	2.73
SS24	3144.857	3376.098	166.03	2.76
SS25	3141.872	3373.481	167.02	2.76
SS26	3138.632	3370.556	168.14	2.75
SS27	3135.566	3367.777	169.30	2.71
SS28	3133.329	3366.029	170.34	2.69
SS29	3130.924	3363.888	171.38	2.62

(2nd set)

31-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3151.941	3383.195	164.59	2.58
SS22	3149.747	3380.572	164.72	2.68
SS23	3147.424	3378.271	165.21	2.73
SS24	3144.668	3375.944	166.10	2.77
SS25	3141.686	3373.363	167.11	2.78
SS26	3138.497	3370.467	168.21	2.76
SS27	3135.426	3367.648	169.35	2.73
SS28	3133.143	3365.936	170.46	2.71
SS29	3130.755	3363.735	171.46	2.64

(3rd set)

8-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3151.959	3383.084	164.51	2.59
SS22	3149.682	3380.496	164.72	2.69
SS23	3147.389	3378.222	165.20	2.72
SS24	3144.600	3375.860	166.10	2.77
SS25	3141.670	3373.314	167.09	2.77
SS26	3138.412	3370.387	168.24	2.76
SS27	3135.358	3367.543	169.36	2.72
SS28	3133.146	3365.824	170.40	2.69
SS29	3130.771	3363.652	171.40	2.66

(4th set)

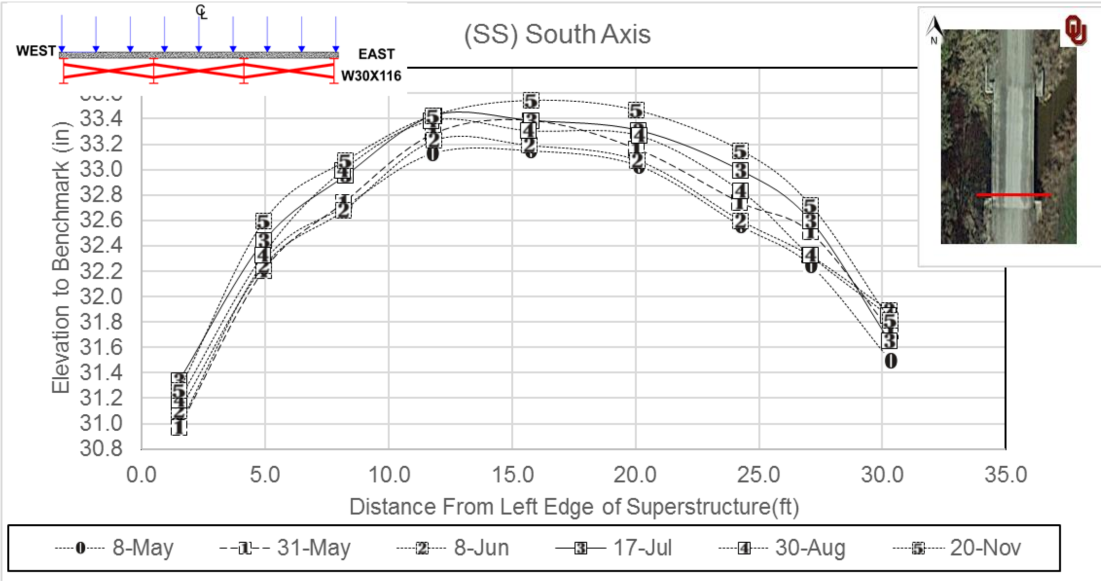
17-Jul				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3152.008	3383.169	164.53	2.61
SS22	3149.715	3380.594	164.76	2.70
SS23	3147.356	3378.297	165.28	2.75
SS24	3144.616	3375.975	166.16	2.79
SS25	3141.624	3373.415	167.19	2.78
SS26	3138.422	3370.453	168.27	2.78
SS27	3135.394	3367.667	169.39	2.75
SS28	3133.153	3365.932	170.45	2.72
SS29	3130.784	3363.799	171.46	2.64

(5th set)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3152.060	3383.183	164.49	2.60
SS22	3149.784	3380.673	164.75	2.69
SS23	3147.517	3378.360	165.18	2.75
SS24	3144.744	3376.073	166.11	2.78
SS25	3141.723	3373.527	167.17	2.78
SS26	3138.455	3370.482	168.25	2.77
SS27	3135.407	3367.736	169.42	2.74
SS28	3133.196	3365.965	170.43	2.69
SS29	3130.824	3363.862	171.46	2.65

(6th set)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS21	3152.057	3383.242	164.53	2.60
SS22	3149.793	3380.633	164.72	2.72
SS23	3147.405	3378.347	165.26	2.76
SS24	3144.702	3376.057	166.13	2.79
SS25	3141.690	3373.471	167.16	2.80
SS26	3138.553	3370.568	168.22	2.79
SS27	3135.430	3367.746	169.40	2.76
SS28	3133.189	3366.007	170.45	2.73
SS29	3130.817	3363.832	171.45	2.65



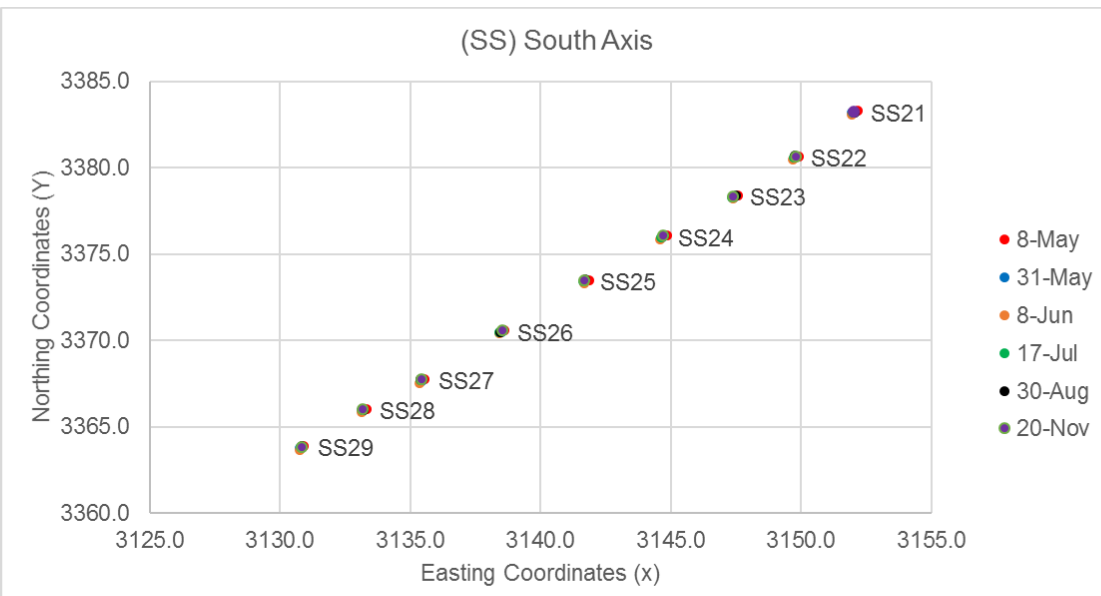


Figure 93: Coordinates of surveyed points on south axis of Bridge No.2 relative to BM22

Table 42: Coordinates of surveyed points on south center axis of Bridge No. 2

(1^{st} set)

8-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3169.006	3364.734	139.80	2.55
CS22	3166.642	3362.508	140.40	2.64
CS23	3164.223	3360.331	141.13	2.70
CS24	3161.708	3357.663	141.75	2.74
CS25	3158.727	3355.376	143.06	2.73
CS26	3155.555	3352.759	144.46	2.70
CS27	3152.613	3350.038	145.71	2.64
CS28	3149.998	3347.656	146.91	2.59
CS29	3148.015	3345.768	147.84	2.53

(2^{nd} set)

31-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3168.861	3364.638	139.86	2.55
CS22	3166.489	3362.434	140.48	2.65
CS23	3164.132	3360.223	141.15	2.71
CS24	3161.469	3357.637	141.94	2.75
CS25	3158.602	3355.210	143.08	2.76
CS26	3155.374	3352.684	144.58	2.73
CS27	3152.471	3349.964	145.80	2.67
CS28	3149.868	3347.549	146.98	2.61
CS29	3147.846	3345.661	147.95	2.55

(3^{rd} set)

8-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3168.901	3364.606	139.81	2.53
CS22	3166.440	3362.369	140.48	2.64
CS23	3164.078	3360.095	141.12	2.72
CS24	3161.463	3357.559	141.90	2.75
CS25	3158.596	3355.141	143.05	2.74
CS26	3155.394	3352.612	144.53	2.73
CS27	3152.471	3349.862	145.75	2.67
CS28	3149.859	3347.444	146.94	2.60
CS29	3147.864	3345.587	147.90	2.54

(4^{th} set)

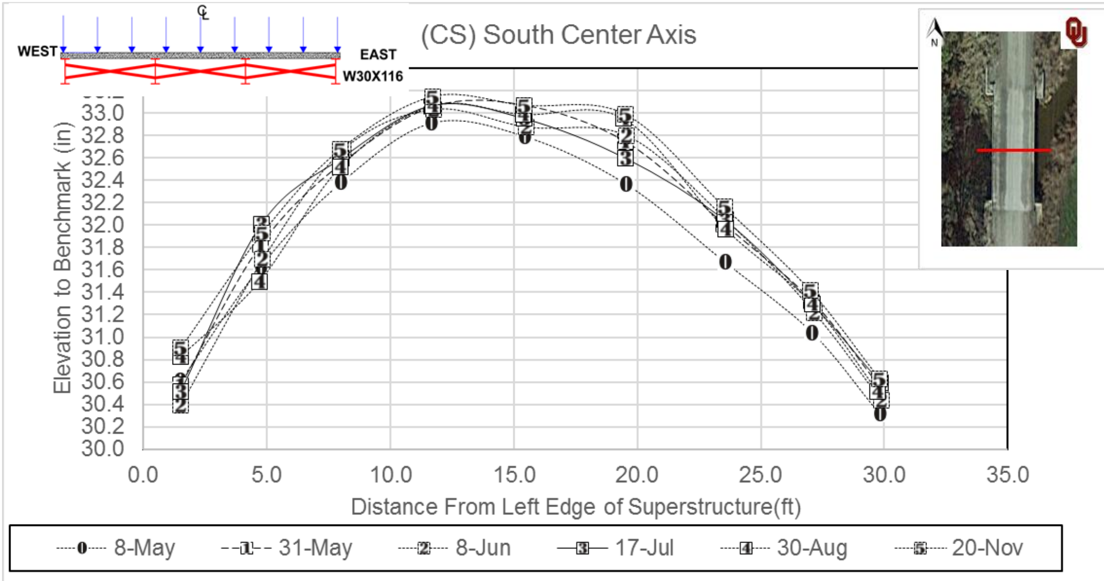
17-Jul				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3168.891	3364.649	139.84	2.54
CS22	3166.463	3362.464	140.52	2.67
CS23	3164.104	3360.240	141.18	2.72
CS24	3161.447	3357.677	141.98	2.76
CS25	3158.602	3355.269	143.11	2.75
CS26	3155.404	3352.684	144.55	2.72
CS27	3152.464	3349.964	145.80	2.67
CS28	3149.856	3347.576	147.01	2.61
CS29	3147.808	3345.689	148.00	2.55

(5^{th} set)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3168.871	3364.728	139.91	2.57
CS22	3166.565	3362.530	140.47	2.62
CS23	3164.137	3360.328	141.20	2.71
CS24	3161.473	3357.710	141.98	2.76
CS25	3158.645	3355.341	143.12	2.75
CS26	3155.469	3352.795	144.55	2.75
CS27	3152.425	3350.036	145.87	2.66
CS28	3149.862	3347.631	147.02	2.61
CS29	3147.966	3345.765	147.89	2.54

(6^{th} set)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
CS21	3168.891	3364.751	139.91	2.58
CS22	3166.470	3362.503	140.53	2.66
CS23	3164.154	3360.358	141.20	2.72
CS24	3161.431	3357.782	142.05	2.76
CS25	3158.557	3355.410	143.23	2.76
CS26	3155.384	3352.848	144.65	2.75
CS27	3152.510	3350.053	145.80	2.68
CS28	3149.954	3347.658	146.95	2.62
CS29	3147.881	3345.781	147.97	2.55



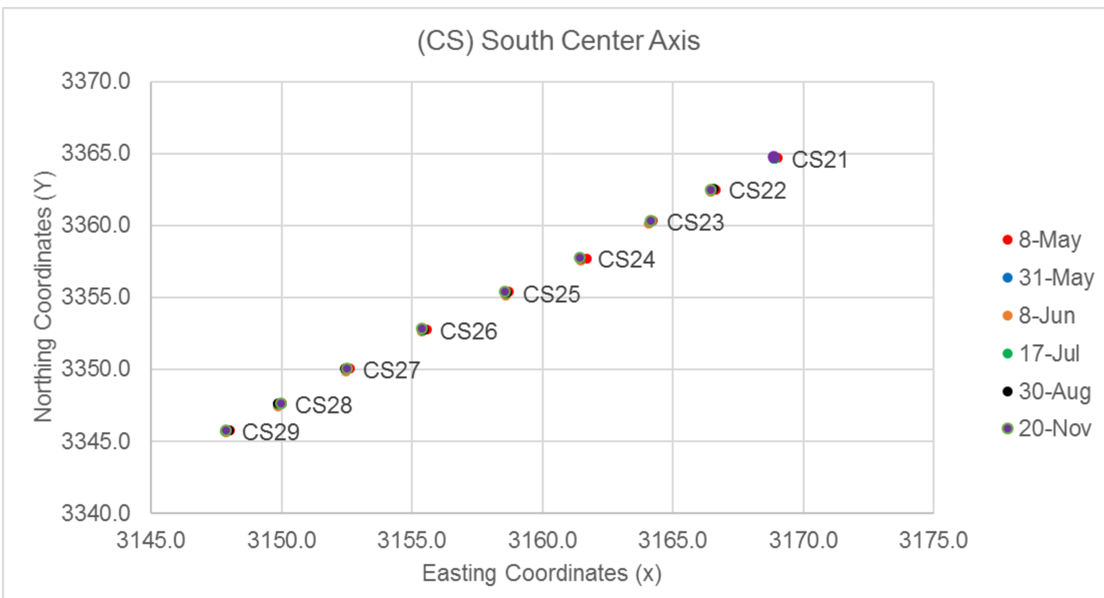


Figure 94: Coordinates of surveyed points on south center axis of Bridge No.2 relative to BM22

Table 43: Coordinates of surveyed points on north center axis of Bridge No. 2

(1st set)

8-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.156	3360.187	133.76	2.54
CN22	3170.797	3358.071	134.44	2.63
CN23	3168.407	3355.758	135.11	2.70
CN24	3165.605	3353.419	136.19	2.73
CN25	3162.717	3350.617	137.19	2.73
CN26	3159.410	3347.991	138.76	2.70
CN27	3156.370	3345.210	140.13	2.64
CN28	3153.937	3342.886	141.26	2.57
CN29	3151.982	3341.037	142.23	2.51

(2nd set)

31-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.044	3360.121	133.81	2.55
CN22	3170.600	3358.032	134.58	2.65
CN23	3168.248	3355.708	135.21	2.70
CN24	3165.461	3353.367	136.28	2.75
CN25	3162.566	3350.563	137.30	2.74
CN26	3159.334	3347.872	138.77	2.72
CN27	3156.222	3345.117	140.22	2.65
CN28	3153.783	3342.806	141.36	2.60
CN29	3151.847	3340.934	142.30	2.54

(3rd set)

8-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.009	3360.043	133.79	2.54
CN22	3170.594	3357.946	134.53	2.63
CN23	3168.228	3355.594	135.16	2.71
CN24	3165.413	3353.222	136.24	2.74
CN25	3162.559	3350.433	137.24	2.74
CN26	3159.285	3347.749	138.75	2.70
CN27	3156.230	3345.017	140.16	2.65
CN28	3153.750	3342.723	141.36	2.58
CN29	3151.831	3340.866	142.29	2.53

(4th set)

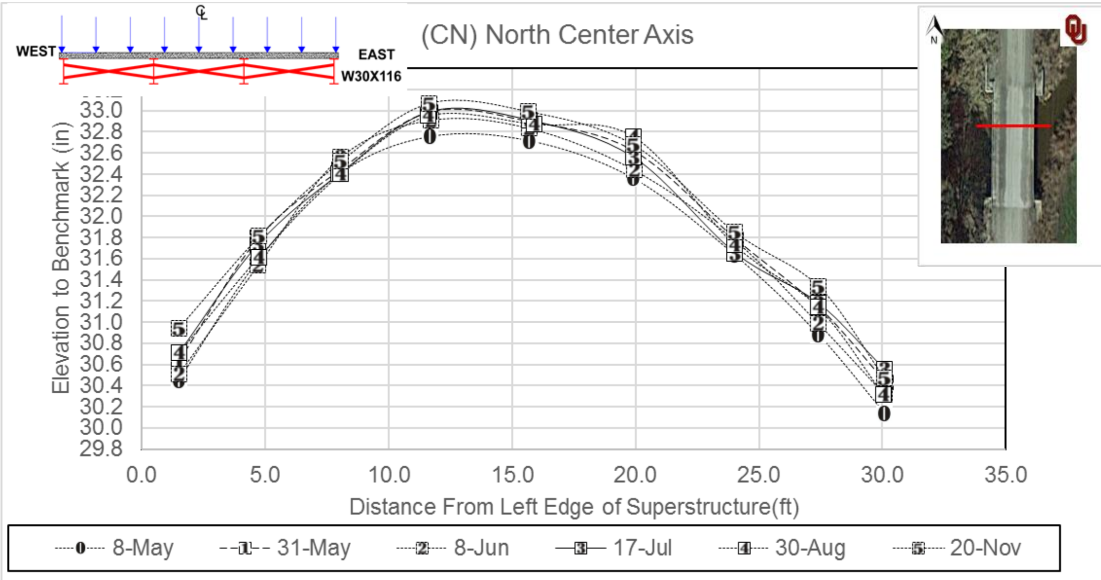
17-Jul				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.018	3360.115	133.83	2.56
CN22	3170.604	3358.041	134.58	2.64
CN23	3168.327	3355.656	135.12	2.70
CN24	3165.482	3353.333	136.24	2.75
CN25	3162.576	3350.538	137.27	2.74
CN26	3159.265	3347.900	138.84	2.71
CN27	3156.227	3345.138	140.22	2.64
CN28	3153.796	3342.818	141.36	2.60
CN29	3151.883	3340.938	142.27	2.55

(5th set)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.110	3360.158	133.78	2.56
CN22	3170.650	3358.068	134.56	2.63
CN23	3168.242	3355.778	135.26	2.70
CN24	3165.548	3353.442	136.25	2.75
CN25	3162.329	3350.617	137.53	2.74
CN26	3159.314	3347.946	138.82	2.73
CN27	3156.283	3345.204	140.20	2.64
CN28	3153.812	3342.884	141.37	2.60
CN29	3151.877	3341.066	142.33	2.53

(6th set)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
CN21	3173.045	3360.210	133.86	2.58
CN22	3170.604	3358.068	134.60	2.65
CN23	3168.265	3355.738	135.21	2.71
CN24	3165.496	3353.435	136.29	2.76
CN25	3162.605	3350.660	137.31	2.75
CN26	3159.314	3347.966	138.83	2.72
CN27	3156.198	3345.295	140.32	2.65
CN28	3153.786	3342.907	141.40	2.61
CN29	3151.857	3341.076	142.36	2.54



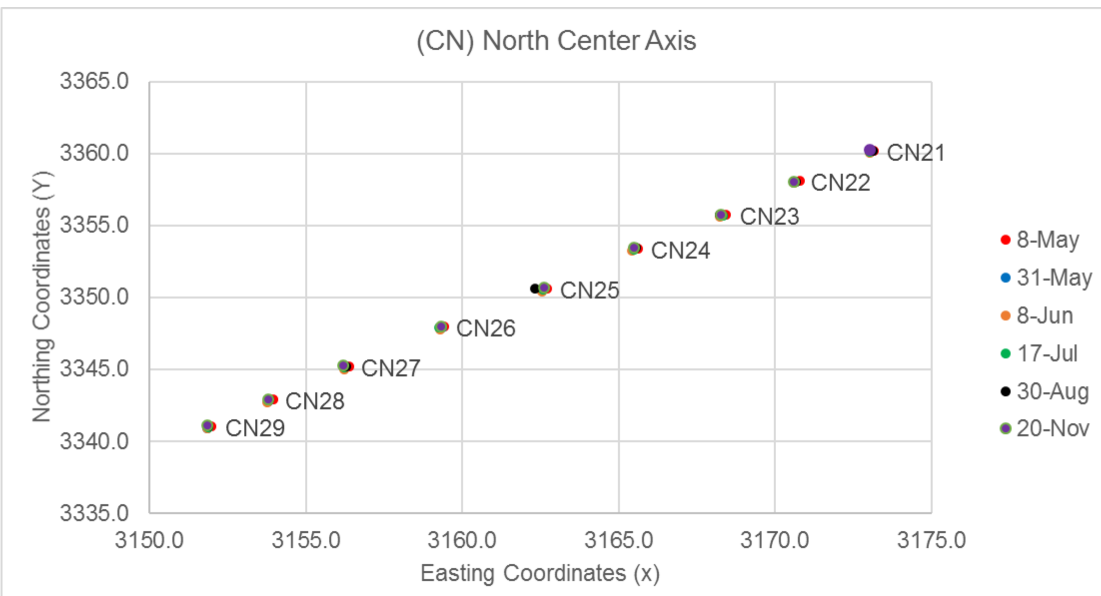


Figure 95: Coordinates of surveyed points on north center axis of Bridge No.2 relative to BM22

Table 44: Coordinates of surveyed points on north axis of bridge No. 2

(1st set)

8-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3190.133	3342.003	109.40	2.59
NN22	3187.992	3340.003	110.10	2.62
NN23	3185.254	3337.582	111.16	2.68
NN24	3182.348	3334.879	112.34	2.71
NN25	3179.605	3332.507	113.66	2.70
NN26	3176.622	3329.784	115.14	2.68
NN27	3173.998	3327.464	116.57	2.64
NN28	3171.903	3325.432	117.71	2.60
NN29	3169.321	3322.891	119.18	2.54

(2nd set)

31-May				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3190.000	3341.962	109.49	2.60
NN22	3187.858	3339.972	110.19	2.64
NN23	3185.178	3337.541	111.20	2.70
NN24	3182.302	3334.834	112.36	2.71
NN25	3179.513	3332.418	113.70	2.71
NN26	3176.511	3329.711	115.21	2.68
NN27	3173.890	3327.355	116.63	2.66
NN28	3171.787	3325.361	117.79	2.61
NN29	3169.228	3322.798	119.24	2.56

(3rd set)

8-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3189.974	3341.857	109.45	2.60
NN22	3187.851	3339.830	110.12	2.64
NN23	3185.174	3337.385	111.13	2.70
NN24	3182.277	3334.787	112.36	2.70
NN25	3179.498	3332.280	113.65	2.70
NN26	3176.476	3329.554	115.17	2.68
NN27	3173.855	3327.257	116.62	2.65
NN28	3171.893	3325.256	117.65	2.60
NN29	3169.242	3322.730	119.20	2.56

(4th set)

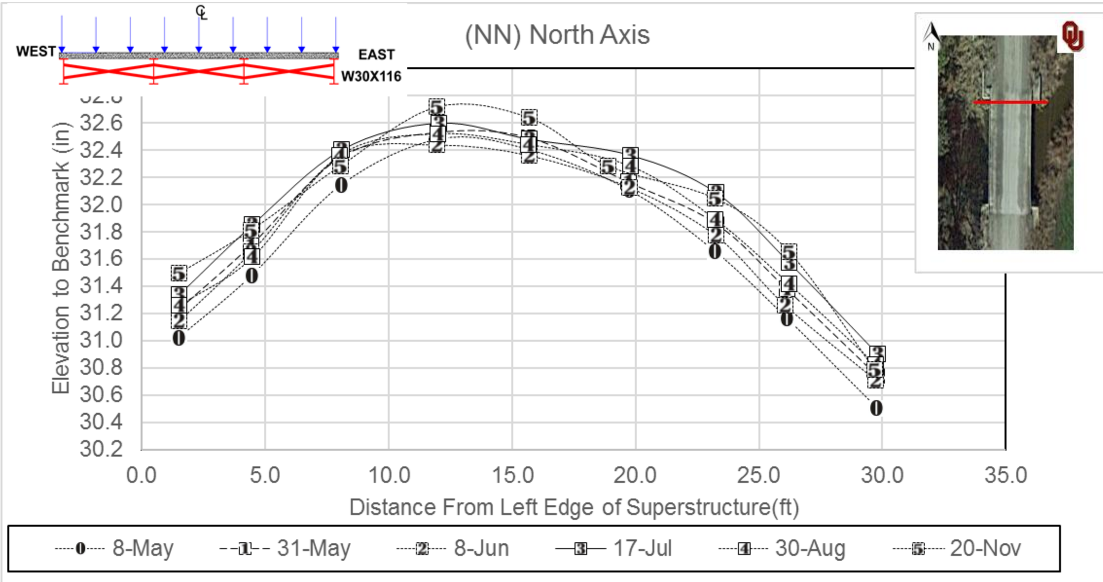
17-Jul				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3190.020	3341.969	109.48	2.61
NN22	3187.854	3339.948	110.18	2.65
NN23	3185.144	3337.546	111.23	2.70
NN24	3182.254	3334.872	112.42	2.72
NN25	3179.544	3332.408	113.67	2.71
NN26	3176.473	3329.679	115.23	2.70
NN27	3173.911	3327.343	116.60	2.67
NN28	3171.759	3325.325	117.80	2.63
NN29	3169.206	3322.815	119.26	2.58

(5th set)

30-Aug				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3190.023	3341.969	109.47	2.60
NN22	3187.831	3339.954	110.20	2.63
NN23	3185.220	3337.576	111.19	2.70
NN24	3182.284	3334.905	112.41	2.71
NN25	3179.538	3332.428	113.68	2.70
NN26	3176.421	3329.754	115.31	2.69
NN27	3173.858	3327.428	116.69	2.66
NN28	3171.729	3325.338	117.84	2.62
NN29	3169.242	3322.884	119.26	2.57

(6^{th} set)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN21	3189.944	3342.044	109.58	2.62
NN22	3187.838	3340.033	110.24	2.65
NN23	3185.154	3337.595	111.25	2.69
NN24	3182.280	3334.948	112.44	2.73
NN25	3179.534	3332.389	113.67	2.72
NN26	3177.165	3330.243	114.84	2.69
NN27	3173.898	3327.425	116.65	2.67
NN28	3171.680	3325.384	117.90	2.64
NN29	3169.223	3322.920	119.29	2.57



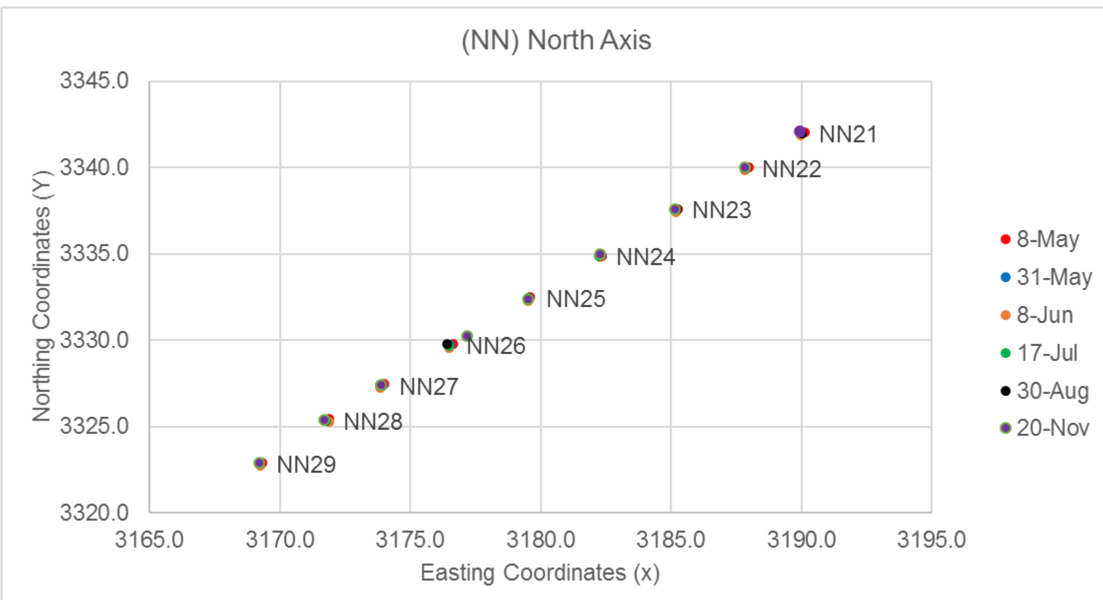
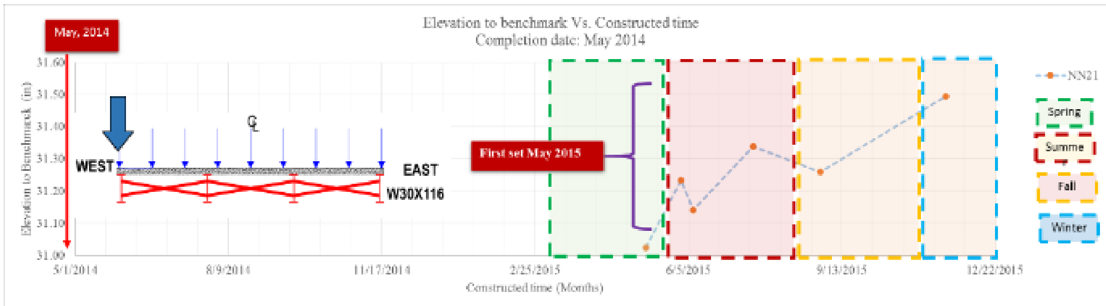
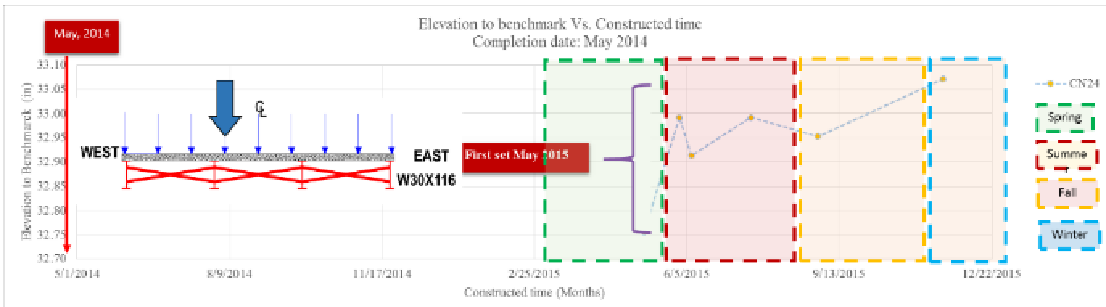


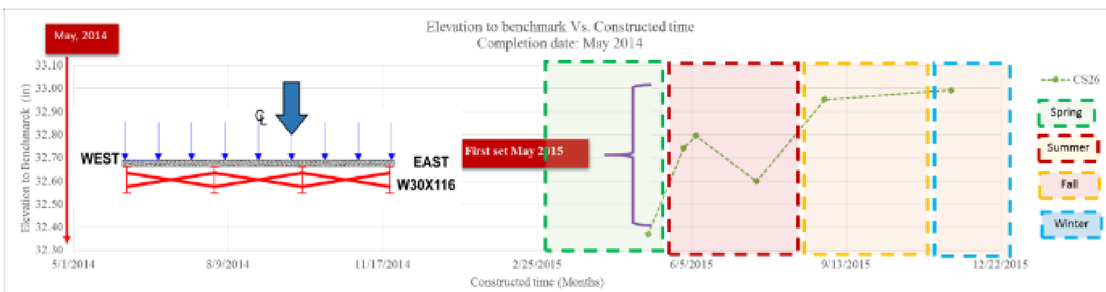
Figure 96: Coordinates of surveyed points on north axis of Bridge No. 2 relative to BM22



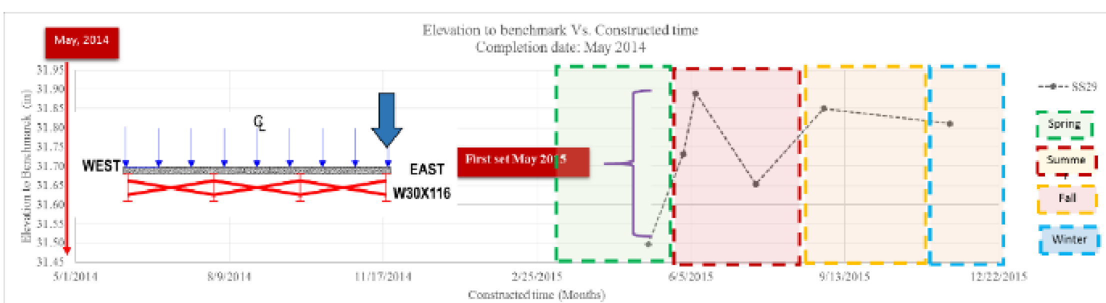
North end



North center

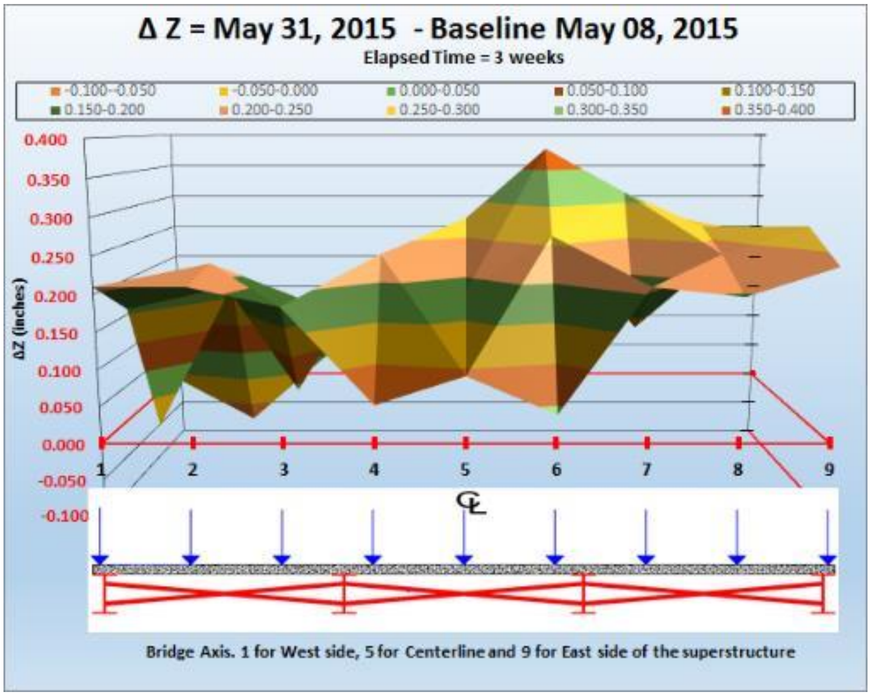


South center

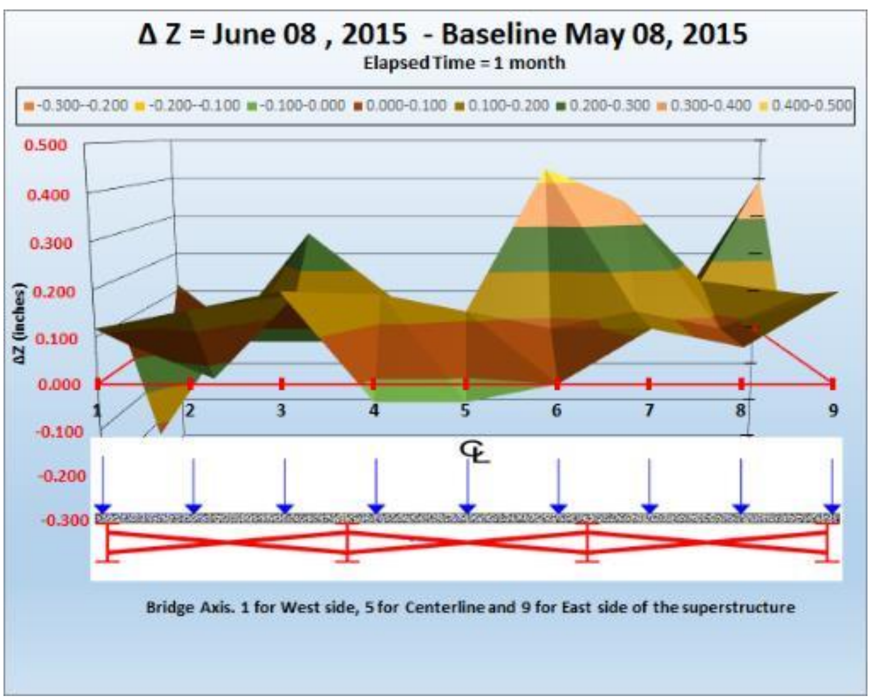


South end.

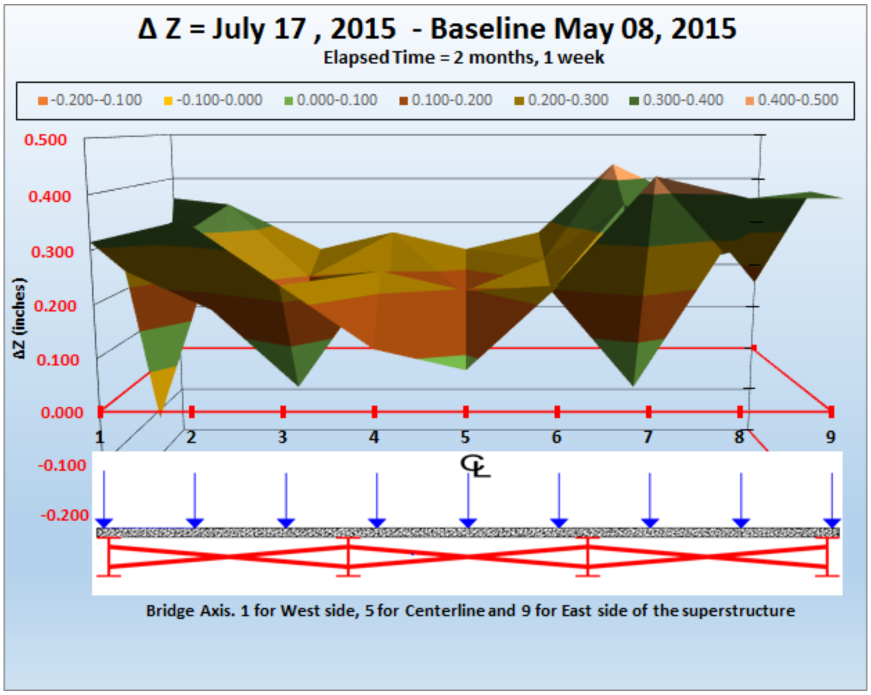
Figure 97: Vertical movements of Bridge No. 2 superstructure based on the survey of its two ends and the center over a 7-month period:


Δ Z = May 31, 2015 - Baseline May 08, 2015
Elapsed Time = 3 weeks
-0.100--0.050
-0.050-0.000
0.000-0.050
0.050-0.100
0.100-0.150
0.150-0.200
0.200-0.250
0.250-0.300
0.300-0.350
0.350-0.400
ΔZ (inches)
0.400
0.350
0.300
0.250
0.200
0.150
0.100
0.050
0.000
-0.050
-0.100
1
2
3
4
5
6
7
8
9
Bridge Axis. 1 for West side, 5 for Centerline and 9 for East side of the superstructure

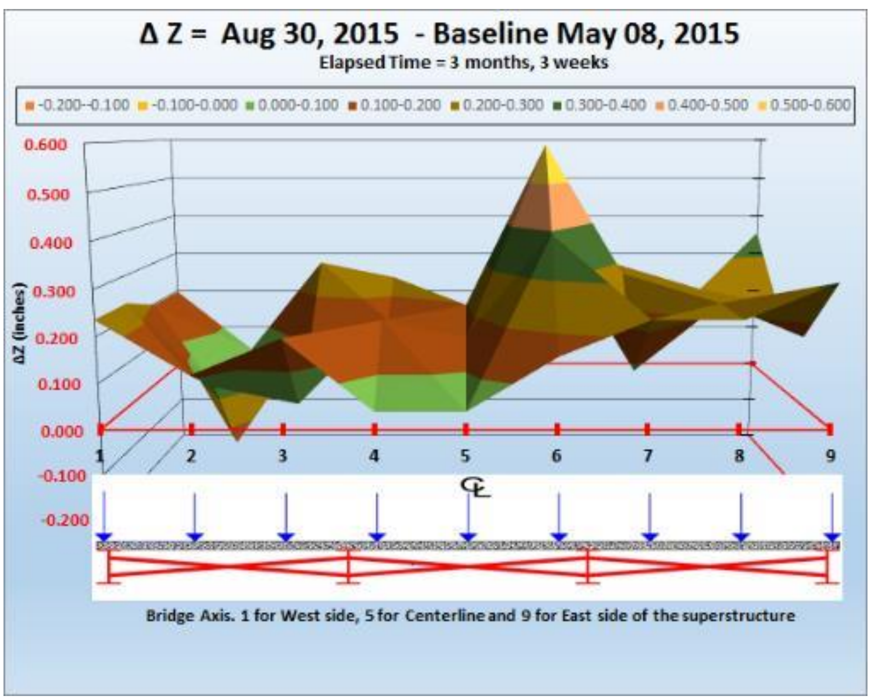
(a)


Δ Z = June 08 , 2015 - Baseline May 08, 2015
Elapsed Time = 1 month
-0.300--0.200
-0.200--0.100
-0.100-0.000
0.000-0.100
0.100-0.200
0.200-0.300
0.300-0.400
0.400-0.500
ΔZ (inches)
0.500
0.400
0.300
0.200
0.100
0.000
-0.100
-0.200
-0.300
1
2
3
4
5
6
7
8
9
Bridge Axis. 1 for West side, 5 for Centerline and 9 for East side of the superstructure

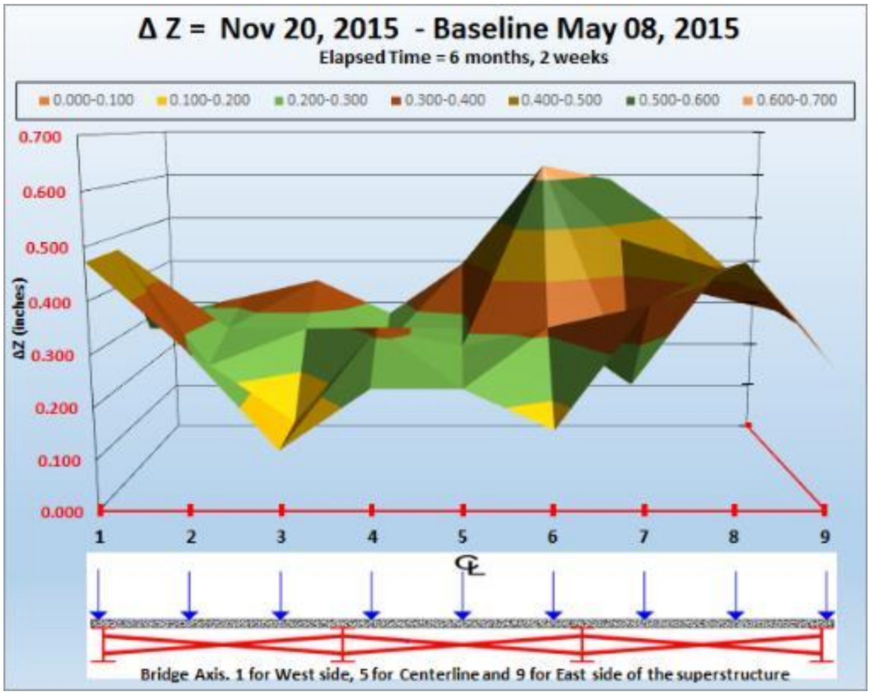
(b)


Δ Z = July 17 , 2015 - Baseline May 08, 2015
Elapsed Time = 2 months, 1 week
-0.200--0.100 -0.100-0.000 0.000-0.100 0.100-0.200 0.200-0.300 0.300-0.400 0.400-0.500
ΔZ (inches)
0.500
0.400
0.300
0.200
0.100
0.000
-0.100
-0.200
1 2 3 4 5 6 7 8 9
Bridge Axis. 1 for West side, 5 for Centerline and 9 for East side of the superstructure

(c)


Δ Z = Aug 30, 2015 - Baseline May 08, 2015
Elapsed Time = 3 months, 3 weeks
-0.200--0.100 -0.100-0.000 0.000-0.100 0.100-0.200 0.200-0.300 0.300-0.400 0.400-0.500 0.500-0.600
ΔZ (inches)
0.600
0.500
0.400
0.300
0.200
0.100
0.000
-0.100
-0.200
1 2 3 4 5 6 7 8 9
Bridge Axis. 1 for West side, 5 for Centerline and 9 for East side of the superstructure

(d)



(e)

Figure 98: Differential vertical movements (ΔZ) of Bridge No. 2 superstructure based on surveyed data of its mid-span and abutment ends over a 6½ month period: (a) – (e)

Note: The diagram on the bottom of each plot shows the corresponding cross section of the bridge indicating the locations of its girders and survey points.

4.11.3 Bridge No. 3

During two (2) separate visits to the bridge sites over the last six months (i.e. 08/30/2015, 10/24/2015, 01/29/2015), two different surveys were carried out on Bridge No. 3 **Table 45** and **Figure 99** show the results for Bridge No. 3.

Table 45: Coordinates of surveyed points on Bridge No. 3

(South Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS31	1061.023	1000.096	0.50	94.02
SS32	1061.451	1001.073	1.57	95.12
SS33	1061.931	1002.167	2.76	96.03
SS34	1062.449	1003.281	3.99	96.73
SS35	1062.934	1004.389	5.20	96.93
SS36	1063.430	1005.434	6.36	96.13
SS37	1063.936	1006.554	7.59	95.42
SS38	1064.440	1007.671	8.81	94.32
SS39	1064.884	1008.611	9.85	92.62

(South Axis) 2nd Set

29-Jan				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS31	1061.023	1000.096	0.50	93.42
SS32	1061.451	1001.073	1.57	93.52
SS33	1061.931	1002.167	2.76	95.22
SS34	1062.449	1003.281	3.99	95.53
SS35	1062.934	1004.389	5.20	95.42
SS36	1063.430	1005.434	6.36	95.22
SS37	1063.936	1006.554	7.59	93.92
SS38	1064.440	1007.671	8.81	93.22
SS39	1064.884	1008.611	9.85	91.52

Note: Coordinates in m. and Elevation in cm.

(Center Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC31	1000.651	1052.314	0.50	92.02
CC32	1001.493	1052.758	1.45	93.12
CC33	1002.589	1053.298	2.67	94.42
CC34	1003.697	1053.844	3.91	95.32
CC35	1004.779	1054.391	5.12	95.63
CC36	1005.993	1054.925	6.45	95.53
CC37	1007.089	1055.481	7.68	95.32
CC38	1008.194	1056.045	8.92	93.82
CC39	1009.090	1056.485	9.92	92.32

(Center Axis) 2nd Set

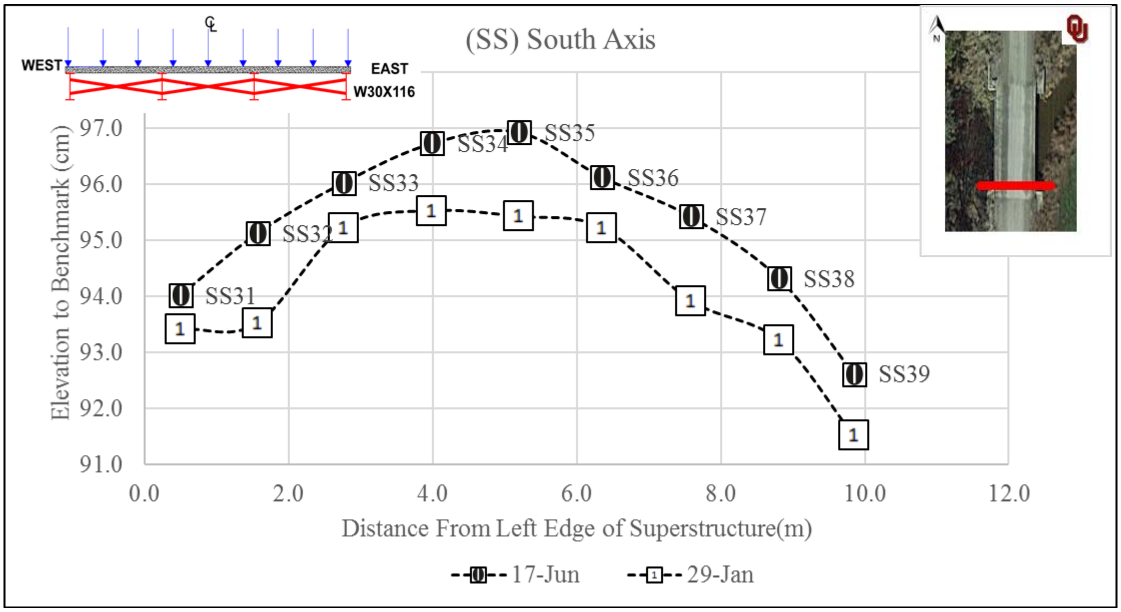
29-Jan				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC31	1000.651	1052.314	0.50	90.91
CC32	1001.493	1052.758	1.45	91.92
CC33	1002.589	1053.298	2.67	93.72
CC34	1003.697	1053.844	3.91	94.62
CC35	1004.779	1054.391	5.12	94.62
CC36	1005.993	1054.925	6.45	94.82
CC37	1007.089	1055.481	7.68	94.82
CC38	1008.194	1056.045	8.92	92.52
CC39	1009.090	1056.485	9.92	90.91

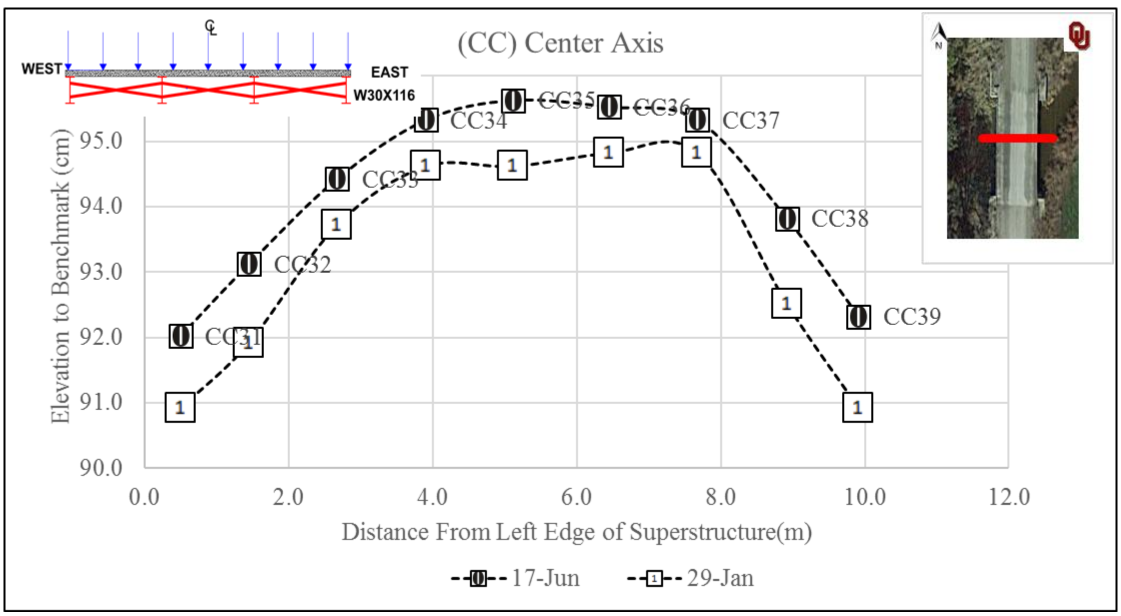
(North Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN31	1043.834	1001.183	0.50	92.62
NN32	1044.302	1001.996	1.44	93.92
NN33	1044.793	1003.155	2.70	94.52
NN34	1045.330	1004.227	3.90	95.42
NN35	1045.860	1005.309	5.10	95.83
NN36	1046.342	1006.457	6.35	94.82
NN37	1046.844	1007.593	7.59	94.52
NN38	1047.313	1008.718	8.81	93.42
NN39	1047.732	1009.672	9.85	92.52

(North Axis) 2nd Set

N/A


(SS) South Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance From Left Edge of Superstructure(m)
SS31
SS32
SS33
SS34
SS35
SS36
SS37
SS38
SS39
17-Jun
29-Jan


(CC) Center Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance From Left Edge of Superstructure(m)
CC31
CC32
CC33
CC34
CC35
CC36
CC37
CC38
CC39
17-Jun
29-Jan

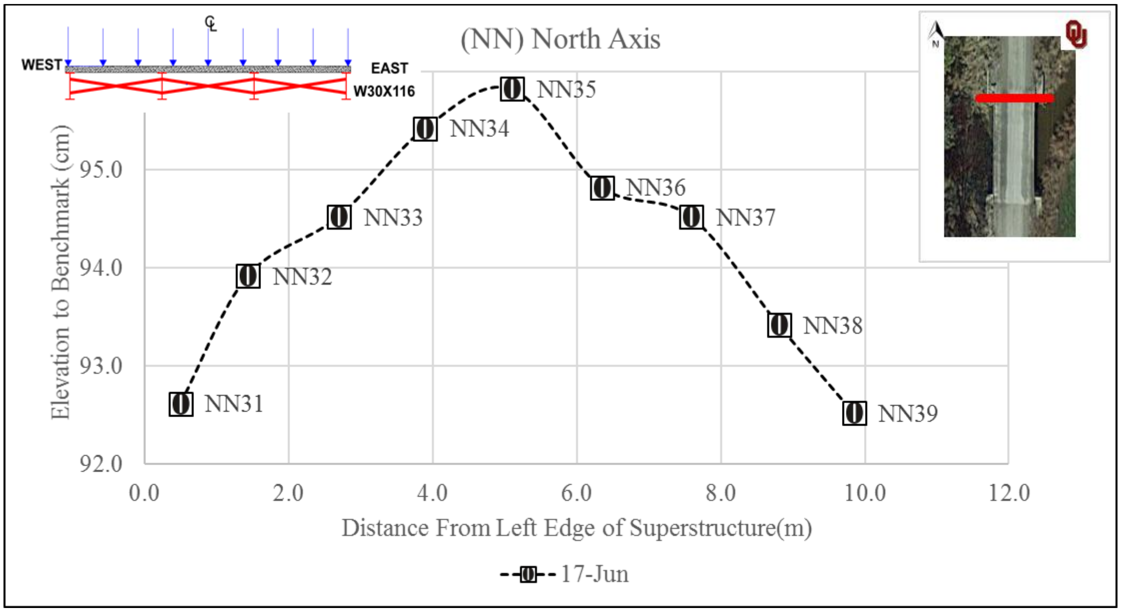


Figure 99: Coordinates of surveyed points on south, center and north axis of Bridge No.3 relative to BM31

4.11.4 Bridge No. 4

During three separate visits to the bridge sites over the last six months (i.e. 08/30/2015, 10/24/2015, 01/29/2016), two different surveys were carried out on Bridge No. 4, **Table 46** and **Figure 100** show the results for Bridge No. 4.

Table 46: Coordinates of surveyed points on Bridge No. 4

(South Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS41	1001.313	968.980	1.50	86.60
SS42	1002.308	968.933	2.50	88.01
SS43	1003.347	968.872	3.54	89.31
SS44	1004.426	968.811	4.62	90.61
SS45	1005.593	968.772	5.79	92.42
SS46	1006.814	968.720	7.01	92.22
SS47	1007.889	968.658	8.08	91.32
SS48	1008.961	968.588	9.16	90.21
SS49	1009.963	968.534	10.16	89.21

(South Axis) 2nd Set

29-Jan				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS41	1001.354	968.996	1.50	85.90
SS42	1002.353	968.946	2.50	86.80
SS43	1003.382	968.882	3.53	88.41
SS44	1004.466	968.827	4.62	89.61
SS45	1005.629	968.784	5.78	91.92
SS46	1006.829	968.736	6.98	91.82
SS47	1007.925	968.666	8.08	90.91
SS48	1008.985	968.591	9.14	89.71
SS49	1009.985	968.559	10.14	88.71

Note: Coordinates in m. and Elevation in cm.

(Center Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC41	1001.786	977.470	1.50	85.90
CC42	1002.763	977.386	2.48	87.81
CC43	1003.832	977.279	3.55	89.41
CC44	1004.884	977.177	4.61	90.61
CC45	1006.073	977.081	5.80	91.72
CC46	1007.320	976.982	7.06	91.01
CC47	1008.372	976.902	8.11	89.51
CC48	1009.428	976.845	9.17	88.31
CC49	1010.401	976.754	10.15	86.50

(Center Axis) 2nd Set

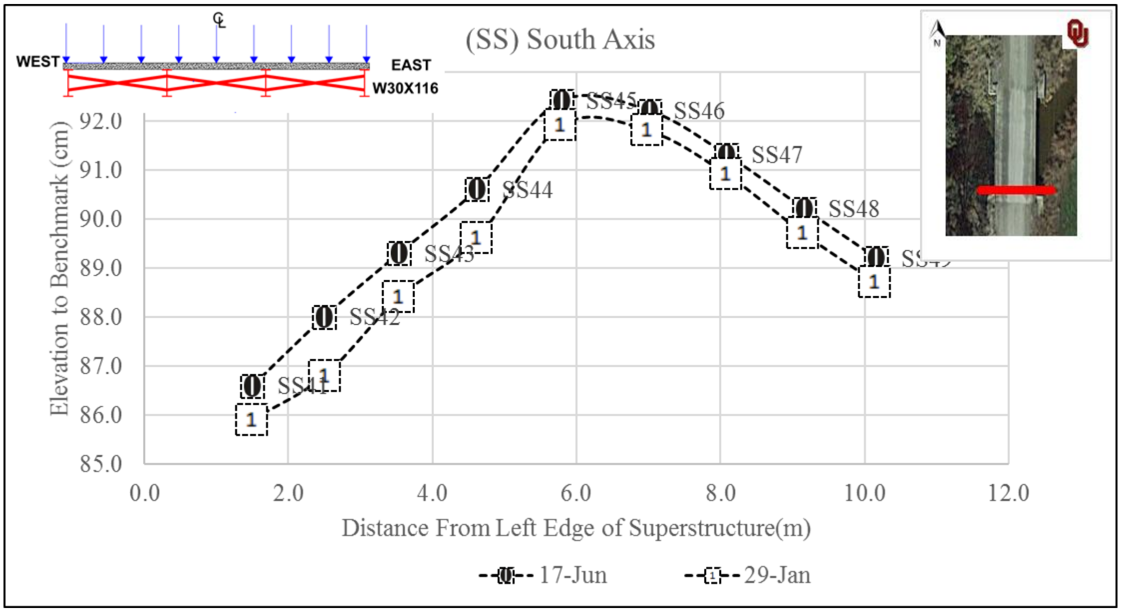
29-Jan				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC41	1001.823	977.479	1.50	85.50
CC42	1002.797	977.397	2.48	87.41
CC43	1003.868	977.291	3.55	88.61
CC44	1004.903	977.204	4.59	89.91
CC45	1006.108	977.102	5.80	90.91
CC46	1007.363	977.008	7.06	90.61
CC47	1008.396	976.947	8.09	88.91
CC48	1009.494	976.847	9.20	87.71
CC49	1010.434	976.779	10.14	85.80

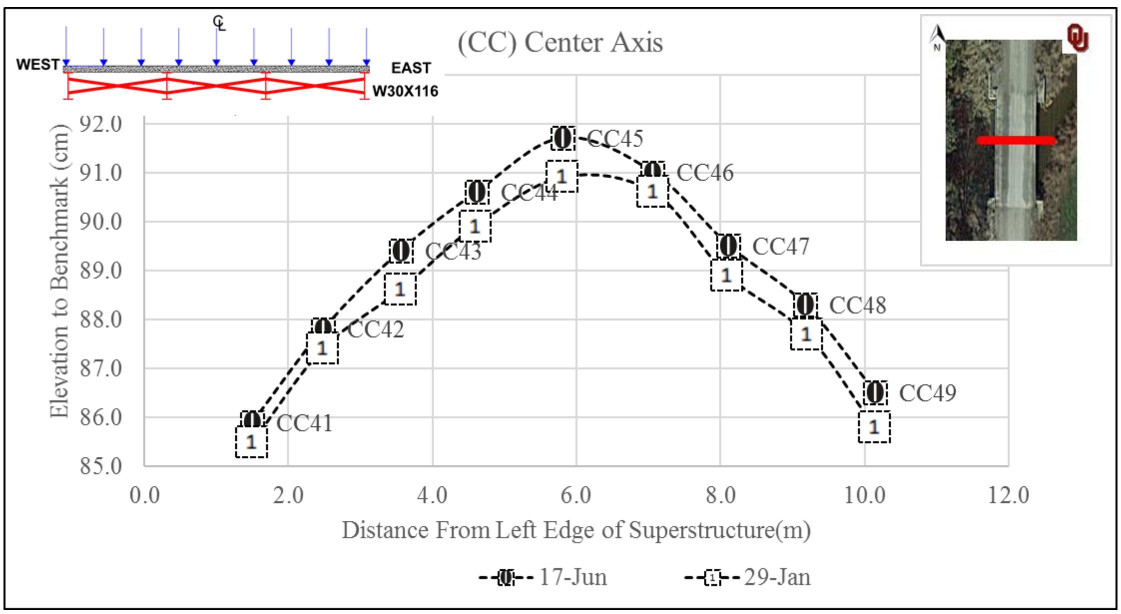
(North Axis) 1st Set

17-Jun				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN41	1002.248	986.061	1.50	90.61
NN42	1003.179	986.003	2.43	91.21
NN43	1004.370	985.936	3.63	92.12
NN44	1005.453	985.889	4.71	92.72
NN45	1006.611	985.843	5.87	92.92
NN46	1007.626	985.737	6.89	91.52
NN47	1008.855	985.723	8.12	89.71
NN48	1009.938	985.645	9.20	88.71
NN49	1010.920	985.584	10.19	87.61

(North Axis) 2nd Set

29-Jan				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN41	1002.285	986.061	1.50	90.31
NN42	1003.213	986.036	2.43	90.81
NN43	1004.407	985.962	3.62	91.42
NN44	1005.508	985.855	4.73	92.12
NN45	1006.619	985.764	5.85	92.42
NN46	1007.643	985.735	6.87	90.91
NN47	1008.875	985.735	8.10	89.11
NN48	1009.970	985.690	9.20	87.61
NN49	1010.951	985.613	10.18	86.50


(SS) South Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance From Left Edge of Superstructure(m)
SS41
SS42
SS43
SS44
SS45
SS46
SS47
SS48
17-Jun
29-Jan


(CC) Center Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance From Left Edge of Superstructure(m)
CC41
CC42
CC43
CC44
CC45
CC46
CC47
CC48
CC49
17-Jun
29-Jan

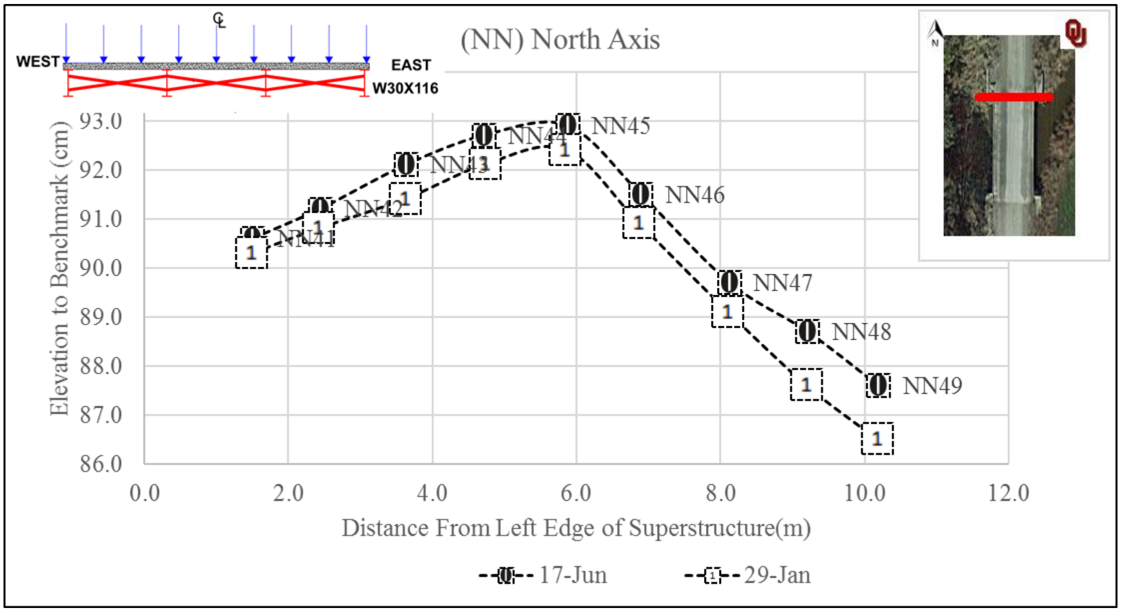


Figure 100: Coordinates of surveyed points on south, center and north axis of Bridge No.4 relative to BM31

4.11.5 Bridge No. 5

During one visit to the bridge on November 20th, one set of settlement control points was surveyed. The baseline for Bridge No. 5 was established. **Table 47** and **Figure 101** show the baseline results for Bridge No. 5. Results show that on the north and south axes the bridge geometry is as expected.

Table 47: Coordinates of surveyed points on Bridge No. 5

(South Axis)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS51	1024.716	1036.320	1.50	8.16
SS52	1025.385	1035.524	2.54	8.37
SS53	1026.066	1034.716	3.60	9.07
SS54	1026.780	1033.813	4.75	10.28
SS55	1027.443	1033.045	5.76	10.48
SS56	1028.200	1032.142	6.94	9.78
SS57	1028.923	1031.266	8.08	9.57
SS58	1029.569	1030.484	9.09	9.47
SS59	1030.161	1029.736	10.04	8.77

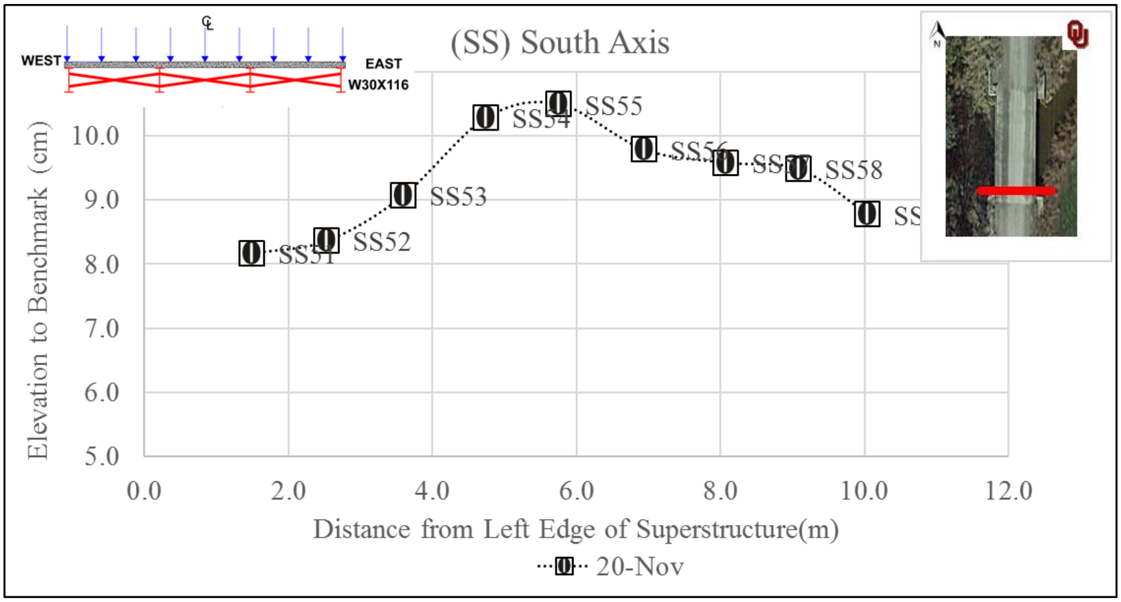
(Center Axis)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC51	1030.936	1041.507	1.50	6.35
CC52	1031.431	1040.778	2.38	8.47
CC53	1031.924	1039.973	3.33	8.57
CC54	1032.879	1039.124	4.60	10.89
CC55	1033.574	1038.140	5.81	10.99
CC56	1034.423	1037.139	7.12	10.58
CC57	1035.123	1036.291	8.22	10.38
CC58	1035.809	1035.473	9.29	10.28
CC59	1036.374	1034.767	10.19	8.67

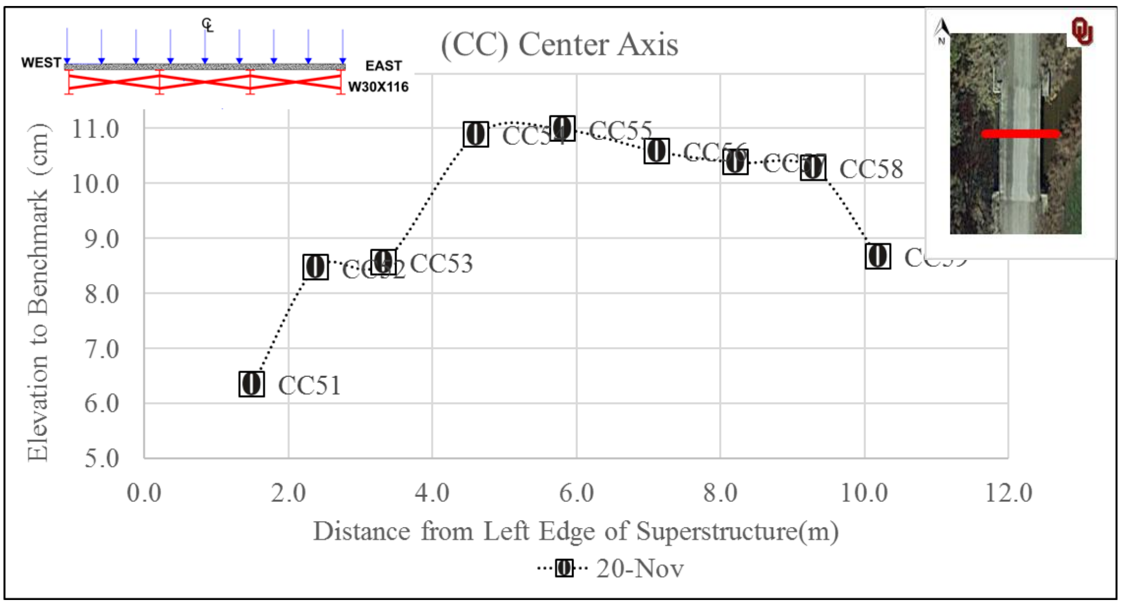
Note: Coordinates in m. and Elevation in cm.

(North Axis)

20-Nov				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN51	1037.877	1047.234	1.50	4.23
NN52	1038.456	1046.538	2.41	5.44
NN53	1039.130	1045.637	3.53	7.76
NN54	1039.700	1044.958	4.42	8.77
NN55	1040.508	1044.024	5.65	8.97
NN56	1041.287	1043.122	6.84	7.36
NN57	1041.972	1042.233	7.97	7.26
NN58	1042.737	1041.347	9.14	6.05
NN59	1043.298	1040.643	10.04	5.34



(SS) South Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance from Left Edge of Superstructure(m)
SS51
SS52
SS53
SS55
SS58
20-Nov



(CC) Center Axis
WEST
EAST
W30X116
Elevation to Benchmark (cm)
Distance from Left Edge of Superstructure(m)
CC51
CC53
CC55
CC58
20-Nov

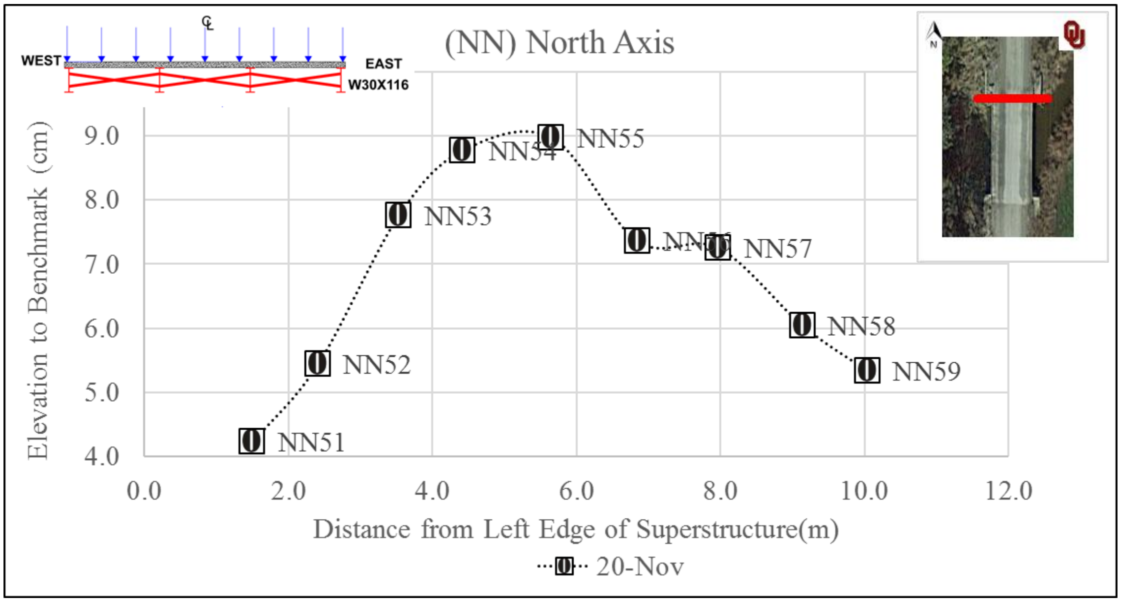


Figure 101: Coordinates of surveyed points on south, center and north axis of Bridge No.5 relative to BM51

4.11.6 Bridge No. 6

In four (4) separate visits to the bridge sites during the period between August 2015 and January 2017 (specifically, on 08/30/2015 and 10/24/2015, 01/29/2016 and 01/20/2017), four (4) slightly different sets of settlement control points were surveyed for Bridge No. 6, **Figure 102** through **Figure 104** show the survey results for Bridge No. 6 from these four separate visits. Survey results for the conventional Bridge No. 6 (i.e. pile foundation) shows approximately 2.5 cm (one inch) of nearly uniform settlement in both the north and south abutments. However, this unexpected result may be attributed to fact that during construction stage the steel piles were not drive to bedrock (Simpson 2016). Also, the bridge has been re-graded several times because a slightly "bump" at the joint appeared. Nonetheless, this bridge has not shown any visible signs of major serviceability or aesthetics-related problems. Nevertheless, the above observations on the performance of the GRS-IBS projects in light of the flooding events and local seismicity to date, confirm that GRS-IBS can indeed provide reliable and cost-effective alternatives to conventional designs for many rural and county roads in Oklahoma.

Table 48: Coordinates of surveyed points on south axis of Bridge No.6

(South Axis) 1st Set

30-Aug-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS61	1012.111	1067.065	1.50	8.65
SS62	1011.215	1067.078	2.40	9.56
SS63	1010.135	1067.040	3.48	9.66
SS64	1008.947	1067.092	4.67	9.46
SS65	1007.788	1067.134	5.83	9.26
SS66	1006.551	1067.130	7.06	10.06
SS67	1005.518	1067.115	8.10	9.96
SS68	1004.449	1067.138	9.17	9.56
SS69	1003.383	1067.150	10.23	9.06

(South Axis) 2nd Set

24-Oct-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS61	1012.096	1067.086	1.50	8.65
SS62	1011.211	1067.099	2.39	8.55
SS63	1010.142	1067.062	3.45	8.65
SS64	1008.973	1067.090	4.62	9.86
SS65	1007.772	1067.107	5.83	9.86
SS66	1006.575	1067.137	7.02	9.86
SS67	1005.537	1067.117	8.06	8.96
SS68	1004.460	1067.146	9.14	8.96
SS69	1003.391	1067.158	10.21	8.55

Note: Coordinates in m. and Elevation in cm.

(South Axis) 3rd Set

29-Jan-16				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS61	1012.084	1067.073	1.50	6.85
SS62	1011.210	1067.095	2.37	7.75
SS63	1010.132	1067.056	3.45	7.75
SS64	1008.989	1067.071	4.60	7.45
SS65	1007.866	1067.091	5.72	7.55
SS66	1006.571	1067.105	7.01	7.55
SS67	1005.539	1067.104	8.05	7.25
SS68	1004.473	1067.144	9.11	7.15
SS69	1003.383	1067.149	10.20	7.25

(South Axis) 4th Set

20-Jan-17				
Point	East (x)	North (Y)	Distance (R)	Elevation
SS61	1012.088	1067.073	1.50	6.95
SS62	1011.203	1067.095	2.39	6.35
SS63	1010.134	1067.056	3.45	6.35
SS64	1008.980	1067.071	4.61	7.25
SS65	1007.790	1067.091	5.80	7.35
SS66	1006.567	1067.105	7.02	6.85
SS67	1005.527	1067.104	8.06	7.05
SS68	1004.483	1067.144	9.11	7.35
SS69	1003.405	1067.149	10.19	7.25

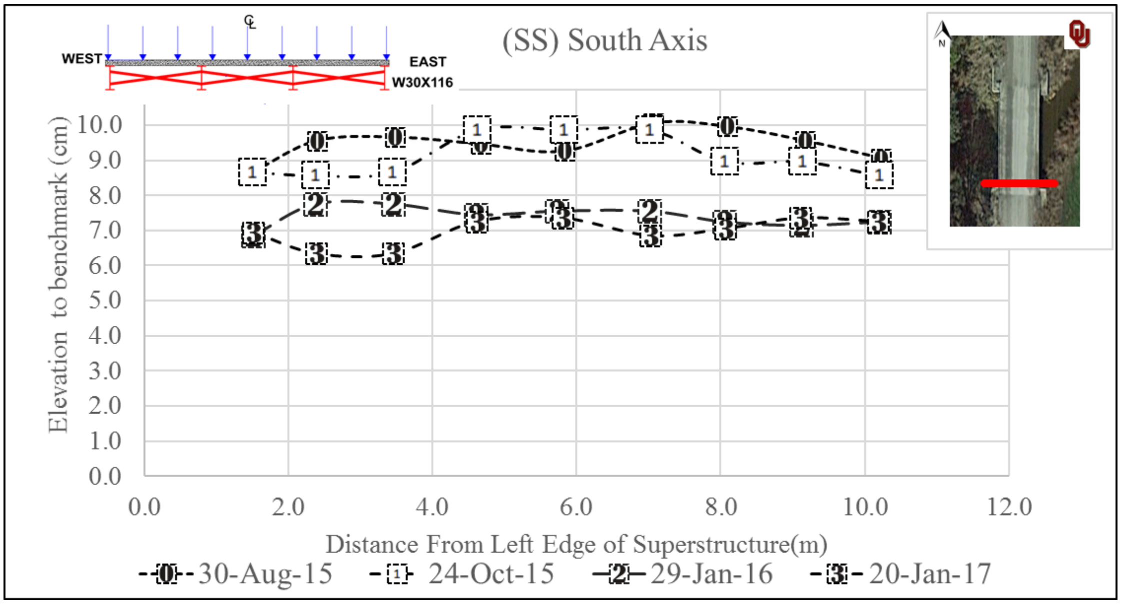


Figure 102: Coordinates of surveyed points on south axis of Bridge No.6 relative to BM61

Table 49: Coordinates of surveyed points on center axis of Bridge No.6

(South Axis) 1st Set

30-Aug-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC61	1012.027	1059.047	1.50	8.25
CC62	1011.203	1059.028	2.32	8.15
CC63	1010.029	1059.042	3.50	5.95
CC64	1008.950	1059.062	4.58	6.95
CC65	1007.715	1059.065	5.81	6.95
CC66	1006.536	1059.123	6.99	8.55
CC67	1005.455	1059.114	8.07	7.35
CC68	1004.315	1059.138	9.21	7.35
CC69	1003.317	1059.231	10.22	7.35

(South Axis) 2nd Set

24-Oct-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC61	1012.047	1059.048	1.50	7.65
CC62	1011.213	1059.039	2.33	7.65
CC63	1010.026	1059.068	3.52	5.95
CC64	1008.945	1059.064	4.60	6.25
CC65	1007.727	1059.091	5.82	6.75
CC66	1006.551	1059.099	7.00	7.65
CC67	1005.457	1059.118	8.09	6.95
CC68	1004.322	1059.140	9.23	7.05
CC69	1003.339	1059.240	10.21	7.05

Note: Coordinates in m. and Elevation in cm.

(South Axis) 3rd Set

29-Jan-16				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC61	1012.050	1059.044	1.50	5.25
CC62	1011.197	1059.012	2.35	5.35
CC63	1010.022	1059.053	3.53	3.94
CC64	1008.961	1059.034	4.59	4.54
CC65	1007.725	1059.067	5.83	4.54
CC66	1006.598	1059.080	6.95	5.45
CC67	1005.463	1059.099	8.09	4.24
CC68	1004.306	1059.131	9.25	4.44
CC69	1003.323	1059.219	10.23	4.44

(South Axis) 4th Set

20-Jan-17				
Point	East (x)	North (Y)	Distance (R)	Elevation
CC61	1012.046	1059.044	1.50	5.05
CC62	1011.224	1059.012	2.32	4.95
CC63	1010.031	1059.053	3.52	4.85
CC64	1008.955	1059.034	4.59	5.25
CC65	1007.770	1059.067	5.78	4.85
CC66	1006.612	1059.080	6.94	5.05
CC67	1005.491	1059.099	8.06	5.15
CC68	1004.323	1059.131	9.23	5.05
CC69	1003.347	1059.219	10.21	4.95

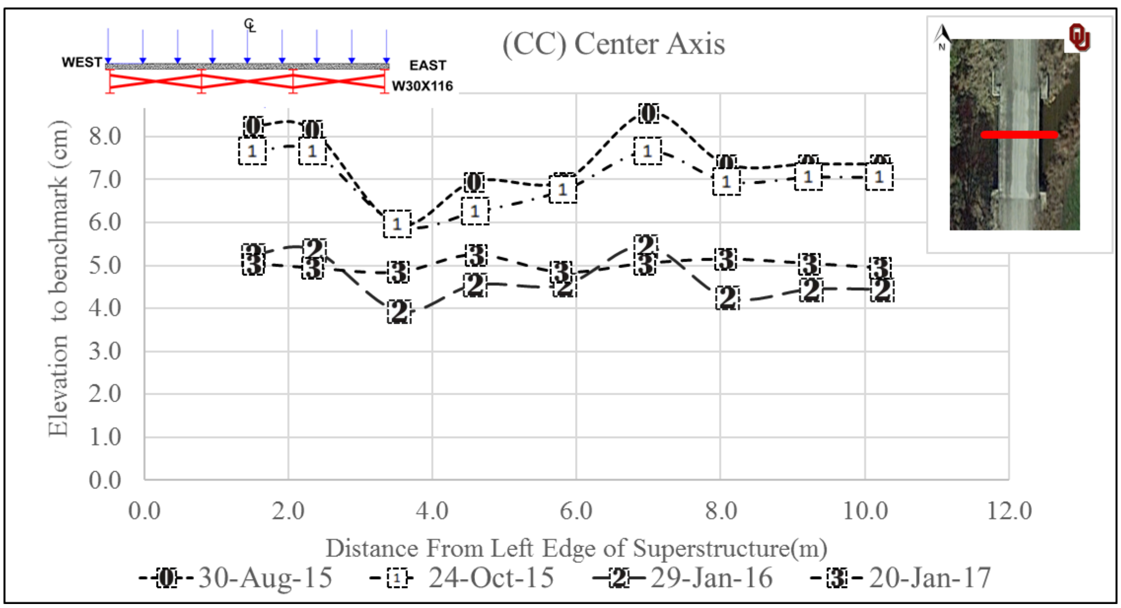


Figure 103: Coordinates of surveyed points on center axis of Bridge No.6 relative to BM61

Table 50: Coordinates of surveyed points on north axis of Bridge No.6

(North Axis) 1st Set

30-Aug-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN61	1012.018	1052.097	1.50	7.05
NN62	1011.178	1052.097	2.34	6.55
NN63	1009.983	1052.134	3.54	6.55
NN64	1008.838	1052.111	4.68	7.25
NN65	1007.682	1052.147	5.84	7.95
NN66	1006.523	1052.117	7.00	7.95
NN67	1005.474	1052.163	8.05	6.35
NN68	1004.300	1052.149	9.22	6.45
NN69	1003.252	1052.151	10.27	6.45

(North Axis) 2nd Set

24-Oct-15				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN61	1012.016	1052.091	1.50	6.45
NN62	1011.190	1052.091	2.33	6.45
NN63	1009.998	1052.122	3.52	6.35
NN64	1008.834	1052.107	4.68	7.75
NN65	1007.680	1052.150	5.84	7.75
NN66	1006.551	1052.092	6.97	7.05
NN67	1005.479	1052.179	8.04	7.05
NN68	1004.289	1052.146	9.23	7.05
NN69	1003.262	1052.158	10.26	6.35

Note: Coordinates in m. and Elevation in cm.

(North Axis) 3rd Set

29-Jan-16				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN61	1012.018	1052.085	1.50	4.14
NN62	1011.185	1052.077	2.33	4.04
NN63	1009.985	1052.101	3.53	4.04
NN64	1008.823	1052.096	4.70	5.55
NN65	1007.684	1052.141	5.84	5.35
NN66	1006.559	1052.094	6.96	5.05
NN67	1005.455	1052.157	8.07	5.05
NN68	1004.287	1052.151	9.23	4.24
NN69	1003.255	1052.145	10.27	4.04

(North Axis) 4th Set

20-Jan-17				
Point	East (x)	North (Y)	Distance (R)	Elevation
NN61	1012.025	1052.085	1.50	4.34
NN62	1011.192	1052.077	2.33	4.14
NN63	1009.979	1052.101	3.55	4.34
NN64	1008.828	1052.096	4.70	5.55
NN65	1007.692	1052.141	5.83	5.65
NN66	1006.553	1052.094	6.97	4.14
NN67	1005.470	1052.157	8.06	4.24
NN68	1004.293	1052.151	9.24	4.24
NN69	1003.253	1052.145	10.28	4.24

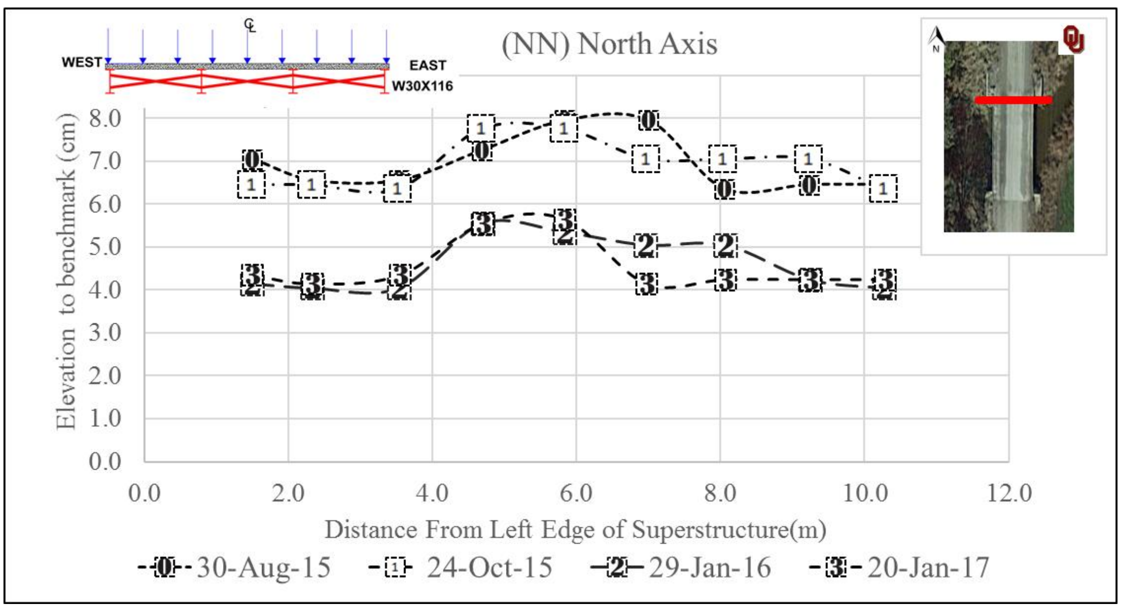


Figure 104: Coordinates of surveyed points on north axis of Bridge No.6 relative to BM61

Chapter 5. Numerical Modeling of Kay County Bridge No. 2 GRS Abutment

5.1 Introduction

The computer program Fast Lagrangian Analysis of Continua (FLAC; Itasca 2005) was used to continue the development and improvement of the numerical models developed by Ngo (2016). **Figure 105** shows an as-built cut-away section for numerical simulation of GRS-IBS Bridge No. 2 in Kay County, OK based on the information obtained from Mr. Tom Simpson at the Bureau of Indian Affairs (BIA) in Anadarko, OK, and Mr. Pete Lively, who is the former Road Foreman at Kay County, District 3. The information on local soils indicated in the figure is based on the geotechnical report discussed in Section 4.6-Geotechnical Data of Kay County, Oklahoma.

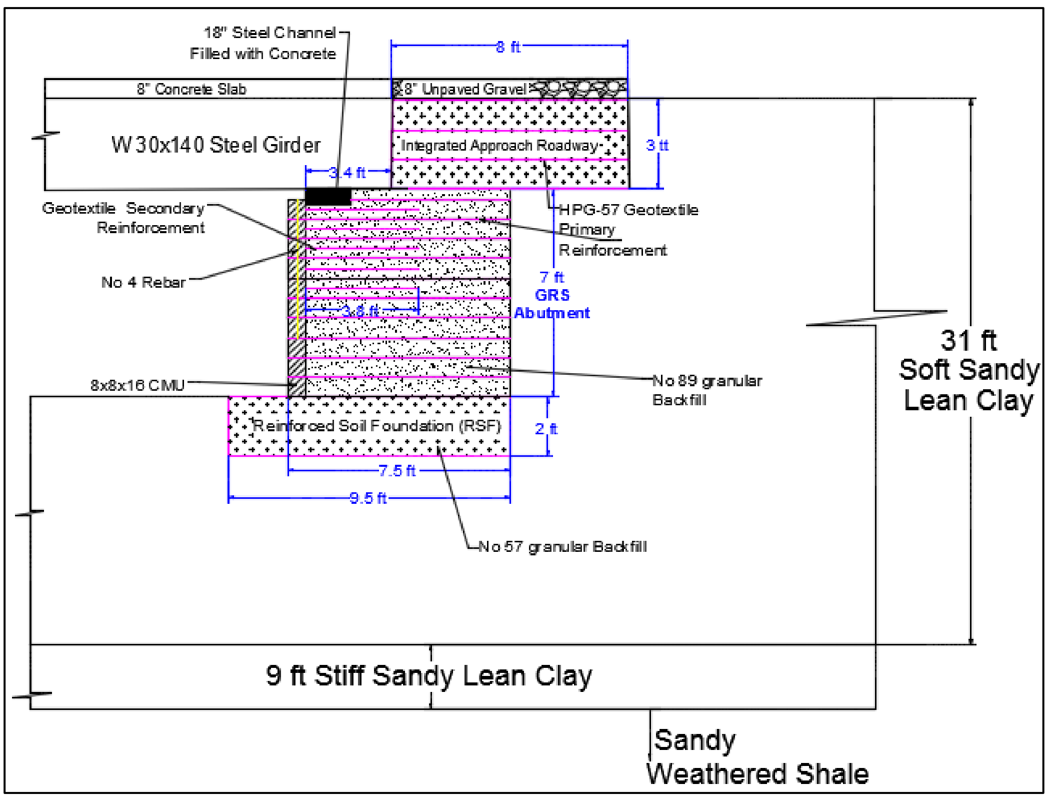


Figure 105: Detailed as-built cross-section of the GRS-IBS Bridge No. 2 in Kay County (Ngo 2016)

5.2 Model Configuration and Material Properties

Ngo (2016) proposed six stages for the numerical model. However, the author added the Model configuration and Geometry Stage as an initial input for the model. Thus, the numerical simulations for the GRS-IBS model were set up in seven different stages as listed below to evaluate abutment deformations during construction, placement of the bridge superstructure, construction of the gravel road, and an equivalent static load of the traffic:

Stage 1: Model configuration and geometry (**Figure 106**)

Stage 2a: Excavation of the abutment area and shallow foundation (RSF) (**Figure 107a**)

Stage 3: Construction of RSF (**Figure 107b**)

Stage 4: Construction of GRS abutment in lifts and placement of reinforcement (**Figure 107c**)

Stage 5: Application of the bridge load equal to 65.32kPa (1,365 psf) on the GRS-IBS abutment through the beam seat (**Figure 107d**)

Stage 6: Application of 18.8kPa (390 psf) surcharge load due to the approach roadway (20 cm ~ 8 in. unpaved gravel road) (**Figure 107e**)

Stage 7: Application of an equivalent static traffic load of 13.2 kPa (280 psf) (**Figure 107f**). Traffic load on the GRS abutment was simulated using an equivalent 13.2 kPa uniform surcharge load (i.e. 0.61 m ~ 2 ft of soil) on the top of the entire model as recommended by FHWA design guidelines (Berg et al. 2009).

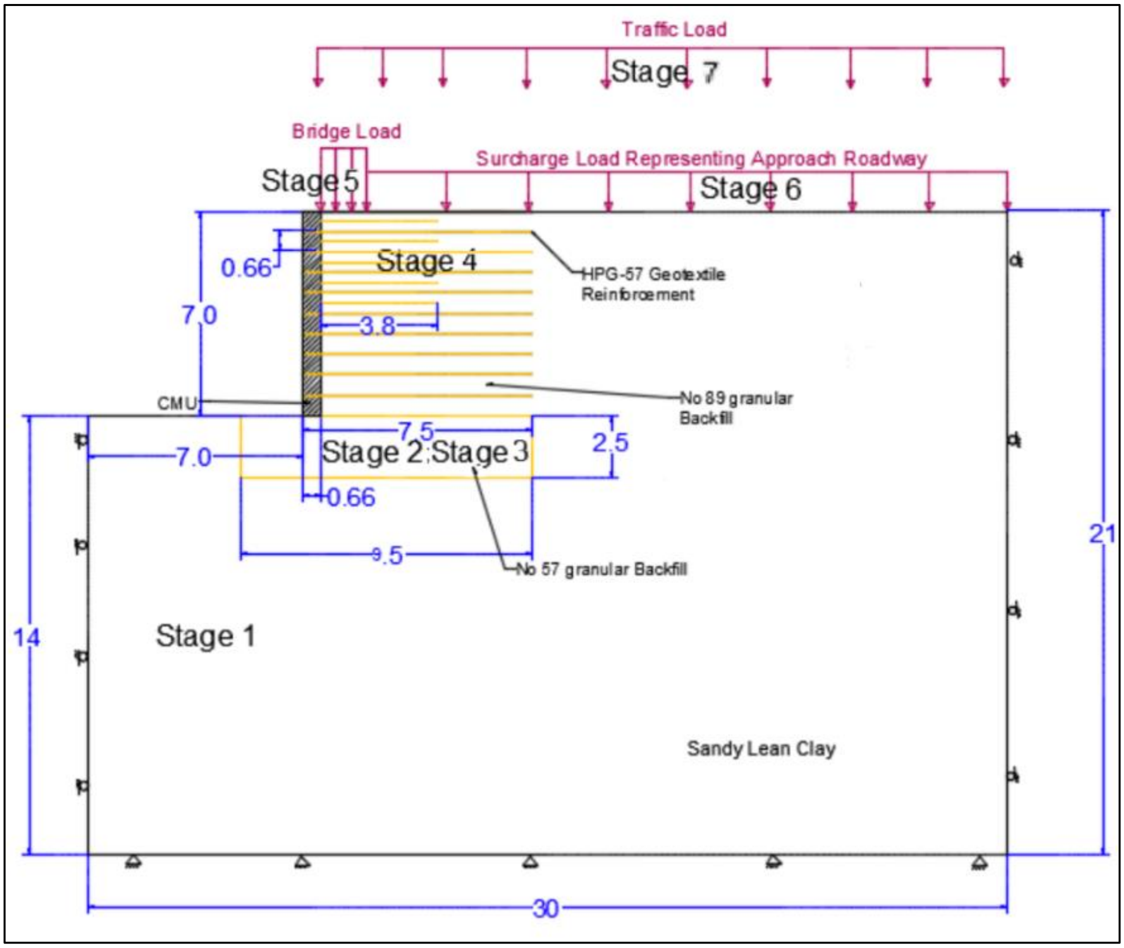
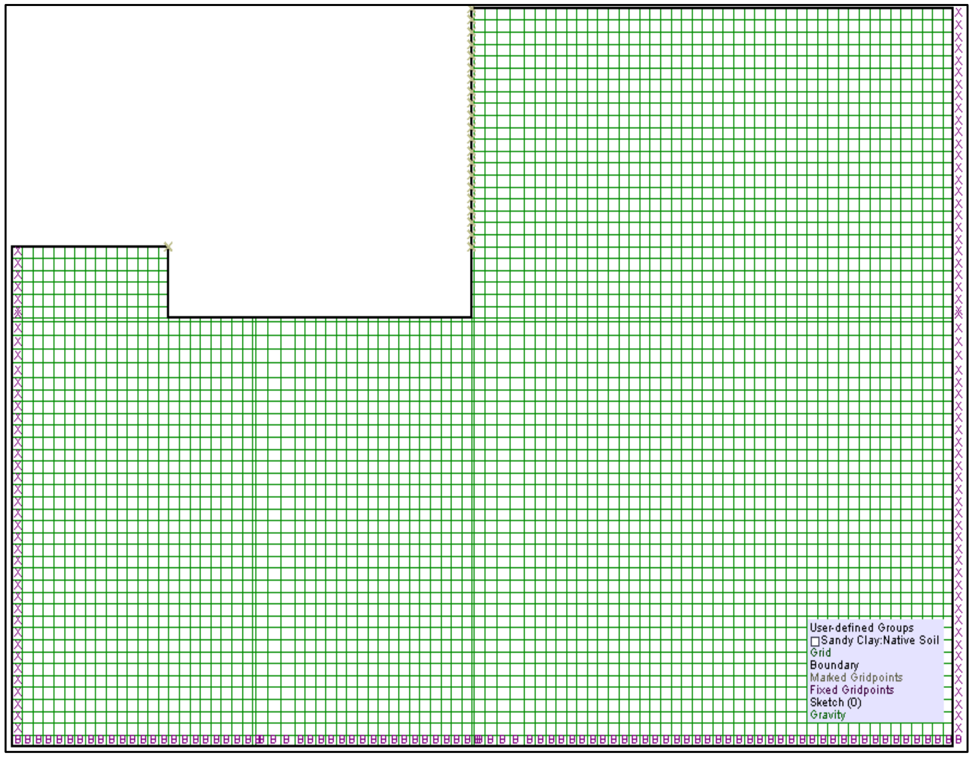
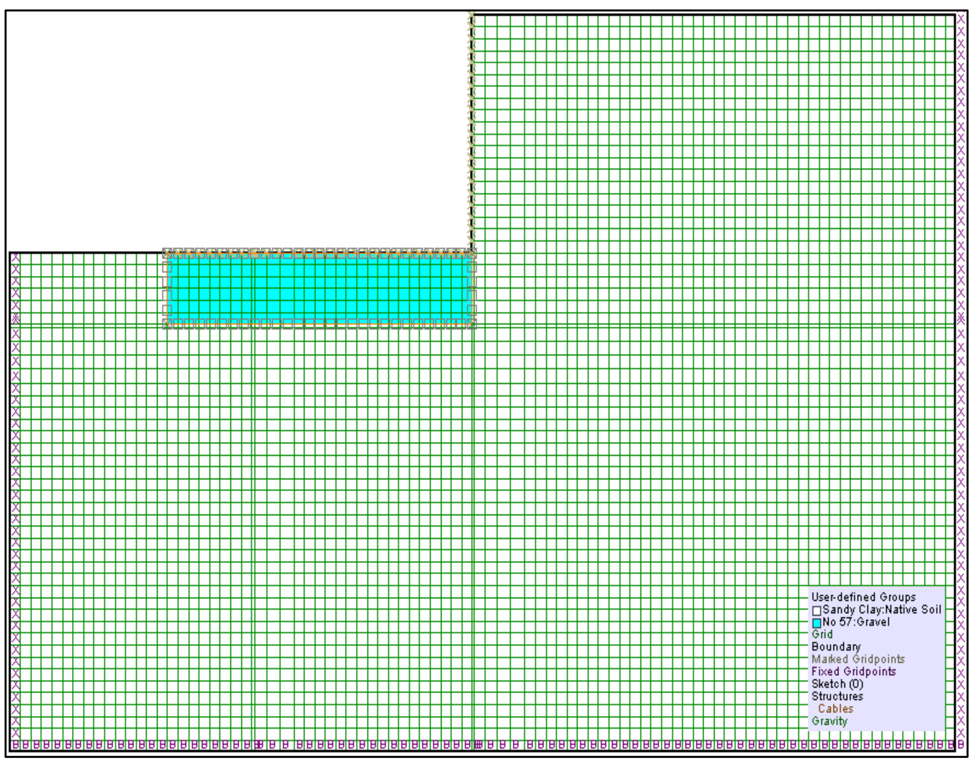


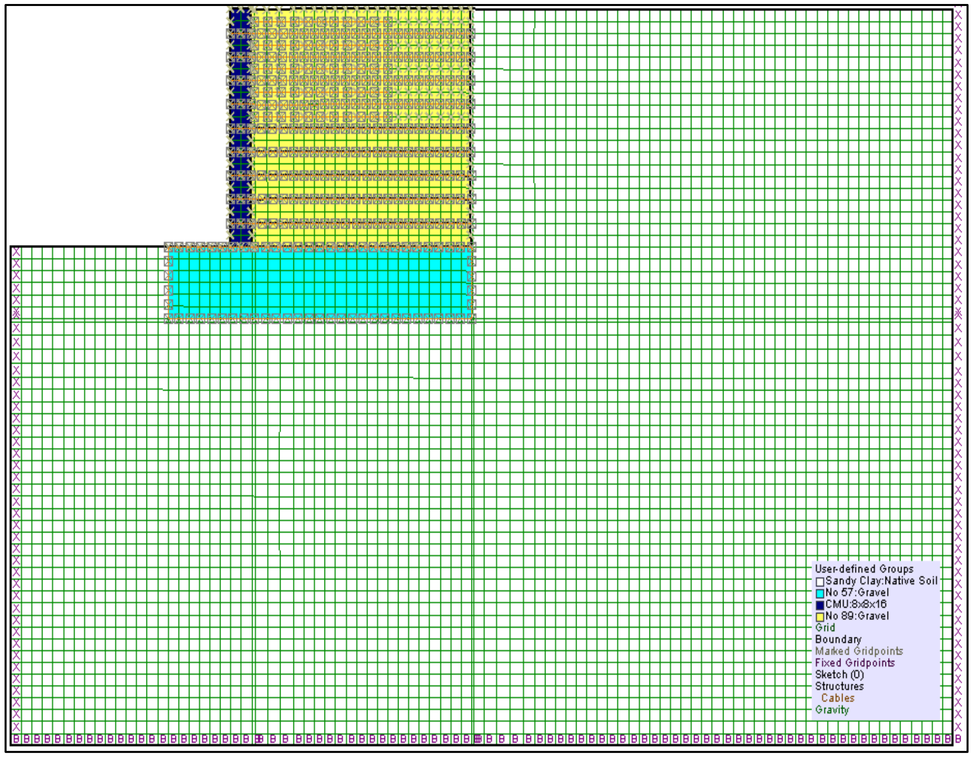
Figure 106: Construction stages in numerical modeling of GRS-IBS Bridge No. 2 (all dimensions are in ft.) (Hatami et al. 2015; Ngo 2016)


User-defined Groups
Sandy Clay:Native Soil
Grid
Boundary
Marked Gridpoints
Fixed Gridpoints
Sketch (0)
Gravity

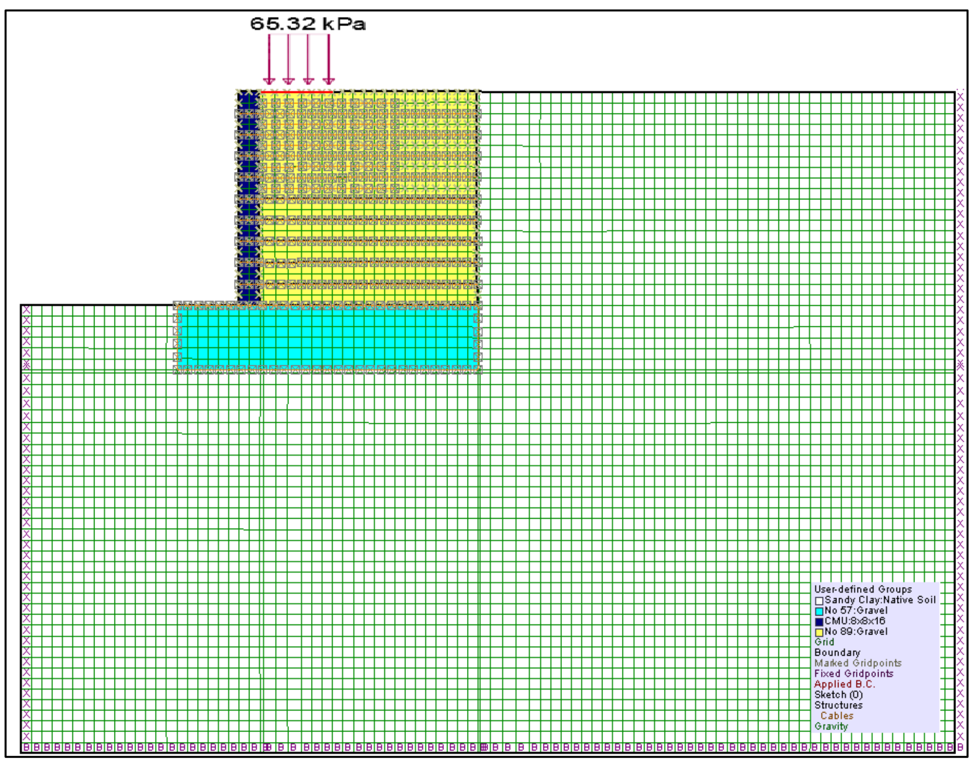
(a)


User-defined Groups
Sandy Clay:Native Soil
No 57:Gravel
Grid
Boundary
Marked Gridpoints
Fixed Gridpoints
Sketch (0)
Structures
Cables
Gravity

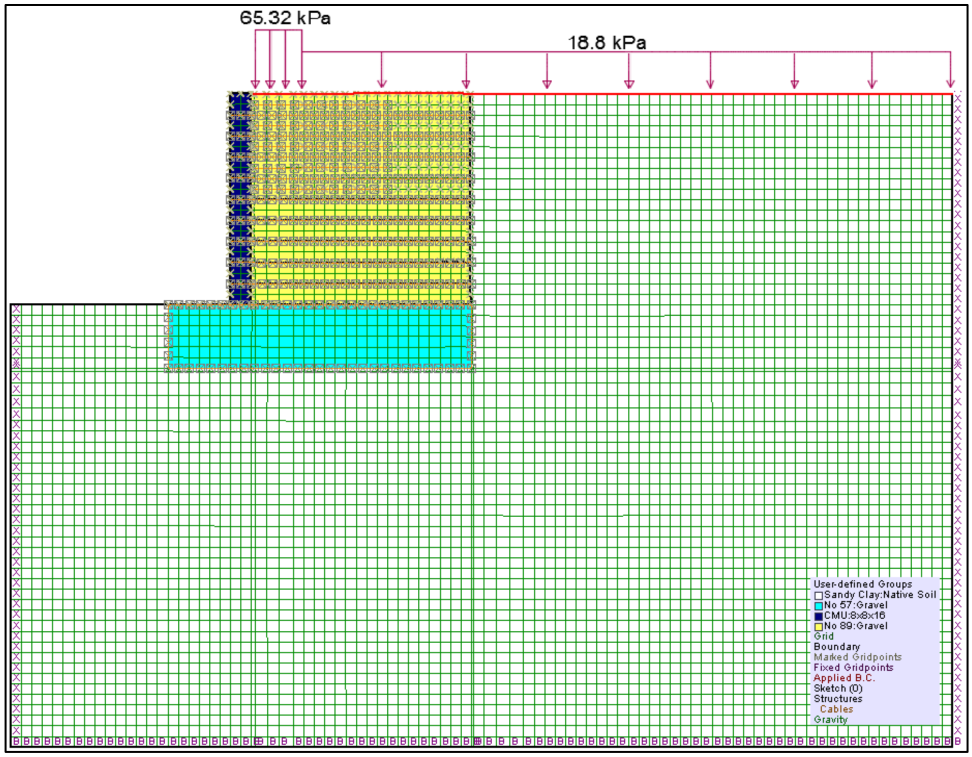
(b)


User-defined Groups
Sandy Clay:Native Soil
No 57:Gravel
CMU:8x8x16
No 89:Gravel
Grid
Boundary
Marked Gridpoints
Fixed Gridpoints
Sketch (0)
Structures
Cables
Gravity

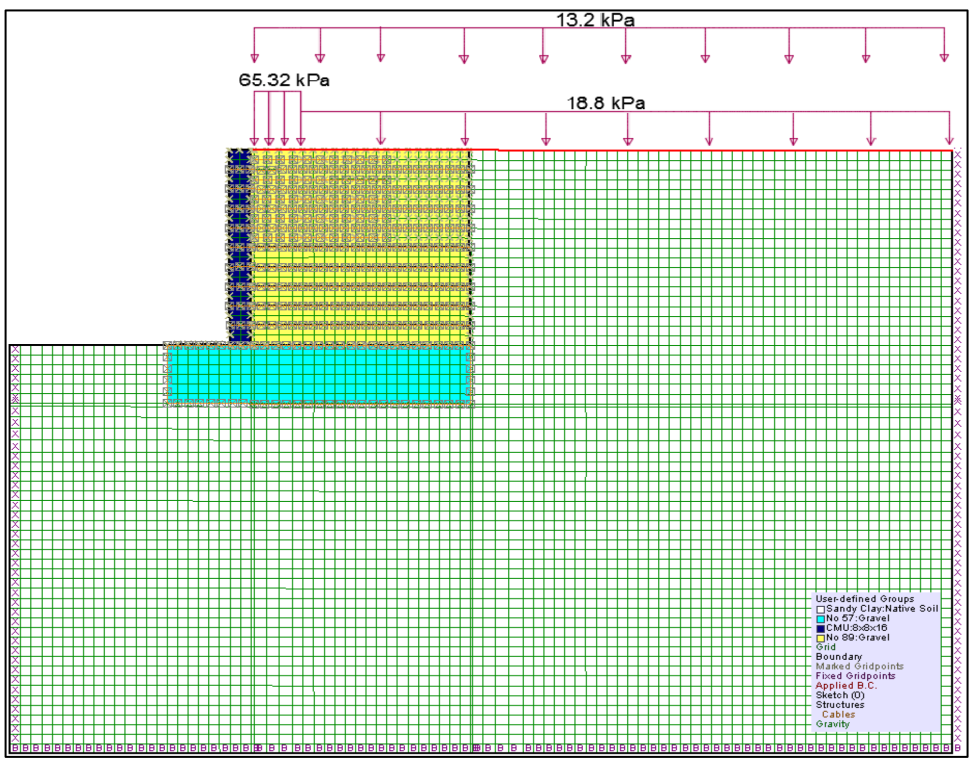
(c)


65.32 kPa
User-defined Groups
Sandy Clay:Native Soil
No 57:Gravel
CMU:8x8x16
No 89:Gravel
Grid
Boundary
Marked Gridpoints
Fixed Gridpoints
Applied B.C.
Sketch (0)
Structures
Cables
Gravity

(d)



(e)

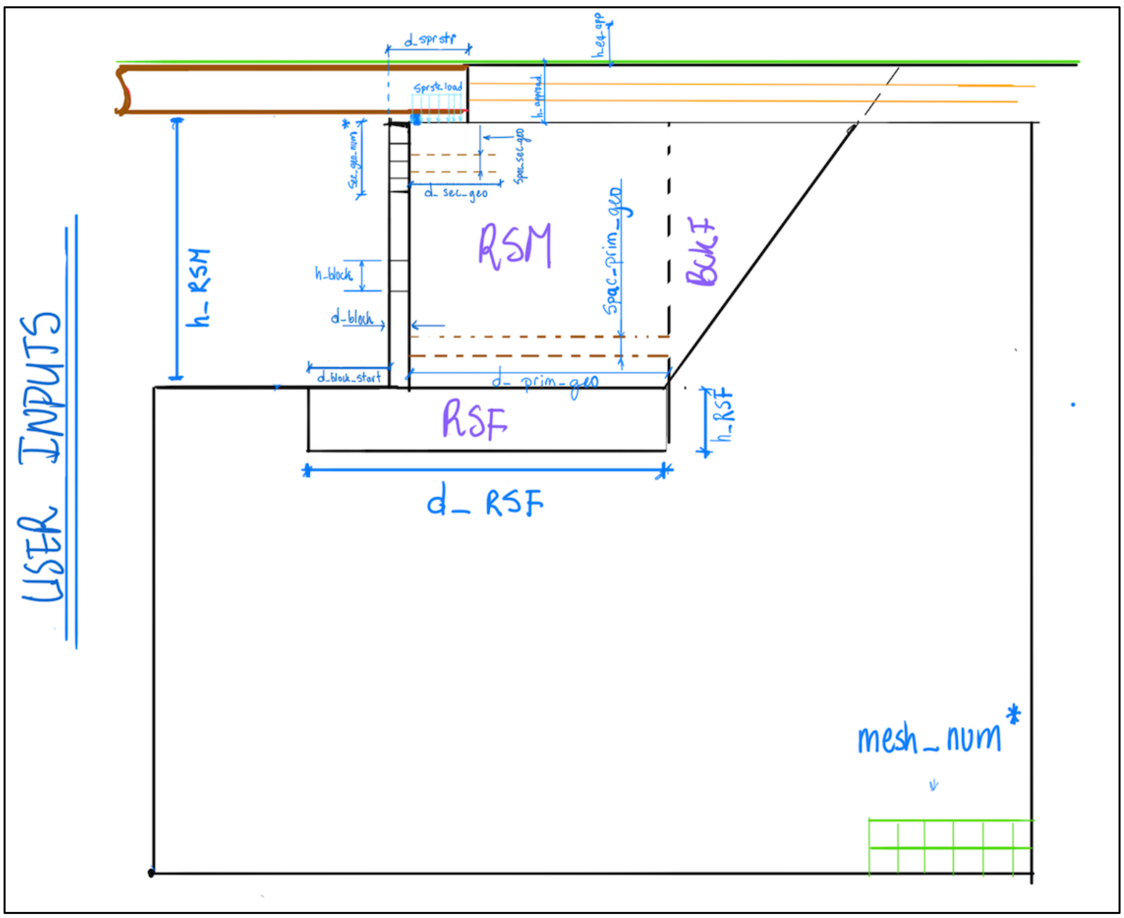


(f)

Figure 107: Numerical modeling of Bridge No. 2 (Ngo 2016; Hatami et al. 2016)

5.3 Model Input

The author set up the numerical model to accept different geometries and material properties for future parametric analysis of different GRS-IBS models. **Figure 108b and Figure 108c** show a detailed list of the geometrical and material parameters that the user can input in either the imperial or SI units according to the simplified design sketch shown in **Figure 108a**, which is consistent with the standard FHWA design drawing shown in **Figure 2** (Adams et al. 2012).



(a)

User Inputs : Geometry						
Input parameter		Variable	Qty	(SI)	Qty	(Imp)
RSF Length	[length]	**d_RSF**	2.8956	m	9.500	ft
RSF height	[length]	**h_RSF**	0.7620	m	2.500	ft
Abutment height	[length]	**h_RSM**	2.1336	m	7.000	ft
Mesh resolution; number of zones in 1 unit		**mesh_num***	6.0000	Unit	6.000	ft
Distance from the start of RSF to the facing block	[length]	**d_block_start**	0.6096	m	2.000	ft
Block Height	[length]	**h_block**	0.2032	m	0.667	ft
Block Depth	[length]	**d_block**	0.2032	m	0.667	ft
Primary reinforcement depth	[length]	**d_prim_geo**	2.2860	m	7.500	ft
Spacing Primary Reinf	[length]	**spac_prim_geo**	0.2032	m	0.667	ft
Secondary reinforcement depth	[length]	**d_sec_geo**	1.1582	m	3.800	ft
Primary Reinf spacing (must be integer of h_block)	[length]	**spac_sec_geo**	0.10160000	m	0.333	ft
Secondary reinforcement number of blocks		**sec_geo_num**	5.0000	unit	5.000	unit
Depth of the superstructure on the abutment	[length]	**d_sprstr**	1.0668	m	3.500	ft
Superstructure Stress	[stress]	**sprstr_load**	-65,320.0000	Pa	-1.36E+03	lbf/ft2
Approach Roadway height	[length]	**h_approad**	1.0058	m	3.300	ft
Equivalent surcharge load height	[length]	**h_eq_traff**	0.6096	m	2.000	ft

(b)

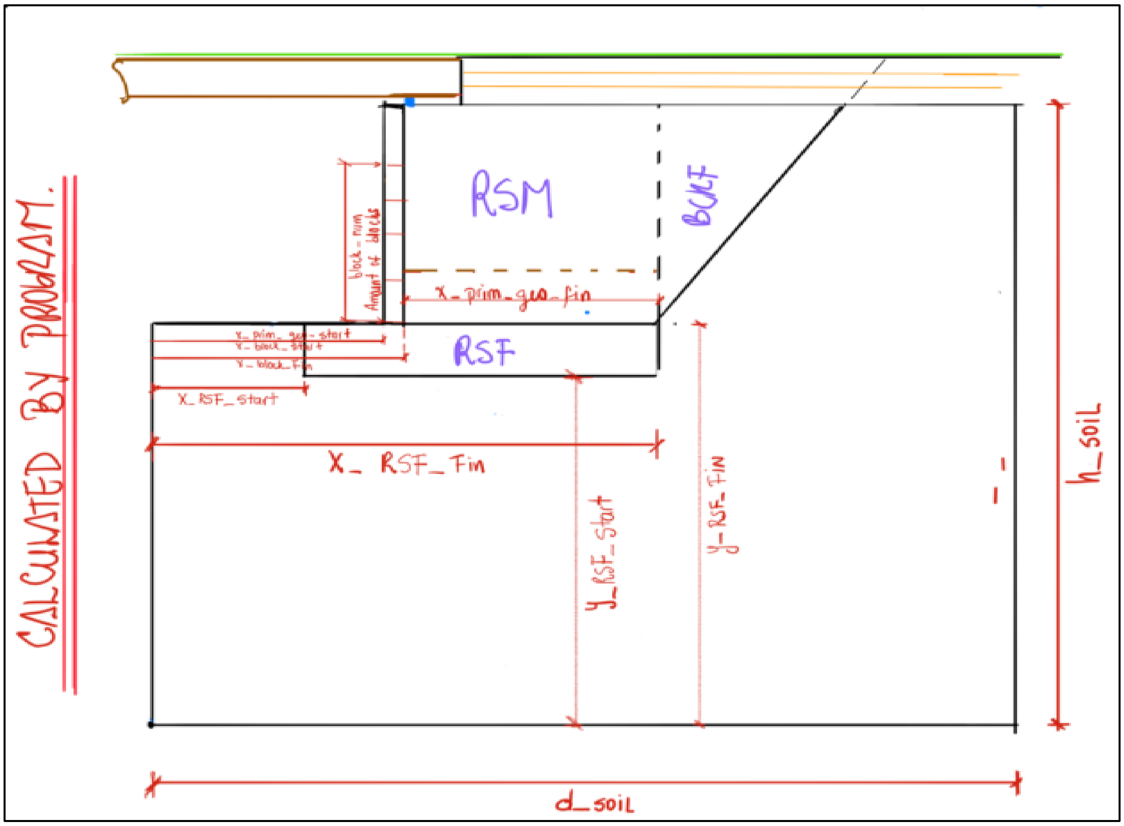
User Inputs: Geometry						
Property		Variable	Qty	(SI)	Qty	(Imp)
Native Soil Mass Density	[mass/volume]	**native_den**	1735.0	kg/m3	3.37	slug/ft3
Native Soil Bulk Modulus	[stress]	**native_bu**	3.33E+07	Pa	6.95E+05	lbf/ft2
Native Soil Shear modulus	[stress]	**native_sh**	1.54E+07	Pa	3.21E+05	lbf/ft2
Native Soil Cohesion	[stress]	**native_coh**	2.00E+04	Pa	4.18E+02	lbf/ft2
Native Soil Friction Angle	[degrees]	**native_fric**	20.0	Dev	20.00	Deg
Cable Area [Structural element 1.4.2]	[length^2]	**cbl_area**	0.00160	m2	6.17E-10	ft2
Cable density (default value)	[mass / volume]	**cbl_density**	0.0	kg/m3	0.00	slug/ft3
Cable elastic modulus	[stress]	**cbl_e**	6.03E+08	Pa	1.26E+07	lbf/ft2
Cable Spacing (default value)	[length]	**cbl_spac**	1.0	m	1.00	ft
Cable Tensile yield strength	[force]	**cbl_yield**	1.93E+04	N	4338.81	lbf
Cable Compressive yield strength	[force]	**cbl_comp**	0.0	N	0.00	lbf
Cable Exposed perimeter	[length]	**cbl_perim**	2.0032	m	6.5721	ft
Cable stiffness of the grout	[force/cable length / displacement]	**cbl_kbond**	48000.0	m	1002.51	lbf/ft/ft
Cable cohesive strength of the grout	[force /cable length]	**cbl_sbond**	45500.0	N/m	3117.74	lbf/ft
Cable frictional resistance of the grout	[degrees]	**cbl_sfric**	26.0	Deg	26.00	deg
Reinforced soil foundation Density	[mass/volume]	**RSF_den**	1937.6	kg/m3	3.76	slug/ft3
Reinforced soil foundation Bulk Modulus	[stress]	**RSF_bu**	1.11E+08	Pa	2.32E+06	lbf/ft2
Reinforced soil foundation Shear Modulus	[stress]	**RSF_sh**	6.30E+07	Pa	1.32E+06	lbf/ft2
Reinforced soil foundation Cohesion	[stress]	**RSF_coh**	0.00E+00	Pa	0.00E+00	lbf/ft2
Reinforced soil foundation Friction angle	[degrees]	**RSF_fric**	52.0	Dev	52.00	Deg
Reinforced soil foundation Dilation	[degrees]	**RSF_dil**	15.0	Dev	15.00	Deg
Reinforced soil mass Density	[mass/volume]	**RSM_den**	1834.9	kg/m3	3.56	slug/ft3
Reinforced soil mass Bulk Modulus	[stress]	**RSM_bu**	1.04E+08	Pa	2.17E+06	lbf/ft2
Reinforced soil mass Shear Modulus	[stress]	**RSM_sh**	6.00E+07	Pa	1.25E+06	lbf/ft2
Reinforced soil mass Cohesion	[stress]	**RSM_coh**	0.00E+00	Pa	0.00E+00	lbf/ft2
Reinforced soil mass Friction angle	[degrees]	**RSM_fric**	44.0	Dev	44.00	Deg
Reinforced soil mass Dilation	[degrees]	**RSM_dil**	14.0	Dev	14.00	Deg
Block Density	[mass/volume]	**blck_den**	2240.0	kg/m3	4.35	slug/ft3
Block Bulk Modulus	[stress]	**blck_bu**	1.10E+10	Pa	2.30E+08	lbf/ft2
Block Shear Modulus	[stress]	**blck_sh**	8.90E+09	Pa	1.86E+08	lbf/ft2

(c)

Figure 108: GRS-IBS abutments model input: (a) Simplified design sketch defining model geometry; (b) Geometry input parameters;(c) Materials input parameters

5.4 Preprocessing Algorithm

Another feature added by the author was the inclusion of a preprocessing algorithm to set up the numerical model using the user input parameters and simplified sketches (**Figure 109**a and **Figure 109b** that are compatible with the standard FHWA design drawing (**Figure 2**) .



(a)

Calculated by the program						
Input parameter	Equation	Variable	Qty	(SI)	Qty	(Imp)
Gravity		**gravity**	9.8100	m/s2	32.185	ft/s2
Depth of Soil Mass	[= d_RSF * 4]	**d_soil**	11.5824	m	38.00	ft
Height of Soil Mass	[= h_RSM*4]	**h_soil**	8.5344	m	28.00	ft
Number of grid zones in depth	[=mesh_num*d_soil]	**d_grid_num**	228.0000	Unit	228.00	unit
Number of grid zones in height	[=mesh_num*h_soil]	**h_grid_num**	51.2064	m	168.00	ft
Total grid point number in depth	[=d_grid_num+1]	**x_grid_num**	69.7992	m	229.00	ft
Total grid point number in height	[=d_grid_num+1]	**y_grid_num**	51.5112	m	169.00	ft
X coordinate of RSF Start	[=d_RSF]	**x_RSF_start**	2.8956	m	9.50	ft
X coordinate of RSF end	[=x_RSF_start + d_RSF]	**x_RSF_fin**	5.7912	m	19.00	ft
Y coordinate of RSF Start	[= h_soil - h_RSM - h_RSF]	**y_RSF_start**	5.6388	m	18.50	ft
Y coordinate of RSF end	[= h_soil - h_RSM]	**y_RSF_fin**	6.4008	m	21.00	ft
Y coordinate of excavation start	[= h_soil - h_RSM]	**y_exc_start**	6.4008	m	21.00	ft
Block_num	[=(int(h_RSM/h_block))]	**block_num**	3.3528	unit	11.00	unit
X coordinate of Block Start	[= = x_RSF_start + d_block_start]	**x_block_start**	3.5052	m	11.50	ft
X coordinate of Block end	[= x_block_start + d_block]	**x_block_fin**	3.7084	m	12.17	ft
Y coordinate of Block Start	[=h_soil - h_RSM]	**y_block_start**	6.4008	m	21.00	ft
Y coordinate of Bloc end	[=h_soil]	**y_block_fin**	8.5344	m	28.00	ft
X Coordinate primary reinforcement start	[= x_block_start]	**x_prim_geo_start**	3.5052	m	11.50	ft
X coordinate of primary reinforcement attached to face	[= x_prim_geo_start + d_prim_geo]	**x_prim_geo_fin**	3.7084	m	12.17	ft
Number of layer from top to bottom of block	[=int(h_block/spac_prim_geo)]	**prim_rear_num**	0.3048	unit	1.00	unit
Number of layer between the prim reinf	[= int (h_block / spac_sec_geo)]	**sec_rear_num**	0.6096	m	2.00	ft
X Coordinate of secondary reinforcement start	[= x_block_start + d_block]	**x_sec_geo_start**	3.7084	m	12.17	ft
X Coordinate of secondary reinforcement end	[=x_sec_geo_start + d_sec_geo]	**x_sec_geo_fin**	4.8666	m	15.97	ft
X Coordinate of Superstructure Start	[= x_block_start + d_block]	**x_sprstr_start**	3.7084	m	12.17	ft
X Coordiante of Superstructure End	[= x_sprstr_start + d_sprstr - d_block]	**x_sprstr_fin**	4.5720	m	15.00	ft
Y Coordinate of Superstructure Start	[=h_soil]	**y_sprstr_start**	8.5344	m	28.00	ft
Y Coordiante of Superstructure End	[=h_soil]	**y_sprstr_fin**	8.5344	m	28.00	ft
Weight of Approach Roadway	[= RSF_den * gravity * h_approad]	**approad_load**	19118.86	Pa	399.61	lbf/ft2
Weight of Equivalent traffic load	[= RSF_den * gravity * h_eq_traff]	**eq_traff_load**	11587.1886	Pa	242.19	lbf/ft2

(b)

Figure 109: GRS-IBS abutments model: (a) Simplified design sketch for preprocessing algortihm; (b) Geometry and material parameters calculated by the FLAC model

5.5 Material Properties

Selected input parameters for the numerical model were determined through laboratory tests and related literature as shown in **Table 51**.

Table 51: Properties of materials used in FLAC simulation of GRS-IBS projects (Ngo 2016)

Backfill AASHTO No.89 gravel (GRS abutment backfill) properties	
Density (kg/m^3)	1834.9
Friction angle (deg)	44
Dilation angle (deg)	14
Bulk Modulus, K (MPa)	104
Shear Modulus, G (MPa)	60
Backfill AASHTO No. 57 gravel (RSF backfill) properties	
Density (kg/m^3) - Modified Proctor	1937.6
Friction angle (deg)	52
Bulk Modulus, K (MPa)	111
Shear Modulus, G (MPa)	63
Sandy lean clay - Native soil properties	
Density (kg/m^3)	1735
Friction angle (deg)	20
Cohesion (kPa)	20
Bulk Modulus, K (MPa)	33
Shear Modulus, G (MPa)	15
Solid 8" × 8" × 16" CMU (facing block) properties	
Young's Modulus, E (MPa)	20000
Poisson's ratio	0.2
Density (kg/m3)	2240
Bulk Modulus, K (MPa)	11111
Shear Modulus, G (MPa)	8333
Geotextile properties (TerraTex HPG-57) Structural element: Cable	
Area, A (m^2) = 1 m width * thickness	0.0016
Tensile strength, T(kN/m) in cross machine direction	19.3
Tensile stiffness, $J_{2\%}=T_{2\%}/0.02$ (kN/m)	965
Young's Modulus, E (MPa) = J/A	603
Geotextile-backfill (No.89) interface	
K_{bond} (N/m/m)	48000
S_{bond} (N/m/m)	45500
Friction Angle, δ (deg)	26
Interface properties between No. 89 backfill and CMU	
Friction angle (deg)	29
Cohesion (kPa)	0
Interface properties between No. 89 backfill and native soil	
Friction angle (deg)	29
Cohesion (kPa)	13

5.6 Parametric Study in the Numerical Model for Bridge No. 2

5.6.1 Influence of Different Facing Blocks

A set of parametric analyses was carried out on GRS facing blocks, which included two additional sizes of 41 cm × 61 cm (16 in. × 24 in.) and 61 cm × 61 cm (24 in. × 24 in.) in cross-sectional view in addition to the control (as built) size of 20 cm x 20 cm (8 in. x 8 in.) blocks. The 61 cm × 61 cm (24 in. × 24 in.) blocks represented those available from a local block manufacturer (Dolese 2016). In this case, surcharge pressures equal to 0.450, 0.395 and 0.355 kPa (i.e 9.39, 8.24 and 7.3 psf) were applied on the top of the blocks, GRS mass (Gravel No. 89) and the retained native soil behind the GRS mass, respectively to compensate for the facing height diffrence in the model with large blocks and keep it comparable to other models. It should be noted that the numerical models represent plane strain conditions, and the results are interpreted for a unit width of facing perpendicular to the plane of analysis. Therefore, the dimension of the block in the running length of the facing is irrelevant to the analysis, and the blocks are merely referred to by their cross-sectional dimensions in the analysis and discussion of results presented in this section. The 41 cm × 61 cm (16 in. × 24 in.) model is used here as an example and can be modified to include other large blocks that are commercially available, as necessary. **Figure 110** shows the numerical models with different size facing blocks that are otherwise comparable to one another with respect to their geometry and material properties. The facing block properties and loading conditions for the two GRS-IBS models are listed in **Table 52** and **Table 53**, respectively.

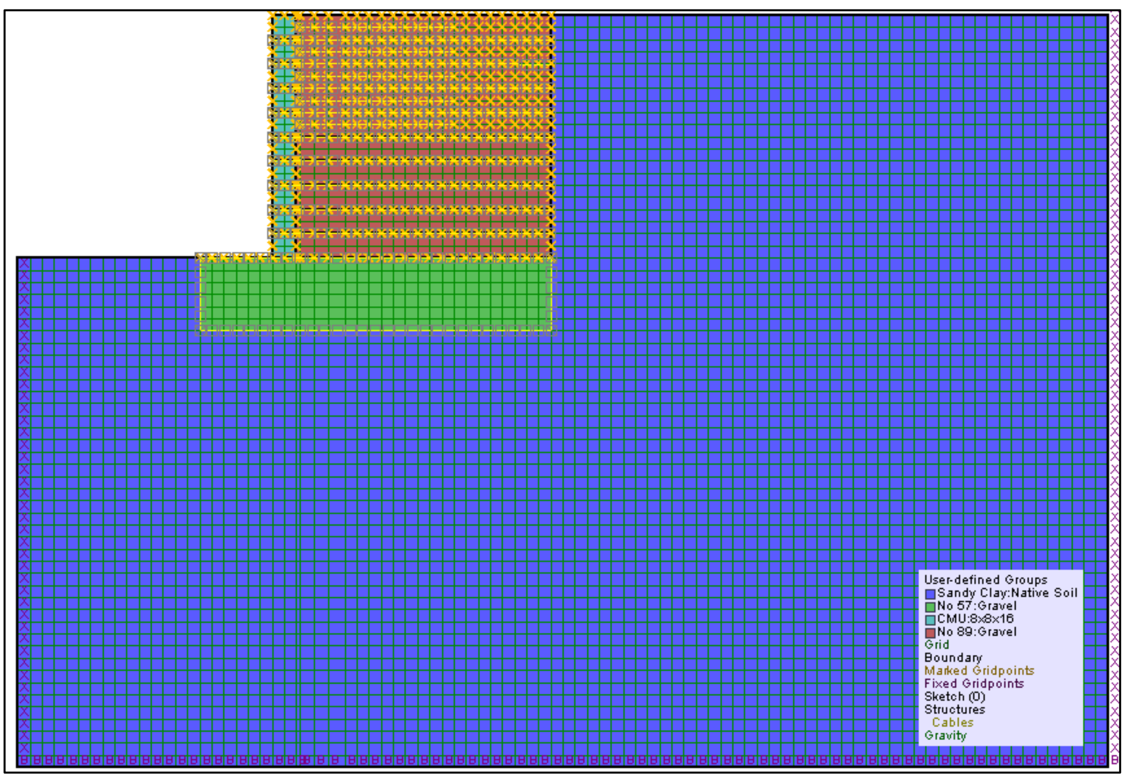
Table 52: Solid CMU properties in GRS-IBS numerical (FLAC) models

(1 kgf/m^3 = 0.062 pcf, 1 MPa = 20.89 ksf)

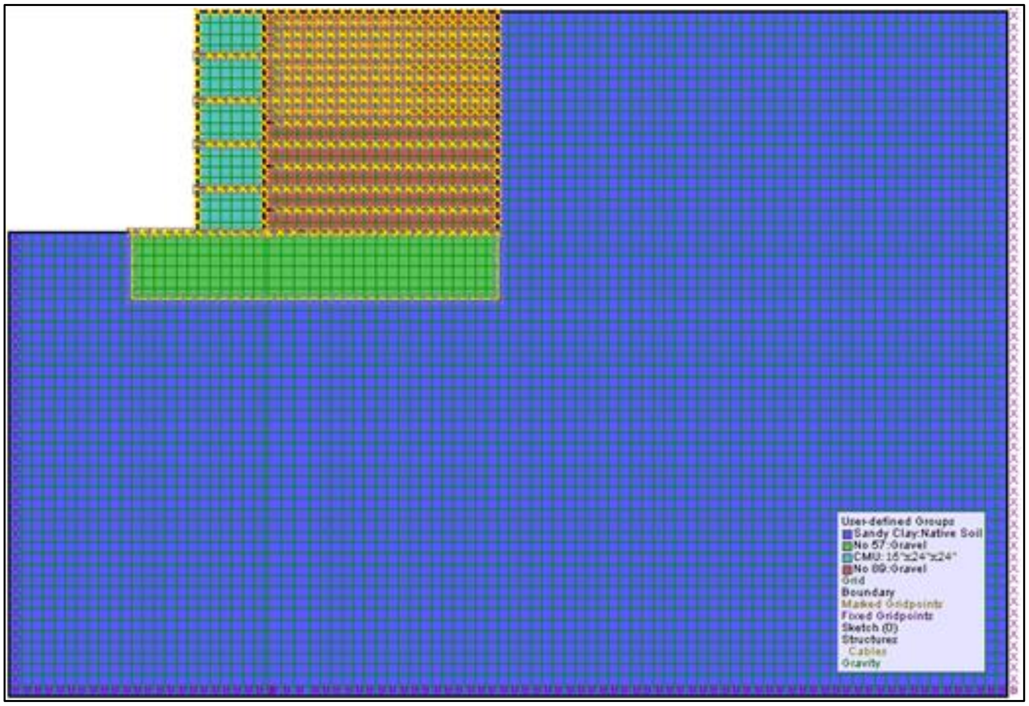
Solid CMU (facing block) properties	
Young's Modulus, E (MPa)	20,000
Poisson's ratio,	0.2
Density, (kg/m^3)	2,240
Bulk Modulus, K (MPa)	11,111
Shear Modulus, G (MPa)	8,333

Table 53: Static loading conditions (applied pressure) in GRS-IBS numerical (FLAC) models (1 kPa = 20.89 psf; 1 kgf/m^3 = 0.062 pcf)

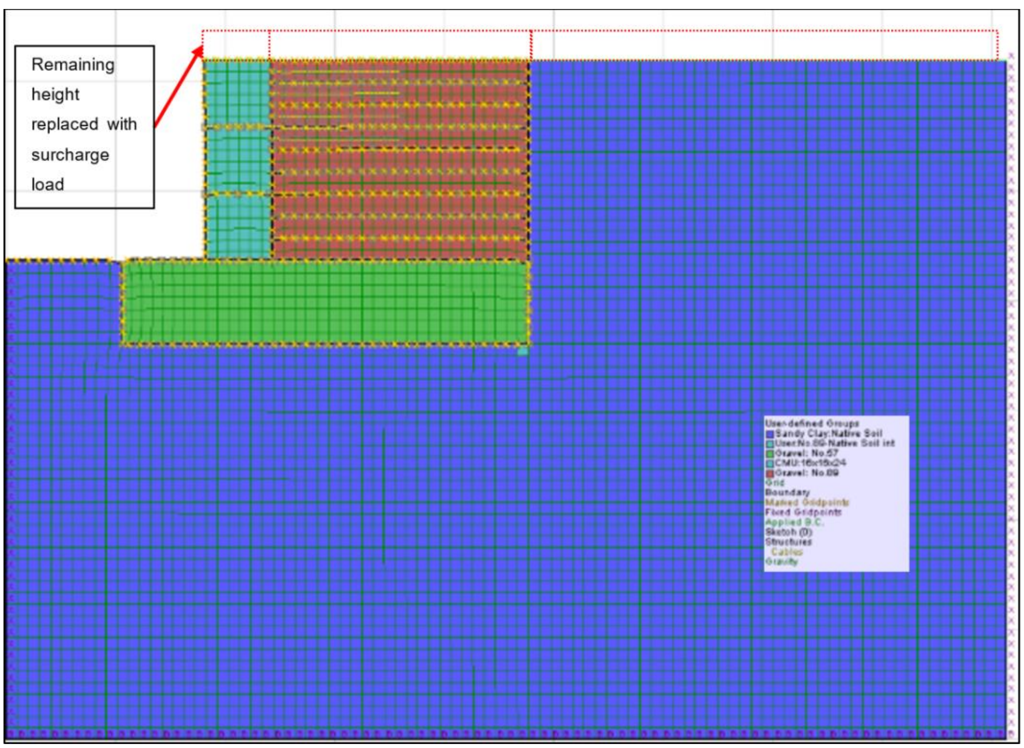
	Beam Bridge	Approach Roadway Equivalent static load, σv
End of Construction (kPa)	65.32	18.8
Traffic Surcharge Load (kPa)	78.52	32.0



(a)



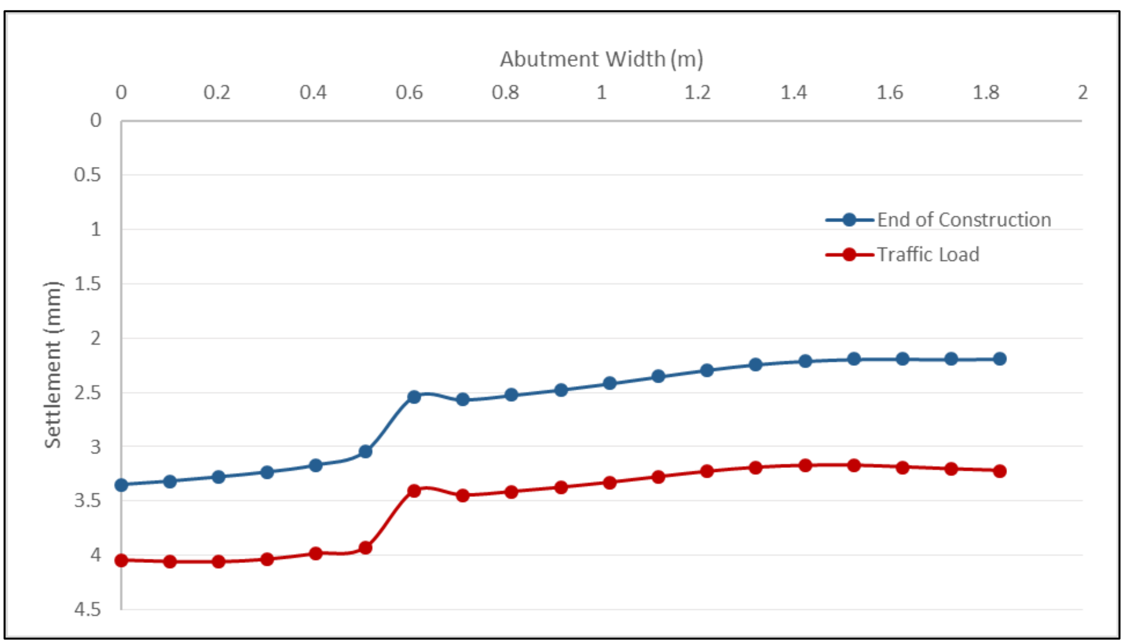
(b)



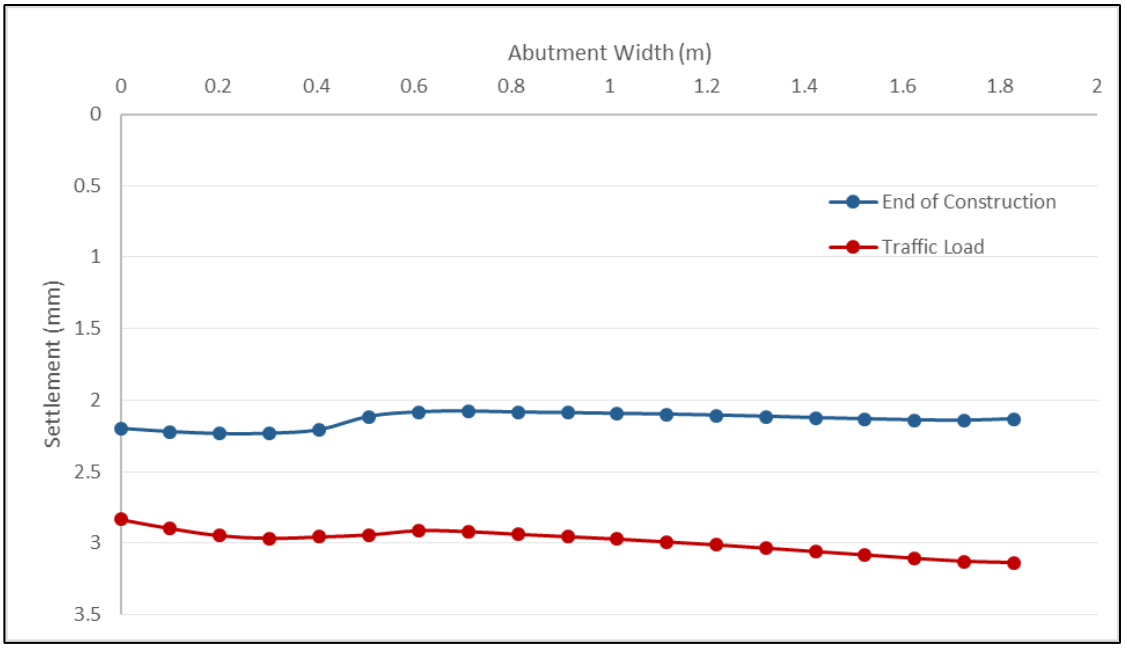
(c)

Figure 110: GRS-IBS FLAC models: (a) 8 in. × 8 in. × 8 in. CMU block facing; (b) 16 in. × 24 in. × 24 in.; and (c) 24 in. × 24 in. × 48 in. 'Dolese' block facing (1 inch = 25.4 mm)

Figure 111 and **Figure 112**, respectively, show predicted settlements at the top and lateral deformations of the GRS-IBS abutment models at the end of construction, and when subjected to a 13.2 kPa (280 psf) uniform surcharge (0.61 m ~ 2 ft of soil) on the top of the abutment representing traffic load, as suggested by FHWA design guidelines (Berg et al. 2009). The numerical models and the corresponding predicted results await future validation. Nevertheless, the predicted results suggest that the performance of the GRS-IBS abutments with both facing types is expected to be satisfactory with relatively small settlements and lateral deformations. The maximum lateral movement of the model with 8 in. × 8 in. CMU facing is predicted to be approximately 3 mm (1/8 in) at one-third of the abutment height from the top, while the corresponding value for the model with 16 in. × 24 in. block facing is negligible. However, the maximum settlements of both models are predicted to be similar (~3 mm = 1/8 in).

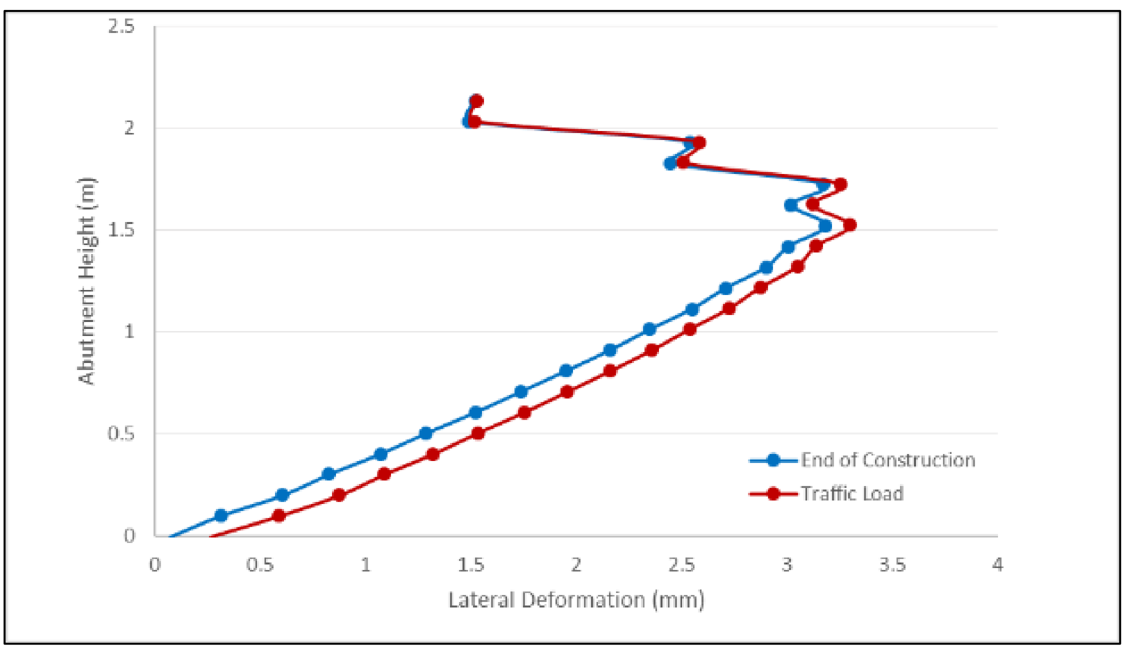


(a)

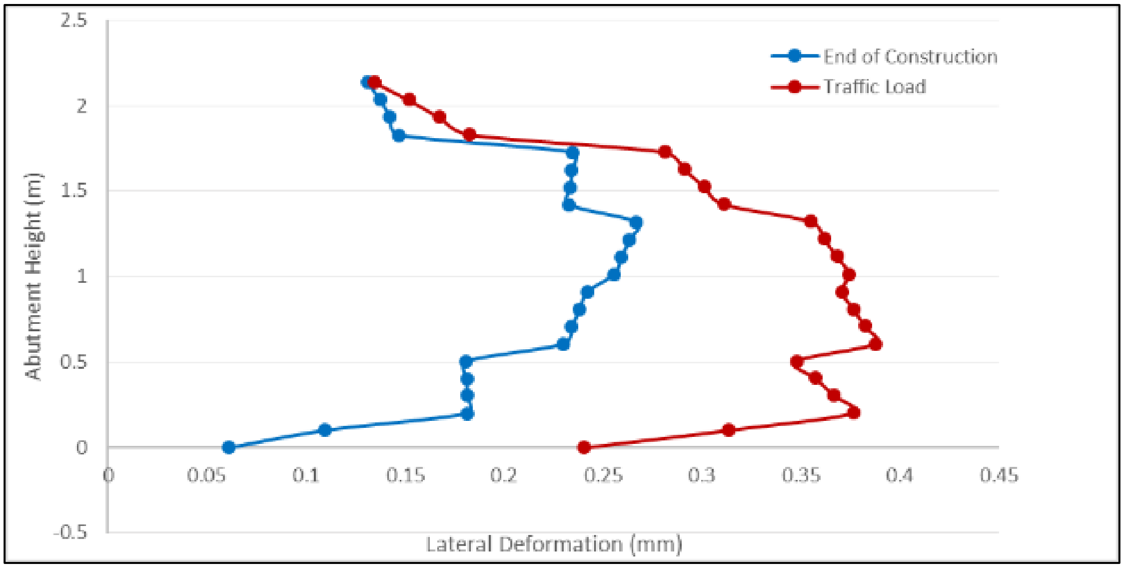


(b)

Figure 111: Predicted bridge settlements at the end of construction and under equivalent traffic load in numerical models with: (a) 8 in. × 8 in. CMU block facing; (b) 16 in. × 24 in. block facing ;(1 inch = 25.4 mm)



(a)



(b)

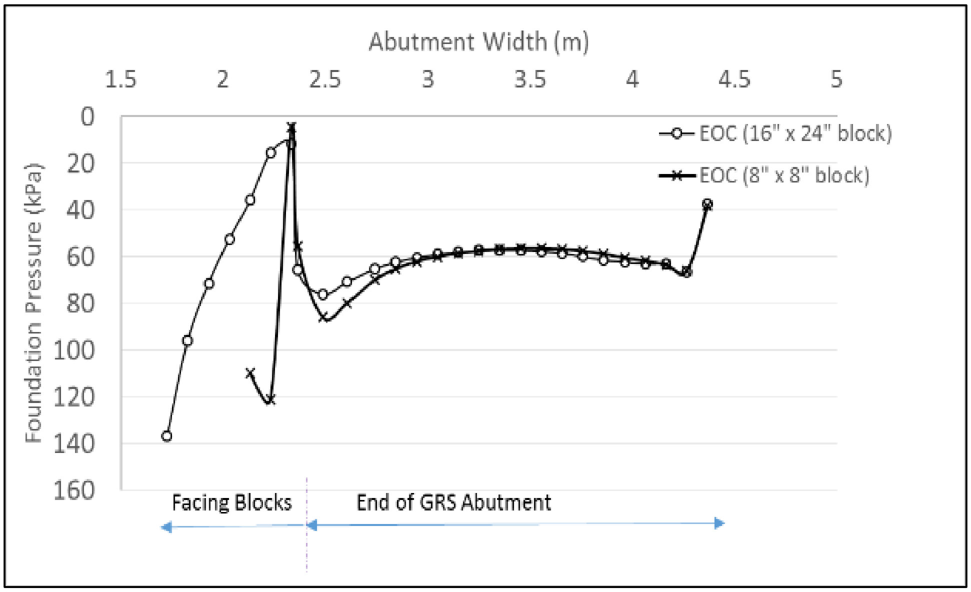
Figure 112: Predicted facing deflections at the end of construction and under equivalent traffic load in numerical models with: (a) 8 in. x 8 in. CMU block facing; (b) 16 in. × 24 in. block facing; ;(1 inch = 25.4 mm)

Foundation pressure and reinforcement load are important factors in GRS-IBS design. One potential concern with the use of large (16 in. × 24 in. × 24 in.) blocks as GRS facing is the bearing pressure on the subgrade soil underneath the block column, which was explored using the FLAC model. On the other hand, it was hypothesized that larger facing blocks would result in reduced reinforcement load, which was also examined in the numerical model. **Figure 113** shows predicted foundation pressure at the bottom of the GRS abutment models at the end of construction and under equivalent traffic load for GRS models with the control and larger concrete blocks. Results indicate that predicted foundation pressures in the two models are comparable. It can also be observed that in both models, pressure distributions in the foundation show increased magnitudes at the toe of the facing and reduced magnitudes toward its back due to down-drag forces at reinforcement connections in addition to an overturning tendency of the facing column away from the GRS backfill. Similar observations have been reported in previous studies (e.g. Hatami and Bathurst 2005).

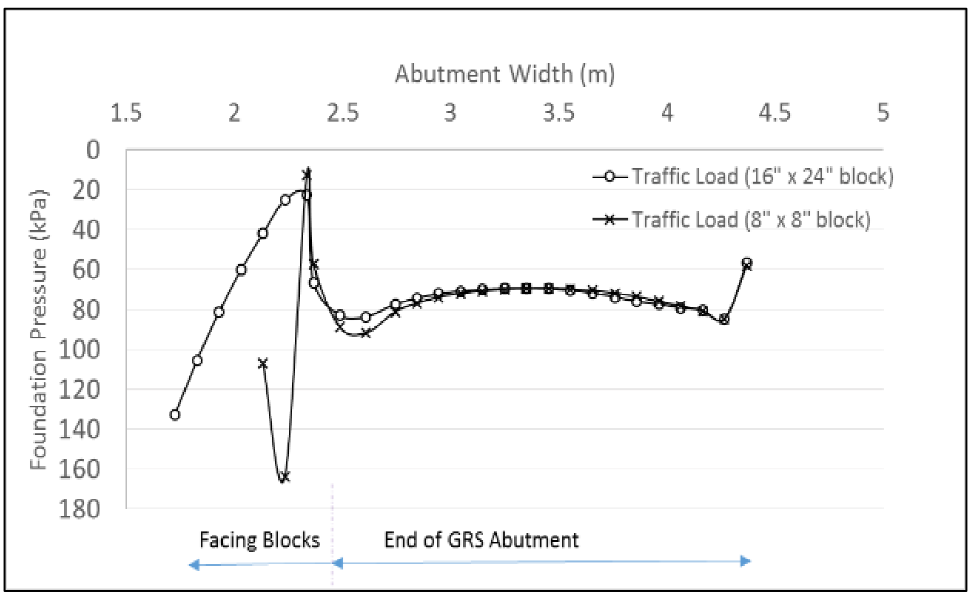
Figure 114 shows predicted axial loads in the reinforcement behind the facing in the two GRS models. The reinforcement was modeled as a series of cable elements (geotextile) that can only resist tension. Data shown in **Figure 114** indicate the following important and interesting results: (1) reinforcement loads are significantly larger in the case of the GRS abutment with smaller (standard) CMU blocks, (2) in both models (i.e. regardless of the size of blocks used in the facing), reinforcement layers that are connected to the facing blocks carry significantly larger connection loads than those terminated immediately behind the facing, and (3) regardless of the block size, significantly larger loads develop toward the top of the GRS mass underneath the bridge abutment in the reinforcement layers that are connected to the facing whereas the distributions of reinforcement load over the height of the GRS mass are more uniform in the layers that are not connected to the facing, especially in the model with large facing blocks, indicating its potential for a more optimum design.

These findings are exciting in that they point to the merit of large-block facing construction in GRS-IBS projects in Oklahoma and other states as per Mr. Sheffert and PI’s early discussions leading to the present work. They also provide added incentive for future work to look for possible field projects to compare the performance of GRS-IBS projects with different facing construction in addition to measuring reinforcement loads in instrumented cases to validate these numerical simulation results in the future.

These efforts will help develop adjustments to the current FHWA guidelines (Adams et al. 2012) for GRS-IBS construction that could lead to more economical design by using large-block facing alternatives.

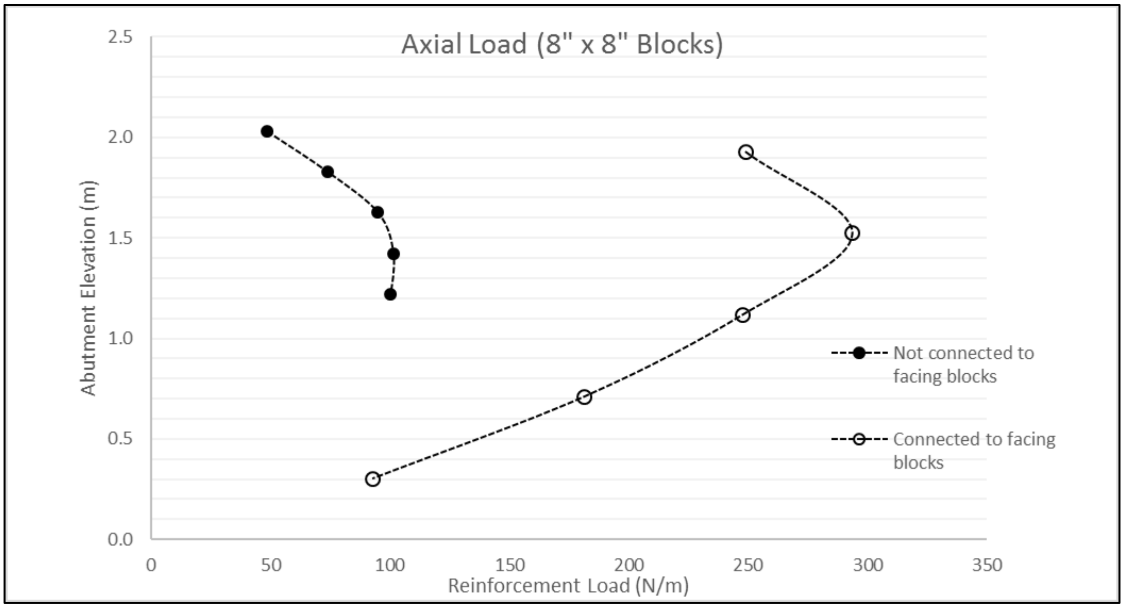


(a)

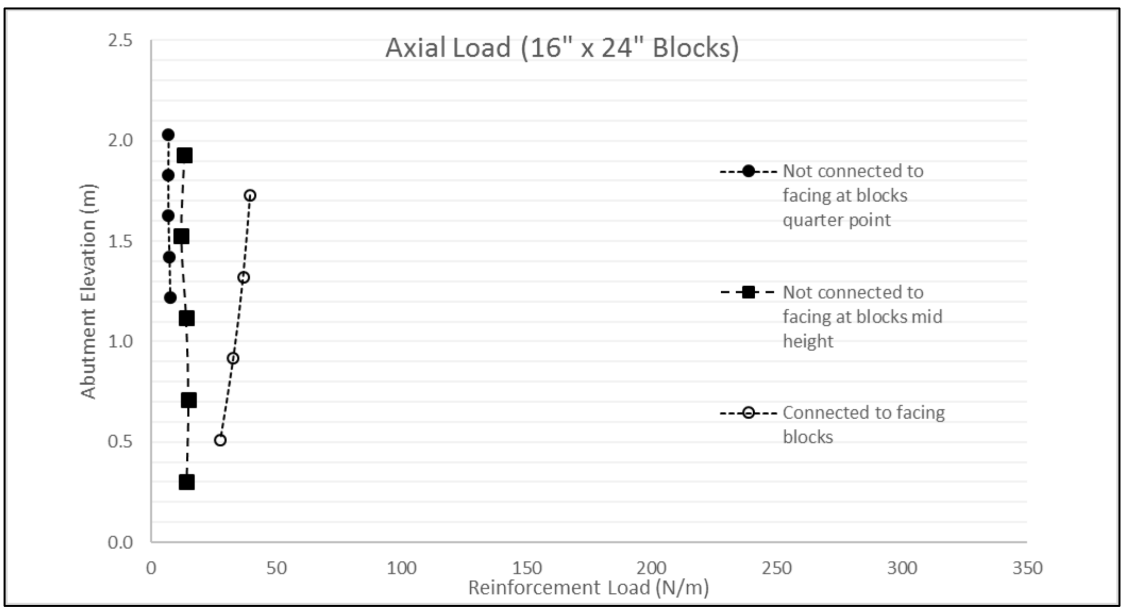


(b)

Figure 113: Comparison of the predicted foundation pressure in the numerical models with 8 in. × 8 in. CMU and 16 in. × 24 in. block facing: (a) end of construction (Stage 6); (b) Under traffic load (Stage 7) using 280 psf (13.2 kPa) equivalent surcharge (100 kPa = 2.1 tsf)



(a)



(b)

Figure 114: Predicted reinforcement loads in the GRS abutment models when subjected to equivalent traffic load: (a) Model with 8 in. × 8 in. CMU block facing; (b) Model with 16 in. × 24 in. CMU block facing (1 lb/ft = 14.6 N/m)

The influence of block size used in the facing on the predicted GRS performance was further examined through a FLAC numerical model with 24 in. x 24 in. facing blocks. Distributions of earth pressure in the GRS fill, reinforcement axial load, settlement at the top of the GRS fill and facing deformation were examined using the numerical model. Example snapshots of model results for end of construction and when subjected to the bridge dead load (Stages 4 and 6, respectively; **Figure 106**) are shown in **Figure 115** and **Figure 116**, respectively.

Maximum reinforcement loads in the GRS abutments with the 8 in. × 8 in. and 24 in. × 24 in. blocks are 299 N/m (20.48 lb/ft) and 214.4 N/m (14.68 lb/ft), respectively (**Figure 116a**) indicating a greater structural contribution of the larger block facing. These values can be compared with the allowable strength (T_{all}) of the geotextile reinforcement used in Bridge No. 2 in Kay County, OK, and calculate a factor of safety using selected values of partial reduction factors (Koerner 2005) as the following:

$$T_{all} = \frac{T_{ult}}{FS * \prod RF}; \;\; FS = \frac{T_{ult}}{T_{all} * \prod RF}$$

$where$:

$$T_{ult} = Wide\ Width\ Tensile\ Strength\ at\ 2\%\ elongation\ (Cross\ Machine\ Direction)$$

$$T_{ult} = 19{,}300\,\frac{N}{m} \approx 1{,}322\,\frac{lb}{ft}\ (\textbf{Table 5})$$

$$\prod RF = RF_{Intallation\ Damage} \times RF_{Creep} \times RF_{Chemical\ \ /\ biological\ degradation}$$

$$\prod RF = \ 2\times4\times1.5; \ \prod RF = 12$$

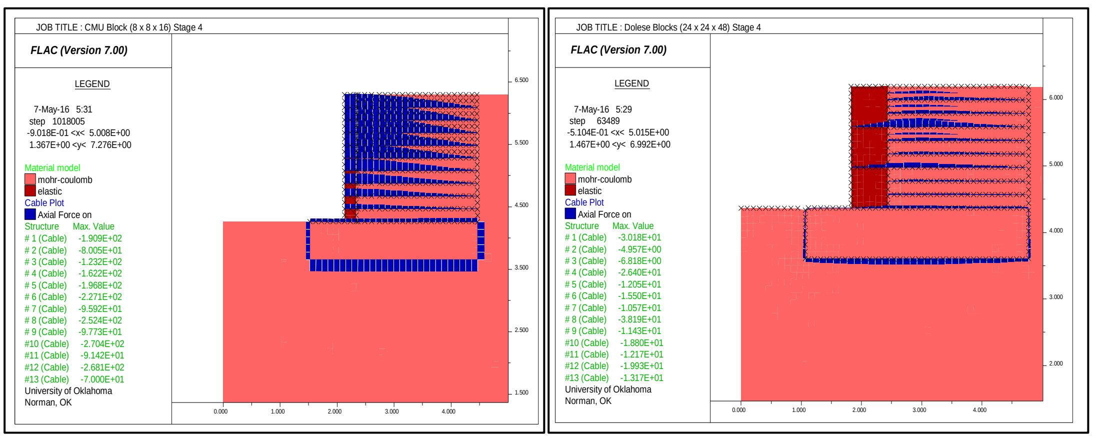
$$T_{all} = \frac{19{,}300\,\frac{N}{m}}{12}; \ T_{all} = 1{,}609\,\frac{N}{m} \approx 110.2\,\frac{lb}{ft}$$

$$FS_{8\,in.x\,8\,in.} = \frac{110.2\frac{lb}{ft}}{20.48\frac{lb}{ft}};\ FS_{8\,in.x\,8\,in.} = 5.4$$

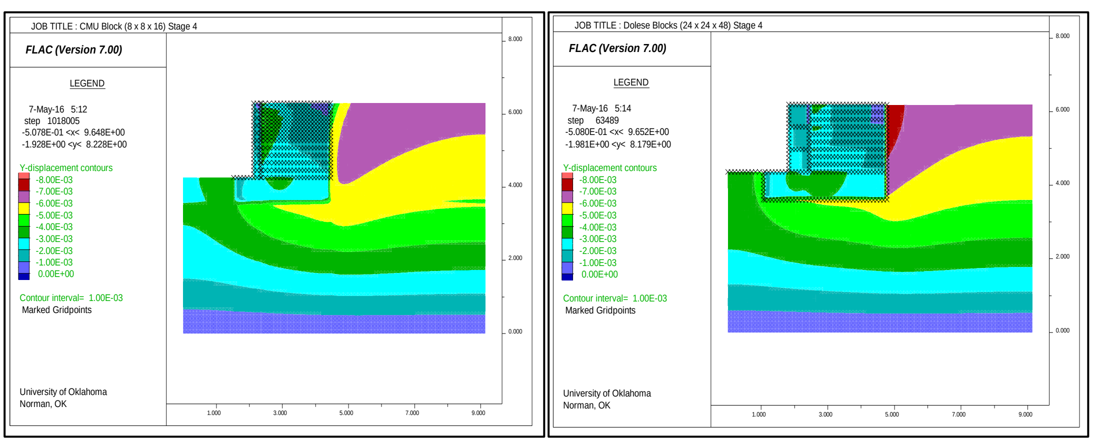
$$FS_{24\,in.x\,24\,in.} = \frac{110.2\frac{lb}{ft}}{14.68\frac{lb}{ft}};\ FS_{24\,in.x\,24\,in.} = 7.5$$

Predicted settlements in the Reinforced Soil Mass (RSM) in **Figure 116b** is also slightly larger in the model with 8 in. × 8 in. facing units relative to those with 24 in. × 24 in. facing blocks. However, settlement contours below the Reinforced Roil Foundation (RSF) in the two models appear to be comparable. Finally, slightly larger facing deformations are predicted for the model with larger blocks (i.e. 12 mm vs. 10 mm; **Figure 116c**), which could be attributed to the fact that a larger number of reinforcement layers' interlock with the blocks in the 8 in. × 8 in. model resulting in a stiffer RSM-block structure as compared to the 24 in. × 24 in. model. Nevertheless, predicted facing deformations in both models are limited to approximately 10 mm ≈ 3/8 in., which can be considered acceptable. It should be noted that FHWA guidelines (Adams et al. 2012) state that: "Since the facing element is not structural in a GRS wall or abutment, any facing element can be used". In addition, no maximum allowable facing deformations have so far been specified in the said guidelines.

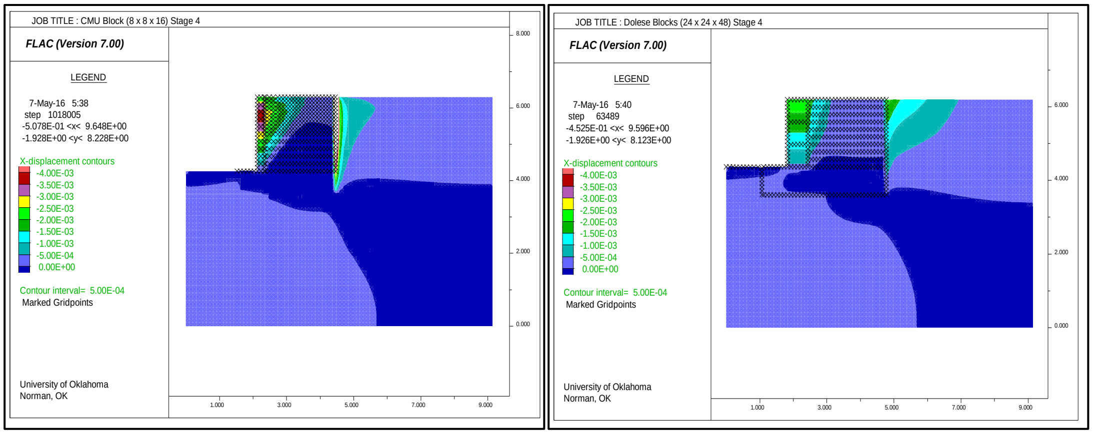
Figure 117 and **Figure 118** show predicted distributions of axial load and axial strains in the reinforcement behind the facing in the two GRS models with different size facing blocks. Data shown in **Figure 118** confirm the following important and interesting results we obtained in an earlier comparative study involving different size blocks: (1) reinforcement loads are significantly larger in the case of the GRS abutment with smaller (standard) CMU blocks, (2) in both 8 in. × 8 in. and 24 in. × 24 in. block GRS models (i.e. regardless of the size of blocks used in the facing), reinforcement layers that are connected to the facing blocks carry significantly larger connection loads than those terminated immediately behind the facing, and (3) regardless of the block size, significantly larger loads develop toward the top of the GRS mass underneath the bridge abutment in the reinforcement layers that are connected to the facing whereas the distributions of reinforcement load over the height of the GRS mass are closer to uniform in the layers that are not connected to the facing, especially in the model with large facing blocks, indicating its potential for a more optimum design.



(a)

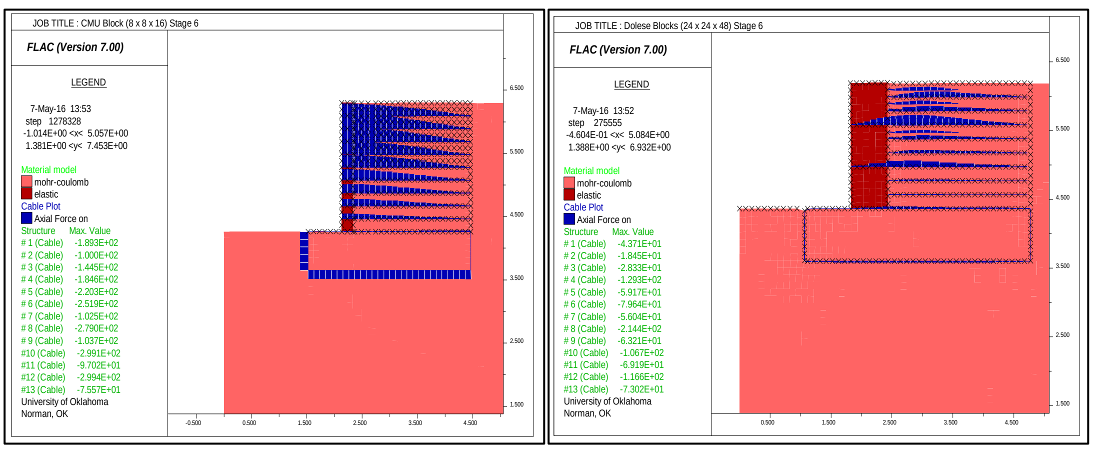


(b)

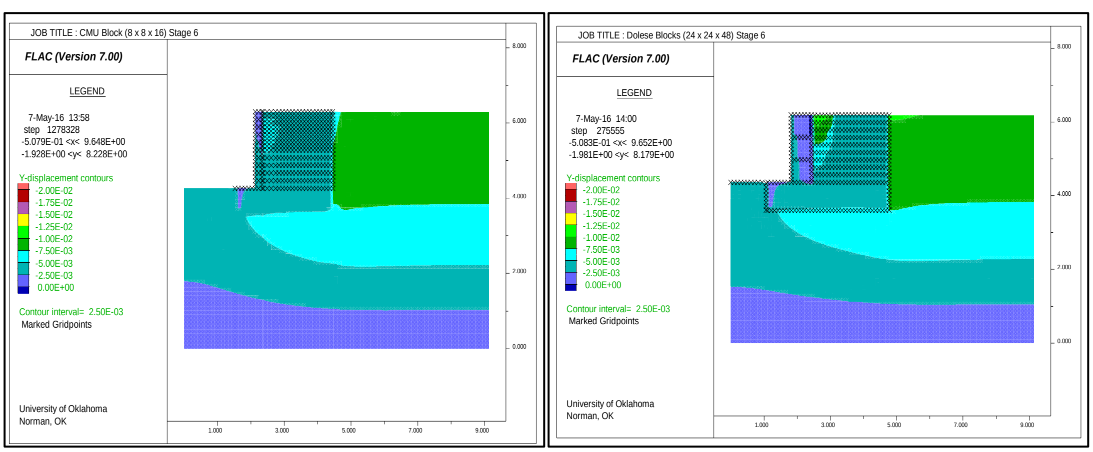


(c)

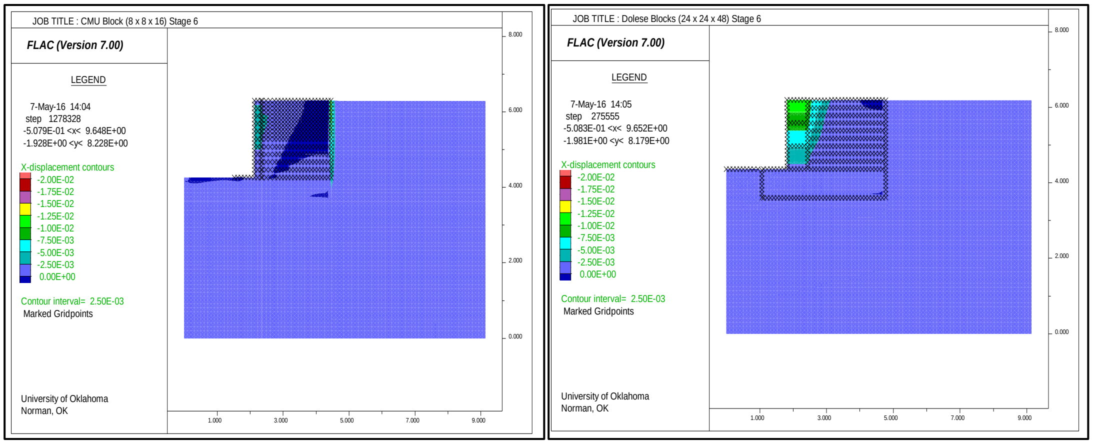
Figure 115: Comparison of GRS-IBS numerical models with 8 in. × 8 in. CMU and 24 in. × 24 in. block facing at the end of construction (Stage 4) and subjected to 65.32 kPa (1,365 psf) bridge load (Stage 6): (a) reinforcement load (N/m); (b) settlement contours (m); and (c) contours of lateral deformation (m)



(a)

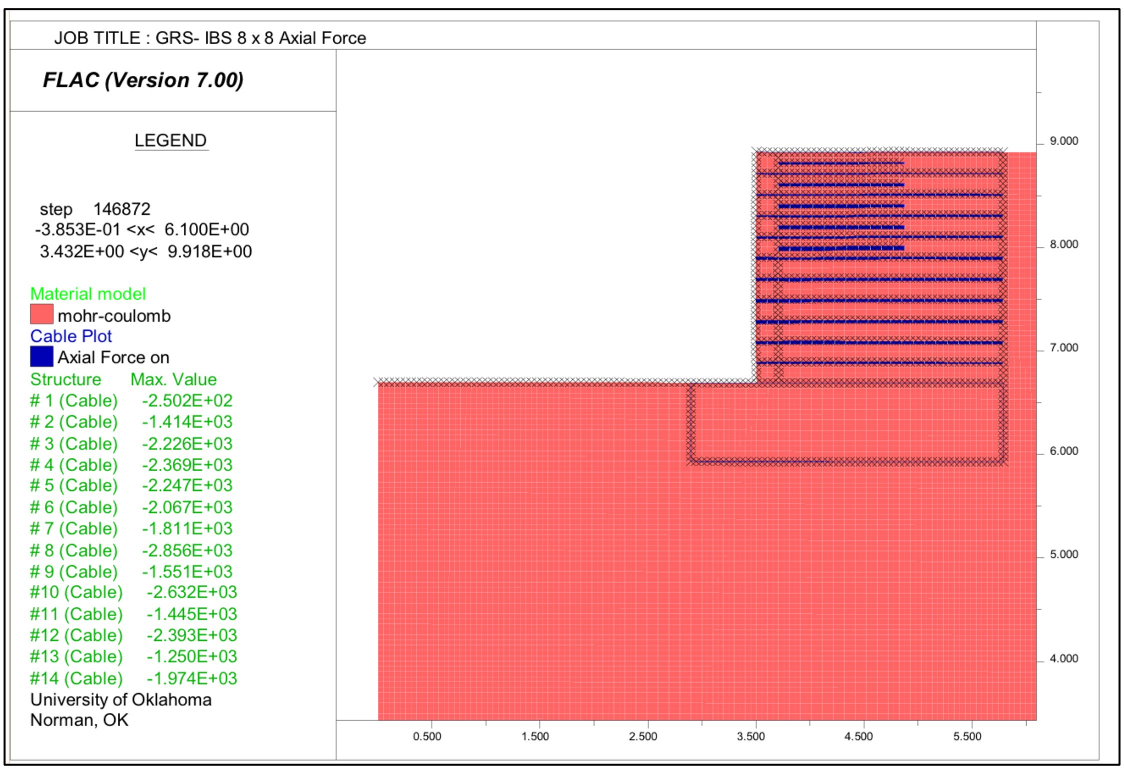


(b)

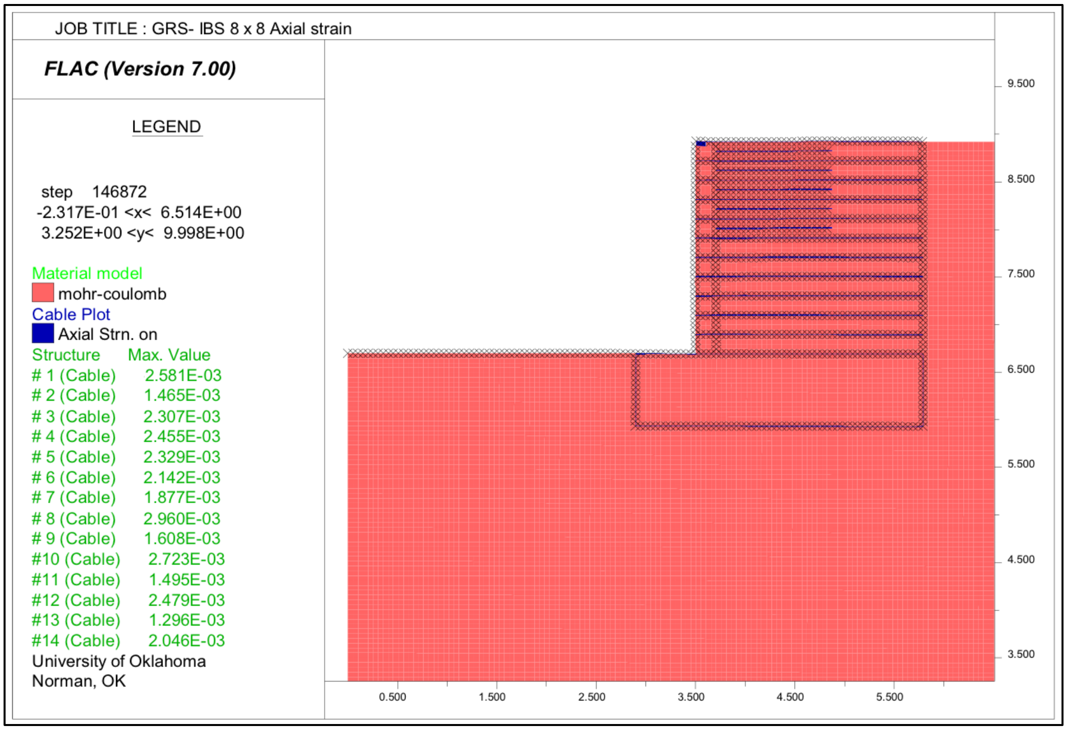


(c)

Figure 116: Comparison of GRS-IBS numerical models with 8 in. × 8 in. CMU and 24 in. × 24 in. 'Dolese' block facing at the end of construction (Stage 6) and subjected to 13.2 kPa (280 psf) traffic load: reinforcement load (N/m); (b) settlement contours (m); and (c) contours of lateral deformation (m)



(a)



(b)

Figure 117: Numerical model of GRS-IBS abutments for Bridge No. 2 in Kay County: (a) predicted axial loads and (b) predicted axial strains

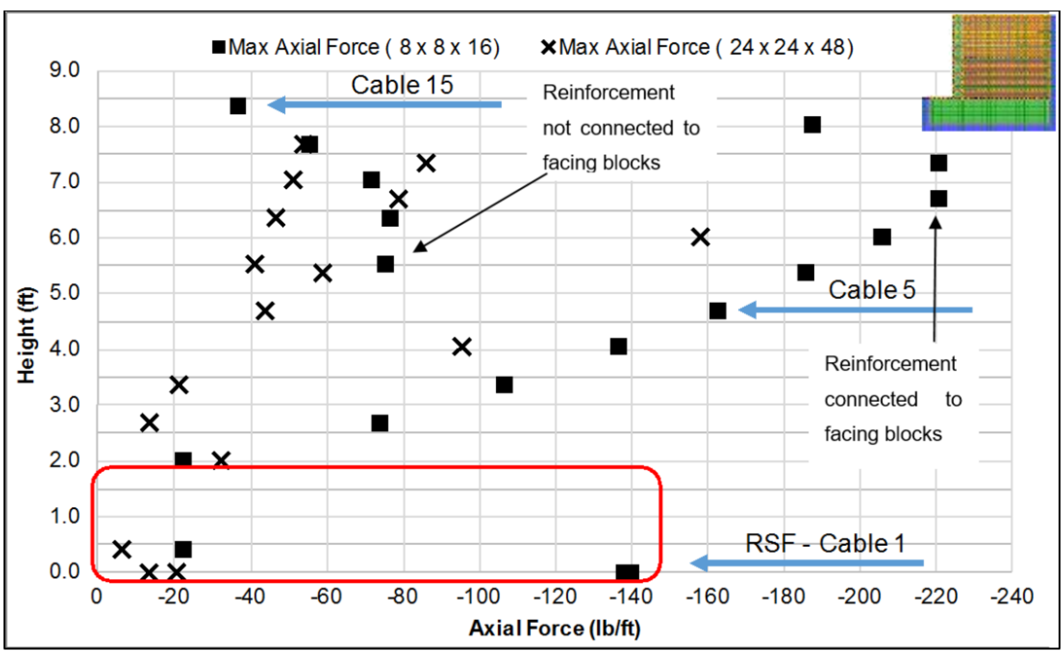


Figure 118: Comparison of predicted axial loads in GRS-IBS numerical models with 8 in. × 8 in. CMU and 24 in. × 24 in. facing at end of construction and subjected to service load. Loading applied in the model includes equivalent uniform surcharge load due to traffic (Stage 7) **Figure 107**

Chapter 6. Conclusions and Recommendations for Future Work

6.1 Summary and Conclusions

GRS-IBS technology has a track-record of field projects across the nation. Although, to date only 250 bridges have been recorded it is estimated that currently, a greater number have been completed. In this study, a database was developed that includes a wide range of data on 144 GRS- IBS projects in the U.S. on which at least some basic information was available. The bridges documented in the database include those from 82 different counties in 44 states. GRS-IBS is an alternative to conventional bridge systems, especially useful in low-volume roads that can be taken as a solution for replacement of many outdated bridges in the nation. The information on the surveyed bridges included their geographical locations, size, geometry and other related design information, geotechnical, hydraulic and traffic data, types of superstructure, facing wall, backfill material and geosynthetic used, performance monitoring methods/results, and feedback from the corresponding local agencies.

The main benefits of GRS-IBS are: (1) Cost reduction of the bridge construction and maintenance, saving up to 62% compared to standard technologies, (2) Fast construction given the fact bridges built with this technology don’t required complex equipment or materials, (3) Flexible design that can be easily adapted according the environment or design requirements, and (4) Elimination of the bump at the end of the bridge which allows a smoother transition between road and bridge and decreases the need for frequent maintenance of bridge roads. The database is used to summarize a set of suitable criteria with respect to the items above for future GRS-IBS projects in Oklahoma.

FHWA guidelines (Adams et al. 2012) on GRS-IBS are extremely conservative. Recommended limits for eligible projects are those with low traffic volume, single span of no more than 42.7 m (140 ft.) and 9.1 m (30 ft.) in abutment height. Although most bridges covered in this report follow these guidelines, several other documented projects have surpassed them and proven to have a good performance (e.g. Bridge No. 169 in Knox County, ME; Ngo 2016). Numerous DOT's reported learned lesson of the GRS-IBS experience. California reported a successful technical challenge, due to its geographic location with an elevation of approximately 2,744 m (9,000 ft.) and seismic peak ground acceleration between 0.40g and 0.50g. Delaware reported three main good practices during construction: (1) Accurate and proper placement of the first layer of the facing block is very important, (2) The first layer must be level, straight, and plumb, and (3) If the edges of the blocks are too smooth, they can slide easily over each other. Therefore, an inward slope (batter) is necessary to accommodate movement. Iowa reported the experience as "successful" based on the following outcomes: (1) Construction costs were 50% - 60% lower than what would be expected for a conventional bridge, (2) Measured settlements were less than 2 cm (0.7 in) and differential settlements were less than 5.8 mm (0.2in), and (3) The projects confirmed the ease and reduced time of construction, and reduced material and labor costs. New York concluded that GRS-IBS construction results in a shorter construction schedule as well as cost savings in materials, labor and equipment. It is also more adaptable, less weather sensitive, less prone to settlement and eliminates the bump and crack at the bridge approach.

From the documented information of the surveyed bridges, we can conclude: (1) The most common facing type is the CMU blocks (8 in. × 8 in. × 16 in.) with 52% followed by sheet piles with 6%, (2) The most used monitoring instrument is the conventional surveying with 35%, pressure cells with 20% and inclinometers with 11%, and (3) The highest reported savings in a project were in Olympic Avenue, IA with 53% -62% of cost savings (Ngo 2016). Additionally, the state of Pennsylvania reported a per square foot abutment cost analysis with an average cost of $95.5 (GRS) vs $208.5 (traditional). All this information was initially compiled in the reported database, which evolved into the development of a project website. This tool will contain design features, good practices and improvement potentials that will be very useful for designing.

In Oklahoma, the set of four GRS-IBS bridges in Kay County together with two other bridges with conventional driven pile support systems all with a 1-mile segment of 44th street in Kay County provided a unique opportunity for this study. The bridge essential information as weather, traffic, geotechnical reports and construction phase were documented in this project. Essentially, all six bridges are subjected to the same conditions. Furthermore, all of them were built by the same crew and GRS-IBS were reported to be more cost effective than their conventional counterparts. Performance monitoring was conducted during this project. Is worth noting that during the monitoring period, the bridges were subject severe weather and seismic events (e. g

April 2015 flooding; Sept 2016 earthquake). The results show, that so far, all the bridges have outperformed. GRS-IBS Bridge No. 2 bridge deck has undergone a seemingly consistent and predominantly upward movement between 6.35 mm and 12 mm(¼ and ½ in.) during this monitoring period. Conventional Bridge No. 6 shows approximately 2.54 cm (one (1) in.) of nearly uniform settlement in both the north and south abutments. However, this unexpected result may be attributed to fact that during construction stage the steel piles were not drive to bedrock (Simpson 2016). Also, the bridge has been re-graded several times because a slightly "bump" at the joint appeared. Nonetheless, this bridge has not shown any visible signs of major serviceability or aesthetics-related problems. Furthermore, baselines were monitored for conventional Bridge No.1 and GRS-IBS Bridges No. 3, 4 and 5. Further monitoring of the bridges movement should determine the validity and accuracy of the initial findings.

The develop of a numerical model helped investigate the influences of select design factors such as size of facing blocks. A computer program Fast Lagrangian Analysis of Continua (FLAC; Itasca 2005) was used to develop the numerical models based on the as-built geometry and construction details of Bridge No. 2 in Kay County and material properties that were either tested or otherwise obtained during this study. Parametric study was carried out which showed: (1) The performance of the GRS-IBS abutment under static loading conditions (e.g. end of each construction stage) is expected to be satisfactory with relatively small deflection. The maximum lateral movement at the end of construction and including the approach roadway (Stage 6) is predicted to be approximately 7 mm (0.27 in.) at slightly above mid-height of the facing wall.

Predicted maximum facing deformation and bridge settlement under traffic loading are 6.1 mm and 7 mm (0.24 in. and 0.26 in.), (2) Predicted deformations of the GRS-IBS abutments in this study are not sensitive to the interface shear stiffness properties assumed in the analysis within the range of values examined, (3) Predicted maximum settlements consistently occur under the abutment beam seat. (4) maximum values of bridge settlement and facing deformation for the most critical case of Sv= 16 in and ϕ= 34o, are limited to 5/16 in. and 5/8 in. (8 mm and 15 mm), respectively. Therefore, the performance of GRS abutments with different combinations of reinforcement spacing and friction angle values in this study could be considered as satisfactory. However, these results need to be validated in the field before they can be applied in practice, and (5) Reinforcement loads are significantly larger in the case of the GRS abutment with smaller (standard) CMU blocks. In both 8 in. × 8 in. and 24 in. × 24 in. block GRS models (i.e. regardless of the size of blocks used in the facing), reinforcement layers that are connected to the facing blocks carry significantly larger connection loads than those terminated immediately behind the facing. Regardless of the block size, significantly larger loads develop toward the top of the GRS mass underneath the bridge abutment in the reinforcement layers that are connected to the facing whereas the distributions of reinforcement load over the height of the GRS mass are closer to uniform in the layers that are not connected to the facing, especially in the model with large facing blocks, indicating its potential for a more optimum design. Further development of the numerical model together with its more rigorous validation can lead to a useful tool for GRS-IBS design and their more widespread acceptance in the U.S. and internationally.

6.2 Recommendations for Future Work

The GRS-IBS database needs to be expanded and maintained periodically. Additionally, it is important to survey more bridges with reported performance monitoring. More research in the documented bridges shall be done in order to complete the current database with missing information as span length, height, superstructure system, facing type and construction methods. This information will help in future designs across the U.S.

The web-page needs to be completed and updated periodically. It is also recommended to implement an auto-update feature. This feature can be based in forms that once submitted will automatically add information to the database. Thus, when the webpage is refreshed the new information will be shown.

Further monitoring needs to be performed in Bridges 1, 3, 4 and 5. These bridges which were initially monitored in this project, can be compared to the reported performance of Bridges No. 2 and No. 6 in Kay County, OK.

Lastly, it is recommended to perform additional developing of the numerical model together with its more rigorous validation based on the results obtained by the performance monitoring. This can lead to a useful tool for GRS-IBS design.

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