**Implementation of Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS) Technology in Alabama**


by

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A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
August 8th, 2020


Keywords: GRS-IBS, accelerated bridge construction, geosynthetics, performance monitoring




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## ABSTRACT

Geosynthetic reinforced soil-integrated bridge systems (GRS-IBS) utilize mechanically stabilized earth (MSE) systems to support single-span bridges. A typical GRS-IBS is made up of three components: the reinforced soil foundation (RSF), geosynthetic reinforced soil (GRS) abutments, and the integrated approach. Each abutment sits directly atop a reinforced soil foundation, which consists of compacted granular material encapsulated in woven geosynthetic. However, a concrete pad can be constructed in lieu of a traditional RSF if the native underlying material is competent rock or very dense sand. Abutments are constructed from the ground up; open-graded or well-graded gravel is placed in lifts, compacted, and overlain by layers of geosynthetic material. This process is repeated until desired roadway elevation is met. Closely spaced layers (typically less than 12 in.) and internal reinforcement that is frictionally (rather than mechanically) attached to the facing material are two unique features that distinguish GRS structures from traditional MSE walls. The integrated approach blends the bridge superstructure into the surrounding geology, resulting in a smooth transition between the roadway approach and the bridge pavement.

This relatively new technology provides a cost-effective alternative to traditional bridge foundation systems, as well as ease of constructability. In 2011 the Federal Highway Administration created an Every Day Counts Initiative to accelerate implementation of these bridges. GRS-IBS has been successfully implemented in several states, with Alabama being the latest. Alabama’s first GRS-IBS was constructed in Marshall County, spanning across Turkey Creek in the northeast area of the state. The robust sandstone geology and low scour potential of

this site provided a conservative option for a test case study. Two 12 ft. tall, 33 ft. wide abutments support seven 1.75 ft. thick, 4 ft. wide, 52 ft. long reinforced concrete beams, pavement, and traffic. Earth pressure and pore-water- pressure vibrating-wire sensors were installed within the abutments, and reflective prisms were placed on the corners of the abutments to monitor lateral and vertical displacement. Earth pressures reached 1800 psf after the concrete beams were placed, and pore-water-pressure has remained near zero- signifying no significant buildup of pore water pressure due to flooding or rainfall within the abutment. Periodic surveys showed that settlement (z) and lateral movement (x,y) of the bridge were minimal. Information pertaining to typical building practices and construction specifications were gathered from multiple state Departments of Transportation. This information was compiled into a draft Special Provision for the Alabama Department of Transportation.

# ACKNOWLEDGEMENTS

I would like to thank Dr. Brian Anderson for his constant guidance, patience, and for helping me find an awesome job. I would also like to thank Dr. Jack Montgomery and Dr. Wesley Donald for their willingness to join my committee. Additionally, I'd like to thank ALDOT, Marshall County, and Bob Pirando, P.E. for their participation in this project which has funded my graduate education. This paper draws heavily from GRS-IBS literature published by Mike Adams (Federal Highway Administration), Jonathan Hogan (Wood PLC), Walter Turnbull (GeoStabilization International), and Dr. David Elton (Auburn University). The following graduate students deserve recognition for their efforts towards this project: Michael Kiernan, Dan Jackson, Ashton Babb, Chukwuma Okafor, and Frank Russell. Finally, I'd like to thank my family for their everlasting support. Above all, I'd like to thank God for blessing me with the abilities to complete this task.

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| | |
|---|---|
| $a_b$ | Setback Distance |
| ALDOT | Alabama Department of Transportation |
| $\alpha$ | Angle in Radians (Equation 6) |
| $\beta$ | Angle in Radians (Equation 7) |
| b | Bridge Seat Bearing Width |
| $B_b$ | Width of Bridge |
| BM | Benchmark |
| $b_{rb,t}$ | Width Over the Abutment Which the Dead Load Acts |
| $B_{total}$ | Base Width |
| de | Clear space |
| DL | Dead Load |
| $d_{max}$ | Maximum Aggregate Size |
| ° | Degrees |
| DOT | Department of Transportation |
| ECP | East Control Point |
| EDC | Every Day Counts |
| EOR | Engineer of Record |
| EPC | Earth Pressure Cell |

| | |
|---|---|
| $F_b$ | Driving Force From the Retained Backfill |
| FE | Finite Element |
| FHWA | Federal Highway Administration |
| $F_{rb}$ | Driving Force From Road Base |
| $F_t$ | Driving Force From Live Load Surcharge |
| Ft | Foot |
| $\gamma_b$ | Unit Weight of the Retained Backfill |
| $\gamma_r$ | Unit Weight of the Reinforced Fill |
| GRS | Geosynthetic Reinforced Soil |
| GRS-IBS | Geosynthetic Reinforced Soil – Integrated Bridge System |
| $>$ | Greater Than |
| $\geq$ | Greater Than or Equal To |
| H | Height of Wall |
| HI | Height of Instrument |
| IBS | Integrated Bridge System |
| $\phi$ | Internal Angle of Friction |
| IM | Impact Allowance |
| in | Inch |
| LaDOTD | Louisiana Department of Transportation and Development |

| | |
|---|---|
| lb | Pound |
| $<$ | Less Than |
| $\leq$ | Less Than or Equal To |
| $K_a$ | Rankine Active Earth Pressure Coefficient |
| $K_{ar}$ | Rankine Active Earth Pressure Coefficient of the Reinforced Fill |
| $K_{ab}$ | Rankine Active Earth Pressure Coefficient of the Retained Fill |
| $K_p$ | Ranking Passive Earth Pressure Coefficient |
| LL | Live Load |
| $(LL+IM)_{total}$ | Governing Abutment Reaction for One Lane |
| $L_{span}$ | Bridge Span Length |
| M-C | Mohr-Coloumb |
| MD | Machine Direction |
| MSE | Mechanically Stabilized Earth |
| $N_{lanes}$ | Number of Design Lanes on the Bridge |
| No | Number |
| PI | Plasticity Index |
| $\pm$ | Plus/Minus |
| psf | Pounds per Square Foot |
| psi | Pounds per Square Inch |

| | |
|---|---|
| $q$ | Surcharge Pressure |
| QA/QC | Quality Assurance/Quality Control |
| $q_b$ | Bridge Dead Load |
| $Q_{LL}$ | Live Load Surcharge |
| $q_{LL}$ | Equivalent Distributed Live Load Pressure |
| $q_{rb}$ | Surcharge Due to the Structural Backfill |
| $q_t$ | Roadway Surcharge from Traffic |
| $R_n$ | Resisting Force |
| RSF | Reinforced Soil Foundation |
| SRW | Segmental Retaining Wall |
| $\sigma_{h,q}$ | Surcharge Loading Pressure |
| $\sigma_{h,rb}$ | Structural Backfill of the Integrated Approach Pressure |
| $\sigma_{h,t}$ | Roadway Surcharge Pressure |
| $\sigma_{h,w}$ | GRS Fill Pressure |
| Su | Undrained Shear Strength |
| $S_v$ | Reinforcment Spacing |
| $\mu$ | Interface Friction Angle Between the Soil and Abutment |
| $T_{req}$ | Required Reinforcement Strength |
| UV | Ultraviolet |
| W | Weight of the GRS Abutment per Unit Width |
| WCP | West Control Point |

| | |
|---|---|
| $W_t$ | Total Resisting Force per Unit Width |
| XMD | Cross Machine Direction |
| z | Depth From the Top of the Wall |

# CHAPTER 1: INTRODUCTION

## 1.1 Overview

Geosynthetic reinforced soil-integrated bridge systems (GRS-IBS) utilize a type of mechanically stabilized earth (MSE) wall to support single-span bridges. Mechanically stabilized earth (MSE) walls built with closely spaced layers of geosynthetic reinforcement and compacted granular backfill directly support bridge superstructure. This technology blends the abutment and roadway for a seamless transition, potentially eliminating the “bump at the end of the bridge” (Adams et al. 2011b). These bridges can be constructed using a small workforce, with little impact on the surrounding landscape. Bridge design is easily modifiable and able to accommodate a wide range of geological, hydrological, and environmental conditions.

**Figure 1. Defiance County, Ohio GRS-IBS spanning Tiffin River (Adams et al. 2011a).**

GRS-IBS was created by the Federal Highway Administration (FHWA) during the Bridge of the Future Initiative as a lower-cost design option for single span bridges across the United States (Adams et al. 2011b). Reports have shown GRS-IBS to be 50-60% less expensive to construct than traditional bridge foundations (White et al. 2012). In 2010 the FHWA began an Every Day Counts (EDC) Initiative in an effort to accelerate implementation of GRS-IBS across the United States. Over 200 bridges have been successfully built since 2010 in a variety of unique environments (Daniyarov 2017).

## 1.2 Objective and Scope

The primary objectives of this study were to observe and monitor the performance of Alabama's first GRS-IBS and to draft a Special Provision for the Alabama Department of Transportation (ALDOT). Several tasks were identified to support these objectives, including:

- Measuring pore pressures and earth pressures within the abutments,
- Measuring geospatial data after construction, and
- Obtaining GRS-IBS construction specifications from multiple state Departments of Transportation (DOT), consolidating relevant data from these documents, and compiling the information into an ALDOT Special Provision.

The bridge monitoring program utilized instrumentation and survey equipment. Pore-pressure and earth-pressure sensors were installed in both abutments during construction; fluctuations due to rainfall and changes in surcharge stress were recorded over time. Lateral and vertical displacement of the abutments were measured periodically. The data collection program spanned over a two year period. GRS-IBS construction specifications were obtained from state DOTs at the regional and national level by means of online communication and individual research; this information was compiled into a working draft of a Special Provision and training modules prepared for ALDOT.

### 1.3 Organization of Thesis

This thesis presents background information on GRS-IBS technology and a review of the related literature. GRS-IBS construction guidelines, modelled after the FHWA GRS-IBS Interim Implementation Guide (Adams et al. 2011a), are outlined. This is followed by a summary of the Marshall County (Turkey Creek) GRS-IBS case study, including a discussion of results from its associated data collection program. The drafting process of ALDOT's construction specification is outlined, with the Special Provision referenced in Appendix A. This thesis concludes with a project summary and conclusions.

CHAPTER 2: Background and Literature Review

## 2.1 Overview of GRS-IBS Technology

GRS-IBS was developed and endorsed by the FHWA to meet an increasing demand for small, single span bridges across the United States. Low cost, ease of constructability, and substantial durability make these systems advantageous in a number of environments. GRS-IBS consists of three main components: the reinforced soil foundation (RSF), geosynthetic reinforced soil (GRS) abutments, and an integrated approach (Figure 2).

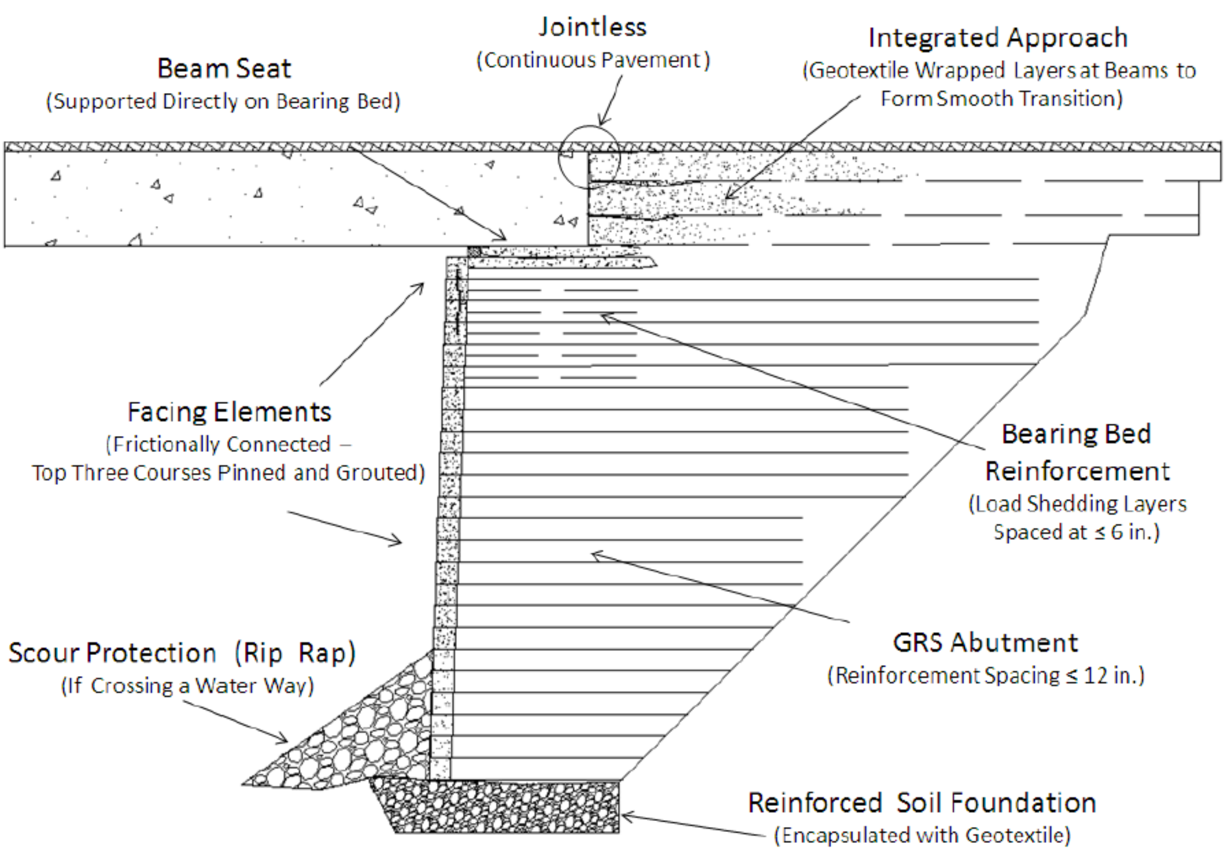


**Figure 2: Typical GRS-IBS cross-section (Adams et al. 2011a)**

The RSF consists of compacted granular material encapsulated in geosynthetic fabric; this technique has been proven to be a viable alternative to deep foundations when constructing on loose, fine-grained, or organic-ridden soil (Adams and Colin 1997). However, if building atop suitable bearing strata, a concrete levelling pad can be used in lieu of a traditional RSF. The pad is generally unreinforced concrete and is not intended as a structural component of the foundation (Berg et al. 2009).

The GRS abutment is constructed directly atop the RSF; this practice adds embedment depth, effectively increasing bearing width and capacity of the abutment (Adams et al. 2011b). Granular backfill is placed in closely-spaced (12" or less) layers of geosynthetic material; the close spacing of reinforcement differentiates GRS structures from traditional MSE walls. MSE technology uses either inextensible metal or extensible geotextile strips that are mechanically connected to proprietary facing elements (Berg et al. 2009). In reality, GRS walls are a type of MSE wall constructed purely with closely spaced layers of geosynthetics that are wrapped or frictionally connected at the face. GRS structures are constructed "from the ground up" and are integrated into the surrounding landscape, as shown in Figure 3.

**Figure 3. GRS-IBS abutment under construction in Defiance County, Ohio (Adams et al. 2011a)**

The integrated approach blends the roadway into bridge superstructure, helping to eliminate the bump at the end of the bridge. Typical bridge structures consist of superstructure placed atop a rigid foundation supported by piles or drilled shafts to minimize the amount of settlement. However, the abutments often settle more than the bridge deck, resulting in a bump at the end of the bridge. Temperature changes and dynamic loading of the bridge deck can also result in a ratcheting effect. Typical construction results in a joint at the end of bridge beams. As the structure translates, soil particles migrate down the joint; loss of soil forms the ratcheting effect, amplifying the bump. GRS-IBS structures are built “into” the surrounding landscape, eliminating the superstructure/roadway interface that is present in traditional bridge construction; this practice mitigates the bump at the end of the bridge.

GRS-IBS implementation provides an array of other benefits, including the following:

- Construction is relatively simple when compared to other bridge-building methods, as these bridge systems can be constructed with common equipment.
- GRS-IBS is significantly cheaper than traditional methods; projects are typically completed in weeks rather than months.
- Project footprints are relatively small, posing minimal environmental concerns. GRS walls are flexible structures; the durable nature of this design leads to satisfactory performance during earthquakes (Keller and Devin 2003).

## 2.2 Components of GRS-IBS

### *2.2.1 Foundation*

Depending on site conditions and native underlying material, the GRS mass can be constructed atop either a reinforced soil foundation (RSF) or a concrete levelling pad. Reinforced soil foundations consist of compacted granular fill material encapsulated with geotextile fabric (Figure 4). An RSF should be used in cases where consolidation settlement could be an issue. A concrete pad may be used in lieu of an RSF if the native underlying material is competent bedrock; additionally, dense sand may also qualify.

**Figure 4. Construction of a reinforced soil foundation for the Maree-Michel bridge in Vermilion Parish (LaDOTD 2016)**

### *2.2.2 Facing Elements*

Facing elements of a GRS abutment are esthetic and are not intended to contribute to design strength of the reinforced soil mass. Aside from aesthetics, the facing elements provide protection from weathering, as well as a form for compaction of backfill material. Modular concrete blocks are the most common facing material (Figure 6); wrapped geosynthetics, gabions, full-height concrete, timber, tires, and shotcrete can be used as well (Wu 1994). A multitude of wrapped-face GRS walls were built by the US Forest Service in the 1970s, but suffered damage due to vandalism, fire, and UV degradation (Berg 1991).

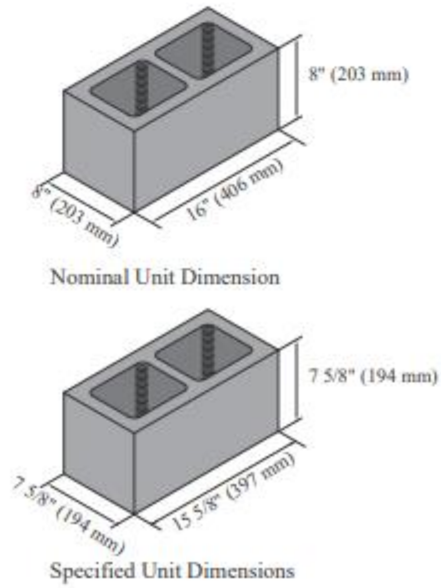


**Figure 5. Schematic of Concrete Masonry Unit (CMU) nominal and specified dimensions (National Concrete Masonry Association 2020)**

Most newer bridges have utilized concrete masonry units (CMU) or segmental retaining wall (SRW) units. Common nominal dimensions are 8 in. x 10 in. x 16 in., and 8 in. x 8 in. x 16 in. Actual dimensions will differ, and it is important to use the actual dimensions in designing and detailing GRS-IBS (Adams et al. 2011a). A minimum compressive strength of 4000 psi is recommended in GRS-IBS applications (Adams et al. 2011a). An absorption limit of 5% is recommended in cooler climates; a freeze/thaw test (ASTM C1262-16) is to be conducted when building in these climates. These tests measure durability and ensure the material meets specifications, and are usually conducted by a state Department of Transportation (DOT). While facing elements are not considered in design strength calculations, large scale performance tests conducted by Nicks et al. (2013) suggest that use of modular concrete blocks as facing elements positively contributes to performance of the GRS structure; bearing capacity significantly increased when CMU blocks were used as facing material, as opposed to no facing material (Figure 7).

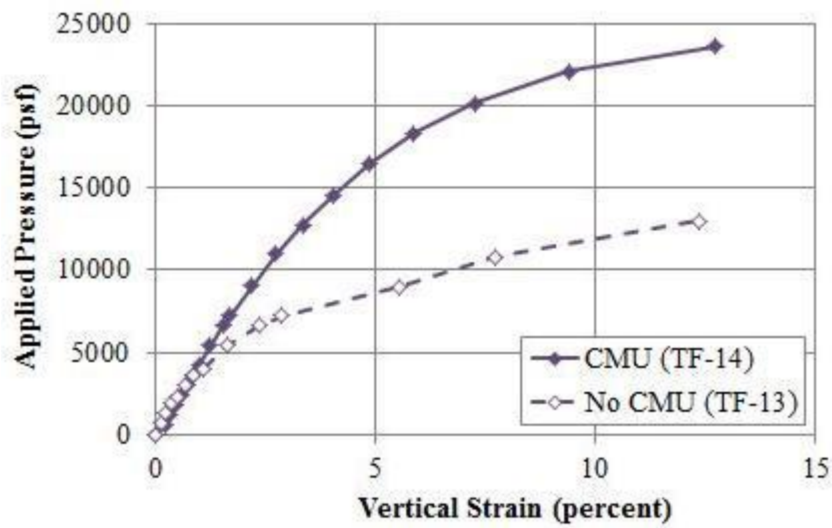


**Figure 6. Performance test results of vertical strain with applied load with and without a facing material. The geosynthetic reinforcement was spaced at 11.25-in with a tensile strength of 3,600-lb/ft (Nicks et al. 2013)**

*2.2.3 Backfill Material*

Backfill material should be well-graded or open-graded, free-draining, and have a maximum particle size of 2 in. Aggregate should be clean, crushed, and angular; the material should also be free from organic matter or deleterious material such as shale or other soft particles that have poor durability (Louisiana Department of Transportation and Development 2014). The material should meet AASHTO T-90 and AASHTO T-104, which control plasticity index (PI) and aggregate soundness. Open-graded aggregate is typically employed in abutment construction; it is not as dense, contains better drainage properties, and is easier to compact (Nicks and Adams 2013). However, well-graded aggregate is recommended for use in the RSF and integrated approach, as greater aggregate density benefit these two structural components. Well-graded backfill should be compacted to 95% of the maximum dry density (AASHTO T-99), while open-graded backfill can be compacted until no visible evidence of further compression exists. Material containing fines should have a moisture content within ± 2% of the optimum. 8 in. lift thickness compacted with vibratory rollers is general practice; however, only hand-operated compaction equipment should be used within 3 ft. of the GRS wall face. Friction angle of open-graded aggregate is commonly estimated as 34° which is a rather conservative design property. While direct shear and triaxial tests can be used to estimate friction angles, the granular nature of GRS backfill makes this difficult, as typical particle diameters are larger than the diameter of a standard direct shear box.

### *2.2.4 Geosynthetics*

Most GRS-IBS structures utilize polypropylene (PP) biaxially woven geotextile reinforcement, similar to that shown in Figure 8. Soil has good compressive strength, but little to no tensile strength; however, geosynthetics perform well in tension. Geosynthetics placed in layers within soil backfill greatly reduces lateral stress (Ingold 1994) and allows GRS and MSE walls to be constructed over soft foundations (Holtz 2008). Filtration is a useful feature of geotextiles; the material allows pore water pressure to dissipate while preventing erosion. Biaxially woven geotextile reinforcement has equal strength in the machine direction (MD) and cross machine direction (XMD); biaxial material is often employed in GRS-IBS applications to reduce the implications of potential construction errors.

**Figure 7. Woven geosynthetic reinforcement material. Geosynthetics are generally stored in rolls (Tencate Geosynthetics 2020).**

The type of reinforcement to be used in a GRS-IBS project depends on lateral stress, spacing of reinforcement, and backfill properties. Lateral stress is created from both dead and live loads imposed during operation. The required reinforcement strength must be less than both the allowable stress and the strength at 2% reinforcement strain. A minimum ultimate tensile strength of 4800 lbs/ft is recommended for most applications. If flooding is a concern, the chosen geosynthetic must accommodate a fast release of water so drawdown conditions do not develop in the GRS mass. Either uniaxial or biaxial geotextile can be used within a GRS mass. Uniaxial has its greatest strength in one direction, while biaxial has equal strength in both direction; biaxial is commonly selected in order to minimize the effects of potential construction errors.

*2.2.5 Design Considerations Near Top of Abutment*

The beam bearing bed receives extra reinforcement to accommodate the additional load imposed by the bridge beams. Geosynthetic reinforcement is spaced more closely in the bearing bed and masonry facing blocks within this region are pinned and grouted with No. 4 rebar and ready-mix cement, as portrayed in Figure 5. The area just behind the bridge beams is called the integrated approach; this section of the abutment is reinforced with equally spaced layers of geosynthetic wrapped around the granular backfill material, with a final layer of geosynthetic wrapped around dense-grade base (Hogan 2018). This practice is intended to alleviate differential settlement between the roadway and abutment.

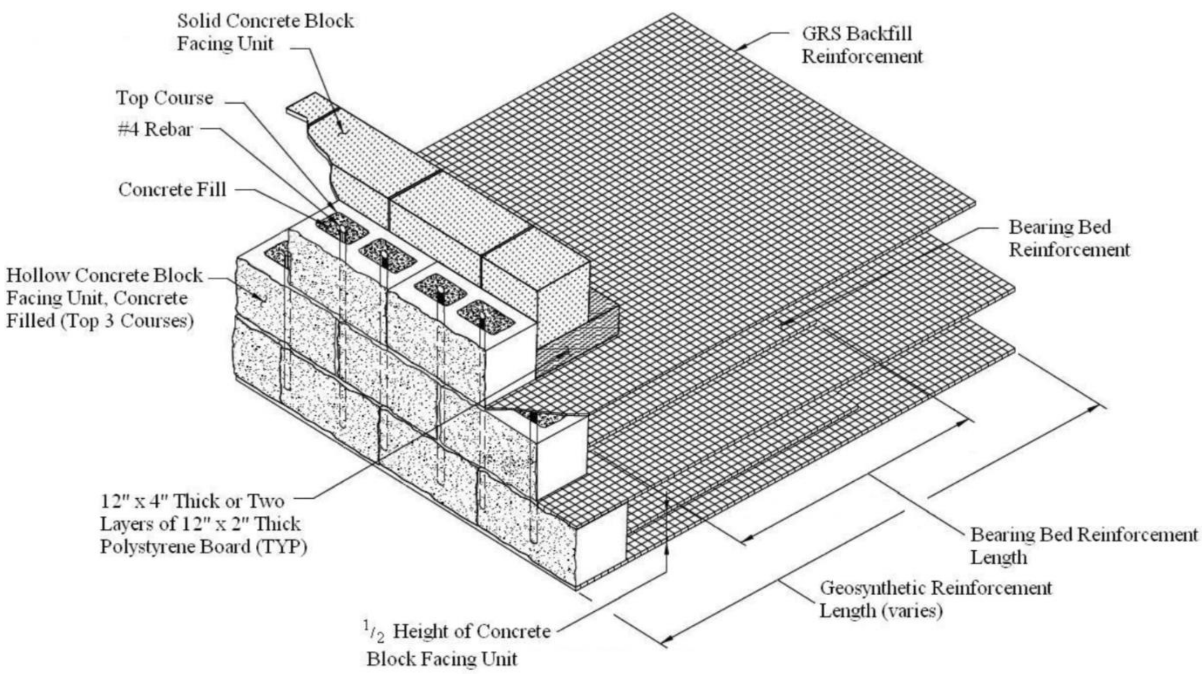


**Figure 8. Beam bearing bed reinforcement pattern (Jones 2012).**

## 2.3 Design Procedure

Design of a GRS-IBS structure is generally conducted using Allowable Stress Design (ASD) methods, as there is not enough significant data on GRS-IBS technology for Load Resistance Factored Design (LRFD). It is worth noting that LRFD can be utilized by normalizing the design to meet ASD factors of safety; however, this eliminates the statistical calibration of LRFD, which is a major benefit of this design methodology. Adams et al. (2011a and 2011b) outlines a 9-step design procedure that is routinely used in GRS-IBS design, which will be discussed throughout this section; a visual schematic of this protocol is portrayed in Figure 10.

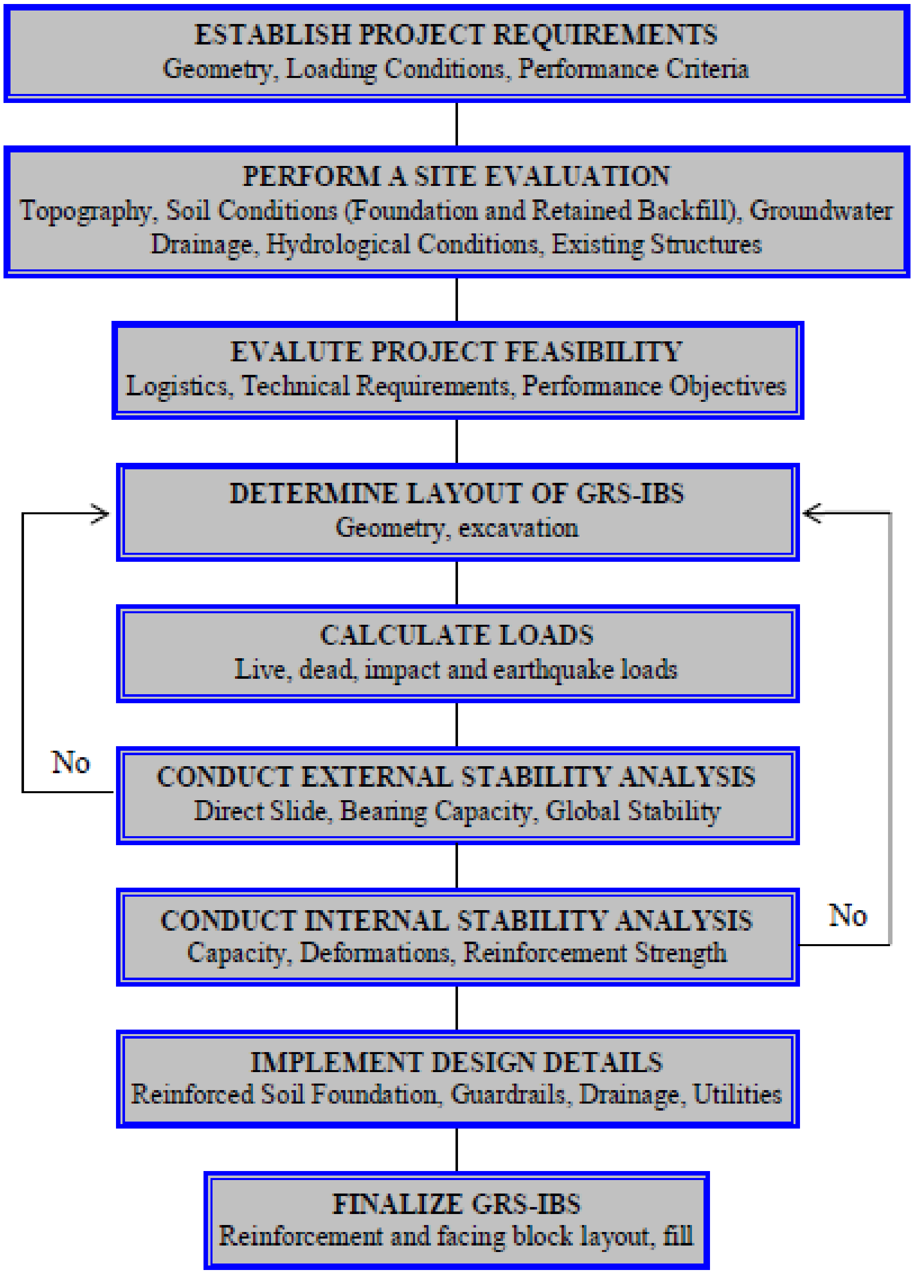


**Figure 9. Nine-step protocol for GRS-IBS design (Adams et al. 2011).**

This design methodology is based on several key assumptions. The geosynthetic reinforcement must be closely spaced (less than 12"), and the reinforced soil mass acts as an internally stabilized composite mass. Granular fill and reinforcing layers strain laterally together with application of a vertical stress until a failure condition is approached. The face of the wall is not considered a structural element, and its presence is not included in design strength calculations. Lateral earth pressure against the wall face is small enough that connection failure is not a concern. Geosynthetic reinforcement and the facing elements are frictionally- not mechanically- connected. Creep of the reinforcement is not a concern when using the recommended granular fill. The steps of Adams's design procedure are detailed throughout this chapter.

*2.3.1 Preliminary Measures*

It is important to first determine the project parameters required to fulfill the needs of the abutment. The height and final elevation of the wall can be deduced from the existing elevations and the final elevation required for the road. An estimate of static loading can be made based on the anticipated surcharge imposed by the bridge superstructure. A traffic analysis can provide an estimate of the expected live loads to be imposed on the abutment. Performance criteria are based on several factors and can be determined by the owner, designer, and contractor in accordance with local code and roadway specifications. Maximum allowable movement- lateral, vertical, and differential across an abutment- is a common performance measure (Adams et al. 2011a), as well as expected lateral stress and pore water pressure within the abutment. Predetermined maximum values will be dependent on the expected lifespan of the structure, as well as any environmental impact expected during or after construction.

An extensive geotechnical site investigation should be completed early in the project life cycle. Necessary measures to be taken while performing a site evaluation are discussed at length by Adams et al. (2011a). There are several specific practices that should be undertaken prior to GRS-IBS construction. It is necessary to examine the existing topography; a topographic map can aid in estimating dimensions of the abutment, as well as obtaining some idea of water flow. In accordance with FHWA procedure, all bridges built over water shall be evaluated for scour, sedimentation, and channel instability (Adams et al. 2011a). Existing structures, roads, and utilities which may influence design must be located. A subsurface investigation should be planned and conducted in accordance with the FHWA Subsurface Investigations manual (Mayne et al. 2002). In-situ properties of native (foundation) and fill (retained earth) soils must be determined; these include unit weight, friction angle, and cohesion, as well as relevant groundwater information. Additionally, maximum particle diameter of the reinforced fill must be known; the designer should obtain this information by contacting the distributor from which the aggregate will be supplied, or make estimates based on aggregate specification that will be written into design. Cost, logistics, engineering and performance requirements should be considered prior to construction. An aggregate source within the relative vicinity should be located (Elton 2014).

### *2.3.2 Determine Layout of GRS-IBS*

The dimensions and layout of a GRS-IBS structure is a function of hydraulic and geotechnical considerations, desired road alignment, and existing elevations. Other considerations, including site preparation, drainage, and scour protection, are discussed at length in the FHWA Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide (Adams et al. 2011a). Wall face geometry, bearing width (b), setback distance ($a_b$), depth and volume of excavation, reinforcement length and spacing, and layout of the integration zone should be analyzed in order to specify GRS-IBS dimensions and layout.

Dimensions of the abutment facing (and wing wall facing, if applicable) can be estimated based on existing topography and desired final grade of bridge to determine the wall face geometry. Bearing width is the width of zone atop the abutment on which bridge girder sits. Steel girders should be placed on a concrete leveling pad, while concrete girders can be placed on either a pad or directly on the reinforced soil mass (Adams et al. 2011a). The bearing width for superstructure should be at least 2 ft. for bridge span lengths ($L_{span}$) less than 25 ft., and 2.5 ft. for spans greater than or equal to 25 ft. The setback distance ($a_b$) is the length of space between the back of the abutment face and beam set where no load is placed on reinforced soil mass. This distance is generally the height of one CMU block, or at least 8 in. A distance of at least 3 in. or 2% of abutment height (whichever is greater) should be located between the top of the uppermost facing block and bottom of the bridge girder (Adams et al. 2011a); this distance is called clear space ($d_e$).

Geosynthetic reinforcement pattern is the primary factor in dictating the depth and volume of a GRS excavation. Geosynthetic length and spacing dictates the strength of the GRS mass. Vertical spacing of geosynthetic reinforcement is to be the height of one SRW block. It is up to the designer to specify length of the reinforcement in each layer. GRS-IBS structures can be designed utilizing either uniform reinforcement length or a truncated length, with truncated patterns being most common choice for GRS-IBS applications (Adams et al. 2011a). Truncated design consists of a shorter reinforcement length near the bottom of the abutment, with a longer reinforcement length near the top; the reinforcement length can gradually increase by each layer, or have multiple same-length layers with groups getting larger near the top (Figure 11). When utilizing a truncated reinforcement pattern, the allowable bearing pressure of the underlying soil should be reduced by 10% (Wu 1994). Additionally, a truncated design offers significant cost savings and reduces the amount of excavation, granular fill, and geosynthetic material required for the project (Wu 1994). Adams et al. (2011a) presents methods of determining the extent of required excavation, which depend on the chosen reinforcement pattern. For uniform length reinforcement design, the initial geosynthetic reinforcement length should be estimated at 70% of the height of wall (0.7H). For a truncated design, the initial base width ($B_{total}$) is the larger of:

- Ratio of $B_{total}/H$ equal to 0.3
- 6 ft. for spans ≥25 ft. & 5 ft. for spans < 25 ft.

The base of the abutment is placed at the calculated scour depth, with the RSF extending 0.25 $B_{total}$ below the scour line and 0.25 $B_{total}$ out from the face of the abutment (Figure 11). Additional excavation may be needed if foundation conditions dictate a larger RSF is required (Adams et al. 2011a).

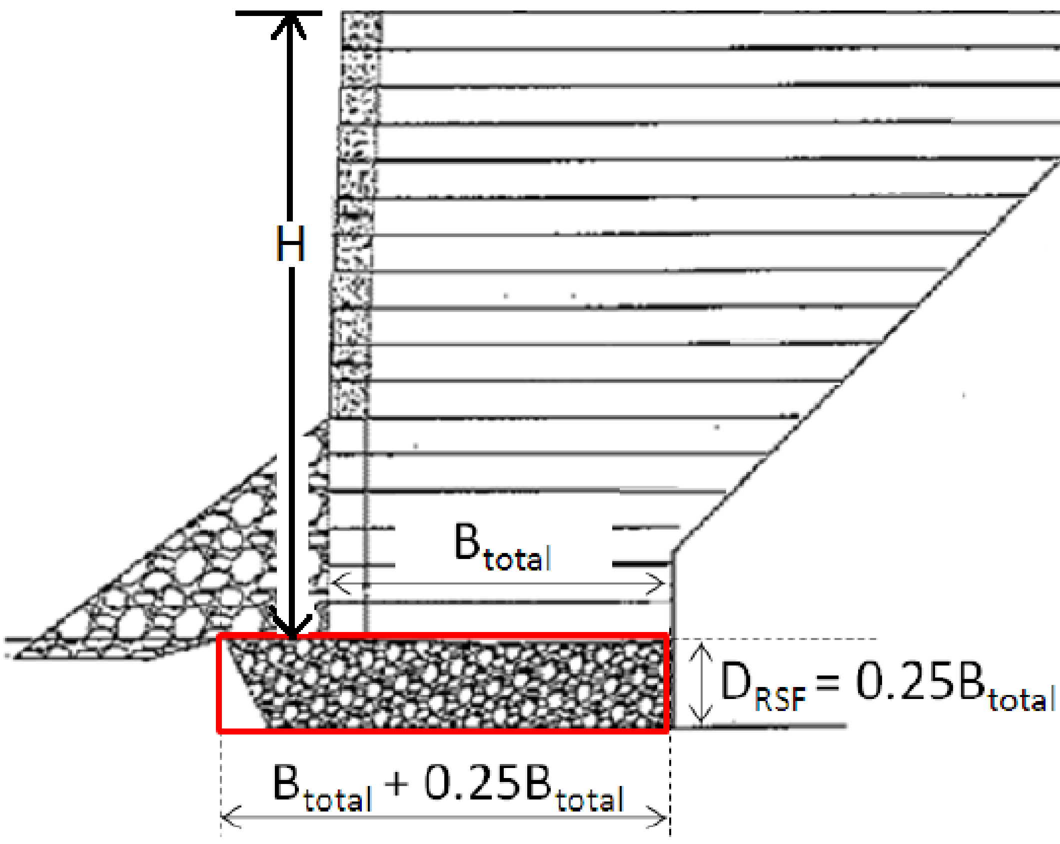


**Figure 10. Illustration of base and RSF dimensions for a truncated reinforcement pattern (after Adams et al. 2011a).**

The bearing reinforcement zone is located directly beneath the bridge seat, acting as an embedded footing in the reinforced soil mass to support the surcharge loading from the bridge (Adams et al. 2011a). The depth of the bearing reinforcement zone can be determined from an internal stability analysis as discussed *2.3.4 Stability Analysis*; however, Adams et al. (2011a) outlines the following guidelines regarding design of the bearing reinforcement zone:

- Geosynthetic spacing within this zone should be half of the primary spacing (half the height of one CMU),
- Width of the bearing reinforcement zone should be at least the width of the bridge seat, plus twice the width of the setback distance,
- There should be at least five bearing reinforcement layers (designated Zone 3 in Figure 12).

The integration zone is essential to alleviating differential settlement between the abutment and bridge deck, also known as the “bump at the end of the bridge”. The reinforcement layers in the integration zone blend into soils extending into the cut slope, thus creating a smooth transition. The integration zone sits directly behind and above the bearing reinforcement zone, as shown in Figure 12. Inclusion of this zone aids in preventing a tension crack from developing at the interface of the reinforced and retained soil mass. The number of reinforcement layers required depends on the height of the superstructure; however, the maximum lift thickness is 12 in.

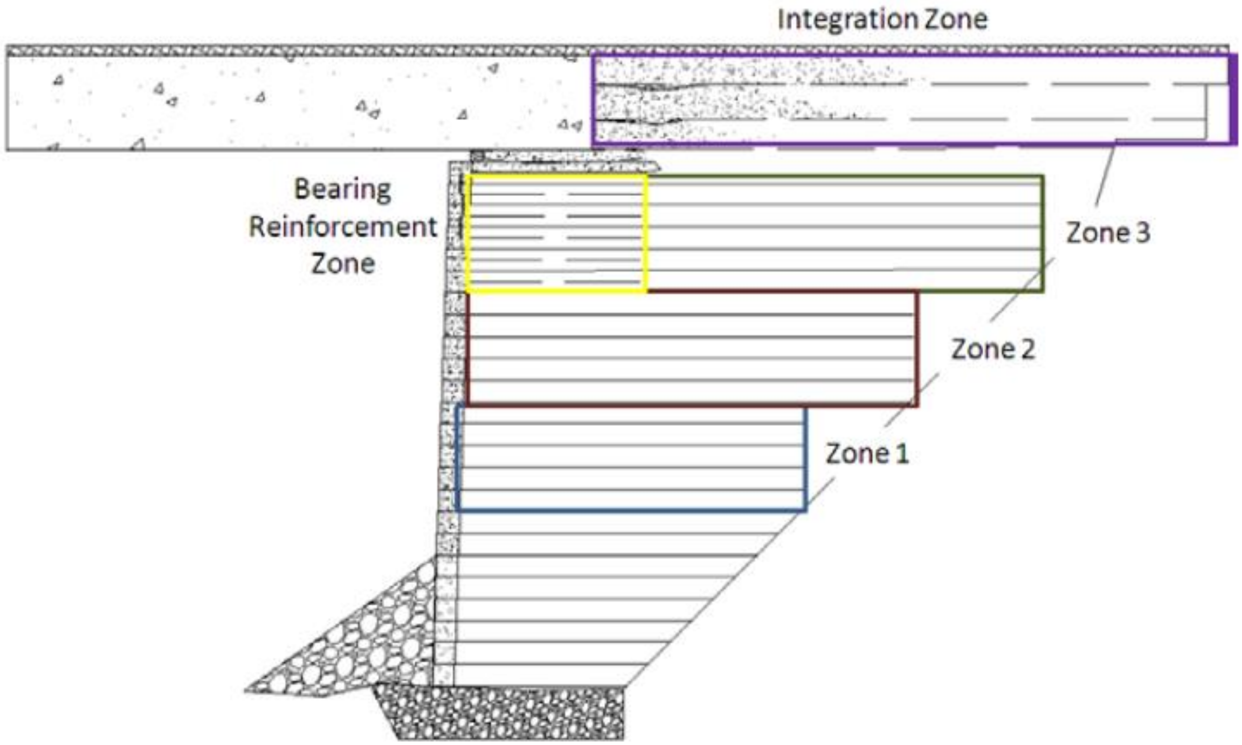


**Figure 11. Reinforcement truncated in zones for a GRS abutment (Adams et al. 2011a).**

*2.3.3 Loading Analysis*

Estimate of loading imposed onto GRS abutment should be calculated for design. Figure 13 shows common loads that should be considered, and Table 1 defines the abbreviations.

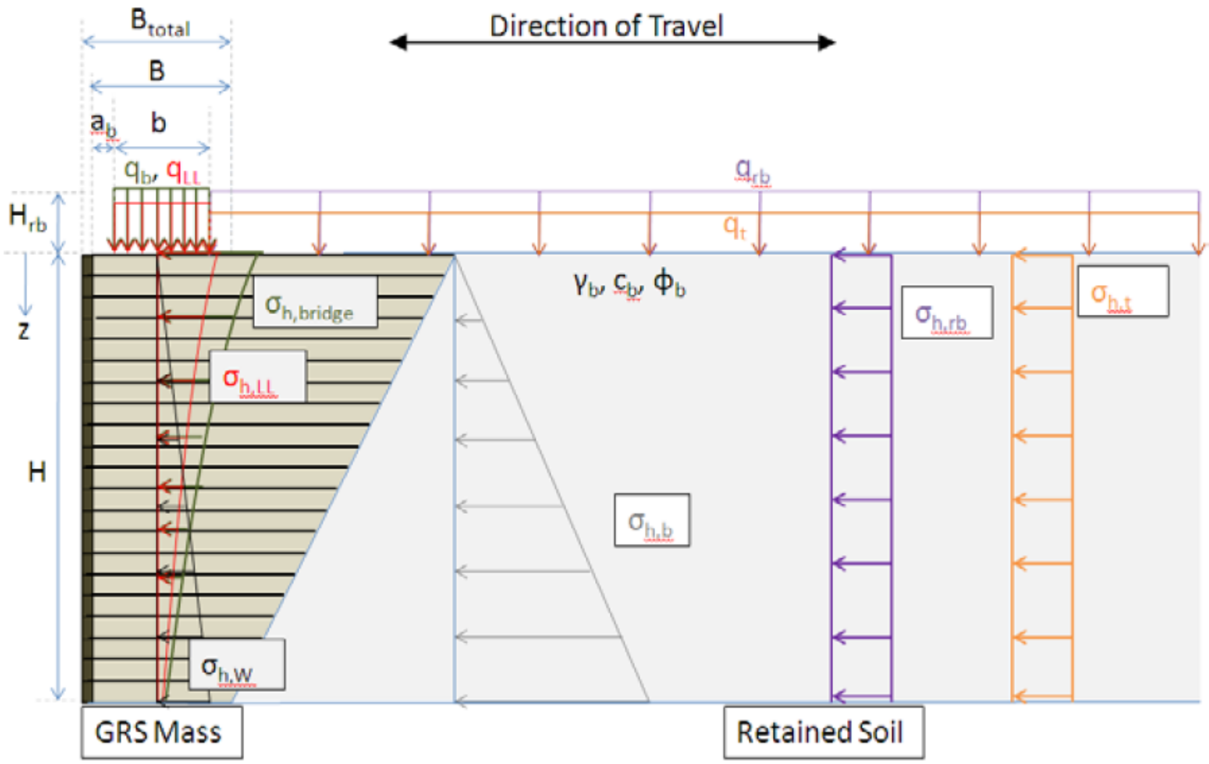


**Figure 12. Typical vertical and lateral pressures on a GRS abutment (Adams et al. 2011a).**

**Table 1. Typical pressures on a GRS abutment (from Adams et al. 2011a)**

| Notation | Parameter |
|---|---|
| $q_t$ | Equivalent roadway LL surcharge |
| $\sigma_{h,t}$ | Lateral pressure due to traffic surcharge within GRS |
| $q_{rb}$ | Surcharge due to the structural backfill |
| $\sigma_{h,rb}$ | Lateral pressure due to road base surcharge within GRS |
| $q_b$ | Equivalent superstructure DL pressure |
| $\sigma_{h,bridge}$ | Lateral stress distribution due to the equivalent superstructure DL pressure |
| $\sigma_{h,b}$ | Lateral stress distribution due to retained soil behind GRS abutment |
| $q_{LL}$ | Equivalent superstructure LL pressure |
| $\sigma_{h,LL}$ | Lateral stress distribution due to equivalent superstructure LL pressure |
| $\sigma_{h,W}$ | Lateral stress due to weight of GRS |

Lateral earth pressure is calculated using Rankine active earth pressure theory. The active earth pressure coefficient ($K_a$) is calculated using Equation 1 (Adams et al. 2011a).

$$K_a = \tan^2(45 - \phi / 2) \qquad \text{Equation 1}$$

where $\phi$ is the internal friction angle in degrees (Adams et al. 2011a).

Four separate loads contribute to the lateral earth pressure: GRS fill pressure ($\sigma_{h,w}$), roadway surcharge pressure ($\sigma_{h,t}$), structural backfill of the integrated approach pressure ($\sigma_{h,rb}$), and surcharge loading pressure ($\sigma_{h,q}$). These pressures can be calculated by Equations 2, 3, 4, and 5 respectively (Adams et al. 2011a).

$$\sigma_{h,w} = \gamma_r z K_{ar} \qquad \text{Equation 2}$$

Where $\gamma_r$ is the unit weight of the reinforced fill, z is the depth from the top of the wall, and $K_{ar}$ is the Rankine active earth pressure coefficient of the reinforced fill (Adams et al. 2011a).

$$\sigma_{h,t} = q_t K_{ab} \qquad \text{Equation 3}$$

where $q_t$ is the roadway surcharge and $K_{ab}$ is the Rankine active earth pressure coefficient of the retained backfill.

$$\sigma_{h,rb} = q_{rb} K_{ab} \qquad \text{Equation 4}$$

where $q_{rb}$ is the surcharge due to the structural backfill (Adams et al. 2011a).

$$\sigma_{h,q} = \frac{q}{\pi}\left[\alpha + \sin(\alpha)\cos(\alpha + 2\beta)\right]K_a \qquad \text{Equation 5}$$

Where q is the surcharge pressure, $K_a$ is the Rankine active earth pressure coefficient, and $\alpha$ and $\beta$ are the angles in radians found using equations 6 and 7 respectively. This design procedure assumes a Boussinesq stress distribution, as this analysis results in higher calculated stresses and a more conservative design than other similar analyses (Adams et al. 2011a).

$$\alpha = \tan^{-1}\left(\frac{x}{z}\right) - \beta \qquad \text{Equation 6}$$

$$\beta = \tan^{-1}\left(\frac{x - b_q}{z}\right) \qquad \text{Equation 7}$$

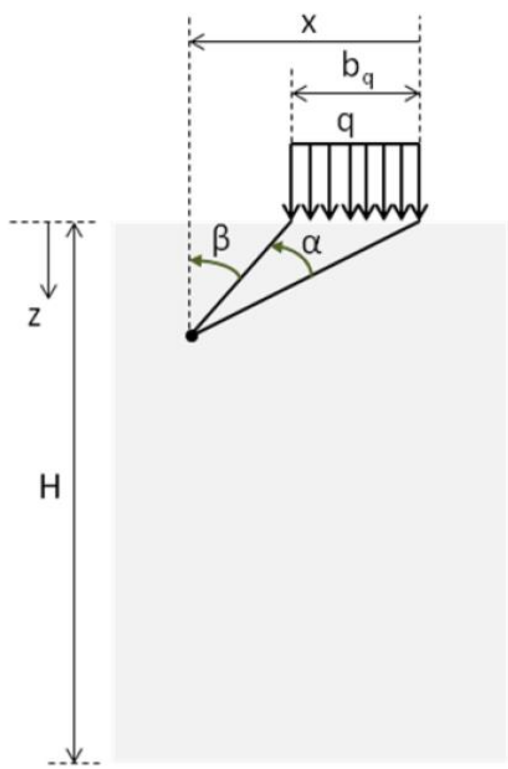


**Figure 13: Boussinesq load distribution with depth for a strip load (Adams et al. 2011a)**

Bridge beams, asphalt, overlay, guardrails, and any other applicable permanent loads related to the superstructure will impose dead load forces on the abutment (Adams et al. 2011a). Traffic loading on the approach pavement is modeled as a live load surcharge ($q_t$). The load is modeled as a height of earth that produces an equivalent lateral effect on the abutment as the vehicular loading (Elton 2014). $q_t$ is dependent upon abutment height and orientation (Adams et al. 2011a). The vehicular live load is increased for impact allowance (IM). Equation 8 shows how to calculate the equivalent distributed live load pressure ($q_{LL}$) on the abutment seat.

$$q_{LL} = \frac{(LL + IM)_{total} N_{lanes}}{b(B_b)} \qquad \text{Equation 8}$$

Where $N_{lanes}$ is the number of design lanes on the bridge, b is the bridge seat bearing width, $B_b$ is the width of the bridge, and $(LL+IM)_{total}$ is the governing abutment reaction for one lane (Adams et al. 2011a).

If the bridge seat bearing width is unknown, the live load should be quantified as a reaction as shown in equation 9 (Adams et al. 2011a).

$$Q_{LL} = (LL + IM)_{total} N_{lanes} \qquad \text{Equation 9}$$

The design bearing pressure should be targeted at 4,000 psf. Dividing the total load (LL+DL) by the area of the bridge seat yields bearing pressure. If the bearing pressure is too high, the width of the bridge seat should be increased (Adams et al. 2011a).

*2.3.4 Stability Analysis*

Once bridge dimensions and loadings have been determined, an external stability analysis of the GRS-IBS structure should be performed; direct sliding, bearing capacity, abutment displacements and global stability are all factors to be considered. Direct sliding refers to horizontal translation of an abutment; both driving and resisting forces of the reinforced soil mass must be calculated to determine this factor of safety. Driving forces from the retained backfill ($F_b$), road base ($F_{rb}$), and roadway live load surcharge ($F_t$) are calculated using equations 10, 11, and 12, respectively (Adams et al. 2011a).

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 \qquad \text{Equation 10}$$

$$F_{rb} = q_{rb} K_{ab} H \qquad \text{Equation 11}$$

$$F_t = q_t K_{ab} H \qquad \text{Equation 12}$$

Where $\gamma_b$ is the unit weight of the retained backfill, $K_{ab}$ is the active earth pressure coefficient for the retained backfill, H is the height of the wall including clear space, $q_{rb}$ is the road base dead load, and $q_t$ is the roadway live load (Adams et al. 2011a). The total driving force is calculated using equation 13 (Adams et al. 2011a).

$$F_n = F_b + F_{rb} + F_t \qquad \text{Equation 13}$$

The resisting force is calculated using equation 14 (Adams et al. 2011a).

$$R_n = W_t \mu \qquad \text{Equation 14}$$

Where $R_n$ is the resisting force, $W_t$ is the total resisting weight per unit width (equation 15), and μ is the interface friction angle between the soil and the reinforcement. If μ is unknown, it can be estimated as the tangent of 2/3 of the reinforced granular fill friction angle as shown in equation 16 (Adams et al. 2011a).

$$W_t = W + q_b b + q_{rb} b_{rb,t} \qquad \text{Equation 15}$$

$$\mu = \tan(\frac{2}{3}\phi) \qquad \text{Equation 16}$$

Where W is the weight of the GRS abutment per unit width, $q_b$ is the bridge dead load, b is the width of the bridge load, $q_{rb}$ is the road base dead load and $b_{rb,t}$ is the width over the abutment where the road base dead load acts (Adams et al. 2011a).

$$W = \gamma_r HB \qquad \text{Equation 17}$$

The resisting forces of the reinforced soil mass must be greater than 1.5 times the driving forces; equation 18 is used to calculate the factor of safety (Adams et al. 2011a).

$$FS_{slide} = \frac{R_n}{F_n} \qquad \text{Equation 18}$$

The vertical pressure at the base of the RSF should be calculated using a Meyerhof distribution; this formulation is outlined by Adams et al. (2011a) using a FS of 2.5 or greater. This calculated vertical pressure is not to exceed the allowable bearing capacity of the underlying soil foundation. Vertical pressure at the base of the RSF can be calculated by summing vertical forces. These forces include the following: Weight of the abutment, weight of the RSF, weight of the facing units, live load exerted on the roadway, width of traffic and road base load over the abutment, road base surcharge, dead loads exerted by the bridge, width of bridge seat, and live load imposed on the superstructure. Stress concentrations due to eccentric loading are also considered.

A global stability analysis should be performed in accordance with a classical slope stability theory; either a rotational or wedge analysis is recommended. Global failure modes should be protected by a safety factor of at least 1.5 (Adams et al. 2011a). These analyses are generally performed using modern software such as SLIDE or FLAC, as large numbers of iterations are carried out in a limit equilibrium analysis. It is imperative to gather quality soil property information for this analysis; otherwise, the critical failure surface may go unnoticed.

Horizontal and vertical displacements of the abutment are estimated assuming zero volume change (Adams et al. 2011a). Vertical displacements can be estimated using a classical settlement analysis. Anticipated settlement should be considered at all locations across a bridge system; differential settlement must be accounted for in the design, and monitored post-construction (Elton 2014); this can be accomplished with a total station surveying instrument. Horizontal displacements are more difficult to estimate, and are often approximated in the design phase. It is assumed the applied factors of safety for external and internal stability of the abutment will ensure lateral deformations are within limits

Ultimate vertical capacity, abutment deformation, and reinforcement breakage are the three internal failure modes that are evaluated by the FHWA GRS-IBS design method (Adams et al. 2011a). It should be noted that pullout resistance is an internal failure mode that is commonly evaluated for MSE structures; however, GRS wall facing is not mechanically attached to internal reinforcement, so this failure mode is not a possibility for GRS structures and is thus not accounted for in design.

The ultimate vertical capacity of the abutment is found either empirically or analytically; Elton (2014) outlines both methods of calculations, while Adams et al. (2011a) provides further information. Ultimate capacity can be determined from performance tests; the test should employ the same geosynthetic and granular material as planned for field use (Elton 2014). The ultimate capacity can be divided by a factor of safety of 3.5 to yield the total allowable pressure on the abutment. Total allowable pressure on the abutment must be less than the summation of the dead load from the bridge and the live load on the superstructure.

Wu et al. (2013) provides a method to calculate the required reinforcement strength (equation 19); this relationship is a function of the lateral stress, reinforcement spacing, and the maximum aggregate size. Since horizontal stress changes with vertical position, strength estimates are needed at each layer of reinforcement.

$$T_{req} = \left[ \frac{\sigma_h}{0.7(\frac{s_v}{d_{\max}})} \right] s_v \qquad \text{Equation 19}$$

Where $S_v$ is the reinforcement spacing and $d_{max}$ is the maximum aggregate size. The horizontal stress within the abutment is a function of the stress imposed by the backfill material, bridge, and roadway DLs and LLs. Horizontal stress can be calculated using equations provided by Adams et al. (2011a). Furthermore, Adams et al. (2011a) recommends that the allowable reinforcement strength be at least a factor of 3.5 less than the ultimate tensile strength and less than the strength at 2% reinforcement strain.

### *2.3.5 Implementation Measures*

Certain design details of a GRS-IBS structure should be given particular attention during the implementation phase. Having a level first course of blocks is an essential starting point to ensure proper alignment of the wall face (Elton 2014). Compaction of fill near the face of the wall should be performed with hand equipment to reduce the movement of facing blocks (Adams et al. 2011a). The bearing reinforcement bed should be composed of at least five reinforcement layers, and a clear space of three inches (or 2% of abutment height) should be ensured by placing the beam seat at a proper setback distance. Cranes should be properly positioned on the GRS mass with outrigger pads, and dragging beams across the wall face should be avoided. Adams et al. (2011a) recommends using steel H posts for guardrail systems. All surface runoff should be diverted away from the structure during the construction phase. Further discussion of design details pertaining to site preparation, the reinforced soil foundation (RSF), geosynthetics, blocks, backfill material, bearing reinforcement bed, beam seat, placement of superstructure, integrated approach, guardrails, and drainage provisions can be found within Chapter 3 of this document.

## 2.4 Case Studies

### *2.4.1 Louisiana: Maree Michel Bridge*

The Maree Michel GRS-IBS was constructed along Louisiana Highway 91 in Vermilion Parish to replace two timber bridges over the Maree Michel Canal. The bridge was designed inhouse by the Louisiana Department of Transportation and Development (LaDOTD) and let to private contractors for bidding. The site was located in a rural setting with an ADT of less than 500 vehicles. The old creek bridge had a span length of 35 ft., while the replacement GRS-IBS had a span length of 72 ft. The bridges spanned irrigation canals with virtually no flow, posing minimal scour potential. The underlying stratigraphy consisted of clay with undrained shear strengths (Su) ranging from 820 psf to 2100 psf, along with a shallow layer of sandy silt with a friction angle ($\phi$) greater than 30°.

Preliminary design consisted of hand calculations which followed the steps outlined by the FHWA Interim Implementation Guide (Adams et al. 2011a). While initially considered, overdesign of the bridge to account for perceived weaknesses of the method was discouraged. A stability analysis spreadsheet was conceived with support from the Bridge Design bureau; span configuration and beam seat size where iteratively determined by using different load and moment combinations. In addition to quickly accommodating minor changes in structural design, the spreadsheet allowed the designers to study the relative impact of various factors. The maximum height of the GRS abutment is approximately 15.6 ft from the bottom of the RSF to the road pavement, the width of the abutment is 43 ft, and the girder span is 72 ft. The abutment's structural fill consisted of an open-graded crushed rock compacted to a minimum of 95% of $\gamma_{d,max}$; backfill material was compacted to 100% of $\gamma_{d,max}$ in the bearing bed and beam seat. Woven geotextile with an ultimate tensile strength of 4800 lb/ft was used as reinforcement. Facing elements consisted of nominal 8 in. x 8 in. x 16 in. concrete masonry units (CMUs) with a compressive strength of 4000 lb/in$^2$. Constructed began on April 6, 2015 and commenced with placement of the integrated approach slab on July 27, 2015.

**Figure 14. Maree-Michel GRS abutment construction behind cofferdam (Rauser 2016).**

Similarly to the Turkey Creek bridge in Marshall County, the Maree-Michel bridge was Louisiana’s first GRS-IBS structure; various types of instrumentation were installed within the GRS abutments to monitor in-service performance. Primary measurements included horizontal and vertical displacements of the wall, settlement of the soil foundation, stress distributions in the GRS mass, and strain distributions along the geosynthetic reinforcements. Pressure cells underneath the RSF measured distribution of the vertical total pressure, and pressure cells behind the wall face measure the horizontal total pressure. Piezometers measured pore water pressure and were used to examine the effective stresses in the GRS-IBS abutment. Electrical resistance-type strain gauges installed onto the geosynthetic reinforcement measured the mobilized strains along the material; resulting tensile forces developed in the geosynthetic reinforcement could be estimated from strain measurements and the elastic modulus of the material. All instrumentation readings showed the overall performance of the GRS-IBS was within acceptable tolerance in terms of measured strains, stresses, settlements, and deformations (Abu-Farsakh et al. 2017).

### *2.4.2 New York State: St. Lawrence County*

The St. Lawrence County Highway Department in New York State has constructed seventeen GRS-IBS bridges since 2009. A bridge inventory showed that there were 81 County-owned bridges that were labelled as deficient; 27 of these were placed on a priority replacement list due to a condition rating of ≤ 4.5 out of 7. GRS-IBS approach was utilized due to the need for an accelerated, economical method of bridge replacement. The Elliot Road project entailed replacing a structure spanning Trout Brook in upstate New York. The existing bridge consisted of two 25 foot spans built in 1929, and the bridge had a condition rating of 3.33 out of 7 as computed for the 2013 NYSDOT Bridge Inspection. The site was composed of silty sands and silt underlain by limestone bedrock. Erosion had occurred due to spring flood waters and ice, as well as the existing North abutment and center pier having poor stream alignment. The replacement GRS-IBS structure included removal of the existing North abutment and center pier, aligning the new North abutment with the stream, and building the South GRS abutment behind the existing South abutment. Superstructure for the replacement bridge consisted of 73 ft. long precast concrete adjacent beams. From start of demolition to final grading and paving took 11 weeks with a material cost of $228,700; this expedited construction schedule was particularly advantageous given New York state’s short construction season. The St. Lawrence County Department of Highways followed the FHWA 9-Step Design procedure (Adams et al. 2011a) for the Elliot Road project. Unique features of this bridge included a levelling pad pinned into bedrock, a heavy precast concrete block lower wall facing to withstand ice impact aligned with the stream, and a setback

GRS "stub wall" of concrete masonry units aligned with the south abutment. These non-typical features demonstrate the potential for GRS-IBS to conform with a variety of site conditions.

**Figure 15. Elliot Road GRS-IBS in St. Lawrence County, New York State. A setback GRS "stub wall" was built atop the existing (south) concrete abutment.**

*2.4.3 Defiance County, Ohio*

Defiance County, Ohio has been leading the charge for GRS-IBS construction in the United States. In 2005, the FHWA began working with the County's engineering department to provide guidance on GRS abutment design. Between 2005 and 2011, Defiance County built 26 GRS-IBS structures at a cost of $3,513,484 (Bloser et al. 2012). Defiance County engineer Warren Schatter stated that after becoming comfortable with the construction process, county construction crews can build an abutment in 3 days; geosynthetic fabric is ordered in bulk and other materials are readily available, resulting in minimal time waiting around (Wichman et al. 2012). Five of these 26 bridges were instrumented to measure vertical and lateral deformations. Over the course of three years the maximum bridge differential settlement was 0.033 ft., and the average GRS settlement was less than 0.1 ft. (Adams et al. 2011a).

**Figure 16. Bowman Road GRS-IBS in Defiance County, Ohio (defiance-county.com)**

# CHAPTER 3: CONSTRUCTION PROCEDURE

## 3.1 Labor, Tools, and Equipment

Most GRS-IBS projects can be constructed with a crew of five workers; one equipment operator, and four laborers. The equipment operator provides support to the labor crew and shall remain central to the project. They are responsible for shaping the excavation to facilitate construction of the RSF and GRS abutment, as well as placing fill material and moving facing units into the work area. Typically, one of the labor crew members has the role of foreman; this member is responsible for layout of excavation limits and grades alignment of the wall face, placement of facing blocks, compaction of fill, placement of geosynthetic reinforcement, and other activities to streamline production and flow of material to the jobsite (Adams et al. 2011a).

GRS construction can be completed using simple tools that are easily available, including the following:

- gravel rakes (as concrete spreaders)
- shovels (flat-blade and spade)
- heavy rakes
- push brooms (to sweep off the top of CMU blocks)
- whisk brooms
- sledgehammer and wood blocks (to adjust misaligned blocks)
- heavy rubber mallets
- spade trowels

- razor knives (to cut reinforcement)
- hand tampers with metal base plates
- chainsaws (to cut reinforcement roll)
- concrete saws
- 5-gallon buckets
- block lifter
- standard concrete mixing and finishing tools

Survey equipment ensures that the abutment is being built to the lines, grades, and batter as shown in plans; this includes:

- total station
- tripod
- prism rods
- backsight prisms
- measuring tapes

A 4 ft. carpenter’s level is needed to check individual block alignment, as well as a laser level or string/chalk line to check an entire row of blocks. A plum bob can be used to check wall batter.

Certain pieces of heavy equipment are needed for GRS construction. Walk-behind vibratory plate tampers are used for backfill compaction; these are generally 200 lbs., and should be 18 in. wide or larger (Figure 18). A riding smooth drum vibratory roller can be used, as long as it is kept 3 ft. from the wall face. A track-hoe excavator should be used for aggregate placement.

Pallet forks for the excavator can be used for moving CMU block in and out of the working area; a backhoe can be used for this task as well. A trash pump and hose should be used for dewatering the foundation excavation.

**Figure 17. Worker using a walk-behind vibratory plate tamper to compact backfill while constructing the Turkey Creek GRS-IBS in Marshall County, AL.**

### 3.2 Site Preparation

GRS abutments are built from the ground up, within the footprint of the structure. Effective use of space ensures an uninhibited construction process. The materials staging area should be located in an area that allows for continuous GRS construction, and should be easily accessible to the excavator- the central piece of equipment. Positioning the excavator inside the wall area allows for easy placement of fill, block, and other materials. Labor should be organized around the work platform, able to assemble construction materials as needed.

Site layout begins with a topographic survey of the bridge site. Once the terrain is identified, excavation limits are staked. Stakes should be located in an area that will remain undisturbed during construction of the base of wall- usually 5 ft. from excavation. Abutment base and wing walls should be constructed within an inch of staked elevations, external GRS abutment and wing walls to be constructed within 0.5 in. of surveyed staked dimensions (Adams et al. 2011a).

All excavations must comply with Subpart P (OSHA 2020). Slopes of excavations are to be shaped for temporary slope stability, safety, and constructability. Excavation slopes should be sufficient to accommodate movement of labor, and design must consider imposed loading by heavy equipment. Drainage provisions should be taken; sloped cuts facilitate movement of water. At the conclusion of the project, any open excavations should be backfilled with crushed aggregate and compacted. Grubbing of vegetation should be included in the excavation work plan. If building in a flooded excavation, several remediation options exist. The use of dewatering pumps is common, but sheet-pile walls or coffer dams with sheeting might be necessary in the presence of excess water; selection of method depends on influx of water at the site (Figure 19).

**Figure 18. (a) Dewatering pumps used to remove flooded excavation at Turkey Creek bridge in Marshall County, AL; (b) reinforced soil foundation constructed inside a cofferdam at Maree-Michel bridge in Vermilion Parrish, LA.**

Sometimes it may be beneficial to construct a GRS-IBS behind existing substructure. Project feasibility, environmental considerations, and other factors need to be assessed before selecting this type of project layout. Building a bridge behind existing substructure often requires removal of the top part of existing abutment walls; this is necessary to provide additional space for width of the new GRS-IBS. Whether built behind an existing structure or not, GRS design process remains the same.

### 3.3 Reinforced Soil Foundation

The depth and footprint of the RSF should be based on external stability, and in some cases a hydraulic analysis. The base is to be cut smooth at a uniform depth; loose and unstable material should be removed. If the excavation's base is left open for any significant amount of time (enough for water to accumulate), the base should be graded to one end to facilitate removal of any intrusion of water with a pump. If the open excavation ever floods, all water should be removed along with soft, saturated soils. The excavation should be backfilled as soon as possible to provide a suitable foundation, as well as facilitate in avoiding adverse weather delays. RSF construction can typically be completed in less than a day, but will be dependent on several factors; size and depth of excavation, type of materials used, equipment present on job site, and experience of the labor crew all factor into this. The base of the excavation is to be compacted prior to construction of the RSF; the base might require proof rolling, and any soft spots or voids should be backfilled with compacted fill material. Compacted granular material is to be encapsulated in geotextile reinforcement that is placed perpendicular to the abutment face; this orientation of the closed face of the RSF provides scour protection. Reinforcement sheets should be measured and sized to fully enclose three sides: the face, and two wing wall sides.

Typical RSF reinforcement spacing is 12 in. As with other reinforcement sheets throughout the abutment, reinforcement should be pulled taut to remove all wrinkles prior to placing and compacting the structural backfill. The RSF should be constructed with the same structural backfill used throughout the rest of the abutment. RSF fill material should be compacted as outlined later in this document.

### 3.4 Compaction

Proper compaction of the structural backfill of the abutments is necessary to ensure adequate GRS performance. Relative compaction of the structural backfill should be at least 95% of its maximum dry density; well-graded aggregate should meet AASHTO T99, while method specification (e.g. three passes of compactor) is recommended for open-graded aggregate. Material containing fines should be compacted at a moisture content ± 2% of optimum. Vibratory rolling equipment can be used to compact lifts up to 8 in. thick; for lifts exceeding this thickness, multiple lifts of smaller thickness should be placed and compacted. Facing blocks should be laid out prior to compaction of fill material, as SRW units provide a form for each lift.

After fill material is placed at its required thickness and graded, all area behind modular blocks should be compacted to the specified density. Any depression behind the facing blocks should be filled level to the top of modular blocks prior to further compaction. Compaction directly behind modular blocks should be performed as to maintain wall alignment while improving density of the fill material behind the modular blocks. Fill material in this area can be rodded or foot tamped while downward pressure is exerted on the modular blocks in order to prevent lateral displacement. For multiple lifts, the height of the top lift should extend slightly above the block in order to compensate for compression of fill during compaction. If a lightweight vibratory plate compactor is available, one should be used in lieu of rodding or foot tamping; however, downward lateral pressure should still be exerted atop the blocks. A larger vibrator compactor can be used for the remainder of fill (3 ft. from face of GRS wall); outward block movement should be periodically checked and adjusted for accordingly.

Quality control (QC) monitoring prior to placement of subsequent lifts is crucial to ensure specifications are met. The most common QC tool is a nuclear density gauge; Clegg hammer, soil stiffness gauge, or falling weight deflectometer are also viable options. Measurements produced by these instruments can be correlated to soil density and moisture content. Method-based compaction methods (e.g. three passes of vibratory compaction equipment) can also be used; this is often seen when using open-graded fills. Open-graded material should be compacted to non-movement or no appreciable displacement, and fills should be visually assessed to confirm this.

### 3.5 Reinforcement

The length of geosynthetic reinforcement layers should follow the cut slope, increasing towards the top of the abutment. The RSF and integrated approach should be constructed of fill material encapsulated with geotextile to confine the compacted granular fill. The strongest direction should run perpendicular to the abutment face. Where one roll ends, the next roll begins; overlapping between sheets is not required. The geosynthetic reinforcement should extend between layers of CMU blocks to provide a frictional connection; the reinforcement should cover a minimum of 85% of the top surface of the CMU block, and excess can be removed by either burning it with a propane torch or cutting it with a razor knife.

The geosynthetic should be laid out so that it is taut, free of wrinkles, and flat. Placement of fill material should start close to the wall face and proceed backward, as to reduce the formation of wrinkles. A conscious effort should be taken during placement of fill to prevent development of wrinkles. Reinforcement splices can occur without overlap. Splice seams should be staggered to avoid continuous break in reinforcement throughout the GRS structure (Adams et al. 2011a). Following this procedure, all splice seams can run either perpendicular or parallel to the wall face. Overlaps of adjacent geosynthetic should be trimmed where in contact with the surface of facing blocks, as to avoid varying geosynthetic thicknesses between rows of CMU blocks. Any seams in geosynthetic should be staggered with each successive layer of the GRS abutment (Adams et al. 2011a).

Special care should be taken when operating equipment on geosynthetic reinforcement. Driving directly atop geosynthetic reinforcement is not allowed; a minimum of 6 in. of granular fill should be placed prior to operating any vehicles or equipment over geosynthetic. In the bearing reinforcement zone, hand operated compaction equipment should be used over 4 in. lifts in order to prevent excessive installation damage of geosynthetic reinforcement. Rubber-tired equipment may pass over geosynthetic reinforcement at speeds less than 5 mph. Skid steers and tracked vehicles can impose significant damage on the geosynthetic reinforcement, and their use should be restricted. If necessary, they may be used provided no sharp turns or sudden braking occur and a minimum 6 in. cover is present. The bearing reinforcement layer provides additional strength in upper GRS wall layers directly beneath the bearing area of the superstructure. Reinforcement is placed behind the facing block in 4 in. layers, rather than being sandwiched between rows of blocks

### 3.6 Wall Face

GRS structures are internally stable, and a variety of facing elements can be used in construction. For simplicity, CMUs are used throughout this section to refer to the facing. For flexible facing other than the CMU block, alternative construction guidelines may need to be followed and/or developed. These other facing systems are described by Wu et al (1994). The general guidelines for GRS-IBS, however, remain the same.

#### *3.5.1. Setting and Levelling Block Courses*

Setting the first course of facing block levelled and graded properly is crucial in maintaining wall alignment for the entire height of abutment. Typically, the first course of CMU blocks is placed directly on top of the RSF. Large aggregate size of the RSF fill material can impede proper leveling; therefore, a thin leveling layer of fine aggregate can help set the facing blocks to grade and prevent rocking. If utilized, this leveling layer should be kept to a maximum thickness of 0.5 in. If the leveling layer exceeds this thickness and there is potential for water to erode and undermine the aggregate, mortar or grout should be placed in the gap between the RSF and first course of blocks.

CMU block wall construction begins at the lowest portion of the excavation, with each layer placed horizontally as shown in project plans. Each layer should be constructed entirely before beginning the next layer. A stretcher or running bond should be maintained between courses of block so that joints between blocks are offset with each row. Since the blocks are dry and stacked without mortar, special care should be taken to avoid cracking the blocks and to maintain a uniform horizontal elevation; this can be achieved by sweeping the top surface of blocks clean of debris before placement of the next layer of CMU block and geosynthetic reinforcement. Gravel between blocks creates point loads, which induces cracking. Additionally, aggregate between blocks causes them to rock, making it difficult to secure a good fit. Blocks should be placed tightly against one another while setting a course. This practice prevents fill material from migrating through seams in the wall face. It is often helpful to walk along the top of blocks before placing the next layer, as this can aid in recognition of inconsistencies along the course.

When placing and compacting fill behind CMU blocks, it is often necessary to set blocks back about 0.5 in. to allow for outward lateral movement of blocks during compaction. Alignment of the abutment wall should be checked for plumbness at least every other layer. Any deviation greater than 0.5 in. must be corrected. Each combination of wall face and backfill reacts differently during compaction. Adjustment of setback distance between block courses should be performed as needed to maintain the necessary batter. Wall face verticality, or batter, should be maintained to conform to the design limits and shape of the abutments. This practice aids in avoiding potential as-built changes in setback distance and clear space (Adams et al. 2011a). While some GRS abutments have been built with poor face alignment without exhibiting signs of instability, deficient wall appearance can become a serviceability issue. Questions may arise over whether the wall was built with poor block alignment, or if the wall has experienced post-construction deformation. Prior to placement of backfill, every other row of blocks should be checked for proper alignment with a string line referenced off the back of facing blocks from wall corner to corner

(Figure 20). A 3 lb. sledgehammer and block of wood can often be used to correct CMU block displacement. However, blocks that are excessively out of alignment need be repositioned after fill material is removed. After being re-checked for alignment, fill material may be replaced and re-compacted.

**Figure 19. A string line being used to check the line and grade of the 6th course of the west abutment.**

*3.6.2. Top of Wall Face*

Due to loading imposed by superstructure, the top three courses of CMU blocks are susceptible to movement. No weight from successive layers atop these courses contributes to this issue. Displacement should be prevented by filling the hollow cores of the top three block courses with concrete wall fill. These courses should be pinned together with No. 4 reinforcing steel bars preferably epoxy-coated and embedded with a minimum of 2 in. cover. Geosynthetic reinforcement should be removed in order to grout and pin these courses. This can be accomplished by cutting reinforcement with a razor knife or by burning the reinforcement. The concrete wall fill should be placed in two steps. After the block void is filled with concrete to the top of the block and the steel reinforcing bar is inserted, a thin layer of the same concrete mix should be placed atop the block to form a coping cap. The coping should then be hand troweled either square or round, and sloped to drain. A wet cast cap is more durable than a dry cast cap, and eliminates the need to furnish and install a separate cap unit.

Once the top of the wall is tied together, care should be taken to avoid any construction activity that may dislodge the top layer of reinforcement. The frictional connection between blocks is strong. When the courses are pinned together, the entire grouted wall face can be pulled out of alignment. If another type of concrete modular block is used for the abutment face, the designer needs to develop a suitable method of connection. Many SRW systems have pre-engineered methods of connection; however, these systems may or may not be compatible with the wall face layout or these pinning and grouting practices. Alternative methods may use concrete adhesives.

### *3.6.3. Batter, Superelevation, Corners and Curves*

Block alignment for battered walls is similar to that of vertical walls. However, when the face wall turns to form the wing wall, it is necessary to trim blocks on either end to account for the reduced wall length. All cuts are to be performed to maintain standard running or stretcher bonds between rows of dry stacked blocks, with vertical joints of each course midway between those of adjoining courses. In certain situations, negative battered GRS walls have been constructed when the top area needed to be larger than the bottom area; road widening is one example of this special case. A negative batter can be created by offsetting CMU blocks by measured amounts in consecutive wall layers, then filled and compacted as specified. It should be noted that this practice is typically limited to walls, and has not been used for GRS abutments. However, this example helps to highlight the stability of closely spaced GRS structures.

In the case of superelevation, the top courses of CMU blocks should be trimmed to match elevation difference or clear space across the abutment. This practice produces a sloped-face wall, and aids in construction of the beam seat. To achieve this, a chalk line should be snapped along the back face of blocks at the superelevation slope; a carpenter's angle finder can aid in marking the cut. A concrete saw should then be used to perform this cut.


**Figure 20. Superelevated GRS wall in St. Lawrence County, NY. It should be noted that this wall was constructed behind an existing concrete abutment (Adams et al. 2011a).**

Most GRS applications utilize right angle wall corners. These should be constructed with specialized CMU corner blocks, that have architectural detail on two sides. Facing wall and wing wall courses should be staggered to form a tight, interlocking, stable corner. Walls with angles other than 90° require more effort; corner blocks must be saw-cut to form an angled face. This practice results in a vertical seam formed at the corner. Open block joints may exist, deeming it crucial to fill these voids with concrete mix and install bent reinforcing steel bar to close and connect the seam at each course of block. This method secures two faces and prevents compaction-induced separation during construction of subsequent layers. This practice could also be used to simply add strength to the corner.

Curved walls can be constructed with SRW blocks in lieu of sharp corners. Radius of the curve will be dictated by the tapered shape of the SRW blocks. A curved wall will create a larger footprint, leading to an increased volume of fill material. The block layout schedule should include details of how parapets link to the sides of superstructure. In the case of a curved wall, rounded corner blocks should be used. Figure 22 shows contractors filling in gaps with non-shrink cement after constructing a curved wall with square SRW blocks. While this will help prevent fill from migrating through the gap of the radius, this is not proper GRS-IBS construction.


**Figure 21. Filling cavities in wall with non-shrink cement (Abernathy 2014).**

### 3.7 Beam Seat

The beam seat is constructed directly above the bearing bed reinforcement zone. The beam seat serves to ensure that the superstructure bears directly atop the GRS abutment, not the wall facing block. It also aids in providing clear space between superstructure and the wall face; this clear space is usually 3 in., or 2% of the abutment height (Figure 23). The beam seat is generally 8 in. thick, composed of two 4 in. lifts of wrapped-face geosynthetic and granular aggregate. Cores of the top three courses of CMU block must be pinned with No. 4 steel reinforcing bar and grouted with concrete prior to construction of the beam seat.

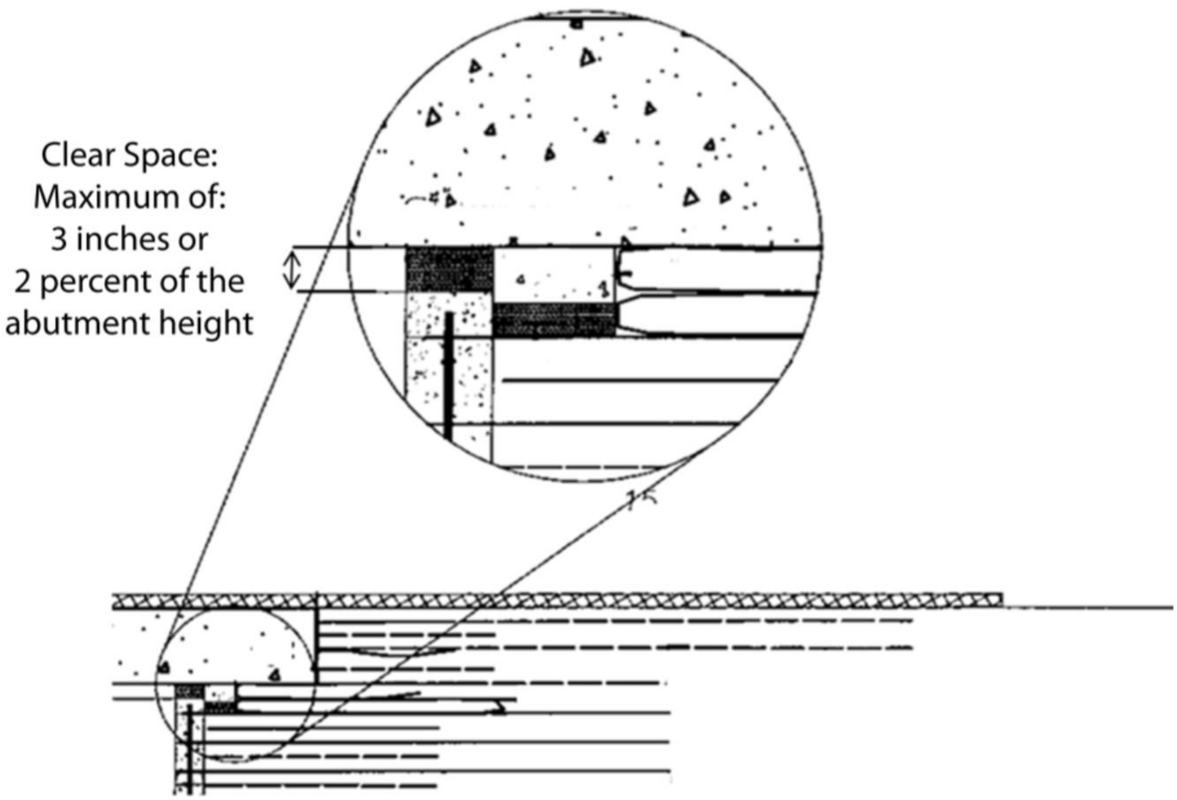


**Figure 22. Illustration of beam seat with clear space distance (Adams et al. 2011a)**

Precut 4 in. thick polystyrene foam board should be placed atop the bearing bed reinforcement zone. Sometimes a thin layer of backfill underneath the foam board is necessary for grading purposes, as well as to ensure proper clear space height and drainage measures are achieved. The foam board should butt against the back face of the CMU block. The exposed edge of the foam board helps to form the nose of reinforcement wrap across the length of bearing area, while the stiffness of the board should allow it to compress as the beam settles.

Solid concrete blocks 4 in. thick should be set atop the foam board across the entire length of the bearing area. The back edge of the top CMU facing block should hold the 4 in. thick concrete blocks in place during compaction. The first 4 in. wrapped layer of compacted fill is used as thickness to the top of the polystyrene board, while the second 4 in. wrapped layer of compacted fill should be placed to the top of the 4 in. solid block. The top of this second layer creates beam space, controlling beam elevation. Surface aggregate atop the beam seat should be graded to about 0.5 in.. This practice aids in seating superstructure, as well as maximize its contact with the bearing area. Placing an additional layer of geosynthetic reinforcement between the beam seat and superstructure provides additional protection of the beam seat, while possibly decreasing sliding resistance between the two elements (Adams et al. 2011a).

As an optional measure, a drip edge can be installed to protect the beam seat and its reinforcement layers from water intrusion (Figure 24). The drip edge also serves to shed potentially corrosive fluids off of the facing blocks, as well as preventing animals from burrowing into the abutment. Aluminum flashing is common material for this element. A precut 4 in. thick polystyrene foam board should be placed on top of the filled-in top course of the concrete block facing units, positioned directly in front of the 4 in. solid concrete blocks. The flashing should be placed in between the bottom of the beams and the polystyrene foam board. The flashing is held in place by the pressure of the beams on the solid concrete blocks. The length of the flashing shall extend beyond the outside edge of the bridge beams and be trimmed to fit against the parapets (New York Department of Transportation 2015).


**Figure 23. Aluminum fascia drip-edge (Adams et al. 2011a).**

The setback is the distance between the back of the facing block and the front of the beam seat (Figure 25). This distance can be established during construction of the beam seat, particularly the placement of block and foam board that are used to form the beam seat wrap. The setback distance is generally 8 in. but can be greater.

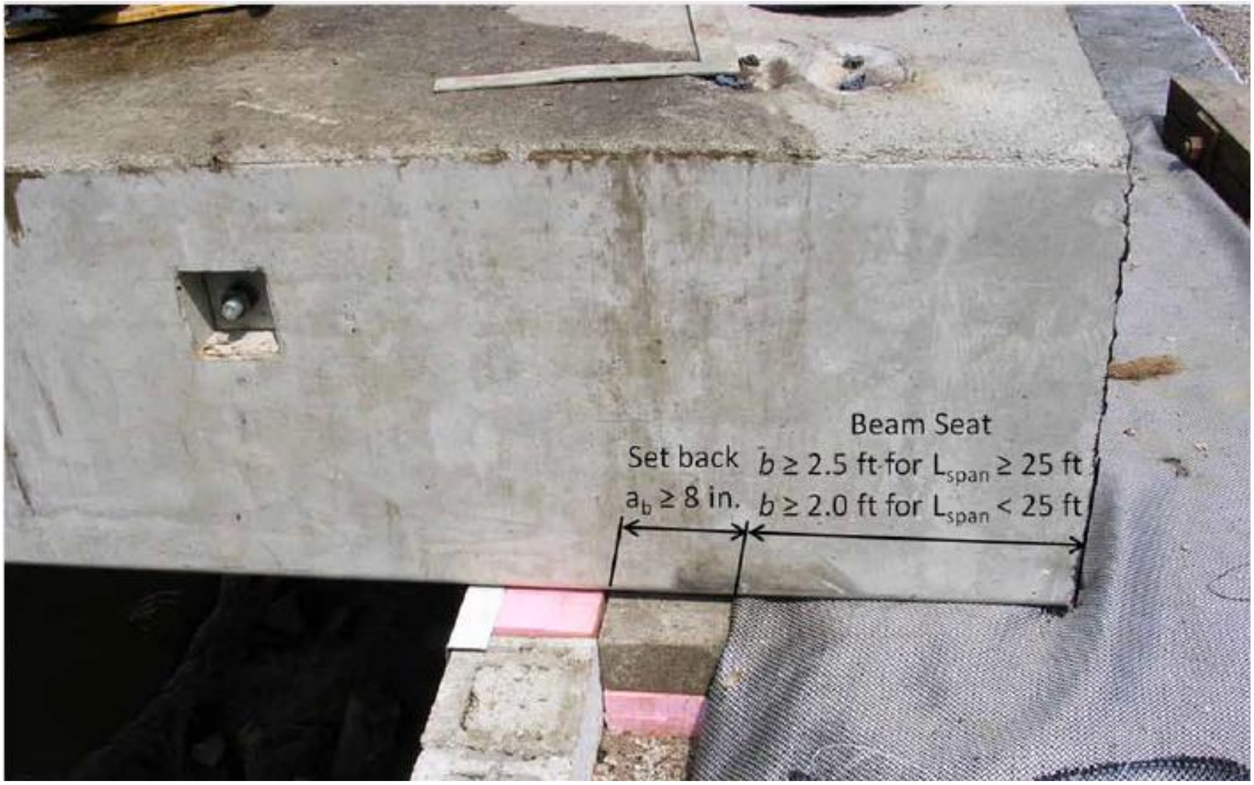


**Figure 24. Bridge beam seat and setback distance dimensions (Adams et al. 2011a).**

### 3.8 Placement of Superstructure

The placement of superstructure should occur following construction of the bridge beam seat. The crane should be positioned atop the GRS mass, centered and away from the wall face. Outrigger pads should be sized as to impose a lesser load on the ground than the factored bearing resistance of the GRS mass. Outrigger pads should be sized for 4000 psf near the face of the wall, but greater loads can be supported with increasing distance from the abutment face; however, larger outrigger pressures should be checked and approved by the Engineer of Record (EOR) (NYDOT 2015).

The bearing surface of the superstructure is the aggregate layer underneath the top layer of geosynthetic reinforcement; thus it is important to set the beams square and level. The grade of the beam seat will control the final elevation of the bridge. Beams should never be dragged over the bridge beam seat surface, as this could create potential for uneven bearing or a void under a beam; voids could create uneven bearing stresses between bridge elements (Adams et al. 2011a). Wing walls and parapets should be constructed after the superstructure is set to ensure a trim fit between these elements. CMU blocks in parapet walls should be trimmed or saw cut for a custom fit against the beam edge of the superstructure, as this practice prevents loss of fill material. If the gap between superstructure and facing blocks is difficult to fill using thin slices of cut facing block, mortar mix or other material should be used to close the gap.

### 3.9 Approach Integration

A properly constructed approach integration is essential for mitigating settlement in front of the bridge beams and minimizing the bump at the end of the bridge; this feature is crucial to successful implementation of GRS-IBS. This is accomplished by compacting and reinforcing the approach fill with wrapped geotextile layers. The integrated approach should be constructed after placement of the superstructure. Geotextile sheets should be trimmed to provide for the planned length after it is wrapped, and placed behind the beam ends. The width of this sheet should allow for wrapping of all sides after compaction of the fill, as wrapping sides prevents lateral migration of the fill. A 6 in. thick layer of fill should be placed and compacted in accordance with road base plans and specifications. After addition of a secondary layer of reinforcement atop the first 6 in. lift, another lift of identical thickness should be placed and compacted. The geosynthetic sheet should then be folded back to wrap the compacted fill layer, and pulled taut free of wrinkles. These steps should be repeated until the integrated approach is approximately 2 in. from the top of the beam grade.

Multiple sheets can be used along the width of the approach as long as all seams are kept perpendicular to the beam ends. The typical wrap reinforcement spacing is 12 in., with intermediate layers at 6 in. and compacted in 6 in. thick lifts. When dealing with reduced-depth beams, wrapped layer spacing may need to be reduced and intermediate layers eliminated. At a minimum, the top two reinforcement layers of the integrated approach should extend 3 ft. over the cut slope; this practice blends the roadway with the GRS mass. The top wrap fold should increase in length with each successive wrapped layer until the fill is 2 in. below the bridge beam grade. Limiting the amount of fines in the backfill material used for the integrated approach is crucial to preventing frost heave. In certain situations, it is beneficial to pre-load the abutment prior to paving; this practice minimizes post-construction deformation or settlement of the GRS mass. This can be achieved by parking fully loaded trucks atop the abutments for several days prior to placing asphalt pavement.

Paving should commence after completion of the integrated approach reinforcement layers. The top layer of reinforcement should be kept approximately 2 in. below the bridge beam grade to allow for a layer of aggregate cover to be placed, which serves to protect the geosynthetic reinforcement from contact with hot mix asphalt. A layer of paving fabric or waterproof membrane should extend over the bridge beams onto the approach roadway; a 3 ft. overlap is recommended to bridge the gap and provide an interface to accommodate thermal movement, minimize surface water infiltration, and prevent cracks in the road (Adams et al. 2011a). If the superstructure has a non-asphaltic wearing surface, the control joint should be detailed to tie the bridge surface with the approach roadway material.

Special care should be taken during guardrail post installation, as excessive driving through geosynthetic reinforcement could compromise the strength of the abutment. Non-displacement steel H-posts are recommended for any railing that is driven through reinforcement layers, while auger pre-drilling prior to setting other types of posts is also possible; both methods are acceptable. In some jurisdictions, guardrail post installation occurs after paving by augering through the asphalt and into the reinforced fill. After posts are set, holes are filled and re-compacted, and an asphalt patch is placed in the area around the post.

### 3.10 Site Drainage

The GRS-IBS construction area should be protected from surface runoff throughout the entire duration of the project. Critical areas include the interface between the GRS wall facing and retained fill, the base of the abutment, and any location where the fill slope meets the wall face. GRS-IBS design should include provisions for surface drainage along the fill slope adjacent to the wing walls, and drainage measures should also be taken at the boundary of wing walls and fill slope. Abutment wing walls are often stepped to reduce excavation; in these situations, termination of wall steps should be sufficiently embedded to prevent problems with erosion. Any drainage swales or channels should be constructed away from the wall to avoid flow directly against wall face (Adams et al. 2011a).

A site preparation plan should contain measures that address grading, diversion trenches, and compaction. The site should be graded to drain away from the GRS mass at the end of every work day. Precipitation should be anticipated each night in order to avoid saturation of foundation and fill material. As an alternative to these grading measures, diversion trenches could be placed around the perimeter of the work area. Any loose soil placed in the GRS construction area should be graded and compacted before stoppage of work each day. Additionally, onsite stockpiles of fill material containing fines should be protected from excess precipitation.

### 3.11 Utilities

All utilities passing through GRS abutments should follow local, state, and federal utility codes. Utilities can be placed in the reinforced zone within a GRS mass, passing in either the parallel or perpendicular direction through the aggregate fill. Geosynthetic reinforcement can be trimmed to accommodate pipes and casing, and extra reinforcement sheets can be added to replace cut out sections. Waterlines within the abutment should be encased in a sleeve pipe to prevent erosion or loss of material should there be a break.

Wall stability, utility ports, repair access, and connections to wall face should be given special considerations when designing utilities within an abutment. Sleeve pipes surrounding waterlines are necessary to ensure leached water exits the abutment without saturating the GRS mass. Pass through portals should be detailed and constructed for fit against the wall face in order to prevent loss of backfill material. Additionally, utility ports should be designed to accommodate any differential movement. Utilities passing through an abutment should be laid out for somewhat easy access should repair or maintenance be necessary. This consideration should pertain not only to the wall's structural stability, but also traffic. Hanging utilities on the wall face is permitted, given the connections are compatible with the wall facing type. These connections should be designed to accommodate lateral and vertical movement associated with the substructure-superstructure interaction.

# CHAPTER 4: TURKEY CREEK GRS-IBS

## 4.1 Introduction

Alabama's first GRS-IBS structure was built in 2018 in Marshall County, spanning Turkey Creek. The site was selected due to its low scour potential and suitable subsurface bearing strata. Construction was completed using design guidance from the FHWA GRS-IBS Implementation Guide (Adams et al. 2011a) and the GRS-IBS Synthesis Report (Adams et al. 2011b), with some modifications. Construction proceeded with few significant issues, although delays due to weather and utility conflicts led to a longer construction period than expected (Hogan et al. 2019). Instrumentation was installed within the abutments to monitor performance; earth pressure, pore water pressure, and geospatial movement were recorded for two years post-construction.

## 4.2 Design of Marshall County GRS-IBS

### *4.2.1 Site Description*

The Turkey Creek GRS-IBS is located in Albertville on Cochran Road which connects US Highway 75 to Marshall County Road 409 (Figure 26). The GRS-IBS spans 40 ft. across Turkey Creek, which has a drainage area of approximately 5 square miles. The 25 year flood elevation for this portion of Turkey Creek is estimated to be 8.8 ft. above the base of the GRS foundation, with a peak runoff rate ($Q_{25}$) of 1850 ft$^3$/s (ALDOT 2017). Despite this relatively high flow, the hard bedrock at the site results in a low scour potential. This native geology was a driving factor in using this site for implementation of Alabama's first GRS-IBS.

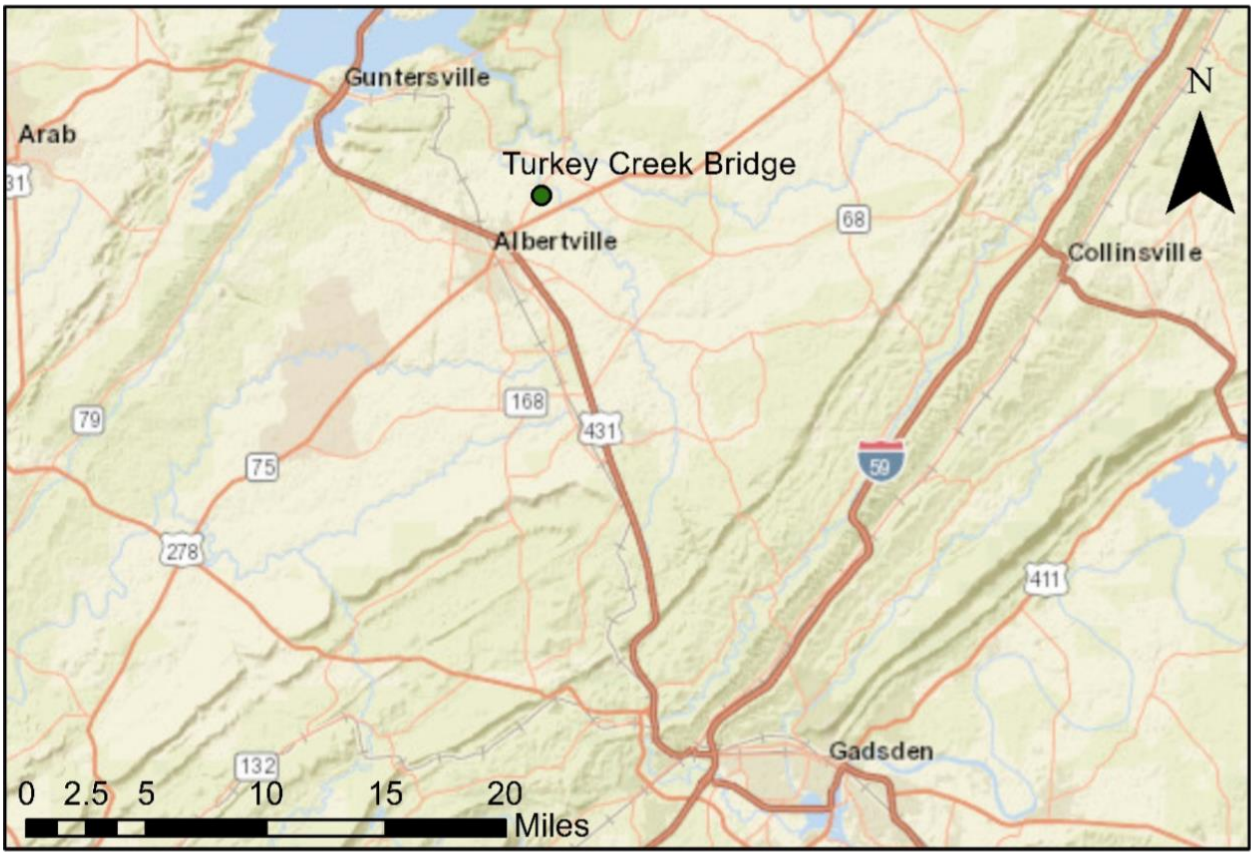


**Figure 25. Location of Marshall County GRS-IBS (Hogan 2018).**

Auburn researchers visited this site in the preliminary stages of the project to visually inspect the condition of the existing bridge. No evidence of scour was encountered near or around the existing abutments or steel I-beam bridge deck supports. During excavation for the GRS-IBS, the channel was widened by approximately 20 ft. in order to lower the velocity of the water and help ensure scour would not become an issue. A portion of the excavated sandstone was placed along the base of the GRS wall; this rip-rap provided additional protection for the new abutments.

*4.2.2 Site Geology*

Marshall County is located within the Sand Mountain region of the Cumberland Plateau, which is the most southerly part of the Appalachian Plateaus province of the Appalachian Highlands Region (Neilson 2007). A further discussion of geological aspects pertaining to this site is included in  Hogan (2018). Rock cores taken at the site (ALDOT 2017) indicate the foundation material was primarily hard sandstone with a 10 to 15-degree dip angle, although thin coal seams were also found. The sandstone had an unconfined compressive strength of 11,300 psi. Borings were completed to depths of 17 ft. and 14.5 ft. on the east and west side of the creek, respectively, and indicated similar material across the site (Figure 27).

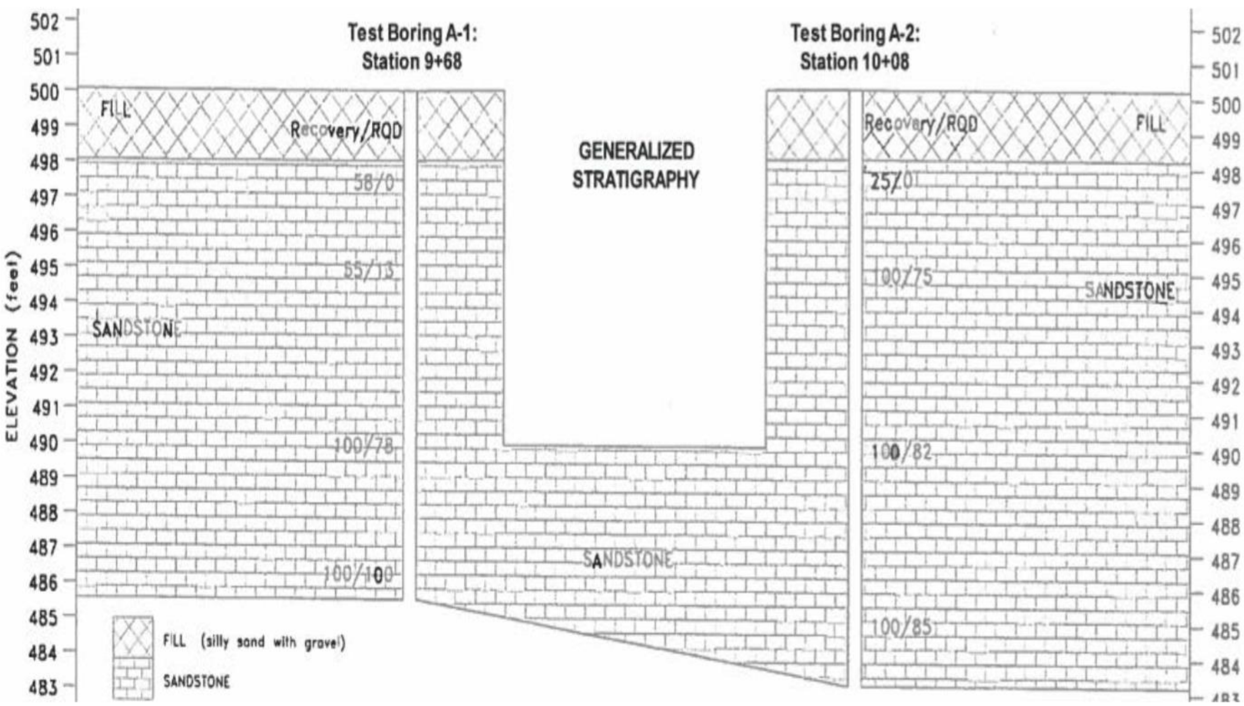


**Figure 26. Borings and generalized stratigraphy of Turkey Creek site (ALDOT 2017).**

*4.2.3 Design*

The Turkey Creek GRS-IBS was designed in accordance with the Federal Highway Administration GRS-IBS Implementation Guide (Adams et al. 2011a) and the GRS-IBS Synthesis Report (Adams et al. 2011b), with some modifications. Initially, three abutment configurations were considered (Elton 2014); constant length, truncated, and stepped reinforcement designs were analyzed to determine factors of safety for bearing and sliding, as well as rock excavation required for each design (Table 2). All three designs provided adequate protection against bearing and sliding failures, so a truncated design was selected in order to reduce the quantity of material that needed to be excavated (Figure 28).

**Table 2: Calculated bearing and sliding factors of safety and volume of rock excavation (after Elton 2014)**

| Type | Constant length | Stepped | Truncated |
|---|---|---|---|
| Bearing FS | 5.7 | 6.0 | 5.9 |
| Sliding FS | 2.9 | 2.0 | 1.9 |
| Estimated rock excavation ($yd^3/yd$) | 12.78 | 9.04 | 8.63 |

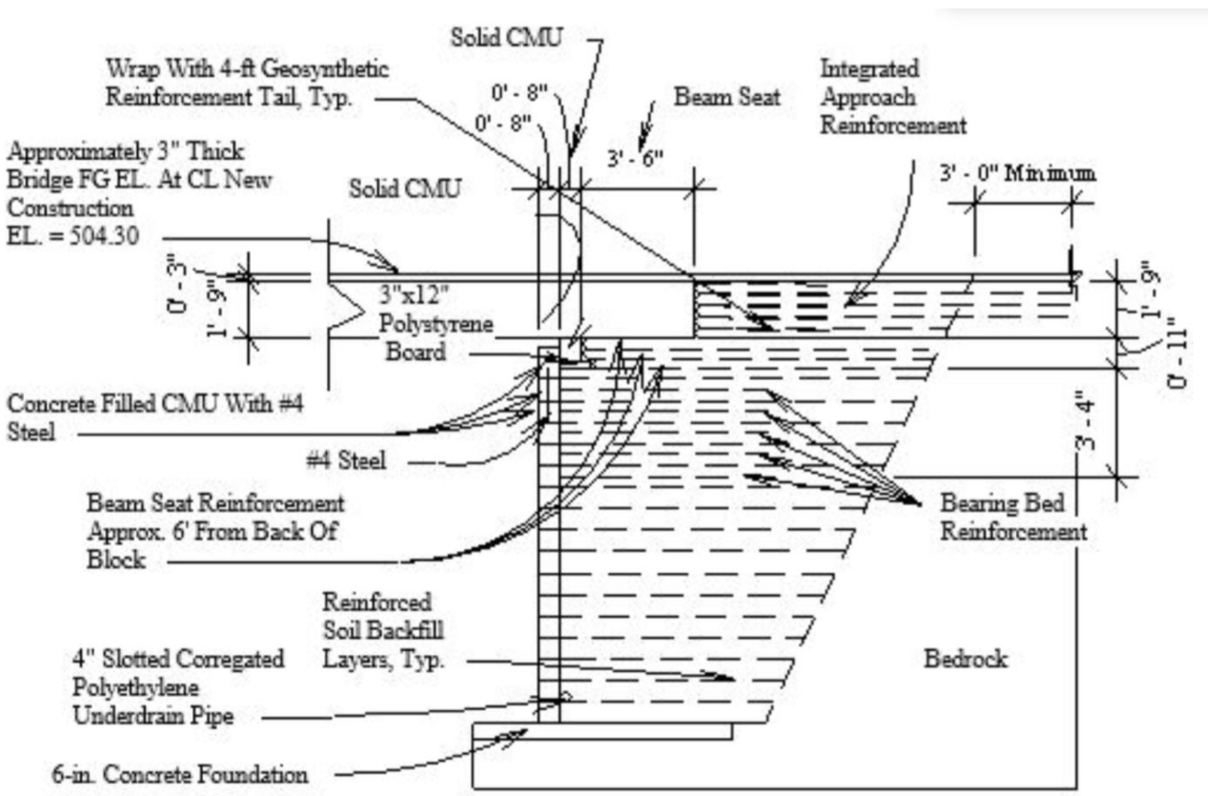


**Figure 27. Design plan using truncated reinforcement (from ALDOT 2017)**

The GRS abutments were approximately 12 ft. high and 33 ft. wide. The wingwalls were 6 ft. wide at the base of the abutment, transitioning to a maximum of 10 ft. wide at the road surface (use figure). A 6 in. thick concrete foundation was constructed on the native underlying sandstone using ready-mixed concrete to serve as a leveling pad for the wall.

The reinforced backfill consisted of No. 89 limestone gravel (Figure 29) with biaxial woven geosynthetic material (Figure 30) spaced every 8 in., which extended to the cut-slope. The geosynthetic material had biaxial strength, which was chosen to decrease the chance of construction misalignment. Additional reinforcement was placed in the bearing bed and beam seat areas to accommodate the extra load imposed by the bridge deck. At the surface, the integrated approach consists of three layers of reinforcement that extended at least 3 ft. beyond the surface projection of the cut-slope to reduce differential settlement.


**Figure 28. AASHTO No. 89 aggregate.**

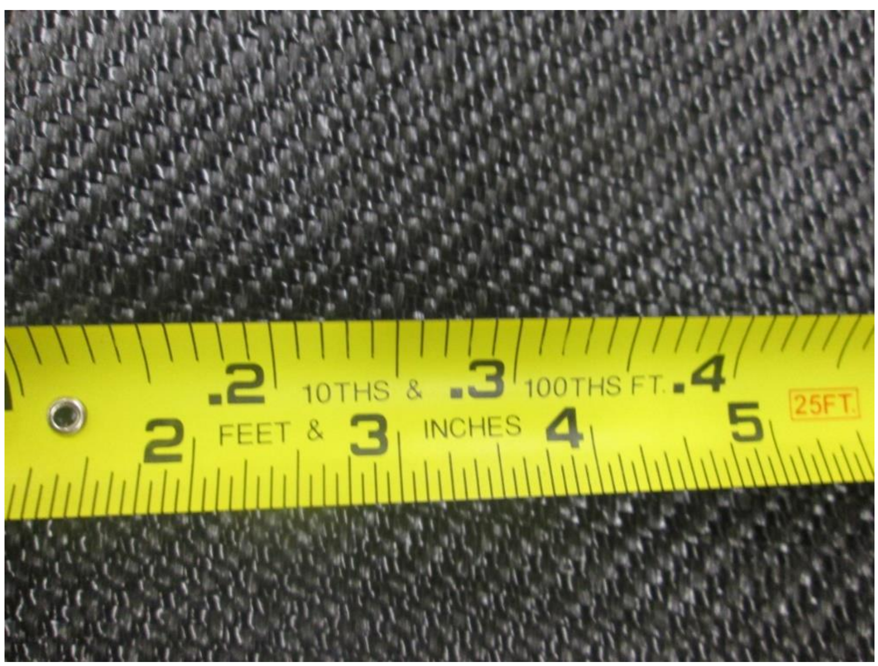


**Figure 29. Biaxial woven polypropylene (PP) geosynthetic with rule for scale (Hogan 2018).**

*4.2.4 Materials*

The GRS abutments were constructed using U.S. Fabrics Type 4800 woven geosynthetics (Figure 31a) and No. 89 limestone gravel (Table 3). Auburn researchers performed Standard Proctor density tests on No. 89 gravel; these tests indicated the as-placed dry unit weight was approximately 105 pcf (Hogan 2018). The geosynthetic material was tested independently by SGI Testing Services, using ASTM D 4595 wide-width tensile strength test (Table 4). The GRS abutments were faced using segmental retaining wall (SRW) cement masonry units (Figure 31b; Table 5). SRW units were selected because they had low absorption, which was considered important to long-term performance of the facing. The bridge deck was constructed using seven 52 ft. long by 4 ft. wide by 1.75 ft. thick precast concrete beams. This layout consisted of five middle beams and two end beams with cross-sectional areas and calculated weights of 4.7 $ft.^2$ and 5.6 $ft.^2$ and 36,956 lbs. and 43,651 lbs., respectively.


**Figure 30. (a) Selected woven geosynthetic; (b) segmental retaining wall masonry units (from Hogan 2018).**

**Table 3. No. 89 gravel gradation results (adapted from ALDOT 2017)**

| Sieve Opening | Test 1 | Specification |
|---|---|---|
| 1/2-in (12.5-mm) | 100.0 | 100 |
| 3/8-in (9.5-mm) | 97.0 | 90-100 |
| #4 (4.75-m) | 33.0 | 20-55 |
| #8 (2.36-mm) | 9.0 | 5-30 |
| #16 (1.18-mm) | 4.0 | 0-10 |
| #50 (300-μm) | 2.0 | 0-5 |

**Table 4. Selected tensile strength measurements of the 4800 geosynthetic (Adapted from SGI Testing Services, LLC 2017)**

| Test No. | Tension at 2% lbs/in | Tension at 5% lbs/in | Tension at 10% lbs/in | Ultimate Strength lbs/in | Ultimate Strain (%) |
|---|---|---|---|---|---|
| 1 | 38 | 146 | 302 | 465 | 18.2 |
| 2 | 45 | 155 | 311 | 446 | 17.6 |
| 3 | 46 | 158 | 310 | 465 | 19.2 |
| 4 | 38 | 147 | 302 | 449 | 17.8 |
| 5 | 37 | 144 | 294 | 454 | 19.8 |
| 6 | 37 | 148 | 309 | 455 | 17.6 |
| Mean | 40 | 150 | 305 | 456 | 18.4 |

**Table 5. Selected test results (ASTM C140-16 and ASTM C1372-16) of segmental retaining wall masonry units (after S&ME 2017)**

| Unit No. | 1 | 2 | 3 | Average |
|---|---|---|---|---|
| Received Weight, lbs | 94.31 | 93.85 | 93.34 | 98.83 |
| Width, in | 10.9 | 11.0 | 10.9 | 10.9 |
| Height, in | 8.0 | 8.0 | 8.0 | 8.0 |
| Front length, in | 18.0 | 18.0 | 18.0 | 18.0 |
| Back length, in | 11.0 | 11.0 | 11.0 | 11.0 |
| Compressive strength, psi | 10,220 | 8,640 | 8,610 | 9,160 |
| Saturated (SSD) wt., lbs | 56.27 | 55.25 | 54.66 | 55.39 |

| | | | | |
|---|---|---|---|---|
| Oven Dry wt., lbs | 54.66 | 53.79 | 53.20 | 53.88 |
| Absorption % | 2.9 | 2.7 | 2.7 | 2.8 |
| Density, pcf | 144.0 | 143.6 | 144 | 143.9 |

Three consolidated-drained (CD) triaxial tests were conducted on the gravel by Auburn researchers. Confining stresses of 7.5 psi, 12.5 psi, and 17 psi were used. An average friction angle of 46 degrees was determined from these tests. The secant shear modulus of 10,129 psi at about 0.6% strain was the limit of the linear range on the stress-strain plots from the CD tests. Additionally, strain hardening became prominent as the material dilated during shearing.

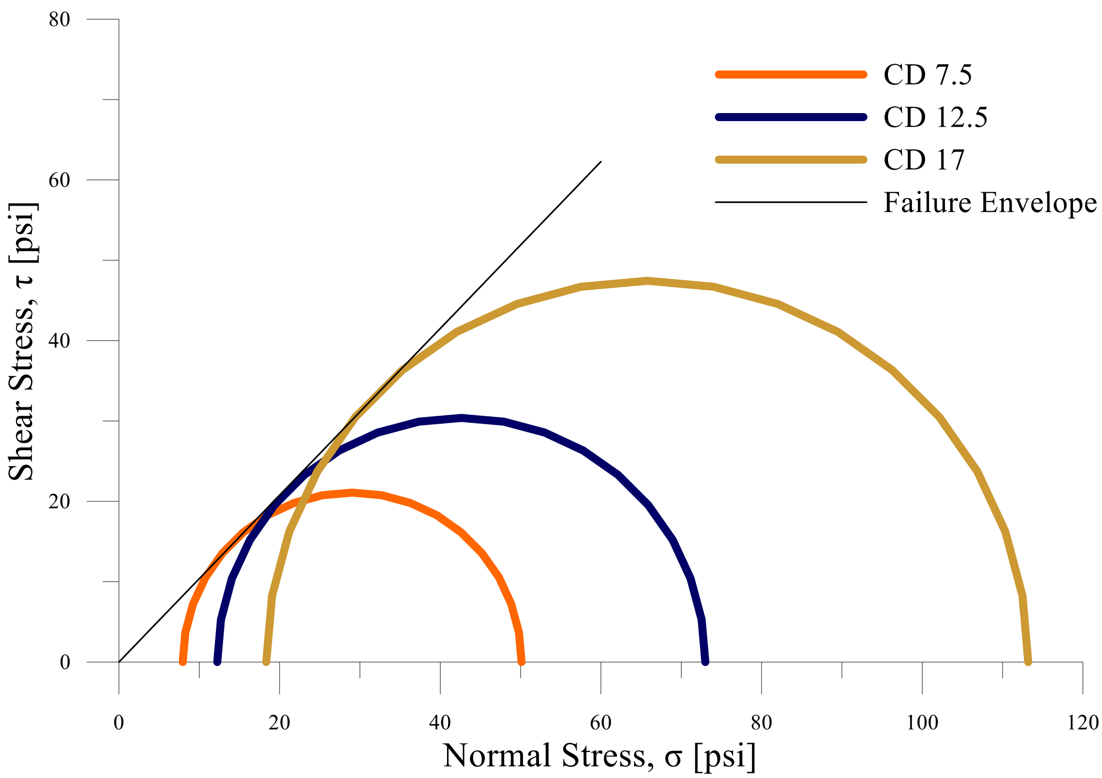


**Figure 31. Mohr-Coulomb failure criteria for selected No. 89 backfill (from Hogan 2018).**

## 4.3 Construction

### *4.3.1 Demolition and Excavation*

Construction began on October 2nd, 2017 and started with removal of the existing bridge and excavation of native material. A hydraulic excavator removed the existing bridge and abutments (Figure 33), as well as surficial soil and rock. Blasting was utilized to excavate hard sandstone to the required elevation, and an excavator was used to remove the blasted material. Berms constructed from native material were used to limit inflow of water from the creek into the excavation; this practice was only mildly effective, and pumps were used to dewater the excavation prior to placement of the concrete pad.

**Figure 32. Existing Bridge Superstructure and Abutments.**

*4.3.2 Concrete Foundation*

The leveling pad was 6 in. thick, 8 ft. deep, and 33 ft. wide. Concrete forms were set at the correct elevation, 490.33 ft., using a traditional tripod-mounted level and grade-rod. Ready-mix concrete was brought to the site and placed using a concrete bucket attached to a hydraulic excavator. Concrete was initially placed around the outside of form boards to keep concrete from leaking out of the formwork. Once concrete around the perimeter hardened, concrete was placed in formwork and finished using traditional hand tools. Surface grading was performed on the finished pad as needed to obtain a level surface for the block placement.

**Figure 33. Construction of east abutment concrete levelling pad foundation.**

*4.3.3 GRS Abutments*

The initial row of masonry SRW blocks for the GRS abutment were placed atop the concrete foundation by hand using a string-line as a guide and the centers of the blocks were filled with concrete. No. 89 backfill was placed in the area behind the blocks prior to being leveled and lightly compacted to be even with the top of the first row of blocks. The fist layer of geosynthetic material was placed at this elevation, then the $2^{nd}$ row of masonry blocks was set into place. Joints between blocks were offset with the row of blocks below; this practice was implemented to minimize seams within the wall face, therefore preventing backfill material from migrating out of the abutment (Figure 35). A 4 in. corrugated drain pipe was placed behind the second course of blocks to allow for drainage of backfill. This process of backfilling masonry blocks with No. 89 backfill material and adding a layer of geosynthetic every 8 in. was repeated for a total of 17 layers at a batter of 1:32. The final 3 rows of masonry blocks were reinforced and grouted using No. 4 reinforcing steel 8 in. on-center and ready mix concrete, respectively (Figure 36).

**Figure 34. $6^{th}$ course of west abutment. Masonry SRW block joints were offset between courses to minimize seams within the wall face.**

**Figure 35. 16th course of east abutment. This is the second of the top three rows, which were pinned with No. 4 rebar and grouted with ready-mix cement.**

The beam seat is the area of the abutment which receives extra reinforcement to carry the load of the bridge deck. The beam seat was located at an elevation of 498.33 ft. ending at the top of the abutment; this area is referred to as the bearing bed. The bearing bed was 4 in. deep and extended 6.5 ft. from the back of the masonry blocks. An 11 in. wide beam seat was constructed immediately below the beam elevation using closely spaced geosynthetic and No. 89 backfill material that extended 6 ft. from the back of the SRW units. A detailed schematic of this configuration can be found in Figure 37.

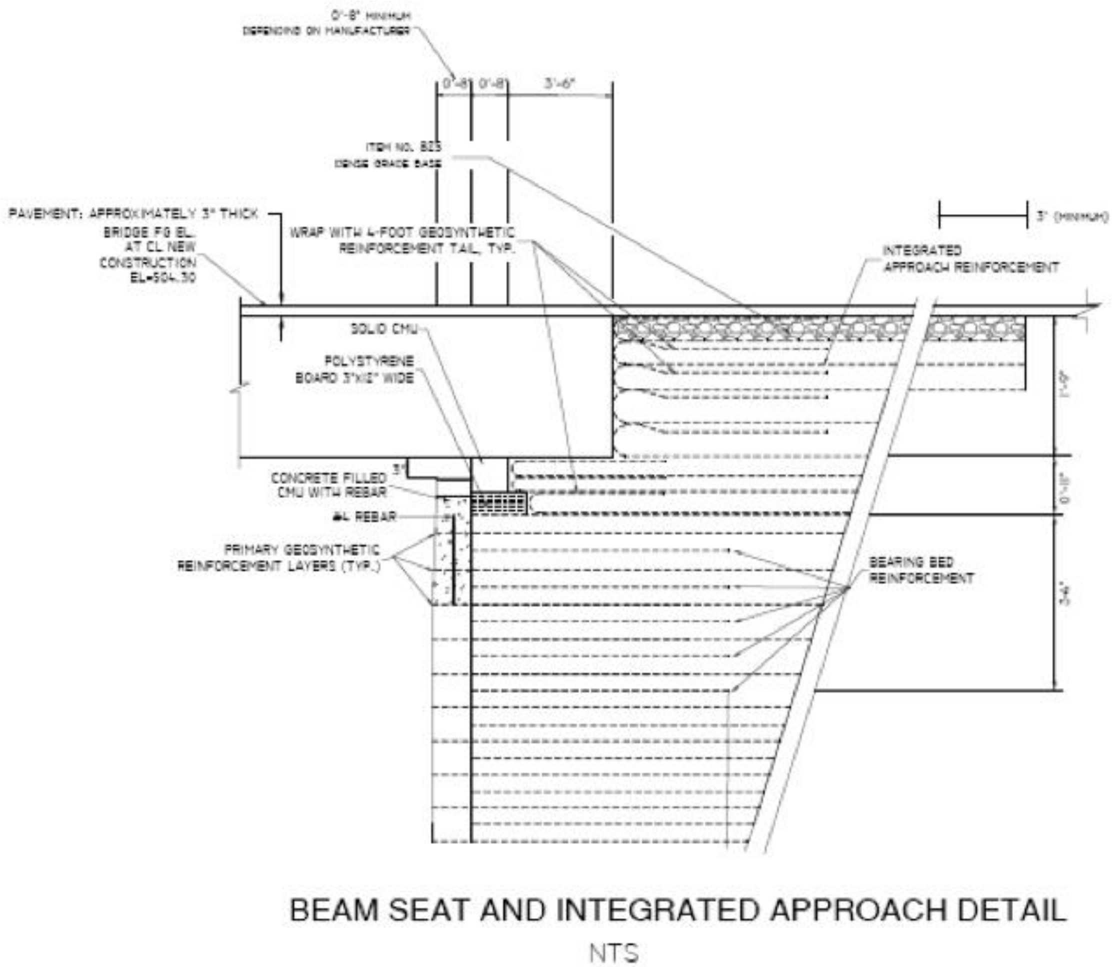


**Figure 36. Beam seat and integrated approach structural details.**

*4.3.4 Beam Placement*

Precast bridge beams were placed on December 8th, 2017- approximately 54 days after placement of the final concrete leveling pad. The bridge beams are directly supported by solid SRW units placed on a 3 in. by 12 in. polystyrene board at the top of the reinforcing layer of abutments; this practice left 3 in. of space between the abutment and bottom of the beams (Figure 38). The beams were placed with a crane prior to being post-tensioned using three 1 in. diameter steel tie-rods (Figure 39).


**Figure 37. Polystyrene foam board used to provide 3 in. of clear space for the east abutment beam seat.**


**Figure 38. Placement of final precast concrete beam atop the beam seat.**

The integrated approach was constructed after placement of bridge beams. This area starts just behind the beams and extends a minimum of 3 ft. from the end of each beam. The integrated approach consists of four closely spaced layers of geosynthetic folded around dense-grade base which ends level with the top of the bridge beams.

### *4.3.5 Final Grading and Paving*

Rain and utility conflicts delayed final grading and paving for approximately two months after the completion of the integrated approach; operations were completed on May 22nd, 2018. Once paving was completed, guardrails were installed and the bridge was open to traffic on June

3rd, 2018. (Figure 40). The entire construction process took approximately 9 months, from demolition of the existing bridge to opening of the GRS-IBS.

**Figure 39. The completed GRS-IBS over Turkey Creek in Marshall County, AL (from Hogan 2018).**

*4.3.6 Construction Issues*

Construction of the abutments proceeded as expected for the most part. However, minor problems with initial placement of the foundation and poor dimensional tolerances of the SRW units caused issues. The EOR prevented a potential mishap early on by checking the span length relative to the layout of the initial row of SRW units. Evidently, the contractor had begun placing the initial row of SRW units such that the final span length would have been greater than the design length of 40 ft.; this error would have increased the pressure imposed by bridge beams onto the abutment by reducing the bearing area.

### 4.4 Project Cost

The total project cost was about $650,000; the bridge itself accounted for approximately $317,000 with roadway construction making up the rest of the cost (Hogan 2018). The costs of each major component of the project were as follows: $21,600 for construction of the two concrete levelling pads, $115,600 for the GRS abutments, $172,410 for the 7 PPC box beams, and $7,200 for 240 lbs. of structural steel (Pirando 2020).

# CHAPTER 5: INSTRUMENTATION AND FIELD MONITORING

Auburn University researchers were tasked with observing the Turkey Creek GRS-IBS and monitoring performance over a period of two years. Pore-pressure and earth-pressure readings were collected continuously by piezometers earth pressure cells connected to data loggers. Periodic surveys were performed to document settlement and lateral displacement of the abutments.

## 5.1 Instrumentation

Two Geokon 4500 standard vibrating wire piezometers were used to monitor pore water pressures within the abutments (Figure 41). A thin wire located within the stainless-steel housing transmits a frequency based on the tension in the wire; the wire is connected to a diaphragm at one end that deflects either in or out, depending on the pore water pressure. The sensors were saturated prior to being placed in sand-filled sleeves, which protected them from damage imposed by backfill material. The cables were routed along the length of the abutment and through a small opening in the SRW units created with a chisel and hammer (Hogan 2018).


**Figure 40. Model 4500 standard vibrating wire piezometer (Geokon 2017)**

Two Geokon 4810 vibrating wire earth pressure cells (EPC) were used for the project (Figure 42). This type of sensor consists of a circular plate underlain by a thin casing that contains oil and a diaphragm. Pressure exerted onto the instrument changes the tension of the vibrating wire. This instrument measures total stress; thus, additional pore pressure sensors were necessary to estimate effective stress. During installation, fine sand was used to cover the sensors to reduce the effects of soil arching and protect the cells from damage due to the angular aggregates.

**Figure 41. Geokon Model Pressure Cell (Geokon.com).**

The east and west EPCs were installed at elevations of 491.00 and 490.33 ft, respectively. This corresponds to 8 in. above the concrete foundation of the east abutment and directly on the concrete foundation of the west abutment. The piezometers are located at an elevation of 491.00 ft, at the approximate center of the abutments near the location of the EPCs (Figure 43). Two Campbell Scientific CRWV3 data loggers were used to measure output signal and supply excitation voltages produced by the pressure cells; these systems are specifically designed to be compatible with vibrating-wire sensors and have 16MB of storage. In addition to recording earth pressure readings, these data loggers also recorded pore water pressure readings. Following placement of the final course of blocks, data loggers were mounted on the southeast and northwest sides of the east and west abutments using concrete screws and construction adhesive.

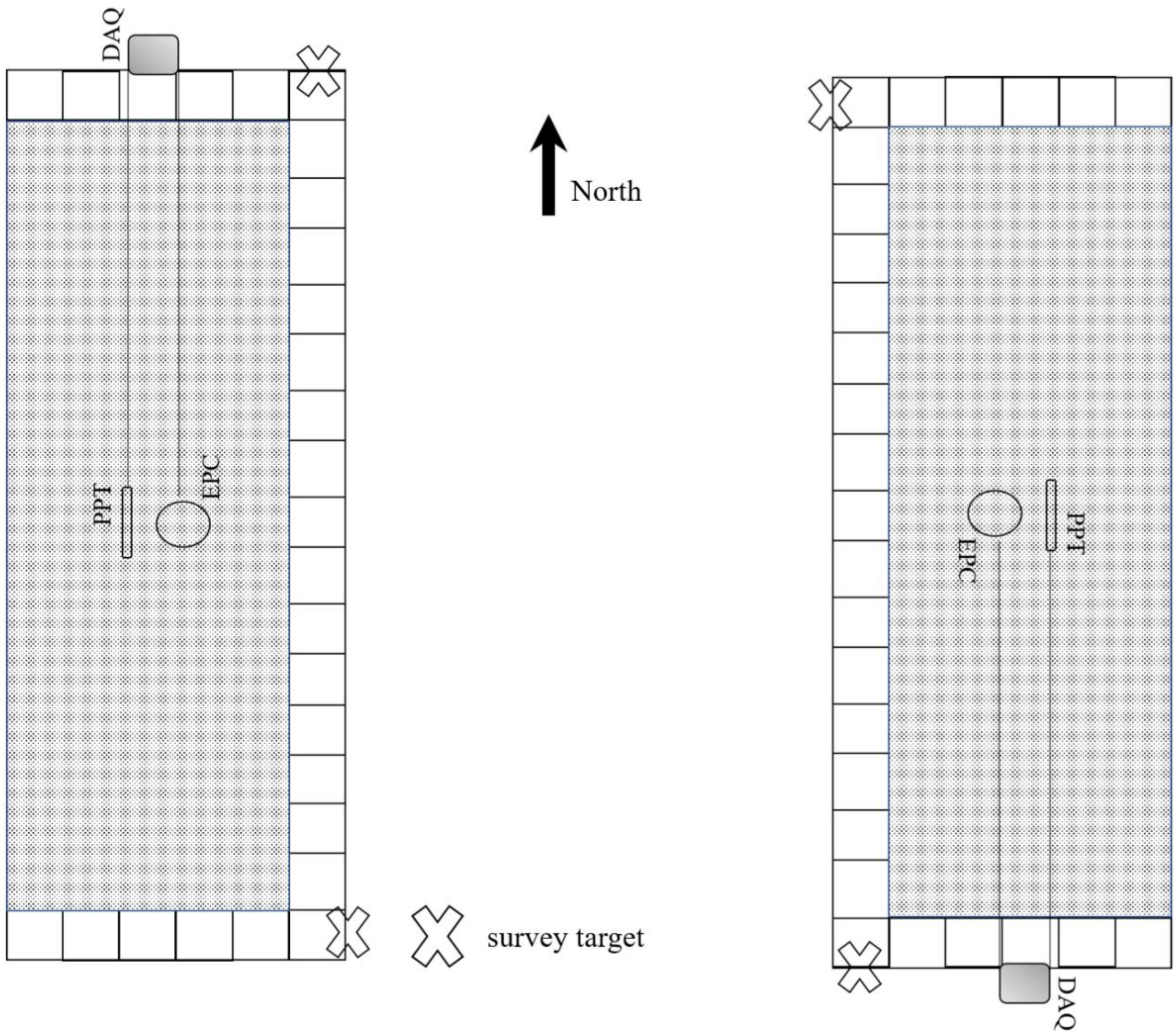


**Figure 42: Plan view of the layout of the sensors and survey targets used on the GRS-IBS structure (from Hogan 2018).**

A Topcon GTS-235W was used to measure geospatial data for the project (Figure 44). This instrument has a laser plumb, as well as vertical and horizontal angle tilt correction sensor that ensures accurate readings. Bernsten RS60 survey targets were attached to corners of the bridge abutments; these targets are reflective and compatible with a total station. The size of the reflective cross section is 1.57 x 1.57 in., therefore the approximate recommended minimum and maximum range is 33 and 328 ft., although according to Berntsen (2018) most total stations can exceed the maximum value. Four survey targets were placed at each corner of the abutments using construction adhesive, and a fifth target was placed on a power pole just east of the project as a bench mark (BM) for subsequent surveys. Surveys began after the bridge beams were placed, on February 9, 2018, and were conducted approximately every month since then.


**Figure 43. Topcon GTS-235W total station (Precision Geosystems).**

## 5.2 Results

### *5.2.1 Earth Pressure*

Readings from the earth pressure sensors in the east and west abutments are shown in Figure 45. Manual readings were taken immediately after abutment construction to establish a zero reading. The reading at this time was approximately 600 psf; this is approximately half of the vertical stress that would be expected based on one-dimensional stress conditions using the height of the backfill and estimated unit weight. This value is consistent with patterns observed experimentally and numerically by Bathurst et al. (2000) and Hatami and Bathurst (2005). This lower vertical stress can be attributed to load shedding due to friction interface of the GRS material and the inner face of the CMU (Hatami and Bathurst 2005). The weight of the precast concrete bridge beams and backfill material is estimated to apply 1200 psf to the surface of the abutment; combined with the 600 psf imposed by the GRS mass, earth pressure sensor readings would be expected to be approximately 1800 psf. The most recent readings show earth pressures for the east and west abutments to be approximately 1700 psf, which is slightly lower than anticipated. However, readings taken between September of 2018 and March of 2019 fall in the range of approximately 1300-1550 psf, suggesting that seasonal temperature changes could influence the pressure sensor readings. While the east abutment pressure readings rose to expected values during the summer of 2019, the west abutment readings still fell within the low range. However, pressure readings for both abutments have remained near 1700 psf during the most recent year, which is only slightly lower than anticipated.

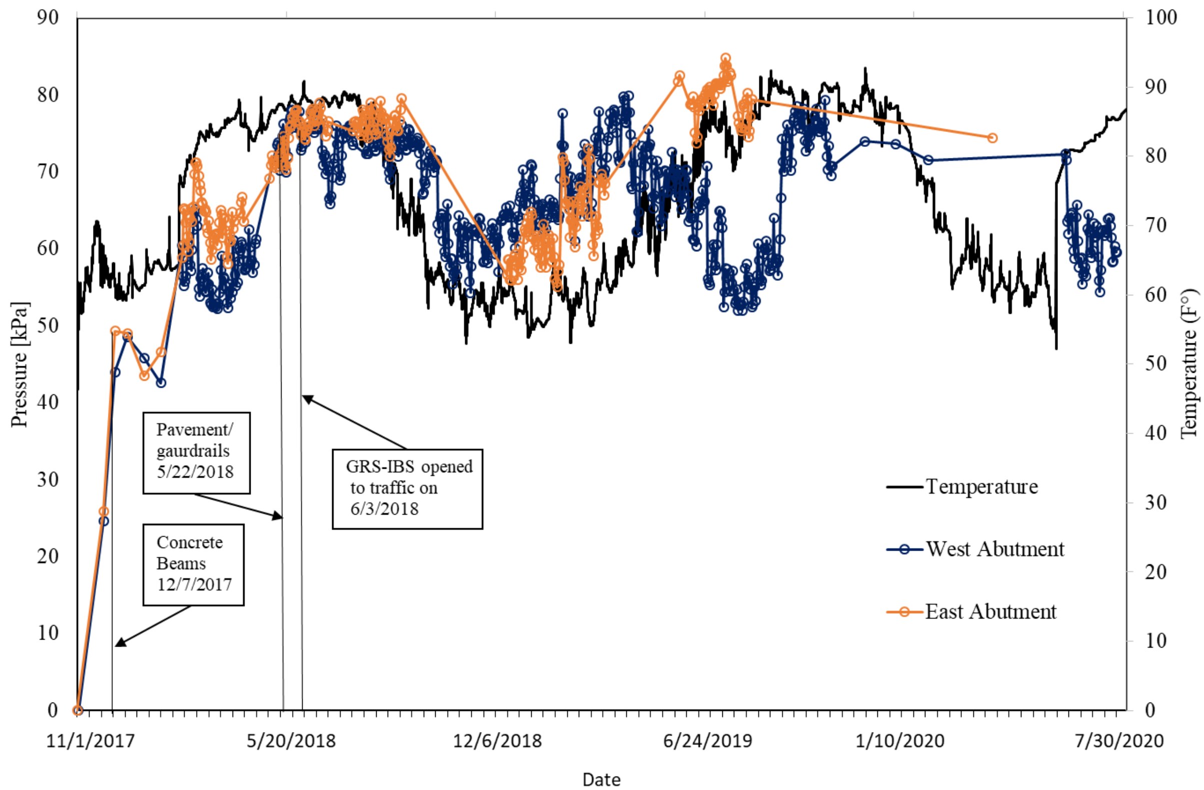


**Figure 44: Earth pressure sensor readings for the east and west abutments.**

*5.2.2 Pore-Water Pressure*

Pore-pressure readings have remained near zero, as expected (Figure 46). The data shows minor oscillations, which are likely a result changes in air pressure and temperature. The pore-pressure sensors have not shown evidence of saturation, indicating the abutments are dry and the creek level has never risen beyond the elevation of the sensor. It should be noted that the piezometers are not designed to read pressure in unsaturated sand or gravel, so the data set could potentially contain inaccuracies. However, the readings are still within the error of the instrument.

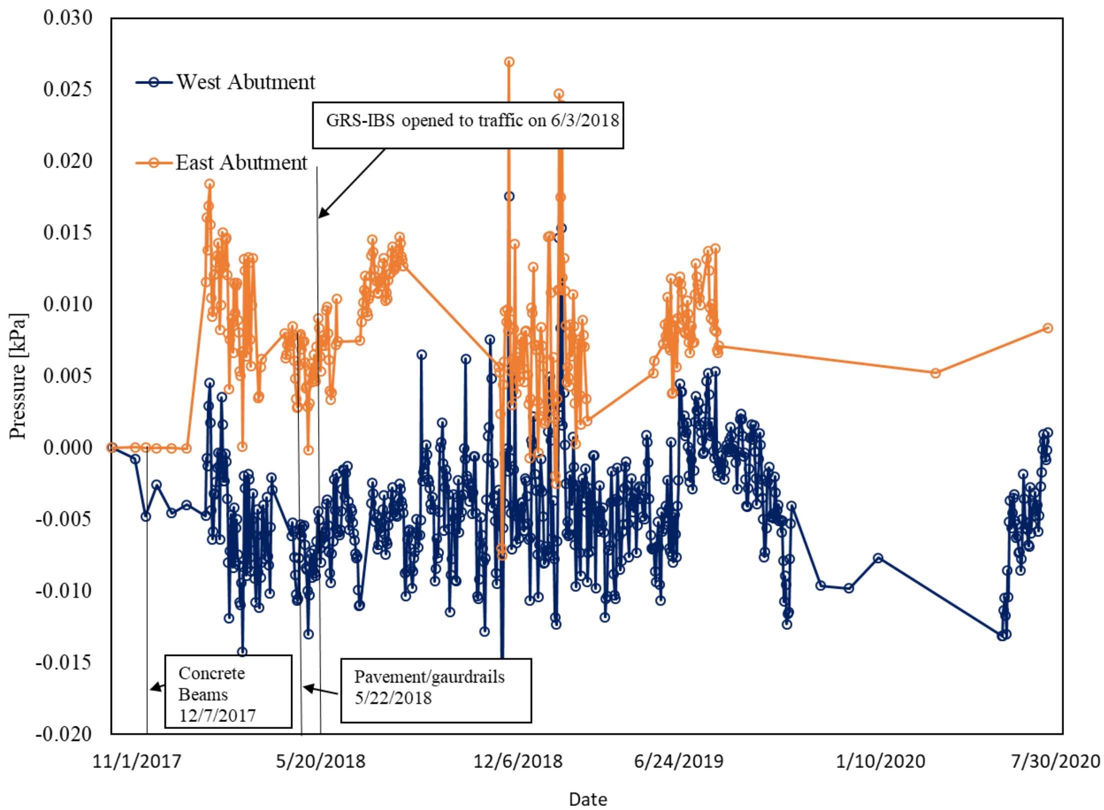


**Figure 45. Pore pressure sensor readings for the east and west abutments.**

*5.2.3 Settlement*

Periodic geospatial surveys indicate the abutment has experienced little to no settlement (Figure 47). The northern corner of the eastern abutment appears to have settled slightly after construction, and the remaining readings do not reflect any settlement of the GRS mass. The abutments were constructed atop hard sandstone, which is not expected to settle significantly over time. Most primary settlement of the abutment was expected to occur immediately after construction; surveys did not begin until a few weeks after the bridge beams were installed, so this movement was not recorded. Overall, settlement readings have remained stable and virtually unchanged. Oscillations in settlement readings are due to instrument and measurement uncertainties.

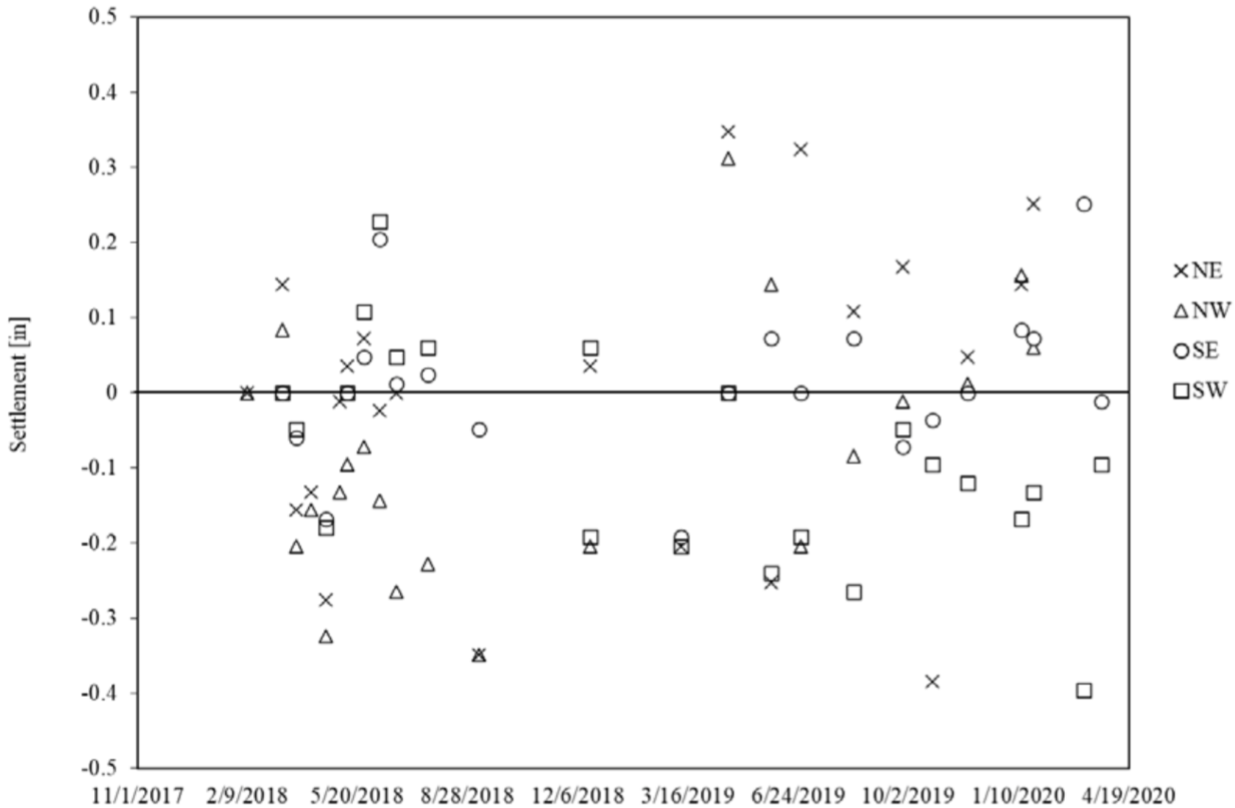


**Figure 46. Settlement measurements of all survey locations.**

Settlement measurements of each abutment corner with uncertainty bands are displayed in Figures 47 through 50. The uncertainty bands shown in these figures were calculated based on measurements taken from the BM height at each control point. Since the BM is assumed to have a constant position, the average uncertainty was estimated by subtracting the lowest BM height measurement from the highest, then dividing by two. Two control points (ECP and WCP) were used for the surveys, so two average uncertainties were calculated. The calculated uncertainties for the ECP and WCP were 0.358 in. and 0.386 in. respectively. The lower limit of each uncertainty band was calculated by subtracting the average uncertainty from the average displacement measurement of that abutment; the upper limit was calculated by adding the average uncertainty to the average displacement measurement. Each abutment had a different average displacement, so four uncertainty bands were calculated and plotted. Most of the uncertainty in these measurements is likely due to errors in measuring the height of the instrument (HI). It should be noted that the ECP was destroyed in May of 2018 and had to be re-established; tampering and vandalism to instrumentation could adversely affect instrument readings. Between May and December of 2018, rainfall eroded the ground around the new ECP, causing the embedded rebar to protrude approximately 0.17 in. above the ground; this control point was originally installed flush to the ground. Before this discrepancy was noted, the instrument operator could have potentially measured the HI from the ground, rather than the top of the control point. Other sources of error include target/total station misalignment, the precision of the total station, and possible movement of the wooden power pole on which the BM was fixed; the contribution of these other factors is likely small compared to the error in measuring the HI (Hogan 2018).

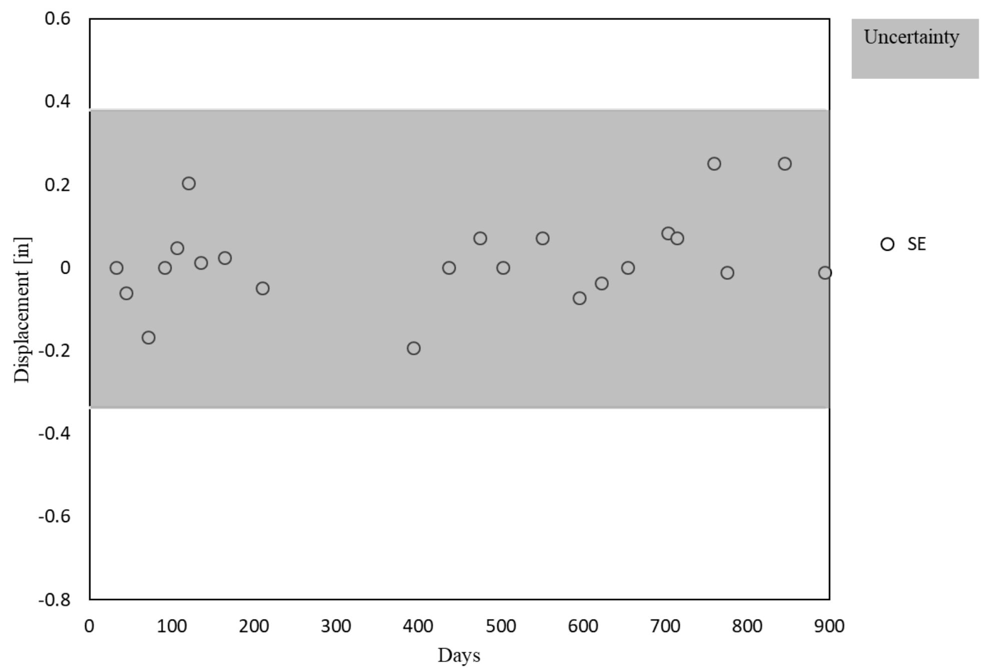


**Figure 47. Settlement measurements of the southeast corner of the east abutment.**

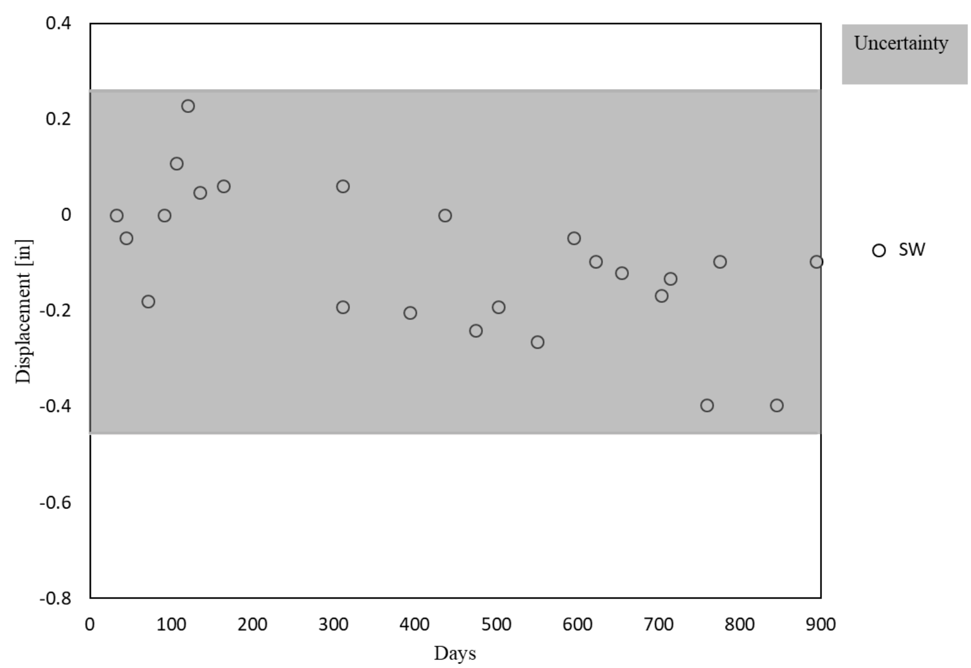


**Figure 48. Settlement measurements of the southwest corner of the west abutment.**

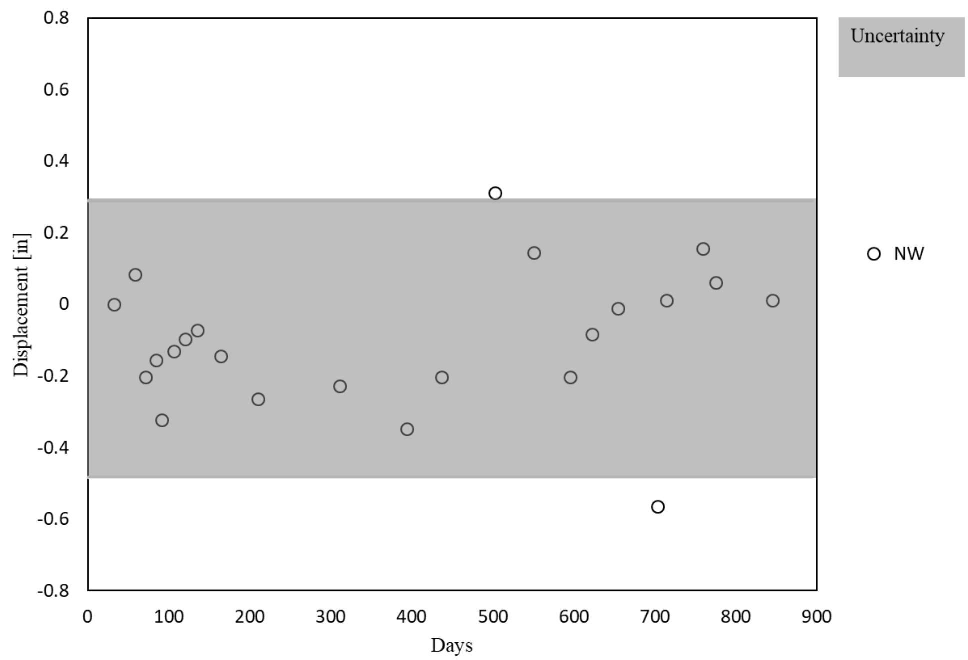


**Figure 49. Settlement measurements of the northwest corner of the west abutment.**

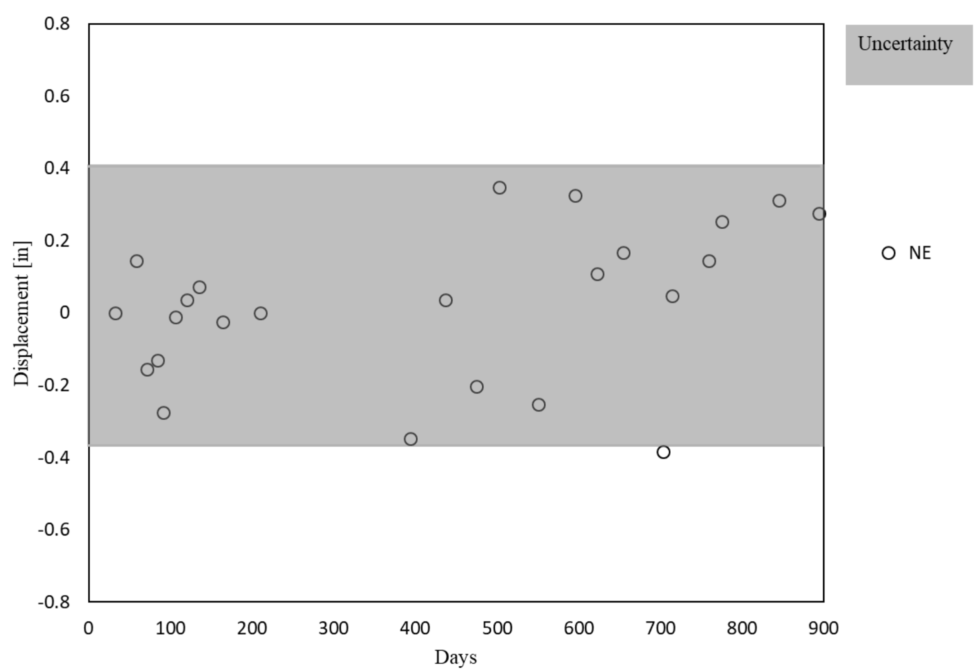


**Figure 50. Settlement measurements of the northeast corner of the east abutment.**

*5.2.4 Lateral Displacement*

As observed during geospatial monitoring, the surveys have not measured any discernable lateral movement of the abutments (Figure 48). Positive lateral displacement is defined as the inward movement of the abutment (i.e. compression). Most recorded movements were within the range of instrument uncertainty; however, a few measurements fall outside of this range. Thermal expansion and contraction of the SRW units is the most probable cause for these displacements, as they oscillate about zero and no discernable trend is noticed.

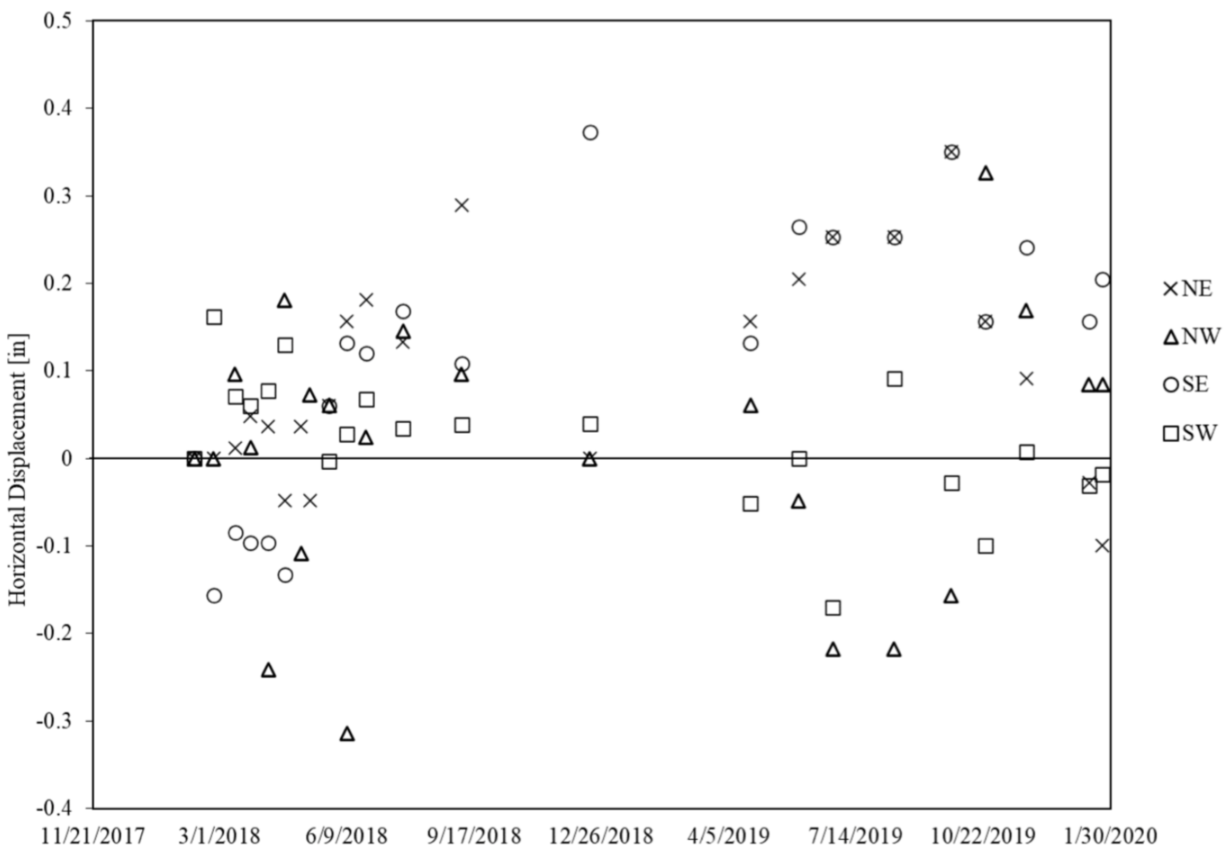


**Figure 51. Lateral displacement measurements of all survey locations.**

Lateral displacement measurements of each abutment corner with uncertainty bands are displayed in Figures 52 through 55. The uncertainty bands shown in these figures were calculated in similar fashion to the settlement uncertainty bands; however, the BM easting was used for reference rather than height. The horizontal angle reading of the BM was set to zero at the start of each survey for both the ECP and WCP.

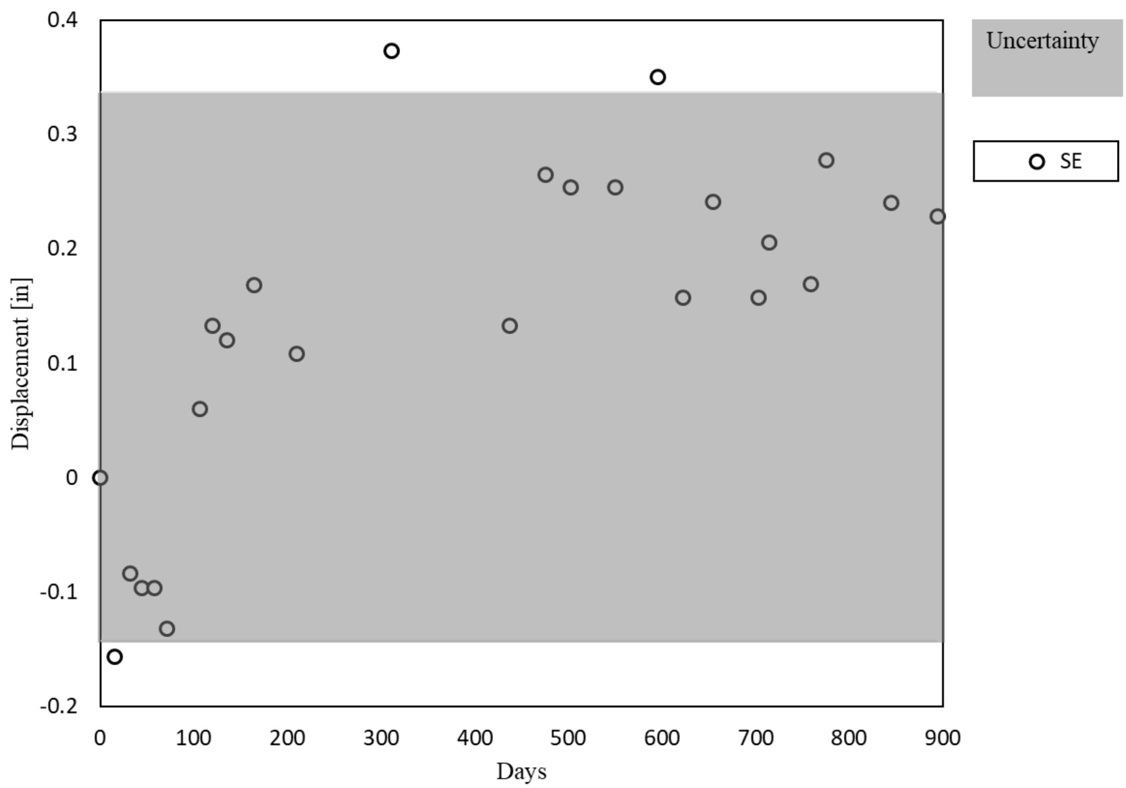


**Figure 52. Measured lateral displacement of the southeast corner of the east abutment.**

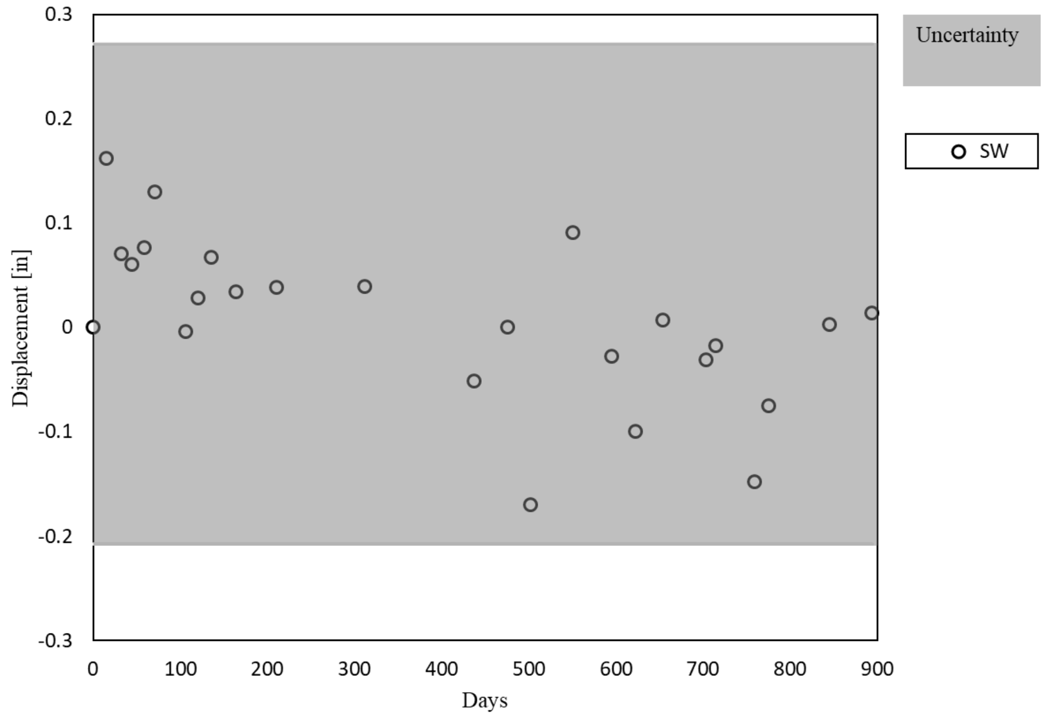


**Figure 53. Measured lateral displacement of the southwest corner of the west abutment.**

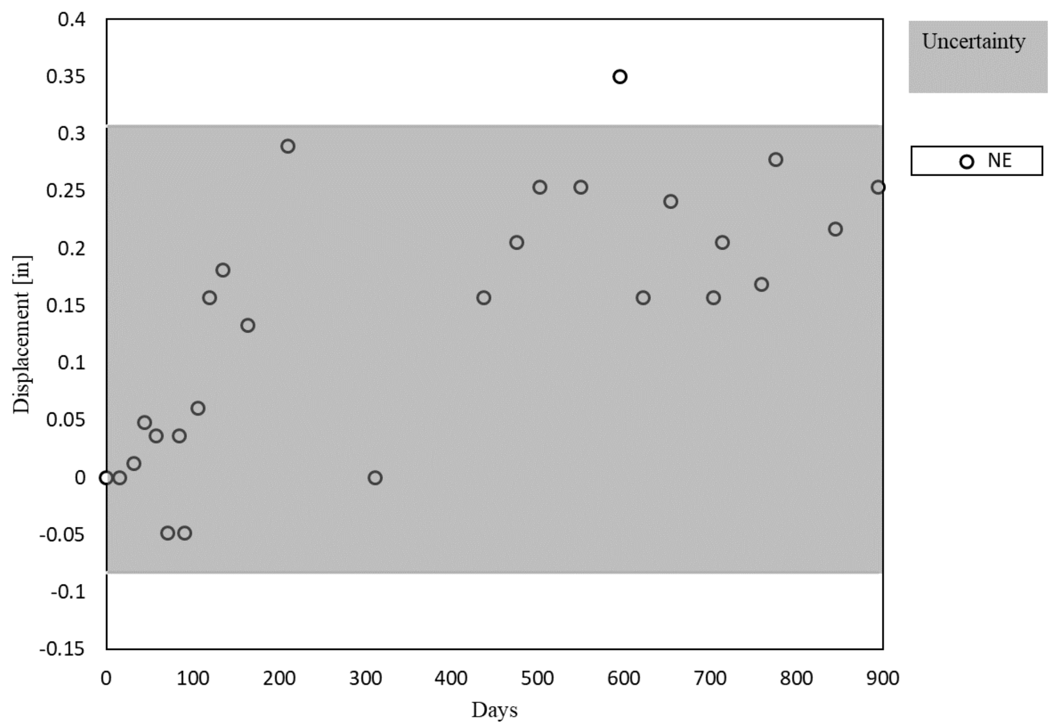


**Figure 54. Measured lateral displacement of the northeast corner of the east abutment.**

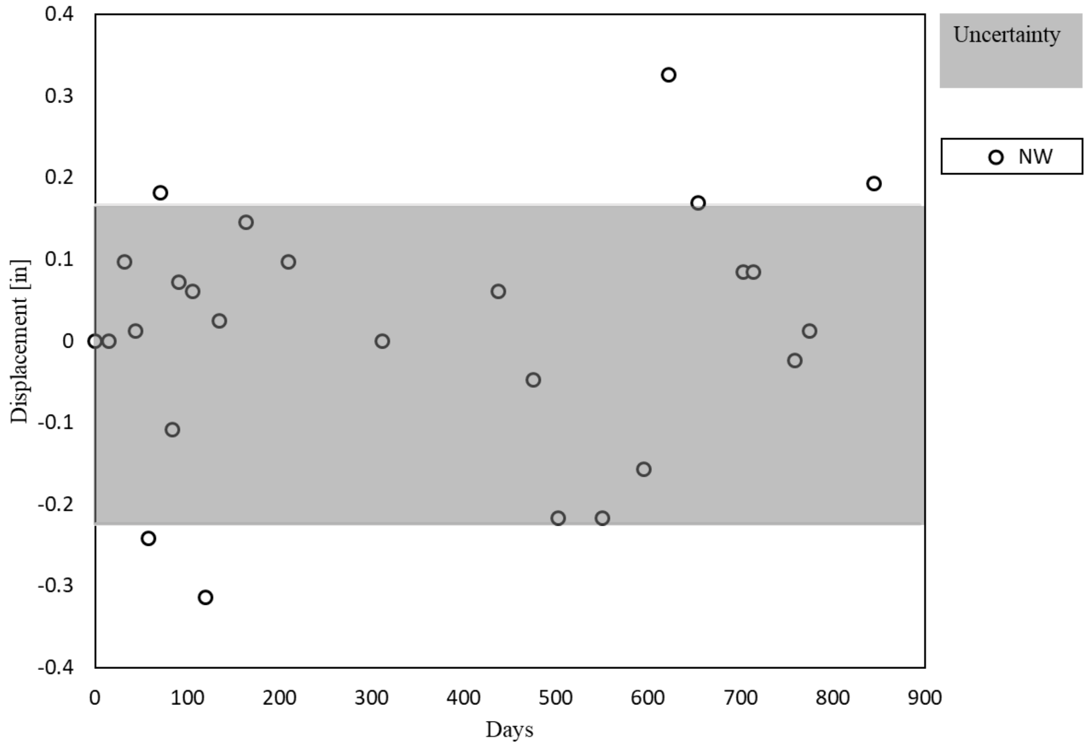


**Figure 55. Measured lateral displacement of the northwest corner of the west abutment.**

# CHAPTER 6: SPECIAL PROVISION

Auburn researchers were tasked with creating a GRS-IBS construction specification for the Alabama Department of Transportation (ALDOT). Information pertaining to GRS-IBS specifications, case studies, and specific experiences was gathered from multiple state DOTs. Relevant information was consolidated into a draft Special Provision, which was brought before the ALDOT Materials and Testing Bureau prior to revision and resubmission. The most recent draft of this Special Provision is included within Appendix A of this report.

## 6.1 State-of-Practice Review

The Auburn geotechnical research team reached out to multiple state DOTs primarily those operating in the southeastern United States; Florida, Kentucky, Louisiana, North Carolina, Virginia, and West Virginia all responded to email requests for information. Further information online was gathered from Puerto Rico, New York State, and the FHWA. The majority of information gathered from the DOTs consisted of Special Provisions, plans and specifications, and case studies.

Several states shared specific advice related to writing a Special Provision. The Virginia Department of Transportation (VDOT) shared notes pertaining to design nuances. The designer stated that the final elevations of the superstructure should be above the wingwalls to a height equivalent to the clear space to allow for settlement; 2 to 4 in. of clear space is recommended between the beam and the wall. He also stated that if a fascia (drip-edge) is not required, then eliminate the foam board on top of the half-height blocks at the top of the GRS wall. He also recommended that CMU type should be explicitly stated in the plans, and not left up to interpretation by the contractor; this includes, solid or hollow blocks, split-face, and color specifications (Weaver 2016). Additionally, the VDOT shared a study that study that documented MSE wall failures; out of 141 case studies, 91% contained geogrid reinforcement (Koerner and Koerner 2012). For this reason, polypropylene (PP) biaxially woven geotextile material is recommended for GRS abutment reinforcement.

The Louisiana Department of Transportation and Development (LaDOTD) wrote a GRS-IBS special provision prior to construction the Maree Michel bridge, drawing from the LaDOTD existing MSE wall specification, FHWA Interim Implementation Guide, and GRS-IBS specifications from other state DOTs. Jesse Rauser, PE stated that special care should be taken in separating guidelines from necessary specifications; for instance, backfill compaction methods should be outlined in the individual plans rather than the general specification (Rauser 2016). He also recommends allowing related personnel, such as district construction inspectors and engineers, to review the plans and specifications prior to publication. The LaDOTD recommends making the specification relevant on a local level; climate concerns and material availability should be accounted for when writing a specification. The Louisiana GRS-IBS special provision was written in June 2012, prior to publication of the FHWA sample specification in August of that year. While this resource was not available for the LaDOTD, they recommend following the FHWA guidelines when writing a state-specific GRS-IBS special provision.

Additionally, LaDOTD shared construction advice related to quality assurance.

- Enforcing the compaction specification is critical; certain best practices, such as hand tamping and walking on blocks, should be required.

- Since a method specification is harder to quantify with records, having an experienced inspector onsite is recommended.

- Modifications to the blocks as a substitute for good compaction should be discouraged.

- A detailed QA/QC plan for the facing elements should be incorporated, with a mechanism to reject facing elements in the field.

- Reviewing plans and specifications with the contractor on a regular basis is encouraged.

## 6.2 Document Structure

Review of GRS-IBS construction specifications from other states suggested the document would be best structured into five sections; these sections were Description (01), Materials (02), Construction (03), Measurement (04), and Payment (05). Description (01) contained contractual language, definitions, and references to outside documents. Materials (02) contained references to ASTM and AASHTO specifications that construction materials must meet, as well as references to ALDOT's general construction specification manual. Construction (03) outlines practices for building a GRS abutment and placement of superstructure; this section draws primarily from the FHWA GRS-IBS Interim Implementation Guide (Adams et al. 2011a), as well as other states' construction specifications. Measurement (04) discusses how units of material and labor are to be quantified. Payment (05) discusses compensation for work performed, unit bid contracts, and specific pay items. It was determined that ALDOT personnel should structure Description (01), Measurement (04), and Payment (05) how they felt most appropriate; several example structures from other states were included in the draft specification in order to facilitate this. Materials (02) and Construction (03) were written independently by Auburn researchers, minus several suggestions made by state personnel.

After an initial draft Special Provision was written, Auburn researchers met with the following state and county personnel: Kaye Chancellor Davis, P.E., ALDOT Assistant Materials and Testing Bureau Chief; Renardo Dorsey, P.E., State Geotechnical Engineer; and Robert "Bob" Pirando, P.E., Marshall County Engineer. State personnel recommended several revisions, which were addressed in the subsequent resubmission. Pirando commented that the CMU minimum compressive strength of 4000 psi and maximum water absorption of 5% that is recommended by the FHWA is likely over-conservative; these specifications for compressive strength and absorption limit were modified to 3100 psi and 10%, respectively. The initial draft also recommended that CMU blocks be tested for freeze-thaw durability in accordance with ASTM C1262. Due to the mild climate in Alabama, this clause was removed. The initial draft stated that aggregate should have a plasticity index (PI) less than or equal to 6, a pH of 4.5 to 9, and an organic content less than 0.5 percent; this clause was removed. Miscellaneous Materials contained a clause that required an asphaltic coating to be shop installed on the concrete beam when embedded between the GRS abutment and wing walls (Figure 49); this clause was removed, as this requirement was specified later in the document. Miscellaneous Materials also stated that an aluminum fascia should be used as a drip edge, but this requirement was removed due to this being a regional issue that is not applicable when construction GRS-IBS structures in the southeastern United States. Construction (03) was organized as to follow a chronological flow in the revised specification.

**Figure 56. Bituminous coating on beam ends at Marshall County GRS-IBS.**

# CHAPTER 7: SUMMARY AND CONCLUSIONS

## 7.1 Summary of Project

Construction of the Turkey Creek GRS-IBS located in Marshall County, AL was completed using design guidance from the FHWA (Adams et al. 2011a). The total project cost was about $650,000; the bridge itself accounted for approximately $317,000 with roadway construction making up the rest of the cost (Hogan 2018). Post-construction monitoring persisted for two years; this included geospatial monitoring of the abutments to identify any settlement or lateral displacement, as well as earth-pressure and pore-pressure readings taken from within the abutment with a CRVW Series 3 datalogger. A draft Special Provision for constructing future GRS-IBS structures was developed for ALDOT.

## 7.2 Recommendations for Further Research

Several studies have indicated that GRS facing elements contribute to the overall strength of the abutment; large-scale performance tests conducted by Nicks et al. (2013) support this statement, although no direct relationship was discovered from these tests. Strength contributed from facing elements is neglected when designing GRS-IBS structures in order to keep the design conservative. If the relationship between facing element selection and strength of the GRS mass could be identified, this step could be included in the FHWA design process (Adams et al. 2011a). This could result in less geosynthetic reinforcement and backfill aggregate being necessary, reducing cost of GRS-IBS implementation.

2D and 3D finite element (FE) models have been used to model the behavior of GRS-IBS structures. LaDOTD researchers used PLAXIS 2016 to evaluate the performance of the Maree-

Michel GRS-IBS (Abu-Farsakh et al. 2020). The linear-elastic with Mohr-Coulomb (M-C) failure criterion model was used to simulate the interface between the geosynthetic and backfill materials, as well as the geosynthetic and facing blocks. A 2D FE parametric study was conducted to evaluate the effect of different variables and parameters on the performance of the GRS-IBS under service loading; lateral displacement of facing, settlement of RSF, maximum strain distribution along the reinforcement, lateral facing pressure, and location of possible failure locus. A similar study could be conducted with Alabama's next GRS-IBS in order to compare any discrepancies that could be present due to differences in regional geology, climate, or native material. The Turkey Creek GRS-IBS was built atop hard sandstone; Auburn researchers hypothesize that the reinforced fill/rock interface contributed to the satisfactory performance of the abutment. If another structure is constructed in similar geology, this interface should be studied and modelled in PLAXIS or a similar program.

### 7.3 Conclusions

The primary objectives of this study were to observe and monitor the performance of Alabama's first GRS-IBS and to draft a special provision for ALDOT

- Pore pressure presence within the abutment has been negligible, and earth pressure measurements have fallen within the range of expected values

- The abutments have experienced little to no settlement, and no discernable lateral displacement has been recorded

- GRS-IBS construction specifications were obtained from multiple state DOTs and relevant information was compiled into a draft Special Provision for the Alabama Department of Transportation.

The Turkey Creek site in Marshall County was selected because of its hard underlying bedrock and minimal scour potential. This was a conservative measure taken to increase the likelihood of optimal performance. Two years of post-construction monitoring have indicated that the abutments are performing as expected. The successful implementation of Alabama's first GRS-IBS structure suggest that this technology could be used to replace single-span bridges across the state in a cost-efficient manner. Auburn University researchers recommend GRS-IBS technology for further implementation within the state.

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# SECTION 545: GEOSYNTHETIC REINFORCED SOIL-INTEGRATED BRIDGE SYSTEM (GRS-IBS)

## 545.01 Description.

This Section shall cover the work of furnishing and installing all materials, labor, equipment, and supervision required for the construction of a Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS).

## 545.02 Submittals.

The submittal of the design and details of a retaining wall is required if the details of the retaining wall are not shown on the plans. The Contractor shall submit 8 copies of the complete details, material requirements and design calculations to the Engineer for review no later than 30 calendar days after the date of the Notice to Proceed.

The contractor shall also submit a Wall Installation Plan that includes:

- name and experience record of the superintendent in charge of the retaining wall installation;
- list of proposed equipment to be used;
- details of the proposed sequence of retaining wall construction;
- details of planned excavation and shoring methods, if shoring is required;
- details of earth reinforcement placement including methods proposed to prevent damage to the reinforcement during subsequent backfill placement.

The design calculations shall include an analysis of the internal and external stability of the wall and all structural connection details of the wall. All proposed details, material requirements and design calculations shall be stamped and signed by a Licensed Professional Engineer licensed in the state of Alabama and not employed by ALDOT. The design shall be in conformance with the requirements given in the current AASHTO Standard Specifications for Highway Bridges as amended by interim revisions.

The Engineer will review the retaining wall installation plan, wall design details, materials requirements, and design calculations for conformance with the plans and specifications. The Engineer will not approve the submittal but will review it to make sure that it is sufficiently complete to allow the construction of the wall.

The Engineer will return the submittal for corrections, distribute the submittal for construction inspection, or contact the Contractor to establish a mutually agreeable date and time for a meeting to discuss the submittal. The Contractor will be notified of changes in the submittal deemed necessary within seven days after the meeting. Retaining wall construction shall not begin until the submittal has been distributed for construction inspection and the Engineer informs the Contractor in writing that the proposed wall details, material requirements, design calculations and wall installation plan are complete. Distribution of the submittal for construction inspection shall not relieve the Contractor of the responsibility to satisfactorily complete the work as detailed on the plans and in the specifications.

Any proposed modification of the installation plan during construction shall be submitted to the Construction Engineer for review and distribution.

## 545.03 Materials.

### (a) Masonry Facing Blocks.

1. CMU shall have a minimum compressive strength of 3100 psi and maximum water absorption of 10 % after 24 hours, tested in accordance with ASTM C90-11b.

2. CMU shall comply with all other requirements of ASTM C1372, Specification for segmental wall units.

3. The height of each individual CMU shall be within 1/16 inches of the specified dimension. The length and width of each individual CMU shall be within 1/8 inches of the specified dimension. Hollow CMU units shall have a minimum face shell thickness of 1.25 inches and a minimum web thickness of 3/4 inches.

4. The CMU units shall be randomly sampled and tested in accordance with ASTM C140-12. Contractor QC testing shall be conducted at a qualified agency or AASHTO accredited laboratory as described by 545.04j.

5. The contractor shall provide units that accept No. 4 rebar to pin blocks together as shown in the contract documents.

6. Agency acceptance testing of the CMU blocks will be performed on a lot basis.

### (b) Backfill Material.

1. Aggregate for GRS backfill shall conform to ALDOT Section 529, shall be free-draining (maximum five percent passing the No. 200 sieve) and have a maximum particle size of two inches.

2. If the GRS-IBS will be in a submerged condition, the backfill gradation shall be open-graded.

3. Backfill material gradation should meet the requirements detailed in Table 1:

**Table 1. GRS abutment backfill gradation requirements (from FHWA)**

| Description | Value | |
|---|---|---|
| | Well-Graded Material | Open-Graded Material |
| Maximum grain size (inches) | 0.5-2 | 0.5-2 |
| Percent passing the No. 200 sieve (percent) (AASHTO T 11-05) | ≤ 12 | ≤ 5 |

## (c) Geosynthetic Reinforcement.

1. Geosynthetic reinforcement shall be biaxially woven polypropylene, high-density polyethylene, or polypropylene-polyester blend geotextile with a minimum ultimate wide-width tensile strength in the machine and cross-machine direction as designated in the Plans, but no less than 4800 lbs/ft.

2. Geosynthetic reinforcement should conform to ALDOT Section 243.

3. Ultimate wide-width tensile strength and strength at two percent shall be measured according to ASTM D4595. Geosynthetic reinforcement strength at 2 percent strain shall be greater than the unfactored required reinforcement strength.

4. Resin type for manufacturing the geotextile shall be identified according to ASTM D4101.

5. UV resistance shall be measured according to ASTM D4355.

6. Manufacturer Certified Test Reports verifying geotextile requirements described herein shall be submitted to the Engineer upon delivery per ALDOT Section 243.

## (d) Miscellaneous Materials.

1. A durable foam board, such as expanded polystyrene filler or equivalent, having a minimum compressive strength of 10 psi, and conforming to ASTM D 6817, may be used to provide a setback and create a bearing buffer between the superstructure and the wall face.

2. Concrete filler shall be Class A concrete per ALDOT Section 501 with a minimum compressive strength of 3000 psi. Furnishing, placing, finishing, and curing of concrete shall be performed in accordance with Section 501.

3. A 4-in. by 1.5-in. aluminum fascia or equivalent shall be used to serve as a drip edge under the superstructure within the clear space to shed potentially corrosive fluids off of the dry cast block and to prevent animals from burrowing into the abutment.

4. Geotextile Paving Fabric shall conform to ALDOT Section 243.

5. The reinforcing steel bar inserted inside the top 3 rows of hollow CMU blocks and corner CMU blocks (bearing bed) shall be No. 4, Grade 60 as per ALDOT Section 835.

# 545.04 Construction.

## (a) Delivery, Storage, and Handling.

1. The contractor shall check the materials upon delivery to ensure that the proper materials has been received. The contractor shall prevent contamination of the materials.

## (b) Excavation and Drainage.

1. The contractor shall ensure proper site grading and drainage so aggregate backfill is not contaminated with runoff soil.

2. All excavation shall comply with ALDOT Section 107, as well as ALDOT Section 210.

3. Excavation shall include provisions for drainage with a sloped cut to facilitate the movement of water downstream and away from the wall.

4. Any over-excavation that forms a pit shall be backfilled with suitable free

draining material and compacted.

## (c) Foundation.

The designer should choose between using a Reinforced Soil Foundation (RSF) or a concrete levelling pad to support the GRS abutment; procedures for both options are detailed within this section.

**Reinforced Soil Foundation**

1. The base of the RSF shall be excavated smooth and to uniform depth; all loose, soft, wet, frozen, organic, or unsuitable material shall be removed from the base and sides of the excavation.

2. The RSF base shall be graded level for the entire area of the base of such backfill, plus an additional 12 inches on all sides or to the limits shown in the plans.

3. The excavation shall be backfilled as soon as possible to avoid adverse weather delays. If this cannot be achieved, the excavation shall be graded to facilitate the removal of any water.

4. The RSF shall be constructed with backfill aggregate placed from the back to the face to roll folds or wrinkles to the free end of the reinforcement layer. Aggregate shall be compacted in lifts no greater than six inches thick.

5. Backfill aggregate shall be graded, leveled, and compacted before encapsulating the RSF.

6. The RSF shall be protected from erosion by encapsulating the RSF in geotextile reinforcement. The geotextile shall be sized to fully enclose the RSF on the face and both sides (wing walls). Corners shall be wrapped tight without exposed aggregate.

7. If the GRS abutment is adjacent to water, the first layer of geosynthetic reinforcement shall be placed on the upstream side of the RSF. Geosynthetic reinforcement shall be overlapped a minimum of three feet. All overlap sections shall be oriented in the area of the RSF so as to prevent running water from penetrating layers of reinforcement.

**Concrete Levelling Pad**

1. If the Contractor elects to use an optional concrete leveling pad, the concrete shall be Class A as specified in ALDOT Section 501. The leveling pad shall extend a minimum of 6 inches from both the toe and the heel of the facing block units.

## (d) Reinforced Backfill & Compaction.

1. The GRS backfill shall be placed onto the geosynthetic reinforcing elements in such a manner that no damage occurs. Placement of backfill materials shall be progressed so as to minimize the development of slack in the reinforcing element.

2. The GRS mass shall be constructed using compacted lifts of 8 in., which are equal to the facing block size.

3. Backfill aggregate shall be placed and compacted from the CMU facing to the back of the GRS excavation to roll folds or wrinkles to the free end of the reinforcement layer.

4. Backfill aggregate shall be compacted in accordance with Section 306.03(b).

5. Only lightweight, hand-operated compaction equipment shall be used within 3 feet of the wall face; this includes mechanical tampers, plates, or rollers.

6. Any damage to CMU blocks or misalignment of wall face as a result of compaction shall be corrected by the contractor prior to placing subsequent lifts.

7. The last lift of backfill aggregate shall be sloped away from the face of the GRS wall. Surface runoff from adjacent areas shall not be allowed to enter the GRS construction area.

## (e) Geosynthetic Reinforcement.

1. The geosynthetic reinforcement shall be pulled taut to remove any wrinkles and lay flat prior to placing and compacting the backfill material.

2. Geosynthetic reinforcement shall cover 100% of the embedment area (from wall face to cut-slope), unless shown otherwise in plans.

3. Primary geosynthetic reinforcement shall be anchored between wall facing layers, covering at least 85% of the facing element surface. Excess reinforcement material showing through the facing shall be removed in accordance with manufacturer’s recommendations.

4. Reinforcing elements shall be placed and secured in accordance with manufacturer’s recommendations. This entails continuous strips without joints, seams, or connections throughout the embedment length. Reinforcing elements should be laid to the line, grade, and orientation shown in the contract documents.

5. Adjacent sections of geosynthetic reinforcement do not need to be overlapped, except when exposed in a wrap-around face system. In this case, overlap or mechanically connect reinforcement rolls per manufacturer's requirements.

6. A minimum 6" backfill cover atop the geosynthetic reinforcement must be present for any equipment operation on an abutment to be permissible.

7. Rubber-tired (no tracked/skid-steer) vehicles may be operated on the abutment provided the operating speed is less than 5-mph, with no sudden braking or sharp turns.

## (f) GRS Wall Facing.

1. CMU block layers shall be erected conforming to lines, grades, and typical sections shown on contract documents and in accordance with the designated manufacturer's installation manual.

2. CMU installation shall begin at the lowest portion of the excavation, with each layer placed horizontally. The first course should be set level and to grade.

3. A thin layer of fine aggregate (not exceeding 0.5 inches in depth) may be used on top of the RSF to facilitate levelling of the first course of CMU blocks. If the levelling course exceeds 0.5 inches, mortar or grout shall be placed between the RSF and CMU course.

4. CMU blocks shall be installed tightly against adjoining CMU blocks, without any visible gaps. CMU facing shall be plumb within 0.25 inches over the height of the face if batter is not required in the plans.

5. Each CMU layer shall be completed and cleaned of any debris and fill materials before installing the next layer of geosynthetic reinforcement and CMU. A stretcher or running bond shall be maintained between courses to ensure that joints between blocks are offset with each row.

6. Level alignment of CMU course shall be checked at least every other layer. Any alignment deviation greater than 0.25 inch shall be corrected.

7. If scour countermeasure (rip-rap) is required, geotextile material shall be placed under the countermeasure and anchored between the first and second course of CMU.

8. CMU blocks displaced out of required alignment during construction shall be carefully moved back into position, using methods that will not damage blocks.

9. Contractor shall replace any CMU or geosynthetic reinforcement that is damaged during construction at no cost.

10. In the case of superelevation, the top course of CMU facing shall be saw-cut to match the elevation difference and clear space across the abutment.

11. Corner details shall be submitted when accommodating corners other than right angles.

12. Facing wall and wing wall courses shall be staggered to form tight interlocking stable corners.

13. The uppermost three layers of CMU shall be filled with Class Aconcrete, pinned with No. 4 steel bar embedded with a minimum of 2-inch concrete cover prior to placement of superstructure.

## (g) Beam Seat.

1. The beam seat shall be constructed directly above the bearing bed reinforcement zone to ensure the superstructure bears on the GRS abutment, not the wall facing units, and provides necessary clear space between the superstructure and wall face.

2. The block elevation beneath the bearing area should be established prior to pinning the concrete block facing units on the abutment wall face.

3. Precut 4 in. thick polystyrene foam board shall be placed on the top of the bearing bed reinforcement. A thin layer of backfill material may be placed beneath the foam board for grading purposes, as well as to ensure proper clear space. The foam board shall be butted against the back face of the concrete block facing unit.

4. 4 in. solid concrete blocks shall be set on top of the polystyrene foam board across the entire length of the bearing area. The back edge of the top concrete block facing unit shall hold the 4 in. concrete block in place during compaction.

5. The first 4 in. wrapped layer of compacted fill shall be used as the thickness to the top of the polystyrene board. The second 4 in. wrapped layer of compacted fill shall be placed to the top of the 4 in. solid block, creating clear space. The top of this layer controls the beam elevation.

6. The surface aggregate of the beam seat shall be graded slightly high (about 0.5 in.) to aid in seating the superstructure and to maximize contact with the bearing area.

7. When the GRS superstructure is built with adjacent precast concrete beams, a layer of geotextile paving fabric shall be installed a minimum distance of three feet over the ends of beams and continuously across beams.

8. An optional aluminum flashing drip edge may be installed prior to setting the bridge beams. Precut 4 in. thick polystyrene foam board shall be placed on top of the filled-in top course of the concrete block facing units, positioned directly in front of the 4 in. solid concrete blocks. The flashing shall be placed in between the bottom of the beams and the polystyrene foam board. The flashing shall be held in place by the pressure of the beams on the solid concrete blocks. The length of the flashing shall extend beyond the outside edge of the bridge beams and be trimmed to fit against the parapets.

## (h) Superstructure placement.

1. Crane shall be positioned as to not damage any aspect of the GRS abutment. Loads exceeding 4000 psf shall not be positioned closer to the GRS facing than the center of the beam seat.

2. Crane shall have outrigger pads sized within the capacity of the GRS mass. Greater loads could be supported with increasing distance from the abutment face if checked by the Engineer.

3. An additional layer of geosynthetic reinforcement shall be placed between the beam seat and the beams.

4. Beams shall be set square and levelled. Beams shall not be dragged over the surface of the beam seat.

5. Wing walls and parapets shall be constructed after the superstructure is set; CMU facing blocks in parapet wing wall should be trimmed or saw cut for custom fit against the beam edge to prevent loss of fill material. Gap should be filled with mortar mix if this gap cannot be filled with thin slices of CMU.

6. Any voids between beam seat and beams shall be filled with additional backfill aggregate, or re-grade the top layer of beam seat backfill aggregate and re-install the beams.

## (i) Integration Zone.

1. The aggregate base and geosynthetic reinforcement layers shall be installed along the back of the superstructure following placement of the superstructure, in maximum lift thicknesses of six inches.

2. Geosynthetic reinforcement shall be wrapped within the approaches at the beam ends and at the sides of the approaches.

3. The top two layers of geosynthetic reinforcement shall be extended a minimum of three feet across the limit of excavation.

4. A minimum of two inches of aggregate base material shall be placed over the top of the final wrap of geosynthetic reinforcement in order to protect against contact with hot mix asphalt.

5. The Contractor shall propose a safe method of guardrail post installation through the geosynthetic reinforcement that does not damage or misalign the CMU facing and provides proper confinement of the posts.

## 545.05 Method of Measurement

The unit of measurement for furnishing the GRS-IBS retaining wall system will be the vertical square footage of wall surface from the top of the leveling pad to the top of the wall. The leveling pad will be paid for in cubic yards as calculated by the required dimensions shown in the plans. The quantity to be paid shall include supply and installation of the GRS-IBS retaining wall system. Excavation of unsuitable materials and replacement with select fill, as directed and approved in writing by the Engineer, shall be paid for under separate pay items.

## 545.06 Basis of Payment.

### (a) Unit Price Coverage.

The accepted quantities of the GRS-IBS retaining wall system will be paid for per square foot of vertical wall face in place as measured from top of the leveling pad, to the top of wall including the cap block, grout and rebar used in the upper three courses of facing blocks, and all required geotextile and riprap materials used for backfill reinforcement. The quantities of the retaining wall system, to determine the area supplied, will be as shown in the plans. Payment for the leveling pad and shall include the preparation and the placement of the pad.

Excavation to install the GRS-IBS system will be paid per cubic yard of excavated material on an unclassified basis as shown in the plans.

### (b) Payment will be made under Item No.:

545-A GRS-IBS Retaining Wall System- per square foot

545-B   Unclassified GRS-IBS Excavation – per cubic yard
545-C   Crushed Stone Leveling Pad – per cubic yard
545-D   Concrete Leveling Pad – per cubic yard