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## ERRATA

### *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*

Publication No. FHWA-HRT-11-026

Dear Customer:

Editorial corrections were made to this report after it was originally published. The following table shows the modifications that were made to this report.

Location	Correction
Page 9, section 2.1	<b>Added</b> “ $RF_{global}$ : Global reduction factor for the geosynthetic to account for long-term strength losses due to installation damage, creep, and durability [dimensionless]
Page 17, last paragraph	<b>Change</b> “gradations are shown in sections 3.3.2.1 and 3.3.2.2” <b>to</b> “gradations are shown in sections 3.3.1.1 and 3.3.1.2”
Page 34, step 6	<b>Change</b> “see section 4.4.7.3.1” <b>to</b> “see section 4.4.7.3”
Page 42, third paragraph	<b>Add</b> “Direct sliding should also be checked at the interface between the RSF and the foundation soils.” at end of paragraph.
Page 44, section 4.3.7	<b>Change</b> “refer to appendix B” <b>to</b> “refer to appendix C”
Page 49, section 4.3.7.3	<b>Change</b> “(2) it must be less than the strength at 2 percent reinforcement strain ( $T_{@ε=2\%}$ ).” <b>to</b> “(2) it must be less than the strength at 2 percent reinforcement strain ( $T_{@ε=2\%}$ ) in the direction perpendicular to the abutment wall face.”
Page 72, first paragraph	<b>Delete</b> two instances of “(see section 4.5.3)”
Page 73, first paragraph	<b>Change</b> “Section 6.5 discusses drainage details” <b>to</b> “Section 7.11 discusses drainage details.”
Page 88, last paragraph	<b>Change</b> “Overlapping between sheets is required.” <b>to</b> “Overlapping between sheets is not required.”
Page 90, fourth paragraph	<b>Add</b> “In the bearing reinforcement zone, hand-operated compaction equipment should be used over the 4-inch lifts to prevent excessive installation damage of the reinforcement.” after second sentence.
Page 137, first paragraph	<b>Change</b> “Refer to section 4.4 for discussion ... and to section 4.5 for the ASD calculation” <b>to</b> “Refer to section 4.3 for discussion ... and to section 4.4 for the ASD calculation”
Page 141, first paragraph	<b>Change</b> “A resistance factor for reinforcement strength ( $\Phi_{reinf}$ ) of 0.4 should be applied to the ultimate strength ( $T_f$ ) to determine the factored reinforcement strength ( $T_{ff}$ ).” <b>to</b> “In addition to a global reduction factor of 2.25 accounting for long-term strength losses ( $RF_{global}$ ) of the geosynthetic, a resistance factor for reinforcement strength ( $\Phi_{reinf}$ ) of 0.9 should be applied to the ultimate strength ( $T_f$ ) to determine the factored reinforcement strength ( $T_{ff}$ ).”

<p>Page 141, equation 93</p>	<p><b>Change</b></p> $\frac{T_{f,f}}{T_{req,f}} = \frac{\Phi_{reinf}(T_f)}{T_{req,f}} = \frac{0.4(T_f)}{T_{req,f}} \geq 1.0$ <p><b>to</b></p> $\frac{T_{f,f}}{T_{req,f}} = \frac{\Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right)}{T_{req,f}} = \frac{0.9\left(\frac{T_f}{2.25}\right)}{T_{req,f}} = \frac{0.4(T_f)}{T_{req,f}} \geq 1$
<p>Page 141, section C.3</p>	<p><b>Change</b> “design example in the ASD format contained in section 4.5.” <b>to</b> “sections of the design example in the ASD format contained in section 4.4.”</p>
<p>Page 145, section C.3.7.3</p>	<p><b>Change</b> “Applying a resistance factor (<math>\phi_{reinf}</math>) of 0.4, the factored reinforcement strength (<math>T_{f,f}</math>) is 1,920 lb/ft.” <b>to</b> “Applying the resistance and global reduction factors of 0.9 and 2.25, respectively, the factored reinforcement strength (<math>T_{f,f}</math>) is 1,920 lb/ft.”</p>

$L_{\text{block}}$	Length of a facing block [L]
$L_{\text{span}}$	Span length of the bridge [L]
$(LL + IM)_{\text{total}}$	Governing abutment reaction for the HL-93 LL model for one lane
$N_{\gamma}$	Dimensionless bearing capacity coefficient
$N_{\text{block}}$	Number of facing blocks in a column
$N_c$	Dimensionless bearing capacity coefficient
$N_{\text{lanes}}$	Number of lanes
$N_q$	Dimensionless bearing capacity coefficient
$q$	Surcharge load [ $F/L^2$ ]
$q_b$	Equivalent superstructure DL pressure [ $F/L^2$ ]
$q_{LL}$	Equivalent superstructure LL pressure [ $F/L^2$ ]
$q_n$	Bearing capacity of the foundation soil [ $F/L^2$ ]
$q_{n,\text{an}}$	Nominal ultimate load-carrying capacity of the foundation using the analytical method [ $F/L^2$ ]
$q_{n,\text{emp}}$	Nominal ultimate load-carrying capacity of the foundation using the empirical method [ $F/L^2$ ]
$q_R$	Factored bearing resistance [ $F/L^2$ ]
$q_{rb}$	Surcharge due to the structural backfill (road base) DL [ $F/L^2$ ]
$q_t$	Equivalent roadway LL surcharge [ $F/L^2$ ]
$q_{\text{ult},\text{an}}$	Ultimate load-carrying capacity of GRS using the analytical method [ $F/L^2$ ]
$q_{\text{ult},\text{emp}}$	Ultimate load-carrying capacity of GRS using the empirical method [ $F/L^2$ ]
$Q_{LL}$	LL reaction load [F]
$RF_{\text{global}}$	Global reduction factor for the geosynthetic to account for long-term strength losses due to installation damage, creep, and durability [dimensionless]
$R_n$	Nominal resisting force for direct sliding calculations [F/L]
$R_R$	Factored resisting force for direct sliding calculations [F/L]
$S_e$	Superelevation angle [deg]
$S_k$	Skew angle [deg]
$S_v$	Reinforcement spacing [L]

used as spacers to form the beam seat (see chapter 7). CMU blocks have been used for GRS construction because they are readily available and inexpensive. They are also compatible with the frictional connection to the recommended reinforcement. Since the facing element is not structural in a GRS wall or abutment, any facing element can be used. With other facing elements, however, special design considerations may apply, and such considerations are beyond the scope of this guide.

### **3.3 BACKFILL MATERIAL**

Backfill selection for GRS-IBS is important because it is a major structural component for the abutment. The backfill must be properly compacted to a minimum of 95 percent of maximum dry density according to AASHTO T-99. Other procedures to determine the degree of compaction can also be used (e.g., modulus-based test methods), as discussed in chapter 7. In GRS-IBS construction, other areas to consider for backfill selections are the RSF and the integrated approach.

Locally sourced aggregates, as long as they meet the material qualifications, are the most economical choice for GRS construction. Most State specifications for aggregate, which are usually met by local quarries and aggregate suppliers, will satisfy the material requirements. Recommendations are provided in this section for GRS abutment, RSF, and approach-way backfills.

It should be noted that some backfill materials are easier to work with than others. Certain backfills are more suitable for compacting behind a given facing element than others. These factors need to be considered when selecting the backfill for a given project.

It has been observed that some fine-grained sands and open-graded coarse aggregates with a maximum grain size greater than 2 inches are difficult to compact directly behind the face of a frictionally connected split face CMU block. The selection of a compatible fill and facing element is therefore necessary for the following purposes:

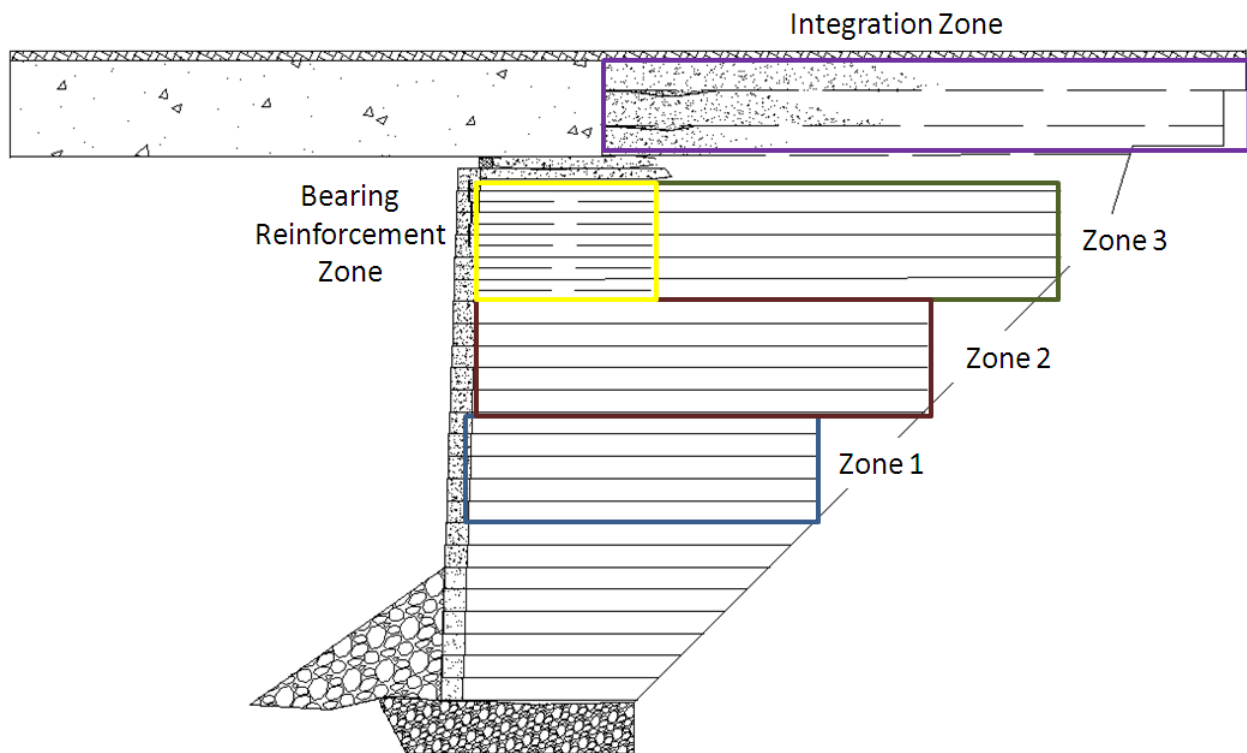
- Ensure adequate compaction directly behind the face.
- Control face alignment.
- Limit post construction lateral deformation.

#### **3.3.1 GRS Abutment Backfill**

Because a GRS abutment is designed to support load, the backfill is considered a structural component. Abutment backfill should consist of crushed, hard, durable particles or fragments of stone or gravel. These materials should be free from organic matter or deleterious material such as shale or other soft particles that have poor durability. The backfill should follow the size and quality requirements for crushed aggregate material normally used locally in the construction and maintenance of highways by Federal or State agencies.

Abutment backfill typically consists of either well-graded or open-graded aggregates (example gradations are shown in sections 3.3.1.1 and 3.3.1.2, respectively). It is recommended that either one of these gradations or a blend in between the two be used as backfill behind GRS abutments. At the time of this report, open-graded aggregates had been selected on all GRS-IBS projects due to the relative ease of construction and favorable drainage characteristics (see appendix A). If the

6. Add a bearing reinforcement zone underneath the bridge seat to support the increased loads due to the bridge (see figure 13). This bearing bed reinforcement serves as an embedded footing in the reinforced soil mass. The bearing bed reinforcement spacing directly underneath the beam seat should be, at a minimum, half the primary spacing (e.g., for an 8-inch primary spacing, the bearing bed reinforcement spacing will equal 4 inches). In general, the minimum length of the bearing bed reinforcement should be twice the setback plus the width of the bridge seat. The depth of the bearing reinforcement zone is determined based on internal stability design for required reinforcement strength (see section 4.4.7.3). At a minimum, there should be five bearing bed reinforcement layers (see figure 13).



**Figure 13. Illustration. Reinforcement schedule for a GRS abutment.**

7. Blend the reinforcement layers in the integration zone to create a smooth transition. The layers should extend to the cut slope, if applicable, with the exception of the top reinforcement layer, depending on the site. This top layer should extend beyond the cut slope to prevent moisture infiltration. The integration zone is part of the integrated approach of GRS-IBS (see figure 13). It is added behind the bridge superstructure to limit the development of a tension crack at the cut slope and reinforced soil interface and to blend the approach way on to the roadway to create a smooth transition. The number of reinforcement layers in the integration zone depends on the height of the superstructure, but each wrapped layer should be no more than 12 inches in height. Additional work is needed to integrate the substructure with the superstructure within the integration zone. This is described in chapter 7.

DL, and  $b_{rb,t}$  is the width over the GRS abutment where the road base DL acts (see figure 14). The LL on the approach pavement and the superstructure are not included as resisting forces because they are transient loads.

$$W = \gamma_r HB \quad (16)$$

Where  $\gamma_r$  is the unit weight of the reinforced fill,  $H$  is the height of the GRS abutment including the clear space distance, and  $B$  is the base width of the GRS abutment not including the wall facing.

The factor of safety against direct sliding ( $FS_{slide}$ ) is computed according to equation 17. The factor of safety must be greater than or equal to 1.5. If not, consider lengthening the reinforcement at the base. **Direct sliding should also be checked at the interface between the RSF and the foundation soils.**

$$FS_{slide} = \frac{R_n}{F_n} \geq 1.5 \quad (17)$$

#### 4.3.6.2 Bearing Capacity

To prevent bearing failure, the vertical pressure at the base of the RSF must not exceed the allowable bearing capacity of the underlying soil foundation. The vertical pressure is a result of the weight of the GRS abutment, the weight of the RSF, the bridge seat load, the LL on the superstructure, and the LL on the approach pavement. The pressure at the base ( $\sigma_{v,base,n}$ ) is calculated according to a Meyerhof-type distribution, shown in equation 18.<sup>(10)</sup>

$$\sigma_{v,base,n} = \frac{\Sigma V}{B_{RSF} - 2e_{B,n}} \quad (18)$$

Where  $\Sigma V$  is the total vertical load on the GRS abutment (calculated in equation 19),  $B_{RSF}$  is the width of the RSF, and  $e_{B,n}$  is the eccentricity of the resulting force at the base of the wall (calculated in equation 20).

$$\Sigma V = W + W_{RSF} + W_{face} + b_{rb,t}(q_t + q_{rb}) + b(q_b + q_{LL}) \quad (19)$$

Where  $W$  is the weight of the GRS abutment (equation 16),  $W_{RSF}$  is the weight of the RSF,  $W_{face}$  is the weight of the facing elements,  $q_t$  is the roadway LL,  $b_{rb,t}$  is the width of the traffic and road base load over the GRS abutment,  $q_{rb}$  is the road base surcharge,  $q_b$  is the bridge DL,  $b$  is the width of the bridge seat, and  $q_{LL}$  is the LL on the superstructure.

$$e_{B,n} = \frac{\Sigma M_D - \Sigma M_R}{\Sigma V} \quad (20)$$

Where  $\Sigma M_D$  is the total driving moment,  $\Sigma M_R$  is the total resisting moment, and  $\Sigma V$  is the total vertical load (equation 19). The moments should be calculated about the bottom and center of the RSF for the specific layout of the GRS abutment. If  $e_{B,n}$  is negative, take  $e_{B,n}$  equal to zero for the term  $B_{RSF} - 2e_{B,n}$ .

Beyond bearing capacity, consolidation settlement should be evaluated to ensure excessive deformations will not occur over the life of the bridge. Design considerations such as excavation and the RSF reduce the pressure on the foundation soil. Nevertheless, settlement of the foundation soil should be assessed as with any other spread footing according to FHWA guidance.<sup>(7)</sup> Determining the criterion for tolerable foundation settlement is left up to the engineer.

A stress history analysis should be conducted to ascertain settlement and stability prediction. Answers to the following questions will provide insight on the stress history for an efficient design:

- Is the site bridge a replacement project built in the same location?
- What was the performance of the existing bridge?
- Were there any chronic maintenance issues associated with the existing structure?
- What was the combined weight of the abutment and superstructure within the footprint of the new bridge foundation? How does that stress compare with the stress of the new structure?
- Does the site involve an excavation equivalent to the weight of the new GRS-IBS?
- Can the new bridge be built behind the existing foundation?

#### **4.3.6.3 Global Stability**

Global stability is evaluated according to classical slope stability theory using either rotational or wedge analysis. To facilitate the global stability check, it is prudent to collect accurate soil property information. Standard slope stability computer programs can then be used to assess the global and compound stability of a GRS structure. The factor of safety for global stability should equal at least 1.5.

#### **4.3.7 Step 7—Conduct Internal Stability Analysis**

The internal stability analysis will vary slightly depending on whether ASD or LRFD is the chosen design method. ASD is presented in this chapter. For guidance on LRFD, refer to [appendix C](#).

##### **4.3.7.1 Ultimate Capacity**

The ultimate vertical capacity of a GRS abutment is found either empirically or analytically. It is recommended that the ultimate capacity be found empirically if possible. A performance test should be conducted to determine the ultimate capacity if the reinforced fill is different from those used in the performance tests reported in this guide (see appendix A). Testing will provide the most accurate results for the design. If a performance test cannot be performed, the analytical method can be used to determine the ultimate capacity.

**4.3.7.1.1 Empirical Method:** Empirically, the results of an applicable performance test using the same geosynthetic reinforcement and compacted granular backfill as planned for the site should be used. The *ultimate vertical capacity* in this case is defined as the stress at which the performance

$$\sigma_{h,t} = q_t K_{ar} \quad (35)$$

Where  $q_b$ ,  $q_{rb}$ ,  $q_t$ , and  $q_{LL}$  are the bridge DL, road base DL, roadway LL, and bridge LL surcharges, respectively, and  $\alpha_b$  and  $\beta_b$  are the angles shown in **Error! Reference source not found.**, found using equation 36 and equation 37, respectively.

$$\alpha_b = \tan^{-1} \left( \frac{b}{2z} \right) - \beta_b \quad (36)$$

$$\beta_b = \tan^{-1} \left( \frac{-b}{2z} \right) \quad (37)$$

The required reinforcement strength ( $T_{req}$ ) must satisfy two criteria: (1) it must be less than the allowable reinforcement strength ( $T_{allow}$ ), and (2) it must be less than the strength at 2 percent reinforcement strain ( $T_{@ \epsilon=2\%}$ ) **in the direction perpendicular to the abutment wall face.**

In design, a minimum value of the ultimate reinforcement strength ( $T_{allow}$ ) is needed to ensure adequate ductility and satisfactory long-term performance. In addition, it is prudent to specify the resistance required at the working load ( $T_{@ \epsilon=2\%}$ ) to ensure satisfactory performance under the in-service condition.

For abutments, a minimum ultimate tensile strength ( $T_f$ ) of 4,800 lb/ft is required. The allowable reinforcement strength ( $T_{allow}$ ) is found by applying a factor of safety for reinforcement strength ( $FS_{reinf}$ ) of 3.5 to the ultimate strength (see equation 38). The required reinforcement strength ( $T_{req}$ ) must be less than  $T_{allow}$ .

$$T_{allow} = \frac{T_f}{FS_{reinf}} = \frac{T_f}{3.5} \quad (38)$$

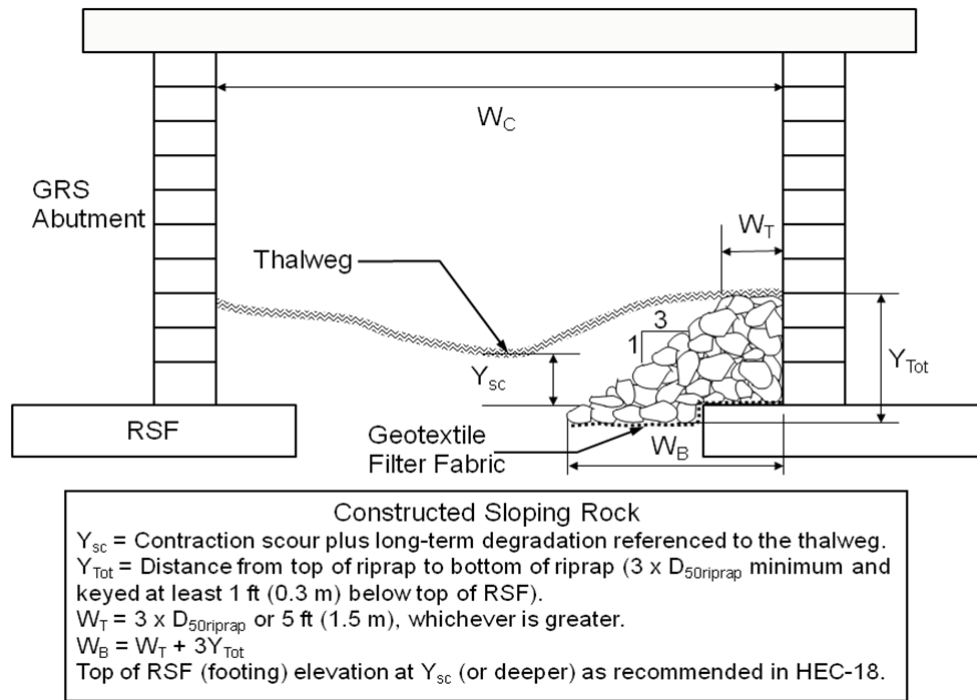
Since geosynthetic reinforcements of similar strength can have rather different load-deformation relationships depending on the manufacturing process and the polymer used, it is important that  $T_{req}$  be less than the strength at 2 percent reinforcement strain. The strength of the reinforcement at 2 percent ( $T_{@ \epsilon=2\%}$ ) is often given by the geosynthetic manufacturer. If  $T_{req}$  is greater than  $T_{@ \epsilon=2\%}$ , a different geosynthetic must be chosen, the ultimate strength must be increased, or the reinforcement spacing must be decreased.

While the strength of the reinforcement can theoretically vary along the height of the GRS abutment, it is recommended that only one strength of reinforcement be used throughout the entire abutment. This simplifies the construction process and avoids placement errors for the reinforcement.

**4.3.7.3.1 Depth of Bearing Bed Reinforcement:** The required reinforcement strength ( $T_{req}$ ) is found at each 8-inch primary spacing layer. If  $T_{req}$  is greater than the allowable reinforcement strength ( $T_{allow}$ ) or the strength at 2 percent strain ( $T_{@ \epsilon=2\%}$ ), then the reinforcement spacing must be reduced to 4 inches to the depth at which  $T_{req}$  is less than  $T_{allow}$  or  $T_{@ \epsilon=2\%}$ . This depth is termed the *bearing reinforcement bed*. The minimum required depth is five courses of block.



- Scour countermeasures:** When scour depth is calculated as described in this section, a designed scour countermeasure is included. Design scour countermeasures include riprap aprons, gabion mattresses, and articulating concrete blocks ~~(see section 4.5.3)~~. The purpose of installing a designed scour countermeasure is to prevent loss of soil from underneath a GRS abutment from scour that occurs at or near the abutment. Soil loss can reduce bearing capacity or lead to settlement, which can cause structural failure ~~(see section 4.5.3)~~. Figure 36 shows a cross section of a typical abutment riprap countermeasure recommended for smaller, more culvertlike structures (flow length through structure is longer than structure width). See HEC-23 for additional details regarding the specific requirements for the design and configuration of this countermeasure.<sup>(5)</sup> Larger, more bridgelike structures (opening length is greater than the flow distance through the structure) must be evaluated for scour using the procedures outlined in HEC-18 and HEC-20 and use a designed countermeasure as outlined in HEC-23.<sup>(14,15,5)</sup>



Source: HEC-23 Figure 18-10

**Figure 36. Illustration. Typical cross section for sloping rock (adapted).<sup>(5)</sup>**

- Inspection:** After construction, scour countermeasure condition and channel instability should be assessed during each regular bridge inspection and after extreme flood events. Any countermeasure failure or significant change in channel condition should be noted and scheduled for repair or stabilization. Without proper inspection and maintenance, a scour countermeasure may fail or a channel may become unstable, which can lead to undermining of an abutment. The FHWA's HEC-20 discusses approaches for evaluating channel instability, and HEC-23 discusses approaches for inspection and monitoring the effects of scour.<sup>(15,5)</sup>

Another hydraulic consideration is drainage. The potential for unbalanced water pressure exists when a wall can become partially submerged by a flood or when surface drainage is not controlled. All GRS structures should include consideration for surface and subsurface drainage. Critical areas are behind the wall at the interface between the GRS mass and the retained fill, at the base of the wall, and any location where a fill slope meets the wall face. For example, the design needs to include provisions for surface drainage along the fill slope adjacent to the wing walls. [Section 7.11](#) discusses drainage details.

## **5.4 SEISMIC DESIGN**

External stability for seismic design will need to be checked for GRS-IBS just like with any other gravity structure. Design considerations for external stability and seismicity include increasing the base width of the wall and increasing the length of the reinforcement at the top of the wall. Additional bearing capacity and overall external stability is generally improved by increasing the base width of the wall. Additional stability is created by increasing the length of the reinforcement at the top of the wall or abutment. This integrated approach has also been shown to be beneficial because it keys the structure into the existing terrain, preventing the development of a failure plane along the cut slope, which can lead to progressive failure.

No seismic design requirements are necessary for the internal stability of GRS-IBS. Reinforced soil walls have been known to perform better than conventional retaining walls under seismic loading, as evidenced by observations of actual performance in strong earthquake events. (See references 16–19.) A National Cooperative Highway Research Program (NCHRP) study is being conducted to establish guidelines for the design and construction of GRS abutments under seismic loading.<sup>(20)</sup> As part of the NCHRP study, a 12-ft-high GRS abutment supporting a bridge load of about 1,000 kips was subject to sinusoidal motions on a shake table. No significant damage or movement was recorded until the acceleration was increased to 1.0 g, at which time base sliding between the GRS abutment and foundation soil became apparent. The superstructure would not have failed due to the deformation of the GRS mass at the 1.0-g acceleration. This experiment suggests that a GRS abutment is capable of withstanding at least low to medium earthquakes without any special provisions.

## **5.5 IMPACT EVENTS**

There is limited information on vehicle impact against GRS-IBS. Typically, GRS walls along roadways are built behind a crash barrier. A niche function of GRS technology, however, is rock-fall protection. That application is not covered in this manual, but it serves to show that GRS is capable of withstanding considerable lateral and vertical impacts without failure or loss of serviceability.

Since the facing elements are not rigidly connected to the reinforcement, hand-operated compaction equipment (e.g., a lightweight mechanical tamper, plate, or roller) is required within 1.5 ft of the front of the wall face. It is very important for adequate GRS performance that the backfill is properly compacted. The top 5 ft of the abutment should be compacted to 100 percent of the maximum density according to AASHTO T-99.

Onsite compaction equipment should be selected to achieve the required density of the fill materials. Considering that compaction is critical to the success of the project, compaction equipment should be in good operating order for efficient use. In addition, backup equipment should be available to provide quality construction throughout the project and to avoid construction delays.

### **7.5.1 Compaction Procedure**

Once fill is placed at the required thickness and graded, all areas behind the CMU block should be compacted to the required density. Any depression behind the facing block should be filled level to the top of the CMU block prior to compaction.

Compaction directly behind the CMU block should be performed in a manner that maintains wall alignment while improving the density of fill behind the block. This can be achieved in the following ways:

- Placing a fill lift directly behind the CMU block face and rodding or foot tamping along the row of CMU block while exerting downward pressure on the block to prevent lateral movement. For multiple lifts, the top lift height is slightly higher than the block to compensate for compression of the fill during compaction.
- Using a lightweight vibratory plate compactor directly behind the CMU block while exerting downward pressure on the block to prevent lateral movement.
- Using larger vibratory compactors for the remainder of the fill area 3 ft from the face of the GRS wall. Check for outward block movement and adjust accordingly.

The most common compaction QC tool is the nuclear density gauge. Other instruments are also available for compaction control such as the Clegg hammer, the soil stiffness gauge, or the falling weight deflectometer. These devices are typically used by correlating their measurements to soil density and moisture content. Method-based compaction specifications can also be used. For open-graded fills, compact to non-movement or no appreciable displacement and assess with visual inspection.

## **7.6 REINFORCEMENT**

Generally, the length of the reinforcement layers will follow the cut slope, as shown in figure 50. While the reinforcement layers in the GRS abutment can be any geosynthetic, the RSF and integrated approach should be constructed and encapsulated with a geotextile to confine the compacted granular fill. The geosynthetic should be placed so that the strongest direction is perpendicular to the abutment face, as shown in figure 51. Where the roll ends, the next roll should begin. **Overlapping between sheets is not required.** The geosynthetic reinforcement should extend between layers of CMU block to provide a frictional connection. The geosynthetic reinforcement

After the geosynthetic is rolled out, it should be laid so that it is taut, free of wrinkles, and flat. The geosynthetic can be held in place with the fill. Placement of fill should be from the wall face backward to remove and prevent the formation of wrinkles in the geosynthetic. A conscious effort should be taken during placement of fill to prevent the development of wrinkles.

Splices of reinforcement can occur without overlap. Splice seams should be staggered to avoid a continuous break in the reinforcement throughout the GRS structure. All splice seams should run perpendicular to the wall face.

Overlaps of adjacent geosynthetic should be trimmed where they are in contact with the surface of the CMU block to avoid varying geosynthetic thicknesses between the CMU block. Any seams in the geosynthetic should be staggered with each successive layer of the GRS abutment. All seams between adjacent sheets of geosynthetic located in the area beneath the footprint of the bridge seat should be perpendicular to the abutment wall face.

### **7.6.1 Operating Equipment on Geosynthetic Reinforcement**

Driving should not be allowed directly on the geosynthetic reinforcement. Place a minimum 6-inch layer of granular fill prior to operating any vehicles or equipment over the geosynthetic reinforcement. **In the bearing reinforcement zone, hand-operated compaction equipment should be used over the 4-inch lifts to prevent excessive installation damage of the reinforcement.** Rubber-tired equipment may pass over the geosynthetic reinforcement at speeds less than 5 mi/h. Skid steers and tracked vehicles can cause considerable damage to the geosynthetic. On one occasion, a track hoe operating on a GRS structure turned and pulled the fabric causing deformation to the wall face. For this reason, it is recommended to restrict the use of these vehicles on GRS structures. If absolutely necessary, use may be permitted provided no sudden braking or sharp turning occur and a minimum 6-inch cover is placed.

### **7.6.2 Bearing Reinforcement Bed**

The bearing reinforcement bed provides additional strength in the upper GRS wall layers directly beneath the bearing area of the superstructure. These reinforcement layers are not sandwiched between two consecutive rows of block but are placed behind the CMU block at 4-inch spacing. This 4-inch reinforcement spacing is generally placed in the top five layers of the GRS abutment or as determined by design (see chapter 4).

Bearing bed reinforcement spacing in superelevated abutment walls requires additional planning. The 4-inch reinforcement spacing needs to be in place for the top five courses of block at the lowest elevation across the abutment wall (see figure 50 and figure 52). The reinforcement schedule will guide field personnel in the proper placement of the geosynthetic along a wall block course.

Once these steps are accomplished, the GRS-IBS can be constructed. The basic design guidelines are the same whether using ASD or LRFD. However, the detailed equations within step 6 and step 7 will differ between the two design methods. In this appendix, only the differences in step 6 and step 7 that result from conversion to the LRFD format are presented. Refer to [section 4.3](#) for discussion on each of these design elements and the equivalent ASD equations and to [section 4.4](#) for the ASD calculation.

### C.2.1 Step 6—Conduct an External Stability Analysis

The external stability of a GRS-IBS is evaluated by looking at the following potential external failure mechanisms:

- Direct sliding.
- Bearing capacity.
- Global stability.

#### C.2.1.1 Direct Sliding

The total factored driving force for LRFD ( $F_R$ ) is calculated in much the same way as in ASD ( $F_n$ ) except load factors are applied to each component of thrust force. Equation 70 modifies equation 13 to include the load factors  $\gamma_{EH\ MAX}$ ,  $\gamma_{ES\ MAX}$ , and  $\gamma_{LS}$ , which are determined using table 16 and table 17

$$F_R = \gamma_{EH\ MAX} F_b + \gamma_{ES\ MAX} F_{rb} + \gamma_{LS} F_t \quad (70)$$

The factored resisting force ( $R_R$ ) is calculated using equation 71. This equation is the LRFD modification of equation 14 that includes a shear resistance factor ( $\Phi_\tau$ ). For sliding,  $\Phi_\tau$  is equal to 1.0.<sup>(9)</sup>

$$R_R = \Phi_\tau (W_{t,R} \mu) \quad (71)$$

Where  $W_{t,R}$  is determined using equation 72, which is equation 15 modified to include the appropriate load factors,  $\gamma_{EV\ MIN}$ ,  $\gamma_{DC\ MIN}$  and  $\gamma_{DC\ MIN}$ , from table 17.

$$W_{t,R} = \gamma_{EV\ MIN} W + \gamma_{DC\ MIN} (q_b b) + \gamma_{ES\ MIN} (q_{rb} b_{rb,t}) \quad (72)$$

For LRFD, the ratio of the factored resistance and the factored driving force must be greater than or equal to 1.0 (see equation 73). If not, consider lengthening the reinforcement at the base.

$$\frac{R_R}{F_R} \geq 1.0 \quad (73)$$

#### C.2.1.2 Bearing Capacity

In this section, the ASD equations to evaluate bearing capacity have been modified to include the appropriate load and resistance factors of LRFD. Equation 74 is the LRFD version of equation 18.

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b \quad (91)$$

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right) \quad (92)$$

For abutments, a minimum wide width tensile strength ( $T_f$ ) of 4,800 lb/ft is required. In addition to a global reduction factor of 2.25 accounting for long-term strength losses ( $RF_{global}$ ) of the geosynthetic, a resistance factor for reinforcement strength ( $\Phi_{reinf}$ ) of 0.9 should be applied to the ultimate strength ( $T_f$ ) to determine the factored reinforcement strength ( $T_{f,f}$ ). The factored required reinforcement strength ( $T_{req,f}$ ) must be less than this factored reinforcement strength ( $T_{f,f}$ ), as shown in equation 93.

$$\frac{T_{f,f}}{T_{req,f}} = \frac{\Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right)}{T_{req,f}} = \frac{0.9\left(\frac{T_f}{2.25}\right)}{T_{req,f}} = \frac{0.4(T_f)}{T_{req,f}} \geq 1 \quad (93)$$

Since geosynthetic reinforcements of similar strength can have rather different load-deformation relationships depending on their material, it is important that the nominal (unfactored)  $T_{req}$  be less than the strength at 2 percent reinforcement strain. The strength of the reinforcement at 2 percent ( $T_{@ \varepsilon=2\%}$ ) is often given by the geosynthetic manufacturer. If the unfactored  $T_{req}$  is greater than  $T_{@ \varepsilon=2\%}$ , either a different geosynthetic must be chosen or the ultimate strength must be increased.

### C.3 DESIGN EXAMPLE (LRFD): BOWMAN ROAD BRIDGE, DEFIANCE COUNTY, OH

In this section, the equations formatted for the LRFD method in section C.2 are demonstrated. For additional details and discussion in support of these calculations, see the corresponding sections of the design example in the ASD format contained in section 4.4.

#### C.3.6 Step 6—Conduct an External Stability Analysis

##### C.3.6.1 Direct Sliding

The driving forces on the GRS abutment are comprised of the lateral forces due to the retained backfill, the road base and the traffic surcharge. The force due to the backfill is calculated in equation 94.

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 = \frac{1}{2}(120)(0.361)(15.58)^2 = 5258 \text{ lb/ft} \quad (94)$$

The lateral force due to the road base and traffic surcharges are calculated in equation 95 and equation 96.

$$F_{rb} = q_{rb} K_{ab} H = 385(0.361)(15.58) = 2165 \text{ lb/ft} \quad (95)$$

$$F_t = q_t K_{ab} H = 298(0.361)(15.58) = 1676 \text{ lb/ft} \quad (96)$$

The vertical deformation is the product of the vertical strain and the height of the GRS mass and is calculated in equation 112.

$$D_v = \varepsilon_v H = 0.003(15.58) = 0.047 \text{ ft} \quad (112)$$

**C.3.7.2.2 Lateral Deformation:** The lateral strain and deformation are found in equation 113 and equation 114.

$$\varepsilon_L = 2\varepsilon_v = 2(0.3\%) = 0.6\% \quad (113)$$

$$D_L = \frac{2D_v}{H}(b + a_b) = \frac{2(0.047)}{15.58}(4 + 0.67) = 0.028 \text{ ft} \quad (114)$$

### C.3.7.3 Required Reinforcement Strength

The strength of the reinforcement used at Bowman Road Bridge is 4,800 lb/ft. **Applying the resistance and global reduction factors of 0.9 and 2.25, respectively**, the factored reinforcement strength ( $T_{f,f}$ ) is 1,920 lb/ft. According to the manufacturer,  $T_{@ \varepsilon=2\%}$  is equal to 1,370 lb/ft. The maximum required reinforcement strength is found as a function of depth, as shown in equation 115.

$$T_{req,f} = \left[ \frac{\sigma_{h,f} - \sigma_c}{0.7 \left( \frac{S_v}{6d_{max}} \right)} \right] S_v \quad (115)$$

The factored lateral stress ( $\sigma_{h,f}$ ) is a combination of the factored lateral stresses due to the road base DL ( $\sigma_{h,rb,f}$ ), the roadway LL ( $\sigma_{h,t,f}$ ), the GRS reinforced soil ( $\sigma_{h,w,f}$ ), and an equivalent bridge load ( $\sigma_{h,bridge,f}$ ). To simplify calculations, the roadway LL and road base DL can be extended across the abutment. The vertical components of these loads are then subtracted from the bridge DL and LL, giving an equivalent bridge load. The lateral stresses due to the equivalent bridge load are then calculated according to Boussinesq theory. The lateral stress is calculated for each depth of interest (each layer of reinforcement). All lateral stresses are calculated in **Error! Reference source not found.**

An example calculation for the required reinforcement strength at a depth ( $z$ ) of 5.3 ft (the eighth reinforcement layer from the top) is shown in equation 116. First, the lateral pressure must be found. Remember, the location of interest is directly under the centerline of the bridge load (where  $x = 0.5b = 0.5(4\text{ft}) = 2 \text{ ft}$ ).

$$\sigma_{h,f} = \sigma_{h,w,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f} = 117 + 297 + 85 + 77 = 576 \frac{\text{lb}}{\text{ft}^2} \quad (116)$$

Where the lateral pressure is found using equation 117 through equation 120.

$$\sigma_{h,w,f} = \gamma_{EH \text{ MAX}}(\gamma_r z K_{ar}) = 1.35 \left[ 110(5) \left( \frac{1 - \sin(48\text{deg})}{1 + \sin(48\text{deg})} \right) \right] = 117 \frac{\text{lb}}{\text{ft}^2} \quad (117)$$