Differential Settlement of a Geosynthetic Reinforced Soil Abutment: Full-Scale Investigation: Summary Report

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The Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) uses alternating layers of closely spaced geosynthetic reinforcement and well-compacted granular fill to support the bridge superstructure and form an integrated roadway approach. This system offers simple and rapid construction, lower costs than traditional alternatives, and reduction or elimination of the bump at the end of the bridge. However, like all shallow foundations, GRS-IBS can be vulnerable to differential settlements beneath the foundation.

This report summarizes the final project report by Kost et al. (2015) that describes research into the behavior of GRS abutments subjected to differential settlements, which may be due to compressible soils beneath the foundation or to scour undermining. A field-scale model was constructed and subjected to carefully controlled differential settlements, and a comprehensive instrumentation program monitored the response of the abutment. The robust response of the abutment under the large differential settlements imposed in these tests indicated that GRS abutments will perform well under the smaller levels of differential settlement that would be expected in field applications. However, if large enough differential settlements occur such that the facing blocks separate, then hydraulic forces could pose a significant hazard to the abutment if the reinforced fill is not adequately protected. Three measures to reduce the vulnerability of the reinforced fill are presented, and a predictive equation was developed to estimate the settlement of the abutment's facing blocks in response to differential foundation settlement. The predictive equation is specific to the conditions of the field-scale test.

The authors recommend that the Virginia Department of Transportation's Structure and Bridge Division consider GRS-IBS as a viable bridge technology. For crossings over water, the authors agree with the recommendation of Adams et al. (2011) that GRS-IBS should be considered only if scour concerns can be adequately addressed. In addition, the authors suggest that GRS-IBS designers consider additional measures to protect the reinforced fill in the event of unanticipated settlements.

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FINAL REPORT

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DISCLAIMER

The project that is the subject of this report was done under contract for the Virginia Department of Transportation, Virginia Center for Transportation Innovation and Research. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. Any inclusion of manufacturer names, trade names, or trademarks is for identification purposes only and is not to be considered an endorsement.

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ABSTRACT

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INTRODUCTION

Geosynthetic reinforced soil (GRS) consists of alternating layers of well-compacted granular fill and closely spaced geosynthetic reinforcement. While GRS systems share many similarities with more familiar mechanically stabilized earth (MSE) systems, proponents of GRS highlight differences between the two systems and provide separate guidelines for design and construction of GRS systems. In particular, Adams et al. (2011) emphasize: (1) composite action of the compacted fill and closely spaced geosynthetic reinforcement layers in GRS systems and (2) frictional connections between the reinforcement and facing elements in GRS walls versus structural connections between the reinforcement and facing elements in most MSE walls.

GRS systems have been utilized in retaining structures since the 1970s. More recently, researchers at the Federal Highway Administration (FHWA) (Adams et al., 1999), the Colorado Department of Transportation (Abu-Hejleh et al., 2002), and the University of Colorado (Wu et al., 2006) have explored applications for bridge abutments. The result has been the development of the GRS Integrated Bridge System, or GRS-IBS. The system consists of a reinforced soil foundation (RSF), a GRS abutment, and an integrated approach (Adams et al., 2011). The bridge superstructure is supported directly on the reinforced soil abutment, without utilizing any deep foundations.

GRS-IBS technology offers a number of advantages over traditional pile-supported bridges, including lower cost, simple and rapid construction, and reduced environmental impact (Adams et al., 2011). The abutment supports both the superstructure and approach and allows these two components to settle uniformly, reducing or eliminating the bump that often forms at the end of the bridge. However, the absence of deep foundations has contributed to a perceived vulnerability of the system to changes in the support condition at the foundation level. Differential settlements have the potential to negatively impact any type of shallow foundation,

including GRS abutments. In particular, because many GRS-IBS bridges are placed along waterways, the possibility of scour-induced settlement must be considered.

While reinforced soil structures are widely considered to possess inherent flexibility, the authors are not aware of any research examining the behavior of a load-bearing GRS structure subjected to variable support conditions at its base. Like all shallow foundations, GRS-IBS structures are designed with the assumption that the structure will not experience detrimental differential settlements. The designer will conduct subsurface explorations, require excavation and replacement of soft soils beneath the RSF, and provide compaction specifications for the subgrade beneath the RSF in an effort to minimize settlements due to compressible soils beneath the foundation (Adams et al., 2011). Likewise, the designer will assess hydraulic flow, scour potential, and channel instability; place the top of the RSF beneath the calculated scour depth; and employ scour countermeasures as appropriate, to guard against scour-induced settlements (Adams et al., 2011). However, it is helpful for both the designer and the owner to understand the potential consequences for the structure if these measures fall short. Additionally, investigating the response of GRS abutments subjected to large differential settlements provides unique insight into their behavior that could not be gained from observations of the structures under normal operating conditions.

PURPOSE AND SCOPE

This project began as a collaboration between the Virginia Department of Transportation (VDOT) and Virginia Tech to provide design support and monitoring for the first GRS-IBS bridge in Virginia. A crossing along Towlston Road near McLean, Virginia, was selected for the pilot project. However, a number of extenuating circumstances, including the washout of the existing structure in September 2011, contributed to significant delays for the project. When it became evident that much of the original scope of work would occur outside of the schedule for this project, Virginia Tech proposed modifications to the scope to examine the effects of differential settlements on GRS-IBS structures.

The purpose of this research was to address a lack of knowledge regarding the performance of GRS-IBS in response to differential settlements, whether these settlements result from the presence of compressible soils beneath the foundation or are due to scour of the subgrade material. This knowledge will help policy-makers, owners, and designers make informed decisions regarding implementation of GRS systems. To this end, a field-scale investigation was carried out at Virginia Tech. The field-scale experiment examined the response of one GRS abutment to differential settlements represented by two different areas and two different depths, for a total of four support-loss conditions. Because the field-scale model lacked an integrated approach, the term "GRS abutments" is used in place of "GRS-IBS" throughout this report when referring to observations specific to the experimental abutment. However, the experimental GRS abutment did include a reinforced soil foundation, a load-bearing GRS wall, and an equivalent bridge load, so the test was representative of a GRS-IBS installation in most respects.

Nine project tasks were established:

1. Develop a test concept to induce carefully controlled settlements beneath a field-scale GRS abutment.

2. Design the field-scale model to adequately represent an in-service abutment while ensuring measurable abutment response and remaining within the constraints of the testing site.

3. Select materials that are representative of materials commonly used for GRS-IBS.

4. Construct the field-scale abutment using techniques and methods representative of typical GRS-IBS construction.

5. Instrument the abutment to observe key indicators of structural response during testing, including settlement of the fill and facing blocks, stress changes within the reinforced fill, and strain changes in the reinforcement.

6. Subject the abutment to differential settlements of carefully controlled magnitude and area, and measure the abutment's response.

7. Reduce and interpret data collected during testing, identifying important trends and behavior of the abutment.

8. Develop a predictive equation to estimate the settlement of the abutment as a function of area of support removed, depth of support removed, and height above the foundation.

9. Develop conclusions and recommendations for VDOT regarding the implementation of GRS-IBS based on the results of this study and the authors' experience in designing and constructing the test abutment, and provide recommendations for future research.

This report summarizes the results of these project tasks. Complete details of this project, particularly with regard to the methods, results, and discussion, are provided in the final project report (Kost et al., 2015). Design and construction procedures (Tasks 1-4) are summarized in the "Methods" section. The extensive instrumentation effort (Task 5) and testing procedures (Task 6) are also summarized in the "Methods" section. Data were collected during testing and reduced. The key conclusions are presented along with the predictive equation in the "Results and Discussion" section (Tasks 7 and 8). The "Conclusions" and "Recommendations" sections (Task 9) follow, and the report concludes with "Benefits and Implementation Prospects."

METHODS

A field-scale testing program was carried out to investigate the response of GRS abutments to differential settlements. The following sections summarize the experimental concept and design, materials, testing site, construction, instrumentation, and testing procedures.

Experimental Concept

A method of simulating settlement beneath a column-supported embankment using geofoam was developed by Sloan et al. (2013). This method was modified to simulate differential settlements beneath a GRS abutment. A field-scale abutment was constructed over a subgrade in which the support conditions were carefully controlled. The primary subgrade material was a well-compacted crushed rock fill. However, in regions where differential settlements would be induced, a portion of the crushed rock was replaced with a stiff, expanded polystyrene (EPS, commonly referred to as geofoam) inclusion. This foam was stiff enough to support the abutment without excessive deformations, and it could later be dissolved using an environmentally friendly solvent to simulate settlement in these regions.

Experimental Design

Thorough consideration was given to appropriate dimensioning of the experimental abutment and the regions from which support would be removed. The objectives throughout the design process were to model a reasonable abutment geometry, to ensure that settlements induced at the foundation level would be large enough to produce measureable deformations at the surface, and to minimize the influence of tests at one corner of the abutment on results observed at the opposite corner. This section details the design of the abutment and reinforced soil foundation, the location of the geofoam inclusions, and a protective wrap that was included behind the facing elements at one level of the abutment.

Abutment and Reinforced Soil Foundation

The test abutment, including its reinforced soil foundation (RSF), was designed and detailed in accordance with FHWA guidelines (Adams et al., 2011). This design procedure is most reflective of the current state of the practice. Figure 1 shows a front view of the abutment configuration with the relevant dimensions marked, and Figure 2 shows a side view of the abutment with the internal reinforcement layout. Consistent with the terminology used in the FHWA guidelines, this report will use the term "length" to refer to the abutment dimension in the horizontal plane of the abutment face, transverse to a non-skewed bridge alignment. The term "width" will refer to the abutment dimension in the horizontal plane of the wing walls, parallel to a non-skewed bridge alignment. The overall dimensions of the abutment, about 10 ft high by 24.5 ft long, represent a geometry that is reasonable for a relatively small full-scale bridge—for example, a one-lane bridge over a small to moderately sized stream.

When possible, the most critical case was selected in designing the abutment. For example, the base width of the abutment was set to 5 ft, the minimum allowed by the FHWA guidelines for small spans. However, many of the dimensions were also constrained by the testing location. For example, the length of the abutment was limited by the size of the concrete mat foundation. The height was limited both by site constraints and by the need to ensure surface expression of the differential settlements at the base (i.e., measureable deformations at the surface of the abutment).
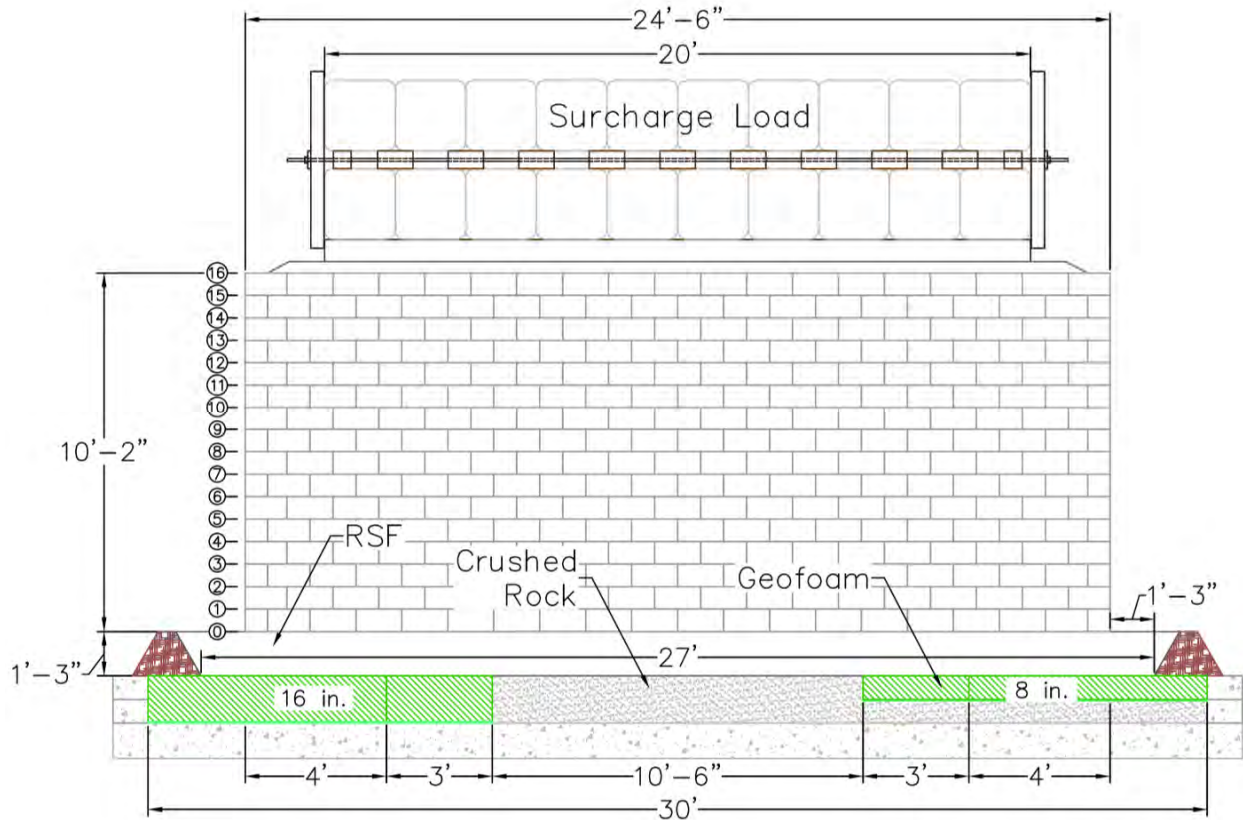
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Figure 1. Front View of Abutment Configuration

The width of the abutment increased at a constant 1H:1V slope over the entire height of the abutment. Because the base of the abutment was placed above the surrounding grade, a ramp of compacted, crushed rock was placed as construction progressed to support the abutment at this 1H:1V slope. This ramp also served to simulate the cut or fill slope behind the abutment, to apply representative horizontal earth pressures to the back of the abutment, and to permit delivery of materials to the top of the abutment throughout construction.

Primary reinforcement was spaced at 8-in vertical intervals throughout the abutment. Secondary reinforcement was placed in the upper five levels at the midpoint of primary reinforcement, resulting in a combined spacing of 4 in for the primary and secondary reinforcement, as shown in Figure 2. The secondary spacing and bearing bed for the surcharge load were detailed in accordance with FHWA guidelines, although some non-structural detailing was omitted. The integrated approach was also omitted from the test abutment.

The RSF extended a minimum of 0.25 times the base width, or 15 in, from the abutment at the face and both wing walls and was 15 in deep. Adams et al. (2011) indicate that reinforcement is commonly spaced at 12 in within the RSF. In order not to exceed this recommendation, one sheet of primary reinforcement was placed within the RSF at a distance of 7.5 in from the bottom and top of the RSF.
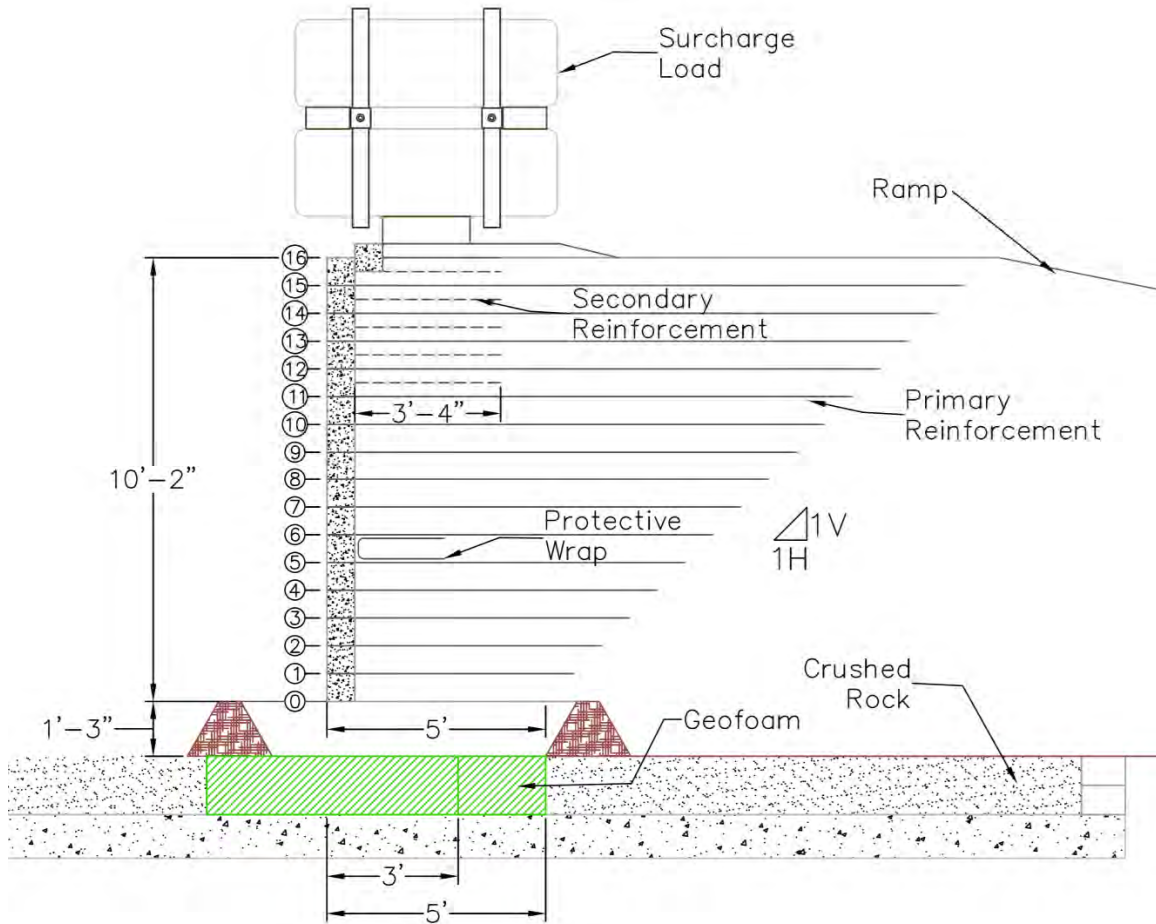
Figure 2. Side View of Abutment Configuration, Showing Reinforcement Spacing

Surcharge Load

A surcharge load consisting of large, precast concrete blocks was applied to the top of the abutment to simulate a bridge dead load. Twenty blocks measuring 2 ft wide by 2 ft high by 6 ft long were stacked on top of a foundation measuring 2 ft wide by 20 ft long, resulting in a bearing pressure of approximately 1750 psf. This load represents the maximum load that the researchers believed could be safely applied during testing without risking toppling of the concrete blocks.

In most GRS-supported bridges, the superstructure is comprised of precast, prestressed concrete box beams that are transversely post-tensioned together. In order to simulate this composite stiffness, and to improve the stability of the surcharge load, the concrete blocks were tensioned together following placement.

Geofoam Inclusions

Geofoam inclusions were placed within the subgrade beneath the abutment and RSF to serve as temporary support. These inclusions were later dissolved to induce controlled settlements beneath the abutment, and therefore their dimensions represent the variables

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investigated in this research. Consequently, considerable care was taken in selecting the dimensions of these inclusions. Figure 3 shows a plan view of the geofoam inclusions.
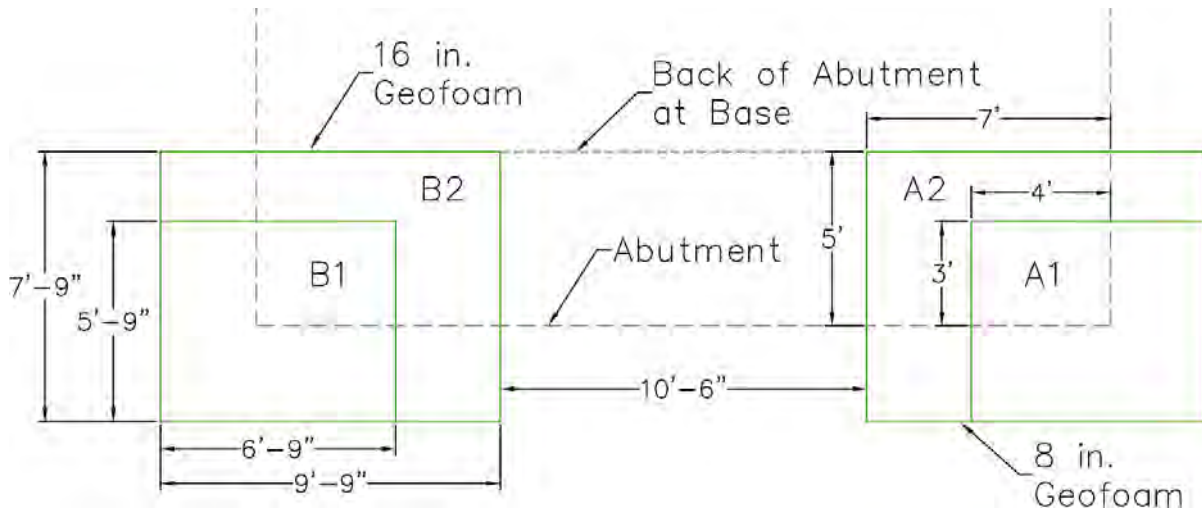

Figure 3. Location of Geofoam Inclusions

The thickness of the geofoam inclusions was selected to ensure that settlement at the base was large enough to result in measureable deformations at the surface of the abutment. The testing site could accommodate geofoam up to 16 in thick, and therefore this thickness was selected for the inclusions on one side (Side B) of the abutment. The inclusions on the opposite side (Side A) were half the thickness of Side B at 8 in thick.

The area in plan of the geofoam inclusions for each testing increment was also selected in an effort to create measureable deformations while minimizing the influence of the tests at one side of the abutment on the tests at the opposite side. After considering a number of possible configurations, an area of 3 ft by 4 ft beneath the abutment was selected for the first test on each side (i.e., Test A1 and Test B1), and an area of 5 ft by 7 ft was selected for the second test on each side. The actual dimensions of the geofoam inclusions were larger in order to completely undermine the RSF and to allow access to the geofoam after construction of the RSF. Table 1 shows the area of base support and the percentage of the base area that was removed during each individual test, as well as the cumulative total of support removed. By the end of testing, large settlements had been induced beneath nearly 60% of the abutment, which represents a very extreme loading condition.

Table 1. Percentage of Abutment Base Area Removed During Each Testing Sequence

Test	Area of Abutment Base Support Removed (ft²)		Percentage of Abutment Base Support Removed	
	Individual	Cumulative	Individual	Cumulative
A1	12	12	9.8%	9.8%
A2	23	35	18.8%	28.6%
B1	12	47	9.8%	38.4%
B2	23	70	18.8%	57.1%

Note: The total base area of the abutment is 122.5 ft².

Protective Wrap

During the experiment's design, a suggestion was provided to the researchers by Andy Zickler of VDOT to explore the effectiveness of a wrap behind the facing units to prevent erosion of the backfill material in case gaps were to form between the facing units. The researchers incorporated this suggestion at one level of the abutment, Level 6, as shown in Figure 2. The wrap was designed to be one continuous length behind the abutment face and both wing walls. A 7.5-ft-wide roll of fabric was used for the wrap, resulting in an embedded length of approximately 3 ft 4 in.

In deciding to incorporate this suggestion, the researchers considered that including additional reinforcement at this level may alter the stiffness of the reinforced fill and therefore influence the stress distribution within the reinforced fill. The geotextile selected for this application was very lightweight in order to minimize this effect, and unnecessary material where the geotextile was folded at the corners of the abutment was removed. The researchers were also concerned that the interface friction angle between the two geotextiles may be less than the interface friction angle between the primary reinforcement and the fill. Consequently, a very thin layer of aggregate was placed between the two geotextiles. The procedure for installing this protective wrap is further described later in this report.

Materials

This section summarizes the materials that were used in the construction and testing of the GRS abutment. Materials were selected that are typical of in-service GRS-IBS bridges.

Fill

A crushed rock fill is the primary structural component of a GRS system. The FHWA guidelines allow for use of either a well-graded fill or an open-graded fill. To date, the vast majority of GRS-IBS projects have used an open-graded fill, and therefore an open-graded fill (ASTM No. 8) was also selected for this project. The No. 8 fill has a maximum particle size of 0.5 in and less than 5% passing the No. 16 sieve.

A well-graded fill must be used for the RSF. For this project, the VDOT No. 21A crushed rock, which has a maximum particle size of 2 in and a fines content between 6% and 12%, was selected for the RSF.

Geotextiles

Two geotextiles were used in the construction of the abutment. The geotextile used for fill reinforcement was a TenCate Mirafi HP570, which is a biaxial, woven polypropylene geotextile having an ultimate tensile strength of 4,800 lb/ft. The HP570 was used for both the primary and secondary layers of reinforcement in the abutment, and also for the RSF.

The second geotextile was used to create the protective wrap behind the Level 6 facing units. The Tencate Mirafi 140N, a very lightweight geotextile for filtration applications, was

selected for this application. The 140N is a needlepunched, non-woven geotextile with an apparent opening size (AOS) of 0.212 mm and a permittivity of 1.7 sec^{-1}, according to the manufacturer.

Facing Units

This project used nominal 8 in by 8 in by 16 in hollow-core concrete masonry units (CMUs) with a compressive strength of 4,000 psi. The project duration was less than six months and occurred in the summer and fall; therefore, resistance to freeze-thaw cycles was not considered.

Geofoam

An Insulfoam EPS39 geofoam was selected for the geofoam inclusions beneath the abutment. In general, geofoam should have a large enough stiffness to ensure that its strain does not exceed 1%, or creep behavior may result. According to the manufacturer, the EPS39 experiences 1% strain at a compressive stress of 15 psi, which is considerably higher than the estimated 11.6 psi of stress applied to the subgrade.

Geonet

A geonet was used as part of the solvent delivery system to evenly distribute the solvent beneath the geofoam inclusions. The geonet used for this project was a SynTec UBXC, which has a thickness of 200 mils and a transmissivity of 2×10^{-3} m^2/s in the machine direction, according to the manufacturer.

Solvent

Expanded polystyrene (EPS) can be readily dissolved using a variety of agents. For this project, a biodegradable, non-caustic solvent known as d-Limonene was used. d-Limonene is derived from concentration of citric oils and is commonly used in the recycling industry to reduce the volume of EPS packaging.

Testing Site

The abutment was constructed at Virginia Tech's Kentland Farm, located about 10 miles west of the Blacksburg campus. At the farm, the researchers have access to a 30-ft-square, 12-in-thick concrete mat foundation. The mat is enclosed by a 16-in-high CMU wall, with the top of the wall at grade with the adjacent ground surface. The mat provides a stable foundation for the abutment and ensures that measured deformations are the result of the induced differential settlements rather than consolidation of the underlying natural soils.

Construction

Construction of the test abutment utilized a crew of two or three students who did not have prior experience with GRS construction. Even so, the crew found that they were able to

carry out the construction operation without undue difficulty. Construction equipment was generally small and hand-operated. A small Bobcat utility vehicle was also used to move fill and other materials during construction.

Construction of the subgrade underlying the test abutment comprised the first phase of construction. Each region of geofoam was carefully packaged in plastic to isolate the geofoam from surrounding regions and to contain the solvent and dissolved EPS. The RSF and the abutment were then constructed over the subgrade, following the guidelines established by the FHWA (Adams et al., 2011). Finally, the surcharge load was placed and tensioned together to provide additional stability to the load and to reflect the composite stiffness of a concrete box-beam superstructure. Additional details of the construction process are provided in the final project report (Kost et al., 2015). The completed abutment is shown in Figure 4.

Figure 4. Completed Abutment

Instrumentation

During construction, five types of instrumentation were installed to monitor the behavior of the abutment. Instrumentation included survey targets, PVC tubing for settlement measurements, earth pressure cells, draw-wire extensometers, and resistance strain gages. With the exception of the survey targets, the location of each instrument is shown in Figure 5 and Figure 6. Survey targets were placed on the CMU blocks comprising the face and wing walls, and survey points were established on the surface of the RSF and the abutment. Some of these survey targets are visible in Figure 4. A total of 214 survey points and targets were placed to provide comprehensive coverage of the abutment.
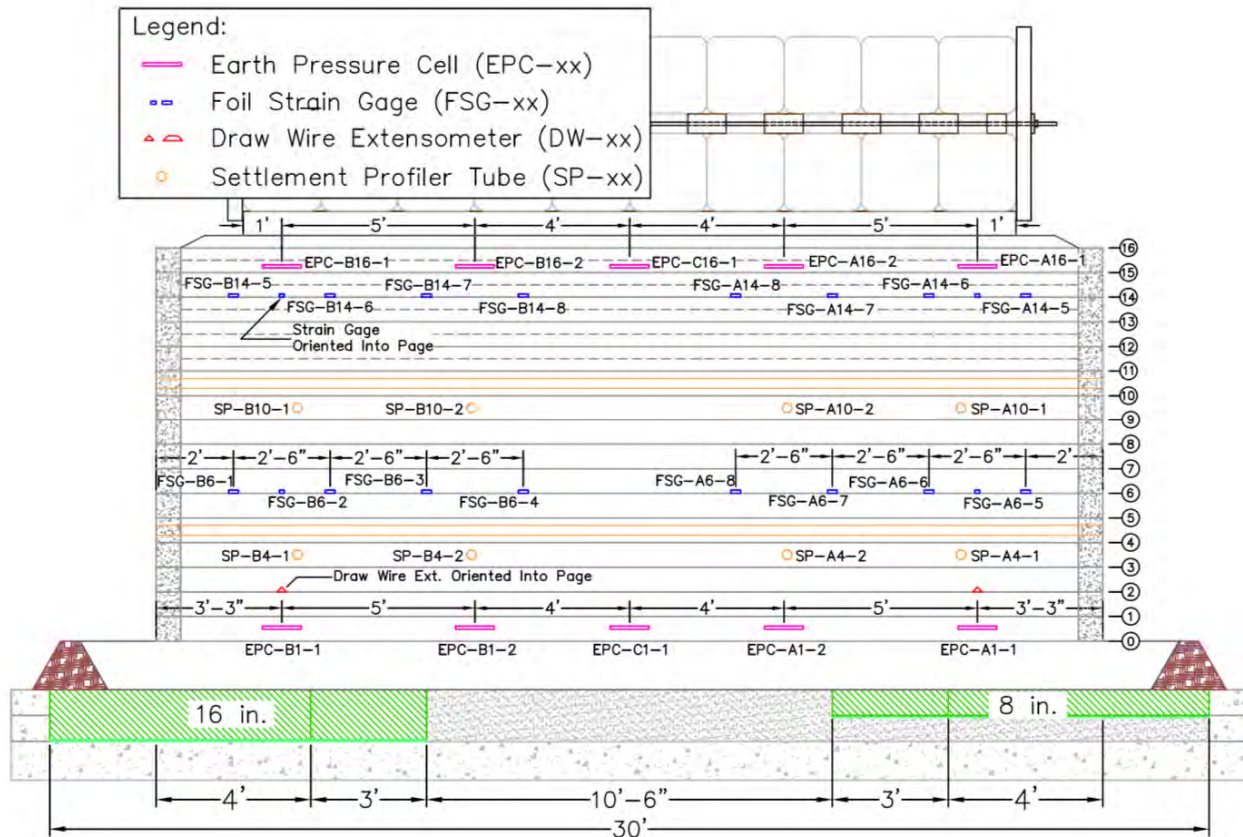
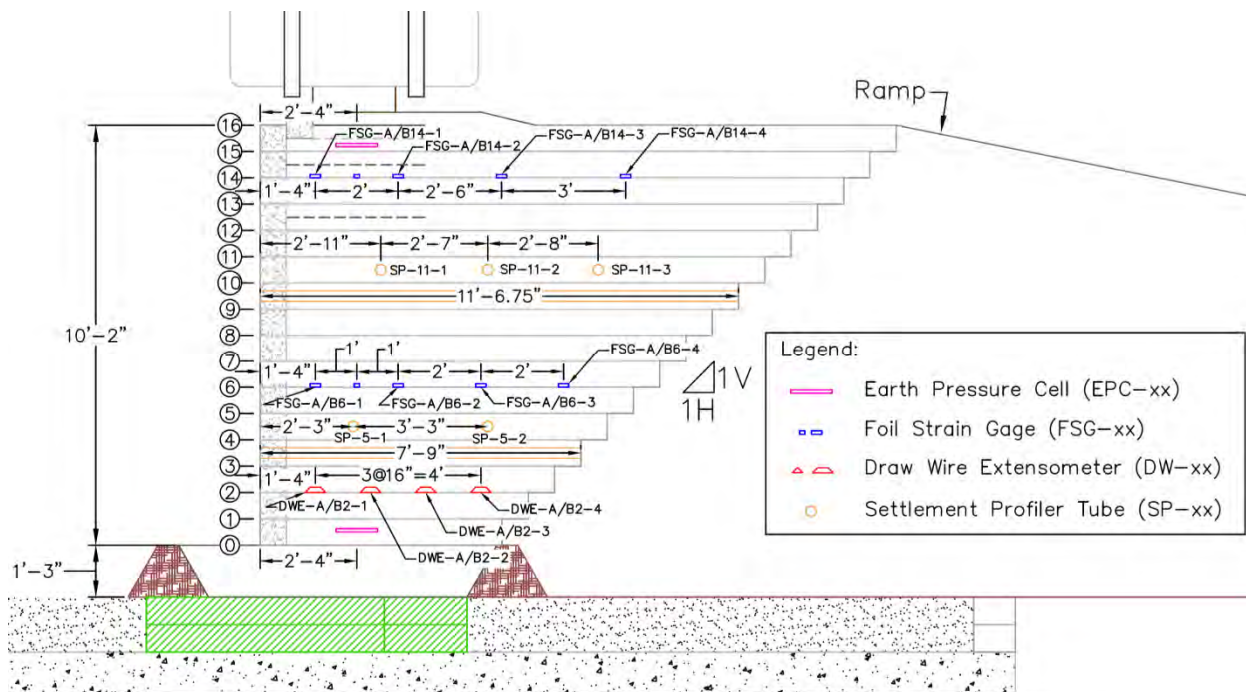**Figure 5. Front View of Instrumentation Configuration**

Figure 6. Side View of Instrumentation Configuration

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The identification number for each instrument begins with a three-letter code identifying the type of instrumentation—e.g., "EPC" identifies an earth pressure cell. The codes are provided in a legend with each figure. The three-letter code is followed by a dash, a letter, and a number. The letter identifies whether the instrument is on Side A (A), Side B (B), or in the center of the abutment (C). For the settlement profiling tubes that run the length of the abutment, this letter is omitted altogether. The subsequent number identifies the abutment level where the instrument is located. This number is followed by a dash and a final number, which identifies the particular instrument at the indicated level. Instruments are numbered from the face toward the back of the abutment, or from the wing wall toward the center of the abutment, depending on the configuration. As an example, EPC-B1-2 denotes the earth pressure cell located on Side B of the abutment, within the Level 1 backfill, second instrument from the wing wall.

Data Acquisition System

A Campbell Scientific CR1000 datalogger was used to record instrument readings for the earth pressure cells, draw wire extensometers, and strain gages during construction and testing. Three Geokon 8032 multiplexors were used to accommodate the large number of instruments, one for each type of instrumentation. An AVW-200 signal analyzer pre-processed the signal from the vibrating wire pressure cells before it was recorded by the datalogger.

Survey data were stored in the total station instrument and transferred to a laptop computer. The settlement profiler was monitored using a Geokon GK-404 handheld readout device, and the data were recorded by hand.

Testing Procedures

Prior to each test, the soil berm used in construction of the RSF was excavated and removed from the perimeter of the area where support would be removed. The intent of removing this berm was to account for the loss of this additional support to the RSF that would occur in some cases of differential settlement, such as scour-induced settlements. After removing the berm, the valve(s) regulating the flow of solvent to the foam region were opened, and solvent was pumped into the packaged foam region.

Following introduction of the solvent, the geofoam dissolved over a period of several hours. Data from total station surveys and earth pressure cells were monitored as the primary indicator of the progress of the test. Once the earth pressure cells exhibited asymptotic trends or the survey data indicated settlements of less than 0.003 ft over 24 hours, the test was considered complete.

RESULTS AND DISCUSSION

This section summarizes the key points from the construction process and the four testing sequences performed on the experimental abutment. A full discussion of observations and instrumentation data from the testing sequences is provided in the final project report (Kost et al., 2015).

General construction and testing observations are presented, and the most important observations from the instrumentation data are summarized. Next, based on the observations of abutment performance, three potential methods of mitigating the vulnerability of the reinforced fill are presented. A discussion of the authors' investigation into repair methods for the abutment follows, and the section concludes with recommendations for future research.

Construction Observations

Although construction of the test abutment utilized a crew of two or three students who did not have prior experience with GRS construction, the crew found that they were able to carry out the construction operation without undue difficulty. In total, construction and instrumentation of the abutment, including placement of the surcharge load, took 35 working days. A photograph of the completed abutment was presented in Figure 4. Instrumentation data were collected during construction and are discussed later in this report.

The FHWA manual by Adams et al. (2011) was consulted as the primary reference for construction. The authors found the construction guidance in the manual to be accessible and generally easy to reference, with a number of helpful photographs.

General Testing Observations

Post-testing deconstruction of the abutment revealed that the geofoam inclusions had been fully dissolved during the testing process. Figure 7 compares the abutment prior to and after all testing sequences. The figure is not intended to show the details of the deformation patterns, but rather to demonstrate that, despite removing large amounts of support from beneath much of the foundation area, the abutment was still able to adequately support the surcharge load. The CMUs and lengths of lumber placed on top of the abutment wall and beneath the surcharge load were a precautionary measure in the event that the load began to tip toward the face. The surcharge load experienced virtually no tipping during the tests, and clear space was maintained between the surcharge load and the precautionary blocking through all tests.

Figure 7. Comparison of Abutment (a) Before and (b) After All Testing Sequences

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The deformation pattern of the facing blocks demonstrates a predictable, stair-step pattern that begins at the edge of the region where support is removed. Particularly for smaller regions of support removal, the CMUs are able to bridge over the region of support loss and minimize settlements of the overlying CMUs. This pattern roughly mirrors the support condition within the reinforced fill, which is able to support the full width of the surcharge load even after support beneath a large portion of the foundation has been removed. On Side A, the deformation patterns induced by Test A1 are erased by the patterns of Test A2, suggesting that the same deformation pattern would be seen if the support of Regions A1 and A2 had been removed at one time rather than in two steps. The behavior on Side B suggests similar trends, although the larger magnitude of settlement increases the difficulty of predicting the settlement had Regions B1 and B2 been removed at the same time.

The blocks at and near the corners of the abutment appeared vulnerable after experiencing settlements. This experiment tested only the response of the abutment to differential settlements. If water action were introduced, as would occur in a scenario of scour-induced settlement or of heavy stream flow after large differential settlements due to foundation compression, it is quite conceivable (and perhaps likely) that the combined effects of buoyant and viscous water forces could remove some of these CMU blocks and expose the backfill to erosion. This scenario could lead to serious damage or failure of the abutment. Some considerations to mitigate this hazard are discussed later in this report.

Instrumentation Data

Important observations from each of the forms of instrumentation in the abutment are presented in the following five sections. A full explanation of these key points is provided in the final project report (Kost et al., 2015).

Survey Targets

A full survey of the abutment was completed before and after each testing sequence, and survey data were also used as a reference when other instrumentation data were analyzed. Important trends are summarized as follows.

- The surcharge load remained stable throughout all testing sequences.

- Settlements at the surface decreased with distance from the region of support loss.

- The maximum angular distortion of the surcharge load in the plane parallel to the abutment face was 0.008, and the maximum average angular distortion between the center and edge of the surcharge load was 0.003. Both measures are within the range of acceptable serviceability values for simply supported spans.

- The reinforced fill significantly reduced the settlements observed at the surface compared to the base of the abutment and provided acceptable performance for the surcharge load.

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A predictive equation was developed using the survey data from the CMU facing blocks. Equation 1 can be used to estimate the settlement of the CMUs if the geometry and magnitude of the region of support loss is known. The *XYZ* coordinate system is shown in Figure 8. Although an attempt was made to formulate the predictive equation in normalized terms, it is not yet known if this normalization would apply to other configurations. So, at present, the predictive equation is specific to the conditions of this test set-up.

$$\frac{S}{S_b} = \left(\frac{X}{0.5S_b + 2.55t_{RSF}}\right)^{1.07} \left(\frac{Y}{0.5S_b + 2.85t_{RSF}}\right)^{1.09} \left[1 - \left(\frac{Z}{1.04(X+Y)}\right)^{1.22}\right]$$
Equation (1)

where: S = settlement at the desired point
S_b = magnitude of support loss beneath RSF
t_{RSF} = thickness of the RSF
X = horizontal distance from edge of support loss to desired point, measured in the plane parallel to abutment face
Y = horizontal distance from edge of support loss to desired point, measured in the plane perpendicular to abutment face
Z = vertical distance from top of RSF to desired point

All terms raised to a power are restricted to a maximum value of 1.
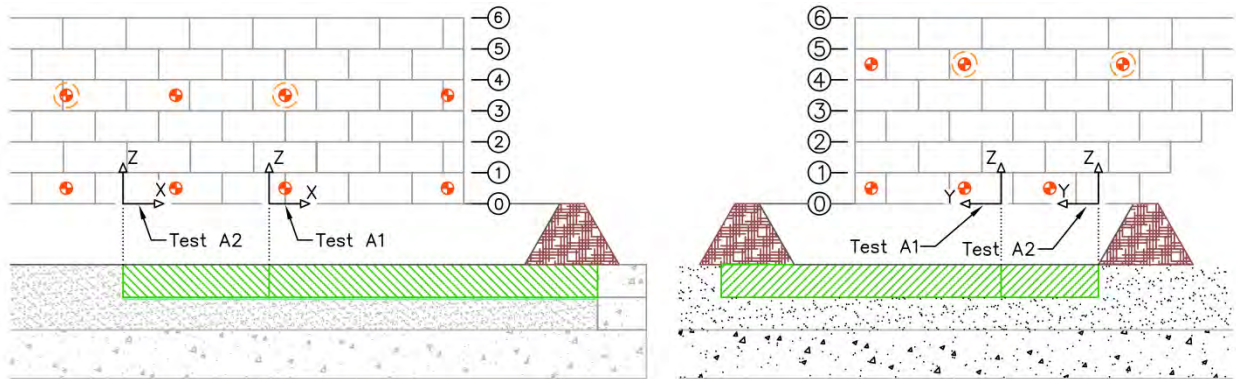
Figure 8. Coordinate System for Predictive Equation

Settlement Profiling Tubes

Settlement data for the reinforced fill were collected following each test using the profiling tubes. Tubes passed through the abutment in both directions, and points of overlap generally recorded consistent settlements. The following important trends were identified:

- In many instances, the maximum settlement of the fill at a given level was greater than the settlement of the facing CMUs at the same level.

- When support is lost beneath part of the foundation, the reinforced fill acted to bridge over the area of support loss. The additional support extends upward and outward

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from the inside edges of the region of support loss. The effect of support loss diminishes with height above the foundation.

- Settlements within the reinforced backfill decreased and the uniformity of settlement increased with height and with distance from the region of support loss due to the effect of the bridging action.

- When the depth of support removal was held constant, settlement of the fill increased with removal of the larger area of support.

- When the area of support removal was small, as in Test A1 and Test B1, the settlements of the fill were relatively similar at the level of the profiling tubes, independent of the depth of support removal.

- When the area of support removal was large, as in Test A2 and Test B2, the settlement of the fill at the levels of the settlement tubes was significantly larger on Side B than Side A, where a larger depth of support was removed on Side B. However, surface expressions of the settlements were not necessarily larger on Side B than Side A.

- The small settlements observed at the surface of the abutment in response to severe differential settlement conditions at the base demonstrate robust performance in the GRS abutment's ability to minimize surface expressions of differential settlements beneath its foundation.

Earth Pressure Cells

The pressure cells were monitored regularly from the time of installation through construction and each of the four testing sequences. Pressure data could not be monitored continuously during Test B1 due to failure of the signal analyzer for the vibrating wire instruments. Baseline readings for the test were known, and final readings were obtained at the end of the test once a replacement signal analyzer had been received and installed. Replacing the signal analyzer is not believed to have impacted the zero readings of the pressure cells. The earth pressure cells identified the following key trends:

- When support is lost beneath part of the foundation, the reinforced soil transfers load to the portion of the foundation where the support conditions are unchanged. Vertical stresses in the fill directly above the region of support loss drop substantially, although this effect diminishes with increasing height above the foundation.

- Only small deformations are required to cause a re-distribution of stresses within the reinforced fill.

- Removing a larger area of support results in more significant stress changes at upper levels of the abutment but in similar stress changes at lower levels. The first observation is expected to hold true for any scenario of support loss; the second

observation may be a function of the geometry of support removal selected for these tests.

- For the magnitudes of foundation settlement investigated in these tests, stress changes in the reinforced fill depend primarily on the area of support removed and not the depth of support removed.

- Stress increases in the regions of the reinforced fill to which load is transferred imply increased tensile demand in the reinforcement at these locations.

Draw Wire Extensometers

The draw wire extensometers at Level 2 of the abutment were monitored regularly from the time of their installation through construction and all testing sequences. During some tests, the displacement data were obscured by settlements of the overlying CMUs and fill, which resulted in erroneously large displacements. Where the displacement values were not affected in this way, the following conclusions were drawn:

- Strains observed at Level 2 during construction were small, generally less than 0.1%.

- During construction, all measured displacements showed that these strains were directed toward the abutment face.

- Strains during Test A1 and Test B1 were small and generally resulted from displacements toward the face.

- Strains during Test A2 were difficult to identify because large settlement of the facing CMUs and reinforcement controlled the observed displacements; strains during Test B2 were completely obscured by settlements of the facing CMUs.

Resistance Strain Gages

Due to multiplexor limitations, only half the gages in the abutment could be monitored at one time. Strain gages on Side A were monitored continuously from the time of installation through both testing sequences on Side A. Following the completion of Test A2, these gages were disconnected, and the gages on Side B were connected and monitored through the completion of testing. The following trends were observed:

- The maximum strain observed at Levels 6 and 14 on Side A during construction and placement of the surcharge load was 0.8%. The maximum strain increase observed during placement of the surcharge load alone was 0.14%.

- The FHWA procedure for calculating lateral strains due to placement of the surcharge load gave a reasonable, if slightly conservative, estimate.

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- The maximum strain observed at Levels 6 and 14 on Side A after construction, Test A1, and Test A2 was 1.0%. Despite large differential settlements beneath the foundation imposing a severe loading condition, observed strains remained well within the acceptable range.

- With the exception of the corner gages, strain increases were relatively uniform, indicating that the increased demand was well-distributed along the reinforcement.

- The significant variability in the response of the corner gages showed that they were likely influenced by curvature of the reinforcement, and perhaps by connection slipping in some cases. It also suggests that stress concentrations are most likely to develop near the face and wing walls of the abutment.

- Test A1 and Test B1, which removed two different depths of support from a smaller area beneath the foundation, resulted in similar strain increases on each side.

- Test A2 and Test B2, which removed two different depths of support from a larger area beneath the foundation, resulted in higher strains at Level 6 in the direction parallel to the abutment face and higher strains at Level 14 in the direction perpendicular to the abutment face. One possible explanation for this behavior, based on the abutment geometry and the area of support removed, is presented in the final project report (Kost et al., 2015).

Potential Mitigation Measures

The large settlements of some CMU facing blocks created gaps between the blocks that would make them vulnerable to removal during periods of elevated stream flow. Three possible measures to provide additional robustness to the GRS abutment are summarized in this section, and further details are given in the final project report (Kost et al., 2015). Determining the appropriateness of implementing these suggestions at a given site is the responsibility of the designer.

Performance of Protective Wrap

The lightweight filtration geotextile proved to be effective at containing and protecting the backfill when gaps formed between facing units. The performance of the wrap when exposed to water action was not evaluated; however, the authors are optimistic that its performance would be satisfactory. The performance of the wrap with an entire facing block removed could not be evaluated due to stability concerns associated with attempting to remove an entire CMU block.

Overall, the protective wrap appears to be a low-cost, easily installed measure that may offer an additional layer of protection for the backfill. Including such a wrap may be a worthwhile consideration for some sites.

Pinning of Corner Blocks

The authors noted that the corners of the abutment appeared particularly vulnerable after settling. One method to mitigate this exposure would be to fill the blocks near the corner with concrete or grout and pin them together using vertical lengths of rebar, similar to way that the upper three courses were pinned together. Adams et al. (2011) describe a similar process for joining the wing wall and face of the abutment when the joint is not a right angle, and they note that such a process can be used to add strength to the wall corners. Pinning these blocks together would likely reduce the possibility of losing a facing block in the event of combined settlements and water action. This process could also be used to reinforce the corners if impact loads are a concern for the site. Additional details and considerations for this mitigation measure are discussed in the final project report (Kost et al., 2015).

Increasing the Base Width of Abutment

The stability of the wall could be increased by increasing the base width of the abutment. This measure will prevent the reinforcement from losing embedment in the case of settlements beneath a large area of the foundation. Additionally, if this measure is combined with the suggestion to pin the corner blocks of the abutment, the authors believe that a synergistic effect may be observed. However, increasing the base width will also increase the overall cost of the abutment by increasing the volume of backfill and reinforcement required to construct the abutment, and in many cases increasing the volume of native soil that must be excavated. This measure is expected to offer the most significant benefits to abutments that would otherwise have a small base width, and the stability improvements it offers will likely decrease as the base width increases.

Investigation of Repairs

Following the four testing sequences, the authors considered whether any measures could be taken to repair the abutment. Pressure grouting was the primary method considered for repairs. Allen Sehn at Hayward Baker, Inc. spoke with the authors over the telephone and, after hearing a description of the problem, expressed his opinion that the likelihood of a successful repair using this or another ground improvement method was low. The primary concern Sehn mentioned is that the confining pressures in front of and beside the wall would not be large enough to contain grout applied with large enough pressures to lift blocks back to an elevation close to their original elevations.

If a GRS abutment were to experience small differential settlements that did not result in structural distress but created gaps between the facing blocks, the authors believe that these gaps could be patched effectively using concrete without significantly affecting the structural behavior of the abutment. Such repairs might not be attractive.

Recommendations for Future Research

This project is, to the authors' knowledge, the first substantial investigation into the effects of differential settlement on GRS abutments. The large scale of the project has produced

a sizeable and unique data set, which presents a singular opportunity for further exploration of this subject. The data set can allow for calibration and validation of a numerical model that can be used to perform parametric studies and identify the influence of different abutment geometries, different bridge loads, different areas of support loss, and different depths of support loss. For example, the effect of support removal beneath the center of the abutment, rather than the corners, can be investigated. The numerical model can also be used to improve upon the predictive equation for settlement of the abutment that is presented in this report. Although the major focus of this research was on the effect of foundation settlements beneath a GRS abutment, much data were also obtained during GRS abutment construction and bridge loading. Hence, the numerical model could be used to reliably perform many numerical experiments at much lower cost than large-scale experiments. The results could then be compared with existing analysis procedures (e.g., Adams et al., 2011) to investigate the validity of those procedures.

A protective wrap consisting of a lightweight filtration geotextile was placed behind the facing blocks at Level 6. The purpose of this trial wrap was to examine its effectiveness in protecting and containing the reinforced fill should gaps form between the facing blocks. While the wrap was effective to this end, an evaluation of its performance when water action is applied was not performed at that time. If VDOT is considering implementing this wrap in a GRS structure that may be subjected to water action, further evaluation of its performance under these conditions would be prudent. The effects of removing one or more facing blocks could also be examined. This evaluation could be performed on a small GRS mass and would not constitute a large undertaking. The authors also placed a thin layer of aggregate between the filtration geotextile and the reinforcing geotextile to increase the interface friction angle. However, it is possible that maintaining adequate quality control oversight of this component would be challenging. Research could also be conducted to examine the interface friction of these two geotextiles to determine whether the thin layer of aggregate is necessary.

CONCLUSIONS

- *A testing concept was developed to induce carefully controlled differential settlements beneath a field-scale GRS abutment.* This testing concept was adapted from previous column-supported embankment tests and used geofoam blocks as temporary support inclusions within the subgrade, which could later be dissolved using an environmentally friendly solvent.

- *A field-scale model was designed and constructed using the FHWA manual by Adams et al. (2011) to accurately represent an in-service abutment.* The authors found the FHWA manual to be a helpful and relatively comprehensive reference. The geometry of the abutment, materials, and construction techniques were representative of typical GRS-IBS construction.

- *Construction of the abutment was simple and efficient.* The abutment was completed in 35 working days by two or three unskilled laborers who were inexperienced with GRS construction techniques, using a small utility vehicle and hand-operated equipment.

- *A comprehensive instrumentation plan was designed and implemented to observe the response of the abutment during testing.*

- *Differential settlements of carefully controlled magnitude and area were induced beneath the foundation of the GRS abutment, and the abutment response was measured.*

- *The GRS abutment demonstrated robust behavior in response to large differential settlements.* While the large magnitude and area of settlement at the base represented an extreme loading condition for this abutment, the settlements expressed at the surface of the abutment were small. Stresses were redistributed within a relatively small height above the foundation following support loss beneath the corners, maintaining support capability at upper levels of the abutment. Strain increases in the reinforcement due to this bridging action were not large and were generally well-distributed along the length of the reinforcement.

- *A predictive equation was developed that can be used to estimate the settlement of the facing blocks at points over the area of support loss.* The equation was calibrated using data from the lower 11 levels (7 ft) of the abutment, and it is best-suited for use in this range. Although an attempt was made to formulate the predictive equation in normalized terms, it is not yet known if this normalization would apply to other configurations. So, at present, the predictive equation is specific to the conditions of this test set-up.

- *The performance of a GRS abutment is expected to be excellent when subjected to normal levels of differential settlement due to compressible soils beneath the foundation.* The robust response of the field-scale GRS abutment to large settlements suggests excellent performance at levels normally observed in typical applications.

- *The performance of a GRS abutment may be severely compromised when exposed to scour-induced settlements, if steps are not taken to protect the backfill from erosion.* Scour-induced settlements imply that water will also be flowing along the face of the abutment. Even after experiencing small settlements, the facing CMUs near the corner were in a very loose condition, and they would be susceptible to removal due to water action. The backfill would then be exposed to erosion by the water, potentially impacting the structural integrity of the abutment.

RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division should consider GRS-IBS as a viable option for new bridges and bridge replacements.* While GRS-IBS has limitations, it is shown in this experiment to be a robust, flexible system that can be constructed efficiently and at low cost.

2. *For bridges crossing over water, VDOT's Structure and Bridge Division should consider GRS-IBS only if scour issues can be appropriately addressed according to the guidelines provided by Adams et al. (2011).* Placing the top of the RSF below the calculated scour depth and/or implementing appropriate scour countermeasures are two possible means of

managing scour concerns. If scour cannot be addressed in a constructible and economical manner, GRS-IBS should not be used.

3. *When GRS-IBS is selected for a water crossing, VDOT's Structure and Bridge Division should include additional measures to protect the backfill in the event of a facing element becoming dislodged.* Three such measures considered in this study are placing a protective wrap behind the face, joining the corner blocks with vertical lengths of rebar and grout or concrete, and increasing the width of the base. The authors recognize that this study was not designed to comprehensively examine these mitigation measures but are confident of their utility after observing the performance of the test abutment.

4. *VDOT's Structure and Bridge Division should inspect in-service GRS-IBS bridges on a regular schedule to identify and mitigate the effects of differential settlements early.* GRS-IBS bridges crossing over water should also be inspected after severe flooding events, according to FHWA guidelines (Adams et al., 2011). If differential settlements introduce gaps between the facing blocks that are severe enough to allow a block to be removed, the gap should be repaired immediately. One repair measure discussed in this report is to patch the gap with concrete.

5. *VDOT's Structure and Bridge Division should use the FHWA manual by Adams et al. (2011) for the design and construction of GRS-IBS bridges.* Until further research validates or updates its recommendations and design procedures, this manual best represents the current state of the practice, and it was useful to the authors in designing and constructing this experimental abutment. For the one performance criterion that could be compared with the values predicted by the FHWA manual—lateral strains—the FHWA procedure gave somewhat conservative results.

6. *VCTIR should consider authorizing additional research into the performance of GRS-IBS bridges that are subjected to differential settlements.* While the field-scale abutment performed well under severe differential settlements, in many cases such settlements may represent the greatest threat to the performance of a GRS abutment near a waterway because of the potential for loosening the frictional connection between blocks. The data set compiled in this field-scale experiment provides an excellent opportunity to improve understanding of the behavior of GRS abutments by developing a numerical model, which can be used to thoroughly investigate the influence of a range of parameters.

BENEFITS AND IMPLEMENTATION

The results of this research have validated a number of assumptions about the performance of GRS abutments. Reinforced soil structures are generally considered to possess inherent flexibility, but the behavior of GRS abutments, which also must support a surcharge load, had never been rigorously examined. This research showed that the experimental GRS abutment offered robust performance when subjected to severe differential settlements and maintained an acceptable level of performance for the surcharge load. However, the abutment also appeared vulnerable if scour undermining were to induce settlements of the facing blocks.

This understanding underscores the importance of complying with FHWA recommendations for managing scour potential, and it allows VDOT to make informed policy and design decisions regarding the implementation of GRS-IBS. GRS-IBS bridges can offer real cost and time savings, and the recommendations presented in this report can be used to target the most appropriate sites for GRS-IBS. Simple, low-cost measures to provide additional protection to GRS abutments along waterways were introduced. Finally, a predictive equation was developed that allows a designer to estimate the settlement of facing blocks when subjected to base settlements of known magnitude and area.

VDOT is implementing GRS-IBS technology through construction projects in its engineering districts, the first of which is the Towlston Road Bridge in Fairfax County. Recommendations from this study are being incorporated by Structure and Bridge division into designs for these structures.

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