

Article

Prediction of Reinforcement Connection Loads in Geosynthetic Reinforced Segmental Retaining Walls Using Response Surface Method

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Abstract: This paper presents an improved earth pressure method that considers the capacity of a wall toe to carry an earth load to predict the connection loads of GRS segmental walls constructed with cohesionless backfills on competent foundations. In this method, a response surface model (RSM) of the lateral earth pressure coefficient replaces the Coulomb active earth pressure coefficient. The parameters of the RSM were determined from numerical studies on the impacts of toe restraint, wall geometry, and backfill properties on the distribution of earth loads between the toe and reinforcement layers. The unknown coefficients of the RSM were obtained through a regression analysis of 705 reinforcement load values from 65 simulated walls. The proposed method was compared to the earth pressure method and the stiffness method using measured connection loads from field and centrifuge GRS segmental walls. The results show that the predictions of the proposed RSM method are in better agreement with the measurements than those of the stiffness method and the earth pressure method, whether under a typical or poor toe restraint condition.

Keywords: geosynthetics; segmental retaining walls; reinforcement connection load; toe load; response surface method; numerical modeling



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1. Introduction

With the expanding scale of urbanization, a large amount of engineering infrastructures, such as buildings, bridges, and highways, are constructed in mountains and along rivers [1–5]. Geosynthetic reinforced segmental retaining walls have been widely used as bridge abutments, slope supporting structures, high fill subgrades, and bank revetments as a result of their low cost, aesthetics, and tolerance to the foundation. The design of geosynthetic reinforced soil (GRS) walls requires the selection of reinforcement materials and the determination of the reinforcement anchorage length for internal stability. In both of these steps, the value of the maximum tensile force in each reinforcement layer needs to be considered. Hence, the maximum reinforcement tensile force is a key value for the internal stability of GRS walls.

Tatsuoka [6] considered that the reinforcement tensile loads in each layer are uniformly distributed within the critical slip surface of GRS walls with hard facings. Allen et al. (2003) [7] and Bathurst et al. [8] differentiated the connection load T_{con} from the maximum reinforcement load in the backfill T_{bmax} , the latter usually acting at the potential slip surface. For GRS walls in a working stress state, T_{con} is generally the maximum tensile force along a reinforcement layer as a result of the down-drag force caused by backfill compaction, the rotation of the facing column, and the differential settlement of the foundation, as shown in Figure 1 [9–12]. Moreover, a field investigation of the failures of six GRS walls revealed that the failures were mainly caused by a connection breakage in the reinforcement layers [13].

This indicates that the connection load may be the largest within the reinforcement layer and an important design parameter for GRS walls in both the working stress and limit states.

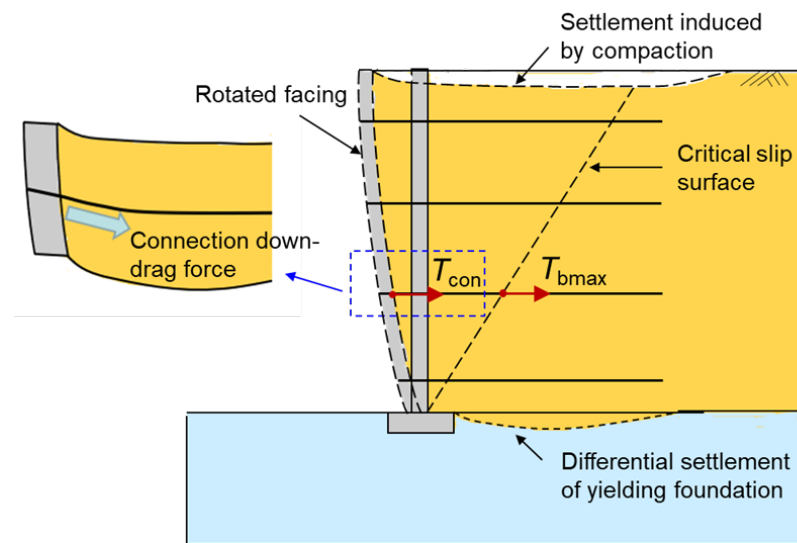


Figure 1. Factors causing connection down-drag force.

There are a few methods for predicting reinforcement connection loads. Gebremariam et al. [14] considered that the earth pressure method, specified in the FHWA (2009) [15] standard, can be used to predict T_{con} for GRS bridge abutments based on a comparison between measured connection loads and theoretical calculations. The earth pressure method is the most widely used for predicting T_{bmax} ; in this approach, T_{bmax} is equated with the active earth pressure calculated using Coulomb's theory within the contributory area of each reinforcement layer. However, this method seriously overestimates the T_{bmax} of GRS walls under working stress conditions, since it ignores the effects of toe resistance, facing stiffness, reinforcement stiffness, and backfill soil cohesion on the reinforcement loads [16]. Overestimation was also observed in the calculated values of the earth pressure method when comparing it to T_{con} under working stress conditions [10,11].

A number of experimental and numerical studies have shown that a significant portion of earth pressures acting on the facing column are counterbalanced by toe resistance [17–21]. Considering the effects of toe resistance on reinforcement loads, Leshchinsky et al. [22] presented a limit equilibrium method to calculate T_{con} for both GRS walls and slopes, in which toe resistance was regarded as a function of the normal load and the interface friction angle at the base of the facing column. Liu et al. [23] included toe resistance in the earth pressure method to estimate T_{con} . In this method, toe resistance was calculated using the equilibrium of the forces and bending moments of the vertical facing column. Some methods for predicting T_{bmax} also included the influence of toe resistance [24–27]. Among them, the stiffness method proposed by Allen and Bathurst [25], which was previously known as the simplified stiffness method, has been adopted in the latest AASHTO (2020) code [28].

It has been shown that the magnitude of toe resistance is affected by a variety of factors, such as toe restraint, wall height, facing batter, facing stiffness, reinforcement spacing, etc. [29–33]. It is not easy to determine the relationship between toe resistance or T_{con} and the enumerated influence factors analytically. This problem can be effectively solved using the response surface method, whereby the relationship can be approximated with a polynomial. The response surface method has been widely used in the probabilistic analysis of reinforced soil structures [34–37]. It has also been used for deterministic issues of reinforced soil walls to predict seismic performances, facing deformations, and reinforcement loads [38–40].

The objective of this study is to propose a reinforcement connection load prediction method that considers the contribution of wall toe to carrying earth loads in geosynthetic

reinforced cohesionless soil segmental walls on competent foundations. First, numerical investigations were conducted on the fractions of earth loads carried by the toe and reinforcement layers and their influence factors (including toe restraint condition, wall height, facing batter, reinforcement spacing, and soil friction angle) based on the numerical modeling of an instrumented field GRS wall. Second, we devised a new method for predicting the connection load by replacing the active earth pressure coefficient with a response surface model (RSM) of the lateral earth pressure coefficient determined using the influence factors of the toe load ratio. Finally, the proposed method was compared to the classical earth pressure method and the stiffness method using the measured reinforcement connection loads from two field walls [10,11] and three centrifuge model walls [32].

2. Numerical Model Validation

2.1. Numerical Model

The numerical models in this study were built based on a geosynthetic reinforced segmental retaining wall that was part of the SR-18 highway in Seattle, Washington [10]. Yu et al. [41] also simulated this wall using the finite-difference program FLAC. Figure 2 shows the numerical model of the SR-18 wall that was built using FLAC. The total wall height was 6.4 m, including a backfill height of 6.1 m and a pavement thickness of 0.3 m. The height of the embedment soil at the wall toe was 0.8 m. The vertical facing column consisted of concrete blocks with 460 mm in length, 300 mm in width, and 200 mm in height. The reinforcement layers had a uniform vertical spacing of 0.6 m and a uniform length of 7.9 m. The spacing between the lowest reinforcement layer and the foundation surface was 0.2 m. A high-density polyethylene uniaxial geogrid (Tensar UXK1100) was used as the reinforcement layer. The backfill was a well-graded, silty, gravelly sand with a maximum particle size of 38 mm and was compacted to reach a 95% maximum dry density (corresponding to a density of 2200 kg/m^3) to construct the wall. The foundation soil consisted of dense, silty, sandy gravel.

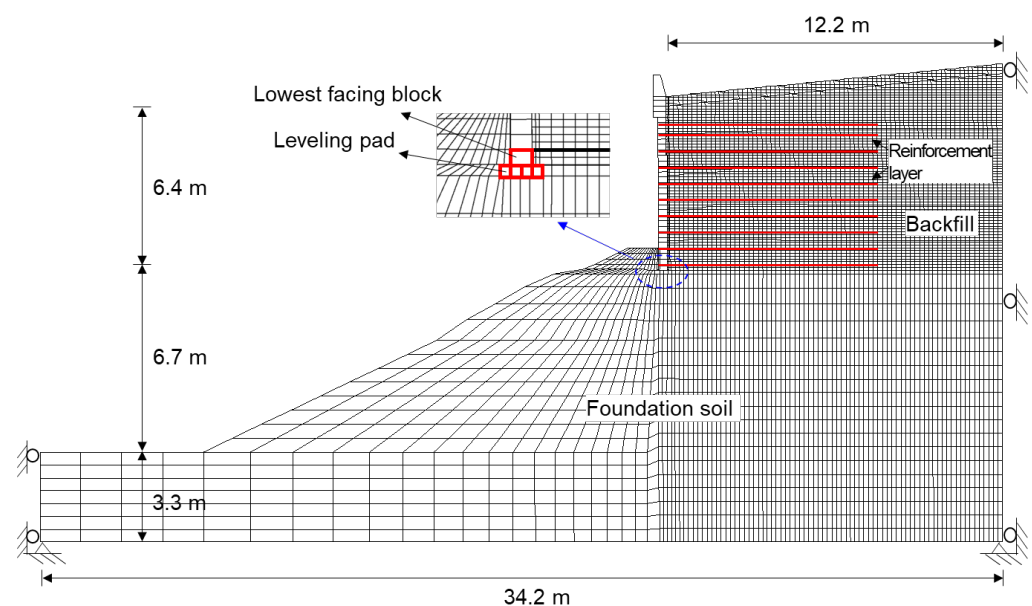


Figure 2. Numerical model of SR-18 wall.

The wall was constructed in layers. The final grading was completed at 4060 h after the beginning of construction. At this time, the wall height was 6.1 m. Pavement construction started at 7200 h and ended at 10,100 h. The final height of the wall was 6.4 m at 10,100 h. The time of 4060 h was considered as the effective end time of construction because, after that, construction progressed very slowly. The different parameters of the wall were also recorded at this time. Hence, the numerical modeling in this paper was conducted for the effective construction stage.

In this numerical modeling, the modeling procedures reported by Hatami and Bathurst [17] were used for backfilling in layers of the reinforced soil wall. The procedures were as follows: (1) generating the grids of the facing block and the backfill soil behind the block, and subsequently adding the related interfaces; (2) cycling the model with a temporary vertical load of 8 kPa on the backfill surface to equilibrium followed by unloading; (3) generating the structural elements to simulate the reinforcements at a specific height of the wall; and (4) repeating the first three steps until the wall model reached the desired height. The left and right boundaries of the numerical model were fixed in the horizontal direction. The bottom boundary of the numerical model was fixed horizontally and vertically.

2.2. Material Constitutive Model and Parameters

The backfill and foundation soil were simulated using the Mohr–Coulomb model. Table 1 presents the input parameters for the soil model. The parameter values were the same as those used by Allen and Bathurst (2014) [10] and Yu et al. (2016) [41]. The backfill friction angle of 54° was determined from the triaxial shear-strength parameters using a plane-strain friction angle calculation method [42]. The plane-strain friction angle was chosen here because the mechanical analysis of retaining walls is a plane strain problem. The adoption of 2 kPa for soil cohesion instead of 0 was done to avoid numerical instability. Since the foundation soil was similar to the backfill, the same parameters were used for both materials for simplicity. The linear elastic constitutive model was selected to model the facing blocks and the leveling pad. The input parameter values for these concrete blocks (listed in Table 1) were determined by referring to Yu et al. (2016) [12].

Table 1. Parameter values for backfill, foundation, and concrete blocks.

Parameters	Backfill and Foundation	Blocks
Density ρ (kg/m ³)	2200	2400
Cohesion c (kPa)	2	-
Plane strain friction angle φ ($^\circ$)	54	-
Dilation angle ψ ($^\circ$)	14	-
Elastic modulus E (MPa)	80	3.2×10^4
Poisson's ratio ν	0.3	0.15

The reinforcements were modeled using the cable element of FLAC. Creep tests were conducted on the geogrids used in the SR-18 wall to obtain the reinforcement stiffness corresponding to the end time of construction (i.e., 4060 h). The test results showed that the creep stiffness at a 1% strain was 310 kN/m [10]. The selection of the stiffness value at a 1% strain was made so that the measured maximum strain in each instrumented reinforcement layer was close to 1%. The other parameters for reinforcements are attributed to the values recommended by Yu et al. [41], as summarized in Table 2.

Table 2. Parameter values for reinforcements.

Parameters	Value
Axial stiffness J (kN/m)	310
Ultimate tensile strength T (kN/m)	62.5
Cross-section area A (m ² /m)	2×10^{-3}
Soil-reinforcement interface cohesion c_{sr} (kN/m)	2.5
Soil-reinforcement interface friction angle φ_{sr} ($^\circ$)	43
Soil-reinforcement interface shear stiffness K_{sr} (MN/m/m)	1

All the concrete–concrete and concrete–soil interfaces in the wall were modeled using a linear spring–slider system, with the strength controlled by the interface friction angle and cohesion and the deformation controlled by the normal and shear stiffness at the

interface. For these two types of interfaces, cohesion is usually close to 0 and the normal stiffness is generally taken as 1000 MPa/m when the interface is horizontal [12,17–19,41]. The interface shear stiffness value is important for predicting the behavior of an interface but is inconvenient to obtain, especially for concrete–concrete interfaces. Hence, large, direct shear tests were performed on the concrete–concrete interfaces with different degrees of roughness to establish an interface shear stiffness calculation model for this type of interface. Partial test details and results were reported by Zhang et al. [32]. The shear stiffness data for each concrete–concrete interface under different vertical loads are plotted in Figure 3a. It should be noted that the interface shear stiffness was normalized by wall height and atmospheric pressure. Figure 3a shows that there was a logarithmic functional relationship between the normalized interface shear stiffness and the tangent values of the interface friction angle under each vertical load. Thus, the interface shear stiffness can be expressed as the following:

$$\frac{K_{si} \cdot H}{P} = \chi(\sigma_n) \cdot \ln(\tan \varphi_i) + \eta(\sigma_n) \quad (1)$$

where K_{si} = the interface shear stiffness, H = the wall height, P = the normal atmospheric pressure (i.e., 101.3 kPa), φ_i = the interface friction angle, σ_n = the interface vertical stress, and $\chi(\sigma_n)$ and $\eta(\sigma_n)$ = coefficients in relation to σ_n . The values of $\chi(\sigma_n)$ and $\eta(\sigma_n)$, corresponding to different vertical stresses, are plotted in Figure 3b and can be expressed as the following:

$$\begin{cases} \chi(\sigma_n) = a_1 \left(\frac{\sigma_n}{P}\right)^2 + b_1 \left(\frac{\sigma_n}{P}\right) + c_1 \\ \eta(\sigma_n) = a_2 \left(\frac{\sigma_n}{P}\right)^2 + b_2 \left(\frac{\sigma_n}{P}\right) + c_2 \end{cases} \quad (2)$$

where $a_{1,2}$, $b_{1,2}$, and $c_{1,2}$ = fitting parameters, and their values are given in Figure 3b. In GRS segmental walls, the interface vertical stress σ_n at the base of the facing is a function of the wall height H : $\sigma_n = H \cdot \gamma_{fb}$, where γ_{fb} = the unit weight of the facing block. The value of γ_{fb} can be considered constant because the plain concrete block used as the facing block usually has a density of 2200 kg/m³ [12,43,44]. Therefore, if H and φ_i are known, the concrete–concrete interface shear stiffness K_{si} can be computed using Equations (1) and (2). Figure 4 shows the values of K_{si} corresponding to H in the range of 3 to 10 m and φ_i in the range of 5° to 45°, calculated using Equations (1) and (2). This shear stiffness calculation model can provide concrete–concrete interface parameters for the numerical simulation of segmental retaining walls.

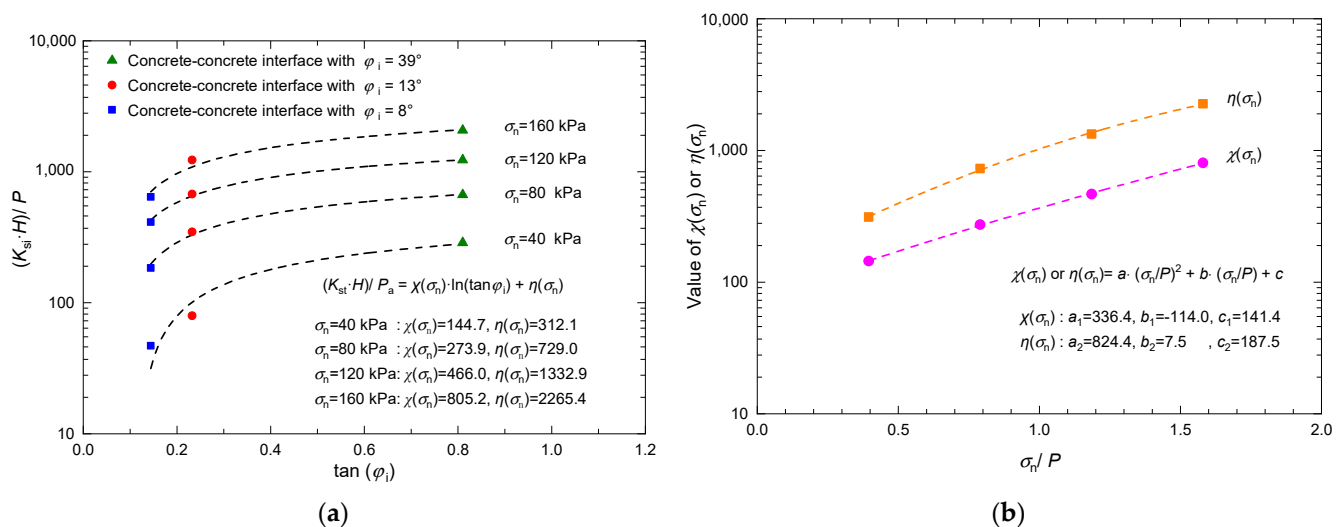


Figure 3. Relationships among the shear stiffness K_{si} , the friction angle φ_i , and the normal stress σ_n of concrete–concrete interface: (a) K_{si} versus φ_i ; and (b) fitting functions $\chi(\sigma_n)$ and $\eta(\sigma_n)$.

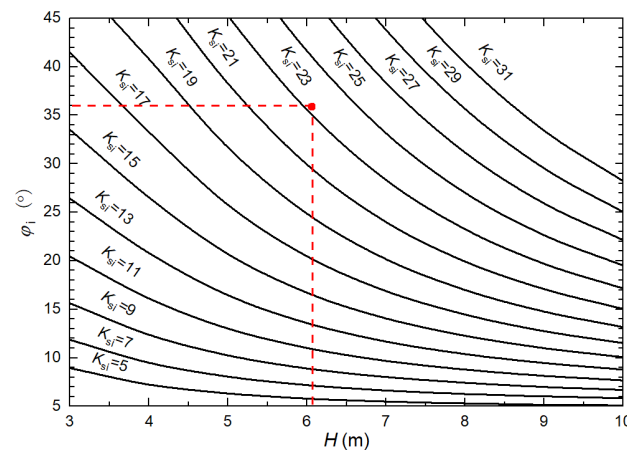


Figure 4. Shear stiffness prediction model of concrete–concrete interface in GRS walls.

For the facing block–leveling pad interface in this wall, the friction angle was taken to be 36° according to the results of an interface shear test performed by Bathurst et al. [8]. The shear stiffness of this interface can be set to 23 MPa, according to Figure 4. A small cohesion of 1 kPa instead of 0 was adopted for this interface to avoid numerical instability. For the interfaces between the facing blocks, the cohesion increased from 0 to 58 kPa as a result of the shear keys between the facing blocks [8]. Also because of the shear keys, the shear stiffness of these interfaces was set to 40 MPa, as recommended by Hatami and Bathurst [17,18] and Yu et al. [41], instead of being determined from Figure 4. A typical normal stiffness of 1000 MPa was applied to these two interfaces.

For the concrete–soil interfaces, such as the facing block–backfill and the leveling pad–foundation soil interfaces, the values of the friction angle and cohesion were the same as those used by Yu et al. [41]. The leveling pad–foundation soil interface had a normal stiffness of 1000 MPa and a shear stiffness of 10 MPa. The value of the shear stiffness was derived from the shear stress–shear displacement curves of a concrete–sand interface reported by Zhang et al. [32]; one of the curves was based on the normal stress that was close to 60 kPa, which is the normal stress value at the base of the leveling pad for this wall. The stiffness parameters for the facing block–backfill interface were taken as a tenth of those for the leveling pad–foundation soil interface, as reported by Hatami and Bathurst [17,18], because the facing block–backfill interface was free in the normal direction. Table 3 shows all the interface parameters described in this section.

Table 3. Parameter values for interfaces.

Parameters	Value
Facing block–leveling pad interface	
Interface cohesion c_{bp} (kPa)	1
Interface friction angle φ_{bp} ($^\circ$)	36
Interface normal stress K_{nbp} (MPa/m)	1000
Interface shear stiffness K_{sbp} (MPa/m)	23
Leveling pad–foundation soil interface	
Interface cohesion c_{pf} (kPa)	1.3
Interface friction angle φ_{pf} ($^\circ$)	43
Interface normal stress K_{nbf} (MPa/m)	1000
Interface shear stiffness K_{spf} (MPa/m)	10
Block–block interface	
Interface cohesion c_{bb} (kPa)	58
Interface friction angle φ_{bb} ($^\circ$)	36
Interface normal stress K_{nbb} (MPa/m)	1000
Interface shear stiffness K_{sbb} (MPa/m)	40
Block–backfill interface	
Interface cohesion c_{bs} (kPa)	1.3
Interface friction angle φ_{bs} ($^\circ$)	43
Interface normal stress K_{nbs} (MPa/m)	100
Interface shear stiffness K_{sbs} (MPa/m)	1

2.3. Validation Results

Figure 5 shows the computed and measured strains in the reinforcement layers that were installed with strain gauges. The computed strain was larger at the reinforcement connection and decreased along the reinforcement length in each layer. This distribution shape of the reinforcement strains agrees with the measurements, with the exception of those for Layer 8. The measured connection strain was the smallest in Layer 8 as a result of some looseness between the facing column and the reinforcement layer, which occurred during the wall's construction [10]. However, this looseness was not captured by the numerical model, which led to a difference in the computed and measured strains at the connection in Layer 8. In general, the computed reinforcement strains agree well with the measurements.

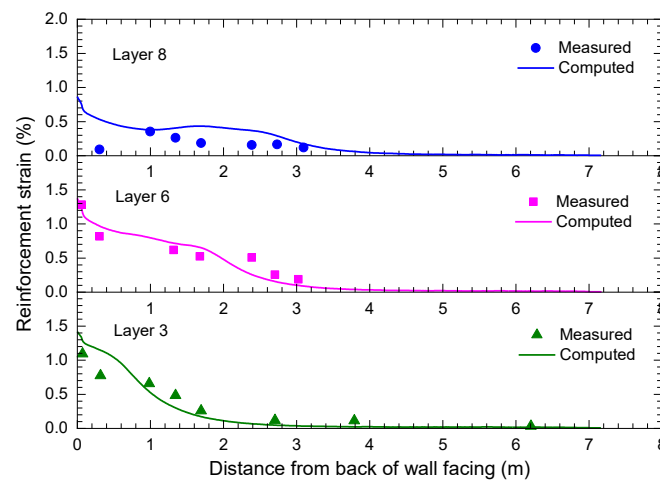


Figure 5. Computed and measured reinforcement strains of SR-18 wall.

Figure 6 shows the measured and computed connection loads T_{con} and the maximum reinforcement loads within the backfill T_{bmax} . The maximum reinforcement load beyond 1 m away from the wall facing was selected for the T_{bmax} value to avoid the influence of the down-drag force motivated by compaction near the connection. The computed values are in good agreement with the measured T_{con} and T_{bmax} . Only the computed T_{con} in Layer 8 is much larger than the measurement as a result of the looseness occurring at the connection in the field wall. The shear strength and stiffness at the facing block–leveling pad interface will significantly influence wall performance [19,20,32]. The encouraging agreement in the reinforcement strains and loads between the numerical and field SR-18 wall indicates that our proposed interface shear stiffness calculation model is reasonable.

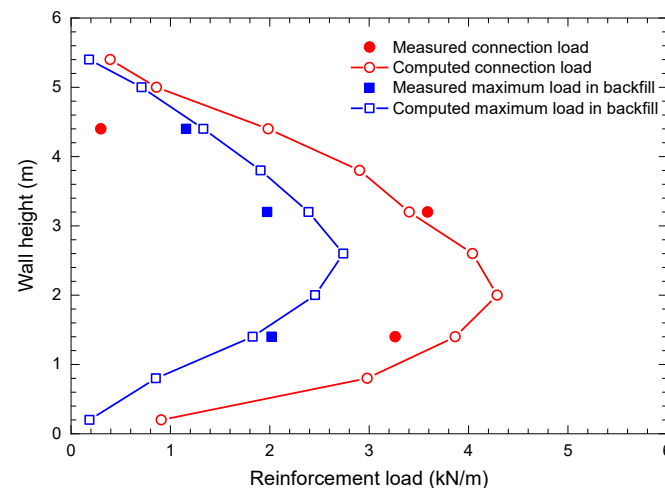


Figure 6. Computed and measured reinforcement loads of SR-18 wall.

3. Toe Load Analysis

3.1. Mechanical Model of the GRS Segmental Wall

In GRS retaining walls with segmental block facings, the integral thrust behind the wall facing is theoretically counterbalanced by the reinforcement connection loads and toe resistance if the facing is considered a beam. Thus, the mechanical model of the GRS segmental wall can be assumed as the following:

$$F_e = \sum T_{con} + F_t \quad (3)$$

where F_e = the integral thrust behind the facing, $\sum T_{con}$ = the total reinforcement connection load, and F_t = the horizontal toe load. Experimental and numerical studies have demonstrated that toe restraint condition has a significant impact on the loads carried by the wall toe and reinforcement layers [9,19–21,30,32,33].

The accuracy of Equation (3) and the impact of toe restraint on this equation was quantitatively analyzed using a numerical approach. The numerical model of the SR-18 wall was modified to have a more typical geometry (Figure 7) and more typical material parameter values (Table 4) for this analysis. The embedment soil at the toe was not included in the baseline numerical model, as shown in Figure 7, considering that the passive soil resistance in front of the toe is usually ignored in the conventional design method [15]. This is because the passive soil resistance may be lost unexpectedly and unpredictably by scouring and excavation during a long period of service. Nonetheless, a comparison of the connection loads in the SR-18 walls with and without the embedment soil is presented to demonstrate the effects of the embedment soil. Figure 8 shows that the influence of the embedment soil on the connection loads in the SR-18 wall can be ignored. Bathurst and Naftchali [45] reported the range of creep stiffnesses at a 2% strain for HDPE geogrids was approximately 330–1120 kN/m. The average value of this range (i.e., 725 kN/m) was selected for this numerical model. The value of the interface friction angle between reinforcement layers and backfill ϕ_{sr} , as shown in Table 4, was determined via the formula $\phi_{sr} = \tan^{-1} (2/3 \times \tan \phi)$, where ϕ = the backfill friction angle [41]. It is noted that parameters with the same values as those of the SR-18 wall (as shown in Tables 1–3) were not listed in Table 4.

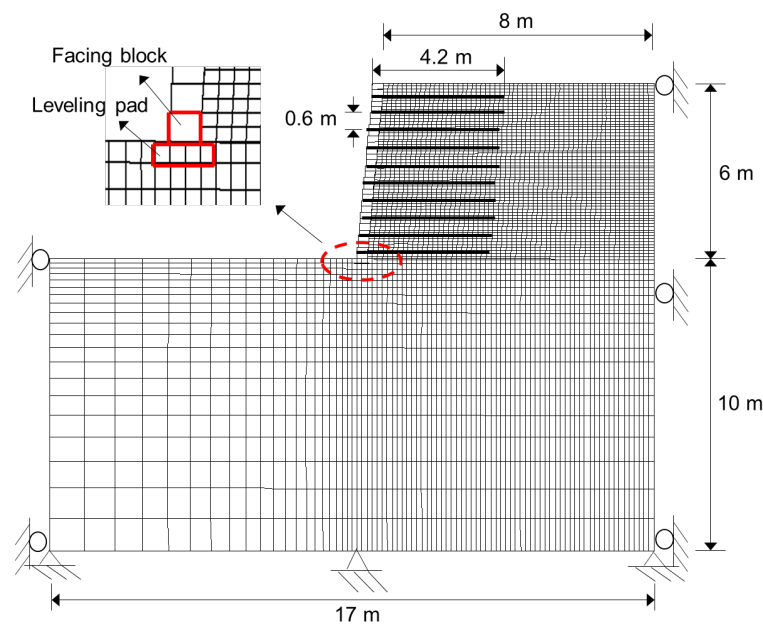
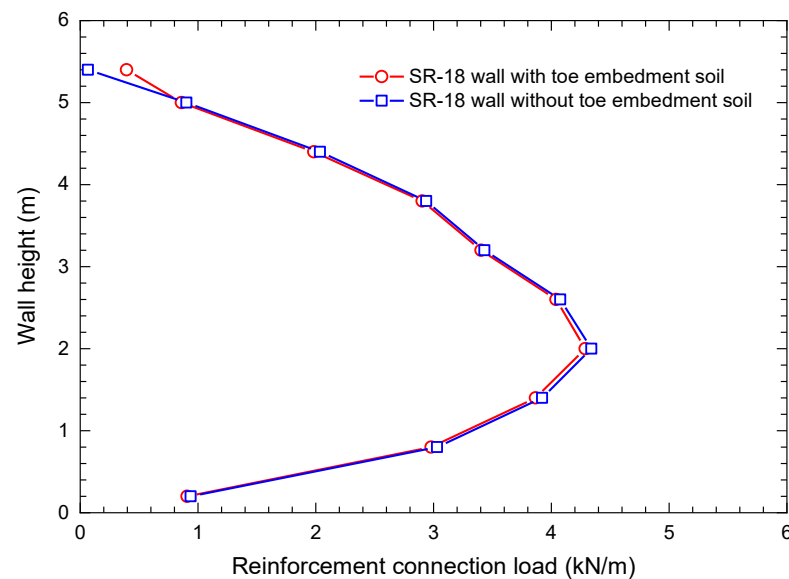


Figure 7. Baseline numerical model of GRS wall.

Table 4. Parameter values for baseline model.

Parameters	Value
Facing block	
Density ρ_b (kg/m ³)	2200
Backfill and foundation	
Density ρ (kg/m ³)	2000
Friction angle φ (°)	40
Dilation angle ψ (°)	10
Elastic modulus E (MPa)	40
Reinforcement	
Axial stiffness (kN/m)	725
Ultimate tensile strength T (kN/m)	115
Soil-reinforcement interface friction angle φ_{sr} (°)	30
Facing block–leveling pad interface	
Interface friction angle φ_{bp} (°)	Variable
Interface shear stiffness K_{sbp} (MPa/m)	Variable
Leveling pad–foundation soil interface	
Interface cohesion c_{pf} (kPa)	1
Interface friction angle φ_{pf} (°)	40
Block–block interface	
Interface cohesion c_{bb} (kPa)	46
Interface friction angle φ_{bb} (°)	57
Block–backfill interface	
Interface normal stress K_{nbs} (MPa/m)	100
Interface shear stiffness K_{sbs} (MPa/m)	1

**Figure 8.** Comparison of connection loads in SR-18 walls with and without toe embedment soil.

The toe restraint condition was quantified using the interface friction angle between the lowest facing block and the leveling pad. The shear strength of the leveling pad–foundation soil interface was not considered in this analysis as a result of its small effect on the capacity of the wall toe to carry load in cases where the leveling pad was embedded in the foundation soil. Zhang and Chen [46] found that, for GRS walls with embedded leveling pads, only the shear resistance at the facing block–leveling pad interface acts as the toe resistance to counterbalance a portion of the horizontal earth load because the passive soil resistance in front of the leveling pad inhibits the development of shear stress and displacement at the base of the leveling pad. In cases of an exposed leveling pad, it is the leveling pad–foundation soil interface that works to carry the earth load because the wall is more likely to slide along this weaker interface. However, even if the leveling pad

becomes exposed as a result of scour and soil erosion at the wall toe, the original leveling pad–foundation soil interface can be regarded as the facing block–leveling pad interface of a wall with the height of an added leveling pad, and this increase in wall height (usually 0.2 m) can be ignored. Hence, only the restraint condition at the facing block–leveling pad interface needs to be investigated. In this paper, the term “toe interface” refers to the interface between the facing block and the leveling pad. The selected values for the toe interface friction angle are presented in Table 5. A change in the interface friction angle influenced the interface shear stiffness, as shown in Figure 4. The interface shear stiffness values corresponding to the selected interface friction angles, determined via Equations (1) and (2), are also shown in Table 5.

Table 5. Parameter values for baseline model and parametric analysis.

Parameters	Value
Baseline model	
Toe interface friction angle (°)	5, 10, 15, 25, 45
Toe interface shear stiffness (MPa/m)	3.6, 10.1, 14.0, 19.1, 26.1
H = 4 m	
Toe interface friction angle (°)	5, 10, 15, 25, 45
Toe interface shear stiffness (MPa/m)	2.3, 7.4, 10.5, 14.5, 20.1
H = 8 m	
Toe interface friction angle (°)	5, 10, 15, 25, 45
Toe interface shear stiffness (MPa/m)	4.4, 12.6, 17.5, 24, 33
H = 10 m	
Toe interface friction angle (°)	5, 10, 15, 25, 45
Toe interface shear stiffness (MPa/m)	4.8, 14.9, 20.9, 28.9, 39.9

Figure 9 shows the loads carried by the wall toe and reinforcement layers and their sums for the different toe interface friction angles. The figure also shows the active earth pressure on the wall facing, calculated using the Coulomb earth pressure equation:

$$E_a = \frac{1}{2} K_a \gamma H^2 \quad (4)$$

where E_a = the active earth pressure, K_a = the active earth pressure coefficient, γ = the unit weight of the backfill, and H = the wall height. K_a is expressed as the following:

$$K_a = \frac{\cos^2(\varphi + \omega)}{\cos^3 \omega \cdot \left(1 + \frac{\sin \varphi}{\cos \omega}\right)^2} \quad (5)$$

where φ = the friction angle of the backfill and ω = the facing batter. Equation (5) is a reductive Coulomb active earth pressure coefficient, which is recommended by FHWA standard [15] for the internal stability analysis of GRS walls. The plane strain friction angle was used for the soil models in the numerical simulations of this paper, whereas a triaxial friction angle should be introduced for K_a [28]. Thus, a triaxial value of 38° was used for φ in Equation (5), according to the relationship between the plane strain and triaxial friction angles proposed by Lade and Lee [42]. It can be seen that the sum of the reinforcement and toe loads (i.e., $\Sigma T_{\text{con}} + F_t$) is closer to the calculated active earth pressure E_a than the total reinforcement connection load ΣT_{con} , but it is still smaller than E_a . The difference between $\Sigma T_{\text{con}} + F_t$ and E_a may be attributed to a small portion of the earth pressure that was counterbalanced by the friction resistances between the facing blocks and between the facing column and backfill soil. The resistance between the facing blocks was not considered in the mechanical model, as shown in Equation (3), for the convenience of application, which may lead to some conservativeness for the predictions of connection loads. It is also noted that the value of $\Sigma T_{\text{con}} + F_t$ is relatively small under toe interface friction angles of no more than 10° . This may be attributed to local soil failure

occurring within the reinforced soil mass, resulting in the earth load acting on the wall facing less than the active earth pressure, in the two weaker toe restraint cases. Local soil shear failure developing in GRS walls with poor toe restraint was also observed in the numerical modeling reported by Huang et al. [19].

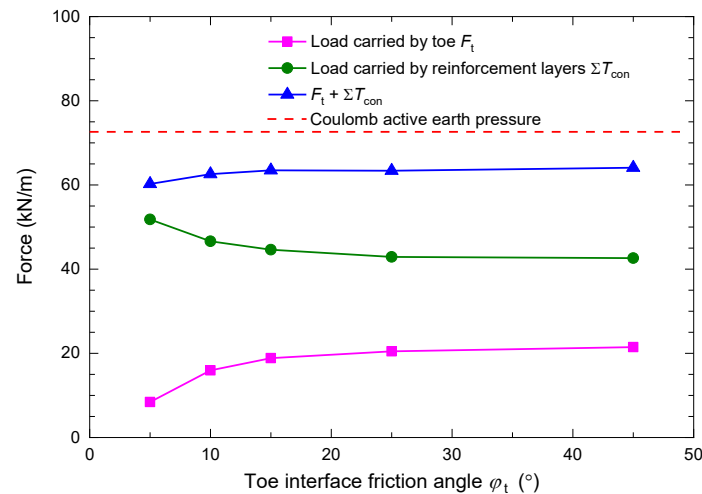


Figure 9. Loads carried by toe and reinforcement layers under different toe restraint conditions.

Figure 10 shows the ratios of the toe load (F_t) and reinforcement load (ΣT_{con}) to the total load ($\Sigma T_{con} + F_t$) under different toe restraint conditions. The toe load ratio increased with the toe interface friction angle, whereas the reinforcement load ratio decreased. The toe load ratio was 14.0% for a toe interface friction angle of 5° and increased to 29.7% when the toe interface friction angle was 15°. When the toe interface friction angle overpassed 15°, the toe load ratio increased slowly, reaching only 33.5% for a toe interface friction angle of 45°. This may have been caused by the influence of toe restraint on the horizontal facing displacements, as shown in Figure 11. When the toe restraint was weaker, a larger shear displacement between the lowest facing block and the leveling pad led to larger horizontal displacements in the bottom part of the wall. The larger, horizontal facing displacements resulted in high reinforcement connection loads and, thus, smaller toe loads. As the toe interface friction angle increased from 5° to 15°, the significant decrease in the facing displacements induced a quick increase in the toe load ratio. When the toe interface friction angle was in the 15–45° range, the negligible change in the facing displacements led to a slow increase in the toe load ratio.

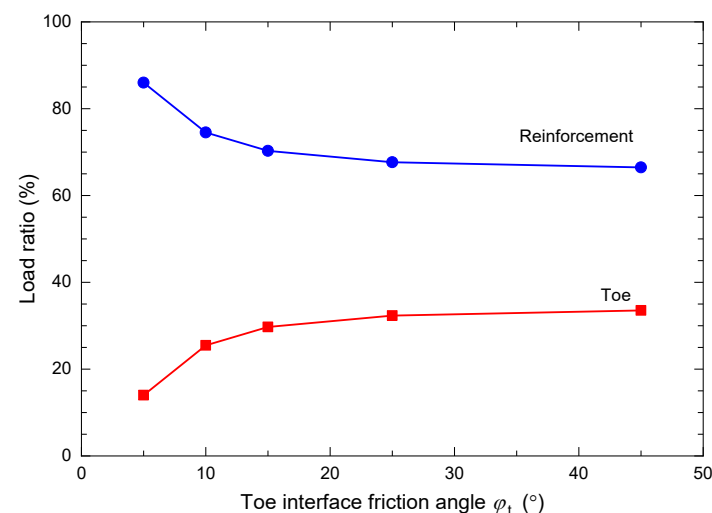


Figure 10. Ratios of toe load and reinforcement load under different toe restraint conditions.

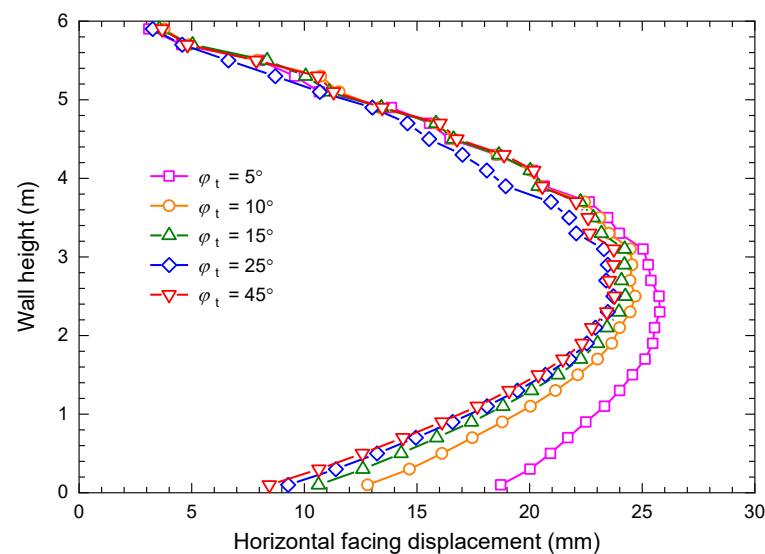


Figure 11. Horizontal facing displacements under different toe restraint conditions.

3.2. Influence Factors of the Toe Load Ratio

The capacity of the wall toe and reinforcement layers for carrying a load is expected to be associated with various factors. For example, a variation in wall height will induce a change not only to the earth load on the facing column but also to the shear stiffness of the toe interface, which may influence the distribution of the total load between the toe and reinforcement layers. To study the influence of different factors on the toe and reinforcement load ratio, we changed the factors of wall height, facing batter, reinforcement spacing and stiffness, and backfill friction angle based on the baseline numerical wall model presented in Figure 7. Tables 5 and 6 provide the factor values and other parameter values affected by these factors. The reinforcement tensile strength in Table 6 was determined using the linear relationship between the creep stiffness values and the ultimate tensile strength for HDPE geogrids, proposed by Bathurst and Naftchali [45]. When one factor was investigated, only the values of this factor and the involved parameters were changed, while the rest of the parameters were set to their baseline values.

Table 6. Values of influence factors.

Factors	Value
Wall height H (m)	4, 6, 8, 10
Facing batter ω (°)	0, 5, 10, 15
Reinforcement stiffness J (kN/m)	330, 725, 1120
Reinforcement tensile strength T (kN/m)	$J/6.3$
Reinforcement spacing (m)	0.4, 0.6, 0.8
Plane strain backfill friction angle φ (°)	35, 40, 45
Backfill dilation angle ψ (°)	$\varphi-30$
Block–backfill interface friction angle φ_{bs} (°)	φ
Leveling pad–foundation soil interface friction angle φ_{pf} (°)	φ
Soil–reinforcement interface friction angle φ_{sr} (°)	$\tan^{-1}(2/3 \times \tan \varphi)$

Figure 12 shows the toe and reinforcement load ratio versus the toe interface friction angle in walls with different heights. Since the variation characteristics of the reinforcement load ratio are opposite to those of the toe, hereafter, only the toe load ratio is discussed. Figure 12 shows that the toe load ratio decreased with wall height under the same toe restraint condition. For example, the toe load accounted for approximately 51% and 22% of the total load in the 4 m and 10 m high walls, respectively, when $\varphi_t = 45^\circ$. This is attributed to the larger reinforcement connection loads motivated by the larger facing displacements

and, thus, the smaller earth load transmitted to the toe interface in the taller walls. As the wall height increased, there was a diminishing influence of wall height on the toe load ratio.

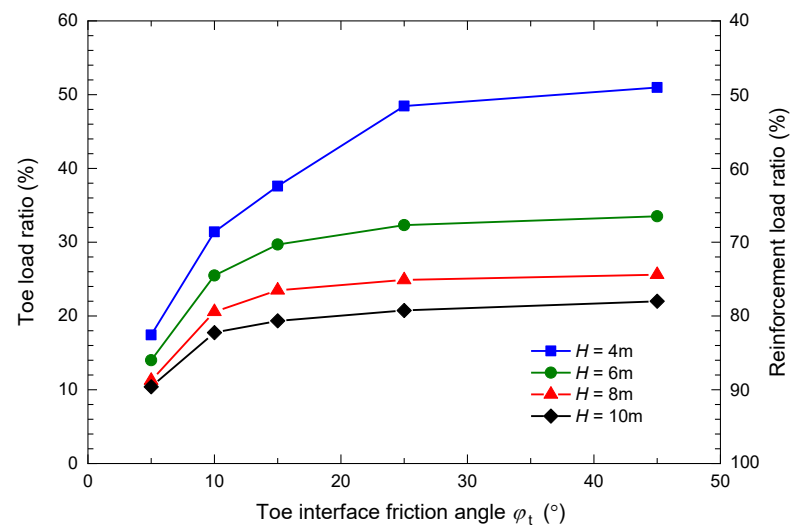


Figure 12. Influence of wall height H on the ratios of toe and reinforcement loads.

Figure 13 shows the effects of the facing batter on the ratio of the earth load carried by the toe and reinforcement layers. The figure shows that the toe load ratio increased with the facing batter under the same toe conditions. As the facing batter increased, the earth load acting on the facing column decreased, resulting in a decrease in the horizontal facing displacements. Lower facing displacements reduced reinforcement connection loads and, thus, the contribution of the reinforcement layers to carrying the earth load, which led to an increase in the toe load ratio. However, the facing batter had little effect on the toe load ratio if it was less than 5° .

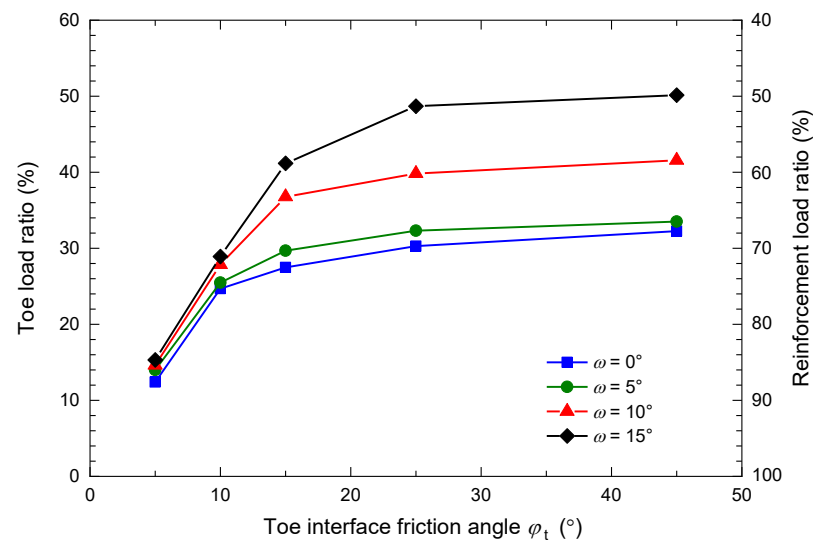


Figure 13. Influence of facing batter ω on the ratios of toe and reinforcement loads.

Figure 14 shows the influence of reinforcement stiffness on the relative contribution of wall toe and reinforcement layers to load capacity. The toe load ratio decreased as the reinforcement stiffness increased at the same toe interface friction angles, but at a diminishing rate. This is because the larger reinforcement stiffness led to a higher capacity of reinforcement layers for carrying loads, resulting in a lower toe load ratio.

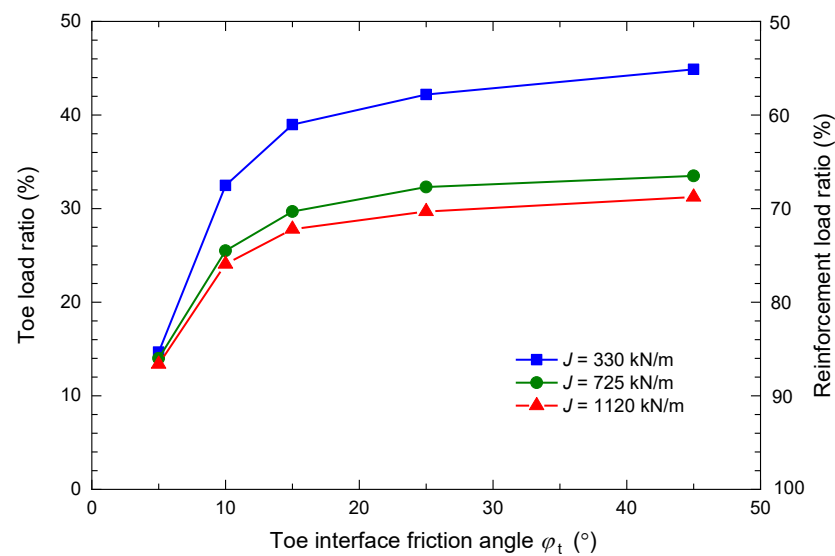


Figure 14. Influence of reinforcement stiffness J on the ratios of toe and reinforcement loads.

Figure 15 shows that reinforcement spacing had less influence on the toe load ratio when $\phi_t \leq 10^\circ$. For the range of $\phi_t = 15\text{--}45^\circ$, the toe load ratio increased with the reinforcement spacing. An increase in reinforcement spacing reduces global reinforcement stiffness [16], which induced a decrease in the fraction of the earth load carried by the reinforcement layers and an increase in the toe load ratio.

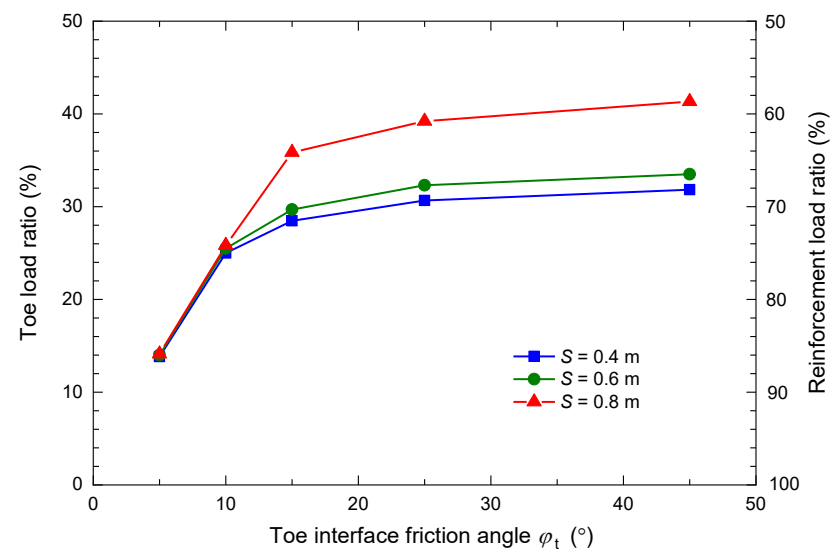


Figure 15. Influence of reinforcement spacing S on the ratios of toe and reinforcement loads.

The effects of the backfill friction angle is presented in Figure 16. The toe load ratio increased with the friction angle of the backfill under the same toe conditions. This is because of a larger normal stress at the toe interface, induced by an increase in the downward friction between the backfill and the wall facing when the backfill friction angle increased. The larger toe interface normal stress resulted in a further increase in the horizontal toe load.

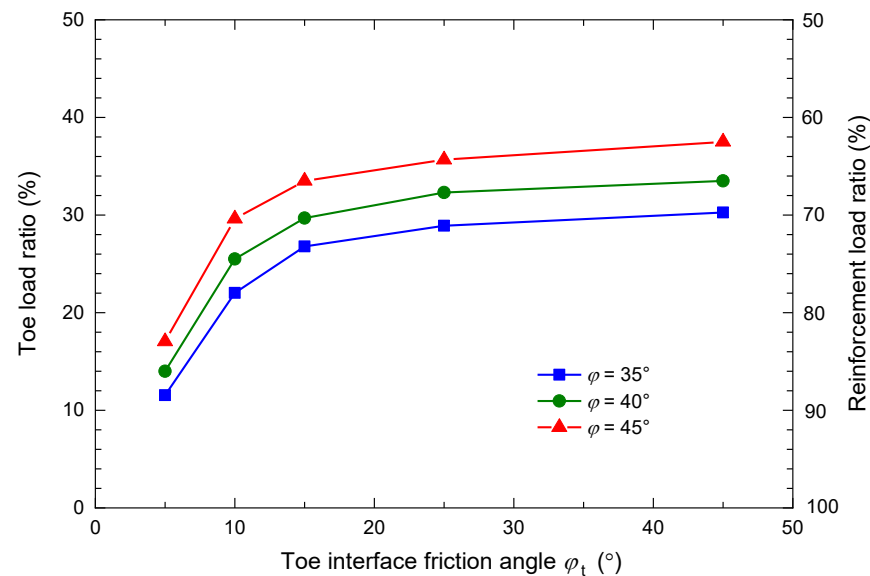


Figure 16. Influence of backfill friction angle ϕ on the ratios of toe and reinforcement loads.

The toe load ratio plotted in Figures 12–16 increased rapidly with toe interface friction angles in the 5–15° range and then changed only slightly in all cases. This indicates that the involved factors had little influence on the distribution shape of the toe load ratio with the toe interface friction angle. Moreover, at the lower toe interface friction angle of 5°, the toe load ratio was less than 20% in all the cases. In a field wall, the friction angles of the interfaces at the wall toe would change with the surface roughness of the facing block and leveling pad and the foundation soil properties. For GRS walls constructed along rivers or seashores or in mountains, the soil at the wall toe and even the toe itself may be scoured away by wave actions, flooding, or debris flows. Tatsuoka et al. [47] reported a practical case where the toe of a geosynthetic railway embankment in the mountains of Kyushu, Japan experienced strong scour and erosion caused by flooding. Tarawneh et al. [48] reported that about 13% of 339 reinforced soil walls supporting bridge abutments inspected in the state of Ohio, USA experienced soil erosion at the wall toe. In these extreme cases, the shear strength of the interfaces at the wall toe must have reduced sharply, resulting in a significant decrease in the toe resistance. Hence, for GRS walls constructed in mountains and by rivers and seashores, it is recommended to ignore the toe resistance to increase margin of safety against reinforcement overstressing. The typical friction angle of the toe interface can be considered as in the range of 25–45°, because the relatively constant ratio of toe load to the integral thrust behind the facing (almost no more than 50% in any condition) is observed in this range, as shown in Figures 12–16. However, in a series of 3.6 m high GRS test walls on a rigid foundation, it was observed that the toe carried nearly 80% of the earth load acting on the facing [9]. This indicates that a rigid foundation may magnify toe load capacity. In addition, the changes in the fractions of toe and reinforcement load with wall height, facing batter, reinforcement stiffness, and spacing in this study are consistent with the observations of other studies [19,21,31].

4. Prediction of Reinforcement Tensile Force

4.1. Calculation Method of Reinforcement Tensile Force

As previously analyzed, the total earth load acting on the facing column is carried by the toe and reinforcement layers together. It can be assumed that the connection load in each reinforcement layer is equal to a portion of the active earth pressures acting on the facing within the contributory area of this layer, which can be expressed as the following:

$$T_{\text{con},i} = K_t S(\gamma z + q) \quad (6)$$

where $T_{\text{con},i}$ = the connection load in a reinforcement layer; K_t = the lateral earth pressure coefficient, which is lower than the active earth pressure coefficient K_a in Equation (5); S = the vertical spacing between adjacent reinforcement layers; γ = the unit weight of backfill; z = the depth of a reinforcement layer below the wall top; and q = the surcharge pressure. According to the influence of toe restraint on the fraction of the total earth load carried by the reinforcement layers, K_t can be considered a function of K_a and the toe interface friction angle φ_t . Furthermore, the wall height H , facing batter ω , reinforcement stiffness J , reinforcement spacing S , and backfill friction angle φ should be considered in the calculation of K_t as a result of their influence on the reinforcement load ratio. In addition, Figure 17 shows that the toe restraint condition affects $T_{\text{con},i}$ differently at different depths; thus, the depth z should also be regarded as a factor related to the value of K_t . Therefore, K_t is a function of K_a , φ_t , H , ω , J , S , φ and z . The backfill cohesion c is not included in the function of K_t because the scope of the proposed method is limited to GRS walls constructed with cohesionless soil. It is noted that c should be considered in the prediction of connection loads in reinforced cohesive soil walls because of its significant effect on the lateral earth pressure.

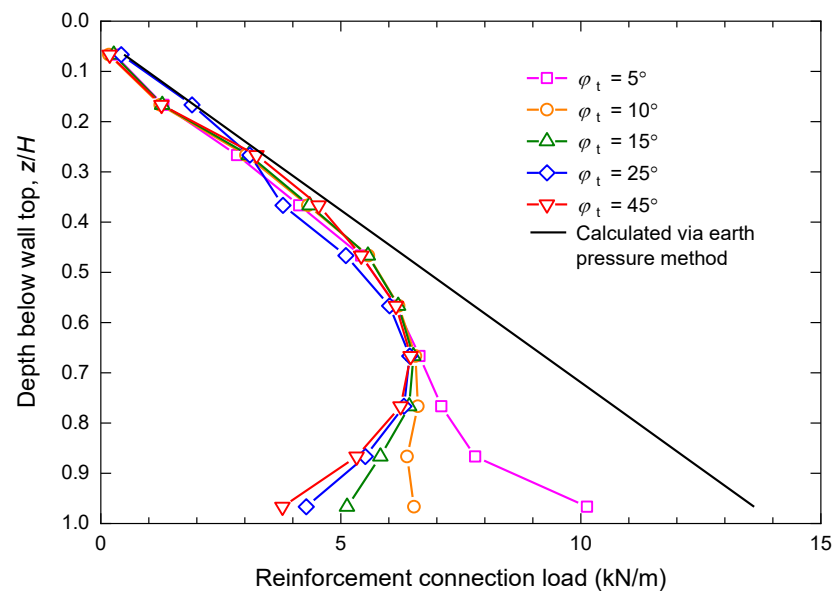


Figure 17. Reinforcement connection loads in baseline model with different toe interface friction angles.

4.2. Response Surface Model of K_t

Although it is difficult to obtain an explicit formulation of K_t , one can do so using the response surface method. The basic idea of the response surface method is to assume a function that includes unknown parameters to replace the implicit actual expression. For example, given the ambiguous functional relationship $Z = f(X_1, X_2, \dots, X_n)$ between the variable Z and the variables $X_1, X_2, \dots, X_n$, the response surface method helps to establish an approximate function $Z' = f'(X_1, X_2, \dots, X_n)$ instead of the actual function through regressing and fitting data from a limited number of experiments. We used the response surface approximation proposed by Myers and Montgomery [49]:

$$y = \beta_0 + \sum_{i=1}^m \beta_i x_i + \sum_{i=1}^m \beta_{ii} x_i^2 + \sum_{i=1}^{m-1} \sum_{j=i+1}^m \beta_{ij} x_i x_j \quad (7)$$

where y = the response value, β_0 , β_i , β_{ii} and β_{ij} = the unsolved coefficients, and x_i and x_j = variables. When y , x_i , and x_j are known, β_0 , β_i , β_{ii} , and β_{ij} can be determined using the least-squares method. In the RSM of K_t , K_t is regarded as the response value, and the active

earth pressure coefficient K_a , toe interface friction angle φ_t , wall height H , facing batter ω , reinforcement stiffness J , reinforcement spacing S , backfill friction angle φ , and depth of reinforcement below the wall top z are regarded as variables. To reduce the number of and normalize the variables, the six above-mentioned factors can be formalized into the three variables as follows:

$$x_1 = \frac{z}{H} \quad (8)$$

$$x_2 = \frac{K_a}{\tan \varphi_t} = \frac{\cos^2(\phi + \omega)}{\cos^3 \omega \cdot \left(1 + \frac{\sin \phi}{\cos \omega}\right)^2 \cdot \tan \varphi_t} \quad (9)$$

$$x_3 = \frac{S \cdot P}{J} \quad (10)$$

where the influences of ω and φ are considered through K_a and P = the normal atmospheric pressure, which is used to normalize the reinforcement stiffness J . Consequently, the RSM of K_t can be written as the following:

$$\begin{aligned} K_t = & \beta_0 + \beta_1 \left(\frac{z}{H}\right) + \beta_2 \left(\frac{K_a}{\tan \varphi_t}\right) + \beta_3 \left(\frac{S \cdot P}{J}\right) \\ & + \beta_{11} \left(\frac{z}{H}\right)^2 + \beta_{22} \left(\frac{K_a}{\tan \varphi_t}\right)^2 + \beta_{33} \left(\frac{S \cdot P}{J}\right)^2 \\ & + \beta_{12} \left(\frac{z}{H}\right) \left(\frac{K_a}{\tan \varphi_t}\right) + \beta_{23} \left(\frac{K_a}{\tan \varphi_t}\right) \left(\frac{S \cdot P}{J}\right) + \beta_{13} \left(\frac{z}{H}\right) \left(\frac{S \cdot P}{J}\right) \end{aligned} \quad (11)$$

A total of 705 values for the connection loads in different reinforcement layers were computed from the 65 numerical GRS walls mentioned previously and were used to establish a database of $T_{\text{con},i}$. A corresponding database of K_t can further be set up using Equation (6). To do so, substitute the values of K_t and the corresponding values of z , H , K_a , S , J , and φ_t into Equation (11). Then, the values of the unknown coefficients β_0 , β_i , β_{ii} , and β_{ij} in Equation (11) can be determined using the least-squares method. Table 7 shows the values of β_0 , β_i , β_{ii} , and β_{ij} . It is noted that in the process of producing the unknown coefficients, the triaxial friction angle values of the backfill (corresponding to the plane strain friction angles selected in numerical modeling) were used in order to make the method user-friendly in practical engineering. This means that the experimental value of the backfill friction angle can be used directly in Equation (11).

Table 7. Values for coefficients of response surface model.

Coefficient	Value	Coefficient	Value
β_0	0.0337	β_5	0.0064
β_1	0.3934	β_6	−1.1232
β_2	−0.0147	β_7	0.0222
β_3	0.5549	β_8	−0.0129
β_4	−0.3731	β_9	−0.4998

4.3. Calculation Result Analysis

Figure 18 presents the 705 values of $T_{\text{con},i}$ calculated using the proposed RSM method versus the numerically computed connection loads, where the proposed method refers to Equation (6) combined with Equation (11) and Table 7. The bias of the proposed RSM method is defined as the ratio of the numerical value to the calculated value:

$$\lambda = \frac{T_{\text{con},n}}{T_{\text{con},c}} \quad (12)$$

where λ = the bias, $T_{\text{con},n}$ = the numerically calculated value of T_{con} , and $T_{\text{con},c}$ = the calculated T_{con} using the proposed method. The mean μ_λ and the coefficient of variation COV_λ of all the biases are 1.0 and 0.41, respectively. Figure 18 shows an even distribution of the data points on both sides of the diagonal with a slope of 1. Moreover, most of the

data points are within the boundaries of $\pm 2\text{COV}_\lambda$ on both sides of the diagonal. In a simplified and approximated manner, the proposed method can estimate the reinforcement connection loads in GRS segmental walls with different toe restraint conditions.

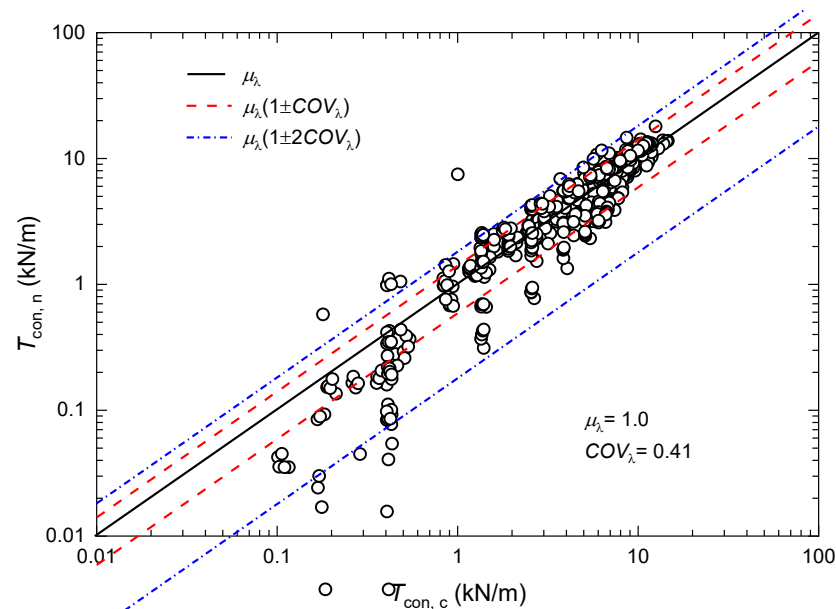


Figure 18. Comparison between the calculated values from the proposed RSM method ($T_{\text{con},c}$) and the connection loads from numerical computation ($T_{\text{con},n}$).

The proposed RSM method was also validated against the measured reinforcement connection loads from five instrumented GRS segmental walls. Table 8 presents information on the five walls. Walls 1, 2, and 3 are the prototype walls corresponding to three centrifuge models with different toe restraint conditions reported by Zhang et al. [32]. Walls 4 and 5 are the field walls reported by Allen and Bathurst [10,11]. Among the five GRS walls, Walls 1, 4, and 5 had normal toe restraints, with toe interface friction angles greater than 25° , while Walls 2 and 3 exhibited poor toe restraints. Figures 19–23 compare the measured connection loads and the values calculated using the RSM method, the classical earth pressure method, and the stiffness method for the five walls. As shown in Figures 19–23, the values calculated using the RSM method are closer to the measurements than those using the other two methods for all the walls.

Table 8. Information on physical GRS segmental walls.

Parameter	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Wall height (m)	3.6	3.6	4.1	10.7	6.1
Facing batter ($^\circ$)	8	8	8	0	0
Number of reinforcement layers	5	5	5	17	10
Reinforcement spacing (m)	0.6	0.6	0.6	0.6	0.6
Reinforcement stiffness (kN/m)	572	572	572	246 for the top 4 layers 393 for the middle 3 layers 598 for the bottom 10 layers	232
Backfill weight (kN/m ³)	15.7	15.7	15.7	21.7	21.7
Backfill friction angle from triaxial or direct shear test ($^\circ$)	36	36	36	47	47
Toe interface friction angle ($^\circ$)	39	13	8	36	36

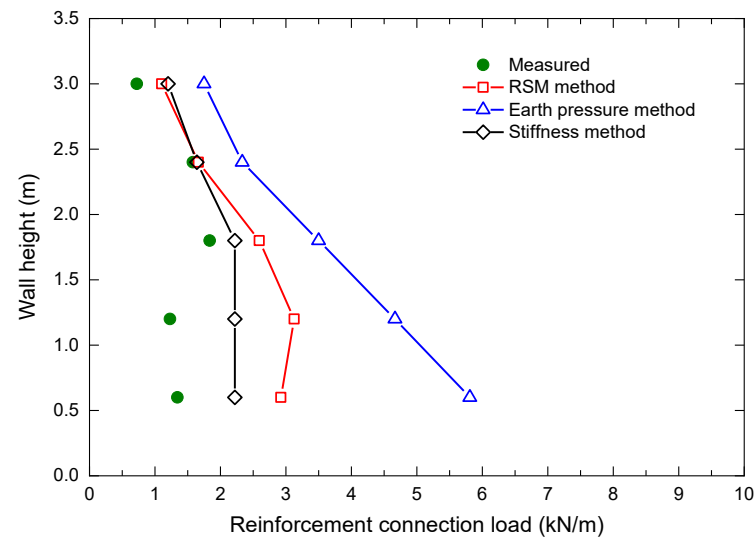


Figure 19. Measured and calculated connection loads in Wall 1.

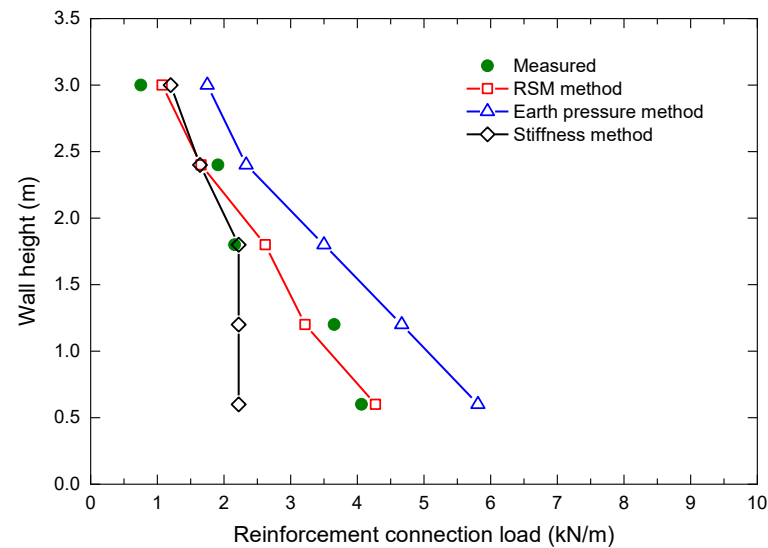


Figure 20. Measured and calculated connection loads in Wall 2.

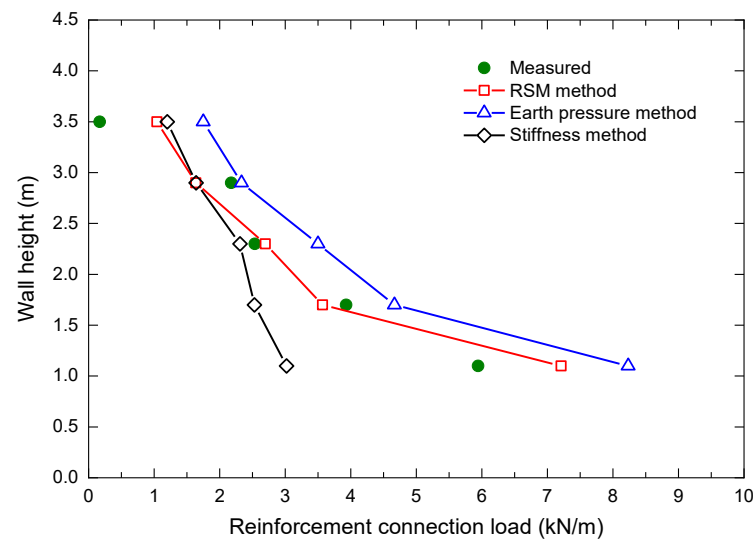


Figure 21. Measured and calculated connection loads in Wall 3.

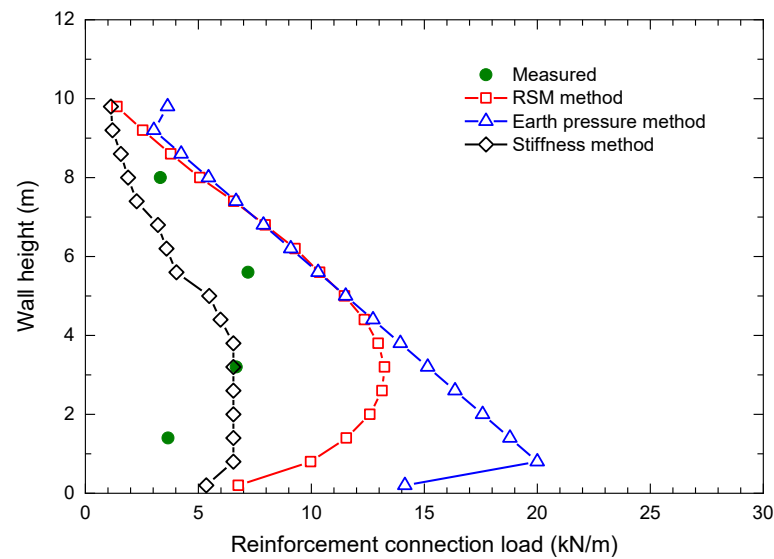


Figure 22. Measured and calculated connection loads in Wall 4.

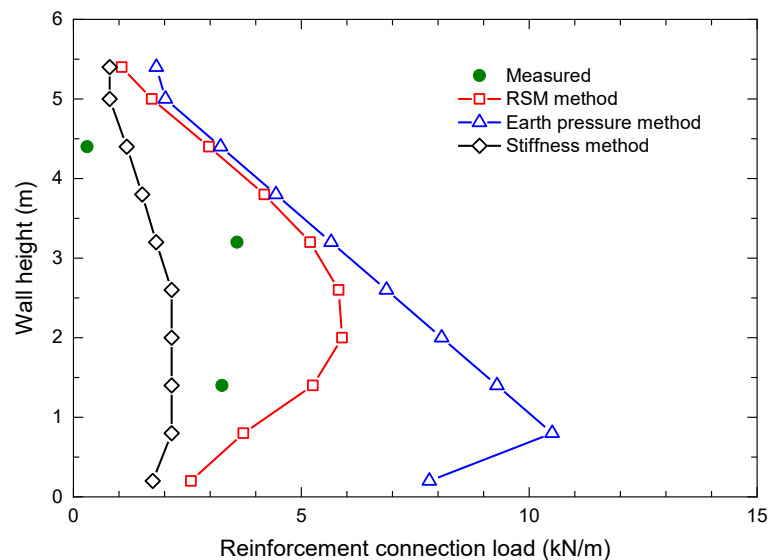


Figure 23. Measured and calculated connection loads in Wall 5.

In Walls 1, 4, and 5, which were under typical toe conditions (Figures 19, 22 and 23), the RSM method-calculated connection loads showed a bulge shape in the lower middle of the wall, which agrees with the connection load distribution in the GRS walls under working stress conditions [12,17,50]. This distribution shape is attributed to the primary contribution of the toe resistance to counterbalancing the earth pressure within the bottom of the wall. The calculated values of the stiffness method show a trapezoidal shape with wall height, but most of them are less than the measurements, especially for the higher walls (Walls 4 and 5). For example, at a height of three meters in Wall 5, the calculated value using the stiffness method is approximately half of the measured value, which is unsafe for design. This is because the stiffness method is deduced from the maximum reinforcement loads in backfill instead of the connection loads. The earth pressure method gives an unrealistic, approximate linear distribution of the connection loads and seriously overestimates the connection loads in the middle and bottom of these three walls. In the bottom of these three walls, the calculated connection loads using the earth pressure method are approximately three times the measured values. This is primarily due to the neglect of the contribution of a restrained toe to carrying earth load in the earth pressure method. In comparison, the RSM method gives more accurate predictions than the above two methods. Nevertheless, the

RSM method still overestimates the connection loads in the middle and bottom parts of the three walls. Within the middle and bottom of these three walls, the RSM method-calculated values are approximately twice the measured values.

The weaker toe restraints in Walls 2 and 3 resulted in an increasing distribution with a depth of the connection loads that is well predicted using the limit equilibrium-based earth pressure method (Figures 20 and 21). The earth pressure method also gives predicted values that agree well with the measurements of the two walls. The stiffness method seriously underestimates the connection loads at the bottom parts of the two walls. For the bottom reinforcement layer of Wall 3, the calculated value using the stiffness method is only half of the measured one. The values calculated using the RSM method show a distribution shape consistent with the measured values, while they are slightly less than the measured ones at some depths (the amplitude is less than 13%).

The proposed RSM method is an easy-to-use reinforcement connection load prediction method for GRS segmental walls that can consider the effects of multiple factors. The proposed method adds two extra parameters (i.e., the interface friction angle between the lowest facing block and the leveling pad φ_t and the reinforcement stiffness J) to the classical earth pressure method. The toe interface friction angle φ_t can be obtained using a simple tilt table test on two concrete blocks [51]. An empirical value can also be assigned to φ_t (e.g., a value between 30° and 40°). However, it should be noted that the value of φ_t must be determined considering the engineering geological conditions of the construction site. If the construction site might experience exogenic geologic processes, such as river erosion or mudslides, the real value of φ_t can be used. If GRS walls are built by rivers, in mountains, or in flood detention areas, the toe will most likely be eroded via scouring, leading to a sharp reduction in its resistance. In this case, it is recommended to set the value of φ_t at less than 15° to seek higher safety margins.

5. Discussion

As stated previously, T_{con} is contributed by the down-drag force induced by backfill compaction, facing rotation, and the differential settlement of a foundation. Unfortunately, the proper calculation of the down-drag force is difficult, and thus, a simple prediction method for T_{con} is still lacking at present. This paper proposes an RSM method for predicting T_{con} based on the back-analysis of numerical GRS wall models that considers the effects of backfill compaction and facing rotation, but not the effects of foundation settlement since the foundation of the wall models is assumed to be rigid.

At present, T_{bmax} that is calculated using the earth pressure method is thought to equal T_{con} since it is seriously overestimated compared to the actual values [8]. Nevertheless, T_{con} could still be larger than T_{bmax} that is calculated based on the limit equilibrium theory when down-drag forces are very significant. Therefore, rupture failures may occur at reinforcement connections if only T_{bmax} is considered in design. At this point, T_{con} is a necessary design parameter for hard-faced GRS walls and needs a prediction method that considers the effects of down-drag force. Certainly, the current RSM method needs further optimization to have more practical value, such as improving model accuracy, extending the connection load database using data from field GRS walls, and including the effects of the differential settlement of yielding foundation.

6. Conclusions

This paper proposed an RSM method for predicting the connection load of GRS segmental walls constructed with cohesionless backfill and on competent foundations. This method replaced the active earth pressure coefficient used in the classical earth pressure method with an RSM of the lateral earth pressure coefficient. The RSM was established using multiple parameters that affect the distribution of the total earth load between wall toe and reinforcement layers. The unknown coefficients of the RSM were determined using the regression analysis of 705 connection load values from 65 numerical GRS segmental wall models. The measured connection loads from two field walls and three centrifuge

test walls were utilized to verify the proposed method. The following conclusions can be drawn from this study:

- (1) In GRS segmental walls with a typical toe restraint, the sum of the total connection load and toe resistance is close to the Coulomb active earth load acting on the facing column.
- (2) The ratio of the earth loads carried by the toe increases with the friction angle of the facing block–leveling pad interface but shows little change for interface friction angles greater than 25°.
- (3) Under the same toe restraint conditions, the toe load ratio increases with the facing batter, reinforcement spacing, and backfill friction angle, and decreases with wall height and reinforcement stiffness. In most cases in this paper, the toe load ratio was not higher than 50%.
- (4) No matter what toe restraint conditions, the predictions of the proposed RSM method are in better agreement with the measured connection loads than those of the stiffness method and the earth pressure method.
- (5) For GRS walls with proper toe restraint conditions, the reinforcement connection loads calculated using the proposed method show a bulge distribution shape, which is consistent with observations in GRS walls under working stress conditions.
- (6) For GRS walls with the loss of toe resistance, the proposed method yields a positive, approximately linear relationship between the connection loads and wall depth, which is close to the results of the earth pressure method.
- (7) The proposed method cannot consider the effects of the cohesion of backfill soil and the down-drag force induced by differential settlement of foundations on the connection loads and needs further optimization.

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References

1. Xue, Y.; Liu, S.; Chai, J.; Liu, J.; Ranjith, P.G.; Cai, C.; Bai, X. Effect of water-cooling shock on fracture initiation and morphology of high-temperature granite: Application of hydraulic fracturing to enhanced geothermal systems. *Appl. Energy* **2023**, *337*, 120858. [\[CrossRef\]](#)
2. Xue, Y.; Ranjith, P.G.; Gao, F.; Zhang, Z.; Wang, S. Experimental investigations on effects of gas pressure on mechanical behaviors and failure characteristic of coals. *J. Rock Mech. Geotech. Eng.* **2023**, *15*, 412–428. [\[CrossRef\]](#)
3. Chen, Y.; Tan, L.; Xiao, N.; Liu, K.; Jia, P.; Zhang, W. The hydro-mechanical characteristics and micro-structure of loess enhanced by microbially induced carbonate precipitation. *Geomech. Energy Environ.* **2023**, *34*, 100469. [\[CrossRef\]](#)
4. Wang, L.; Dong, Z.; Li, N.; Wang, W.; Tian, Y.; Xu, S. Experimental study on the mechanism of ice layer formation in fractured rock masses in cold regions. *Sci. Rep.* **2022**, *12*, 17954. [\[CrossRef\]](#)
5. Chen, Y.; Zhao, W.; Jia, P.; Han, J.; Guan, Y. Dynamic behavior of an embedded foundation under horizontal vibration in a poroelastic half-space. *Appl. Sci.* **2019**, *9*, 740. [\[CrossRef\]](#)
6. Tatsuoka, F. Recent practice and research of geosynthetic-reinforced earth structures in Japan. *J. GeoEng.* **2008**, *3*, 77–100.
7. Allen, T.M.; Bathurst, R.J.; Holtz, R.D.; Walters, D.L.; Lee, W.F. A new working stress method for prediction of reinforcement loads in geosynthetic walls. *Can. Geotech. J.* **2003**, *40*, 976–994. [\[CrossRef\]](#)
8. Bathurst, R.J.; Miyata, Y.; Nernheim, A.; Allen, A.M. Refinement of K-stiffness method for geosynthetic reinforced soil walls. *Geosynth. Int.* **2008**, *15*, 269–295. [\[CrossRef\]](#)
9. Bathurst, R.J.; Vlachopoulos, N.; Walters, D.L.; Burgess, P.G.; Allen, T.M. The influence of facing stiffness on the performance of two geosynthetic reinforced soil retaining walls. *Can. Geotech. J.* **2006**, *43*, 1225–1237. [\[CrossRef\]](#)
10. Allen, T.M.; Bathurst, R.J. Design and performance of 6.3-m-high, block-faced geogrid wall design using K-stiffness method. *J. Geotech. Geoenviron. Eng.* **2014**, *140*, 04013016. [\[CrossRef\]](#)

11. Allen, T.M.; Bathurst, R.J. Performance of an 11 m high block-faced geogrid wall designed using the K-stiffness Method. *Can. Geotech. J.* **2014**, *51*, 16–29. [[CrossRef](#)]
12. Yu, Y.; Bathurst, R.J.; Allen, T.M.; Nelson, N. Physical and numerical modelling of a geogrid-reinforced incremental concrete panel retaining wall. *Can. Geotech. J.* **2016**, *53*, 1883–1901. [[CrossRef](#)]
13. Xiao, C.; Gao, S.; Liu, H.; Du, Y. Case history on failure of geosynthetics-reinforced soil bridge approach retaining walls. *Geotext. Geomembr.* **2021**, *49*, 1585–1599. [[CrossRef](#)]
14. Gebremariam, F.; Tanyu, B.F.; Christopher, B.; Leshchinsky, D.; Zornberg, J.G.; Han, J. Evaluation of required connection load in GRS-IBS structures under service loads. *Geosynth. Int.* **2020**, *27*, 620–634. [[CrossRef](#)]
15. FHWA. *Design of Mechanically Stabilized Earth Walls and Reinforced Slopes*; No. FHWA-NHI-10-024; Berg, R.R., Christopher, B.R., Samtani, N.C., Eds.; Federal Highway Administration: Washington, DC, USA, 2009; Volume I.
16. Allen, T.M.; Bathurst, R.J. Improved simplified method for prediction of loads in reinforced soil walls. *J. Geotech. Geoenviron. Eng.* **2015**, *141*, 04015049. [[CrossRef](#)]
17. Hatami, K.; Bathurst, R.J. Development and verification of a numerical model for the analysis of geosynthetic-reinforced soil segmental walls under working stress conditions. *Can. Geotech. J.* **2005**, *42*, 1066–1085. [[CrossRef](#)]
18. Hatami, K.; Bathurst, R.J. A numerical model for reinforced soil segmental walls under surcharge loading. *J. Geotech. Geoenviron. Eng.* **2006**, *132*, 673–684. [[CrossRef](#)]
19. Huang, B.; Bathurst, R.J.; Hatami, K. Influence of toe restraint on reinforced soil segmental walls. *Can. Geotech. J.* **2010**, *47*, 885–904. [[CrossRef](#)]
20. Ehrlich, M.; Mirmoradi, S.H. Evaluation of the effects of facing stiffness and toe resistance on the behavior of GRS walls. *Geotext. Geomembr.* **2013**, *40*, 28–36. [[CrossRef](#)]
21. Mirmoradi, S.H.; Ehrlich, M.; Dieguez, C. Evaluation of the combined effect of toe resistance and facing inclination on the behavior of GRS walls. *Geotext. Geomembr.* **2016**, *44*, 287–294. [[CrossRef](#)]
22. Leshchinsky, D.; Ling, H.; Hanks, G. Unified design approach to geosynthetic reinforced slopes and segmental walls. *Geosynth. Int.* **1995**, *2*, 845–881. [[CrossRef](#)]
23. Liu, H.; Yang, G.; Hung, C. Analyzing reinforcement loads of vertical geosynthetic-reinforced soil walls considering toe restraint. *Int. J. Geomech.* **2017**, *17*, 04016140. [[CrossRef](#)]
24. Leshchinsky, D.; Zhu, F.; Meehan, C.L. Required unfactored strength of geosynthetic in reinforced earth structures. *J. Geotech. Geoenviron. Eng.* **2010**, *136*, 281–289. [[CrossRef](#)]
25. Allen, T.M.; Bathurst, R.J. Application of the Simplified Stiffness Method to design of reinforced soil walls. *J. Geotech. Geoenviron. Eng.* **2018**, *144*, 04018024. [[CrossRef](#)]
26. Xie, Y.; Leshchinsky, B.; Yang, S. Evaluating reinforcement loading within surcharged segmental block reinforced soil walls using a limit state framework. *Geotext. Geomembr.* **2016**, *44*, 832–844. [[CrossRef](#)]
27. Franco, Y.B.; Utili, S.; Da Silva, J.L. Design of reinforced cohesive soil walls accounting for wall facing contribution to stability. *Géotechnique*, 2022, in press. [[CrossRef](#)]
28. AASHTO (American Association of State Highway and Transportation Officials). *LRFD Bridge Design Specifications*, 9th ed.; American Association of State Highway and Transportation Officials: Washington, DC, USA, 2020; p. 1912.
29. Leshchinsky, D.; Vahedifard, F. Impact of toe resistance in reinforced masonry block walls: Design dilemma. *J. Geotech. Geoenviron. Eng.* **2012**, *138*, 236–240. [[CrossRef](#)]
30. Mirmoradi, S.H.; Ehrlich, M.; Magalhães, L.F.O. Numerical evaluation of the effect of foundation on the behaviour of reinforced soil walls. *Geotext. Geomembr.* **2021**, *49*, 619–628. [[CrossRef](#)]
31. Mirmoradi, S.H.; Ehrlich, M. Effects of facing, reinforcement stiffness, toe resistance, and height on reinforced walls. *Geotext. Geomembr.* **2017**, *45*, 67–76. [[CrossRef](#)]
32. Zhang, W.; Chen, J.F.; Yu, Y. Influence of toe restraint conditions on performance of geosynthetic-reinforced soil retaining walls using centrifuge model tests. *Geotext. Geomembr.* **2019**, *47*, 653–661. [[CrossRef](#)]
33. Xu, P.; Hatami, K.; Yang, G.; Li, T.; Liang, X. Influence of facing toe condition on the bearing capacity of full-height panel MSE walls. *Geosynth. Int.* **2022**, *29*, 593–609. [[CrossRef](#)]
34. Sayed, S.; Dodagoudar, G.R.; Rajagopal, K. Finite element reliability analysis of reinforced retaining walls. *Geomech. Geoenviron. Int. J.* **2010**, *5*, 187–197. [[CrossRef](#)]
35. Yu, Y.; Bathurst, R.J. Probabilistic assessment of reinforced soil wall performance using response surface method. *Geosynth. Int.* **2017**, *24*, 524–542. [[CrossRef](#)]
36. Peng, M.; Sun, R.; Chen, J.F.; Rajesh, S.; Zhang, L.M.; Yu, S.B. System reliability analysis of geosynthetic reinforced soil slope considering local reinforcement failure. *Comput. Geotech.* **2020**, *123*, 103563. [[CrossRef](#)]
37. Pramanik, R.; Mukherjee, S.; Babu, G.S. Deterministic and probabilistic prediction of maximum wall facing displacement of geosynthetic-reinforced soil segmental walls using multivariate adaptive regression splines. *Transp. Geotech.* **2022**, *36*, 100816. [[CrossRef](#)]
38. Lee, K.Z.Z.; Chang, N.Y. Predictive modeling on seismic performances of geosynthetic-reinforced soil walls. *Geotext. Geomembr.* **2012**, *35*, 25–40. [[CrossRef](#)]
39. Lin, B.H.; Yu, Y.; Bathurst, R.J.; Liu, C.N. Deterministic and probabilistic prediction of facing deformations of geosynthetic-reinforced MSE walls using a response surface approach. *Geotext. Geomembr.* **2016**, *44*, 813–823. [[CrossRef](#)]

40. Bathurst, R.J.; Yu, Y. Probabilistic prediction of reinforcement loads for steel MSE walls using a response surface method. *Int. J. Geomech.* **2018**, *18*, 04018027. [[CrossRef](#)]
41. Yu, Y.; Bathurst, R.J.; Allen, T.M. Numerical modeling of the SR-18 geogrid reinforced modular block retaining walls. *J. Geotech. Geoenviron. Eng.* **2016**, *142*, 04016003. [[CrossRef](#)]
42. Lade, P.V.; Lee, K.L. *Engineering Properties of Soils*; Report UCLA-ENG-7652; University of California: Los Angeles, CA, USA, 1976.
43. Damians, I.P.; Bathurst, R.J.; Josa, A.; Lloret, A.; Albuquerque, P.J.R. Vertical-facing loads in steel-reinforced soil walls. *J. Geotech. Geoenviron. Eng.* **2013**, *139*, 1419–1432. [[CrossRef](#)]
44. Mirmoradi, S.H.; Ehrlich, M. Numerical evaluation of the behavior of GRS walls with segmental block facing under working stress conditions. *J. Geotech. Geoenviron. Eng.* **2015**, *141*, 04014109. [[CrossRef](#)]
45. Bathurst, R.J.; Naftchali, F.M. Geosynthetic reinforcement stiffness for analytical and numerical modelling of reinforced soil structures. *Geotext. Geomembr.* **2021**, *49*, 921–940. [[CrossRef](#)]
46. Zhang, W.; Chen, J.F. Numerical modeling of geosynthetic reinforced soil retaining walls with different toe restraint conditions. *Geotext. Geomembr.* **2023**, *51*, 16–29. [[CrossRef](#)]
47. Tatsuoka, F.; Tateyama, M.; Koseki, J.; Yonezawa, T. Geosynthetic-reinforced soil structures for railways in Japan. *Transp. Infrastruct. Geotechnol.* **2014**, *1*, 3–53. [[CrossRef](#)]
48. Tarawneh, B.; Bodour, B.A.; Masada, T. Inspection and risk assessment of mechanically stabilized earth walls supporting bridge abutments. *J. Perform. Constr. Facil.* **2018**, *32*, 04017131. [[CrossRef](#)]
49. Myers, R.H.; Montgomery, D.C. *Response Surface Methodology: Process and Product Optimization using Designed Experiments*, 2nd ed.; John Wiley and Sons: New York, NY, USA, 2002.
50. Miyata, Y.; Bathurst, R.J.; Miyatake, H. Performance of three geogrid-reinforced soil walls before and after foundation failure. *Geosynth. Int.* **2015**, *22*, 311–326. [[CrossRef](#)]
51. Xiao, C.; Han, J.; Zhang, Z. Experimental study on performance of geosynthetic-reinforced soil model walls on rigid foundations subjected to static footing loading. *Geotext. Geomembr.* **2016**, *44*, 81–94. [[CrossRef](#)]

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