Geosynthetics, J. Kuwano & J. Koseki (eds)

Numerical modelling of a geosynthetic reinforced soil retaining wall subjected to seismic loading

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Keywords: reinforced soil retaining walls, numerical modelling, seismic loading

ABSTRACT: The paper presents a parametric study that was carried out to analyze the influence of the restraining condition at the toe of the facing panel, the input motion, facing panel rigidity and facing panel inertial forces on the seismic response of a 6 m high geosynthetic-reinforced soil retaining wall with a continuous facing panel. The two-dimensional explicit dynamic finite difference program FLAC (Fast Lagrangian Analysis of Continua) was used. The study shows that the restraining condition at the toe of facing panel has great influence particularly on the reinforcement tensile loads. The facing panel rigidity has a large influence on the pattern of lateral displacements and on reinforcement tensile loads.

1 INTRODUCTION

In recent earthquakes the performance of reinforced soil retaining walls was diverse. The Hyogoken-Nambu Earthquake caused serious damage to conventional masonry retaining walls, unreinforced concrete gravity-type retaining walls and cantilever type steel-reinforced concrete retaining walls, while geogrid-reinforced soil retaining walls, having a full-height concrete facing, performed very well during the earthquake (Tatsuoka et al. 1996). On the other hand, the Chi-Chi earthquake, in Taiwan, caused serious damage to reinforced-soil retaining walls using keystones as facing (Koseki & Hayano 2000).

Usually reinforced soil retaining walls are designed using limit-equilibrium pseudo static methods. These methods are dependent only on peak ground acceleration, and disregard the effects due to duration of seismic action, frequency, foundation condition, stiffness of the reinforcement, facing type and other factors.

In this work the two-dimensional finite difference program Fast Lagrangian Analysis of Continua (FLAC) was used to carry out parametric analyses (Itasca 2005). FLAC is an explicit dynamic code, suitable for modelling large distortions and dynamic response of earth structures. This code has also been used to investigate seismic response of reinforced soil retaining walls in Royal Military College of Canada (Bathurst & Hatami 1998, El-Emam 2003).

2 STATIC MODELLING

2.1 *General*

The study regards a reinforced wall of height H = 6 m with ten (n = 10) horizontal reinforcement layers, uniformly spaced, of length L = 4.2 m, attached to a continuous facing panel. The wall and soil regions were supported by a stiff foundation with 1 m thick.

The reinforcement length, L, was selected to give L/H = 0.7, where H is the height of the structure. This ratio value of L/H is the minimum recommended by the FHWA (Elias et al. 2001) for static design.

The numerical grid is illustrated in Figure 1. The width of the backfill was extended to 35 m beyond

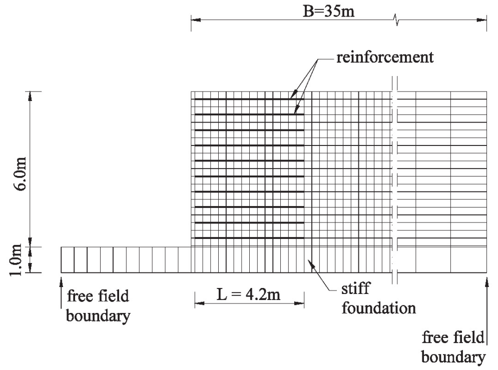

Figure 1. Numerical grid for the sliding case.

the back of the facing panel and free-field boundary conditions (Itasca 2005) were applied to left and right side vertical boundaries. The grid was selected to represent an infinitely wide region.

The fill was modelled as a purely frictional elastoplastic material, with a Mohr-Coulomb yield function and a non-associated flow rule ($\psi = 0°$). The friction angle of the soil was $\phi = 35°$ and the unit weight $\gamma = 22$ kN/m^3. The bulk and shear modulus values of the soil were K = 50.0 MPa and G = 23.1 MPa, respectively. The wall facing was assumed as an elastic material. For the reference case it was assumed a facing panel with thickness equal to 0.15 m, 10 GPa and 0.2 for the Young modulus and Poisson ratio, respectively.

The wall was constructed in 20 layers and it was assumed that the wall facing was fully supported in the horizontal direction during construction. The panel supports were released in sequence from the top of the structure.

The reinforcement layers were modelled using linear elasto-plastic cable elements with negligible compressive strength. The interface between the reinforcement and the soil was modelled by a grout material (Itasca 2005) with an interface friction angle of 29° and a bond stiffness of 5×10^6 kN/m/m.

The linear elastic stiffness value for the reinforcement was taken equal to 1000 kN/m.

The facing panel-backfill interface was modelled using interface elements with a friction angle of 20°, a normal stiffness kn = 2×10^6 kPa/m and a shear stiffness ks = 2×10^6 kPa/m.

As regards the restraining condition at the toe of the facing panel, two conditions were analyzed. The facing panel could be hinged at the toe (Figure 2a), or free to slide (Figure 2b).

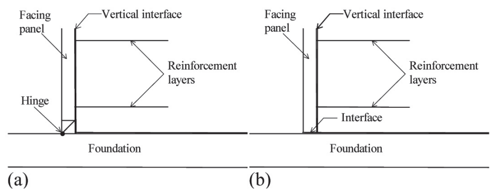

Figure 2. Conditions at the toe of the facing panel: (a) hinged; (b) free to slide.

The hinged case (Figure 2a) corresponds to a situation in which the facing panel was fixed to the foundation but was free to rotate. For the sliding case (Figure 2b) the facing panel was seated on a thin layer of soil, extended across the full width of the grid, with friction angle equal to 20° and remaining parameters having the same values of backfill soil properties. At this case the wall is free to slide horizontally and rotate about the toe.

2.2 *Influence of the restraining condition at the toe of the facing panel*

The influence of the restraining condition at the toe of the facing panel on the lateral displacements and reinforcement connection loads at the end of construction is illustrated on the Figure 3a and b, respectively. Note that the reinforcement loads are greatest at the connections for both restraining conditions.

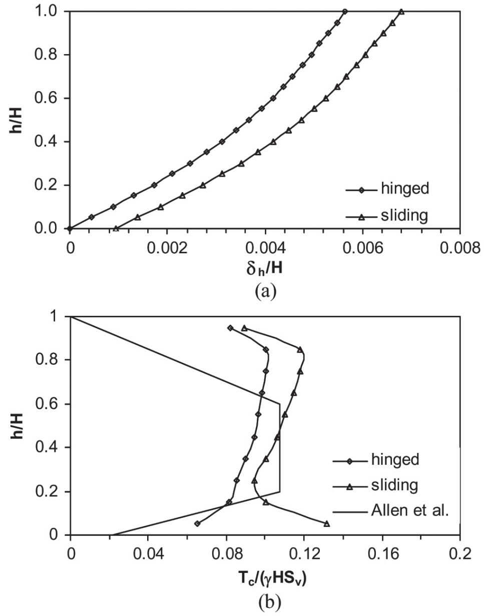

Figure 3. Influence of the toe restraining condition: (a) – on the normalized lateral displacements; (b) – on the normalized connection loads.

The lateral displacement (δ_h) was normalized by the wall height (H). The reinforcement connection loads appear normalized by γHS_v where, γ is the unit weight of the soil, H is the wall height and S_v is the vertical spacing between reinforcement layers. It is also represented the distribution proposed by Allen et al. 2003 developed using a database of 11 full-scale geosynthetic walls and considering the stiffness of the various wall components. The distribution proposed by Allen et al. 2003 presented in the Figure 3b considers the facing stiffness factor (one of the factors considered) equal to unit.

The pattern of the lateral displacements for the two conditions is similar. The sliding case leads to greater displacements and reinforcement forces, particularly near the toe. The proposal of Allen et al. 2003 seems to underestimate the tensile loads particularly at lower and upper layers.

More details about the influence of the restraining condition at the toe of the facing panel can be found in Vieira et al. 2005.

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3 DYNAMIC MODELLING

3.1 *Seismic action and fundamental frequency*

Two types of seismic action were considered in the dynamic analyses: an earthquake ground motion artificially generated with the program SIMQKE (Gasparini & Vanmarcke 1976) according to NP-ENV 1998-1-1 (NP-ENV 1998-1-1 2000) for the greater seismicity area of Portugal (Figure 4a) and a variable amplitude single frequency harmonic motion with identical peak ground acceleration (Figure 4b). Two values of frequency were analyzed for the variable amplitude harmonic acceleration: 4 Hz (Figure 4b), close to the wall fundamental frequency and 2 Hz.

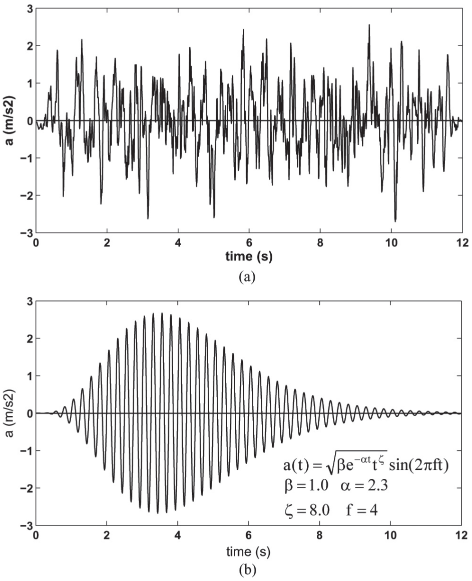

Figure 4. Input accealeration: (a) – earthquake ground motion; (b) – variable amplitude single frequency harmonic acceleration.

The fundamental frequency for a two-dimensional linear elastic medium, contained by two rigid vertical boundaries and a rigid base, and subjected to horizontal base excitation can be expressed as (Bathurst & Hatami 1998):

$$f_1 = \frac{1}{4H}\sqrt{\frac{G}{\rho}} \cdot \sqrt{1 + \left(\frac{2}{1-\nu}\right)\left(\frac{H}{B}\right)^2} \quad (1)$$

where f_1 = frequency, in Hz, corresponding to the first mode shape; H = height of the medium; G = shear modulus; ρ = mass density; ν = Poisson's ratio and B = width of the backfill. The second square root term represents the modification of the one-dimensional frequency formula for an elastic infinitely long uniform medium.

In the limit for an infinitely wide medium, equation 1 becomes the fundamental frequency formula for a one-dimensional elastic medium:

$$f_1 = \frac{1}{4H}\sqrt{\frac{G}{\rho}} \quad (2)$$

Considering that the width of the backfill was extended to 35 m beyond the back of the facing panel and free-field boundary conditions were applied to left and right side vertical boundaries (Figure 1), the fundamental frequency of the wall can be estimated by equation 2. According to this equation the fundamental frequency of the structure is 4.2 Hz.

The fundamental frequency of the structure was also evaluated with FLAC, applying an impulse at its base and letting the structure freely shake. The Fast Fourier Transformation (FFT) of the velocity record at one point of the retaining soil is presented in the Figure 5. This figure corroborates that the frequency corresponding to the first mode is approximately 4.2 Hz.

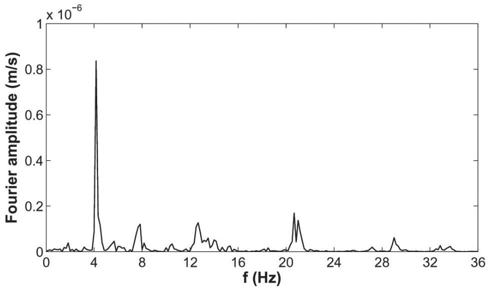

Figure 5. Fast Fourier transformation for one velocity record.

3.2 *Influence of the restraining condition at the toe of the facing panel on the seismic behaviour*

Figure 6 presents the normalized lateral displacements of the wall facing and maximum tensile loads at the end of the seismic motion (Figure 4a) for the two restraining conditions at the toe. As expected, sliding case leads to greater lateral displacements however, the top lateral displacement is nearly the same. El-Emam 2003 carried out reduced-scale shaking table tests of geosynthetic reinforced soil retaining walls and concluded that for acceleration amplitudes smaller than 0.3 g, the hinged toe and sliding toe models showed approximately the same top lateral displacement.

The maximum reinforcement load in the bottom layer is significantly larger for the sliding case. In the other layers the differences are not significant.

Facing panel normalized horizontal displacements at bottom reinforcement layer level, for the two restraining conditions, are presented in Figure 7a. Figure 7b illustrates the connection load time histories

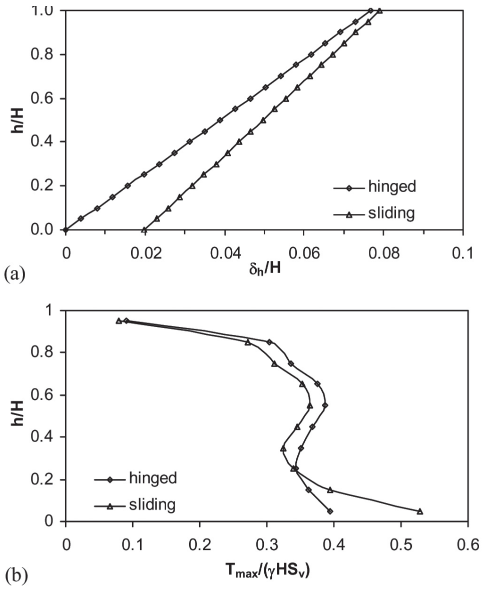

Figure 6. Influence of the restraining condition at the toe: (a) on the normalized lateral displacements; (b) on the normalized maximum reinforcement loads.

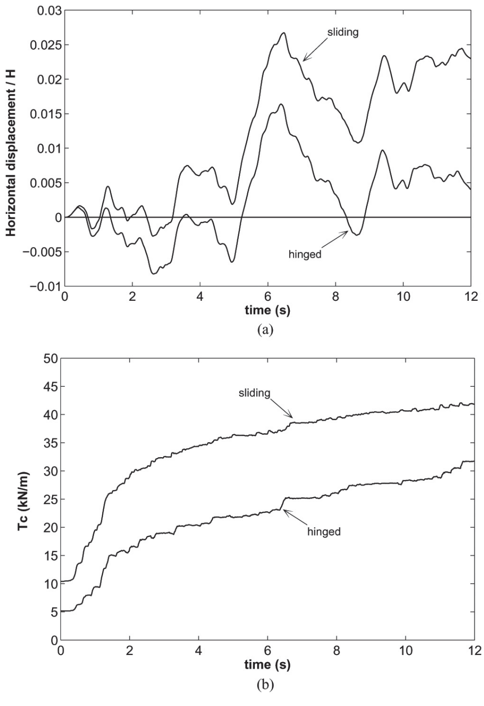

Figure 7. Influence of the restraining condition at the toe. Time histories relating to the bottom reinforcement layer: (a) facing panel normalized horizontal displacement at reinforcement level; (b) connection loads.

at the bottom reinforcement layer for hinged and sliding toe facing panel. Connection loads accumulate with time during the seismic motion. The large connection load occurred for the sliding case is partly consequence of the load measured at the end of construction (see also Figure 3b). For the sliding case the connection load at the bottom reinforcement layer increased approximately four times during the seismic loading while, for the hinged toe condition, the increase was six times the value measured at the end of construction.

3.3 *Influence of the input motion*

In order to investigate the influence of input motion on dynamic response of reinforced soil retaining walls, as was said in 3.1, three solicitations were considered: an artificially generated earthquake and a variable amplitude harmonic motion with frequencies equal to 2 and 4 Hz. Figure 8 illustrates the normalized lateral displacements and the normalized maximum reinforcement loads for these solicitations and for the hinged model.

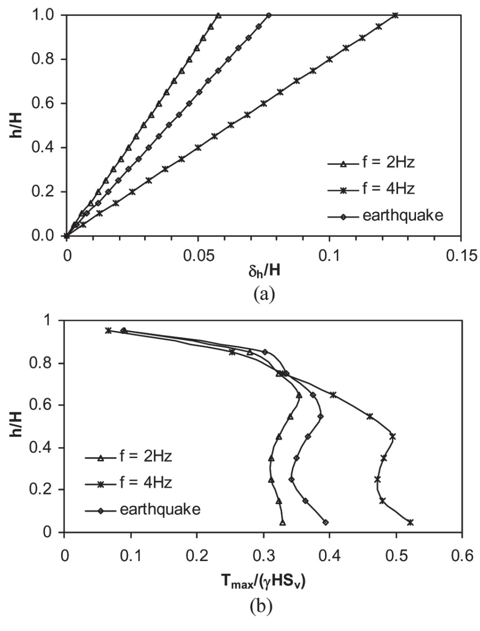

Figure 8. Influence of input motion: (a) on the normalized lateral displacements; (b) on the normalized maximum reinforcement loads.

The variable amplitude harmonic motion with frequency of 4 Hz induces, not unexpectedly, very large lateral displacements and reinforcement loads.

It should be noted that this frequency is close to the fundamental frequency of the structure. This input motion is much more aggressive to the structure than the earthquake loading. On the other hand, the variable amplitude harmonic motion with frequency of 2 Hz is less aggressive than the earthquake loading.

Note that the earthquake ground motion, presented in Figure 4a, contains a range of significant frequencies between 0.5 and 10 Hz.

The simple single frequency harmonic motion may be useful to analyze the response of reinforced soil retaining structures, however the magnitude of the response may be excessive or diminished.

Bathurst & Hatami 1998 also concluded that a variable amplitude harmonic acceleration with frequency close to the fundamental frequency of the structure may be more aggressive than an earthquake motion with identical peak ground acceleration.

Normalized top horizontal displacement histories for the three dynamic motions are represented in Figure 9. It can be observed that the normalized top horizontal displacement for the harmonic motion with f = 4 Hz increase continuously with time.

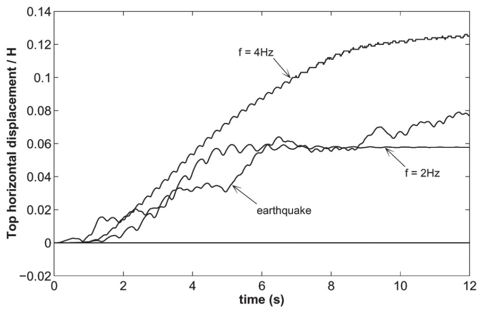

Figure 9. Normalized top horizontal displacement histories for the three dynamic solicitations.

3.4 *Influence of facing panel rigidity*

In what concerns facing panel rigidity (EI), two situations were analyzed. A flexible facing panel with rigidity equal to 66 kNm^2 and a rigid facing panel with EI = 2812 kNm^2. In this section, the facing panel thickness was assumed constant and the elastic modulus was changed.

Figure 10 illustrates the influence of facing panel rigidity on the normalized lateral displacements and normalized maximum reinforcement loads. It can be noted the bending of the facing panel when it is more flexible. More or less unexpected, the top lateral displacement increases with facing rigidity. El-Emam 2003 in his reduced-scale shaking table tests observed that for model walls with a thick facing panel the top lateral displacement was larger compared to model walls with a thin facing panel. He considered that this phenomenon was due the greater destabilizing inertial forces developed in the thick facing panel models.

The results presented in Figure 10 are related to equal facing panel weight, therefore the destabilizing inertial forces theoretically are the same. In fact, the bending of the flexible facing panel leads to smaller

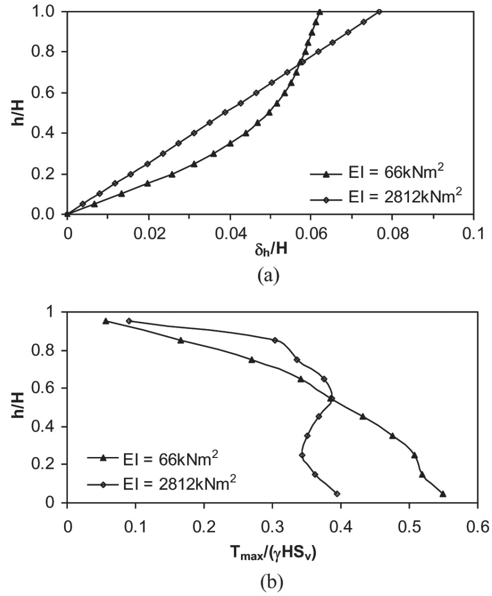

Figure 10. Influence of facing panel rigidity: (a) on the normalized lateral displacements; (b) on the normalized maximum reinforcement loads.

lateral displacements at the upper zone of the wall.

Regarding the normalized maximum reinforcement loads, it can be observed that the increase of facing panel rigidity leads to greater reinforcement loads at upper layers and the opposite trend at lower layers. For the flexible facing panel, the maximum reinforcement load distribution tends to a triangular shape. The conclusions of El-Emam 2003 shaking table tests were similar.

Figure 11a illustrates the connection load histories at the bottom reinforcement layer for the two values of the facing panel rigidity.

As above-mentioned, the decrease of facing panel rigidity leads to greater reinforcement load at the bottom layer. The time histories of connection load normalized by the load at the end of construction (T_o) are presented in Figure 11b. The normalized connection load histories are similar. This results from the fact that greater facing panel rigidity leads to lower connection load at the bottom layer for the static situation (end of construction), (see the first instant in Figure 11a). Note that at the end of seismic loading, the greatest normalized connection load occurs for the rigid facing panel (EI = 2812 kNm^2).

3.5 *Influence of the facing panel inertial forces*

With the purpose of isolate the influence of the inertial forces developed in the facing panel, other analyses were made for the flexible facing panel (EI = 66 kNm^2). In the first analysis, the total weight of the wall face (W) was decreased approximately 2.5 times

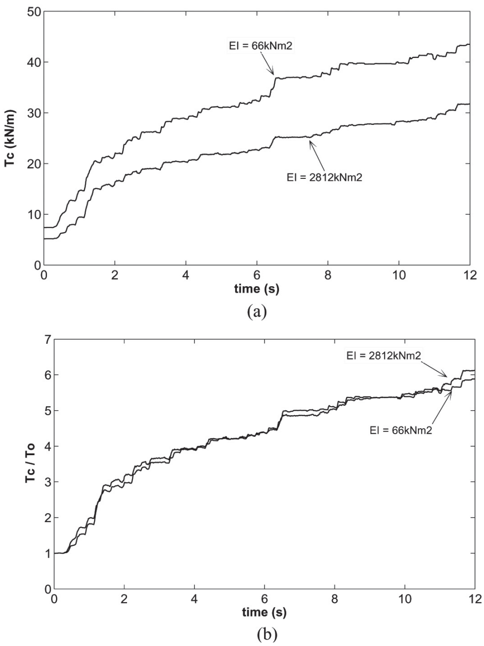

Figure 11. Influence of facing panel rigidity: (a) connection load histories at the bottom reinforcement layer;
(b) connection load normalized by the load at the end of construction (T_o).

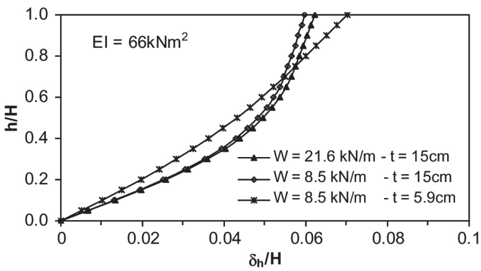

Figure 12. Influence of facing panel inertial forces on normalized lateral displacements.

(W = 8.5 kN/m – t = 15 cm in the Figure 12). In the other analysis, the facing panel thickness (t) and the elastic modulus was changed, but giving the same facing rigidity, and total weight (W = 8.5 kN/m – t = 5.9 cm in the Figure 12).

The observation of the Figure 12 shows that, as expected, for the same facing panel geometry, greater inertial forces lead to larger lateral displacements. For the same inertial forces, the lateral displacements are quite distinct when the geometry of the facing panel is different. This may result from the way the hinged toe condition was modelled (beam elements triangularly arranged - Figure 2a). The influence of facing panel geometry will be presented in future publication.

4 CONCLUSIONS

This study leads to the following conclusions:

- when the toe of the facing panel is free to slide, it is necessary special attention to the reinforcement tensile loads developed at the lower layers;
- a variable amplitude harmonic motion may be more aggressive than an earthquake input motion, particularly when the frequency is close to the fundamental frequency of the structure;
- a more flexible facing panel may lead to smaller lateral displacements at the upper part of the wall but greater reinforcement loads at lower layers;
- for the hinged toe condition, as expected, greater facing panel inertial forces leads to larger lateral displacements. However, for the same inertial forces the pattern of lateral displacements is different when the geometry of the facing panel is changed. The influence of facing panel geometry should be investigated with more detail.

It should be noted that the conclusions of this study are limited to model retaining walls with continuous facing panel, uniform backfill and rigid foundation.

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