

**Research Paper**

Seismic Finite Element Analyses on Relative Settlements of Tall Geosynthetic Reinforced Soil Walls

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ABSTRACT

The underappreciated significance of perpendicular-to-facing relative settlements of geosynthetic reinforced soil (GRS) walls in seismic analyses is evident in the current literature, particularly when compared to along-facing settlements. Consequently, this study conducted a series of seismic finite element analyses to evaluate the perpendicular-to-facing settlements of a 20 m GRS wall during 36 significant worldwide earthquakes. Static and dynamic verifications of numerical modeling were performed by comparing the results of a 3.6 m physical GRS wall model and a 1 m shaking table test at the Royal Military College (RMC) of Canada, respectively. The reinforced soil modeling utilized the hardening soil constitutive model with small strain stiffness (HS-small). Additionally, the facing of the GRS wall model comprised 0.5 m × 1.5 m concrete panels. The findings emphasized the critical nature of relative deformations between the facing and reinforced zones, and between the reinforced and retained zones, in addition to absolute settlements. Moreover, the amount of relative settlements was found to be correlated with the intensity of seismic excitations. The correlations between relative settlements and intensity of earthquakes were also proposed based on the results of the current research.

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1. Introduction

The utilization of geosynthetic-reinforced soil (GRS) walls has seen a significant increase in recent years, particularly in transportation-related applications such as airport runways, bridge abutments, and highway widenings. This surge can be attributed to the lower construction costs, improved deformation tolerance, and satisfactory performance under seismic loadings. Additionally, the construction of taller GRS walls has become more prevalent, replacing traditional rigid concrete gravity walls as outlined in AASHTO (2020). According to Koerner & Koerner (2018), the global construction of geosynthetic-reinforced mechanically stabilized earth (MSE) walls exceeded 200,000 by 2018.

The literature encompasses numerous investigations on the dynamic performance of GRS walls with short to regular heights (Krishna & Latha, 2012; Ling et al., 2004; Krishna & Bhattacharjee, 2017; Xu et al., 2023). Despite these valuable research, there is notably limited focus on tall walls, with only a few studies available (Alavi et al., 2024; Lelli et al., 2015; Runser et al., 2001; Stuedlein et al., 2012, 2010). Hence, there is a necessity for further research to comprehensively explore the seismic performance of tall GRS walls across various aspects.

Differential (or relative) settlements of GRS walls can manifest as either along-facing (longitudinal orientation) or perpendicular-to-facing (between facing and reinforced zone) occurrences. While some guidelines, such as FHWA (Berg et al., 2009a, 2009b), offer recommendations for along-facing settlements, there are presently no criteria for allowable perpendicular-to-facing relative deformations (named relative or differential settlements in this study hereafter). According to Kim et al. (2010), these relative deformations could be more destructive than total settlements in some cases. Their assessment of an 11.5 m MSE wall demonstrated that differential settlements led to excessive tensile stresses in reinforcements, resulting in wall failure.

Furthermore, a six-year observation of an MSE wall revealed 10 cm relative settlements between facing and reinforced zones due to the consolidation

process (Bartlett et al., 2015). Furthermore, Sadat et al. (2018) showed that relative settlements could cause significant deformations, elevating lateral earth pressures and geogrid strains.

Additionally, previous parametric studies illustrated that the normalized values of differential settlements and the effectiveness of reinforcement parameters were considerably different in short and tall GRS walls (Alavi et al., 2023a, 2023b).

Seismic loadings can lead to critical damage and deformations in tall geostuctures, such as GRS walls. The significance of differential settlements and their repercussions underscores the imperative to investigate such deformations due to seismic loads. Therefore, the present study assesses the relative settlements of a tall GRS wall (20 m) under worldwide significant earthquakes (36 records) through dynamic finite element analyses. For static validation of numerical modeling, a large-scale physical model of an MSE wall at the Royal Military College of Canada (RMC) was employed (Bathurst et al., 2000). Additionally, the seismic validation of numerical modeling was evaluated using the results of RMC's shaking table tests on GRS walls conducted by El-Emam and Bathurst (2004). The concrete panels were considered for facing elements and the minimum allowable ratio of 0.8 was used for reinforcement length to wall height ratio (L/H) (Berg et al., 2009a, 2009b). Furthermore, backfill soil modeling employed a hardening soil constitutive model with small-strain stiffness (HS-small).

2. Details of Finite Element Seismic Simulations

2.1. Numerical Modeling

The static verification of numerical modeling was conducted using the results of the physical modeling of the RMC Wall 1 (Bathurst et al., 2000). The RMC Wall 1 comprises dimensions of 3.6 m in height, 3.3 m in width, and 5.95 m in length, featuring a battered facing at an angle of 8° . The facing includes 15×30 cm concrete modular blocks, while six layers of polypropylene geogrids serve as reinforcements with an L/H ratio of 0.7 and vertical spacing of 0.6 m (Figure 1a). Moreover, the Wall 8 of dynamic shaking table experiments at RMC was considered for seismic validation of the numerical modeling (Figure 1b) (El-Emam &

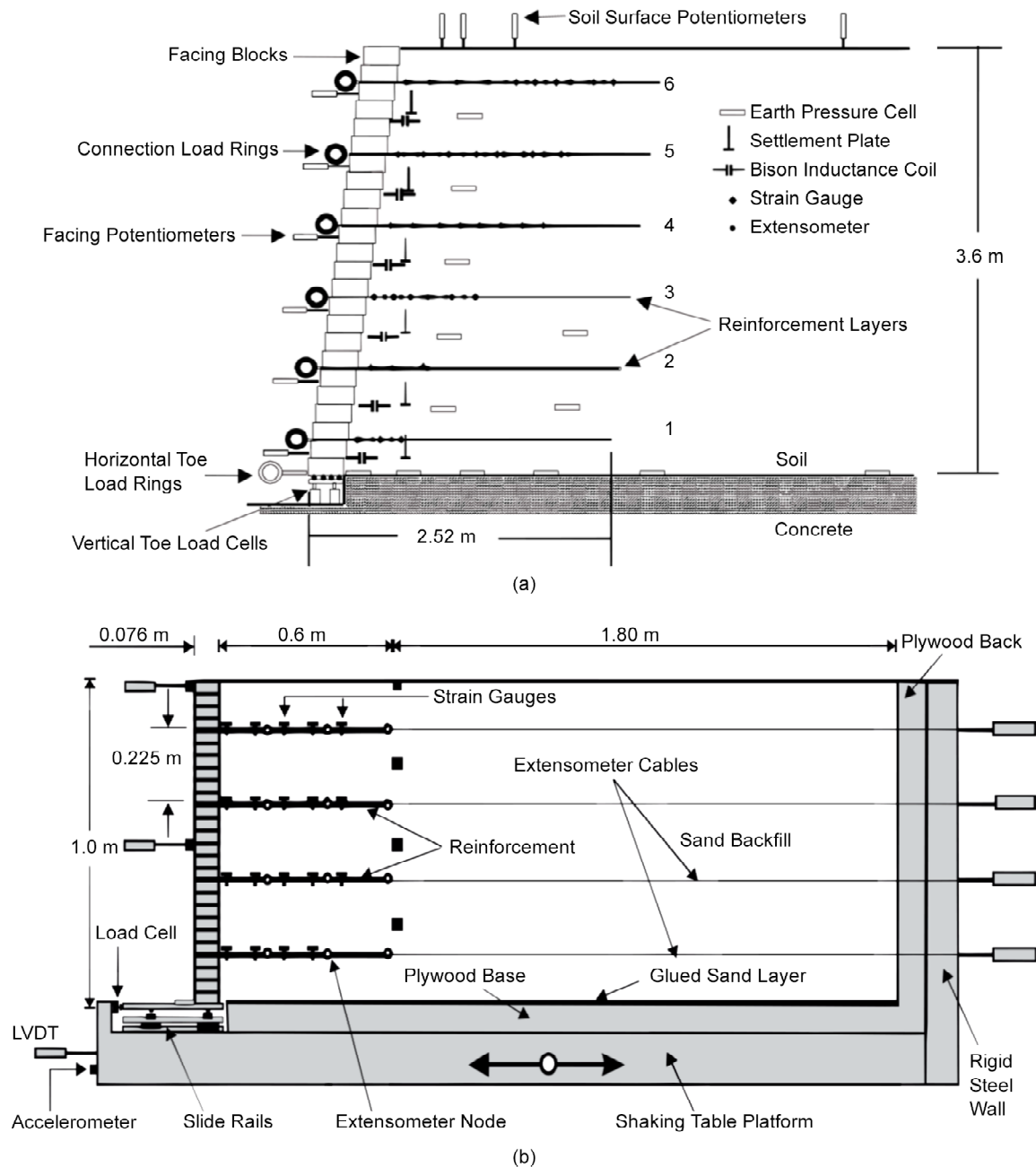


Figure 1. Configuration of (a) RMC's large-scale Wall 1 (static verification case) (Bathurst et al., 2000), and (b) RMC's shaking table Wall 8 (dynamic verification case) (El-Emam & Bathurst, 2004).

Bathurst, 2004). Stage constructions and physical model boundary conditions were considered in the corresponding finite element analyses conducted in Plaxis 2D software (Brinkgreve et al., 2019). Detailed model parameters employed for the verification processes of Wall 1 (static validation) and Wall 8 (dynamic validation) are presented in Table (1). It is noteworthy to mention that interface properties were established based on recommendations from El-Emam et al. (2004); Hatami & Bathurst (2006), as detailed in Table (1).

The hardening soil with small strain (HS-small)

constitutive model proposed by Benz (2007), which implements a hyperbolic function to predict stress-strain relationship of soils, was considered for backfill soil modeling.

The HS-small model has been successfully employed for geotechnical applications in literature (Brinkgreve et al., 2010; Ezzeldin & El Naggari, 2021; Obrzud, 2010). Equations (1) and (2) demonstrates tangent stiffness for primary oedometer loading test (E_{oed}) and secant stiffness in standard drained triaxial test (E_{50}) as an example, respectively:

Table 1. Material properties in numerical modeling of GRS walls.

Attribute	Unit	Case		
		Wall 1	Wall 8	20 m Wall
Backfill:		HS-Small		
Unit weight (γ)	kN/m ³	16.8	15.7	18.64
Friction angle (φ)	Deg (°)	40	56	42
Cohesion (c)	kPa	0.2	0.1	0.1
Dilation angle (Ψ)	Deg (°)	11	14.5	16
Secant stiffness in triaxial test (E_{50}^{ref})	kN/m ²	31600	15000	30000
Tangent stiffness in oedometer test (E_{oed}^{ref})	kN/m ²	31600	9000	30000
Unloading/reloading stiffness in triaxial test (E_{ur}^{ref})	kN/m ²	95000	60000	90000
Stiffness stress dependency factor (m)		0.37	0.55	0.55
Unloading / reloading Poisson's ratio (ν_{ur})		0.2	0.08	0.25
Reference stress (p^{ref})	kN/m ²	100	100	100
Horizontal earth pressure coefficient (k_0^{nc})		0.35	0.17	0.4
Failure ratio (R_f)		0.94	0.9	0.9
Shear modulus of small strains ($\varepsilon<10^{-6}$) (G_0^{ref})	kN/m ²	117000	30000	108000
Threshold shear strain ($G_s = 0.722 G_0$) ($\gamma_{0.7}$)		0.0002	0.0002	0.0002
Rayleigh damping (α, β)			1.57, 0.0015	0.06271, 1.59E-05
Foundation:		Mohr-Coulomb		
Unit weight (γ)	kN/m ³	-	-	24
Friction angle (φ)	Deg (°)	-	-	41
Cohesion (c)	kPa	-	-	25000
Poisson's ratio (ν)		-	-	0.25
Elastic modulus (E)	GPa	-	-	9
Rayleigh damping (α, β)		-	-	0.6281 , 1.59E-04
Geogrid:		Elastoplastic		
Stiffness at 2% strain (J)	kN/m	100	90	1000
Ultimate strength (T_y^a)	kN/m	14	13	180
Facing:		Linear Elastic		
Unit weight (γ)	kN/m ³	22	18	22
Elastic modulus (E)	GPa	25	200	25
Poisson's ratio (ν)		0.2	0.2	0.2
Interface:				
Hinged toe: normal stiffness (K_n)	MN/m ³	-	1200	-
Hinged toe: shear stiffness (K_s)	MN/m ³	-	1200	-
Block-block: normal stiffness (K_n)	MN/m ³	1000	-	1000
Block-block: shear stiffness (K_s)	MN/m ³	40	-	40
Soil-facing: normal stiffness (K_n)	MN/m ³	100	100	100
Soil-facing: shear stiffness (K_s)	MN/m ³	1	60	1
Fixed-end anchor:				
Axial stiffness (J)	MN/m/m	10	-	

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cos \varphi - \sigma'_s \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m \quad (1)$$

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos \varphi - \sigma'_s \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m \quad (2)$$

where E_{50}^{ref} is a reference stiffness modulus corresponding to the reference confining pressure

(p^{ref}), E_{oed}^{ref} is a reference stiffness modulus corresponding to the reference confining pressure, p^{ref} is reference stress for stiffnesses (assumed to be 100 kPa), m is constant representing power of stress-level dependency, σ'_s is minor principal effective stress, c is cohesion, and φ is internal friction angle. Figure (2) depicts acceptable predictions regarding verification results of soil

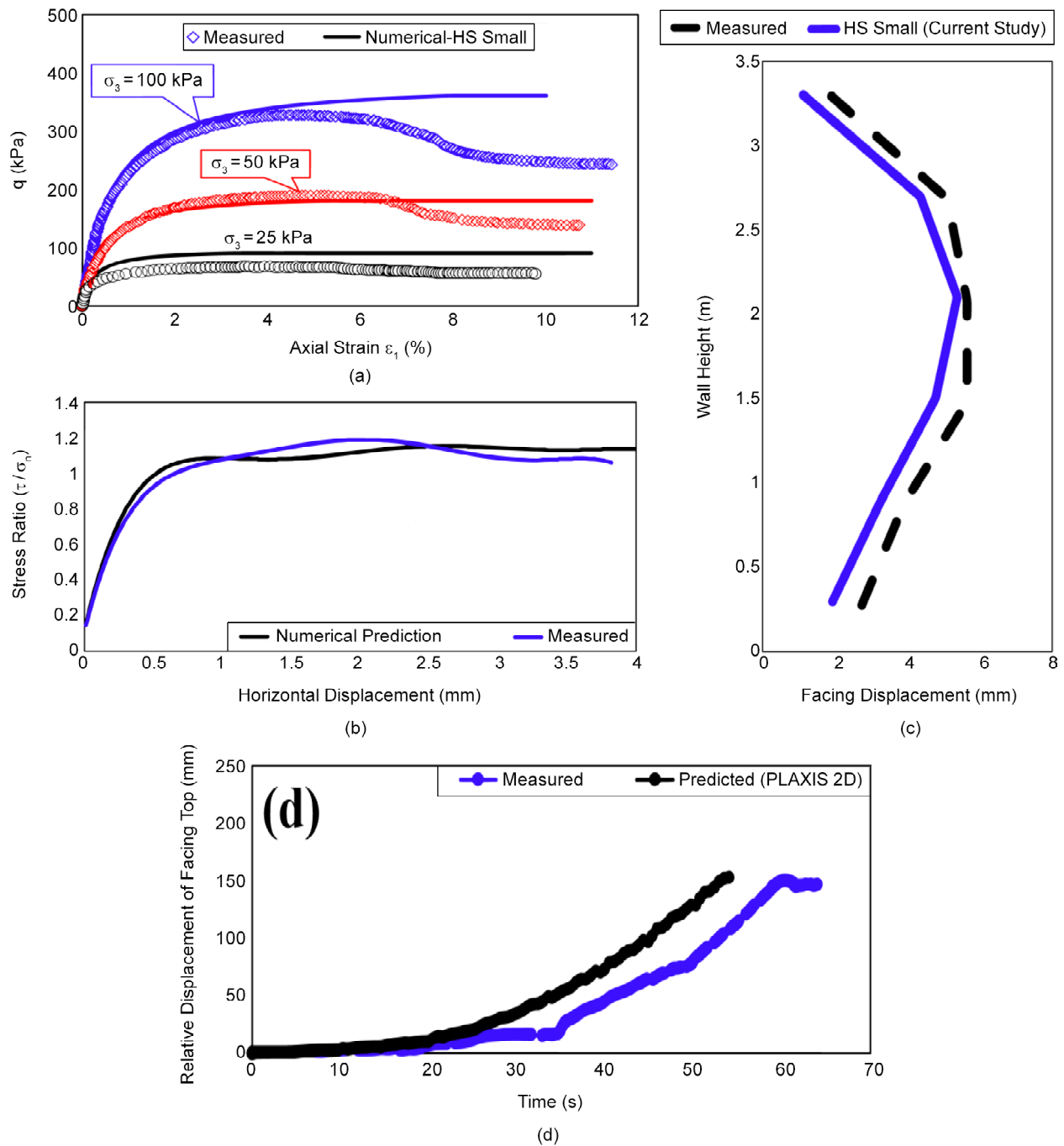


Figure 2. Numerical modeling validation: (a) Soil behavior in static case, (b) Soil behavior in dynamic case, (c) Deformations in static case and (d) Deformations in dynamic case.

behavior and deformations of numerical modeling (i.e., Wall 1 and Wall 8), respectively.

Until 2018, Koerner & Koerner (2018) reported that among 320 damaged MSE walls, 11 were over 16 m in height and 20 were over 20 m in height. Thus, this study investigates the relative settlements of GRS walls under seismic loading through finite element analyses on a typical 20 m wall shown in Figure (3). The numerical models considered the minimum acceptable L/H for seismic loading scenarios as proposed by FHWA

(i.e., $L/H = 0.8$, $L = 16$ m). The HS-small model was implemented to model the Hostun dense sand for the backfill soil (reported by Benza, 2007). Moreover, the properties of the foundation soil were derived from a 6 m GRS wall in Kansas, as outlined by (Pierson, 2008; Pierson et al., 2008), to replicate the foundational condition of an actual GRS wall.

In the concept of the mesh size of numerical modeling recommendations of the previous studies have been considered. Increasing the number of elements with a finer mesh size results in

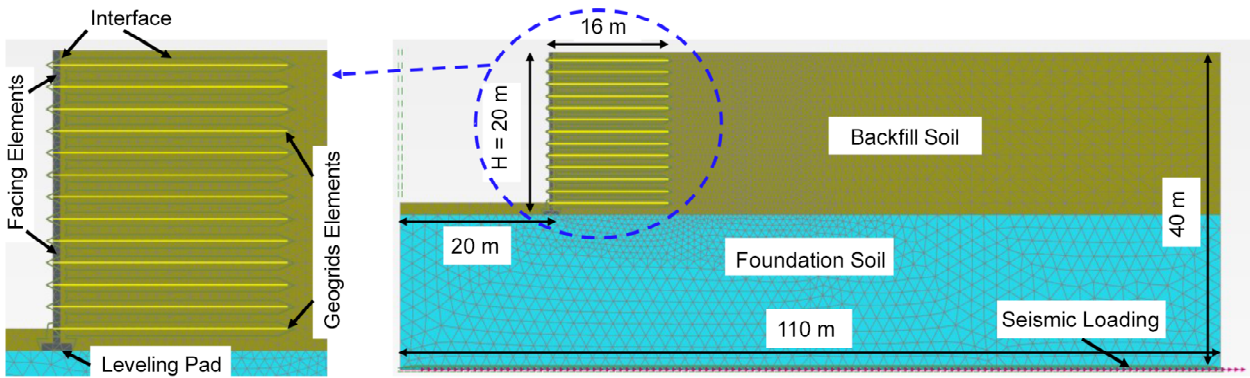


Figure 3. Detailed description of a GRS wall numerical model for evaluating relative deformations.

simultaneously increasing the accuracy of the model and analysis time. Hence, an adequate mesh size should be considered in the numerical modeling to avoid a long analysis time. Therefore, a fine-size mesh was generated in the numerical model. The automatically meshing option in the Plaxis software considers very finer mesh size near any inclusions in the model, which means very fine mesh size in the reinforced soil zone and corresponding geogrids. Hence, over 7000 soil elements were generated in model with the maximum element size of about 2.5 m. The propagating wave in the model may experience numerical distortion depending on the modeling conditions in a seismic analysis. The accuracy of wave transmission is influenced by the frequency content of the input wave and the wave speed characteristics of the system (Itasca Consulting Group, 2011). In this regard, Kuhlemeyer & Lysmer (1973) indicated that to accurately represent wave transmission through a model, the element size (L) should be limited to one-tenth to one-eighth of the wavelength (λ) associated with the highest frequency component of the input wave.

$$L \leq \frac{\lambda}{10} \text{ or } \frac{\lambda}{8}, \lambda = \frac{C}{f} \quad (3)$$

where C is the shear wave velocity and f is the frequency of loading. Considering a maximum frequency content of 10 Hz and with shear wave velocity of more than 375 m/s for the compacted reinforced soil, the maximum element size should be limited to 3.75-4.68 m. Hence, considering this criteria, an acceptable mesh dimensions with the maximum element size of 2.5 m was considered in the numerical modeling. Besides, it should be noted

that this values represent the maximum element size, while the average element size was about 1 m, considering the element size of less than 0.4 m in adjacent of the geogrids.

The interface properties were considered the same as the full-scale Wall 1 due to the similarity between the strength and stiffness parameters of the materials. It should be noted that the interface properties of Wall 1 were derived using a series of laboratory interface shear tests (Hatami & Bathurst, 2006). A detailed description of material characteristics of the investigated GRS wall case is provided in Table (1). In reference to the boundary conditions shown in Figure (3) of the numerical model, free-field boundary conditions were employed on the model sides to replicate the semi-infinite soil condition and attenuate the reflected secondary waves within the model. Furthermore, a compliant base boundary condition was applied at the base of the model to absorb reflected waves and facilitate the direct application of the input accelerogram, in accordance with the guidelines outlined in the Plaxis manual (Brinkgreve et al., 2019).

2.2. Seismic Loading Scenarios

Table (2) presents the characteristics of 36 records from eight significant earthquakes, employed in numerical modeling. There are different criteria in literature to estimate the energy propagated during an earthquake. These criteria involve only the amount of energy submitted by the faults, not the exact amount of energy absorbed by structures. Hence, the currently well-known seismic loading parameters cannot fully represent the destructive power of an earthquake record.

Table 2. Properties of selected seismic records.

Record	Earthquake	Station	Magnitude (M _w)	PGV (m/s)	I _a (m/s)
1	Bam	Abaragh	6.6	0.036	0.083
2	Bam	Mohammad Abad-e-Madkoon	6.6	0.12	0.158
3	Manjil	Abhar	7.37	0.49	1.849
4	Manjil	Abbar	7.37	0.39	3.972
5	Manjil	Abbar	7.37	0.474	5.954
6	Imperial Valley-06	Bonds Corner	6.53	0.47	3.699
7	Imperial Valley-06	Bonds Corner	6.53	0.479	5.852
8	Chi-Chi	ILA067	7.62	0.122	0.572
9	Chi-Chi	TCU095	7.62	0.377	2.674
10	Loma Prieta	Capitola	6.93	0.29	2.248
11	Loma Prieta	Capitola	6.93	0.355	4.254
12	Loma Prieta	Waho	6.93	0.282	3.244
13	Loma Prieta	Waho	6.93	0.341	5.415
14	Loma Prieta	Bran	6.93	0.404	4.928
15	Loma Prieta	Bran	6.93	0.542	5.069
16	Tottori	OKY004	6.61	0.134	2.754
17	Tottori	OKY004	6.61	0.231	6.798
18	Tottori	TTR007	6.61	0.288	2.762
19	Tottori	TTR007	6.61	0.478	4.588
20	Northridge-01	LA - Century City CC North	6.69	0.233	1.157
21	Northridge-01	LA - Saturn St	6.69	0.394	1.572
22	Northridge-01	LA - Hollywood Store FF	6.69	0.279	1.902
23	Northridge-01	Beverly Hills - 12520 Mulhol	6.69	0.265	2.292
24	Northridge-01	Beverly Hills - 12520 Mulhol	6.69	0.359	2.893
25	Northridge-01	Castaic - Old Ridge Route	6.69	0.522	2.769
26	Northridge-01	Castaic - Old Ridge Route	6.69	0.567	3.153
27	Northridge-01	Beverly Hills - 14145 Mulhol	6.69	0.55	3.062
28	Northridge-01	Beverly Hills - 14145 Mulhol	6.69	0.642	4.461
29	Tohoku	Ootawara	9	0.39	8.572
30	Tohoku	Yabuki	9	0.568	9.842
31	Tohoku	Yabuki	9	0.587	10.546
32	Tohoku	Batou	9	0.353	11.809
33	Tohoku	Mito	9	0.399	12.613
34	Tohoku	Mito	9	0.305	13.063
35	Tohoku	Iwanuma	9	0.438	15.854
36	Tohoku	Kasama	9	0.601	17.695

In this regard, recognizing the limitations of the peak ground acceleration (PGA) as an accurate measure of earthquake destructive power (Baziar & Alavi, 2024; Krishna & Bhattacharjee, 2017), an effort has been made to encompass records featuring a diverse range of peak ground velocity (PGV) and Arias intensity (I_a) parameters. The I_a is the cumulative energy per unit of weight in a non-damped single-degree-of-freedom system containing fundamental frequencies with uniform distribution (Bradley, 2015):

$$I_a = \frac{\pi}{2g} \int_0^t [a(t)]^2 dt \quad (4)$$

where $a(t)$ is earthquake acceleration time history, t_{max} is the earthquake duration, and g represents

the gravity acceleration.

3. Results and Discussion

The deformation mode of the numerical model under end of construction condition and under seismic loading condition is demonstrated in Figure (4). It should be noted that the reset displacement to zero option in the Plaxis software was enabled in the seismic loading stage to capture wall deformations only during earthquake record. Sabermahani et al. (2009) revealed the deformation mode of GRS walls is highly dependent on the reinforcement stiffness. The results of Figure (4) illustrates a bulging deformation mode (maximum displacement at mid-height) in the case of end of construction condition (Figure 4a), while the

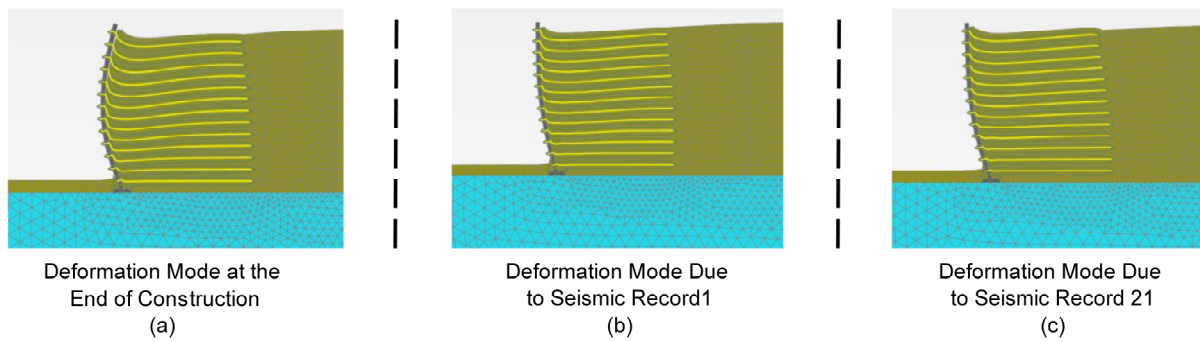


Figure 4. Deformation mode of the numerical model: (a) at the end of construction stage, (b) during record 1 of Table 2, and (c) during record 21 of Table (2).

deformation mode was overturning (maximum displacement at top) during seismic loading conditions (Figures 4b and 4c). Hence, the nature of induced loading also affects the deformation mode of the GRS walls.

Figure (5) illustrates the top settlements of the 20 m GRS wall model in response to various seismic records. Six different zones were considered to evaluate relative displacements. These zones include the facing, the maximum deformation in the reinforced zone (approximately 5 m away from the facing), the end of the reinforced zone (approximately 15.5 m away from the facing), the beginning of the retained zone (situated at the back of the geogrids, approximately 16.5 m away from the facing), the middle of the retained zone (approximately 45 m away from the facing), and a location far from the facing (approximately 60 m away from the facing).

The findings presented in Figure (5) demonstrate two critical locations in which significant relative deformations occurred. One of these locations was between the facing and reinforced zone. The differential settlements between the end of the reinforced zone and the beginning of the retained zone had also high values and were a potentially high-risk point. These two zones were both prone to have the most differential settlements, and the most critical location varied depending on the intensity of the seismic records. The graphs of Figure (5) illustrate that differential deformations due to seismic loadings include a large percentage of absolute deformations and need to be noticed, as well. Furthermore, it was observed that models exhibiting greater absolute deformations also demonstrated more relative settlements. It is worth noting that increasing the reinforcement length

may reduce the differences in settlement between the reinforced and retained zones, as the minimum L/H value was considered in the current study. Moreover, it seems that after a 60 m (equal to $3H$ in the current study) distance away from facing, absolute settlements are relatively low.

Figure (6) represents the settlements of the GRS wall model during the record 1 of Table (2). The maximum settlement occurred nearly behind the facing. These deformations decreased until reaching the end of the reinforced zone, resulting in differential settlements which should be controlled during design analysis. A breakpoint was observed in Figure (6b) in the back of the reinforced zone due to increasing settlements. This surge can be attributed to the overturning mode of the wall and the reinforced zone acting as a block. This behavior is in line with the assumptions of various design guidelines of the GRS walls. Finally, the settlements decreased to a negligible value by getting away from the facing, as denoted in Figures (5) and (6b).

To gain a more comprehensive understanding of the effect of seismic excitations on the differential settlements, further research has been conducted in the current study. These investigations involved examining the variations of normalized relative deformations resulting from various seismic records, as represented in Figure (7). The results in Figure (7) were used to observe and evaluate the correlations between differential settlements and different seismic events, providing valuable insights into the destructive power of seismic excitations and their impact on the relative deformations of GRS walls.

In this regard, Figures (7a) and (7b) present the effects of increasing PGV and I_a , respectively, on

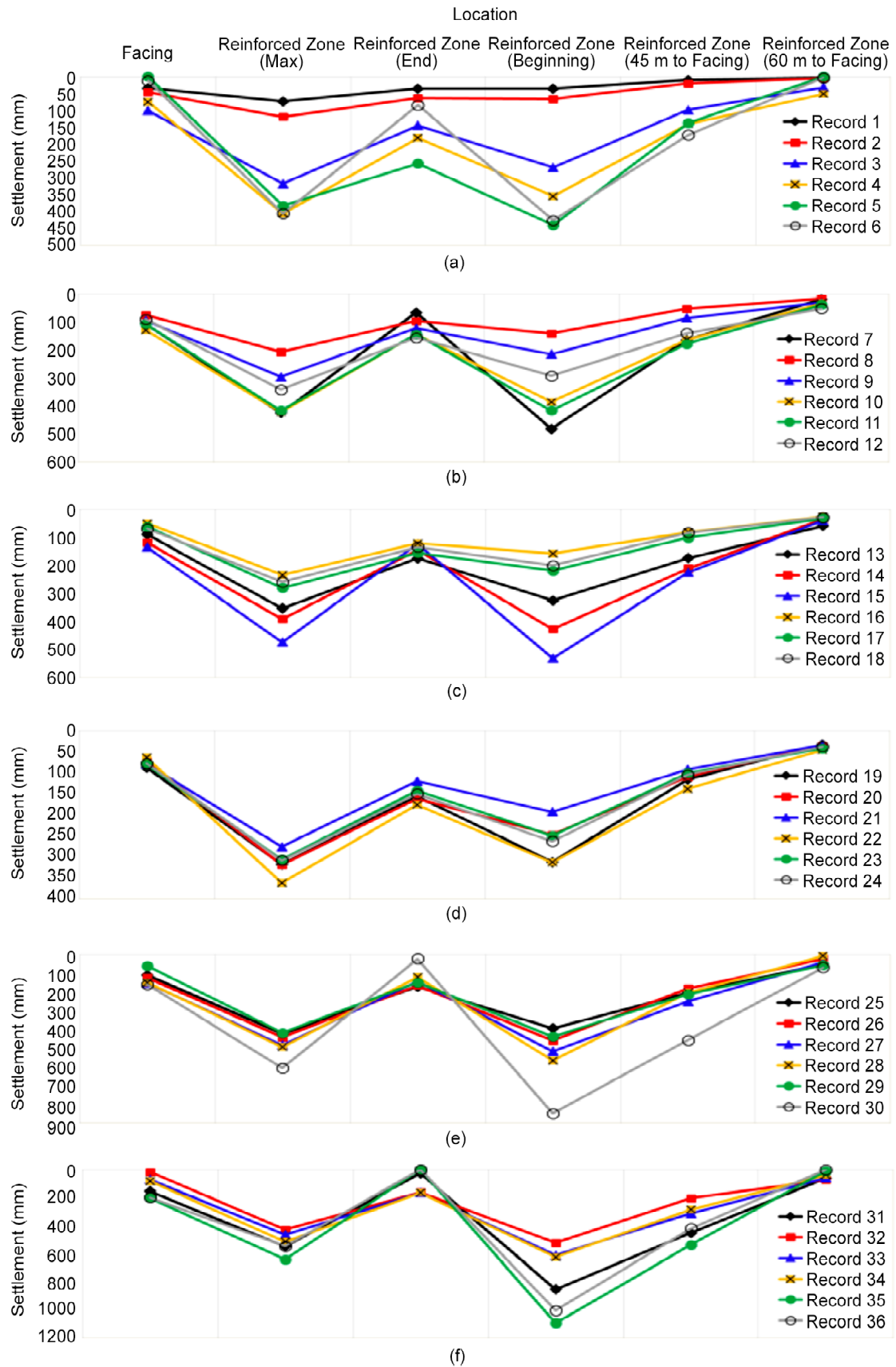


Figure 5. Settlements of the top of the 20 m GRS wall model due to various seismic records.

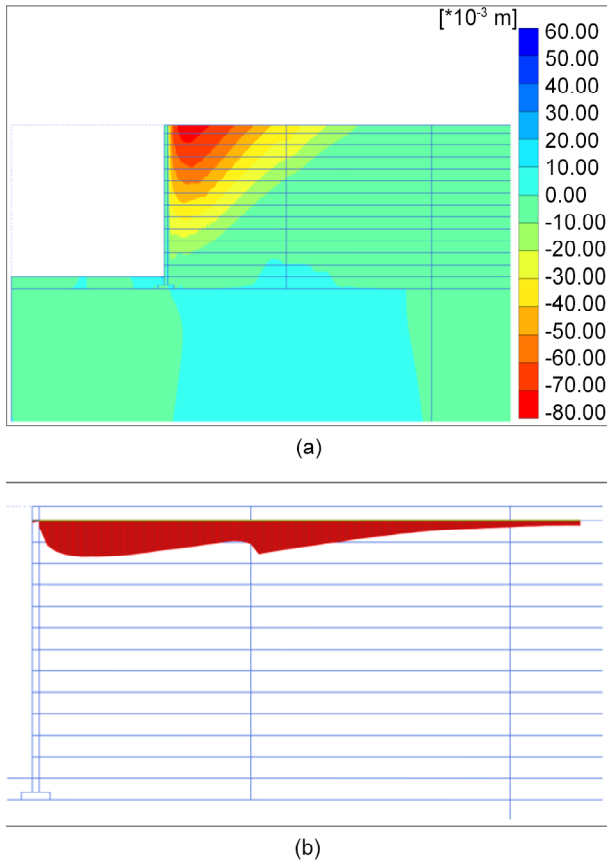


Figure 6. Settlements of the numerical model during the record 1 of Table (2): (a) deformation of whole model and (b) deformation distribution perpendicular to facing.

the normalized differential settlements between the facing and reinforced zone with wall height (i.e., $\Delta S_{\text{Facing - Reinforced zone}} / H$).

Considering that both graphs in Figures (7a) and (7b) are derived based on the same database of records and models, it becomes apparent that the variations in $\Delta S_{\text{Facing - Reinforced zone}} / H$ were more dependent on PGV values than on I_a . Hence, it emphasizes the more significant influence of PGV values on such relative settlements. Furthermore, the rising trend of $\Delta S_{\text{Facing - Reinforced zone}} / H$ decreased after increasing I_a to values greater than 10 m/s.

Figures (7c) and (7d) present the variations of normalized differential settlements in the other mentioned critical location, between reinforced and retained zones ($\Delta S_{\text{Reinforced zone - Retained zone}} / H$). It is noteworthy to point out that, in contrast to the trends observed in Figure (7b), there are sharp rising trends in both graphs of Figures (7c) and (7d) for all PGV and I_a values. Upon closer examination of the relative deformations depicted in Figure (7), it is evident that the $\Delta S_{\text{Facing - Reinforced zone}} / H$ values were limited to about 2.5%, while $\Delta S_{\text{Reinforced zone - Retained zone}} / H$ had higher values up to

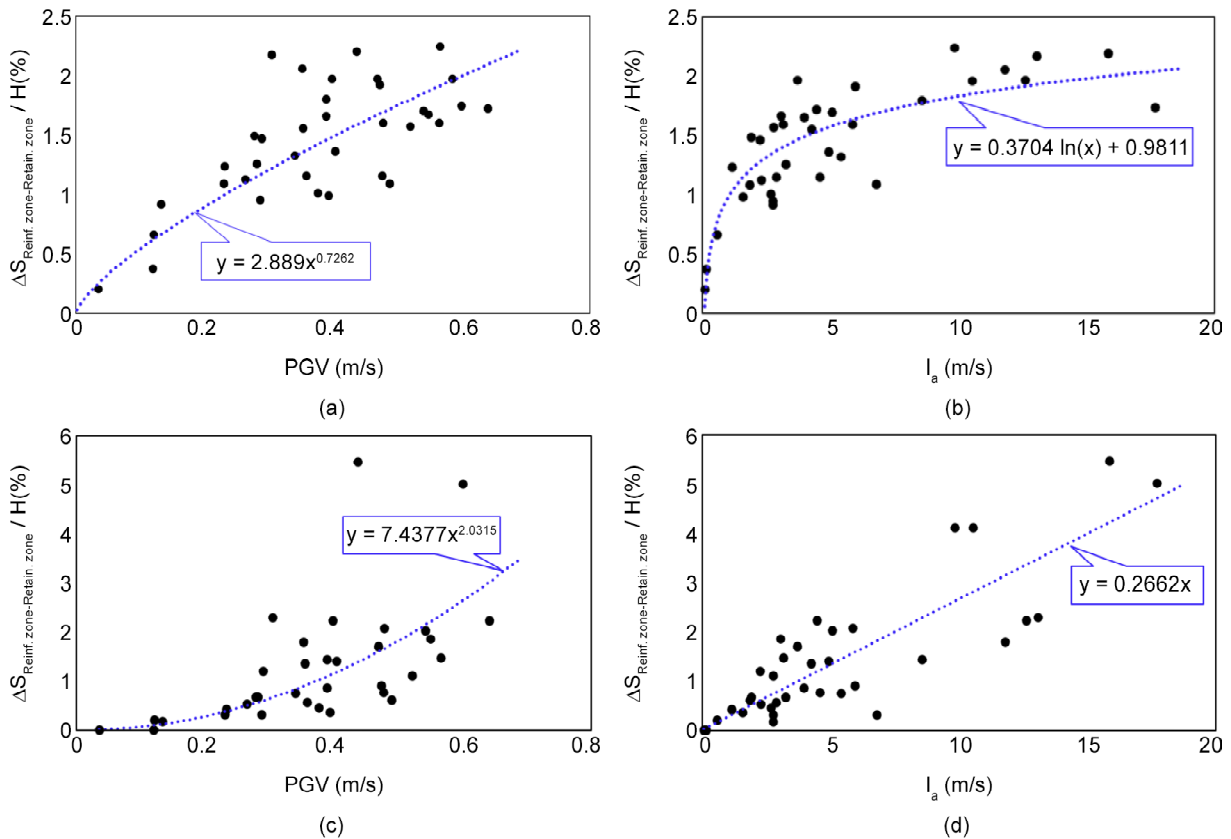


Figure 7. (a) Differential settlements between facing and reinforced zone due to PGV variations, (b) differential settlements between facing and reinforced zone due to I_a variations, (c) differential settlements between retained and reinforced zones due to PGV variations, and (d) differential settlements between retained and reinforced zones due to I_a variations.

5.5%, which is considerably high.

4. Summary and Conclusion

The study has assessed the relative settlements of a tall GRS wall (20 m) under 36 worldwide significant seismic records, utilizing dynamic finite element analyses. To validate the numerical modeling under static loading conditions, a large-scale physical model of an MSE wall at the RMC was modeled in the Plaxis 2D software.

Furthermore, the seismic validation of the numerical modeling was conducted based on the outcomes of RMC's shaking table tests on GRS walls. In the case of the facing elements, 1.5×0.5 meter concrete panels were modeled, with a minimum allowable ratio of 0.8 employed for the reinforcement length to wall height ratio. Besides, the backfill soil modeling incorporated a hardening soil constitutive model with small-strain stiffness. The major outcomes of this research study can be listed as the following:

- The deformation mode of the numerical model varied at the end of the construction conditions and due to seismic loading conditions. The bulging deformation mode was observed at the end of the construction stage of the wall, while the overturning mode was the main deformation mode during earthquake records.
- Relative deformations were found to be particularly significant in two specific areas of a tall GRS wall model. These critical locations include the between the facing and reinforced zone, as well as between the reinforced and retained zones. The maximum settlements occurred at the back of the facing and it decreased until the end of the reinforced zone. A breakpoint was observed then at the back of reinforced zone due to increasing settlements. Finally, the settlements were reduced by getting away from facing and became negligible. These findings emphasize the importance of closely monitoring and addressing deformations in these key areas to ensure the non-occurrence of differential settlements and their consequences in GRS walls.
- Seismic records have shown that there are significant relative settlements when compared to absolute settlements, and these settlements

are directly affected by the intensity of earthquakes. This means that the amount of such relative deformations can be directly linked to the force of the earthquake.

- The absolute settlements at a distance of $3H$ away from facing were relatively low and negligible. Hence, there was a safe distance zone for the 20 m GRS wall considering such problematic deformations.
- The differential settlements between the facing and reinforced zones showed a rising trend due to the increasing I_a of seismic records. However, the slope of this rising trend began to decrease after the I_a values exceeded 10 m/s.
- Finally, the equation of four trends of correlations between 1: PGV and 2: I_a values of earthquake records and relative deformations between 1: facing and reinforced zones, and 2: between reinforced and retained zones, were predicted based on the results of the current study.

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