Seismic Responses of GRS Walls with Secondary Reinforcements Subjected to Earthquake Loading

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Abstract: Secondary reinforcement has been proven to be effective in increasing the performance of geosynthetic-reinforced soil (GRS) walls under working stress conditions, enabling an eco-friendlier environment. However, the seismic responses of GRS walls with secondary reinforcements are still unclear. In this study, in-depth finite element analyses were used to investigate the seismic responses of GRS walls with secondary reinforcement subjected to earthquake motions. The numerical procedure was first validated using measurements obtained from both a field GRS wall with secondary reinforcement and benchmark large-scale shaking table tests. Then, the validated GRS walls procedure was utilized to explore the effects of secondary reinforcement length and stiffness, the vertical spacing of the primary reinforcement, and wall height on the seismic responses. Based on the study, the following findings can be drawn: (i) the secondary reinforcement length and stiffness under various wall heights and peak ground accelerations (PGAs) have a limited influence on the relative lateral facing displacement and acceleration amplification, however, they can significantly decrease the connection load and the maximum reinforcement load; (ii) increasing the length of the secondary reinforcement is more effective for reducing the connection load and the maximum reinforcement load than increasing the stiffness of the secondary reinforcement; (iii) the effect of secondary reinforcement is more evident for greater wall height, the larger vertical spacing of primary reinforcement, and smaller PGA; and (iv) GRS walls with secondary reinforcement could ease the acceleration amplification. The study has highlighted the salient effect of secondary reinforcement on GRS wall performance under seismic conditions.

Keywords: geosynthetics; reinforced soil retaining wall; secondary reinforcement; seismic response; finite element analyses

1. Introduction

Geosynthetic-reinforced soil (GRS) walls have been successfully used in many practical applications as alternatives to conventional retaining walls due to their good performance, lower cost, and economic efficiency. Several guidelines for the design of practical GRS walls are available from the American Association of State Highway and Transportation Officials (AASHTO) [1], the British Standards Institution (BSI; BS 8006-1:2010+A1:2016 (BSI)) [2], the United States Federal Highway Administration (FHWA) [3], and the United States National Concrete Masonry Association (NCMA) [4]. Applications using geosynthetic reinforcements are commonly designed with a vertical spacing (VS) of 0.6 m. This relatively large spacing may lead to a high reinforcement strength near the back of the wall facing, which may result in a connection failure [5,6]. To alleviate the primary reinforcement load and increase internal stability, shorter reinforcements (referred to as secondary reinforcements) set between primary reinforcements have recently been utilized.
Several researchers have evaluated the secondary reinforcement effect on GRS wall performance under working stress conditions using analytical methods [7,8], experimental studies [9–11], field tests [12–14], and numerical analyses [5,6]. For example, the limit equilibrium method was used by Han and Leshchinsky (2006) [7], and Leshchinsky et al. (2014) [8], to investigate the effect of secondary reinforcements in GRS walls. Additionally, Zornberg et al. [9,10] utilized a geotechnical centrifuge to study the failure mechanisms of geosynthetic soil slopes with secondary reinforcements. Furthermore, Lelli et al. [12] and Jiang et al. [13,14] constructed and monitored GRS walls with/without secondary reinforcement in the field to understand the performance of GRS walls with secondary reinforcement by comparing the wall facing deflection, the strains of primary and secondary reinforcements, etc., of the GRS walls. Jiang et al. [5,6] employed finite difference analysis (FDA) to evaluate the secondary reinforcement effect on the performance of GRS walls. These results confirmed that installing a secondary reinforcement between primary reinforcements can reduce the lateral displacement of the wall facing, the bulge of the wall facing, the connection load, and the maximum reinforcement load in the primary reinforcement. Although a large amount of research has been conducted on the performance of GRS walls with secondary reinforcement under working stress conditions, concerns still remain regarding the response of secondary reinforcements to seismic motions and the benefits of secondary reinforcement under dynamic conditions.

Extensive studies, experimentally and numerically, have been carried out to analyze the performance of GRS walls during seismic loading, including the relative lateral facing displacement, lateral earth pressure, crest settlement, acceleration amplification, and tensile reinforcement in reinforcement layers [15–26]. The results of these studies showed that critical factors including reinforcement length, wall height, soil stiffness, soil strength, earthquake motions, and reinforcement spacing played important roles in the performance of GRS walls during seismic loading.

In this study, a series of dynamic finite element analyses were conducted to investigate GRS walls under seismic loading to explore the effect of secondary reinforcement on the seismic performance of the GRS walls. First, the finite element procedure was validated by simulating a field GRS wall with secondary reinforcement and modular-block geosynthetic-reinforced soil (MB-GRS) walls built by Jiang et al. [5,13,14] and Ling et al. [27], respectively. Then, a parametric study was carried out to explore the effect of the secondary reinforcement length and stiffness, vertical spacing of the primary reinforcement, and wall height on the performance of the GRS walls after seismic loading by investigating relative lateral facing displacement, connection load, and maximum reinforcement load in primary reinforcement, and acceleration amplification. Additionally, different scales of seismic motions were considered in the parametric study. It is hoped that the results of this study can provide insights for the seismic design of GRS walls with secondary reinforcement.

2. Validation of the Dynamic Finite Element Procedure

The finite element procedure used in this study is based on the PLAXIS 2D commercial nonlinear finite element code. To verify the capability of the finite element procedure, three benchmark cases, one involving a field GRS wall with secondary reinforcement under working stress conditions [5,13,14], and two large-scale shaking table tests on MB-GRS walls (walls 1 and 2) [27], were carefully adopted and reproduced.

2.1. A Field GRS Wall with Secondary Reinforcement

The cross-section of the field GRS wall with secondary reinforcement is shown in Figure 1. The 11.7 m height vertical wall was constructed on the foundation bedrock, containing backfill (reinforced soil zone and retained soil zone), wall facing, wall cap, embedment soil, backslope, and primary and secondary reinforcement layers. Four types of uniaxial polyethylene geogrids and one type of biaxial polypropylene geogrid were adopted as primary and secondary reinforcements, respectively. The secondary and primary reinforcement lengths were 1.3 m and 18.3 m, respectively. The secondary reinforcement layers were applied between two primary reinforcement layers.
The dimensions of the facing blocks were 0.3 m in width and 0.2 m in height while the wall cap was 0.3 m in width and 0.1 m in height. To minimize the boundary effects on the numerical simulations, the foundation bedrock was created in 72 m width (extended to 36 m in front of the wall facing) and 20 m depth.

Table 1 shows the input parameters of the backfill (reinforced soil zone) used in this verification analysis. The input parameters were obtained and modified based on Jiang et al. [5], where the cap yield model was adopted to simulate the backfill behavior. In this verification analysis, the behavior of the backfill soil was characterized by the hardening soil (HS) model following the conversion process reported by Itasca [28] and Liu et al. [29]. The converting formulations are shown as follows:

\[
G_{ref} = \frac{E_{orf}}{2(1 + v_{ur})} 
\]

\[
K_{iso,ref} = \frac{(1 + 2K_0)E_{orf}}{3} 
\]
where $G_{ref}^e$ is reference elastic tangent shear modulus; $E_{ref}^{ur}$ is reference Young’s modulus for unloading and reloading; $K_{ref}^{iso}$ is reference bulk modulus; $E_{ref}^{ooed}$ is reference tangent stiffness modulus.

### Table 1. Input parameters of backfill (reinforced soil zone) for validation analysis.

| Material                  | Constitutive Model | $\gamma$ (kN/m$^3$) | $c$ (kPa) | $\phi$ (°) | $\psi$ (°) | $E_{ref}^{ooed}$ (kPa) | $E_{ref}^{ur}$ (kPa) | $m$    | $\nu$ |
|---------------------------|--------------------|----------------------|-----------|------------|------------|------------------------|---------------------|--------|-------|
| Backfill (reinforced soil zone) | HS                 | 18.1                 | 0         | 52         | 8          | 14,690                 | 78,000              | 0.52   | 0.2   |

Note: $\gamma$ = unit weight; $c$ = cohesion; $\phi$ = soil peak friction angle; $\psi$ = dilation angle; $E_{ref}^{ooed}$ = reference stiffness modulus; $E_{ref}^{ur}$ = reference Young’s modulus for unloading and reloading; $m$ = Stress dependence exponent; $\nu$ = Poisson’s ratio.

Four types of primary reinforcements (UX1, UX2, UX3, and UX4) and one type of secondary reinforcement (BX) were characterized as linear-elastic. The stiffnesses for UX1, UX2, UX3, UX4, and BX are 360, 407, 637, 860, and 330 kN/m, respectively. The foundation bedrock and facing blocks were simulated as linear-elastic. The properties of foundation bedrock and facing blocks are listed in Table 2. The backfill (retained soil zone), backslope, and embedment soil were defined using the Mohr–Coulomb criterion (MC), and their parameters are shown in Table 2. The block–block interfaces ($\delta = 57^\circ$), soil–block interfaces ($\delta = 40^\circ$), soil–uniaxial geogrid interfaces ($\delta = 47^\circ$), soil–biaxial geogrid interfaces ($\delta = 40^\circ$), and block–embedment soil interfaces ($\delta = 28^\circ$) were defined using the MC, where $\delta$ is the friction angle of interfaces.

### Table 2. Input parameters for validation analysis [5].

| Material                  | Constitutive Model | $\gamma$ (kN/m$^3$) | $c$ (kPa) | $\phi$ (°) | $\psi$ (°) | $E$ (kPa) | $\nu$ |
|---------------------------|--------------------|----------------------|-----------|------------|------------|-----------|-------|
| Backfill (retained soil zone) | MC                 | 16.8                 | 1         | 34         | 0          | 20,000    | 0.3   |
| Backslope                | MC                 | 20                   | 1         | 34         | 0          | 20,000    | 0.3   |
| Embedment                | MC                 | 16.8                 | 1         | 34         | 0          | 2000      | 0.3   |
| Foundation bedrock       | Linear-elastic     | 20                   | -         | -          | -          | 2000,000  | 0.2   |
| Facing block             | Linear-elastic     | 15                   | -         | -          | -          | 2000,000  | 0.25  |

Note: $\gamma$ = unit weight; $c$ = cohesion; $\phi$ = soil peak friction angle; $\psi$ = dilation angle; $E$ = Elastic modulus; $\nu$ = Poisson’s ratio.

The finite element procedure for simulating the field GRS wall with secondary reinforcement consisted of eight steps: (a) the foundation bedrock was activated and reached an equilibrium under gravitational force; (b) a layer of the backfill (reinforced soil zone and retained soil zone) and facing blocks was activated; (c) primary and secondary reinforcements were applied in the numerical model; (d) corresponding interfaces (e.g., the block–block interfaces and the soil–block interfaces) were activated; (e) 8 kPa of uniform vertical stress was placed at the top of reinforced soil to represent more accurate compaction-induced stress; (f) the model reached the new equilibrium; (g) the model was lifted sequentially by staged constructions until a total height of 11.7 m was reached (repeating the steps of (b)–(f) until a total height of 11.7 m was reached); (h) on the top of the reinforced soil, the backslope was then constructed.

The bottom boundary was assigned to be fixed in all directions (fully fixed) and both sides of the wall were roller boundaries (normally fixed) [5]. The roller boundaries enabled vertical direction displacement and fixed the horizontal direction displacement. The field GRS wall with secondary reinforcement was modeled by 15-node triangular solid elements. Figure 3 shows the mesh and the boundary conditions of the field GRS wall with the secondary reinforcement model used in this study.
Figure 3. The mesh and boundary conditions of the field GRS wall with secondary reinforcement model.

The results of lateral facing displacement and maximum reinforcement load in the primary reinforcement, after the construction of the backslope, were selected to verify the finite element procedure. The finite element method (FEM) results were compared with the measured data of Jiang et al. [5], as shown in Figure 4. The overall responses of the GRS wall with secondary reinforcement were satisfactory and can be reproduced.

Figure 4. Comparison of the calculated and measured values: (a) lateral facing displacement; (b) maximum reinforcement load.

Figure 4a shows the lateral facing displacement calculated by the finite element method (FEM) and measured values from the field test after the construction of the backslope. It is shown that the overall lateral facing displacement calculated by the finite element method can capture the trend of the measured lateral facing displacement. The comparison of maximum reinforcement load in the primary reinforcement, calculated by FEM and measured values from the field test after the backslope was constructed, is shown in Figure 4b. It is shown that the maximum reinforcement load in the primary reinforcement calculated by the FEM tracked the maximum reinforcement load distribution along with the depth of the field test wall.
A Large-Scale Shaking Table on an MB-GRS Wall

Figure 5 shows the cross-section of the MB-GRS walls (walls 1 and 2). The MB-GRS walls were 3 m in height (0.2 m thick foundation and 2.8 m thick backfill soil) with a facing inclined 12° from the vertical and consisted of backfill, wall facing, foundation, and reinforcement layers. A polyester (PET) geogrid was utilized for the reinforcement. The reinforcement length was 0.73 H, where H = 2.8 m is the height of the wall with vertical spacings of 0.4 m and 0.6 m, respectively. The modular blocks, each weighing 34 kg, were 0.2 m in height and 0.3 m in width. To avoid unrealistic wave reflection during the shaking table tests, expanded polystyrene boards (EPS) with a thickness of 10 cm were applied [27].

Figure 5. The cross-section of the modular-block geosynthetic-reinforced soil (MB-GRS) wall: (a) wall 1; (b) wall 2.

Table 3 shows the input parameters of backfill and foundation soil that was used in the verification. The backfill soil and foundation soil were Tokachi sand ($D_{50} = 0.27$ mm, $G_s = 2.668$, $C_u = 2$, $e_{max} = 1.291$, and $e_{min} = 0.781$) with different relative densities ($D_r$) of 55% and 90%, respectively. The behavior of the backfill soil and foundation soil was simulated with a hardening soil small strain (HSS) model and HS model. The HSS model accounts for the damping with the strain increase and decay of stiffness. The shear stiffness ratio $G_s/G_0$ was introduced by Hardin and Drnevich [30] and updated by Santos and Correia [31]:

$$\frac{G_s}{G_0} = \frac{1}{1 + 0.385 \left| \frac{\gamma}{\gamma_{0.7}} \right|}$$

where $G_s$ is the secant shear modulus, $G_0$ is the initial or very small-strain shear modulus, $\gamma$ is the shear strain, and $\gamma_{0.7}$ is the value corresponding to $G_s/G_0 = 0.7$. The triaxial test reported by Ling et al. [27,32] was used to calibrate the model. The specimens of backfill soil and foundation, under drained deviatoric loading, were subjected to confining pressures of 30 and 70 kPa. Figure 6 shows the behavior of the foundation and the backfill soil and the calibration results under triaxial tests.

Table 3. Input parameters of the large-scale test wall for validation analysis (backfill and Foundation).

| Material         | Constitutive Model | $\gamma$ (kN/m$^3$) | $c$ (kPa) | $\phi$ (°) | $\psi$ (°) | $E_{ref}^{50}$ (kPa) | $E_{ref}^{oed}$ (kPa) | $E_{ref}^{ur}$ (kPa) | $m$ | $G_0^{ref}$ | $\gamma_{0.7}$ |
|------------------|--------------------|----------------------|-----------|------------|-------------|---------------------|----------------------|---------------------|-----|----------------|----------------|
| Backfill soil    | HSS                | 14.3                 | 2.5       | 37.3       | 7.3         | 5000                | 5800                 | 17,000              | 0.5 | $3 \times 10^4$ | $4 \times 10^{-4}$ |
| Foundation soil  | HS                 | 15.8                 | 2.3       | 40.5       | 10          | 14,800              | 14,000               | 44,400              | 0.5 | -              | -              |

Note: $\gamma =$ unit weight; $c =$ cohesion; $\phi =$ soil peak friction angle; $\psi =$ dilation angle; $E_{ref}^{50}$ = reference stiffness modulus; $E_{ref}^{oed}$ = reference tangent stiffness modulus; $E_{ref}^{ur}$ = reference Young’s modulus for unloading and reloading; $m =$ Stress dependence exponent; $G_0^{ref}$ = reference shear modulus at very small strains; $\gamma_{0.7}$ = small strain at which the secant shear modulus is equal to 0.7$G_0^{ref}$. 


The reinforcements were assumed to be linear-elastic with a stiffness of 550 kN/m. Previous studies have shown that, under cyclic loading, reinforcements exhibit hysteretic behavior [33]. However, using finite element analyses, Liu [34] found that the seismic response was not affected by the hysteretic behavior and that the effect of reinforcement stiffness could be captured by the secant value [11,35,36]. In the present study, the modular blocks and EPS were modeled as linear-elastic. The properties of the modular blocks were \( E = 2 \times 10^6 \text{kPa} \) and \( \nu = 0.2 \). The properties of the EPS were \( E = 200 \text{kPa} \) and \( \nu = 0.2 \). The block–block interfaces \( (\delta = 35^\circ) \), soil–block interfaces \( (\delta = 27^\circ) \), block–geogrid interfaces \( (\delta = 35^\circ) \), and soil–geogrid interfaces \( (\delta = 34^\circ) \) were defined using the MC.

The dynamic finite element procedure consisted of two steps: (a) a staged construction simulation step and (b) a dynamic simulation step, which were utilized to capture the static responses and seismic responses, respectively. In the construction simulation, the wall was lifted sequentially, in which 0.2 m thick soil and block layers were placed until a final height of 3 m was reached. The seismic loading was applied to the model base and was activated following the construction simulation step. The seismic motions (north–south component) reported by Ling et al. [27] were used in this simulation. Viscous damping of 5% was considered for the soil element during the dynamic analysis. The MB-GRS wall was modeled by 15-node triangular solid elements.

For the model’s boundary conditions, roller boundaries (normally fixed) were assigned to both sides of the wall and the bottom boundary was assigned to be fixed in all directions (fully fixed) [36,37].
The mesh and the boundary conditions of the numerical model utilized in this study are shown in Figure 7.

![Figure 7. The mesh and boundary conditions: (a) wall 1; (b) wall 2.](image)

The FEM results of the validation were evaluated by comparing the measured data of Ling et al. [27] with (a) the relative lateral facing displacement, (b) maximum acceleration, and (c) maximum reinforcement load after the first and second shakings, as shown in Figure 8.

Figure 8a shows the calculated and measured values of the relative lateral facing displacement. The relative lateral facing displacement was calculated based on the reference point, as shown in Figure 7. There was a good alignment of the calculated and measured relative lateral facing displacements under seismic loading. Figure 8b shows a comparison between the calculated and measured acceleration amplification after the first and second shakings. In the results for the 0.4 g Kobe earthquake, the calculated acceleration amplification satisfactorily reproduced the measured acceleration amplification. On the other hand, for the results for the 0.8 g Kobe earthquake, the acceleration amplification calculated by the FEM gives slightly smaller values than the measured values. These minor differences in results are likely attributed to the soil strain-softening behavior, which may be more pronounced under the higher-intensity earthquake loading. A comparison of the calculated and measured maximum reinforcement load is shown in Figure 8c. The calculated maximum reinforcement loads were in good trend when compared to the measured values. Similar discrepancies in the calculated and measured values are also found in Liu et al. [37] and Fan et al. [36]. This may be attributed to the locations of the sensors utilized to measure the reinforcement loads in the large-scale shaking table test. Overall, the calculated distribution trend of the maximum reinforcement load in each layer is similar to the measured values.
The FEM results of the validation were evaluated by comparing the measured data of Ling et al. [27] with (a) the relative lateral facing displacement, (b) maximum acceleration, and (c) maximum reinforcement load after the first and second shakings, as shown in Figure 8.

Figure 8. Comparison of the calculated and measured values: (a) relative lateral facing displacement; (b) maximum acceleration; (c) maximum reinforcement load.

Based on the validations, the finite element approach used in this study exhibits a good capability to simulate the behavior of GRS walls with secondary reinforcement as well as the response of GRS walls subjected to seismic loading.

3. Finite Element Model

The objective of this study is to explore the secondary reinforcement effect on the seismic performance of GRS walls. To this end, a series of parametric studies was carried out. The critical factors considered in this study include the secondary reinforcement length and stiffness, the wall height (H), and the vertical spacing of the primary reinforcement layers. To explore the influence of the critical factors, one set of parameters was utilized as a baseline case. The details of the investigated critical factors and baseline case are listed in Table 4.
The heights (H) of the three walls were 3, 6, and 9 m, and all three walls had a length of 50 m. The large length of the wall was intended to prevent the influence of boundary deformation in the study area. The width of the finite element model was modeled to be sufficiently large, as will be addressed in the following section. The length of each primary reinforcement was set as 0.7 H beyond the back of the facing and the length of the secondary reinforcement ranged from 0.9 to 1.8 m (0.15 H to 0.3 H). Two vertical spacings of 0.4 m and 0.6 m were used in the layers of primary reinforcement, and the secondary reinforcement layers were placed within the primary reinforcement layers (see Figure 5a). Vertical modular block facings were assumed, and the width and height of the modular blocks were 0.3 and 0.2 m, respectively. The foundation soil was assumed to be underlain by stiff rock with a depth of 10 m. The mesh and the dimension of the numerical model used in this study are shown in Figure 9.

The backfill, modular blocks, and the interface between the backfill soil and the modular block and the interface between the modular blocks used in the parametric studies were characterized by HSS, the Linear-Elastic model, and the MC constitutive model, respectively, and the parameters were assumed to be the same as the validation of the large-scale shaking table test. The foundation was modeled as linear-elastic, with $E = 2 \times 10^6 \text{ kPa}$, $\nu = 0.2$, and $\gamma = 20 \text{ kN/m}^3$. The primary and secondary geogrid reinforcements were assumed to be linear-elastic, and the former was assumed to have a stiffness of 1500 kN/m while the latter was assumed to have a stiffness ranging from 500 to 5000 kN/m. In this study, the wall with a wall height of 6 m, secondary reinforcement length and stiffness of 0.9 m and 500 kN/m respectively, and vertical spacing of 0.6 m under peak ground acceleration of 0.8 g was considered as a baseline case.

In the dynamic finite element analysis, the staged construction step of the GRS walls was first conducted to reach the initial equilibrium of deformation and stress before the dynamic simulation step [21,26]. The GRS walls were then subjected to the validated 1995 Kobe earthquake (north–south component), which was employed as the input motion in the dynamic analyses, as shown in Figure 10. The duration of the Kobe earthquake was obtained after Kramer [38]. The 1995 Kobe earthquake (north–south component) was scaled to peak ground accelerations (PGAs) of 0.2, 0.4, and 0.6 g to evaluate the effects of the PGA on the dynamic response. In order to prevent spurious wave reflection inside the soil body, the artificial boundary must be set up on the actual boundary when conducting the
dynamic simulation step. In this study, the viscous boundary conditions in PLAXIS 2D were used to ensure the precision of the simulation results. The viscous boundaries used in PLAXIS 2D were based on Lysmer and Kuhlmeyer [39]. The absorbed normal and shear stresses at the viscous boundaries in the x-direction are

\[ \sigma_n = -C_1 \rho V_P \dot{u}_x \]  
\[ \tau = -C_2 \rho V_S \dot{u}_y \]

where \( \rho \) is the unit weight of the materials; \( V_P \) and \( V_S \) are the velocities of the P and S waves, respectively; \( \dot{u}_x \) and \( \dot{u}_y \) are the particle velocities in the x- and y-directions, respectively; and \( C_1 \) and \( C_2 \) are the relaxation coefficients used to modify the effect of absorption. The use of \( C_1 = 1 \) and \( C_2 = 1 \) is suggested by PLAXIS [40], to result in a reasonable absorption of any waves reaching the boundary.

![Acceleration vs Time Graph](image)

**Figure 10.** The validated 1995 Kobe earthquake (north–south component) employed in the dynamic analyses.

4. Results

4.1. Effects of Secondary Reinforcement Length

4.1.1. Relative Lateral Facing Displacement

The effect of the secondary reinforcement length on the relative lateral facing displacement was investigated using five cases, namely no secondary reinforcement (i.e., 0 m) and secondary reinforcement lengths (\( L_{\text{secondary}} \)) of 0.9, 1.2, 1.5, and 1.8 m. In this study, the relative lateral facing displacements were calculated based on the reference point, as shown in Figure 11, and the results were normalized by the height. As shown in Figure 11, the walls with secondary reinforcement have slightly lower relative lateral facing displacements. However, it was shown that increasing the secondary reinforcement length has an insignificant effect on reducing the relative lateral facing displacement.
4.1.2. Connection Load and Maximum Reinforcement Load

The results of the distributions of the connection load and the maximum reinforcement load in the primary reinforcement are presented in Figure 12a,b, respectively, and the sum of the connection load and the maximum reinforcement load of the wall in each primary reinforcement layer are shown in Table 5. It can be observed that the secondary reinforcement of walls can help reduce the connection load and maximum reinforcement load in the primary reinforcement. With increasing secondary reinforcement length, the connection load and maximum reinforcement load decrease. Thus, it is demonstrated that the length of the secondary reinforcement is more effective for reducing the connection load and maximum reinforcement load than the relative lateral facing displacement under earthquake loading. However, the effect of increasing the length of the secondary reinforcement on reducing the connection load and maximum reinforcement load decreases with increasing secondary reinforcement length.

Figure 11. The effect of the length of the secondary reinforcement on the relative lateral facing displacement.
Figure 12. The distributions of (a) the connection load; (b) the maximum reinforcement load in the primary reinforcement.

Table 5. Sum of connection loads and maximum reinforcement loads of walls in each primary reinforcement layer (effect of secondary reinforcement length).

| Secondary Reinforcement Length (m) | Connection Load (kN/m) | Maximum Reinforcement Load (kN/m) |
|-----------------------------------|------------------------|----------------------------------|
| 0                                 | 163.81                 | 165.95                           |
| 1.2                               | 134.22                 | 144.58                           |
| 1.5                               | 117.65                 | 132.65                           |
| 1.8                               | 107.47                 | 120.75                           |
| 2.1                               | 101.90                 | 113.401                          |

4.1.3. Acceleration Amplification

Figure 13 shows the effect of the secondary reinforcement length on variations of acceleration amplification in the reinforced zones at 20 and 30 m distances from the facing, over the height of the backfill. The acceleration in the reinforced zone was based on a 1.0 m distance from the back of the facing blocks. The acceleration amplification is defined herein as the ratio of the peak response acceleration at the measured location to the peak acceleration on the base. It can be seen that the amplification is similar at distances of 20 and 30 m from the wall facing. The similar acceleration responses at these two distances prove that the wall length used in this study is large enough to prevent unrealistic waves from the side boundary. This conclusion applies to all of the five cases (i.e., no secondary reinforcement (0 m) and secondary reinforcement lengths of 0.9, 1.2, 1.5, and 1.8 m). To save space, only the results of the GRS walls without the secondary reinforcement are presented herein.
Figure 13. The effect of the secondary reinforcement length on variations of acceleration amplification.

As shown in Figure 13, the acceleration amplification out of the reinforced zone increases nonlinearly over the backfill height. The acceleration amplification can be considered negligible or uniform in the lower part (vicinity of the foundation) and intensifies in the upper part of the backfill. A comparison of different secondary reinforcement lengths showed that the influence of the secondary reinforcement length on the acceleration amplification was negligible; however, the acceleration amplification in walls without secondary reinforcement was slightly larger than in the walls with secondary reinforcement, which is concluded to be due to the slight increase in relative lateral facing displacement.

4.2. Effects of Secondary Reinforcement Stiffness

4.2.1. Relative Lateral Facing Displacement

The effect of secondary reinforcement stiffness on the relative lateral facing displacement is shown in Figure 14. Six cases were considered, namely no secondary reinforcement (i.e., 0 kN/m) and secondary reinforcements with stiffnesses of 500, 1000, 1500, 2000, and 5000 kN/m. The results indicate that increasing the secondary reinforcement stiffness had a minor influence on the relative facing displacement. The behavior of the relative lateral facing displacement in Figure 14 is comparable to that in Figure 11, indicating that, under strong earthquake loading, increasing the secondary reinforcement length or stiffness cannot effectively reduce the relative lateral facing displacement.
4.2.2. Connection Load and Maximum Reinforcement Load in Primary Reinforcement

Figure 15a,b show the influence of secondary reinforcement stiffness on the distributions of the connection load and the maximum reinforcement load in the primary reinforcement respectively, and the sum of the connection load and the maximum reinforcement load of the wall in each primary reinforcement layer are shown in Table 6. It was found that increasing the secondary reinforcement stiffness had a slight influence on the connection load and the maximum reinforcement load for the walls adopting secondary reinforcement. The walls adopting the secondary reinforcement led to similar distributions of connection load and maximum reinforcement load. Nevertheless, the walls adopting the secondary reinforcement all had a smaller connection load and a smaller maximum reinforcement load than the wall without secondary reinforcement. A comparison of the influence of the secondary reinforcement length and stiffness shows that increasing the secondary reinforcement length is a more effective way to reduce the connection load and maximum reinforcement load under earthquake loading.
Figure 15. The influence of the secondary reinforcement stiffness on the distributions of (a) the connection load; (b) the maximum reinforcement load in the primary reinforcement.

Table 6. Sum of connection loads and maximum reinforcement loads of walls in each primary reinforcement layer (effect of secondary reinforcement stiffness).

| Secondary Reinforcement Stiffness (kN/m) | Connection Load (kN/m) | Maximum Reinforcement Load (kN/m) |
|-----------------------------------------|------------------------|-----------------------------------|
| 0                                       | 163.81                 | 165.95                            |
| 500                                     | 134.22                 | 144.58                            |
| 1000                                    | 131.69                 | 142.86                            |
| 1500                                    | 131.19                 | 142.83                            |
| 2000                                    | 129.90                 | 141.23                            |
| 5000                                    | 128.75                 | 139.67                            |

4.2.3. Acceleration

Figure 16 shows the variations of acceleration amplification in the reinforced zone considering various secondary reinforcement stiffnesses over the height of the backfill. These results are similar to those shown in Figures 13 and 16, indicating that the acceleration amplification was less influenced by either the length or stiffness of the secondary reinforcement.
4.3. Effects of Wall Height

Figure 17a–d show the effect of wall height on the relative lateral facing displacement, connection load, maximum reinforcement load, and the acceleration amplification of the wall with/without secondary reinforcement, respectively. The results for the 3, 6, and 9 m high walls with and without secondary reinforcement are presented. As shown in Figure 17a, higher relative lateral facing displacements occur in the higher wall. Similar results can be seen in the higher walls, where the effect on reducing relative lateral facing displacement is insignificant.

As shown in Figure 17b,c, secondary reinforcement can be an effective method for reducing the connection load and the maximum reinforcement load for various wall heights. To further evaluate the effect of the wall height, the maximum reinforcement loads of the 3, 6, and 9 m high walls in each primary reinforcement after shaking were added together, as shown in Table 7. From the results, it is evident that installing secondary reinforcement is more effective in the higher walls. Under the same $L_{\text{secondary}}/H$ ratio (0.15), the 9 m high wall experienced a 15.3% reduction in the overall reinforcement load, while the 3 and 6 m high walls experienced 1.06% and 12.9% reductions, respectively.

Table 7. Sum of the maximum reinforcement loads of walls in each primary reinforcement layer.

| Height of GRS Walls (m) | Secondary Reinforcement Length (m) | VS (m) | PGA (g) | Value (kN/m) | Difference |
|------------------------|-----------------------------------|--------|---------|--------------|------------|
| 3                      | 0                                 | 0.6    | 0.8     | 25.714       | 1.06%      |
| 3                      | 0.45                              | 0.6    | 0.8     | 25.442       |            |
| 6                      | 0                                 | 0.6    | 0.8     | 165.95       | 12.9%      |
| 6                      | 0.9                               | 0.6    | 0.8     | 144.58       |            |
| 9                      | 0                                 | 0.6    | 0.8     | 363.89       | 15.3%      |
| 9                      | 1.35                              | 0.6    | 0.8     | 308.31       |            |
| 6                      | 0                                 | 0.4    | 0.8     | 181.87       | 9.4%       |
| 6                      | 0.9                               | 0.4    | 0.8     | 164.72       |            |
| 6                      | 0                                 | 0.6    | 0.2     | 102          |            |
| 6                      | 0.9                               | 0.6    | 0.2     | 78.77        | 22.8%      |
As shown in Figure 17d, under the same seismic loading, secondary reinforcement does not notably affect the acceleration amplification for the three wall heights.

Figure 17. The effect of wall height on (a) the relative lateral facing displacement; (b) connection load; (c) maximum reinforcement load; (d) the acceleration amplification of the wall with and without secondary reinforcement.
4.4. Effects of the Vertical Spacing of the Primary Reinforcement Layer

Figure 18a–d show the effect of vertical spacing on the relative lateral displacement, connection load, maximum reinforcement load, and acceleration amplification of the wall with/without secondary reinforcement, respectively. The walls with a VS of the primary reinforcement layer of 0.6 and 0.4 m were compared. As shown in Figure 18a, a small difference was observed between the walls with and without secondary reinforcement and between the walls with a VS of 0.4 and 0.6 m. This result also proved, as above, that decreasing the VS of the primary reinforcement layer has a slight influence on the overall displacement.

Figure 18. The effect of vertical spacing on (a) the relative lateral displacement; (b) connection load; (c) maximum reinforcement load; (d) and acceleration amplification of the wall with and without secondary reinforcement.
From Figure 18b,c, it is shown that the walls with secondary reinforcement have a smaller connection load and maximum reinforcement load. The maximum reinforcement loads in each layer were summed up, as shown in Table 7. For the walls with a VS of 0.4 m, the reinforcement load decreased by 9.4% when a secondary reinforcement was installed, while for the walls with a vertical spacing of 0.6 m, the corresponding decrease was 12.9%. This indicates that the effect of secondary reinforcement on reducing the reinforcement load is diminished with reduced VS of the primary reinforcement layer.

As shown in Figure 18d, the effect of the vertical spacing on acceleration amplification can be negligible. Similar findings were also reported by Ling et al. [20].

4.5. Effects of Peak Ground Acceleration (PGA)

Figure 19a–d show the effect of PGA on the relative lateral displacement, connection load, maximum reinforcement load, and acceleration amplification of walls with/without secondary reinforcement, respectively. Four different PGAs of 0.2, 0.4, 0.6, and 0.8 g were compared. As shown in Figure 19a, with increasing input PGA, the relative lateral facing displacement of the wall increases. The relative lateral facing displacements are nearly linear on the upper part of the wall under PGAs of 0.2 and 0.4 g. Under PGAs of 0.6 and 0.8 g, the distributions of relative lateral facing displacements are nonlinear over the wall height. The results demonstrate that, for the same wall configuration, under a smaller PGA, secondary reinforcement has a slightly greater effect on reducing the relative lateral facing displacement. Under PGAs of 0.2 and 0.8 g, the relative lateral facing of the wall with secondary reinforcement is 17.6% and 3.4% lower, respectively, than the wall without secondary reinforcement. This suggests that the benefit of secondary reinforcement is lower during strong earthquakes.

As shown in Figure 19b,c, installing secondary reinforcement can reduce the connection load and maximum reinforcement load under various PGAs. To compare the effect of secondary reinforcement under different PGAs, the maximum reinforcement loads in each layer after the shaking were added together. Similar to the results for relative lateral facing displacement, it was found that secondary reinforcement is more effective at reducing the maximum reinforcement load under smaller PGA. As shown in Table 7, for PGAs of 0.2 and 0.8 g, the maximum reinforcement load was lower by 22.8% and 12.9%, respectively, in walls with secondary reinforcement compared to those without secondary reinforcement.

Furthermore, as shown in Figure 19d, a smaller PGA was found to give a higher acceleration amplification. Additionally, the acceleration amplifications for various PGAs are all slightly smaller in walls with secondary reinforcement compared to those without secondary reinforcement.

Figure 19. Cont.
1. Under earthquake loading, the relative lateral facing displacement was slightly lower in walls with secondary reinforcement and the large-scale shaking table tests by Jiang et al. (2015, 2016) showed the same trend. As shown in Table 7, for PGAs of 0.2 and 0.8 g, the maximum reinforcement load was lower by 22.8% and 12.9%, respectively, in walls with secondary reinforcement compared to those without secondary reinforcement.

2. As shown in Figure 19, installing secondary reinforcement can reduce the connection load and maximum reinforcement load under various PGAs. To compare the effect of secondary reinforcement under different PGAs, the maximum reinforcement loads in each layer after the shaking were added together. Similar to the results for relative lateral facing displacement, it was found that secondary reinforcement is more effective at reducing the maximum reinforcement load under smaller PGA. As shown in Table 7, for PGAs of 0.2 and 0.8 g, the maximum reinforcement load was lower by 22.8% and 12.9%, respectively, in walls with secondary reinforcement compared to those without secondary reinforcement.

Furthermore, as shown in Figure 19d, a smaller PGA was found to give a higher acceleration amplification. Additionally, the acceleration amplifications for various PGAs are all slightly smaller in walls with secondary reinforcement compared to those without secondary reinforcement.

5. Conclusions

A finite element investigation of GRS walls with/without secondary reinforcement under seismic loading was presented. The validated finite element procedure was based on the field GRS wall with secondary reinforcement and the large-scale shaking table tests by Jiang et al. (2015, 2016, 2018) [5,13,14], and Ling et al. (2005) [27], and was utilized to carry out a parametric study of the seismic response of GRS walls with/without secondary reinforcement by assessing the relative lateral facing displacement, connection load, maximum reinforcement load, and acceleration amplification. Based on the study, the following can be concluded:

1. Under earthquake loading, the relative lateral facing displacement was slightly lower in walls with secondary reinforcement than in walls without secondary reinforcement. However, no significant difference in relative lateral facing displacement was observed when the length or stiffness of the secondary reinforcement were increased. Under different wall heights, vertical spacings of the primary reinforcement layer, and PGAs, secondary reinforcement was found to have only a small influence on the relative lateral facing displacement.
2. Increasing the length or stiffness of the secondary reinforcement can reduce the connection load and maximum reinforcement load in the primary reinforcement. The benefit of increasing the secondary reinforcement length is greater than the benefit of increasing its stiffness: compared to increasing the secondary reinforcement length, increasing its stiffness led to a smaller reduction in the connection load and maximum reinforcement load in the primary reinforcement. The secondary reinforcement effect for the performance of the walls is more effective for reducing the connection load and maximum reinforcement load than for reducing the relative lateral facing displacement.

3. The effect of secondary reinforcement on decreasing the connection load and maximum reinforcement load is greater for higher walls, larger vertical spacings of the primary reinforcement layer, and smaller PGAs.

4. The GRS walls with secondary reinforcement may help reduce the effect of the acceleration amplification, increasing the overall performance of GRS walls subjected to earthquake loadings. However, the number of critical factors investigated in this study is limited, and consequently, future studies should consider more critical factors to provide more insights on the seismic design of GRS walls with secondary reinforcement. This study indicates that installing secondary reinforcement between primary reinforcements can increase the performance of GRS walls under seismic conditions.

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References
1. AASHTO. LRFD Bridge Design Specifications, 6th ed.; American Association of State Highway and Transportation Officials: Washington, DC, USA, 2012.
2. BSI. BS 8006-1:2010+A1:2016 Code of Practice for Strengthened/Reinforced Soils and Other Fills; BSI: London, UK, 2016.
3. Berg, R.R.; Christopher, B.R.; Samtani, N.C. Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes; FHWA-NHI-10-024; National Highway Institute Federal Highway Administration, Department of Transportation: Washington, DC, USA, 2009; Volume I.
4. NCMA. Segmental Retaining Walls Best Practices Guide for the Specification, Design, Construction and Inspection of SRW Systems; National Concrete Masonry Association: Herndon, VA, USA, 2016.
5. Jiang, Y.; Han, J.; Zornberg, J.; Parsons, R.L.; Leshchinsky, D.; Tanyu, B. Numerical analysis of field geosynthetic-reinforced retaining walls with secondary reinforcement. Geotechnique 2018, 69, 122–132. [CrossRef]
6. Jiang, Y.; Han, J.; Parsons, R.L. Numerical evaluation of secondary reinforcement effect on geosynthetic-reinforced retaining walls. Geotext. Geomembr. 2020, 48, 98–109. [CrossRef]
7. Han, J.; Leshchinsky, D. General analytical framework for design of flexible reinforced earth structures. J. Geotech. Geoenviron. 2006, 132, 1427–1435. [CrossRef]
8. Leshchinsky, D.; Kang, B.; Han, J.; Ling, H. Framework for limit state design of geosynthetic-reinforced walls and slopes. Transp. Infrastruct. Geotechnol. 2014, 1, 129–164. [CrossRef]
9. Zornberg, J.G.; Sitar, N.; Mitchell, J.K. Performance of geosynthetic reinforced slopes at failure. J. Geotech. Geoenviron. 1998, 124, 670–683. [CrossRef]
10. Zornberg, J.G.; Sitar, N.; Mitchell, J.K. Limit equilibrium as basis for design of geosynthetic reinforced slopes. J. Geotech. Geoenviron. 1998, 124, 684–698. [CrossRef]
11. 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]

12. Lelli, M.; Laneri, R.; Rimoldi, P. Innovative reinforced soil structures for high walls and slopes combining polymeric and metallic reinforcements. *Procedia Eng.* **2015**, *125*, 397–405. [CrossRef]

13. Jiang, Y.; Han, J.; Parsons, R.L.; Cai, H. *Field Monitoring of Mechanically Stabilized Earth Walls to Investigate Secondary Reinforcement Effects*; No. KS-15-09; Kansas Department of Transportation Bureau of Research: Lawrence, KS, USA, 2015.

14. Jiang, Y.; Han, J.; Parsons, R.L.; Brennan, J.J. Field instrumentation and evaluation of modular-block MSE walls with secondary geogrid layers. *J. Geotech. Geoenviron.* **2016**, *142*, 05016002. [CrossRef]

15. Segrestin, P.; Bastick, M.J. Seismic design of reinforced earth retaining walls—the contribution of finite elements analysis. In Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan, 7 October 1998; pp. 577–582.

16. Cai, Z.; Bathurst, R.J. Seismic response analysis of geosynthetic reinforced soil segmental retaining walls by finite element method. *Comput. Geotech.* **1995**, *17*, 523–546. [CrossRef]

17. Hatami, K.; Bathurst, R.J. Effect of structural design on fundamental frequency of reinforced-soil retaining walls. *Soil Dyn. Earthq. Eng.* **2000**, *19*, 137–157. [CrossRef]

18. Hatami, K.; Bathurst, R.J. Investigation of seismic response of reinforced soil retaining walls. In Proceedings of the International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, USA, 29 March 2001; [CD-ROM]; Paper No. 7.18.

19. Bathurst, R.J.; Hatami, K.; Alfaro, M.C. *Geosynthetics and Their Applications, Geosynthetic-Reinforced Soil Walls and Slopes—Seismic Aspects*; Thomas Telford: London, UK, 2002; Chapter 14.

20. Ling, H.I.; Liu, H.; Mohri, Y. Parametric studies on the behavior of reinforced soil retaining walls under earthquake loading. *J. Eng. Mech.* **2005**, *131*, 1056–1065. [CrossRef]

21. Liu, H.; Ling, H.I. Seismic responses of reinforced soil retaining walls and the strain softening of backfill soils. *Int. J. Geomech.* **2011**, *12*, 351–356. [CrossRef]

22. Liu, H.; Wang, X.; Song, E. Reinforcement load and deformation mode of geosynthetic-reinforced soil walls subject to seismic loading during service life. *Geotech. Geomembr.* **2011**, *29*, 1–16. [CrossRef]

23. Guler, E.; Selek, O. Reduced-scale shaking table tests on geosynthetically-reinforced soil walls with modular facing. *J. Geotech. Geoenviron.* **2014**, *140*, 04014015. [CrossRef]

24. Panah, A.K.; Yazdi, M.; Ghalandarzadeh, A. Shaking table tests on soil retaining walls reinforced by polymeric strips. *Geotech. Geomembr.* **2015**, *43*, 148–161. [CrossRef]

25. Ren, F.; Zhang, F.; Xu, C.; Wang, G. Seismic evaluation of reinforced-soil segmental retaining walls. *Geotech. Geomembr.* **2016**, *44*, 604–614. [CrossRef]

26. Liu, H.; Hung, C.; Cao, J. Relationship between Arias intensity and the responses of reinforced soil retaining walls subjected to near-field ground motions. *Soil Dyn. Earthq.* **2018**, *111*, 160–168. [CrossRef]

27. Ling, H.I.; Mohri, Y.; Leshchinsky, D.; Burke, C.; Matsushima, K.; Liu, H. Large-scale shaking table tests on modular-block reinforced soil retaining walls. *J. Geotech. Geoenviron.* **2005**, *131*, 465–476. [CrossRef]

28. ITASCA. *User’s Manual FLAC2D: Fast Lagrangian Analysis of Continua, Version 7.0*; Itasca Consulting Group: Minneapolis, MN, USA, 2011.

29. Liu, H. Nonlinear elastic analysis of reinforcement loads for vertical reinforced soil composites without facing restriction. *J. Geotech. Geoenviron.* **2010**, *136*, 04010002. [CrossRef]

30. Hardin, B.O.; Drnevich, V.P. Shear modulus and damping in soils: Design equations and curves. *J. Soil Mech. Found.* **1972**, *98*, 667–692.

31. Santos, J.A.; Correia, A.G. Reference threshold shear strain of soil. Its application to obtain a unique strain-dependent shear modulus curve for soil. In Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, Instanbul, Turkey, 27–31 August 2001; pp. 267–270.

32. Ling, H.I.; Mohri, Y.; Leshchinsky, D.; Liu, H.; Burke, C. Finite-element simulations of full-scale modular-block reinforced soil retaining walls under earthquake loading. *J. Eng. Mech.* **2010**, *136*, 653–661. [CrossRef]

33. Bathurst, R.J.; Cai, Z. In-isolation cyclic load-extension behavior of two geogrids. *Geosynth. Int.* **1994**, *1*, 1–19. [CrossRef]

34. Liu, H. *Finite-Element Simulation of the Response of Geosynthethic-Reinforced Soil Walls*. Ph.D. Thesis, Columbia University, New York, NY, USA, 2003.
35. Allen, T.M.; Bathurst, R.J. Improved simplified method for prediction of loads in reinforced soil walls. *J. Geotech. Geoenviron.* **2015**, *141*, 04015049. [CrossRef]

36. Fan, C.; Liu, H.; Cao, J.; Ling, H.I. Responses of reinforced soil retaining walls subjected to horizontal and vertical seismic loadings. *Soil Dyn. Earthq.* **2020**, *129*, 105969. [CrossRef]

37. Liu, H.; Yang, G.; Ling, H.I. Seismic response of multi-tiered reinforced soil retaining walls. *Soil Dyn. Earthq.* **2014**, *61*, 1–12. [CrossRef]

38. Kramer, S.L. *Geotechnical Earthquake Engineering*; Prentice Hall: Upper Saddle River, NJ, USA, 1996.

39. Kuhlemeyer, R.L.; Lysmer, J. Finite element method accuracy for wave propagation problems. *J. Soil Mech. Found. Div.* **1973**, *99*, 421–427.

40. Plaxis. *Reference and Material Models Manual*; Plaxis BV: Delft, The Netherlands, 2019.

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