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Final Research Report

IMPLEMENTATION OF GEOSYNTHETIC REINFORCED SOIL – INTEGRATED BRIDGE SYSTEM (GRS-IBS) TECHNOLOGY IN ALABAMA

Submitted to

The Alabama Department of Transportation

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16. Abstract

Geosynthetic Reinforced Soil-Integrated Bridge Systems (GRS-IBS) use closely spaced layers of geosynthetic reinforcement and compacted granular backfill to directly support a bridge deck and blend the abutment and roadway approach for a smooth transition. These systems are designed for areas where a single span is sufficient to bridge over a gap in the roadway. Alabama has recently completed its first GRS-IBS in Marshall County, which is located within the Sand Mountain region of the Appalachian Plateau. The robust mechanical properties of the native material provided a good location to construct the first GRS-IBS in Alabama. Two 12-ft tall by 33-ft wide GRS abutments were constructed to support the load of seven, 1.75-ft thick by 4-ft wide by 52-ft long, prestressed concrete box beams, pavement and traffic load. Construction of the GRS abutments was completed in three phases: excavation of the native sandstone, forming and placement of the concrete foundation, and segmental retaining wall (SRW) masonry unit and reinforced backfill placement. The bridge beams and integrated approach were placed after the GRS structure was completed. The integrated approach consisted of four layers of geosynthetic wrapped around No. 89 stone and a final layer of geosynthetic wrapped around dense grade base and covered with asphalt pavement. 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.

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DISCLAIMERS

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Scott George	ALDOT, Materials and Test Engineer, Montgomery
Kaye Chancellor Davis	ALDOT, Deputy State Materials and Tests Engineer, Montgomery
Robert Pirando	Marshall County Engineer

ABSTRACT

Geosynthetic Reinforced Soil-Integrated Bridge Systems (GRS-IBS) use closely spaced layers of geosynthetic reinforcement and compacted granular backfill to directly support a bridge deck and blend the abutment and roadway approach for a smooth transition. These systems are designed for areas where a single span is sufficient to bridge over a gap in the roadway. Alabama has recently completed its first GRS-IBS in Marshall County, which is located within the Sand Mountain region of the Appalachian Plateau. The robust mechanical properties of the native material provided a good location to construct the first GRS-IBS in Alabama. Two 12-ft tall by 33-ft wide GRS abutments were constructed to support the load of seven, 1.75-ft thick by 4-ft wide by 52-ft long, prestressed concrete box beams, pavement and traffic load. Construction of the GRS abutments was completed in three phases: excavation of the native sandstone, forming and placement of the concrete foundation, and segmental retaining wall (SRW) masonry unit and reinforced backfill placement. The bridge beams and integrated approach were placed after the GRS structure was completed. The integrated approach consisted of four layers of geosynthetic wrapped around No. 89 stone and a final layer of geosynthetic wrapped around dense grade base and covered with asphalt pavement. 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.

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CHAPTER 1: INTRODUCTION

1.1 Overview

Geosynthetic reinforced soil-integrated bridge system (GRS-IBS) technology is a system of closely spaced layers of geosynthetic reinforcement and compacted granular backfill that directly supports the bridge and blends the abutment and roadway for a seamless transition (Adams et al. 2012). GRS-IBS can be constructed using a small workforce with little impact on the surrounding landscape and the design can be easily modified to accommodate site conditions. GRS construction was first used by the U.S. Forest Service during the 1970s to construct wrapped face walls in the Siskiyou and Olympic National Forests in Oregon and Washington, respectively (Steward and Mohney 1982).

During the 1980s, the Colorado Department of Transportation (CDOT) modified GRS structures for use as retaining walls along roadways (Wu 1994). GRS-IBS was created by the Federal Highway Administration (FHWA) during the Bridge of the Future Initiative to serve as a lower cost design option for single span bridges in the United States (Adams et al. 2012). Reports have shown GRS-IBS to be 50 to 60-percent less expensive to construct than traditional bridge foundations and abutments (White et al. 2012).

The first GRS-IBS was constructed in Albertville, AL by Marshall County. Construction began in October 2017 and the new bridge was open in June of 2018. Figure 2 shows placement of the bridge beams atop completed abutments. The bridge has performed well since opening with minimal movements and no significant pore pressures in the backfill.

1.2 Study Purpose/Objectives

The objective of this project was to provide ALDOT with the information and resources to successfully deploy GRS-IBS technology to state and local engineers in Alabama. Specific objectives were to:

1) observe and document the construction of a GRS-IBS trial bridge
2) assess the performance of a GRS-IBS trial bridge
3) create an Alabama GRS-IBS design guide
4) develop a GRS-IBS technology transfer mechanism for Alabama

Figure 1 GRS-IBS Construction in Marshall County, AL (Hogan 2018)

1.3 Report Organization

This report presents information on design, construction and performance of GRS-IBS. Chapter 2 contains background information and literature review. Chapter 3 presents the design of the Marshall County GRS-IBS. Chapter 4 describes the construction of the Marshall County GRS-IBS. Chapter 5 presents the instrumentation and field monitoring. Chapter 6 contains the special provision and Chapter 7 contains the project summary, conclusions, and recommendations.

CHAPTER 2: BACKGROUND AND LITERATURE

2.1 Overview

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).

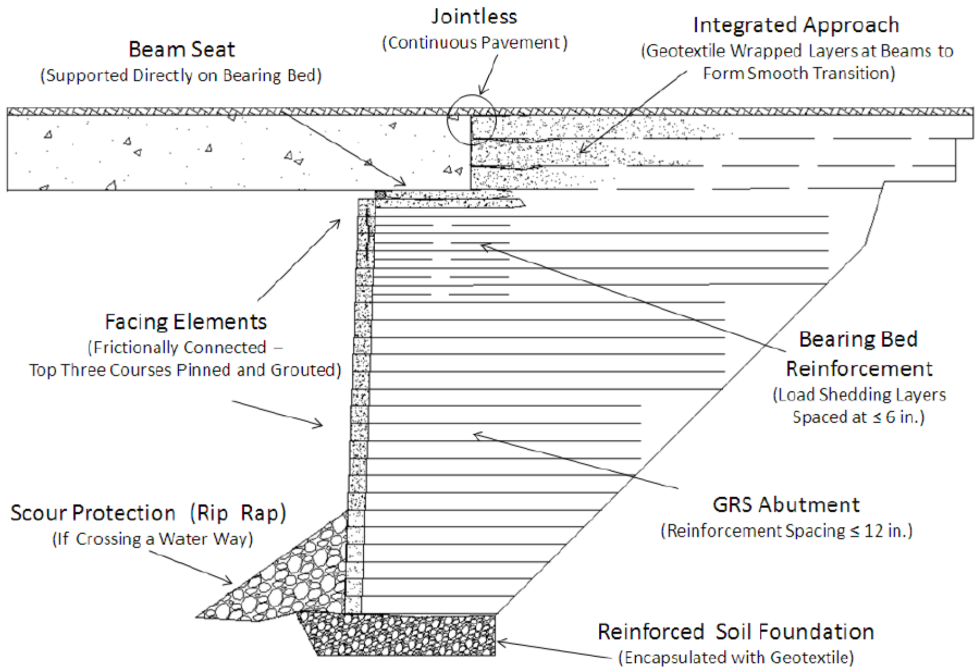

Figure 2 Typical GRS-IBS cross-section (Adams et al. 2011)

The foundation of a GRS abutment can be constructed of reinforced soil or concrete, depending on the site conditions and the native material being constructed upon. A reinforced soil foundation (RSF) is composed of granular fill material that is compacted and encapsulated with a geotextile fabric (Adams et al. 2012).

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. 2012). 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). GRS structures are constructed "from the ground up" and are integrated into the surrounding landscape, as shown in Figure 2.

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. 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.

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. 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 or dense sand. An RSF provides embedment and increases the bearing width and capacity of the GRS abutment and prevents water from infiltrating underneath the GRS mass from a river or stream crossing.

2.2.2 Facing Elements

The primary purposes of the facing element are to provide protection from weathering, serve as a façade, and provide a form for compaction of the backfill material (Adams et al 2011). Modular concrete blocks are the most common facing material; 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). Most newer bridges have utilized concrete masonry units (CMU) or segmental retaining wall (SRW) units. Common nominal dimensions of these units are 8 in. x 10 in. x 16 in., and 8 in. x 8 in. x 16 in. A minimum compressive strength of 4000-psi is required and an absorption limit of 5-percent is required in colder climates (Adams et al. 2011). Large-scale performance tests (PT) conducted by Nicks et al.

(2013) showed that facing materials do contribute to the overall performance of a GRS structure (Figure 3), although it is conservative not to include these benefits in design calculations.

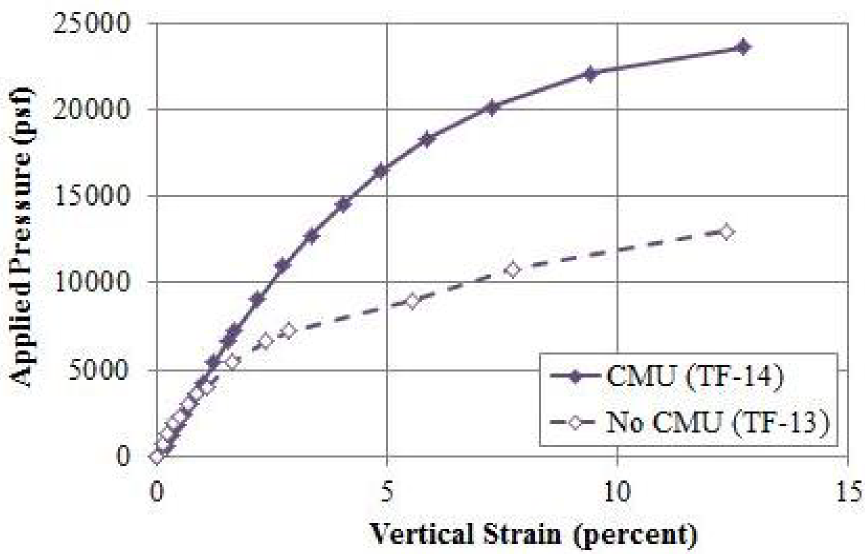

Figure 3 Performance test results of vertical strain with applied load with and without a facing material. The geosynthetic reinforcement was spaced at 11.25-in with a tensile strength of 3,600-lb/ft (Nicks et al. 2013)

2.2.3 Backfill Material

Gradations for backfill material should be either open-graded or well-graded gravel (Table 1), although lower quality granular or natural fill materials can be used if the quantity of fines is less than 12-percent and a performance test is conducted (Adams et al. 2011). Furthermore, the material should satisfy American Association of State and Highway Transportation Officials (AASHTO) specifications T-90 and T-104 which limit the plasticity index (PI) and measure the aggregate soundness, respectively. Open-graded backfills are generally used for this application since they are not as dense, have good drainage, and are relatively easy to compact (Nicks and Adams 2013); however, well-graded aggregates are recommended for use in the RSF and integrated approach so that a denser compaction may be achieved (Adams et al. 2011). Compaction of the backfill material should be 95-percent of the maximum dry density according to the AASHTO T-99 specification. Adams et al. (2011) recommended using eight-inch lifts compacted with vibratory rollers, and a material with fines should have a moisture content within $\pm 2\%$ of optimum. Open-graded stone can be compacted until no further movement can be visually noticed, but, in all cases, hand operated compaction equipment should be used within 1.5-ft of the face of the abutment and the top 5-ft should be compacted to 100-percent of the maximum density (Adams et al. 2011). Friction angle of open-graded aggregate is commonly estimated as 34° which is a rather conservative design value (Nicks 2013).

Table 1: Selected well and open-graded aggregate specifications (after Adams et al. 2011)

Well-Graded		Open-graded	
Sieve Size	**Percent Passing**	**Sieve Size**	**Percent Passing**
¾ inch	100	½ inch	100
1 inch	94 – 100	3/8 inch	90 – 100
3/8 inch	63 – 72	No. 4	20 – 55
No. 10	32 – 41	No. 8	5 – 30
No. 40	14 – 24	No. 16	0 – 10
No. 200	6 – 12	No. 50	0 – 5

2.2.4 Geosynthetics

Most GRS-IBS structures have been constructed with polypropylene (PP) biaxially woven geosynthetics (Figure 4); however, other types can be used. Geogrids are polymeric, planar geosynthetics that are formed by intersecting elements known as ribs (Shukla 2002). The method used to connect the ribs can be extrusion, bonding, or interlacing with openings, termed apertures, that range in size from 0.79 to 6-in and are classified as uniaxial or biaxial (Turnbull 2014) (Figure 5). The apertures provide space for soil to interact with the geogrid, therefore providing additional strength (Figure 6).

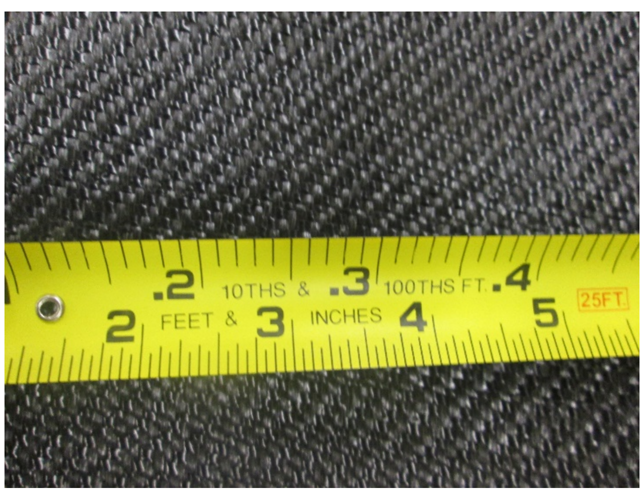

Figure 4 Biaxial woven polypropylene (PP) geosynthetic with rule for scale

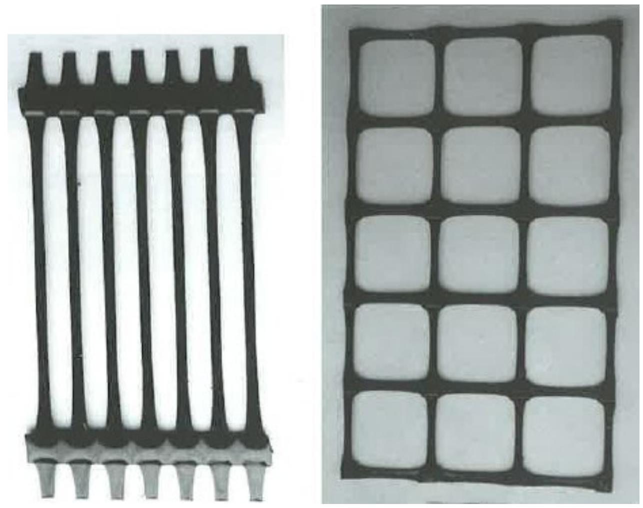

Figure 5 Uniaxial and biaxial geogrids on the left and right respectively (Shukla 2002)

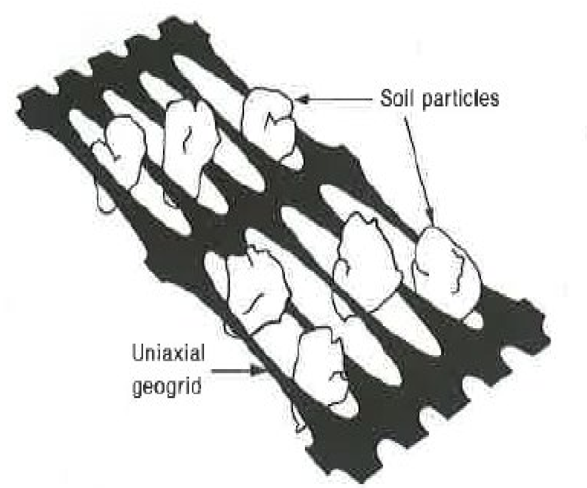

Figure 6 Uniaxial geosynthetic and soil particles interlocked within the aperture space (Shukla 2002)

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 (Adams et al 2012). A minimum ultimate tensile strength of 4800 lbs/ft is recommended for most applications. 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. Standardized testing is used to determine the strength and other material properties of geosynthetics used in GRS-IBS structures (Table 2).

Table 2 Standardized test methods used for geosynthetics in GRS-IBS

Test Procedure	Designation
Wide Width Tensile (geotextiles)	ASTM D4595-17
Wide Width Tensile (geogrids)	ASTM D6637-15
Specific Gravity (HDPE only)	ASTM D1505-18
Melt Flow Index (PP and HDPE)	ASTM D1238-13
Inherent Viscosity (PET only)	ASTM D4603-18
Single Rib Tensile (geogrids)	ASTM D6637-15

2.3 Design Procedure

The FHWA design method (Adams et al. 2011) involves GRS structures that have a vertical or slightly battered wall face and a height that does not exceed 30-ft, usually with a 2:1 base to height ratio or greater. The span of the bridge deck is generally limited to 140-ft. 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. (2011 and 2012) outlined a 9-step design procedure that is routinely used in GRS-IBS design (Figure 7).

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.

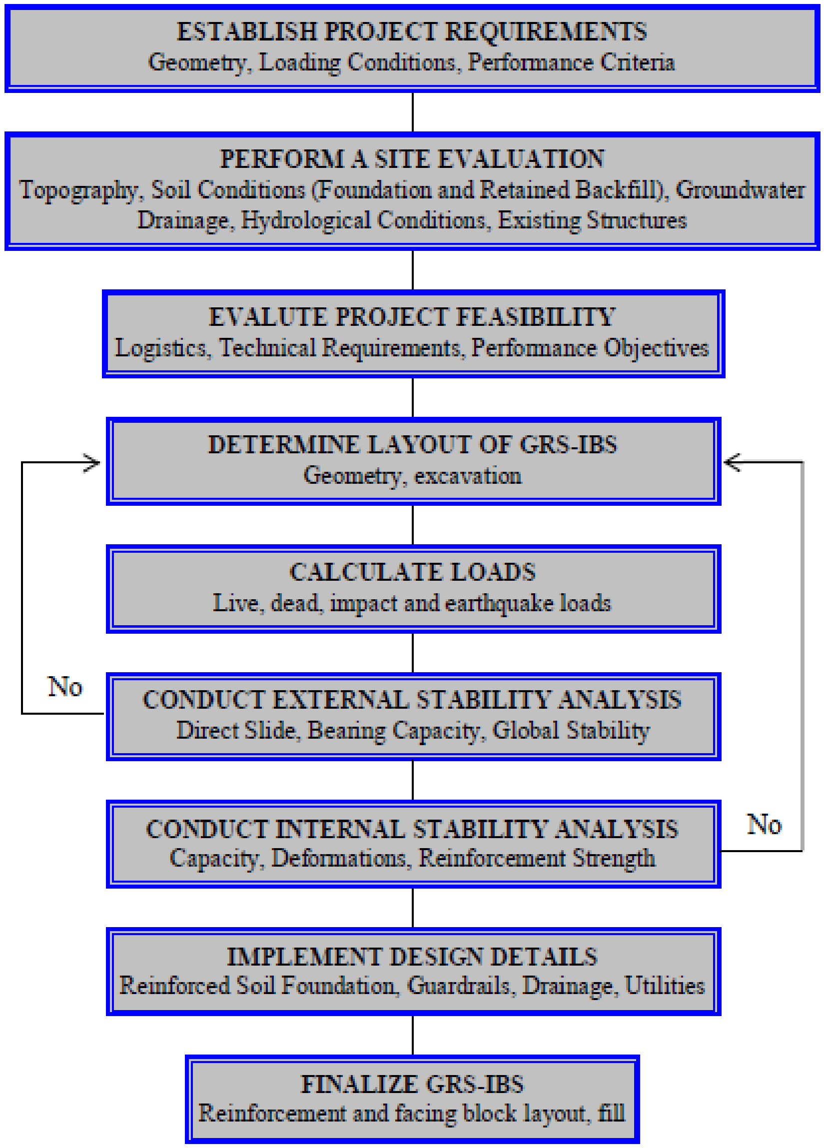

Figure 7 Nine-step protocol for GRS-IBS design (Adams et al. 2011).

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. Maximum allowable lateral, vertical, and differential movement across an abutment is a common performance measure (Adams et al. 2011).

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. 2011). 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.

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. 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 to specify GRS-IBS dimensions and layout.

Bearing width is the width of zone atop the abutment on which bridge girder sits. 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 clear space (d_e) 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. 2011).

Vertical spacing of geosynthetic reinforcement is to be the height of one SRW block. GRS-IBS structures are either of uniform or truncated reinforcement length, with truncated patterns being most common for GRS-IBS applications (Adams et al. 2011). 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 8). When utilizing a truncated reinforcement pattern, the allowable bearing pressure of the underlying soil should be reduced by 10% (Wu 1994). 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 $0.25B_{total}$ below the scour line and $0.25\ B_{total}$ out from the face of the abutment (Figure 8).

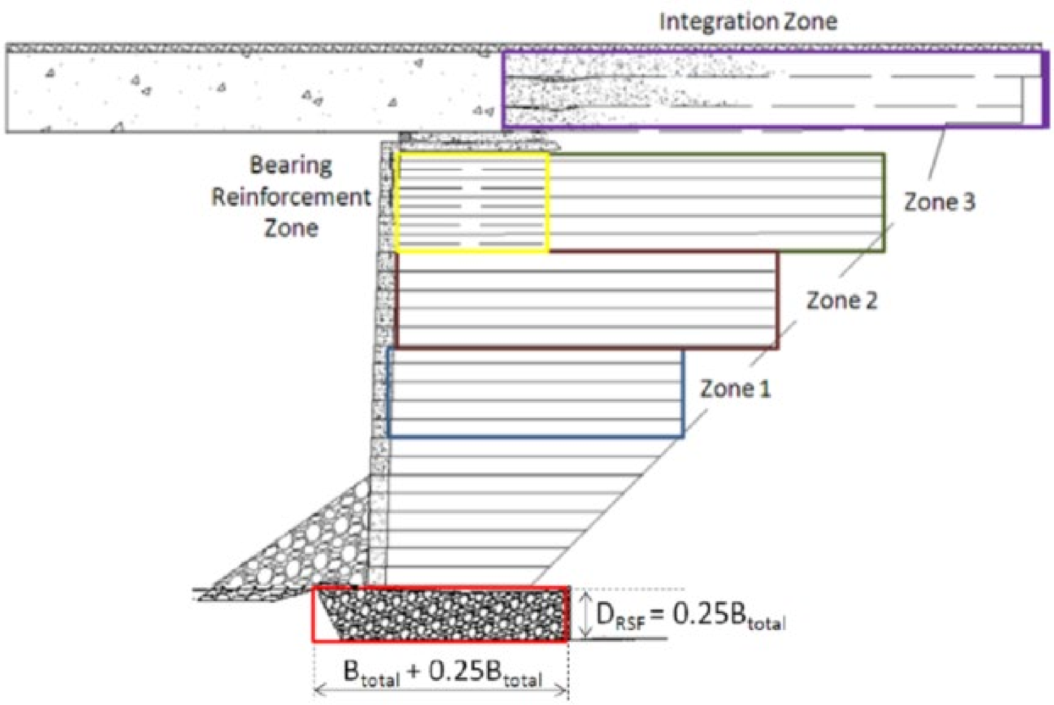

Figure 8 Illustration of base and RSF dimensions for a truncated reinforcement pattern (after Adams et al. 2011).

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. 2011). The following guidelines were outlined 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 8).

The integration zone is essential to alleviating differential settlement between the abutment and bridge deck. 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.

2.3.3 Loading Analysis

Estimate of loading imposed onto GRS abutment should be calculated for design. Figure 9 shows common loads that should be considered, and Table 3 defines the abbreviations.

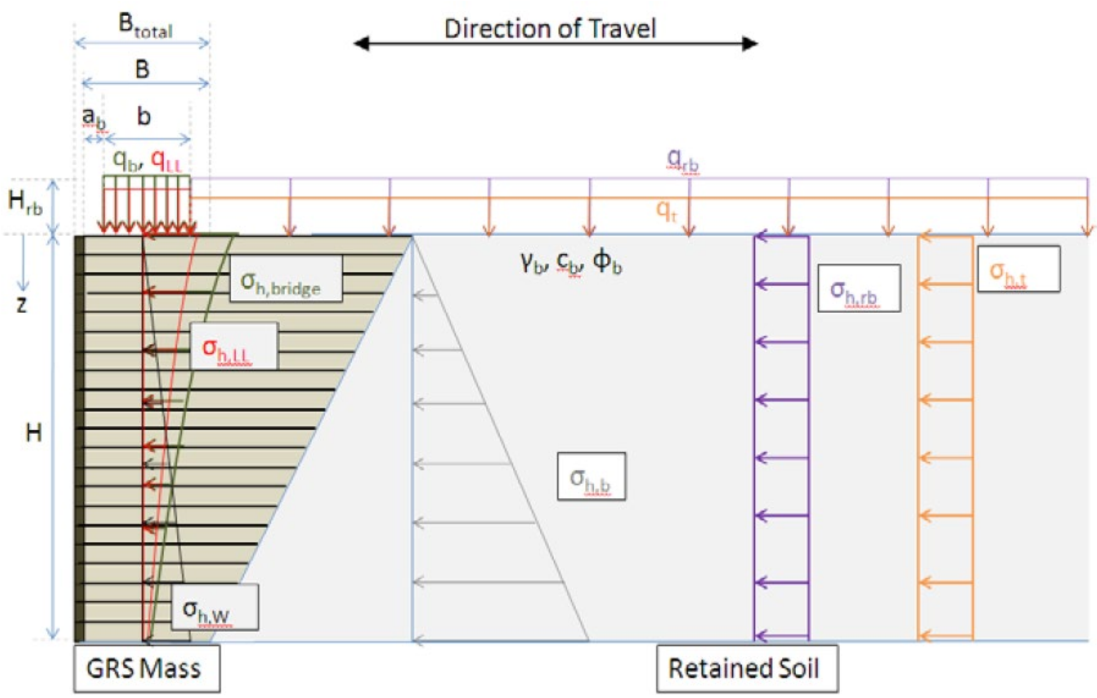

Figure 9 Typical vertical and lateral pressures on a GRS abutment (Adams et al. 2011).

Table 3 Typical pressures on a GRS abutment (from Adams et al. 2011)

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. 2011).

$$K_a = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{1}$$

where ϕ is the internal friction angle in degrees.

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. 2011).

$$\sigma_{h,w} = \gamma_T z K_{ar} \tag{2}$$

Where γ_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. 2011).

$$\sigma_{h,t} = q_t K_{ab} \tag{3}$$

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

$$\sigma_{h,rb} = q_{rb} K_{ab} \tag{4}$$

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

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

where q is the surcharge pressure, K_a is the Rankine active earth pressure coefficient, and α and β are the angles in radians found using equations 6 and 7 respectively.

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

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

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.

$$q_{LL} = \frac{\left(LL + IM\right)_{total} N_{lanes}}{b\left(B_b\right)} \tag{8}$$

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. 2011). 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. 2011).

$$Q_{LL} = \left(LL + IM\right)_{total} N_{lanes} \tag{9}$$

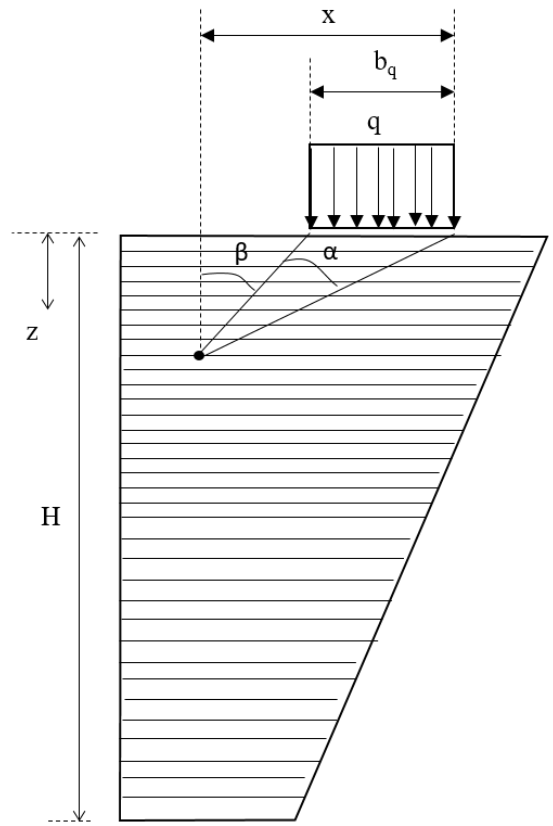

Figure 10 Boussinesq load distribution with depth for a strip load (Adams et al. 2011)

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. 2011).

2.3.4 External Stability Analyses

Once bridge dimensions and loadings have been determined, external stability analyses of the GRS-IBS structure should be performed; direct sliding, bearing capacity, abutment displacements and global stability are all factors to be considered. 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. 2011).

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 \tag{10}$$

$$F_{rb} = q_{rb} K_{ab} H \tag{11}$$

$$F_t = q_t K_{ab} H \tag{12}$$

The total driving force is calculated using equation 13 (Adams et al. 2011).

$$F_n = F_b + F_{rb} + F_t \tag{13}$$

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

$$R_n = W_t \mu \tag{14}$$

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 (Equation 16; Adams et al. 2011).

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

$$\mu = \tan\left(\frac{2}{3}\phi\right) \tag{16}$$

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. 2011).

$$W = \gamma_T HB \tag{17}$$

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. 2011).

$$FS_{slide} = \frac{R_n}{F_n} \geq 1.5 \tag{18}$$

The vertical pressure at the base of the RSF should be calculated using a Meyerhof distribution and 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.

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. 2011).

Horizontal and vertical displacements of the abutment are estimated assuming zero volume change (Adams et al. 2011). 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).

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.

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

where S_v is the reinforcement spacing and d_{max} is the maximum aggregate size. Furthermore, Adams et al. (2011) 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). 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.

CHAPTER 3: DESIGN OF THE MARSHALL COUNTY GRS-IBS

3.1 Site description

The Marshall County GRS-IBS is located on Cochran Road in Albertville, AL, which is lightly traveled and connects U.S. Highway 75 to County Road 409 (Figure 11). The bridge spans approximately 40-ft. across Turkey Creek. The drainage area for this portion of Turkey Creek is approximately 5 square miles and the 25-year flood elevation of Turkey Creek is estimated to be 8.77-ft above the base of the GRS foundation with a peak runoff rate (Q_{25}) of 1850-ft^3/s (ALDOT 2017).

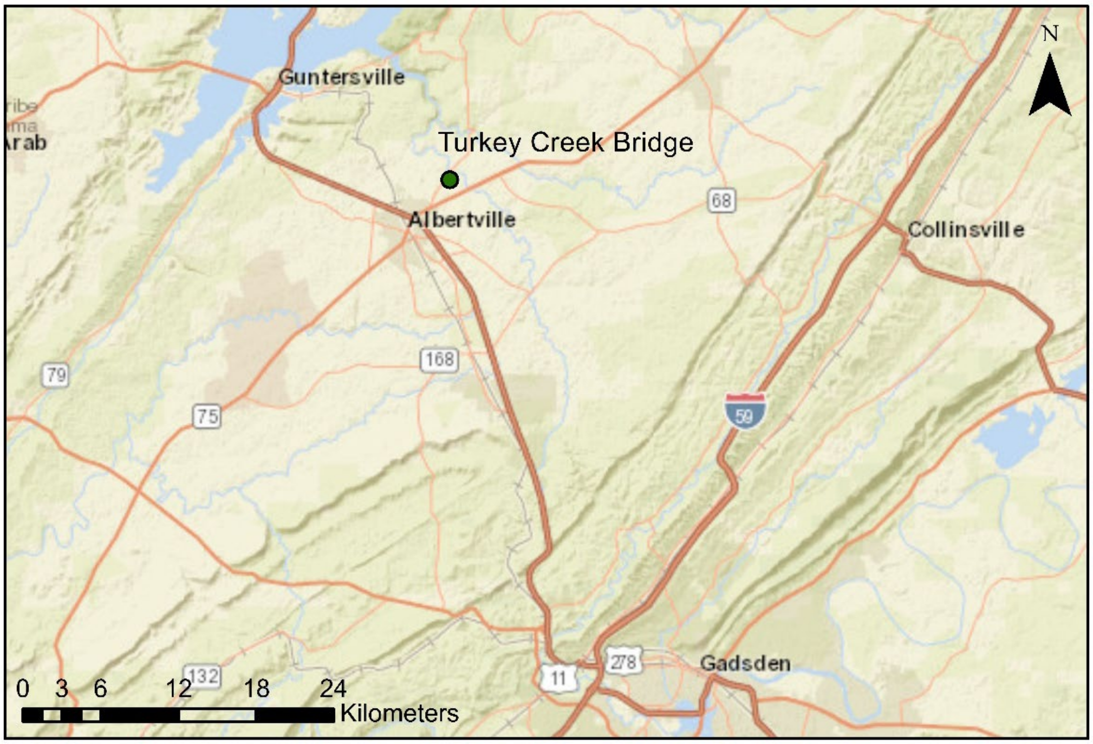

Figure 11 Location of GRS-IBS construction site (Hogan et al. 2019)

3.2 Site Geology

Marshall County is part of the Sand Mountain region of the Cumberland Plateau (Neilson 2007). Sand Mountain is a sub-maturely dissected synclinal plateau of moderate relief capped by the Pottsville Formation (Pomeroy and Thomas 1985). The Pottsville is separated into four fields based on coal production: the Warrior, Cahaba, Coosa, and Plateau fields. These fields were once connected but are separated today due to folding, faulting, and erosion of the highland areas (Adams et al. 1926). The project site is within the Plateau field; a cross-section of the Cahaba field reveals the general stratigraphy of the area. This consists of interbedded sandstones, siltstones, claystone, shale, and coal beds, with orthoquartzite conglomerate at the base (Peavy 2008). Rock cores taken at the site (ALDOT 2017) indicate the foundation material is primarily hard sandstone with a 10 to 15-degree dip angle, although thin coal seams were also found. An unconfined compressive strength of 11,300-psi was reported for the sandstone. Borings were completed to a depth of 17 and 14.5-ft on the east and west side of the creek, respectively, and indicated similar material across the site (Figure 12).

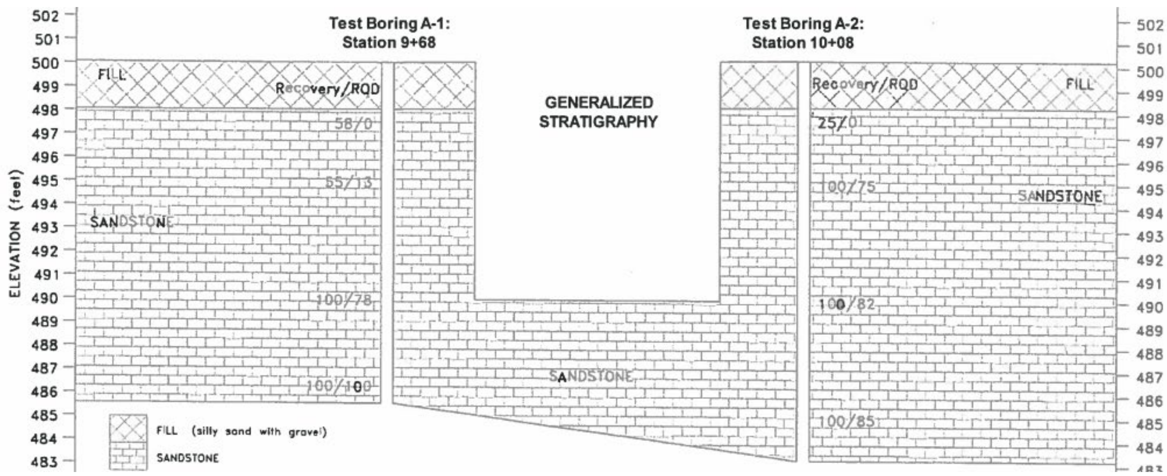

Figure 12 Generalized stratigraphy of GRS-IBS construction site (ALDOT 2017)

3.3 Design

The GRS-IBS at Turkey Creek was designed by Terracon for Marshall County using the guidance presented in the Federal Highway Administration GRS-IBS Implementation Guide (Adams et al. 2011) and the GRS-IBS Synthesis Report (Adams et al. 2012), with some modifications. Initially, three abutment reinforcement configurations were considered and were compared to determine the most efficient design. Constant length, truncated, and stepped reinforcement designs were analyzed to determine the factors of safety for bearing and sliding as well as the volume of rock excavation that would be required for each design (Table 4).

Table 4 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

The truncated method was chosen since the quantity of material that needed to be excavated was the lowest of the three. The overall dimensions of the abutments are approximately 12-feet in height and 33-feet width, while the wingwalls are approximately 6-feet wide at the base, transitioning to a maximum of 10-feet wide at the road surface (Figure 13). A 6-inch thick concrete foundation was constructed on the native sandstone using ready-mix concrete to serve as a leveling pad for the wall. The reinforced backfill was spaced every 8-in and extended to the cut-slope. Additional reinforcement was placed in the bearing bed and beam seat areas to accommodate the extra load of the bridge deck (Figure 13). At the surface, the integrated approach consists of 3-layers of reinforcement that extended at least 3-ft beyond the surface projection of the cut-slope.

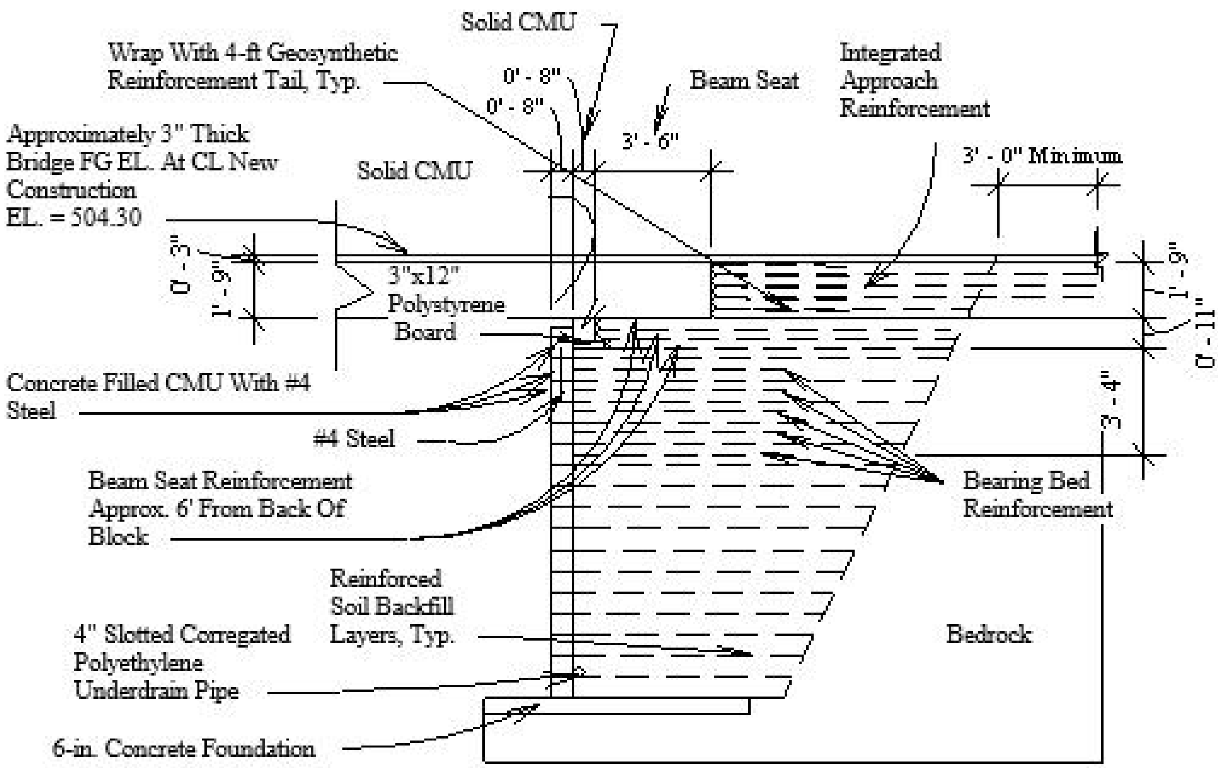

Figure 13 Design profile of GRS-IBS in Albertville, AL (adapted from ALDOT 2017).

3.3.1 Materials

The GRS abutments were constructed using U.S Fabrics Type 4800 woven geosynthetic and No. 89 limestone gravel (Table 5) with an estimated unit weight of 105pcf. This geosynthetic material was tested independently, by SGI Testing Services, using ASTM D 4595 wide-width tensile strength test (SGI Testing Services 2017.). The GRS abutments were faced using segmental retaining wall (SRW) cement masonry units. The bridge deck was constructed using seven 52-ft long by 4-ft wide by 1.75-ft thick prestressed concrete beams. This 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.

Table 5 No. 89 stone 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 6 Selected tensile strength measurements of the 4800 geosynthetic (Adapted from SGI Testing Services 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 7 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

Consolidated drained triaxial tests were conducted on the No. 89 stone at confining stresses of 7.5, 12.5, and 17-psi, the friction angle of the material was estimated to be 46-degrees (Figure 14). The secant shear modulus was estimated to be 10,129-psi using 0.6-percent strain as the limit of the linear range on the stress-strain plots (Figure 15).

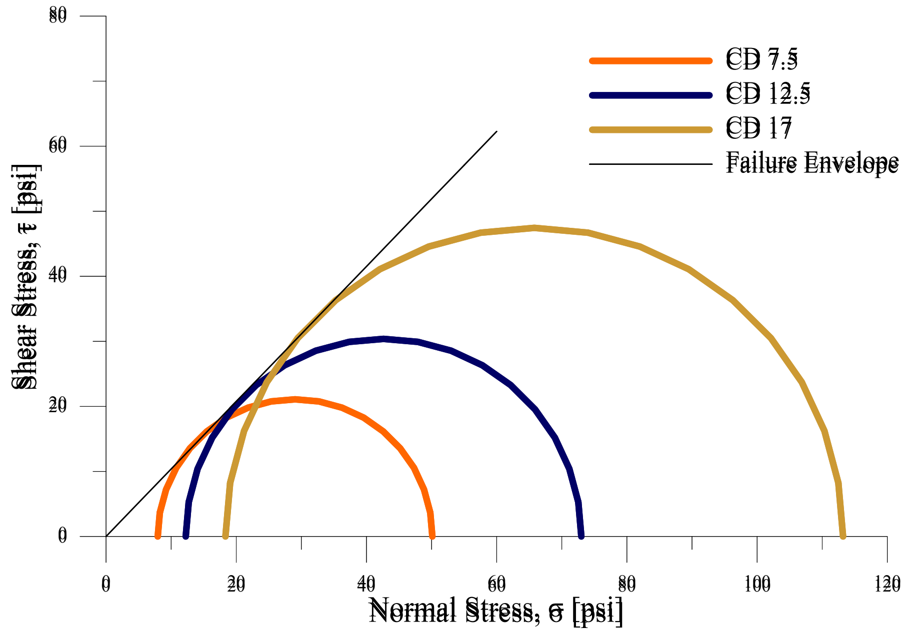

Figure 14 Mohr-Coulomb failure criteria for No. 89 backfill.

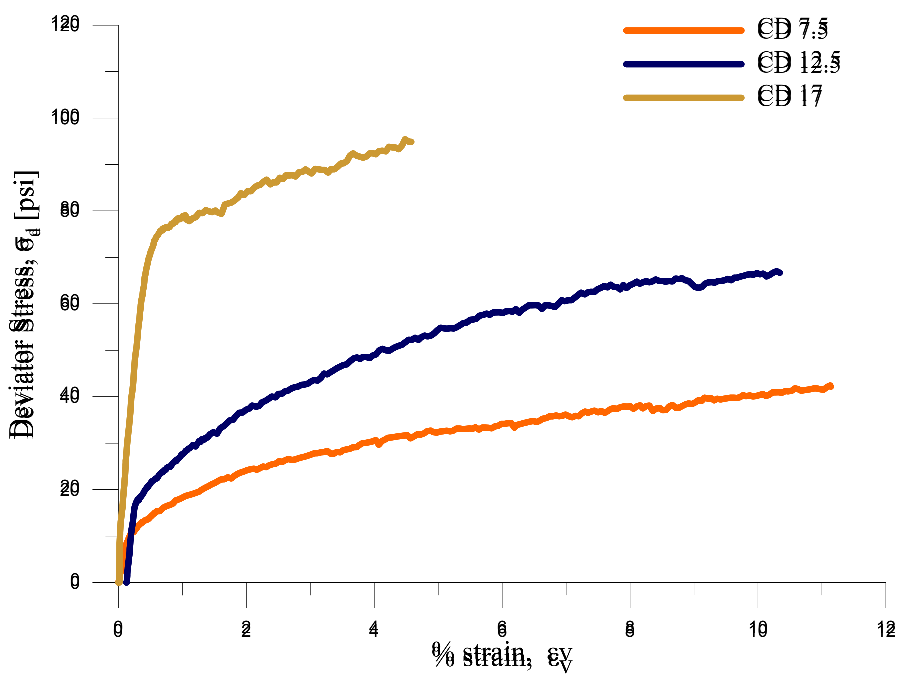

Figure 15: Stress-strain behavior for No. 89 backfill material.

CHAPTER 4:CONSTRUCTION OF THE MARSHALL COUNTY GRS-IBS

4.1 Demolition and Excavation

Construction of the Marshall County GRS-IBS bridge began on October 2nd, 2017 with the removal of the existing bridge and excavation of the native material (Figure 16a). A hydraulic excavator was used to remove the existing bridge and abutments and the surficial soil and rock. As the foundation material was primarily hard sandstone, blasting was needed to excavate the abutment down to the required elevation. The excavator was used to remove the blasted material (Figure 16b). Berms constructed from the native material were used to limit the inflow of water from the creek into the excavation, but this was only mildly effective and pumps were used to dewater the excavation prior to placement of the concrete pad.

Figure 16 (a) Removal of the existing bridge at Turkey Creek; (b) removal of blasted sandstone, on the western side of Turkey Creek.

4.2 Concrete Foundation

A concrete leveling pad was placed in each abutment after completion of the excavation. The leveling pad was 6 inches thick, 8 feet wide and 33 feet long. The concrete forms were set at the correct elevation, 490.33-ft, using a traditional tripod mounted level and grade-rod (Figure 17a). Ready-mix concrete was brought to the site and placed using a concrete bucket attached to a hydraulic excavator (Figure 17b). Concrete was initially placed around the outside of the form boards to keep concrete from leaking out of the formwork. Once the concrete placed around the perimeter hardened, concrete was placed within the formwork and finished using traditional hand tools. Surface grinding was performed on the finished pad as needed to obtain a level surface for block placement.

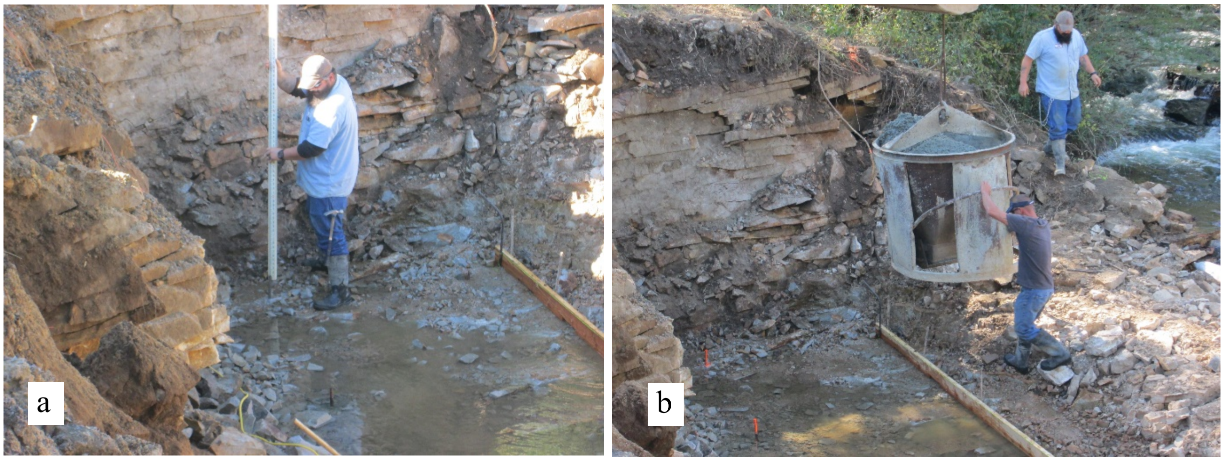

Figure 17 Leveling form boards in the excavation for placement of the concrete pad; (b) Concrete being placed using a concrete bucket suspended from a hydraulic excavation

4.3 GRS Abutment

The initial row of masonry blocks for the GRS abutment was placed on the concrete foundation by hand using a string-line as a guide and the centers of the blocks were filled with concrete. The area behind the blocks was filled with No. 89 gravel, which was leveled and lightly compacted to be even with the top of the first row of blocks (Figure 18a). The first layer of geosynthetic material was placed at this elevation (Figure 18b) then the second row of masonry blocks were set into place. Joints between the blocks were offset with the row of blocks below. A 4-in corrugated drain pipe was placed behind the second course of blocks to allow for drainage of the backfill. The process of backfilling the masonry bocks with No. 89 stone 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 three rows of masonry blocks were reinforced and grouted using No. 4 rebar bars placed on 8-in centers and ready-mix concrete, respectively.

The area of the abutment directly below the bridge beam seat received extra reinforcement to carry the extra load of the bridge deck. Beginning at elevation 498.33-ft and ending at the top of the abutment, the spacing of the geosynthetic for the bearing bed is 4-in and extends 6.5-ft from the back of the masonry blocks. An 11-in beam seat was constructed immediately below the beam elevation using closely spaced geosynthetic and No. 89 stone, that extends 6-ft from the back of SRWs. The final GRS wall is shown in Figure 19.

Figure 18 Placement of initial layer of No. 89 stone to top of first row of masonry blocks and (b) placement of geosynthetic and second row of masonry blocks.

Figure 19 The finished GRS abutment prior to placement of the bridge beams.

4.4 Beam Placement

Precast bridge beams were placed on December 8th, 2017, approximately fifty-four days after placement of the final concrete bearing pad. The bridge beams are directly supported by a solid concrete masonry unit (CMU) placed on a 3 x 12-in polystyrene board at the top of the reinforcing layer of the abutments, therefore leaving a 3-in space between the abutment and bottom of the beams (Figure 20). The beams were placed using a crane and then post-tensioned using three 1-in diameter steel tie-rods.

The integrated approach, placed after the beams were set, started just behind the beams and extended a minimum of 3-ft from the end of the beam. The approach consisted of four, closely spaced, layers of geosynthetic folded around No. 89 stone and a final layer of geosynthetic folded around dense grade base stone ending level with the top of the bridge beams. The completed approach is shown in Figure 21.

Figure 20 Bridge beam setting on top of CMU with 3-in gap between abutment and beam and integrated approach reinforcement layer. The circular opening for post-tensioning can be seen at the end of the beam.

Figure 21 Placed bridge beams and the integrated approach.

4.5 Final Grading and Paving

Rain and utility conflicts delayed final grading and paving for approximately two months after completion of the integrated approach. These operations were completed on May 22nd, 2018. Once paving was complete, guardrails were installed and the bridge was opened for traffic on June 3, 2018. The finished road and bridge is shown in Figure 22. The entire construction process took approximately nine months from demolition of the existing bridge to opening of the bridge.

Figure 22 The completed GRS-IBS over Turkey Creek in Marshall County, AL.

4.6 Construction Issues

Construction of the abutments went as expected for the most part. However, minor problems with the initial placement of the foundation, and poor dimensional tolerances of the SRW units caused issues. The engineer on record (EOR) prevented a mishap 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 would have reduced the bearing area, thus increasing the pressure on each abutment. Fortunately, the EOR made sure the issue was corrected before the project was allowed to move forward. Furthermore, as the wall was constructed, the contractor had to trim the corner

SRW units to match the row units. While this was an easy solution, it is imperative to ensure that each layer is level, otherwise gaps and loss of the frictional connection would occur. The gaps could provide access for backfill material to migrate out, therefore reducing the structural integrity.

4.7 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. 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

The Turkey Creek GRS-IBS was monitored and assessed for 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

5.1.1 Time-lapse Cameras

Two Wingscapes time-lapse cameras with a resolution of 8.0-megapixals were used for the project. The cameras were set to 15 and 30-min. time intervals and placed to obtain the best view possible before, during, and after construction. Sixteen-GB data cards were used to store images.

5.1.2 Piezometers

Two 4500 Geokon standard vibrating wire piezometers were used for the project (Figure 23 a). These sensors have a thin wire located within a stainless-steel housing that transmits a frequency based on the tension of the wire (Figure 23 b). The wire is connected to a diaphragm at one end that deflects in or out based on the applied pressure. The typical accuracy and resolution of a vibrating wire sensors is equal to 0.01 and 0.025-percent of the full scale output, respectively (Geokon 2021). For these sensors to produce accurate results they must be fully saturated; however, they can operate in partially saturated clays, although if there is difference in the pore water and air pressures, the sensor will measure the air pressure.

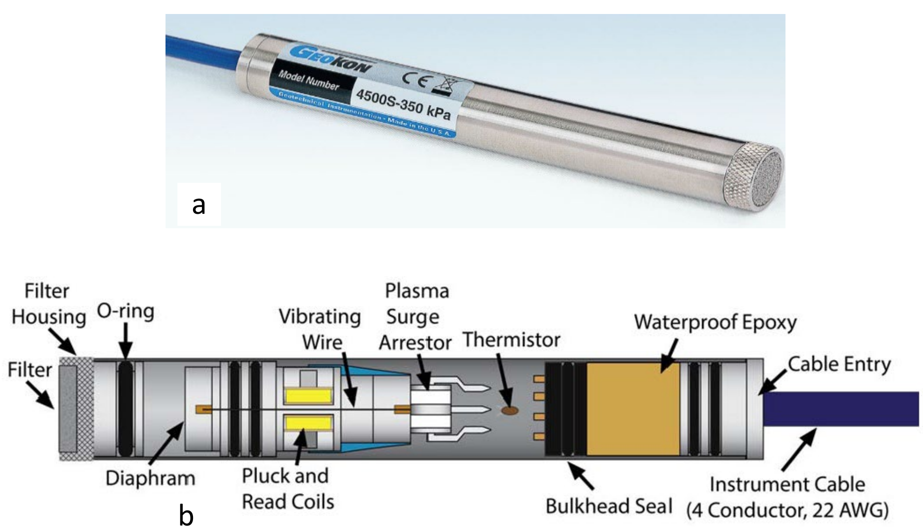

Figure 23 (a) Model 4500 standard vibrating wire piezometer (b) Internal components of vibrating wire sensor (Geokon 2021)

The sensors were saturated, then placed in sand-filled sleeves to protect them from being damaged by backfill material. The cables were routed along the length of the abutment and through a small opening in the SRW units.

5.1.3 Earth Pressure Cells (EPC)

Model 4810 vibrating wire earth pressure sensors were used for the project (Figure 24 a). This instrument measures total stress, therefore, pore pressure sensors are needed so the effective stress can be estimated. Each EPC was installed approximately 3-ft from the face of the SRW units and at elevations of 491.00 and 490.33-ft, for the east and west abutments respectively. This corresponds to 8-in above the concrete foundation of the east abutment and directly on the concrete foundation of west abutment. During installation, finely graded sand was used to cover the sensors (Figure 24 b) to reduce potential for damage from the fill and to reduce the effects of arching.

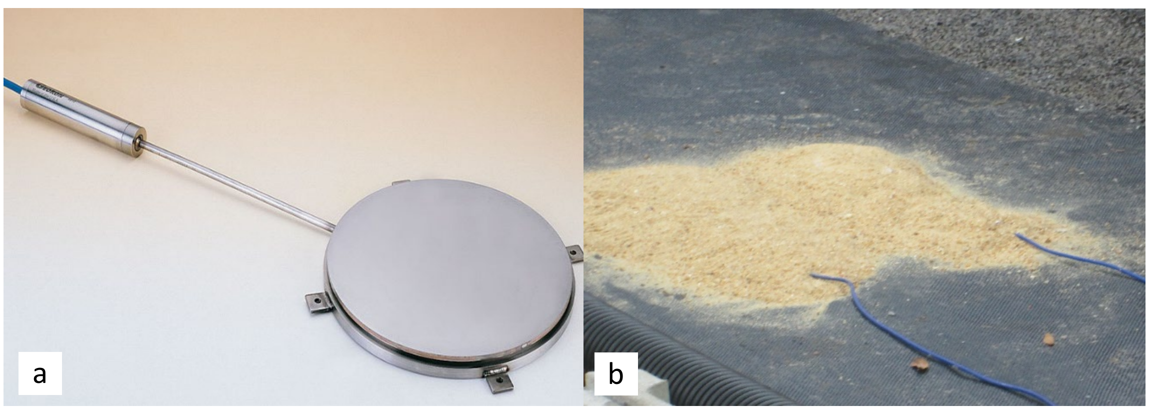

Figure 24 (a) Model 4810 contact pressure cell (Geokon 2020) (b) Installed earth and pore pressure sensors covered with sand to mitigate arching

5.1.4 Data Loggers (DAQ)

Two Campbell Scientific CRVW3 data loggers were used to provide excitation voltages, measure the output signal and log sensor data at regular intervals. These DAQs are specifically designed to be compatible with vibrating wire sensors and have 16-MB of storage. The units were pre-programmed to perform data reduction by adding the sensor conversion constants prior to installation. The data loggers were mounted on the southeast and northwest sides of the east and west abutment using concrete screws and construction adhesive. Once secured, the instrumentation cables were encased in conduit and mounted to the SRW units and routed into the data loggers. Readings were collected continuously since February 2, 2018 and the data were downloaded during each site visit.

5.1.5 Survey Instrument and Targets

A Topcon GTS-235W was used to measure geospatial data for the project. Bernsten RS60 survey targets were attached to corners of the bridge abutments. The size of the reflective cross section was 1.57 x 1.57 in. and the approximate recommended minimum and maximum range is 33 and 328 ft (Bertsten 2017). Four survey targets were placed at each corner of the abutments using construction adhesive (Figure 25), 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.

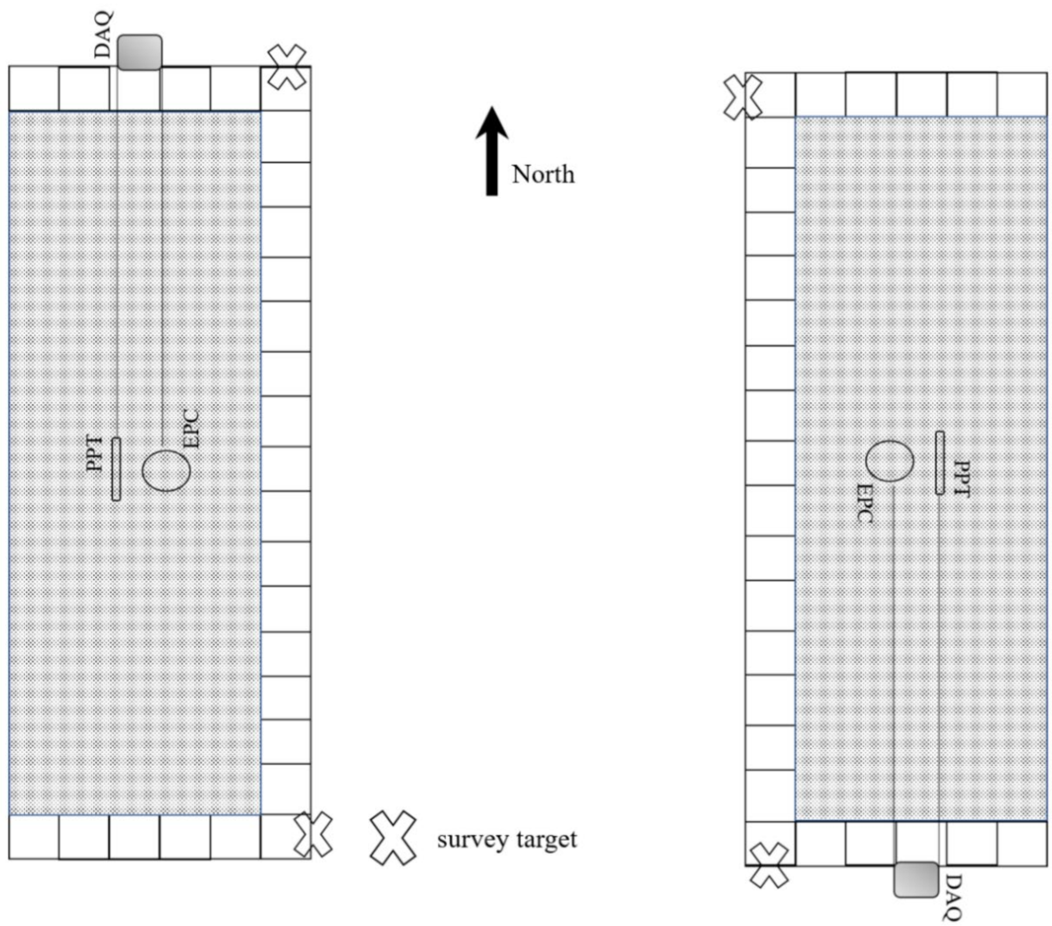

Figure 25 Plan view of the layout of the sensors and survey targets used on the GRS-IBS structure (Hogan 2018)

5.2 Results

5.2.1 Earth Pressure

Readings from the earth pressure sensors in the east and west abutments are shown in Figure 26. Manual readings were taken immediately after abutment construction to establish a zero reading which was roughly 600 psf. This was approximately half of the vertical stress estimated based on one-dimensional stress conditions. 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 were 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 were expected to be approximately 1800 psf. The most recent readings showed earth pressures to be about 1700 psf. However, readings taken between September of 2018 and March of 2019 were in the range of 1300-1550 psf. 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.

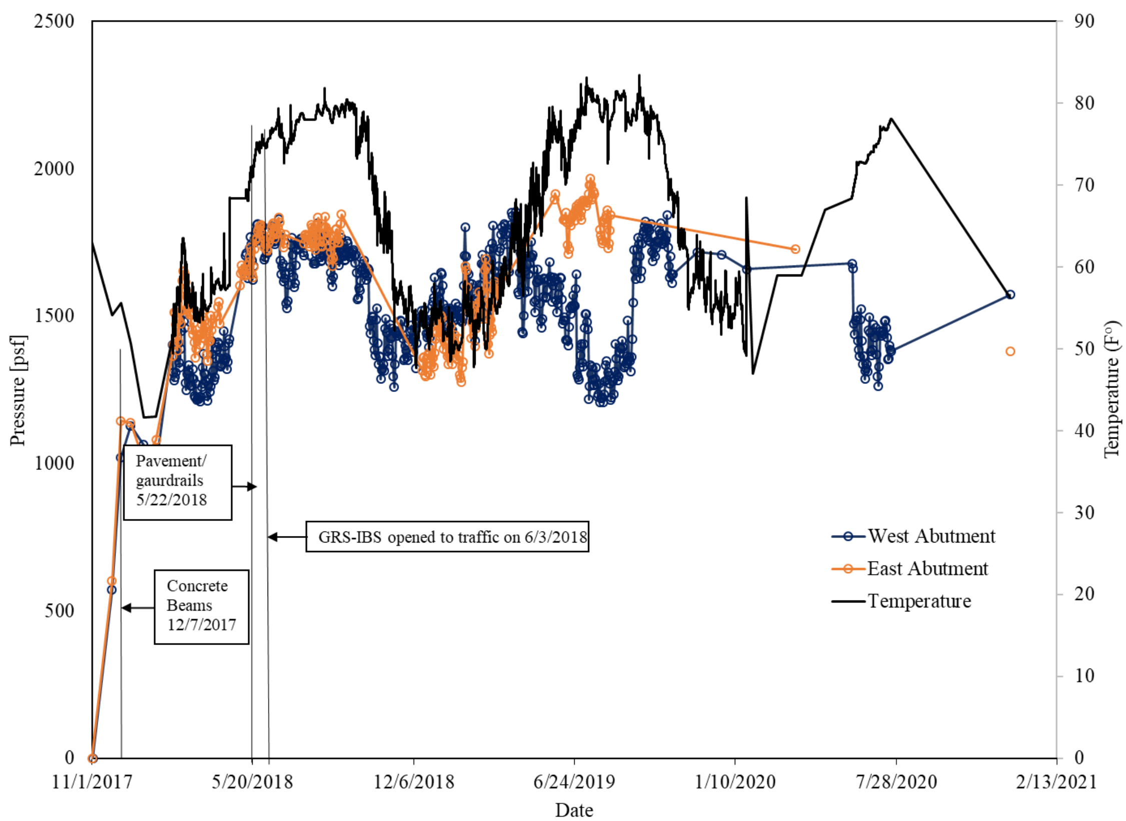

Figure 26 Earth pressure sensor readings for the east and west abutments

5.2.2 Pore-Water Pressure

Pore-pressure readings were insignificant, as expected (Figure 27). The data showed minor oscillations, which are likely due to changes in air pressure and temperature. The pore-pressure sensors have not shown evidence of saturation, indicating the abutments were 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 were still within the margin of error of the instrument.

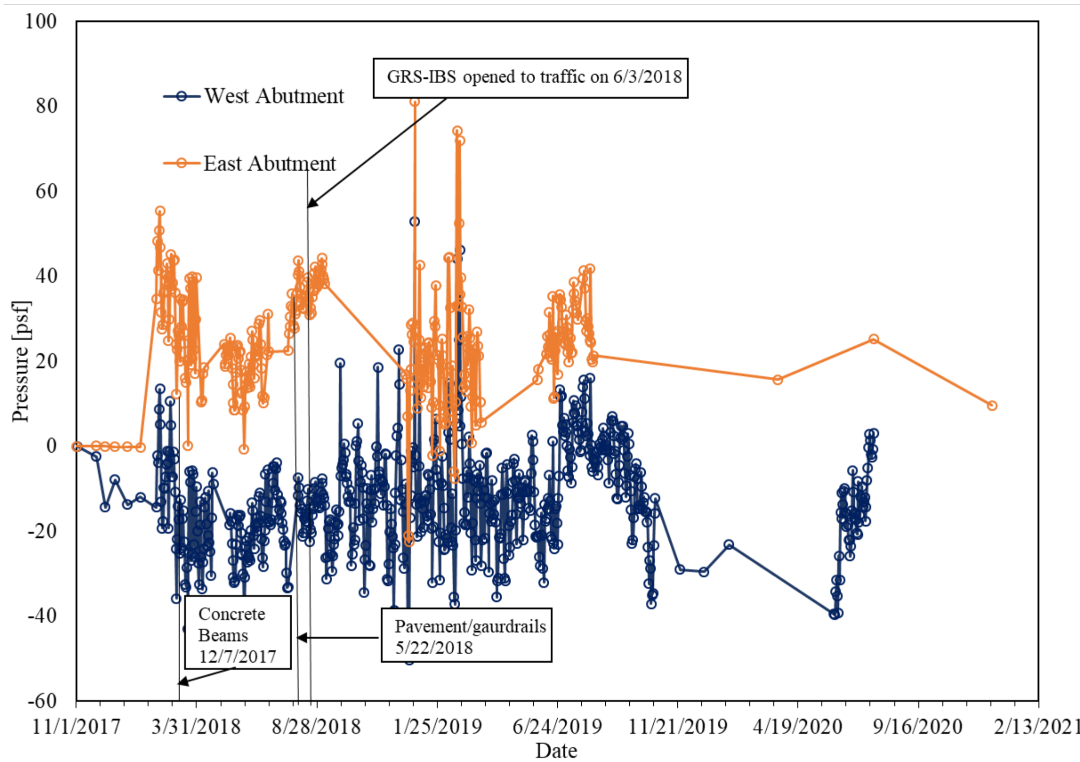

Figure 27 Pore pressure sensor readings for the east and west abutments.

5.2.3 Settlement

Geospatial surveys showed little to no settlement (Figure 28). The northern corner of the eastern abutment appears to have settled slightly after construction, and the remaining readings did not reflect any settlement of the GRS mass. The abutments were constructed atop hard sandstone, which is not expected to settle significantly over time. Overall, settlement readings have remained stable and virtually unchanged. Oscillations in settlement readings were due to instrument and measurement uncertainties. Uncertainty bands were calculated based on measurements taken from the benchmark height at each control point (Hogan 2018). The scatter in the settlement data was generally within the uncertainty bands and less than the settlement that would be expected based on Adams et al. (2012). The uncertainty in these measurements was likely due to errors in measuring the instrument height which is typically done using a metal tape.

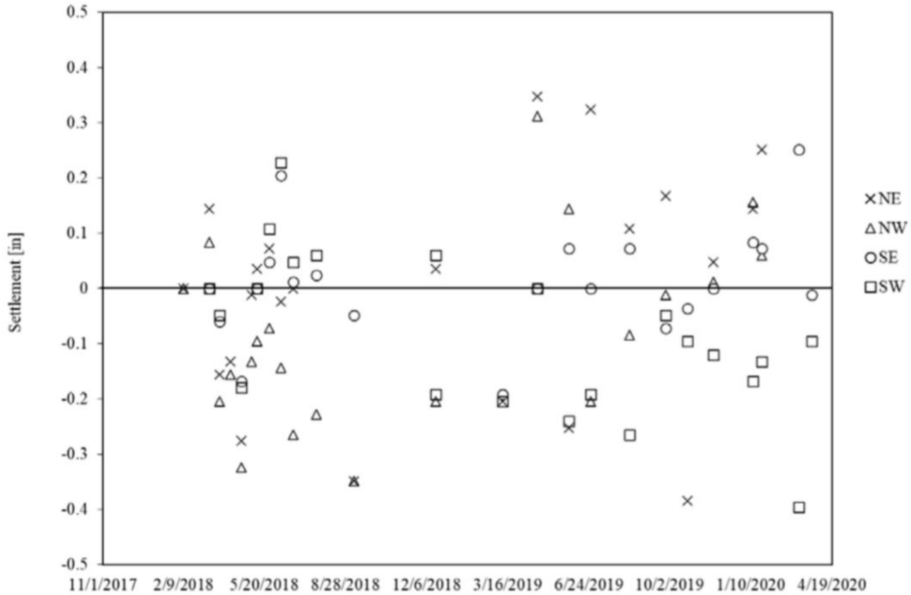

Figure 28 Settlement measurements of all survey locations.

5.2.4 Lateral Displacement

As observed during geospatial monitoring, the surveys have not measured any discernable lateral movement of the abutments. 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 fell outside of this range and were most likely caused by the thermal expansion and contraction of the SRW units. A composite plot of all lateral displacement measurements is shown in Figure 29.

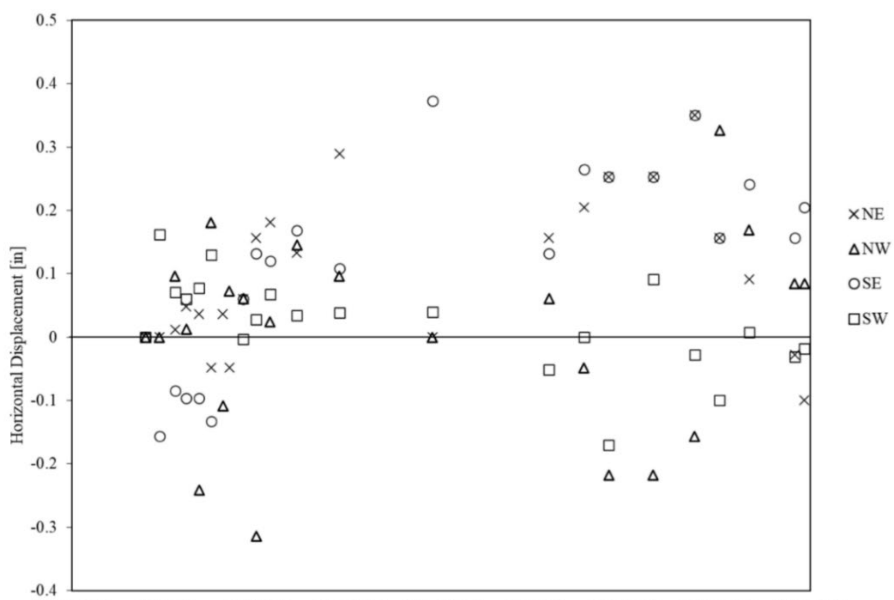

Figure 29 Lateral displacement measurements of all survey locations.

CHAPTER 6: SPECIAL PROVISION

Information pertaining to GRS-IBS specifications, case studies, and specific experiences was gathered from multiple sources including the FHWA and state DOTs from Florida, Kentucky, Louisiana, North Carolina, Virginia, New York and West Virginia; relevant information was consolidated into a draft Special Provision. The most recent draft of this Special Provision is included in Appendix A. The development process is described below.

6.1 State-of-Practice Review

Several states shared specific advice related to the 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 allow for settlement; 2 to 4 in. of clear space is recommended between the beam and the wall. Also, when a fascia (drip-edge) is not required, then the foam board on top of the half-height blocks at the top of the GRS wall should be eliminated. 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 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 2014) wrote a GRS-IBS special provision prior to the construction of the Maree Michel bridge. Their experience showed 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). Related personnel, such as district construction inspectors and engineers, should be allowed to review the plans and specifications prior to publication. The LaDOTD recommended making the specification relevant on a local level; climate concerns and material availability should be accounted for when writing a specification. 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 state DOTs (Florida, Kentucky, Louisiana, North Carolina, Virginia, and West Virginia) 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, AASHTO and ALDOT's general construction specification manual. Construction (03) outlines practices for building a GRS abutment and placement of superstructure. 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.

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. These individuals recommended several revisions. 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; 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.

CHAPTER 7: SUMMARY, CONCLUSIONS, AND RECOMMENDATION

7.1 Summary of Project

The first GRS-IBS in Alabama, shown in Figure 23, was constructed in Albertville to replace an existing stream crossing bridge on Cochran Road over the span of approximately 9 months between October 2017 and June 2018. 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. 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. A draft Special Provision for constructing future GRS-IBS structures was developed for ALDOT.

Figure 30 Turkey Creek GRS-IBS December 2020.

All phases of construction were visually documented with time-lapse cameras. The cameras were beneficial in keeping records of the events that took place during construction and insured images were captured when the Auburn team as not onsite. This data was augmented with other handheld camera photos taken during site visits.

Settlements and horizontal displacement were measured using conventional surveying equipment (total station and reflective targets). Surveys showed little to no settlement of the GRS mass. The East control point was destroyed in May1 2018 and had to be rebuilt. 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. This could have contributed to uncertainties in measurements before it was noted. Other sources of error include target/total station misalignment, instrument height measurement, precision of the total station, and possible movement of the wooden power pole on which the BM was fixed.

Earth pressure cells, and piezometers were installed in the abutments to estimate the stresses that developed due the weight of the backfill, and loads applied at the surface. The piezometers showed that pore pressures remained near zero. The earth pressure cells provided a good indication of the pressures that developed within the abutments and a good estimation of the vertical stress shedding due to friction.

7.2 Conclusions

The 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. Conclusions drawn from the research include:

- Pore pressure presence within the abutment was negligible, and earth pressure measurements were within the range of expected values
- The abutments experienced little to no settlement, and no discernable lateral displacement was recorded
- The Marshall County GRS-IBS was cost effective, constructed without the need for a specialty contractor, and was a suitable substitute for pile supported abutments.
- 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.

Two years of post-construction monitoring 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.

7.3 Recommendations for Further Research

Earth pressure cells should be used so vertical and horizontal stresses can be estimated. Piezometers should be installed at each earth pressure cell location so effective stress can be determined. Strain gages could also be a useful tool to measure reinforcement, horizontal and vertical abutment strains. The strains can be used to estimate the stresses within the abutment or on the facing material.

Large-scale performance tests conducted by Nicks et al. (2013) showed that GRS facing elements contribute to the overall strength of the abutment, although no direct relationship was discovered from these tests. Establishing the relationship between facing element selection and strength of the GRS mass could result in less geosynthetic reinforcement and backfill aggregate needed, hence, reducing cost of GRS-IBS implementation.

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. A similar study could be conducted with Alabama's next GRS-IBS. The Turkey Creek GRS-IBS was built atop hard sandstone; the reinforced fill/rock interface contributed to the satisfactory performance of the abutment. If another structure is constructed in similar geology, this interface could be modelled and studied.

7.4 Technology Transfer

The technology transfer mechanism recommended by the authors is to develop a module (approximately 2 hours) that can be delivered through ATAP summarizing the design and construction of GRS-IBS. A brief version of this report (without the field testing aspects) complementing the module, will be provided as a guide for ALDOT, County, and City engineers considering GRS.

There were a host of resources developed in the first two rounds of FHWA Every Day Counts: EDC-1(2011-2012) and EDC-2(2013-2014) culminating in the more recent document Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems by Adams and Nicks (2018). A bibliography of the key GRS design resources will be provided.

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APPENDIX A: WORKING DRAFT OF SPECIAL PROVISION

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 have 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