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Numerical study on maximum reinforcement tensile forces in geosynthetic reinforced soil bridge abutments

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ABSTRACT: This paper presents a numerical study of the maximum reinforcement tensile forces in geosynthetic reinforced soil (GRS) bridge abutments. The backfill soil was characterized using a nonlinear elasto-plastic constitutive model that incorporates a hyperbolic stress-strain relationship with strain softening behavior and the Mohr-Coulomb failure criterion. The geogrid reinforcement was characterized using a hyperbolic load-strain-time constitutive model. The GRS bridge abutments were numerically constructed in stages, including soil compaction effects, and then loaded in stages to the service load condition (i.e., applied vertical stress = 200 kPa) and finally to the failure condition (i.e., vertical strain = 5%). A parametric study was conducted to investigate the effects of geogrid reinforcement, backfill soil, and abutment geometry on reinforcement tensile forces at the service load condition and failure condition. Results indicate that reinforcement vertical spacing and backfill soil friction angle have the most significant effects on magnitudes of maximum tensile forces at the service load condition. The locus of maximum tensile forces at the failure condition was found to be Y-shaped. Geogrid reinforcement parameters have little effect on the Y-shaped locus of the maximum tensile forces when no secondary reinforcement layers are included, backfill soil shear strength parameters have moderate effects, and abutment geometry parameters have significant effects.

KEYWORDS: Geosynthetics, Geosynthetic reinforced soil, Bridge abutment, Numerical modeling, Internal stability, Tensile force

1. Introduction

Geosynthetic reinforced soil (GRS) bridge abutments have been widely used for transportation infrastructure and several case histories are reported in the literature (Abu-Hejleh et al. 2002; Adams et al. 2011a; Saghebfar et al. 2017). Although these structures show good field performance in terms of acceptable deformations under service load conditions, design methodologies continue to evolve. The design process includes assessment of internal and external stability to size the geometry of the structure and select appropriate backfill soil and reinforcement materials.

With regard to internal stability, reinforcement is selected based on an assumed distribution (i.e., locus) of maximum tensile forces. For the Simplified design method (Berg et al. 2009; AASHTO 2012), this locus coincides with the assumed failure surface, which initially follows the Rankine active failure surface from the toe of the abutment and then moves vertically upward to the heel of the bridge seat. The maximum tensile force in each reinforcement layer is calculated along the locus using a 2:1 distribution for bridge surcharge load, the Rankine active earth pressure coefficient (K_a), and the total lateral stress over the appropriate tributary area. For the geosynthetic reinforced soil-integrated bridge system (GRS-IBS) design method (Adams et al. 2011b), the locus is assumed to lie on the vertical centerline of the bridge seat. The maximum tensile force in each reinforcement layer is calculated based on the ultimate bearing capacity of GRS composite structures (Wu and Pham 2013; Wu et al. 2013) with total lateral stresses for the service load condition. The locus of maximum tensile forces is a key assumption for each method and the dependency of this locus on reinforcement parameters, backfill soil properties, and abutment geometry is largely unknown.

Numerical studies have been conducted to investigate the response of GRS bridge

abutments under service load conditions (Helwany et al. 2003, 2007; Ambauen et al. 2016; Zheng and Fox 2016a, 2017), and generally have indicated relatively small lateral facing displacements and bridge seat settlements. Most of these studies have focused on the deformation response, and fewer have investigated the magnitudes and locations of maximum tensile forces in the reinforcement. Xie et al. (2016) calculated maximum reinforcement tensile forces in GRS walls under a surcharge load using limit analysis and assuming a log-spiral failure surface, and reported important effects from backfill soil friction angle, wall batter, and bridge seat setback distance. Ardah et al. (2017) conducted numerical simulations for a GRS-IBS abutment and found that the locus of maximum tensile forces differs from the Rankine active failure surface and may change with increasing bridge load. This suggests that the locus of maximum tensile forces at the service load condition may not be representative of the locus at the failure condition. Other numerical studies have found that strain softening of the backfill soil and nonlinear response of the geosynthetic reinforcement are likely to be important for characterization of the response of GRS bridge abutments under high applied stress conditions approaching failure (e.g., Walters et al. 2002; Hatami and Bathurst 2006; Liu and Ling 2012; Yang et al. 2012; Zheng and Fox 2016b, Zheng et al. 2018).

This paper presents a numerical study of the magnitudes and locations of maximum tensile forces for GRS bridge abutments at both service load and failure conditions. The study consists of a baseline case, followed by a parametric study of the effects of geogrid reinforcement, backfill soil, and abutment geometry. Results provide insights with regard to the internal stability of GRS bridge abutments and the Simplified and GRS-IBS design methods for these structures.

2. Numerical model and baseline case

The two-dimensional finite difference program *FLAC Version 7.0* (Itasca Consulting Group 2011) was used for the current investigation. Zheng and Fox (2016a) developed a *FLAC* model to simulate the response of GRS bridge abutments under service load conditions and validated the model using field measurements for the Founders/Meadows Parkway Bridge in Castle Rock, Colorado (Abu-Hejleh et al. 2000, 2001). Zheng et al. (2018) enhanced this model by incorporating strain softening for the backfill soil and nonlinear stiffness for the geogrid reinforcement and concluded that these effects are relatively small for GRS bridge abutments under service load conditions but significant for high applied stress conditions approaching failure. The current investigation is based on the *FLAC* model and numerical simulations described in Zheng et al. (2018).

2.1. Model geometry

The finite difference grid and boundary conditions for a baseline case GRS bridge abutment model are shown in Figure 1. The model represents a single-span bridge system with a span of $L_b = 30$ m and symmetrical support structures on both ends. Each end support structure consists of a lower GRS wall, bridge seat, upper GRS fill, and approach roadway. Only the right-hand side of the bridge system was simulated due to symmetry. The GRS bridge abutment has a lower wall height of $h = 5.0$ m and a total height of $H = 6.9$ m. An L-shaped bridge seat with a thickness of 0.4 m rests on top of the abutment and has a setback distance of $a_b = 0.2$ m from the back of the wall facing. The clear distance between the top of the facing and the bottom of the bridge beam d_e is equal to the bridge seat thickness (0.4 m). The clearance height for the bridge beam above the foundation soil is 5.4 m, which satisfies the FHWA minimum requirement of 4.9

m for interstate highways (Stein and Neuman 2007). The bridge seat has a contact length of $L_c = 1.0$ m with the bridge beam on the upper surface and a contact length of $L_s = 1.5$ m with the backfill soil on the lower surface. There is a 100 mm-wide vertical expansion joint between the bridge beam and bridge seat. Assuming a span-to-depth ratio of $R_{sd} = L_b / D = 20$, the depth of the bridge beam is $D = 1.5$ m. A upper GRS fill lies behind the bridge seat, has a thickness of 1.8 m, and is covered by a 0.1 m-thick concrete roadway. The geosynthetic reinforcement has uniform length $L_r = 3.5$ m ($0.7h$) and vertical spacing $S_v = 0.2$ m for both the lower GRS wall and upper GRS fill. No secondary (i.e., bearing bed) reinforcement is included under the bridge seat. To minimize the influence of boundary conditions on system response, the foundation soil has a depth of 10 m ($2h$) and the rear boundary of the model is located at a distance of 20 m ($4h$) from the wall facing. Model lateral boundaries are fixed in the horizontal direction and are free to move in the vertical direction, whereas the bottom boundary is fixed in both the horizontal and vertical directions. Horizontal coordinate x is measured toward the right-hand side from the back of the wall facing and vertical coordinate z is measured upward from the top surface of the foundation soil.

2.2. Soils

Zheng and Fox (2016a) simulated the static response of GRS bridge abutments under service load conditions using a nonlinear elasto-plastic model with a hyperbolic relationship and the Mohr-Coulomb failure criterion. For the current investigation, the model was enhanced to incorporate strain softening behavior of the backfill soil at larger strain levels to simulate abutment response up to failure conditions. The tangent elastic modulus E_t , unloading-reloading

modulus E_{ur} , bulk modulus B , and tangent Poisson's ratio ν_t are expressed as follows (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' = 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. Equations (1-4) were implemented into *FLAC* using *FISH* subroutines to update the stress-dependent soil moduli during each simulation. To account for strain softening behavior, the friction angle and dilation angle were defined as piece-wise linear functions of incremental plastic shear strain and calibrated using triaxial test data.

Properties for the backfill soil in the reinforced and retained soil zones are based on experimental data for a well-graded angular sand that meets the AASHTO and FHWA specifications for use in GRS bridge abutments. Consolidated-drained triaxial compression tests were conducted on sand specimens at five levels of effective confining stress. The tests were numerically simulated and soil parameters were back-calculated from the experimental results.

Piece-wise linear relationships for changes in friction angle and dilation angle with increasing incremental plastic shear strain ε_p , which occurs once the soil reaches the Mohr-Coulomb failure criterion, are shown in Figure 2. The soil has peak values of friction angle and dilation angle of $\phi'_p = 46^\circ$ and $\psi_p = 18^\circ$, respectively, for $\varepsilon_p = 0$ to 4%. For $\varepsilon_p = 4$ to 15%, the soil experiences post-peak strain softening where both angles decrease linearly. For $\varepsilon_p \geq 15\%$, the soil responds with constant volume (i.e., steady state) friction angle and dilation angle of $\phi'_{cv} = 43^\circ$ and $\psi_{cv} = 0^\circ$, respectively. A summary of the backfill soil properties is provided in Table 1, and a comparison of measured and simulated triaxial test results using these back-calculated soil parameters is shown in Figure 3. The nonlinear stress-strain behavior before peak strength and post-peak strain softening behavior are well characterized using this approach. The simulated volume change response with increasing axial strain also is in good agreement with the measured data for effective confining stresses up to 69 kPa and in lesser agreement at the two higher effective confining stress levels. A firm foundation soil was used for all simulations so that the response of the GRS bridge abutments would not be significantly affected by foundation soil conditions. The foundation soil is characterized using the elastic-perfectly plastic Mohr-Coulomb model with $E = 80$ MPa, $\nu = 0.3$, $c' = 2$ kPa, $\phi' = 54^\circ$, and $\psi = 14^\circ$.

2.3. Reinforcement

The reinforcement was specified as a high-density polyethylene (HDPE) uniaxial geogrid and characterized using the hyperbolic load-strain-time model proposed by Allen and Bathurst (2014a, 2014b). Yu et al. (2016) provided additional model details and validation using field measurements for two GRS walls. Tangent stiffness of the reinforcement J_t is calculated as:

$$J_t = \frac{1}{J_0 \left(\frac{1}{J_0} + \chi \varepsilon \right)^2} \quad (5)$$

where J_0 = initial tangent stiffness, ε = tensile strain, and χ = empirical fitting parameter. Figure 4 shows tensile behavior and provides material properties for the reinforcement for the baseline case, indicated as Geogrid-2. Geogrid-1 and Geogrid-3 are discussed later for the parametric study. The stiffness parameters for Geogrid-2 are $J_0 = 1054t^{-0.0697}$ kN/m and $\chi = 0.0359$ m/kN, where t = time (hr.). Yu et al. (2016) reported that stiffness values for these HDPE geogrids are not significantly affected by practical construction times of interest and, for simplicity, can be taken as constant during construction. Following this procedure, an end-of-construction time $t = 3600$ hr. was specified for the current simulations. The tensile behavior for Geogrid-2 is characterized by $J_0 = 596$ kN/m with stiffness decreasing nonlinearly with increasing strain. Geogrid reinforcements were characterized using cable elements and rigidly connected to the facing blocks.

182

183 2.4. Structure components

184 The concrete modular facing blocks, bridge seat, and roadway were modeled as linearly
 185 elastic materials with unit weight $\gamma = 23.5$ kN/m³, elastic modulus $E = 20$ GPa, and Poisson's
 186 ratio $\nu = 0.2$. The bridge beam was modeled as a solid block of unit width ($L_b \times D \times 1$) of
 187 linearly elastic material with $E = 20$ GPa and $\nu = 0.2$. The unit weight of the bridge beam γ_b
 188 was increased in stages to produce increasing vertical load on the lower GRS wall. The average
 189 applied vertical stress on the bottom surface of the bridge seat is $q_v = L_b D \gamma_b / 2L_s$. In addition to
 190 vertical load, the bridge imposes a horizontal frictional restraining force that resists lateral

deformations of the bridge abutment and is important to include for modeling purposes (Zheng and Fox 2016a).

2.5. Interfaces

Interface shear strengths for the numerical model are defined using interface friction angle δ'_i and interface adhesion c'_i , and summarized in Table 2. Soil interface shear strength parameters (e.g., soil-geogrid, soil-block, and soil-bridge seat) are characterized using reduction factor RF defined as:

$$RF = \frac{\tan \delta'_i}{\tan \phi'_p} = \frac{c'_i}{c'} \quad (6)$$

Block-block interfaces are characterized using $\delta'_i = 36.0^\circ$ and $c'_i = 58$ kPa, based on data reported by Yu et al. (2016). Bridge beam-bridge seat interfaces are characterized using $\delta'_i = 21.8^\circ$ and $c'_i = 0$, based on a friction coefficient of 0.4 for elastomeric bearing pads (Caltrans 1994).

2.6. Modeling procedures

The GRS bridge abutment was numerically constructed in stages and then monotonically loaded in stages to failure. The initial condition corresponds to equilibrium of the foundation soil under gravitational forces. The lower GRS wall was constructed in layers on top of the foundation soil, with each layer consisting of one soil lift, one facing block, and the necessary interfaces. Geogrid reinforcement layers were placed at specified elevations, depending on the simulation. A temporary uniform 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 effect of compaction

(Hatami and Bathurst 2006; Zheng et al. 2017). Khabbazian et al. (2011) found that a prescribed minimum soil modulus value has an important effect on simulations using hyperbolic models. Following the work of Hatami and Bathurst (2005, 2006), a minimum modulus of 10 MPa, corresponding to the initial slope of the stress-strain curve for $\sigma'_3 = 34$ kPa, was selected to avoid excessively low soil stiffness under low confining stress. Once staged construction of the lower GRS wall was completed, the bridge seat was placed, the upper GRS fill was constructed in layers behind the bridge seat, and the approach roadway was placed on the upper GRS fill. To load the abutment, the unit weight of the bridge beam was increased in stages; first in increments of 50 kPa to produce $q_v = 200$ kPa, and then in increments of 100 kPa to produce an average vertical strain of $\varepsilon_v = 5\%$ for the lower GRS wall, where ε_v is based on the difference between the average downward displacement of the bridge seat and average downward displacement of the underlying foundation soil. After each construction and loading stage, the numerical model was resolved to an equilibrium condition.

3. Simulation results for baseline case

The vertical stress-strain relationship for the lower GRS wall for the baseline case is shown in Figure 5. FHWA specifies a maximum allowable applied vertical stress of $q_v = 200$ kPa as the service limit and a vertical strain of $\varepsilon_v = 5\%$ from the vertical stress-strain relationship as the strength limit (Berg et al. 2009; Adams et al. 2011b). These are defined as the service load condition and the failure condition, respectively, in the current study. Based on the relationship for the baseline case shown in Figure 5, the service load condition yields $\varepsilon_v = 0.8\%$ and the failure condition yields $q_v = 917$ kPa. As vertical stress increases, the vertical strain is

approximately linear for q_v up to 400 kPa and then increases nonlinearly for higher applied vertical stress.

3.1. Service load condition

Figure 6 presents a plot of maximum tensile force versus elevation for geogrid layers in the lower GRS wall at the service load condition of $q_v = 200$ kPa. The profile shows generally increasing values with increasing elevation. The maximum tensile force is smallest (1.2 kN/m) in the lowermost geogrid layer ($z = 0.2$ m) and then increases significantly with elevation between $z = 0.2$ m and $z = 2$ m. From $z = 2$ m to $z = 4$ m, the profile is nearly uniform with values approximately equal to 5 kN/m. Maximum tensile force increases again near the top of the wall with the highest value (6.9 kN/m) occurring in the uppermost geogrid layer ($z = 4.8$ m).

The maximum tensile force profiles obtained using the GRS-IBS and Simplified methods are also shown for comparison in Figure 6. Values for the GRS-IBS method were calculated using the soil peak friction angle of 46° , whereas values for the Simplified method were calculated using both 46° and the maximum allowable friction angle of 40° . A maximum allowable friction angle is specified for the Simplified method to avoid underestimation of tensile forces in the reinforcement for high strength soils (AASHTO 2012). Maximum tensile forces obtained using the Simplified method for $\phi'_p = 46^\circ$ are nearly uniform, ranging from 5.2 kN/m to 6.0 kN/m, and in close agreement with those from the numerical simulation for $z > 1$ m. For the uppermost geogrid layer ($z = 4.8$ m), the Simplified method yields a slightly lower value (6.0 kN/m). When the peak friction angle is reduced to $\phi'_p = 40^\circ$, the Simplified method yields maximum tensile forces that are higher by approximately 2 kN/m at all elevations. Maximum tensile forces obtained using the GRS-IBS method are much larger than the simulated

values and the Simplified values, with a minimum value of 13.1 kN/m at $z = 2.8$ m and a highest value of 22.0 kN/m in the uppermost geogrid layer. Figure 6 indicates that the GRS-IBS method is significantly more conservative than the Simplified method with regard to maximum tensile forces in the reinforcement at the service load condition. This is attributed to the different formulations for the two methods; the Simplified method was calibrated based on field measurements under working stress conditions whereas the GRS-IBS method was developed based on the ultimate bearing capacity of GRS composite structures (Wu and Pham 2013; Wu et al. 2013).

3.2. Failure condition

The distributions of tensile force for five geogrid layers in the lower GRS wall for the baseline case at the failure condition of $\varepsilon_v = 5\%$ are shown in Figure 7, where the locations of maximum values are indicated using solid symbols. The distributions indicate double maxima for the upper two geogrid layers, such as the two values near the toe and heel of the bridge seat for the uppermost layer 23, and a single maximum for lower layers. The maximum tensile force occurs near the centerline of the bridge seat in layer 13 and near the facing block connection in the lowermost layer 3.

Corresponding locations of maximum tensile force for all geogrid layers in the lower GRS wall for the baseline case are plotted in Figure 8(a). The plot shows a Y-shaped locus that starts at the toe and heel of the bridge seat, converges at approximately $z = 2.8$ m, and then moves downward to the toe of the wall. This is in good agreement with the observation of a Y-shaped reinforcement rupture pattern from large-scale loading tests on GRS composite structures (Pham 2009). The loci of maximum tensile forces specified by the Simplified and GRS-IBS

methods are also shown for comparison in Figure 8(a). For the Simplified method, the locus initially follows the Rankine active failure surface (i.e., inclined at $45^\circ + \phi'_p/2$ from horizontal) from the toe of the wall up to $x = a_b + L_s$, and then moves vertically upward to the heel of the bridge seat, which matches closely with the right-hand side of the Y-shaped locus from the numerical simulation, and similar reinforcement rupture pattern was also observed from large-scale loading tests on GRS composite structures (Pham 2009). For the GRS-IBS method, the locus of maximum tensile forces follows the vertical centerline of the bridge seat and bears little resemblance to the Y-shaped locus from the numerical simulation.

A plot of soil shear strain contours for the baseline case at the failure condition, as discussed by Zheng et al. (2018), is shown in Figure 8(b). The failure mechanism is a combination of punching shear failure of the bridge seat and internal shear failure of the lower GRS wall, as indicated by the formation of large shear strain zones. The triangular punching failure under the bridge seat is in agreement with the locus of maximum tensile forces for the upper reinforcement layers in Figure 8(a). The internal shear failure surface of the lower GRS wall is bilinear and extends vertically downward from the heel of the bridge seat to approximately the mid-height of the wall and then diagonally to the toe of the wall. This surface is also plotted in Figure 8(a) and differs from the locus of maximum reinforcement tensile forces, which indicates differences in mobilized strains in the soil and reinforcement at the failure condition.

4. Parametric study

A parametric study was conducted to investigate the effects of reinforcement, backfill soil, and abutment geometry parameters on maximum tensile forces in the geogrid reinforcement.

For each series of simulations, only the variable of interest was changed and the other variables were held constant and equal to values for the baseline case as described previously. Results are presented for magnitudes of maximum tensile forces at the service load condition ($q_v = 200$ kPa) and locations of maximum tensile forces at the failure condition ($\varepsilon_v = 5\%$).

4.1. Service load condition

4.1.1. Reinforcement

The effects of reinforcement vertical spacing, stiffness, and length, and number of secondary reinforcement layers on the profile of maximum tensile forces at the service load condition are shown in Figure 9. Figure 9(a) indicates that maximum tensile forces increase significantly at all elevations as the vertical spacing S_v increases from 0.2 m to 0.6 m. Highest values from the profiles are 6.9 kN/m, 11.2 kN/m, and 15.4 kN/m for $S_v = 0.2$ m, 0.4 m and 0.6 m, respectively. Simulations were conducted for the three geogrids shown in Figure 4, with Geogrid-1, Geogrid-2, and Geogrid-3 having progressively higher tensile stiffness. The results in Figure 9(b) indicate that maximum tensile forces increase with increasing reinforcement stiffness at the mid-height and upper sections of the GRS wall. Figure 9(c) indicates that increasing reinforcement length from $0.5 h$ to $1.1 h$ had little effect on the maximum tensile forces, especially for $L_r \geq 0.7 h$. Secondary reinforcement layers often are included below the bridge seat to provide additional support. In the current study, numerical simulations were conducted for secondary reinforcement layer number $n_{sr} = 0, 5, 10$, and 15 , where $n_{sr} = 0$ indicates no secondary reinforcement and $n_{sr} = 15$ indicates 15 layers of secondary reinforcement between elevations $z = 2.0$ m and 5.0 m. The results in Figure 9(d) indicate that maximum tensile forces

decrease at elevations where the secondary reinforcement layers are added; however, the effect is not as significant as reduction of vertical spacing for primary reinforcement, as shown in Fig. 9(a).

4.1.2. Backfill soil

The effects of backfill soil shear strength parameters on maximum reinforcement tensile forces at the service load condition are shown in Figure 10. Simulations were conducted for friction angle $\phi'_p = 38^\circ, 42^\circ, 46^\circ, \text{ and } 50^\circ$, with $\phi'_{cv} = 35^\circ, 39^\circ, 43^\circ, \text{ and } 47^\circ$, respectively. Corresponding friction angles for soil interfaces were obtained using Equation (6). Figure 10(a) indicates that maximum tensile forces decrease significantly with increasing friction angle at all elevations, which occurs because higher shear strength allows the backfill soil to carry a greater fraction of the applied load. The highest maximum tensile force is located in the uppermost geogrid layer for each case and decreases from 9.3 to 6.0 kN/m as ϕ'_p increases from 38 to 50°. Simulations were also conducted for $c' = 0, 5, 10, \text{ and } 15$ kPa, with corresponding values of soil interface adhesion obtained using Equation (6). The maximum tensile force profiles are shown in Figure 10(b) and indicate that maximum tensile forces decrease with increasing cohesion; however, the effect becomes less significant for higher cohesion values, especially for $c' \geq 10$ kPa. One source of increased cohesion for GRS bridge abutments is apparent cohesion for unsaturated soil conditions, with greater c' values expected for moist backfill soils and higher fines content.

4.1.3. Abutment geometry

The effects of abutment geometry parameters, including bridge seat setback distance a_b ,

bridge seat length L_s , and lower GRS wall height h , on maximum reinforcement tensile forces at the service load condition are shown in Figure 11. Figure 11(a) indicates that maximum tensile forces decrease significantly with increasing bridge seat setback distance, especially at the top of the wall. A larger setback moves the applied bridge load away from the wall facing and decreases the maximum force carried by the reinforcement layers. Values for $a_b = 1.4$ m are much smaller than for the other cases, which is consistent with observations reported by Xie et al. (2016). Figure 11(a) also shows that the highest maximum force value occurs in the uppermost geogrid layer for $a_b = 0.2$ m and in lower layers for the other cases. Figure 11(b) indicates that bridge seat length has a moderate effect on maximum tensile force profiles, with values increasing as bridge seat length increases from 1.0 to 2.5 m. Simulations were conducted for GRS bridge abutments with $h = 3$ to 9 m and maximum tensile forces are plotted versus normalized elevation (z/h) in Figure 11(c). The effect of abutment height is relatively small, with higher tensile force values generally observed for taller walls, especially near the lower part of the structure.

4.2 Failure condition

4.2.1. Reinforcement

The effects of geogrid reinforcement parameters on the locus of maximum tensile forces at the failure condition are shown in Figure 12. The plots for $S_v = 0.2$ m, 0.4 m, and 0.6 m display close agreement in Figure 12(a), which suggests that vertical spacing has little effect on the locus of maximum force values. Similarly, Figures 12(b) and 12(c) indicate only minor effects of reinforcement stiffness and reinforcement length. Figure 12(d) indicates that the elevation of the convergence point for the Y-shaped locus moves lower with increasing number

of secondary reinforcement layers. Also, for $n_{sr} \geq 10$, the maximum tensile forces on the left-hand side of the Y-shaped locus occur near the facing block connections.

4.2.2. Backfill soil

The effects of backfill soil shear strength parameters on the locus of maximum tensile forces at the failure condition are shown in Figure 13. The elevation of the convergence point for the Y-shaped locus moves slightly lower with increasing friction angle from $z = 3.2$ m for $\phi'_p = 38^\circ$ to $z = 2.8$ m for $\phi'_p = 50^\circ$. This is consistent with an expected steeper angle of the triangular punching zone (i.e., Figure 8b) with increasing friction angle. Similar trends are observed in Figure 13(b) and indicate that the elevation of the convergence point decreases with increasing cohesion from $z = 2.8$ m for $c' = 0$ kPa to $z = 2.2$ m for $c' = 15$ kPa.

4.2.3. Abutment geometry

The effects of bridge seat setback distance, bridge seat length, and height of the lower GRS wall on the locus of maximum tensile forces at the failure condition are shown in Figure 14. For increasing bridge seat setback distance, Figure 14(a) indicates a similar elevation of the convergence point ($z \approx 2.8$ m) and progressively greater asymmetry for the upper part of the Y-shaped locus. In Figure 14(b), the Y-shaped locus displays a wider upper section and lower convergence point, from $z = 3.4$ m for $L_s = 1.0$ m to $z = 2.0$ m for $L_s = 2.5$ m, with increasing bridge seat length. Simulations conducted for increasing height of the lower GRS wall ($h = 3$ m to 9 m) indicate the elevation of the convergence point is approximately 1.5 m to 2.0 m below the bridge seat in each case.

4.2.4. Discussion

The results in Figures 12-14 indicate that geogrid reinforcement parameters have little effect on the Y-shaped locus of the maximum tensile forces when no secondary reinforcement layers are included, backfill soil shear strength parameters have moderate effects, and abutment geometry parameters have significant effects. The upper section of the Y-shaped locus is similar to the triangular punching zone for the bearing capacity failure of a shallow foundation, as indicated by the decreasing elevation of the convergence point with increasing backfill soil friction angle and bridge seat length.

The locus of maximum tensile forces as specified in the Simplified method (Berg et al. 2009) is also plotted in Figures 12-14 for comparison. The specified loci are in good agreement with the right-hand side of the Y-shaped locus from the numerical simulation for each case in Figures 12 and 13. In Figure 14, the specified locus and simulated locus are in lesser agreement for a long bridge seat, large setback distance, and tall abutment (e.g., $L_s \geq 2.0$ m, $a_b \geq 0.6$ m, $h \geq 7$ m). In general, results of the numerical simulations indicate that the locus specified by the Simplified method can provide a first approximation for maximum tensile forces at the failure condition for GRS bridge abutments.

5. Conclusions

This paper presents a numerical study of the maximum reinforcement tensile forces in geosynthetic reinforced soil (GRS) bridge abutments. The backfill soil was characterized using a nonlinear elasto-plastic constitutive model that incorporates a hyperbolic stress-strain relationship with strain softening behavior and the Mohr-Coulomb failure criterion. The geogrid reinforcement was characterized using a hyperbolic load-strain-time constitutive model. A

parametric study was conducted to investigate the effects of geogrid reinforcement, backfill soil, and abutment geometry on reinforcement tensile forces at the service load condition and failure condition. The following conclusions are reached:

1. Maximum tensile forces at the service load condition for the baseline case are slightly smaller than values calculated using the Simplified method, and much smaller than values calculated using the GRS-IBS method.
2. Results from the parametric study indicate that reinforcement spacing and backfill soil friction angle have the most significant effects on the magnitudes of maximum tensile force at the service load condition. The maximum tensile forces increase significantly with increasing reinforcement spacing and decreasing backfill soil friction angle.
3. The locus of maximum reinforcement tensile forces at the failure condition displays a Y-shaped pattern that starts at the toe and heel of the bridge seat, converges at a certain depth, and then moves downward to the toe of the lower GRS wall. The right-hand side of this locus is in good agreement with the locus specified by the Simplified method. The locus of maximum tensile forces is distinct from the internal failure surface of the lower GRS wall, which is attributed to the differences in mobilized strains in the soil and reinforcement at the failure condition.
4. Geogrid reinforcement parameters have little effect when no secondary reinforcement layers are included, backfill soil shear strength parameters have moderate effects, and abutment geometry parameters have significant effects on the Y-shaped locus of maximum tensile forces for the lower wall of a GRS bridge abutment.

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Table 1. Soil properties.

Property	Value
Unit weight, γ (kN/m ³)	17.3
Elastic modulus number, K	334
Unloading-reloading elastic modulus number, K_{ur}	401
Elastic modulus exponent, n	0.66
Failure ratio, R_f	0.67
Bulk modulus number, B	254
Bulk modulus exponent, m	0
Atmospheric pressure, p_a (kPa)	101.3
Cohesion, c' (kPa)	0
Peak friction angle, ϕ'_p (°)	46
Constant volume friction angle, ϕ'_{cv} (°)	43
Peak dilation angle, ψ_p (°)	18
Constant volume dilation angle, ψ_{cv} (°)	0

Table 2. Interface properties.

Property	Soil-geogrid ^a	Soil-block Soil-bridge seat ^b	Block-block ^c	Bridge beam- bridge seat ^d
Friction angle, δ'_i	41.4°	33.9°	36.0°	21.8° ^d
Adhesion, c'_i	0	0	58 kPa	0

^a Average of data ($RF = 0.85$) from Vieira et al. (2013)

^b Data ($RF = 0.65$) from Ling et al. (2010)

^c Yu et al. (2016)

^d Caltrans (1994)

Figure 1. Finite difference grid and boundary conditions for the baseline case GRS bridge abutment model.

(a)

(b)

Figure 2. Relationships for soil parameters with increasing incremental plastic shear strain: (a) friction angle; (b) dilation angle.

Figure 3. Comparison of measured and simulated triaxial test results: (a) deviator stress vs. axial strain; (b) volumetric strain vs. axial strain.

Figure 4. Tensile behavior for three HDPE geogrids ($t = 3600$ hr.): (a) tensile force-strain relationship; (b) tangent stiffness (parameters from Yu et al. 2016).

Figure 5. Vertical stress-strain relationship for lower GRS wall for the baseline case.

Figure 6. Maximum reinforcement tensile forces in lower GRS wall for the baseline case at the service load condition ($q_v = 200$ kPa).

Figure 7. Distributions of reinforcement tensile force for the baseline case at the failure condition ($\varepsilon_v = 5\%$).

(a)

(b)

Figure 8. Simulation results from the baseline case at the failure condition ($\varepsilon_v = 5\%$): (a) locus of maximum reinforcement tensile forces; (b) contours of soil shear strain magnitude (Zheng et al. 2018).

(a)

(b)

(c)

(d)

Figure 9. Effects of geogrid reinforcement parameters on maximum reinforcement tensile forces at the service load condition: (a) vertical spacing; (b) stiffness; (c) length; (d) number of secondary reinforcement layers.

(a)

(b)

Figure 10. Effects of backfill soil shear strength parameters on maximum reinforcement tensile forces at the service load condition: (a) friction angle; (b) cohesion.

Figure 11. Effects of abutment geometry parameters on maximum reinforcement tensile forces at the service load condition: (a) bridge seat setback distance; (b) bridge seat length; (c) lower GRS wall height.

(a)

(b)

(c)

Figure 12. Effects of geogrid reinforcement parameters on locus of maximum reinforcement tensile forces at the failure condition: (a) vertical spacing; (b) stiffness; (c) length; (d) number of secondary reinforcement layers.

Figure 13. Effects of backfill soil shear strength parameters on locus of maximum reinforcement tensile forces at the failure condition: (a) friction angle; (b) cohesion.

647 **Figure 14.** Effects of abutment geometry parameters on locus of maximum reinforcement tensile
648 forces at the failure condition: (a) bridge seat setback distance; (b) bridge seat length; (c) lower
649 GRS wall height.
650