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Numerical simulation of the deformation response of geosynthetic reinforced soil mini-piers

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ABSTRACT: This paper presents a numerical investigation of the deformation response of geosynthetic reinforced soil (GRS) mini-piers under service load conditions. The backfill soil was characterized using a nonlinear elasto-plastic constitutive model that incorporates a hyperbolic stress-strain relationship and the Mohr-Coulomb failure criterion. The geotextile reinforcement was characterized using linearly elastic elements with orthotropic stiffness and strength. Various interfaces were included to simulate the interaction between different components. The three-dimensional numerical model was validated using experimental data from GRS mini-pier loading tests, including average settlements and maximum lateral facing displacements. Simulation results from a parametric study indicate that backfill soil friction angle, backfill soil apparent cohesion, reinforcement spacing, and reinforcement stiffness have the most significant effects on settlements and lateral facing displacements for GRS mini-piers under service load conditions.

KEYWORDS: Geosynthetics, Geosynthetic reinforced soil, Mini-pier, Deformation, Service load, Numerical model

1. INTRODUCTION

The Federal Highway Administration (FHWA) has developed a large-scale test method to measure the deformation behavior of geosynthetic reinforced soil (GRS) composites, called the GRS mini-pier loading test (Adams et al. 2011a; Nicks et al. 2013a). A GRS mini-pier consists of a column of compacted granular backfill soil and geosynthetic reinforcement layers that are frictionally connected to external facing blocks. To conduct the loading test, vertical loads are applied incrementally to a concrete footing placed on the top soil surface and the vertical settlement is measured. The resulting vertical stress-strain relationship is used to estimate deformations for a geosynthetic reinforced soil-integrated bridge system (GRS-IBS) abutment constructed using same backfill soil and geosynthetic reinforcement materials (Adams et al. 2011a, 2011b).

Field and laboratory loading tests have been conducted on GRS piers and abutments, and generally indicate satisfactory performance under service load conditions and relatively large bearing capacity (Adams 1997; Gotteland et al. 1997; Ketchart and Wu 1997; Wu et al. 2001, 2006; Nicks et al. 2013a, 2016; Adams et al. 2014; Iwamoto et al. 2015). Lee and Wu (2004) reviewed the results of several such tests and concluded that the ultimate bearing capacity can be high for closely spaced geosynthetic reinforcement (i.e., vertical spacing ≤ 0.3 m) and well-graded, well-compacted backfill soil. In a closely-spaced GRS system, geosynthetics can significantly improve the stiffness and strength of the GRS composite (Wu and Pham 2013; Wu et al. 2013). Nicks et al. (2013a, 2016) conducted a series of loading tests on GRS mini-piers and found that deformations were smaller for well-graded backfill soil versus open-graded backfill soil. Results also indicate that the facing blocks provided additional confinement and yielded greater ultimate bearing capacity than for similar GRS mini-piers without facing blocks by a

factor of 1.2 to 2.2. Using elastic theory, Adams et al. (2014) concluded that the stiffness for a three-dimensional (3D) unconfined mini-pier is lower than for a GRS abutment in plane strain (i.e., 2D) conditions, whereas the ultimate bearing capacity for both structures is approximately the same (i.e., within 15% to 20%).

Due to the size, difficulty, and cost of these large-scale loading tests, Nicks et al. (2013a) suggested that it is difficult to isolate the effects of different parameters, such as backfill soil or reinforcement, using experimental techniques alone. Therefore, numerical modeling can be used to better understand these effects. Thus far, numerical studies have been conducted for GRS abutments in 2D conditions (Helwany et al. 2003, 2007; Wu et al. 2006; Fakharian and Attar 2007; Zheng et al. 2014, 2017a; Ambauen et al. 2016; Zheng and Fox 2016, 2017; Ardah et al. 2017), and results from these investigations generally show relatively small lateral facing displacements and bridge seat settlements under service load conditions. Associated parametric studies indicate that relative compaction of backfill soil, reinforcement spacing, reinforcement stiffness, and bridge load have significant effects on resulting deformations (Helwany et al. 2007; Ambauen et al. 2016; Zheng and Fox 2016; 2017). GRS piers and abutments are 3D structures and thus 3D numerical modeling ultimately is needed to better simulate the deformation response.

This paper presents a numerical study of the deformation response of GRS mini-piers under service load conditions. A 3D finite difference model was developed and validated using experimental data and a parametric study was conducted to investigate the effects of various backfill soil and reinforcement parameters. Assumptions currently used for the service limit design are evaluated based on results from the parametric study.

2. NUMERICAL MODEL

The finite difference program *FLAC3D Version 5.0* (Itasca Consulting Group 2015) was used for the current investigation to simulate the deformation response of GRS mini-piers DC-1 and DC-3, as described by Nicks et al. (2013a, 2016). Mini-piers DC-1 and DC-3 have different backfill soils and the same geometry and geosynthetic materials. Results from the 3D numerical simulations are compared with experimental results, including average settlements and lateral facing displacements.

2.1. Model geometry

The geometry of the DC-1 and DC-3 GRS mini-piers is shown in Figure 1. The mini-piers have height = 2.0 m and a square cross section ($1.4 \text{ m} \times 1.4 \text{ m}$). The inside dimensions of the backfill soil are $1.0 \text{ m} \times 1.0 \text{ m}$, which yields a height-to-base ratio of 2:1. This ratio matches the typical aspect ratio for a triaxial test soil specimen. Each vertical facing measures $1.4 \text{ m (width)} \times 2.0 \text{ m (height)}$ and has 10 courses of concrete masonry unit (CMU) blocks, with each course consisting of 3.5 blocks and each block measuring $0.4 \text{ m (length)} \times 0.2 \text{ m (width)} \times 0.2 \text{ m (height)}$. Vertical loads are applied to the pier through a concrete footing block that rests on the top soil surface and has dimensions of $0.9 \text{ m} \times 0.9 \text{ m} \times 0.5 \text{ m}$.

Both mini-piers were constructed using the same woven polypropylene geotextile with orthotropic tensile stiffness and strength (i.e., different values in the machine direction (MD) and cross-machine direction (CMD)). Reinforcement layers were placed in an alternating pattern (i.e., rotated 90°) to balance the overall stiffness of the structure and avoid failure in a predetermined lower stiffness direction. The mini-piers had primary reinforcement with a vertical spacing of $S_v = 0.2 \text{ m}$ and two layers of bearing bed reinforcement ($n_{br} = 2$) at the top,

as shown in Figure 1(b). Primary reinforcement layers had dimensions of $1.4 \text{ m} \times 1.4 \text{ m}$ and were placed between the CMU blocks with frictional connections. Bearing bed reinforcement layers had dimensions of $1.0 \text{ m} \times 1.0 \text{ m}$ and were not connected to the CMU blocks.

2.2. Material models and properties

The backfill soil for DC-1 was an open-graded gravel with maximum aggregate size $d_{max} = 12.7 \text{ mm}$, coefficient of uniformity $C_u = 2.36$, and coefficient of curvature $C_z = 1.19$ (Nicks et al. 2013a, 2015, 2016). The DC-1 soil is classified as AASHTO No. 8 according to the American Association of State Highway and Transportation Officials (AASHTO) M 43 specification and as GP, poorly graded gravel, according to the Unified Soil Classification System (USCS). The backfill soil for DC-3 was also an open-graded gravel, classified as AASHTO No. 57 and GP, with $d_{max} = 25.4 \text{ mm}$, $C_u = 2.05$, and $C_z = 1.18$ (Nicks et al. 2013a, 2015, 2016).

The backfill soils were modeled as a nonlinear elasto-plastic material with a hyperbolic stress-strain relationship and the Mohr-Coulomb failure criterion. The tangent elastic modulus E_t , unloading-reloading modulus E_{ur} , bulk modulus B , and tangent Poisson's ratio ν_t are expressed as (Duncan et al. 1980):

$$E_t = \left[1 - \frac{R_f(1 - \sin \phi')(\sigma'_1 - \sigma'_3)}{2c' \cos \phi' + 2\sigma'_3 \sin \phi'} \right]^2 K p_a \left(\frac{\sigma'_3}{p_a} \right)^n \quad (1)$$

$$E_{ur} = K_{ur} p_a \left(\frac{\sigma'_3}{p_a} \right)^n \quad (2)$$

$$B = K_b p_a \left(\frac{\sigma'_3}{p_a} \right)^m \quad (3)$$

$$\nu_t = \frac{1}{2} - \frac{E_t}{6B} \quad (4)$$

where σ'_1 and σ'_3 = major and minor principal effective stresses; ϕ' = friction angle; c' = apparent cohesion; R_f = failure ratio; K = elastic modulus number; n = elastic modulus exponent; p_a = atmospheric pressure; K_{ur} = unloading-reloading modulus number; K_b = bulk modulus number; m = bulk modulus exponent; and ν_t has a range of 0 to 0.49. The soil model accounts for nonlinear behavior and has been used to simulate the static response of GRS abutments under service load conditions (Fakharian and Attar 2007; Zheng and Fox 2016, 2017). In the current investigation, a stress-dependent friction angle is specified using the following equation (Duncan et al. 1980):

$$\phi' = \phi'_o - \Delta\phi' \log_{10} \left(\frac{\sigma'_3}{p_a} \right) \quad (5)$$

where ϕ'_o = friction angle at $\sigma'_3 = p_a$, and $\Delta\phi'$ = reduction of friction angle for a ten-fold increase in σ'_3 / p_a . This equation is needed to accommodate the wide range of applied stresses for the mini-pier loading tests. Equations (1-5) were implemented into FLAC using FISH subroutines to update values of stress-dependent soil modulus and friction angle during the course of each simulation.

Backfill soil parameters were calibrated using measured data from large-size consolidated-drained triaxial compression tests reported by Gebrenegus et al. (2015) and Nicks et al. (2015) for the same AASHTO No. 8 and AASHTO No. 57 soils, respectively. The tests were simulated using *FLAC3D* and the measured and simulated results are shown in Figures 2 and 3. For both soils, the nonlinear stress-strain behavior prior to peak shear strength is accurately reproduced and the simulated dilation behavior is also in reasonable agreement with

the measured data. The post-peak strain softening response at high axial strain is not accurately reproduced by the numerical model, which does not affect the findings of the current study with focus on service load conditions. A summary of calibrated parameters for the AASHTO No. 8 and AASHTO No. 57 backfill soils is presented in Table 1.

Geotextile reinforcement was modeled using linearly elastic structural elements with no bending stiffness and different tensile stiffness in the MD and CMD. Reinforcement parameters are provided as Geotextile-1 in Table 2. The elastic modulus E_r is calculated using the tensile stiffness at 5% axial strain $J_{5\%}$ as:

$$E_r = J_{5\%} / t_r \quad (6)$$

where t_r is the geotextile thickness (constant). Geotextile-1 has $t_r = 1$ mm, and $E_r = 192$ MPa and $E_r = 438$ MPa in the MD and CMD, respectively. The CMU blocks and concrete footing were modeled as elastic materials with modulus $E = 20$ GPa and Poisson's ratio $\nu = 0.2$ (Khalaf et al. 1992). Soil interface shear strength parameters were characterized using a reduction factor RF defined as:

$$RF = \frac{\tan \delta'_i}{\tan \phi'_p} = \frac{c'_i}{c'} \quad (7)$$

where δ'_i is the interface friction angle and c'_i is the interface adhesion. The friction angles for various interfaces are provided in Table 3.

2.3. Modeling procedures

The numerical models of the GRS mini-piers were constructed in layers. Each layer consisted of one lift of soil, one course of CMU blocks, and the necessary interfaces. Geotextile reinforcement layers were placed at specified elevations, depending on the simulation, in an

alternating pattern. A temporary uniform vertical surcharge stress of 8 kPa was applied to the top surface of each soil lift, and then removed prior to placement of the next lift to simulate the effects of backfill soil compaction. This modeling approach has been used in several previous studies (Hatami and Bathurst 2006; Guler et al. 2007; Huang et al 2010; Yu et al. 2016; Zheng and Fox 2017; Zheng et al. 2017a). After mini-pier construction was completed, a concrete footing was placed on top for load application. The loading schedule for each test is provided in Table 4. Vertical stress was applied incrementally, with the time for each increment ranging from 5 to 30 minutes, and was increased when the settlement rate slowed to less than 0.076 mm between any two readings (Nicks et al. 2013a). For the numerical simulations, a uniform vertical stress q_v was applied on top of the concrete footing and increased in stages following the loading schedule in Table 4. For each construction and loading stage, the numerical model was resolved to an equilibrium condition under gravitational forces before proceeding to the next stage. A sensitivity analysis also was conducted to investigate the effect of mesh size on the simulation results. Considering the computational accuracy and efficiency, a total of 18,420 elements was selected for the 3D numerical model of each mini-pier. Mesh elements are shown in Figures 1(b) and 1(c).

3. MODEL VALIDATION

Measured and simulated values of average footing settlement and average vertical strain with increasing applied vertical stress for DC-1 are shown in Figure 4(a). The data are in close agreement for vertical stresses $q_v \leq 600$ kPa and provide validation for the numerical model for service load conditions. At higher stress levels, the simulated values deviate from the measured results and indicate lower settlements and strains. The geotextile reinforcement was modeled as a

linearly elastic material and thus does not undergo tensile stiffness reduction with increasing strain leading to rupture. Similar to findings of Zheng et al. (2017b) for GRS bridge abutments, this likely caused the underestimation of settlements at high vertical stress in the numerical simulations. The calculated vertical stress-strain curve from the analytical method proposed by Hoffman and Wu (2015) is also shown in Figure 4 and indicates similar agreement with the experimental measurements, including an underestimation of settlements starting at approximately 600 kPa. Simulated lateral facing displacement profiles for vertical stresses ranging from 130 kPa to 404 kPa for DC-1 are shown in Figure 4(b), and indicate increasing displacements and lower elevations for the point of maximum displacement with increasing vertical stress. The simulated maximum lateral facing displacement is 2.0 mm at elevation $z = 1.7$ m for $q_v = 130$ kPa, and increases to 9.8 mm at $z = 1.3$ m for $q_v = 404$ kPa. Measured values of maximum lateral facing displacement for DC-1 reported by Nicks et al. (2016) are also shown in Figure 4(b), and are equal to 4.8 mm at $z = 1.7$ m for $q_v = 200$ kPa and 9.0 mm at $z = 1.4$ m for $q_v = 400$ kPa. These measurements are in close agreement with the simulated maximum values in terms of both magnitude and location, and provide additional validation for the numerical model used in this study.

Corresponding measurements and simulation results for DC-3 are shown in Figure 5. The simulated settlements are in good agreement with measured results for vertical stresses $q_v \leq 500$ kPa and slightly smaller than the measured values beyond 500 kPa. Similar to DC-1, the Hoffman and Wu (2015) method provides values of settlement that are slightly lower than the measurements at high stress levels. In Figure 5(b), the simulated maximum lateral facing displacements occur at $z = 1.5$ m and 1.3 m for $q_v = 200$ kPa and 400 kPa, respectively, which

are lower than the elevation of $z = 1.7$ m for the measured results. However, the magnitudes of simulated and measured maximum lateral facing displacement are in reasonable agreement.

Although the numerical simulations underestimate settlements at high applied vertical stress, simulated values of average settlement and maximum lateral facing displacement are in good agreement with measured data for service load conditions, as shown in Table 5. A vertical stress of 200 kPa represents a typical bridge load and a vertical stress of 400 kPa corresponds to a high bridge load (Nicks et al. 2016). The differences between simulated and measured settlements for $q_v = 200$ kPa and 400 kPa, and maximum lateral facing displacement for $q_v = 400$ kPa are within $\pm 7\%$, whereas the differences in maximum lateral facing displacements for $q_v = 200$ kPa are larger (-29.0% for DC-1 and -12.9% for DC-3). In general, the values in Table 5 suggest that the current numerical model can reasonably simulate the deformation response of GRS mini-piers for applied vertical stresses q_v as high as 400 kPa.

4. PARAMETRIC STUDY

A parametric study was conducted using the same modeling approach to investigate the effects of various backfill soil and reinforcement parameters on the deformation response of GRS mini-piers under service load conditions. A baseline case is first described, and the effects of backfill soil friction angle, backfill soil apparent cohesion, reinforcement spacing, reinforcement stiffness, and bearing bed reinforcement are investigated through a parametric study. Similar to results in Table 5, the parametric study focuses on the average settlements and maximum lateral facing displacements under vertical stresses of 200 kPa and 400 kPa, and the results are provided in Tables 6 and 7.

4.1. Baseline case

Backfill soil properties for the baseline case model match the AASHTO No. 8 soil (Table 1) except that the peak friction angle is $\phi'_p = 46^\circ$. The geotextile reinforcement corresponds to Geotextile-4 in Table 2 and has tensile stiffness $J_{5\%} = 700$ kN/m in the MD and $J_{5\%} = 788$ kN/m in the CMD, with an average value of 744 kN/m. The ultimate tensile strength (70 kN/m) is isotropic (i.e., same in the MD and CMD) and satisfies the minimum requirement for GRS-IBS design (Adams et al. 2011a). No bearing bed reinforcement is included for the baseline case. After the mini-pier was numerically constructed in stages, loading was applied with vertical stress q_v increasing from 0 kPa to 400 kPa in increments of 25 kPa.

Simulated settlements and lateral facing displacements for the baseline case are shown in Figure 6. The settlements and vertical strains increase almost linearly with increasing vertical stress. The settlements at $q_v = 200$ kPa and 400 kPa are 13.9 mm and 31.9 mm, respectively, and yield vertical strains of 0.70% and 1.60%. The vertical stress at 0.5% strain $q_{0.5\%}$, defined as the service limit state for the GRS-IBS design method (Adams et al. 2011a), is 153 kPa for the baseline case. Lateral facing displacements also increase with increasing vertical stress, as shown in Figure 6(b). The displacement profiles are relatively uniform for vertical stress smaller than 100 kPa. As the vertical stress increases, the lateral facing displacements near the top of the mini-pier increase significantly, and the maximum value occurs at the elevation of $z = 1.7$ m for $q_v \geq 100$ kPa. Maximum lateral facing displacements for $q_v = 200$ kPa and 400 kPa are 3.9 mm and 10.8 mm, respectively, and the corresponding lateral strains are 0.78% and 2.16% based on the half-width of the backfill soil (0.5 m).

4.2. Backfill soil friction angle

Numerical simulations were conducted for peak friction angle $\phi'_p = 38^\circ, 42^\circ, 46^\circ$, and 50° . Corresponding friction angles for soil-block and soil-geotextile interfaces were obtained using Equation (7). Results are shown in Figure 7(a) and indicate that the settlement increases nonlinearly with increasing applied vertical stress for each friction angle and decreases significantly with higher friction angles. Figure 7(b) shows that settlement decreases nonlinearly with increasing friction angle for both $q_v = 200$ kPa and 400 kPa, with the effect becoming less significant for higher ϕ'_p values. Figure 7(c) shows the lateral facing displacement profiles for $q_v = 200$ kPa and 400 kPa. Similar to settlements in Figure 7(b), lateral facing displacements decrease significantly with increasing friction angle. A higher friction angle produces higher soil stiffness at any given effective stress, and hence lower deformations under the applied vertical stress.

4.3. Backfill soil apparent cohesion

Backfill soil can display apparent cohesion due to unsaturated conditions, and this may be significant depending on the fines content and the shape of the soil-water retention curve. Numerical simulations were conducted for backfill soil apparent cohesion values of $c' = 0$ kPa, 5 kPa, 10 kPa, and 15 kPa. Corresponding values of adhesion for soil-block and soil-geotextile interfaces were obtained using Equation (7). Figure 8 shows that apparent cohesion has important effects on both settlements and lateral facing displacements. The vertical stress-strain relationships in Figure 8(a) indicate that settlement decreases with increasing apparent cohesion, and the effect becomes less significant at higher values, as shown in Figure 8(b). Figure 8(c) shows that increasing apparent cohesion can reduce lateral facing displacements for both $q_v =$

200 kPa and 400 kPa, and the effect also becomes progressively less significant at higher values. For design work, $c' = 0$ is typically assumed to be conservative because apparent cohesion can vary during the service life of a structure depending on moisture conditions.

4.4. Reinforcement spacing

Numerical simulations were conducted for reinforcement spacing $S_v = 0.1$ m, 0.2 m, and 0.4 m. In each case, a soil lift thickness of 0.2 m was maintained for the numerical construction procedure. Results in Figure 9 show that reinforcement spacing has significant effects on settlements and lateral facing displacements. In Figure 9(a), settlement increases nonlinearly with increasing applied vertical stress for each reinforcement spacing, and this nonlinearity is most pronounced for $S_v = 0.4$ m. Figure 9(b) indicates that settlement increases almost linearly with increasing reinforcement spacing for both $q_v = 200$ kPa and $q_v = 400$ kPa, and the effect is more significant at the higher stress level. Lateral facing displacements are shown in Figure 9(c) and also increase significantly with increasing reinforcement spacing for $q_v = 200$ kPa and $q_v = 400$ kPa.

4.5. Reinforcement stiffness

Numerical simulations were conducted for Geotextile-2, Geotextile-3, Geotextile-4, and Geotextile 5 (Table 2), which have average values of reinforcement stiffness $J_{5\%} = 375$ kN/m, 457 kN/m, 744 kN/m, and 1050 kN/m, respectively. Results are shown in Figure 10. As reinforcement stiffness increases from 375 kN/m to 1050 kN/m, the settlement decreases from 16.4 mm to 12.9 mm for $q_v = 200$ kPa, and from 40.0 mm to 28.9 mm for $q_v = 400$ kPa. Similar

to other variables for the parametric study, the nonlinearity of the effect is more pronounced at the higher applied stress level. Lateral facing displacements are shown in Figure 10(c) and indicate similar trends.

4.6. Bearing bed reinforcement

Bearing bed reinforcement layers are typically included under the bridge seat to provide additional support for the GRS-IBS design method (Adams et al. 2011a). Figure 11 shows the settlements and lateral facing displacements for bearing bed reinforcement layer number $n_{br} = 0$, 2, and 4. The settlements are nearly the same for $n_{br} = 0$, 2, and 4 up to an applied vertical stress of 150 kPa. Beyond 150 kPa, settlements are slightly smaller for more bearing bed reinforcement layers. As shown in Figure 11(c), lateral facing displacements become smaller with increasing bearing bed reinforcement layers. This is consistent with the observations that bearing bed reinforcement can reduce lateral facing displacements but is unlikely to reduce settlement under typical design-level vertical stresses (≤ 200 kPa) for GRS-IBS abutments (Nicks et al. 2013a; Zheng and Fox 2017).

5. DISCUSSIONS

5.1. Vertical stress

The vertical stress-strain relationships obtained from GRS mini-pier loading tests can be used to estimate the vertical compressions of GRS-IBS abutments and also prescribe the allowable vertical stresses based on deformation tolerance criteria (Nicks et al. 2016a). FHWA defines the vertical stress at 0.5% vertical strain as the service limit for GRS abutments (Adams et al. 2011a). According to this criterion, values of vertical stress at the service limit $q_{v,0.5\%}$ for

each simulation case of the parametric study are presented in Table 8. Backfill soil friction angle has a significant effect, with $q_{v,0.5\%}$ increasing 54% as ϕ'_p increases from 38° to 50° . Values of $q_{v,0.5\%}$ show a similar percentage increase as c' increases from 0 to 15 kPa. Reinforcement spacing and reinforcement stiffness also have important effects, whereas bearing bed reinforcement has a small effect on the vertical stress at the service limit in this study. At higher stress levels corresponding to the strength limit, experimental and numerical studies have indicated that the effect of bearing bed reinforcement can be more important (Nicks et al. 2013a; Zheng et al. 2017b).

FHWA provides a semi-empirical relationship to calculate the vertical stress at the service limit for GRS composites, which is defined as 10% of the ultimate bearing capacity (Adams et al. 2011a; Nicks et al. 2013a). The ultimate bearing capacity is calculated as (Pham 2009; Wu et al. 2011):

$$q_{ult} = [\sigma'_c + 0.7^{\frac{S_v}{6d_{max}}} (\frac{T_{ult}}{S_v})] K_p + 2c' \sqrt{K_p} \quad (8)$$

where σ'_c is the effective confining stress (typically taken as zero to be conservative), and K_p is the Rankine passive earth pressure coefficient. Equation (8) does not account for bearing bed reinforcement. Using Equation (8) and $\sigma'_c = 0$, calculated values of vertical stress at the service limit ($10\% q_{ult}$) are presented in Table 8 for each simulation case of the parametric study. With one exception ($S_v = 0.1$ m), vertical stresses obtained at the service limit from the numerical simulations are larger, by a ratio of 1.6 or more, than the values calculated using Equation (8). The ratio of simulated to calculated values shows the most variability with respect to the reinforcement spacing parameter, with ratios ranging from 0.63 to 8.38 as S_v increases from 0.1

to 0.4 m. This indicates that Equation (8) is sensitive to the effect of S_v on q_{ult} . Equation (8) is applicable for GRS composites with close reinforcement spacing (i.e., $S_v \leq 0.3$ m), which may have yielded the underestimation of ultimate bearing capacity for $S_v = 0.4$ m. The average ratio of simulated to calculated values for all cases from the parametric study is 2.39. Considering only cases that strictly meet the requirements on reinforcement spacing ($S_v \leq 0.3$ m) and ultimate tensile strength ($T_{ult} \geq 70$ kN/m) for GRS-IBS abutments (i.e., excluding $S_v = 0.4$ m, Geotextile-2, and Geotextile-3), the average ratio is 1.83. Overall, the results in Table 8 suggest that the assumed semi-empirical relationship of 10% q_{ult} for the vertical stress at the service limit is conservative.

5.2. Volume change

In the current FHWA design guideline for GRS bridge abutments (Adams et al. 2011a), the maximum lateral facing displacement is estimated using the assumption of zero volume change. This assumption can be evaluated using the numerical simulation results from the parametric study. The volumetric strain ε_v for the GRS composite is calculated as (Nicks et al. 2016):

$$\varepsilon_v = (\Delta V_l + \Delta V_v) / V \quad (9)$$

where ΔV_l is the lateral volume change (expansion), ΔV_v is the vertical volume change (contraction), and V is the initial volume of the backfill soil (2 m³). The lateral volume change was calculated from lateral facing displacement profiles and was assumed to be the same for all four facings and the vertical volume change was calculated from average settlement (Nicks et al. 2016). The volumetric strains at the vertical stresses of 200 kPa and 400 kPa from the parametric

study are given in Table 9, where positive values indicate expansion and negative values indicate contraction. In general, volumetric strains are less than 1% for $q_v = 200$ kPa and less than 3% for $q_v = 400$ kPa. These volumetric strains are relatively small and suggest that the zero volume change assumption may be reasonable.

6. SUMMARY AND CONCLUSIONS

This paper presents a numerical investigation of the deformation response of GRS mini-piers under service load conditions. The backfill soil was characterized using a nonlinear elastoplastic constitutive model that incorporates a hyperbolic stress-strain relationship and the Mohr-Coulomb failure criterion. The geotextile reinforcement was characterized using linearly elastic elements with orthotropic stiffness and strength. Various interfaces were included to simulate the interaction between different components. The following conclusions are reached:

1. The 3D numerical model was validated for service load conditions using experimental data from two large-scale GRS mini-pier loading tests. Simulation results are in good agreement with measurements of average settlement and lateral facing displacement for applied vertical stresses q_v as high as 400 kPa.
2. A parametric study was conducted to investigate the effects of backfill soil and reinforcement parameters. Simulated average settlements range from 8.3 mm to 21.4 mm for $q_v = 200$ kPa, and maximum lateral facing displacements range from 1.7 mm to 7.2 mm. The results indicate that backfill soil friction angle, backfill soil apparent cohesion, reinforcement spacing, and reinforcement stiffness have important effects on settlements and lateral facing displacements under service load conditions. Bearing bed reinforcement had a small effect on

settlement and a more important effect on lateral facing displacements for the conditions simulated.

3. The FHWA method to calculate the vertical stress at the service limit for GRS composites (i.e., 10% of the ultimate bearing capacity) was evaluated based on simulation results from the parametric study. The average ratio of simulated to calculated values is 1.83 for cases that meet the requirements for GRS-IBS abutments, which suggests that the FHWA method is conservative.
4. The zero volume change assumption for estimation of maximum lateral facing displacements for GRS mini-piers is reasonable for the conditions investigated in the parametric study. Further investigation is needed to verify this assumption for GRS-IBS abutments.

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NOTATIONS

Basic SI units are given in parentheses.

B bulk modulus (kPa)

C_u coefficient of uniformity

C_z coefficient of curvature

410	c'	apparent cohesion (kPa)
411	c'_i	interface adhesion (kPa)
412	d_{max}	maximum aggregate size (mm)
413	E	elastic modulus (kPa)
414	E_r	elastic modulus for reinforcement (kPa)
415	E_t	tangent elastic modulus (kPa)
416	E_{ur}	unloading-reloading modulus (kPa)
417	$J_{5\%}$	secant stiffness at 5% tensile strain for reinforcement (kN/m)
418	K	elastic modulus number
419	K_b	bulk modulus number
420	K_p	coefficient of passive earth pressure
421	K_{ur}	unloading-reloading modulus number
422	m	bulk modulus exponent
423	n	elastic modulus exponent
424	n_{sr}	number of bearing bed reinforcement layers
425	p_a	atmospheric pressure (kPa)
426	$q_{0.5\%}$	vertical stress at 0.5% vertical strain (kPa)
427	q_{ult}	ultimate bearing capacity (kPa)
428	q_v	applied vertical stress (kPa)
429	RF_i	reduction factor for interface shear strength

430	R_f	failure ratio
431	S_v	reinforcement vertical spacing (m)
432	T_{ult}	ultimate strength of reinforcement (kN/m)
433	t_r	thickness of reinforcement (mm)
434	ΔV_l	lateral volume change of GRS composite (m ³)
435	ΔV_v	vertical volume change of GRS composite (m ³)
436	V	initial total volume of GRS composite (m ³)
437	δ'_i	interface friction angle (°)
438	ε_v	volumetric strain (%)
439	ν	Poisson's ratio
440	ν_t	tangent Poisson's ratio
441	σ'_1	major principal effective stresses (kPa)
442	σ'_3	major principal effective stresses (kPa)
443	σ'_c	effective confining stress (kPa)
444	ϕ'	friction angle (°)
445	ϕ'_o	friction angle at $\sigma'_3 = p_a$ (°)
446	ϕ'_p	peak friction angle (°)
447	$\Delta\phi'$	reduction of friction angle for ten-fold increase in σ'_3 (°)

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Table 1. Backfill soil parameters.

Soil	γ (kN/m ³)	K	K_{ur}	n	R_f	K_b	m	p_a (kPa)	c' (kPa)	ϕ'_p (°)	$\Delta\phi'$ (°)	ψ (°)
AASHTO No. 8 ^a	17.7	429	515	0.50	0.79	306	0	101.3	0	46.9	8.8	13.0
AASHTO No. 57 ^b	15.6	361	433	0.44	0.82	188	0.05	101.3	0	45.1	7.8	9.0

^a calibrated from data reported by Gebrenegus et al. (2015)^b calibrated from data reported by Nicks et al. (2015)

Table 2. Reinforcement parameters.

Reinforcement	Tensile stiffness at 5% strain,	Ultimate tensile strength,
	$J_{5\%}$ (kN/m) ^a	T_{ult} (kN/m) ^a
	MD $\times$ CMD	MD $\times$ CMD
Geotextile-1	192 $\times$ 438	70 $\times$ 70
Geotextile-2	354 $\times$ 396	39 $\times$ 36
Geotextile-3	406 $\times$ 508	52 $\times$ 52
Geotextile-4	700 $\times$ 788	70 $\times$ 70
Geotextile-5	1050 $\times$ 1050	105 $\times$ 84

^a based on data provided by manufacturer

Table 3. Interface friction angles.

	Soil-geotextile ^a	Soil-block/footing ^b	Geotextile-block ^c	Block-block ^d
DC-1	42.3°	34.8°	37.0°	35.0°
DC-3	40.2°	32.8°	37.0°	35.0°

^a based on average of data for $RF = 0.85$ from Vieira et al. (2013)

^b based on data for $RF = 0.65$ from Ling et al. (2010)

^c from Nicks et al. (2013b)

^d from Ling et al. (2010)

Table 4. Loading schedule and measured results for DC-1 and DC-3 (Nicks et al. 2013a).

Stage	DC-1		DC-3	
	Vertical stress (kPa)	Settlement (mm)	Vertical stress (kPa)	Settlement (mm)
1	128	8.4	35	6.1
2	164	10.9	92	13.2
3	232	14.7	148	19.8
4	295	20.8	206	26.2
5	356	25.9	265	32.8
6	396	30.2	319	39.1
7	441	34.3	374	46.5
8	504	39.6	429	53.8
9	569	47.0	495	61.5
10	633	53.8	558	70.4
11	698	66.0	626	78.2
12	782	75.9	690	85.6
13	827	82.3	756	94.0
14	890	90.7	821	102.6
15	962	97.0	884	112.3

Table 5. Results for DC-1 and DC-3.

	Vertical stress (kPa)	Average settlement			Maximum lateral facing displacement		
		Measured ^a (mm)	Simulated ^b (mm)	Difference ^c (%)	Measured ^a (mm)	Simulated ^b (mm)	Difference ^c (%)
DC-1	200	13.4	13.9	3.7	4.8	3.4	-29.0
	400	31.6	32.1	1.6	9.0	9.6	6.7
DC-3	200	26.3	24.6	-6.5	6.2	5.4	-12.9
	400	51.7	49.2	-4.8	11.2	11.5	2.7

^a data reported by Nicks et al. (2016)^b values linearly interpolated from data in Figures 4 and 5^c = (simulated value – measured value)/measured value

Table 6. Average settlements and average vertical strains from parametric study

		Average settlement (mm)		Average vertical strain (%)	
		200 kPa	400 kPa	200 kPa	400 kPa
Backfill soil friction angle	38°	21.4	49.5	1.07	2.48
	42°	17.1	39.9	0.86	2.00
	46°	13.9	31.9	0.70	1.60
	50°	11.9	26.3	0.60	1.32
Backfill soil apparent cohesion	0 kPa	13.9	31.9	0.70	1.60
	5 kPa	11.2	27.0	0.56	1.35
	10 kPa	9.4	22.0	0.47	1.10
	15 kPa	8.3	19.3	0.42	0.97
Reinforcement spacing	0.1 m	11.9	25.3	0.60	1.27
	0.2 m	13.9	31.9	0.70	1.60
	0.4 m	17.8	44.2	0.89	2.21
Reinforcement stiffness	375 kN/m	16.4	40.0	0.82	2.00
	457 kN/m	15.6	37.6	0.78	1.88
	744 kN/m	13.9	31.9	0.70	1.60
	1050 kN/m	12.9	28.9	0.65	1.45
Bearing bed reinforcement	0	13.9	31.9	0.70	1.60
	2	13.3	30.8	0.67	1.54
	4	12.3	27.5	0.62	1.38

Table 7. Maximum lateral facing displacements and maximum lateral strains from parametric study.

		Maximum lateral facing displacement (mm)		Maximum lateral strain (%)	
		200 kPa	400 kPa	200 kPa	400 kPa
Backfill soil friction angle	38°	7.2	19.3	1.44	3.86
	42°	5.3	15.0	1.06	3.00
	46°	3.9	10.8	0.78	2.16
	50°	2.9	8.2	0.58	1.64
Backfill soil apparent cohesion	0 kPa	3.9	10.8	0.78	2.16
	5 kPa	2.5	8.6	0.50	1.72
	10 kPa	1.9	6.3	0.38	1.26
	15 kPa	1.7	5.3	0.34	1.06
Reinforcement spacing	0.1 m	2.7	7.3	0.54	1.46
	0.2 m	3.9	10.8	0.78	2.16
	0.4 m	5.4	16.2	1.08	3.24
Reinforcement stiffness	375 kN/m	4.8	14.7	0.96	2.94
	457 kN/m	4.5	13.3	0.90	2.66
	744 kN/m	3.9	10.8	0.78	2.16
	1050 kN/m	3.4	9.5	0.68	1.90
Bearing bed reinforcement	0	3.9	10.8	0.78	2.16
	2	3.5	10.0	0.70	2.00
	4	2.9	7.9	0.58	1.58

Table 8. Vertical stress at the service limit from parametric study.

		Simulated vertical stress at 0.5% vertical strain, $q_{v,0.5\%}$ (kPa)	Calculated 10% ultimate bearing capacity, 10% q_{ult} (kPa)	Ratio (simulated/ calculated)
Backfill soil friction angle	38°	112	58	1.93
	42°	132	69	1.91
	46°	153	84	1.82
	50°	172	104	1.65
Backfill soil apparent cohesion	0 kPa	153	84	1.82
	5 kPa	181	87	2.08
	10 kPa	211	89	2.37
	15 kPa	232	92	2.52
Reinforcement spacing	0.1 m	170	269	0.63
	0.2 m	153	84	1.82
	0.4 m	134	16	8.38
Reinforcement stiffness	375 kN/m	139	43	3.23
	457 kN/m	142	63	2.25
	744 kN/m	153	84	1.82
	1050 kN/m	161	101	1.59
Bearing bed reinforcement	0	153	84	1.82
	2	158	-	-
	4	165	-	-

Table 9. Volumetric strains from parametric study.

		Volumetric strain ε_v at $q_v = 200$ kPa (%)	Volumetric strain ε_v at $q_v = 400$ kPa (%)
Backfill soil friction angle	38°	0.93	2.80
	42°	0.57	2.00
	46°	0.31	1.23
	50°	0.15	0.75
Backfill soil apparent cohesion	0 kPa	0.31	1.23
	5 kPa	0.05	0.81
	10 kPa	-0.02	0.34
	15 kPa	-0.02	0.18
Reinforcement spacing	0.1 m	0.13	0.64
	0.2 m	0.31	1.23
	0.4 m	0.55	2.14
Reinforcement stiffness	375 kN/m	0.47	1.94
	457 kN/m	0.41	1.66
	744 kN/m	0.31	1.23
	1050 kN/m	0.25	0.99
Bearing bed reinforcement	0	0.31	1.23
	2	0.29	1.20
	4	0.19	0.89

Figure 1. GRS mini-pier: (a) isometric view; (b) elevation view; (c) plan view.

(a)

(b)

Figure 2. Comparison of measured and simulated triaxial test results for AASHTO No. 8: (a) deviator stress vs. axial strain; (b) volumetric strain vs. axial strain (measured data reported by Gebrenegus et al. 2015).

(a)

(b)

Figure 3. Comparison of measured and simulated triaxial test results for AASHTO No. 57: (a) deviator stress vs. axial strain; (b) volumetric strain vs. axial strain (measured data reported by Nicks et al. 2015).

Figure 4. Simulation results for DC-1: (a) settlements and vertical strains; (b) lateral facing displacements (measured data reported by Nicks et al. 2013a, 2016).

Figure 5. Simulation results for DC-3: (a) settlements and vertical strains; (b) lateral facing displacements (measured data reported by Nicks et al. 2013a, 2016).

Figure 6. Simulation results for the baseline case: (a) settlements and vertical strains; (b) lateral facing displacements.

Figure 7. Effects of backfill soil friction angle: (a) settlements and vertical strains vs. vertical stress; (b) settlements and vertical strains for $q_v = 200$ kPa and 400 kPa; (c) lateral facing displacements for $q_v = 200$ kPa and 400 kPa.

Figure 8. Effects of backfill soil apparent cohesion: (a) settlements and vertical strains vs. vertical stress; (b) settlements and vertical strains for $q_v = 200$ kPa and 400 kPa; (c) lateral facing displacements for $q_v = 200$ kPa and 400 kPa.

Figure 9. Effects of reinforcement spacing: (a) settlements and vertical strains vs. vertical stress; (b) settlements and vertical strains for $q_v = 200$ kPa and 400 kPa; (c) lateral facing displacements for $q_v = 200$ kPa and 400 kPa.

Figure 10. Effects of reinforcement stiffness: (a) settlements and vertical strains vs. vertical stress; (b) settlements and vertical strains for $q_v = 200$ kPa and 400 kPa; (c) lateral facing displacements for $q_v = 200$ kPa and 400 kPa.

Figure 11. Effects of bearing bed reinforcement layers: (a) settlements and vertical strains vs. vertical stress; (b) settlements and vertical strains for $q_v = 200$ kPa and 400 kPa; (c) lateral facing displacements for $q_v = 200$ kPa and 400 kPa.