Effect of reinforcement spacing on the performance of bridge abutment by scaled model test

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ABSTRACT

Recently, construction of GRS (Geosynthetic reinforced soil) structures with closely spaced layers of geosynthetic reinforcement has been increased significantly. In order to achieve direct comparison between GRS structures and the conventional mechanically stabilized earth structures using geosynthetic (GMSE) under the same working condition, three model tests with a bridge abutment as a prototype were designed in this research. Under the vertical load of 100 kPa in simulating the weight of the bridge deck, important parameters such as stresses and deformations of the model abutment were measured, and the effect of reinforcement spacing on the performance of different reinforced soil structures was evaluated. According the test results, the overall better performance of GRS abutment was proved in terms of measured strains and stresses. In addition, comparisons were made between the measurements and the analytical predictions following solutions available in literature, and the accuracy as well as limitations of different calculation methods was discussed.

Keywords: reinforced soil, geosynthetic reinforced soil (GRS), mechanically stabilized earth (MSE), model test, bridge abutment

1 INTRODUCTION

Geosynthetic-reinforced soil (GRS) refers to alternating layers of compacted granular fill reinforced with closely-spaced geosynthetic reinforcements (i.e., spacing equal or less than 0.3 m) (Adams et al., 2012). Mechanically stabilized earth using geosynthetic (GMSE) is the reinforced soil with larger reinforcement spacing, which is commonly seen in reinforced structures such as retaining walls, railway subgrades, slopes, etc. According to the research of Adams et al. (2007a; 2007b), Keller and Devin (2003), and Saghebfar et al. (2017), compared with GMSE, GRS shows advantages in higher strength, lower lateral deformation and tolerance of differential settlements due to its smaller reinforcement spacing and better compaction of backfill material. Recently, usage of GRS technology in bridge-supporting structures have been extensively increased, particularly for small single-span bridges (Adams et al., 2011).

A primary difference between GRS mass and GMSE mass lies in the load carrying mechanism, which also leads to the difference in mechanical responses and deformation characteristics. Plenty of previous research has been focused on the characteristics of GRS and GMSE by means of laboratory experimental study regarding to bearing capacities, vertical compression, and lateral deformation (Wu et al., 2011; Adams et al., 2012; Benjamim et al., 2015). However, direct comparison between the two types of reinforced soil structures under the same loading condition is still limited. In particular, fewer attention has been paid to the distribution of additional vertical stress induced by surcharge loads (e.g., concentrated load by bridge deck), which directly affects calculations of the lateral earth pressure and reinforcement force.

In this paper, a series of scaled model tests were designed in order to provide insight into the mechanical responses and deformation characteristics of both the GRS and GMSE abutment based on the similarity theory with a contraction ratio of 1/2. Different reinforcement spacing were applied in the model abutment to simulate the corresponding reinforced soil structures. Important parameters such as stresses and deformations of the model abutment were measured and verified against analytical calculation following several design methods.

2 SCALE MODEL TEST

2.1 Model abutment

The geometry of a bridge footing on top of an abutment is typically considered as a plane strain problem since the ratio of the length of the footing to its width is commonly between 7 and 14 (Adams et al.,
Therefore, tests conducted in this study simulated the model abutment in a plane strain condition. Geometry of the model is presented in Fig. 1. The abutment had dimensions of 220 cm (length) \times 150 cm (height) \times 100 cm (width), which was constructed inside a test box made of reinforced steel. Plastic sheets and lubricating oil were applied on both side walls of the test box. The similarity ratio to the prototype in length is chosen as 1/2, and as 1 for the packing density, shown in Table 1.

![Fig. 1. Schematic diagram of the test apparatus.](image)

| Variable        | Scaling Factor | Scaling Factor for \( \lambda = 2 \) |
|-----------------|----------------|---------------------------------|
| Length          | \( \lambda \)  | 2                               |
| Material density| 1              | 1                               |
| Stress          | \( \lambda \)  | 2                               |
| Strain          | 1              | 1                               |
| Modulus         | \( \lambda \)  | 2                               |
| Stiffness       | \( \lambda^2 \) | 4                               |

### 2.2 Material

The maximum and minimum diameter of the granular backfill were 4.8 mm and 0.25 mm respectively. The mean particle size \( D_{50} \) of the sand was 1.9 mm. The uniformity coefficient and the coefficient of curvature are 4.21 and 0.47, respectively. The maximum and the minimum dry unit weight are 18.5 kN/m\(^3\) and 14.9 kN/m\(^3\). During construction, the sand was compacted with a small hand operated compaction equipment layer by layer to reach the density \( \gamma_d \) of 1760 kg/m\(^3\), corresponding to a relatively density \( D_r \) of 80%. Based on large-size direct shear test, an angle of internal friction of 40° and cohesion of 0 kPa were obtained for the sand.

The geosynthetic used in the model test was a biaxial geogrid made of polyethylene with an ultimate tensile strength of 24.8 kN/m and tensile strength at 2% strain of 7.3 kN/m. The rib thickness and width of the geogrid were 4 mm and 5 mm respectively, and the aperture sizes of the geogrid were 40 mm \times 40 mm.

As shown in Fig. 2, modular blocks (geo-gabions) with different dimension were used as facing elements so as to adapt to different reinforcement spacing, which were 100 cm (length) \times 10 cm (width) \times 10 cm (height) and 100 cm (length) \times 10 cm (width) \times 15 cm (height).

![Fig. 2. Geo-gabions used as facing elements.](image)

### 2.3 Testing program

The commonly used vertical reinforcement spacing \( S_v \) in GRS abutment is 0.2 m or 0.3 m (Bueno et al., 2005), while for GMSE structures it is 0.6 m (Han et al., 2017). Thus, \( S_v \) was selected as 0.1 m, 0.15 m and 0.3 m respectively in the current study, given the length similarity ratio of 2. In other words, the first two test cases (named after Case A and Case B) are to simulate the GRS structure and the last one (named after Case C) is to simulate GMSE structure.

During construction of the model abutment, each geo-gabion was be placed tightly against the adjoining one, preventing gaps from which fill material might escape. For the two cases in modeling GRS abutment, frictional connection was applied between the reinforcement layer and facing blocks. That was, the geogrid was directly embedded in between the adjacent gabions. For the test case in modeling GMSE abutment, the geogrid was tied and fixed by lead wire onto the gabions to achieve the mechanical connection. Throughout the model preparation, the facing units were supported by the horizontal fixities to ensure the verticality of the facing system. And the fixities were removed after the model abutment was completed.

In reality, the load of bridge deck is imposed on the abutment instantly. Similarly, the loading of 100kPa was applied through the load plate at one time after completing the model construction.

### 2.4 Instrumentation

The primary measurements included the vertical settlements of the abutment surface, horizontal deformations of the facing wall, distribution of additional vertical stresses induced by footing load, and the distribution of strains along the geosynthetic reinforcements. Correspondingly, three different types of instrumentations were used: linear variable differential transformers (LVDT), earth pressure cells.
(epc), and strain gauges.

As shown in Fig.3, six LVDTs were employed, with one pair on the top of loading plate and the other four near the face wall (30, 75, 105, 135 cm from the bottom). A total of five earth pressure were installed underneath the loading plate along its centerline at different depths (15, 45, 75, 105, 135 cm from the bottom). Strain gauges were installed uniformly on four reinforcement layers at different depths (30, 60, 90, 120 cm from the bottom) to measure the developed strains along reinforcement layers. The segment length between two adjacent strain gauges at the same height was 0.4 m, and the left one is placed at the location of 0.1 m behind the back of facing wall. All the LVDTs, strain gauges and earth pressure cells were initialized to zero after the construction prior to loading.

Fig. 3. Test monitoring program.

3 RESULTS AND DISCUSSIONS

3.1 Deformations

Vertical settlement $D_v$ of the model abutment was obtained by averaging the data from the two LVDTs placed upon the loading plate. According to the test results, $D_v$ under the static load of 100 kPa for three test cases were 8.67 mm, 17.92 mm and 23.15 mm respectively, corresponding to the vertical strain of 0.58%, 1.19% and 1.54%. It could be inferred that vertical settlement was directly influenced by reinforcement spacing. With smaller value of $S_v$, the vertical settlement of the abutment would also be smaller.

![Fig. 4. Distribution of lateral displacement along height.](image)

Fig. 4 shows the lateral displacement $D_L$ along the wall height in three test cases. $z$ represents the distance from the measured point to the bottom of test model, as shown in Fig.3. A reduction in the magnitude of $D_L$ could be observed from upper of the wall to the bottom. Compared with Case A and Case B, an obvious larger lateral displacement was found in Case C, indicating the efficient restraint effect from the closely spaced reinforcement.

The above results proved that both the vertical settlement and lateral displacement were much smaller in GRS masses than in GMSE structure. Wu (1994) and Adams et al. (2012) proposed the methods to estimate the maximum lateral displacement of facing wall, as is shown in Equation 1 and Equation 2.

$$D_{L_{\text{max}}} = \varepsilon_d \left( H \frac{1.25}{b_{\text{q,vol}}} \right)$$

$$D_{L_{\text{max}}} = \frac{2b_{\text{q,vol}}D_v}{H}$$

Here $D_{L_{\text{max}}}$ is the maximum lateral displacement (mm), $\varepsilon_d$ is the maximum reinforcement strain, $H$ is the wall height (m), $b_{\text{q,vol}}$ is the width of the load along the top of the wall including the setback.

As is known that Wu's method is a semi-empirical relationship derived from related monitoring and experimental data, while the method by Adams conservatively considers a zero volume change in the GRS mass, in which the lateral deformation is assumed as a triangular shape and the vertical deformation is assumed uniform. The comparison in Table 2 between the measured values and the calculated values of lateral displacement showed that in most cases, the analytical solutions underestimated the lateral movement of facing wall. Compared with Adams’s method, differences between measurement and calculated values were more significant in Wu’s method. Additionally, both Adams's and Wu’s method provided closer results for case A and case B in representing GRS abutment than for case C of GMSE abutment.

![Table 2. Comparison between the theoretical value and measured value of the maximum lateral displacement.](image)

| Test case | Measured value (mm) | Theoretical value (mm) | Error (%) |
|-----------|---------------------|------------------------|-----------|
|           | Adams               | Wu                     | Adams     | Wu       |
| A         | 5.49                | 4.62                   | 4.50      | -15.8    | -18.0    |
| B         | 8.59                | 9.56                   | 6.95      | 11.3     | -19.1    |
| C         | 19.35               | 12.35                  | 9.00      | -36.2    | -53.5    |

3.2 Additional vertical stresses

Fig.5 shows the distribution of measured additional vertical pressure at different elevations under the applied
Pressure of 100 kPa. It was observed that pressure cells located at deeper layers showed significantly lower values than the pressure cells close to the loading plate. Difference among three test cases was found to be small.

The measured additional vertical stress induced by footing load was further compared to the calculated values. Two solutions were employed here. One is Boussinesq’s solution recommended by Adams et al. (2012). The other is proposed by the American Association of State Highway and Transportation Officials (AASHTO), in which the additional vertical stress is simplified as a truncated uniform vertical distribution of 2 vertical to 1 horizontal (2 to 1) (AASHTO, 2014).

Equations 3 to 6 show the detailed calculation of additional vertical stress induced by footing load using the simplified truncated 2 to 1 distribution method:

\[ \Delta \sigma_z = \frac{q}{D_1} \]  
\[ D_1 = b + z, \quad z \leq z_{cr} \]  
\[ D_1 = a_b + b + \frac{z}{2}, \quad z > z_{cr} \]  
\[ z_{cr} = 2a_b \]

where \( q \) is the applied footing load (kPa); \( D_1 \) is the effective width of the applied load at any depth (m); \( b \) is the footing width (m, \( b = 0.3 \) m in this study); \( z \) is the calculating depth below the top of the abutment (m); \( z_{cr} \) is the depth where the effective width intersects the back of facing wall (m); \( a_b \) is the distance from the back of facing wall to the front edge of the loading plate (m, \( a_b = 0.1 \) m in this study).

The calculated results using both Boussinesq’s solution and AASHTO method were also plotted in Fig. 5. In general, values calculated under both Boussinesq’s solution and AASHTO method were in reasonable agreement with test results. Particularly, AASHTO method provided closer value in comparison to the measured value than Boussinesq’s solution at upper layers of the abutment. It was worthy to notice that within the depth ranging from 0 to 30 cm (i.e., the depth of \( 1b \) where \( b \) is the footing width), larger vertical stress was predicted by Boussinesq’s solution than AASHTO method. However, below this depth, Boussinesq’s solution was more conservative. Similar conclusions has also been mentioned by Salgado (2008) that AASHTO method underestimated the vertical stress within the vertical depth ranging between 0 to 1.5\( b \) below the load while overestimated it at a greater depth. The fact that the 2 to 1 distribution was truncated by the back of wall facing in this study may result in the difference of the critical depth.

3.3 Reinforcement strains

A total of 14 strain gauges installed on four different layers of geosynthetics allows to measure the distribution of the tensile strain, which also helps to quantify the mobilized tensile reinforcement forces.
Fig. 6 presents strain distribution along the reinforcements at different layers. It could be found that the magnitude and distribution of strain in reinforcement layers varied in depth of the abutment. Magnitude of the measured strain decreased with the increase of the depth, indicating that the geogrid further from the loading plate was less affected.

By comparing the strains at the same depth among three test cases, it was observed that in most conditions Case A with the smallest reinforcement spacing showed the smallest magnitude of strain, especially in the area away from the facing wall. In all the four reinforcement layers, distribution patterns in Case A and Case B were similar, while larger variations in strain were found in Case C, indicating the different performance of reinforcement in the two types of model abutment as a result of the change in reinforcement spacing.

### 3.4 Reinforcement tensile forces

Based on the strain measurement along the reinforcement layers mentioned above and the elastic modulus of the reinforcement material, the tensile forces developed in the geosynthetic reinforcement can be estimated. For GRS structures, FHWA (Adams et al., 2012) suggests a semi-empirical equation for calculating the tensile force of reinforcement, as shown in Equation 7.

\[
T_r = \left[ \frac{\sigma_h}{0.7} \right] S_v
\]

(7)

in which \(T_r\) is the reinforcement tensile force (kN/m), \(S_v\) is the reinforcement spacing (m), \(d_{\text{max}}\) is the maximum grain size of backfill (m), and \(\sigma_h\) is the total lateral stress within the GRS abutment at a given depth and location (kPa, calculated in Equation 8).

\[
\sigma_h = \sigma_{h,w} + \Delta\sigma_{h,q}
\]

(8)

where \(\sigma_{h,w}\) is the lateral earth pressure using Rankine’s active stress condition (kPa), \(\Delta\sigma_{h,q}\) is the additional lateral stress induced by surcharge loads (bridge load, roadway, road base), calculated according to Boussinesq theory (kPa).

For GMSE structure, the maximum horizontal force acting on the reinforcements, \(T_{\text{max}}\), is determined by AASHTO method following Equation 9 and Equation 10 (AASHTO, 2014).

\[
T_{\text{max}} = \sigma_{H_{\text{max}}} S_v
\]

(9)

\[
\sigma_{H_{\text{max}}} = \sigma_v k_v + \Delta\sigma_v k_v + \Delta\sigma_H
\]

(10)

Here, \(\sigma_{H_{\text{max}}}\) is the maximum horizontal stress at the layer \(i\) (kPa); \(\sigma_v\) is the vertical soil stress due to self-weight (kPa); \(\Delta\sigma_v\) is the additional vertical soil stress due to footing load (kPa); \(k_v\) is lateral earth pressure coefficient, same value with the active earth pressure coefficient \(k_a\); \(\Delta\sigma_H\) is the other additional lateral pressure (kPa), equals to 0.

![Graph of calculated vs measured values](chart.png)

Fig. 7 Comparison between the calculated value and measured value of the maximum reinforcement forces.

Fig. 7 presents the results of the tensile force of the geogrid, including the measured value and the calculated value by FHWA and AASHTO method. It could be observed that the deviation between the calculated value by FHWA method and the measured value became more obvious with the increase of reinforcement spacing. On the other hand, results by AASHTO method were much closer to the measured value, demonstrating a higher reliability in reinforcement tensile force calculation for both GRS and GMSE structures.

As is shown in the empirical expression in Equation 7, it is noted that the calculated results by FHWA method is affected by the magnitude of the maximum particle size. In order to clarify this issue, relationship between the calculated tensile force and the maximum particle size under \(\sigma_h\) equaling 20 kPa was plotted in Fig. 8. It was observed that a slight change in \(d_{\text{max}}\) might cause...
large variation in the calculated value of axial force in reinforcement, especially for the smaller $d_{\text{max}}$ (less than 4 mm) and larger reinforcement spacing ($S_v$ larger than 30 cm). Therefore, the applicability of FHWA method is limited to specific condition in packing gradation and reinforcement spacing.

Fig. 8. Variation of tensile force along geosynthetics with the change of $D_{\text{max}}$ under different reinforcement spacing

4 CONCLUSIONS

In this research, three model tests were performed to investigate the influence of reinforcement spacing on the behavior of GRS and GMSE abutment under the same loading condition. Comparisons were made between the measured results and the calculated ones following the design specifications and other researchers’ method. Conclusions from the current study can be summarized as follows.

1. The measurements of vertical settlement of the model abutment and horizontal deformation of the facing wall indicated that the restraint effect in GRS mass was obviously increased due to the closely-spaced geosynthetic material.

2. The magnitude and distribution of strain in reinforcement layers varied with the reinforcement spacing and depth of the abutment. Distribution of local strain along reinforcement in abutment with smaller reinforcement spacing was more uniform, indicating a more obvious composite characteristics.

3. Measured additional vertical stress induced by footing load did not change significantly by the reinforcement spacing. In addition, the measured values compared well with the predicted stresses by using Boussinesq’s solution and AASHTO method.

4. In the analysis of reinforcement tensile force, results by AASHTO method were in good agreement with the measured value for both GRS and GMSE abutment, while the accuracy of FHWA method was proved to be sensitive to the change of particle size and reinforcement spacing.

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REFERENCES

1) AASHTO. (2014): LRFD bridge design specifications. 7th ed., Washington, DC, USA.
2) Adams, M.T., Kanop, K. and Wu, J.T.H. (2007a): Mini pier experiments-geosynthetic reinforcement spacing and strength as related to performance. Proceedings of Geo-Denver 2007. Geosynthetics in Reinforcement and Hydraulic Applications, Geotechnical special publication 165, ASCE, Reston, VA, USA.
3) Adams, M.T., Schlatter, W. and Stabile, T. (2007b): Geosynthetic reinforced soil integrated abutments at the bowman road bridge in Defiance County, Ohio. Proceedings of Geo-Denver 2007. Geosynthetics in Reinforcement and Hydraulic Applications, Geotechnical special publication 165, ASCE, Reston, VA, USA, 16-26.
4) Adams, M.T., Nicks, J., Stabile, T., Wu, J. T. H., Schlatter, W. and Hartmann, J. (2011): Geosynthetic reinforced soil integrated bridge system: Synthesis report. Report No. FHWA-HRT-11-027, Federal Highway Administration, McLean, VA.
5) Adams, M.T., Nicks, J., Stabile, T., Wu, J.T.H., Schlatter, W. and Hartmann, J. (2012): Geosynthetic reinforced soil integrated bridge system-interim implementation guide. Report No. FHWA-HRT-11-026, Federal Highway Administration, McLean, VA, USA.
6) Benjamin, C. V. S., Bueno, B. S. and Zornberg, J. G. (2015): Field monitoring evaluation of geotextile-reinforced soil-retaining walls. Geosynthetics International, 14(2), 100-118.
7) Bueno, B. S., Benjamin, C. V. S. and Zornberg, J. G. (2005): Field performance of a full-scale retaining wall reinforced with nonwoven geotextiles. Proceedings of American Society of Civil Engineers Geo-Frontiers Congress, Austin, Texas, United States, 166, 1-9.
8) Han, J., Jiang, Y. and Xu, C. (2017): Recent advances in geosynthetic-reinforced retaining walls for highway applications. Frontiers of Structural & Civil Engineering, 5, 1-9.
9) Keller, G. and Devin, S. (2003): Geosynthetic-reinforced soil bridge abutments. Transportation Research Board 1819(2), 362-368, Proceedings of 8th International Conference on Low Volume Roads, Reno, NV.
10) Saghebfar, M., Abu-Farsakh, M., Ardash, A., Chen, Q. and Fernandez, B.A. (2017): Performance monitoring of geosynthetic reinforced soil integrated bridge system (GRS-IBS) in Louisiana. Geotextiles and Geomembranes, 45(2), 34-47.
11) Salgado, R. (2008): The engineering of foundations. Vol. 888. New York: McGraw-Hill, USA.
12) Wu, J.T.H. (1994): Design and construction of low cost retaining walls: the next generation in technology. Report No. CTI-UCD-1-94, Colorado Transportation Institute, Denver, CO.
13) Wu, J.T.H., Pham, T.Q. and Adams, M.T. (2011): Composite behavior of geosynthetic reinforced soil mass. Report No. FHWA-HRT-10-077, the US Federal Highway Administration, McLean, VA, USA.