Seismic Performance of Single-Propped Retaining Walls

by

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Abstract

This thesis analyzed the dynamic performance of single-propped retaining walls in dry sand under different seismic excitations using the finite difference software FLAC v7.0 (Itasca). The structure comprises two reinforced concrete diaphragm walls connected by a row of cross-lot struts that is used to support a 9.5m deep, 18m wide excavation in dry sand. After simulating the excavation as a staged construction, a suite of thirty-two (32) different seismic inputs were applied at the base of the model. The non-linear, inelastic soil behavior was represented by the advanced PB constitutive model (generalized effective stress soil model) developed by Papadimitriou et al. (2002). In order to avoid spurious reflections of shear waves on the vertical boundaries of the finite difference model, the analyses used periodic boundary conditions.

The performance of the structure was investigated by considering the wall deflections, bending moments, earth pressures and surface settlements for each of the applied ground motions. Based on the horizontal deflection of the walls, three distinct categories of performance were observed and characterized. Results of the parametric study were correlated with the characteristics of the ground motions from which wall deflections and bending moments showed clear correlations with peak ground acceleration and Arias intensity.

Thesis Supervisor: Andrew J. Whittle
Title: Edmund K. Turner Professor of Civil and Environmental Engineering
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1 Introduction

1.1 Problem Statement

Retaining structures of different types are used to support soil during excavation projects, and may either serve as temporary support systems or be part of the permanent structure. The current design methodology for seismic loads incorporates a performance-based design method, where the engineer has to select a maximum, allowable horizontal movement of the retaining wall while the design earthquake forces are applied quasi-statically on the system via the pseudo-static approach. However, recent numerical and experimental studies suggest that the pseudo-static approach implemented in the current performance-based design method can often underestimate the actual total movement of retaining structures.

Nowadays, the analysis of a retaining structure subjected to a seismic excitation can be simulated quite accurately using numerical methods (finite element or finite difference methods). Advanced constitutive models have been developed by many researchers that are able to capture key aspects of soil behavior under cyclic loading. Although these models are quite complicated and can be difficult to calibrate for specific soils, they are useful in research and can contribute to the development of design methods for the dynamic response of retaining structures.

The scope of this thesis is to examine the dynamic response of an excavation supported by two retaining walls and a row of cross-lot strut, constructed in dry sand, using the advanced constitutive model of Papadimitriou et al. (2002) for sands. The numerical analyses performed follow closely the procedures presented by Miriano et al. (2015) which are also used to validate results obtained by these analyses. The main difference introduced in this study is that the retaining structure is subjected to a suite of thirty-two (32) different ground motions, such that correlations between the basic seismic motion characteristics and response parameters can be estimated.
Chapter 1: Introduction

1.2 Organization

A brief summary of the work performed in each chapter of this thesis is presented below.

Chapter 2 summarizes the current design methodology of retaining structures under dynamic loads, and presents a detailed review of the recent research studies and their findings regarding the dynamic response of retaining walls.

Chapter 3 describes the numerical procedure used in this research to simulate the dynamic response of the retaining system. The chapter also summarizes the PB soil model (Papadimitriou et al., 2002) and its input parameters.

Chapter 4 states the scope of the parametric analyses and presents their results. Specific analyses are selected for observation of the different horizontal deflection modes. Correlations are then explored between the basic seismic motion characteristics and the response of the retaining structure.

Chapter 5 presents the main conclusions derived from this research and suggests possible topics for future research.

Appendix A catalogs the acceleration time history and the corresponding Fourier spectra for each of the applied ground motions.

Appendix B catalogs the bending moments, earth pressures and wall deflections together with recorded ground surface settlements (in the retained soil) for each of the excitations.
2 Literature Review

2.1 Introduction

This Chapter reviews the basic literature regarding the response and analysis of retaining structures under seismic loads. Experimental studies (Conti et al. 2012) and numerical ones (Callisto and Soccodato, 2010; Miriano et al., 2015) have been performed in order to i) investigate the seismic response of retaining walls; ii) identify the principal soil-wall failure mechanisms that develop; iii) explore the correlation of different support parameters (ex. embedded depth, wall stiffness) with the accumulated displacements and bending moments of the wall; and finally; iv) propose corrections/improvements to the conventional seismic design based on the Newmark sliding block theory so that to predict more accurately the wall’s behavior. The following paragraphs consider the conventional methodology as well as the most important findings of these recent studies regarding the seismic design of retaining structures.

2.2 Pseudo-static Analysis

It is very common in practice to assess the dynamic behavior of an existing structure by applying the soil forces caused by the earthquake statically over time. Such analysis is usually referred to as pseudo-static analysis. In a pseudo-static analysis, seismic soil forces are constant in space and time similarly to the gravitational forces (weight of the backfill material, weight of the wall, etc.).

The basic principle of this method can be found on the sliding block theory originally proposed by Newmark (1965), evaluating the dynamic response of embankments and dams. The sliding block theory, extended so as to include the dynamic analysis of retaining walls (Seed and Whitman, 1970; Richards and Elms, 1979; Whitman, 1990; Huang et al, 1992) based on concepts for a sliding block on a plane:

The block of mass, m, is subject to a horizontal acceleration \( a(t) \) applied on the base of the mass. The critical value of the acceleration that mobilizes the full friction resistance between the base of the block and the horizontal plane is referred to as \( a_c \). As long as \( a(t) < a_c \) the block and the plane move together, in other words there is no relative displacement between the block and the plane. When \( a(t) = a_c \) relative displacements start to occur. The maximum acceleration that the block may experience is equal to \( a_c \), since for greater values of
acceleration relative displacement occurs. Now, let’s assume that the acceleration of the base stops suddenly at time, \( t=t_0 \). At time \( t_0 \), the base moves with a velocity, \( v_0 \), while the block has a lower velocity (since \( a_c < a(t) \), for a certain time period). The question arises is when will the block and plane start moving together no more relative displacements? The answer is when the block and plane will have the same velocity.

It is now evident how the pseudo-static approach works. The engineer determines the critical acceleration, \( a_c \) of the retaining wall by performing a limit equilibrium analysis (with earth pressures estimated by Mononobe-Okabe equations) with active and passive soil forces. Once the critical acceleration is defined, a comparison between \( a_c \) and \( a(t) \) imposed by the earthquake (i.e. the accelerogram of the desired seismic motion) is carried out and the time intervals in which \( a(t) > a_c \) are determined. By double-integrating the acceleration for both the structure (\( a_c = \text{constant} \)) and ground motion at those intervals, we can calculate the displacements of the structure and the soil. The difference between the displacement values at the same time intervals generates the relative displacement between the soil and structure and hence defines the permanent displacement found at the end of ground seismic event.

Even though pseudo-static analysis is widely used in conventional civil engineering for the seismic assessment of structures, the method has some drawbacks that need to be stated. According to the previous paragraphs, the wall does not deform while \( a_c > a(t) \). However, experimental tests on physical scale models (centrifuge model tests) of cantilever and anchored walls (Zeng, 1990; Zeng and Steedman, 1993) have revealed that walls deform either by sliding or rotating even before the critical acceleration, \( a_c \) of the system is attained. As a result, the accumulation of permanent displacements may be much larger than the value that pseudo-static analysis predicts. Callisto and Soccodato (2007) showed that in order for the pseudo-static analysis to match the results in terms of accumulated relative displacements of the numerical analyses, a fraction of the critical acceleration should be used, since if the total value is considered, pseudo-static analysis tends to under-estimate the accumulation of relative displacements, which is the basic design parameter of the performance-based methodology.

2.3 Current Design Methodology

The current seismic design of gravity retaining walls (Figure 2.1) is based on a performance-based procedure proposed by Richards and Elms (1979), using the framework of the pseudo-static analysis. The first step requires a target maximum value of displacement, \( D \) to be selected which the wall can experience in the case of the design earthquake. It is very common in practice this allowable maximum displacement to be correlated with some characteristics of the design seismic motion such as peak ground acceleration (PGA) and/or peak ground velocity (PGV).
Chapter 2: Literature Review

The desired displacement, D:

\[ D = 0.087 \left( \frac{V^2}{A g} \right) \left( \frac{N}{A} \right)^{-4} \]  \hspace{1cm} (2.1)

where

- \( V \) is the maximum velocity coefficient (given by local seismic codes/regulations)
- \( A \) is the maximum acceleration coefficient (given by local seismic codes/regulations)
- \( g \) is the gravitational acceleration (9.81 m/s\(^2\))
- \( N \) is the maximum friction resistance coefficient

After the calculation of the desired displacement, \( D \), the horizontal acceleration coefficient, \( k_h \), is computed from the following equation:

\[ k_h = G D^{-\frac{1}{4}} \] \hspace{1cm} (2.2)

where

- \( G \) is the displacement coefficient (given by local seismic codes/regulations)

The required mass of the retaining wall, \( M_w \), is given by the following equation:

\[ M_w = \frac{1}{g} C_{IE} E_{AE} \] \hspace{1cm} (2.3)

where

\[ C_{IE} = \frac{\cos(\delta + \beta) - \sin(\delta + \beta) \tan(\phi_p)}{(1-k_v) \tan(\phi_p) - \tan(\theta)} \] \hspace{1cm} (2.4)

\[ E_{AE} = \frac{\gamma H^2 g (1-k_v) \cos(\phi - \theta - \beta)^2}{2 \cos(\theta) \cos(\beta)^2 \cos(\delta + \beta + \theta) \left[ 1 + \frac{\sin(\phi + \delta) \sin(\phi - \theta - \beta)}{\cos(\delta + \beta + \theta) \cos(\phi - \beta)} \right]^2} \] \hspace{1cm} (2.5)

\( C_{IE} \) and \( E_{AE} \) are the inertia coefficient and the active soil force, respectively.

Figure 2.1 illustrates the free body diagram of a retaining wall along with the main variables defined from the above equations. Richards and Elms (1979) also concluded that the point where \( E_{AE} \) applies should be taken equal to half of the wall height, an assumption regarded as sufficient for the majority of engineering designs. Table 2.1 provides the definition of the rest of the parameters stated in Equation 2.4 and 2.5. In addition, Equation 2.5 is derived from the Mononobe – Okabe analysis, which is widely-used in practice for the assessment of the
dynamic behavior of earth structures such as retaining walls, dams, embankments, etc. The last calculation step in the performance-based procedure is to ensure that the wall is sliding rather than tilting due to the ground motion. That can be done by using Equation 2.6. Equation 2.6 calculates a minimum geometric distance, $x_o^{lim}$ that the wall design should meet so as the primary source of deformation to be movement rather than tilt (Figure 2.1). The parameter $x_o$ is defined as the distance between the inner toe of the wall and the point where the resultant force on the base of the wall acts. If $x_o > x_o^{lim}$, then the wall slides and its overall design is regarded as satisfactory.

$$x_o^{lim} = \frac{h \left[ \cos(\beta + \delta) + \tan(\beta) \sin(\beta + \delta) \right] + C_{IE} F \left[ k_h y + (1-k_v) \bar{x} \right]}{\sin(\beta + \delta) + (1-k_v) C_{IE} F}$$

where

- $h$ is the height of the resultant soil force ($h=h/2$)
- $F$ is the safety factor
- and $(\bar{x}, \bar{y})$ are the coordinates of the gravitational centroid of the wall

*Table 2.1: Definition of Parameters*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$</td>
<td>Unit weight of the soil</td>
</tr>
<tr>
<td>$H$</td>
<td>Vertical height of the retaining wall</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Vertical acceleration coefficient</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Soil/wall friction angle</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Wall slope</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Inertia angle</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>Wall/base soil friction angle</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Soil friction angle</td>
</tr>
</tbody>
</table>
Chapter 2: Literature Review

Figure 2.1: Free body diagram of a retaining wall (after Richards and Elms, 1979)

This methodology is referred to as performance-based, since the anticipated movement of the wall is up to a maximum predefined displacement. If instead the wall is designed to resist all movements (no relative displacement), higher values of the static safety factor, $F$ must be chosen that will lead to less-economic wall designs. In this case, the forces exerted on the wall due to seismic loading will be also very high and thus, the gravity retaining wall may start rotating, putting the integrity of the structure into danger. Based on the above-mentioned, such a wall behavior usually leads to an uneconomic designs and should be avoid. On the other hand, the methodology of Richards and Elms (1979) allows for some deformation in terms of wall sliding which is predefined by the engineer and treated as secure/allowable. As a result of the wall movement, much lower forces will be exerted on the wall and a more economic design will be possible.

2.4 Dynamic Response of Retaining Walls

Nowadays, researchers and geotechnical engineers are able to simulate accurately dynamic problems of soil-structure interaction with numerical analysis using finite element/difference software and advanced constitutive models that are capable of capturing the complex aspects of soil behavior under monotonic and cyclic loading. In addition, centrifuge experiments of scale-models are available to validate computed performance using numerical analyses. Researchers are able to compare the results of the numerical analyses and the experimental data with that of the pseudo-static approach and hence, propose new guidelines and design approaches for a more accurate seismic design of new structures and evaluation of the existing ones.
Callisto and Soccodato (2010) ran twenty-four (24) parametric analysis with FLAC v5.0 (Itasca) using two different seismic inputs and flexible cantilever retaining walls with varying embedded length, $d$, and bending stiffness, $E_I$, in an effort to evaluate the importance of the embedded length of the wall and its bending stiffness on the dynamic response. Figure 2.2 shows the finite difference grid and the basic geometric characteristics considered by these authors. The grid zones have a maximum element size of 0.5m and 1.8m near the walls and the bedrock, respectively. The soil profile consists of a dry, cohesionless, coarse-grained soil with density, $\rho = 2.04\text{Mg}/\text{m}^3$ and friction angle, $\phi = 35^\circ$. The walls are simulated by elastoplastic structural beam elements with interface friction angle, $\delta = 20^\circ$.

**Figure 2.2:** Finite difference grid-basic geometric entities (after Callisto and Soccodato, 2010)

Soil behavior is represented by an elastoplastic constitutive model with a non-associated flow rule coupled with a Mohr-Coulomb failure criterion. The model is able to reproduce hysteresis loops in load cycles that follow Masing (1926) rules with a non-linear shear stress-strain ($\tau$-$\gamma$) relation based on the following equation:

$$\frac{\tau}{G_0} = \frac{G_s(\gamma)}{G_0} \gamma = M_s \gamma$$

(2.7)

where:

$G_s(\gamma)$ is the secant shear modulus

$G_0$ is the elastic small strain modulus

and $M_s(\gamma)$ is the normalized secant shear modulus
Chapter 2: Literature Review

The $G_s(y)/G_0$ degradation curve implemented to the model is given by:

$$M_s = y_0 + \frac{a}{1+\exp\left(\frac{-\log_{10}y-x_0}{b}\right)}$$

(2.8)

with input parameters: $\alpha = 0.9762$, $b = -0.4393$, $x_0 = -1.285$, $y_0 = 0.03154$.

Two seismic inputs were used (Figure 2.3) for the evaluation of the seismic response of the retaining walls. The accelerograms are applied on the base of the model which is simulated as a rigid bedrock. To avoid spurious effects, free-field lateral boundaries were implemented on the sides of the model (Figure 2.2).

Figure 2.3: Time histories and Fourier spectra of the two input seismic motions (after Callisto and Soccodato, 2010)

The characteristics of the parametric analyses are shown in Figure 2.4, including the wall embedded length, $d$, the elastic bending stiffness of the wall, $E_I$, the wall yielding moment, $M_y$ (if a yielding moment is not stated, the wall’s behavior is elastic), the soil friction angle, $\phi$, the multiplier of the initial small-strain stiffness of the soil, $K_0$, and the critical acceleration, $a_c$.

Based on the results of the numerical analyses (cases 06-11 from Figure 2.4), the authors concluded that the increase in the wall stiffness produces smaller relative displacements and larger moment variation. However, at the end of the seismic input, the moment has a single value independent of the wall stiffness. The time-histories of the relative displacement between the top and toe of the wall and the maximum moment for two different values of the wall stiffness, $E_I$ are shown in Figure 2.5.
Chapter 2: Literature Review

<table>
<thead>
<tr>
<th>Number</th>
<th>d (m)</th>
<th>$E$ (kN m²/m)</th>
<th>$M_r$ (kN m/m)</th>
<th>Recording</th>
<th>$\varphi$ (°)</th>
<th>$K_w$</th>
<th>$a_r$ (g)</th>
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<td>--</td>
<td>T</td>
<td>35</td>
<td>1.000</td>
<td>0.398</td>
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</tbody>
</table>

Note: Bold identifies the reference analysis.

**Figure 2.4:** Parametric analysis summary (after Callisto and Soccodato, 2010)

**Figure 2.5:** Time history of a) relative top-toe displacement b) maximum bending moment, for two different values of the wall stiffness, $E_I$ (after Callisto and Soccodato, 2010)
From Figure 2.5, we can observe that the bending moment of both walls follows a similar pattern, with larger values for the stiffer wall but similar results at the end of the seismic excitation. However, there is a significant difference in the accumulation of relative displacements (Figure 2.5a) shown between the wall with the very low stiffness (0.2EI) and the wall with the high stiffness (20EI). The accumulation of relative displacements for a wall with bending stiffness equal to EI is very similar to the results of the high-stiffness wall. In other words, it seems that there is an upper limit associated with effects of the bending stiffness on the relative displacements. In fact, as the bending stiffness, EI is increasing, the wall becomes more and more rigid. Above a certain value of EI, the wall behaves as a rigid body and the accumulation of relative displacements will be constant and independent of the wall stiffness. As a result, we can distinguish the wall’s total movement into two parts, a rigid-body movement and a horizontal deflection. When the wall stiffness is larger than EI, the total wall displacement is due to the rigid body rotation. When the wall stiffness is low, the total movement is primarily horizontal deflection.

Figure 2.6 shows how the relative displacement changes with the bending stiffness of the wall. As the bending stiffness ratio increases, the wall’s relative displacements tend to become similar to the displacements of the rigid wall. For a given bending stiffness ratio, the difference between total displacements and displacements of the rigid wall are equal.

Figure 2.6: Variation of relative displacement with the bending stiffness (after Callisto and Soccodato, 2010)

Finally, the effect of the embedded length, d on the relative displacements of the wall and the bending moment was investigated. Figure 2.7 depicts the change of the postseismic
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relative displacements, \( u_r \), and the bending moment, \( M \), with increasing embedded length, \( d \). In these cases, the elasto-plastic wall tends to experience larger relative displacements than the elastic wall. However, as the embedded length, \( d \), increases that difference becomes smaller and smaller. As far as the bending moment is concerned, the elastic wall experiences larger moments while the elasto-plastic tends to have smaller ones. For a given embedded length, \( d \), the difference of the moments that the two wall types (elastic, elasto-plastic) experience remains approximately constant (and equal to 80kN-m/m) (Figure 2.7).

![Figure 2.7](image)

**Figure 2.7:** a) Postseismic relative displacements and b) bending moment change due to an increasing embedded length for the two seismic excitations (after Callisto and Soccodato, 2010)

Conti et al. (2012) performed centrifuge tests on model retaining walls in order to investigate the phenomena that arise during the seismic loading and dominate the soil–wall interaction, and to propose complementary design procedures to the conventional pseudo-static analysis.

Nine centrifuge tests were performed on cantilever and single-propped retaining walls (Figure 2.8a and 2.8b), using the 5m radius beam-centrifuge at the University of Cambridge. Figure 2.8 and 2.9 show the geometry of the two models and the values of the basic geometric characteristics of these tests. In Figure 2.9, \( D_{ri} \) and \( D_{rf} \) are the initial and final soil relative
density, respectively, CW notation denotes a cantilever wall test and PW represents a single-propped wall test.

Figure 2.8: The geometry of the two models a) cantilevered walls and b) one-propped walls (after Conti et al., 2012)
Figure 2.9: Values of each test’s basic geometric entities (after Conti et al., 2012)

The soil used in the tests was a fine silica sand (Leighton Buzzard, Fraction E Sand 100/170), with specific gravity of solids, $G_s = 2.65$, minimum and maximum void ratio, $e_{\text{min}} = 1.014$ and $e_{\text{max}} = 0.613$, respectively and critical friction angle, $\phi'_{cv} = 32^\circ$. During the tests, different types of instrumentation (piezoelectric accelerometers, strain gauges, linear variable differential transducers, load cells) were used for monitoring the accelerations of the walls, bending moments, strut axial loads and horizontal deflections.

Finally, each model (CW1 to PW4, Figure 2.9) was subjected to a sequence of five (5) seismic base motions with basic properties shown in Figure 2.10.

Figure 2.10: Basic earthquake features (after Conti et al., 2012)

Figures 2.11a, b, d and f summarize the results for the propped-wall tests, while Figure 2.11c, e and g show the results for cantilever wall models. Moreover, it should be noted that all the structural loads (axial loads in props and bending moments) are normalized with their corresponding static value to facilitate comparison between models with different soil properties. In the last two charts of the Figure, the critical acceleration of propped and
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cantilevered walls is computed with the methods of Neelakantan et al. (1992) and Blum (1931), respectively, for different values of soil’s critical friction angle and relative density.

As far as the maximum structural loads (bending moments on the wall, axial loads in props) (Figure 2.11a, b, c) are concerned, it can be seen that increasing the amplitude of the acceleration leads to an increase in the maximum values of the structural loads. The maximum loads on the propped walls seem to be in good agreement with the predictions of pseudo-static analysis. More specifically, for the loose sand, the critical acceleration computed was $a_c = 0.44g$, and Figure 2.11b shows that when this value is attained, the moments exerted on the wall remain constant to a specific value, as the soil strength is completely mobilized and the wall starts deforming. On the other hand, from Figure 2.11c (cantilever wall), we can observe that the moments on the wall are always larger than the predictions from pseudo-static analysis. This is generally attributed to the fact that pseudo-static analysis does not take into account the progressive reduction of friction angle in the soil with the accumulation of shear strains.

Regarding the relative displacements of the walls, Figures 2.11d and 2.11e show that there is no trend that correlates the amplitude of the acceleration with the wall displacement. Furthermore, relative displacements occur even when the applied acceleration is smaller than the critical acceleration of the system. Based on those results, Conti et al. (2012) tried to calculate the critical acceleration in order the experimental data and the pseudo-static analysis to produce similar results in terms of accumulated relative displacements. Their conclusion was that there is no such value of critical acceleration. The only way to do this would be to consider a different value of critical acceleration for each seismic input (i.e., they conclude that the critical acceleration is not a constant but varies with the different ground motions).

One possible explanation for the phenomenon of varying critical acceleration deals with the redistribution of earth pressures applied on the wall and the progressive mobilization of the passive resistance in front of the wall. As Conti et al. (2012) note, when the seismic motion takes place, the applied acceleration (which at first is much smaller than the critical value, $a_c$) leads to an increase in the active earth pressures exerted on the retaining wall. From the wall equilibrium, larger active earth pressures mobilize a larger portion of the passive resistance. The only way for that to happen is by having the wall tilting towards excavation. As the induced acceleration varies, the progressive tilting of the wall would take place only in the time intervals when the acceleration exerted on the wall has a larger value from the previous maximum acceleration that the wall experienced. At some point, however, the maximum acceleration equals the critical acceleration and full passive soil resistance becomes mobilized and the wall will continue tilting without further changes in acceleration.
Figure 2.11: Summary of the results of centrifuge model tests in LBS sand, a) maximum axial load in props, b), c) maximum bending moments, d), e) horizontal wall displacements, f), g) critical accelaration for different values of soil's relative density and critical friction angle (after Conti et al., 2012)
Miriano et al. (2015) performed a series of numerical analyses of single-propped retaining walls, using two different constitutive models in order to evaluate how the constitutive model assumptions affect the predictions of seismic soil–structure interaction.

Two sets of numerical simulations were performed with the finite element code, ABAQUS (Standard v6.4). The two soil models implemented to each set were the widely-used elastic-perfectly plastic Mohr-Coulomb constitutive model and the advanced Papadimitriou-Bouckovalas model (PB; Papadimitriou et al., 2002) for sands. The latter combines a kinematic elastoplastic framework with a Ramberg-Osgood formulation in order to simulate hysteretic behavior of sands at different levels of strain. In order to compare the results produced by the two different models, the Mohr–Coulomb model was calibrated based to results of the static/excavation stage generated using the PB model. Moreover, Rayleigh damping has been adopted with the Mohr–Coulomb model so as to represent as accurately as possible the material degradation curves and damping ratio. The soil parameters used for the numerical analyses correspond to Nevada sand, with the input values shown in Figure 2.12.

<table>
<thead>
<tr>
<th>$\Gamma$</th>
<th>$\lambda$</th>
<th>$\alpha_c^c$</th>
<th>$\alpha_e^c$</th>
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<th>$B$</th>
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<th>$k_c^d$</th>
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Table 2: MC model constants for Nevada Sand

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<tr>
<th>Variable</th>
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<th>$\nu$ (--)</th>
<th>$c$ (kPa)</th>
<th>$\phi$ (deg)</th>
<th>$\psi$ (deg)</th>
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<td>0.0</td>
<td>32</td>
<td>15</td>
<td></td>
<td></td>
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</table>

Figure 2.12: PB and MC model input parameters for Nevada sand (after Miriano et al., 2015)

The finite element grid and the basic geometric assumptions of the simulation are shown in Figure 2.13. The simulation of the problem was divided into six (6) separate stages: i) calculation of the geostatic soil stresses, ii) activation of structural elements (two walls), iii–v) simulation of the excavation and installation of the prop, vi) application of the seismic excitation at the base of the model.
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For the dynamic analysis, two seismic inputs were applied at the base of the model which represented a rigid bedrock. The acceleration time histories of the two motions are shown in Figure 2.14. In order to avoid spurious effects, the vertical boundaries were placed one hundred (100) meters away from the center of the excavation (Figure 2.12). In addition, periodic boundary conditions were implemented, to ensure that the vertical boundaries move together during the seismic excitations.

Figure 2.13: The finite element grid – basic geometry and dimensions sand (after Miriano et al., 2015)

Figure 2.14: Acceleration time histories for a) Colfiorito and b) Assisi earthquakes (after Miriano et al., 2015)

Figures 2.15, 2.16 and 2.17 show the results in terms of horizontal wall deflection and bending moments for both constitutive models and seismic motions. It can be observed from these
Figure 2.15: Horizontal deflection of the walls for both constitutive models and seismic excitations earthquakes (after Miriano et al., 2015)

On the other hand, significant differences occur between predictions of the two models during the dynamic stage. The PB model predicts different behavior for the two walls. The left wall does not experience a net movement but rotates at mid-height while the right wall inwards towards the excavation. In contrast the MC model produces more symmetric horizontal deflections for the two walls.

As far as the bending moments are concerned (Figures 2.16, 2.17), the MC model predicts larger values of $M_{\text{max}}$ than the PB model for both seismic excitations. This is due to the fact that when using the MC model, soil responds elastically to most of the seismic cycles, without developing hysteretic damping that will lead to energy dissipation. The elastic soil behavior results in larger earth pressures exerted on the wall and hence, larger structural loads. In contrast, the small yield surface implemented in the PB model, allows the development of irreversible strain even at small shear strain levels.

It was therefore concluded that for an accurate representation of the dynamic response of retaining walls, bounding-surface models should be used since simple elastic – perfectly plastic models fail to represent accurately the soil behavior under cyclic loading.
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Figure 2.16: a) Bending moments and b) the max/min moment envelopes of the left wall for both constitutive models and the Colfiorito earthquake (after Miriano et al., 2015)

Figure 2.17: a) Bending moments and b) the max/min moment envelopes of the left wall for both constitutive models and the Assisi earthquake (after Miriano et al., 2015)
2.5 Conclusions

This chapter has reviewed the current design methodology of retaining walls along with the widely-used pseudo-static approach. Recent research studies (experiments and numerical simulations) highlighted limitations of these simplified methods, but have generated results for only a small number of seismic ground motions. Further work is needed to enrich these results and enable refinements in current design practice.
3 Numerical Analyses

3.1 Introduction

This chapter describes the methodology used in the current study of the dynamic response of single-propped retaining wall systems. The main components of the numerical analysis are as follows:

a. The finite difference code FLAC (Fast Lagrangian Analysis of Continua v7.0, Itasca, 2011) was used. The basic point that differentiates FLAC from the majority of the other available computational mechanics programs (most commercial codes used for geomechanics are based on displacement based finite element method, e.g. Plaxis, etc.) is that FLAC uses an explicit solver scheme in order to solve the difference equations. The basic characteristic of this explicit scheme are that equilibrium is approximated at each calculation step through of matrices. As a result, the code becomes ideal for the simulation of highly non-linear problems, such as those examined in this thesis.

b. The model used for the representation of the soil behavior under cyclic loading is the elastoplastic PB soil model for sands proposed by Papadimitriou et al. (2002). The model combines a bounding surface framework for large cyclic strains, with a Ramberg-Osgood type hysteretic formulation for energy dissipation in small levels of shear strain.

c. The input parameters of the PB (2002) model were chosen to match those used in the analyses by Miriano et al. (2015) and correspond to the properties of Nevada sand.

The validation of the proposed algorithm was assessed by comparing results of wall displacements and bending moments at both the (quasi-static) excavation and dynamic seismic loading stages with the numerical results of Tamagnini et al. (2015) for the Assisi earthquake.
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3.2 PB soil model (Papadimitriou et al., 2002)

The PB model was adopted for the representation of the sand behavior under cycling loading. The main feature of this model is that it combines a kinematic-hardening elastoplastic framework to reproduce soil behavior at large cyclic strains with a Ramberg-Osgood type hysteretic formulation for smaller strains. The reversible part of the model response is characterized by hypoelastic constitutive equations with hysteresis:

\[
\dot{\sigma}'_{ij} = K_t \dot{\varepsilon}_{kk} \delta_{ij} + 2G_t \varepsilon_{ij},
\]

where:
- \( \dot{\sigma}'_{ij} \) is the effective stress tensor
- \( \dot{\varepsilon}_{kk} \) is the volumetric strain rate
- \( \varepsilon_{ij} \) is the small strain rate
- and \( K_t \) and \( G_t \) are the state-dependent tangent bulk and shear moduli, respectively

The tangent bulk and shear moduli are given by the following equations:

\[
K_t = \frac{2(1+\nu)}{3(1-2\nu)} G_t,
\]

\[
G_t = \frac{G_{max}}{T(\chi_{r})'},
\]

where:
- \( G_{max} = \frac{B p'}{0.3 + 0.7 e^2} \sqrt{\frac{p'}{p_a}} \)

where:
- \( e \) is the void ratio
- \( p_a \) is the atmospheric pressure
- \( p' \) is the mean effective stress
- and \( B \) is a material constant

The scaling factor \( T \geq 1 \) depends on the scalar quantity:

\[
\chi_{r} = \frac{1}{\sqrt{2}} \|r - r^{ref}\|,
\]

where
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\[ \|X\| := \sqrt{X_{ij}X_{ij}}, \] the Euclidean norm of the second-order tensor \(X_{ij}\)

which measures the “distance” of the current stress ratio tensor \(r\) from a known reference state \(r^{\text{ref}}\). In particular, two possible cases are considered for the reference state: a) virgin loading paths, for which \(r^{\text{ref}}\) is the geostatic stress ratio tensor; b) unloading paths, for which \(r^{\text{ref}} = r^{\text{SR}}\), this last quantity being the stress ratio tensor at the shear reversal (SR) point. The function \(T\) is given by the expression:

\[
T = \begin{cases} 
1 + k \left( \frac{1}{\alpha_1} - 1 \right) \left( \frac{X_r}{\eta_1} \right)^{k-1} & \text{virgin loading} \\
1 + k \left( \frac{1}{\alpha_1} - 1 \right) \left( \frac{X_r}{\eta_1} \right)^{k-1} & \text{shear reversal}
\end{cases}
\]

subject to:

\[ T \leq 1 + k \left( \frac{1}{\alpha_1} - 1 \right). \] (3.7)

In Equation (3.6), \(\alpha_1\) and \(\eta_1\) are positive scalars, while \(k \geq 1\) is a constant equal to 2.0. The variable \(\eta_1\) is related to a characteristic amplitude of shear strain \(\gamma_1\) (a model constant) according to:

\[ \eta_1 = \alpha_1 \left( \frac{G_{\text{max}}}{p_{\text{SR}}} \right) \gamma_1, \] (3.8)

where:

\[ G_{\text{max}}^{\text{SR}} \] is the max shear modulus

and \(p'_{\text{SR}}\) is the mean effective stress \(p'\) at the last shear-reversal (SR) state

For the first shearing path in particular, \(G_{\text{max}}^{\text{SR}} = G_{\text{max}}^0 \) and \(p'_{\text{SR}} = p'_0\), (i.e., variable \(\gamma_1\) is related to the values of \(G_{\text{max}}\) and \(p'\) at consolidation). Parameter \(\gamma_1\) may be interpreted as a threshold strain beyond which any further degradation in the overall shear stiffness is due to the development of plastic shear strain (\(G_t = G_{\text{min}}\) for \(\gamma \geq \gamma_1\) as shown in Figure 3.1).

As far as the inelastic response is concerned, the model is characterized by an open conical yield surface (YS), which can rotate around the cone apex at the origin of the stress space as originally proposed by Dafalias and Manzari (1997), and by three additional open wedge-type surfaces with apex at the origin of stress space: the critical state surface (CSS), the bounding surface (BS) and the dilatancy surface (DS), as shown in Figure 3.2.
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The yield surface (YS) has the following analytical expression:

\[ f(\sigma, \alpha) = \frac{3}{2} (r - a) : (r - a) - m^2, \]  

(3.9)

where

- \( \alpha \) is the a tensor describing the orientation of the yield surface in stress space \((\sigma')\)
- \( r \) is the tensor of deviatoric stress normalized by mean effective stress
- and \( m \) is the tangent of the cone opening angle (typically very small)

![Shear stress-strain relation](image)

**Figure 3.1:** Shear stress-strain \((\tau - \gamma)\) relation according to the proposed Ramberg – Osgood type formulation – effect of \(\alpha_1\) (after Papadimitriou et al., 2002)

The model is capable of reproducing critical-state (CS) conditions, (i.e. continuing deviatoric deformations at constant effective stress). The CS locus in the void ratio versus \(\log\) mean effective stress plane \((e - \log p')\) is given by the following:

\[ e_c = \Gamma - \lambda \log_e \left( \frac{p}{p_a} \right), \]  

(3.10)

where

- \( e_c \) = \( \Gamma \) at \( p' = p_a \)
- and \( \lambda \) is the slope of the critical state line
The CS locus in stress space is given by:

\[ F^c(\sigma, \alpha) = \frac{3}{2} \alpha^c \cdot \alpha^c - (\alpha^c)^2 = 0.0 , \] (3.11)

where \( \alpha^c \) and \( \alpha^c \) are given by the following expressions:

\[ \alpha^c = \tilde{g}(\tilde{\theta}, c_M) M_c^c - m , \] (3.12)

\[ \alpha^c = \sqrt{\frac{3}{2}} \alpha^c \mathbf{n}, \quad \mathbf{n} := \frac{r-a}{\|r-a\|} , \] (3.13)
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In Equation (3.12), $M^c_c$ is a material constant giving the slope of the CSL in the $q$-$p$ space, $c_M = M^c_M/M^c_c$ is the ratio of critical-state slopes in axisymmetric extension and compression, respectively, and $\bar{g}$ is a given function of $c_M$ and lode angle $\bar{\theta}$ of the tensor $\bar{r} = r - a$.

The rotation of the cone axis in stress space is limited by the bounding surface (BS) which has the expression:

$$F^b(\sigma, \alpha, \psi) = \frac{3}{2} a^b \cdot a^b - (a^b)^2 = 0.0,$$  \hspace{1cm} (3.14)

where $a^b$ and $a^b$ are given by the following expressions:

$$a^b = g(\bar{\theta}) M^b_c - m,$$  \hspace{1cm} (3.15)

$$a^b = \sqrt{\frac{2}{3}} a^b n M^b_c = M^c_c + k^b_c < -\psi >, \hspace{0.5cm} \psi = e - e_c(p),$$  \hspace{1cm} (3.16)

In Equation (3.16), $k^b_c$ is a material constant controlling the change in peak stress ratio in axisymmetric compression with changes in soil density, quantified by the state parameter $\psi$ as defined by Been and Jefferies (1985). If $\psi < 0$ (soils denser than critical), the peak failure conditions can occur at stress ratios higher than the corresponding critical-state value, which is reached asymptotically upon further deviatoric deformation (when $e \to e_c$ and $\psi \to 0$).

The plastic flow direction is provided directly in terms of a prescribed dilatancy function, $D$:

$$\hat{\epsilon}^p = \hat{\gamma} Q = \hat{\gamma} \left( n + \frac{D}{3} I \right), \hspace{0.5cm} D = A_0 \left( a^d - a \right) \cdot n,$$  \hspace{1cm} (3.17)

where

$$a^d = \frac{2}{3} a^d n,$$  \hspace{1cm} (3.18)

$$a^d = \bar{g}(\bar{\theta}, c_M) M^d_c - m,$$  \hspace{1cm} (3.19)

$$M^d_c = M^c_c + k^d_c \psi,$$  \hspace{1cm} (3.20)

In Equation (3.17), $\hat{\gamma} \geq 0$ is the plastic multiplier, $A_0 > 0$ and $k^d_c$ are material constants and $M^d_c$ is the slope of the phase transformation line in axisymmetric compression, function of soil density via $k^d_c$.  

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The set of evolution equations for the state variables is completed by the hardening law for the tensorial internal variable $\alpha$, given by:

$$\dot{\alpha} = \dot{\gamma} h_b h_f (\alpha^b - \alpha),$$

(3.21)

According to Equation (3.21), the rate of rotation of the YS is related to its distance to the BS. Limiting conditions (i.e. $\dot{\alpha} = 0$) occur when the YS axis is in the same direction as the BS. The functions $h_b$ and $h_f$ are hardening functions controlling the plastic modulus of the material. In particular, $h_f$ is related to an additional tensorial internal variable (the fabric tensor, $F$) that takes into account the effects of fabric evolution with accumulated plastic strains.

The plastic modulus $K_p$ depends on the distance from the bounding surface $d^b$ as:

$$K_p = p h_b h_f d^b,$$

(3.22)

All parameters of Equation (3.22) are nonnegative, except for $d^b$ that controls the sign of plastic modulus. The scalar parameter $h_f$ is an empirical macroscopic index which describes the effect of fabric evolution during shearing and parameter $h_b$ is defined as:

$$h_b = h_0 \frac{|d^b|}{<d^b_{ref} |d^b|>},$$

(3.23)

where $h_0$ is a user-defined positive constant and $d^b_{ref}$ is a reference distance corresponding to the $\theta$-related “diameter” of the bounding surface.

In the PB model, fabric evolution affects the plastic strain rate $\dot{\varepsilon}^p$ through an empirical factor $h_f$ that scales the plastic modulus $K_p$ (Equation (3.22)) and uses a macroscopic second-order fabric tensor $F$, defined as:

$$h_f = \frac{1+<F\cdot I>^2}{1+<F\cdot I>},$$

(3.24)

If the fabric tensor $F$ is decomposed in a spheric part $f_{s}/3$ and in a deviatoric part $f$ as follows:

$$F = f + \left(\frac{f_{s}}{3}\right) I,$$

(3.25)

the empirical factor $h_f$ can be rewritten as:

$$h_f = \frac{1+<f_{p}>^2}{1+<F\cdot n>},$$

(3.26)
Chapter 3: Numerical Analyses

There is a correlation of fabric evolution to dilative or contractive behavior, and consequently to the plastic volumetric strain rate $\dot{\varepsilon}_v^p$:

\[
\dot{f}_p = H \dot{\varepsilon}_v^p ,
\]

\[
\dot{f} = -H < -\dot{\varepsilon}_v^p > (C n + f) ,
\]

where $H$ and $C$ are model parameters. $C$ can be obtained from the expression:

\[
C = \max(f_p) ^2 ,
\]

Finally, the parameter $H$ is related to the initial conditions by:

\[
H = H_0 \left( \frac{\sigma_{10}}{p_a} \right)^{-\zeta} < -\psi_0 > ,
\]

where $H_0$ and $\zeta$ are positive constants, and $\psi_0$ and $\sigma_{10}$ are, respectively, the value of the state parameter at consolidation and the value of the major principal effective stress at consolidation.
3.3 Numerical Simulation of One-Propped Retaining Walls

Figures 3.3 and 3.4 show the basic geometric entities of the finite differences grid and a part of the grid as constructed in FLAC for the simulation of the problem.

![Figure 3.3: Basic geometric entities of the problem](image1)

![Figure 3.4: Part of the finite differences grid constructed in FLAC](image2)

The final depth of the excavation is equal to 9.5m, thus the embedded length, d of the walls at the final stage is 5m. The finite difference mesh comprises a uniform mesh of 20200 square elements (with dimensions 0.50m x 0.50m). The dimensions of the elements were chosen carefully, so to avoid filtering of high frequencies when the seismic excitations are applied at the bottom of the model. For each of the seismic input motions that were applied on the model, the element size, $\Delta x$ was calculated according to FLAC's recommendation for dynamic analysis and the suggested expression:
Chapter 3: Numerical Analyses

\[ \Delta x \leq \frac{\lambda}{10}, \quad (3.31) \]

where

\( \lambda \) is the wavelength of the seismic motion.

The minimum value of the elements size was then selected and used for all the seismic motions.

The retaining walls and row of strut were represented using beam elements. In particular, each wall was discretized in 28 segments while the strut was comprised by a single continuous beam element (Figure 3.5).

In order to capture accurately the relative movement that occurs between the walls and the surrounding soil, interface elements were implemented on both sides of each wall (Figure 3.6). Interface elements are placed on both sides of walls (Figure 3.6). The interface elements connect the adjacent soil and structure through shear and axial elasto-plastic springs (with specified stiffness and strength parameters). During the wall movement, as long as the force exerted on those springs remains smaller than the yield value, there is no slip between wall and soil. When the force becomes larger than the yield value of the springs, slip and/or separation occur.

---

**Figure 3.5:** Walls and strut discretization
Figure 3.6: The two interfaces implemented on the left wall a) the blue crosses are points of the interface on the left side of the wall b) the red crosses are points of the interface on the right side of the wall

FLAC provides many different options regarding the behavior of the interface elements when their yield value is exceeded. The user can choose between slip, separation, both or neither options. The user should be able to identify those characteristics that best describe the behavior of soil-structure interaction and activate the appropriate options that simulate the problem most accurately. In the current analyses, wall-soil interaction allowed both slip and separation to occur.

The interface stiffness was evaluated by the following expression proposed by the FLAC manual:

\[ K_n = K_s = \frac{[K + \frac{4}{3}G]}{\Delta x_{min}}, \]  

(3.32)

where

\( K_n \) is the normal stiffness of the elasto-plastic spring
Chapter 3: Numerical Analyses

\[ K_s \text{ is the shear stiffness of the elasto-plastic spring} \]

\[ K = \frac{3E(1-v)}{(1+v)(1-2v)} \]

\[ G = \frac{E(1-v)}{(1+v)(1-2v)} \]

and \( \Delta x_{\text{min}} \) is the smallest width of the adjoining zone in the normal direction (0.5m)

The simulation of the problem was performed in two distinct phases, each one having its own unique characteristics. The first phase includes the geostatic conditions calculation and staged construction of the excavation; while the second phase involves the implementation of the seismic inputs and the monitoring of the dynamic response of the retaining walls and the strut.

3.3.1 Excavation Phase

This phase involves the static loading of the walls due to staged excavation. The boundary conditions at the base of the model are simple roller connections that allow only horizontal movements and the vertical boundaries are rollers as well, allowing soil deformations in the vertical direction but no horizontal movement.

The excavation phase includes the following stages:

1. geostatic conditions
2. wall installation
3. excavation to 2.5m below ground level
4. installation of the strut
5 - 8. subsequent 1.5m excavation stages
9. last 1.0m excavation

In **Stage 1**, the implementation of the advanced PB constitutive model for the sand takes place. The characteristics and the main features of this model were discussed in paragraph 3.3. As it was mentioned above, the input parameters of the PB model were chosen to match those selected by Tamagnini et al. (2015) which were used in their analyses and correspond to the properties of Nevada sand proposed by Papadimitriou et al. (2002). After the PB model is assigned to each soil element, the geostatic conditions are calculated to generate the appropriate in situ stress and state variables (\( \sigma'_{vo}, \sigma'_{vp} \) and \( e \)) from which the initial elastic stiffness values are obtained.

In **Stage 2**, the two retaining walls are installed. Specifically, we activate the appropriate beam and interface elements and assign properties to them. The properties of the retaining walls are shown in Table 3.1. The beam elements are continuous in the out-of-plane direction and...
the dead weight is applied instantaneously (fully drained conditions). Deformations are not considered for this phase.

Table 3.1: Diaphragm Wall Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit Weight, $\gamma$ (kg/m$^3$)</th>
<th>$E$ (kPa)</th>
<th>Wall Thickness, $t$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>25</td>
<td>2.0e7</td>
<td>1.0</td>
</tr>
</tbody>
</table>

In Stages 3 and 4, the first excavation lift and the installation of the strut are performed. The strut has a rectangular cross-section with the following parameters (Table 3.2):

Table 3.2: Strut Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit Weight, $\gamma$ (kg/m$^3$)</th>
<th>$E$ (kPa)</th>
<th>Cross-section Area, $A$ (m$^2$/m)</th>
<th>Out-of-plane Spacing (m)</th>
<th>$I_x$ (m$^4$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>25</td>
<td>2.0e7</td>
<td>0.25</td>
<td>2.0</td>
<td>4.03e-4</td>
</tr>
</tbody>
</table>

Finally, in Stages 5 – 9, the remaining excavation lifts take place until the required excavation depth is reached.

3.3.2 Dynamic Phase

After the static phase has been completed, the dynamic response of the model is captured by applying a seismic motion at its base. All the ground motions used in this research were obtained from the PEER database and their acceleration time history, $a(t)$ is defined in time steps, $\Delta t=0.01s$. The base of the model is assumed to be a rigid bedrock and is simulated as such. In other words, the roller boundary conditions that were implemented in the previous phase remain the same at this phase, too. On the other hand, the vertical boundary conditions change from rollers to periodic constraints. Periodic boundary conditions connect the nodes on the left side and right side vertical boundaries (master and slave nodes, respectively), hence the degrees of freedom of the master and the slave nodes are the same.

The use of periodic boundary conditions comes with a significant advantage and a disadvantage that each user has to assess based on the specific problem of interest. As far as the advantage is concerned, it is a very straightforward and easy procedure implementing this type of constraint in the model compared to other more sophisticated conditions (such as free-field boundaries). On the other hand, in order to avoid reflections of shear waves on the vertical boundaries, they must be placed far from the area of interest, leading to a significant increase in the computation time for the analysis. Finally, it is also obvious that for periodic boundary conditions to be applied, the model must be symmetrical.
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The last step before the dynamic analysis is performed, is to take into account the fact that a very small amount of hysteretic damping (approximately 2%) develops in the soil even for negligible shear strains (small strain damping ratio). There are various procedures described in the literature on how to include this damping in the dynamic analysis. The most widely used procedure assumes Rayleigh damping (Rayleigh et al., 1945). However, it should be noted that the implementation of Rayleigh damping in the analysis leads to a considerable decrease (more than two times) in the timestep that the code uses for each iteration, thus the required computation time for the numerical analysis increases drastically (Hashash et al., 2010). FLAC provides an alternative to Rayleigh damping called “local damping” that can be directly implemented into the model without reducing the timestep of the analysis. According to FLAC’s recommendations for dynamic analysis, the appropriate local damping, \( a_L \), that corresponds to a damping ratio of 2% is given by the expression:

\[
a_L = \pi D,
\]

where

\( D \) is the required damping ratio (in this case 2%)

3.4 Validation of the Algorithm

The validation of the proposed algorithm was assessed by comparing its results in terms of relative wall displacements and bending moments at both the excavation and the dynamic stages with the numerical results presented by Miriano et al. (2015) for the Assisi earthquake. The Assisi earthquake’s acceleration time history and Fourier spectrum are shown in Figure 3.7.

![Assisi earthquake's acceleration time history (left) and Fourier spectrum (right)](image)

*Figure 3.7: Assisi earthquake’s acceleration time history (left) and Fourier spectrum (right)*
Chapter 3: Numerical Analyses

Figures 3.8 and 3.9 depict the results generated by the current FLAC analyses 1. at the end of excavation; and 2. at the end of seismic record with corresponding results of Miriano et al. (2015) in terms of horizontal wall deflections and bending moments. In Figure 3.9, there is no curve for the post-seismic bending moment exerted on the right retaining wall for Miriano et al. (2015), since those values were not reported on the related paper.

It can be seen from those results that there is excellent agreement between the current FD analyses and the published simulations for both response parameters (displacements and moments) at each stage (static, dynamic) for the two walls (left, right). The small differences are likely related to differences in numerical approximations (finite differences vs finite elements).

Figure 3.8: Left and right wall displacements comparison
Chapter 3: Numerical Analyses

Figure 3.9: Left and right wall moments comparison
Interpretation of Results

4.1 Introduction

This Chapter presents results of the parametric analyses and considers correlations between the basic characteristics of the ground motions and the computed wall responses. For each seismic excitation, the bending moments and earth pressures exerted on the two walls, their horizontal deflections, ground surface settlements (behind the retaining structures) and the axial strut loads were recorded. At first, a classification of the walls responses based on their horizontal movement is performed. Afterwards, in order to explore how the wall’s dynamic behavior depends on basic parameters of the applied seismic motions, correlation charts are presented between the different entities. The results of the parametric analyses for each seismic input can be found in Appendix B.

4.2 Basic Seismic Input Characteristics

A suite of thirty-two different seismic excitations were selected from the PEER database. Table 4.1 summarizes the records name, peak ground acceleration (PGA), $a_{\text{max}}$, peak ground velocity (PGV), $v_{\text{max}}$ (single-integration of acceleration time history), peak ground displacement, $d_{\text{max}}$ (double-integration of acceleration time history) and Arias intensity. The Arias intensity of a seismic motion is defined as:

$$I_a = \frac{\pi}{2g} \int_0^\infty [a(t)]^2 \, dt,$$  \hspace{1cm} (4.1)

where

$g$ is the acceleration of gravity (9.81 m/s$^2$)

and $a(t)$ is the recorded acceleration of the motion at time $t$.

Seismic motions denoted “nga” are natural records while the remaining four (4) “sim” records are synthetic earthquakes (Table 4.1). The acceleration time history and the Fourier spectrum of each motion can be found in Appendix A.

Figure 4.1 shows the peak ground acceleration, $a_{\text{max}}$, and the Arias Intensity corresponding to the predominant period, $T$ for each ground motion. As it can be observed from this Figure,
Chapter 4: Interpretation of Results

de the ground motions selected cover a wide range of peak ground acceleration and Arias intensity values.

Table 4.1: Seismic Motions Characteristics

<table>
<thead>
<tr>
<th>#</th>
<th>Name</th>
<th>$a_{max}$ (g)</th>
<th>$v_{max}$ (cm/s)</th>
<th>$d_{max}$ (cm)</th>
<th>Arias Intensity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>nga0779</td>
<td>0.966</td>
<td>217.080</td>
<td>263.250</td>
<td>16.730</td>
</tr>
<tr>
<td>2</td>
<td>nga0033</td>
<td>0.357</td>
<td>21.470</td>
<td>3.810</td>
<td>0.450</td>
</tr>
<tr>
<td>3</td>
<td>nga0145</td>
<td>0.157</td>
<td>21.610</td>
<td>5.220</td>
<td>0.370</td>
</tr>
<tr>
<td>4</td>
<td>nga0150</td>
<td>0.433</td>
<td>98.460</td>
<td>30.680</td>
<td>1.540</td>
</tr>
<tr>
<td>5</td>
<td>nga0448</td>
<td>0.423</td>
<td>50.550</td>
<td>18.340</td>
<td>1.360</td>
</tr>
<tr>
<td>6</td>
<td>nga0451</td>
<td>0.710</td>
<td>103.250</td>
<td>48.050</td>
<td>5.770</td>
</tr>
<tr>
<td>7</td>
<td>nga0632</td>
<td>0.156</td>
<td>3.970</td>
<td>0.210</td>
<td>0.083</td>
</tr>
<tr>
<td>8</td>
<td>nga0648</td>
<td>0.134</td>
<td>5.750</td>
<td>0.290</td>
<td>0.064</td>
</tr>
<tr>
<td>9</td>
<td>nga0649</td>
<td>0.180</td>
<td>4.900</td>
<td>0.550</td>
<td>0.100</td>
</tr>
<tr>
<td>10</td>
<td>nga0669</td>
<td>0.180</td>
<td>20.330</td>
<td>3.820</td>
<td>0.510</td>
</tr>
<tr>
<td>11</td>
<td>nga0676</td>
<td>0.184</td>
<td>20.630</td>
<td>4.510</td>
<td>0.410</td>
</tr>
<tr>
<td>12</td>
<td>nga0684</td>
<td>0.025</td>
<td>1.250</td>
<td>0.110</td>
<td>0.011</td>
</tr>
<tr>
<td>13</td>
<td>nga0791</td>
<td>0.072</td>
<td>20.900</td>
<td>25.790</td>
<td>0.169</td>
</tr>
<tr>
<td>14</td>
<td>nga0802</td>
<td>0.512</td>
<td>82.300</td>
<td>64.980</td>
<td>2.900</td>
</tr>
<tr>
<td>15</td>
<td>nga0954</td>
<td>0.163</td>
<td>8.020</td>
<td>0.830</td>
<td>0.428</td>
</tr>
<tr>
<td>16</td>
<td>nga0969</td>
<td>0.078</td>
<td>3.390</td>
<td>1.820</td>
<td>0.059</td>
</tr>
<tr>
<td>17</td>
<td>nga0982</td>
<td>0.570</td>
<td>152.080</td>
<td>169.660</td>
<td>6.470</td>
</tr>
<tr>
<td>18</td>
<td>nga0983</td>
<td>0.570</td>
<td>152.080</td>
<td>169.660</td>
<td>6.470</td>
</tr>
<tr>
<td>19</td>
<td>nga1012</td>
<td>0.261</td>
<td>27.390</td>
<td>4.810</td>
<td>0.980</td>
</tr>
<tr>
<td>20</td>
<td>nga1013</td>
<td>0.511</td>
<td>127.360</td>
<td>84.990</td>
<td>3.540</td>
</tr>
<tr>
<td>21</td>
<td>nga1014</td>
<td>0.036</td>
<td>2.480</td>
<td>0.410</td>
<td>0.028</td>
</tr>
<tr>
<td>22</td>
<td>nga1023</td>
<td>0.165</td>
<td>4.140</td>
<td>1.130</td>
<td>0.097</td>
</tr>
<tr>
<td>23</td>
<td>nga1031</td>
<td>0.146</td>
<td>7.420</td>
<td>0.580</td>
<td>0.140</td>
</tr>
<tr>
<td>24</td>
<td>nga1055</td>
<td>0.240</td>
<td>12.270</td>
<td>1.070</td>
<td>0.730</td>
</tr>
<tr>
<td>25</td>
<td>nga1057</td>
<td>0.135</td>
<td>18.570</td>
<td>4.510</td>
<td>0.310</td>
</tr>
<tr>
<td>26</td>
<td>nga1086</td>
<td>0.604</td>
<td>39.050</td>
<td>4.200</td>
<td>1.300</td>
</tr>
<tr>
<td>27</td>
<td>nga1642</td>
<td>0.301</td>
<td>7.430</td>
<td>0.500</td>
<td>0.210</td>
</tr>
<tr>
<td>28</td>
<td>nga2399</td>
<td>0.039</td>
<td>5.680</td>
<td>1.750</td>
<td>0.031</td>
</tr>
<tr>
<td>29</td>
<td>sim0003</td>
<td>0.137</td>
<td>22.560</td>
<td>9.510</td>
<td>0.460</td>
</tr>
<tr>
<td>30</td>
<td>sim0005</td>
<td>0.090</td>
<td>15.250</td>
<td>6.810</td>
<td>0.130</td>
</tr>
<tr>
<td>31</td>
<td>sim0006</td>
<td>0.107</td>
<td>19.920</td>
<td>6.960</td>
<td>0.150</td>
</tr>
<tr>
<td>32</td>
<td>sim0007</td>
<td>0.024</td>
<td>3.220</td>
<td>2.700</td>
<td>0.007</td>
</tr>
</tbody>
</table>
Chapter 4: Interpretation of Results

Figure 4.1: a) Peak ground acceleration, $a_{\text{max}}$ and b) Arias intensity corresponding to the predominant period of each seismic motion

4.3 Classification of the Wall Response according to its Horizontal Movement

From the complete results presented in Appendix B, we can categorize the responses based on three distinct patterns of wall deflection. The reason for selecting the final seismically-induced horizontal deflection as the key is primarily because the horizontal movement of the walls during the seismic motion determines the magnitude of the earth pressures that develop and hence, the bending moments developed on the walls.

Figures 4.2-4.4 illustrate the key features of retaining system performance for each of the three classes of seismically-induced wall deformations.

4.3.1 Category 1

Figure 4.2a to d illustrate the basic characteristics of Category 1 earth retaining system response. The seismically-induced wall deflections (Figure 4.2c) for the selected earthquake (nga0983) show that both diaphragm walls rotate around a point that is close to the strut connection for each wall. Large horizontal wall deflections are observed at the toe of the two walls. In this example both walls move to the center of the excavation ($\delta_h=140\text{mm}$ on both sides) showing a large closure of the excavation.

Figures 4.2a, b summarize the envelopes of maximum and minimum bending moments and earth pressure diagrams that during the earthquake event. The results show maximum moments in the range $M_{\text{max}} = 1.5-1.6\text{MNm/m}$ occurring at approximately min-height (8m below ground surface). These values are comparable to the yield moment of a reinforced concrete diaphragm wall (in the range $M_y = 1.5-2.0\text{MNm/m}$ for reinforced concrete diaphragm walls).
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Figure 4.2: Earth retaining system response analysis results, Category 1 a) min/max bending moment envelopes, b) min/max earth pressure envelopes, c) seismically-induced horizontal wall deflections and d) seismically-induced ground surface settlement
Figure 4.3: Earth retaining system response analysis results, Category 2 a) min/max bending moment envelopes, b) min/max earth pressure envelopes, c) seismically-induced horizontal wall deflections and d) seismically-induced ground surface settlement
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Figure 4.4: Earth retaining system response analysis results, Category 3 a) min/max bending moment envelopes, b) min/max earth pressure envelopes, c) seismically-induced horizontal wall deflections and d) seismically-induced ground surface settlement
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In general, severe ground motions cause densification of the soil that produce a net decrease in void ratio, \( e \) and hence, in the relative soil density, \( D_r \). The large free-field settlements (far from the excavation) may be explained based on this phenomenon. Specifically, the largest ground surface settlements (320mm, Figure 4.2d) occur approximately 10m behind the wall and are comparable to the free-field (far-field) settlement. The walls also undergo 280mm settlement (exactly behind the wall) which is approximately 80-85% of the far-field behavior.

A possible explanation for the computed ground settlements may relate to the performance of the retaining system where both walls rotate around a point close to the strut connection. Near the strut connection the wall deflects little if any, while the part of the wall above the rotation point deflects towards the retained side. As a result, the wall exerts a net pressure against the ground (passive earth conditions) producing lower ground settlements.

4.3.2 Category 2

Figure 4.3 summarizes typical results for Category 2 wall movements. In this case, there is a very small horizontal wall movement with maximum net deflections, \( \delta_h = 10-20\text{mm} \) at mid-height. There is almost no seismic-induced movement at the top and toe of each wall. All of the analyses included in Category 2 show maximum wall deflections less than \( \delta_h \leq 20\text{mm} \). The maximum bending moments developed on the two walls have an average value, \( M_{\text{max}} = 0.5\text{MNm/m} \) which is considerably less than the yield moment for a reinforced concrete diaphragm wall.

The computed ground surface settlements are maximum at the wall and decrease with distance, such that for free-field, \( \delta_h = 28\text{mm} \). Even though the wall deflections are small, the ground settlement in Category 2 can be as large as 40mm.

4.3.3 Category 3

The last category includes all the cases in which one wall translates towards the excavation and the other rotates about a point close to the excavated grade elevation. This inward movement causes a net translation of the earth support system. The example in Figure 4.4 shows the right wall with maximum horizontal translation, \( \delta_h = 100\text{mm} \) while the left wall rotates with 50mm inward movement at the toe and 100mm outward movement at the top. This behavior may be attributed to the directionality of the base seismic motion. Vytiniotis (2011) proposes a scalar measure of directionality:

\[
I_d = \frac{\pi}{2g} \int_0^\infty (-a(t))^2 dt - \frac{\pi}{2g} \int_0^\infty (-a(t))^2 dt, \quad (4.2)
\]

where:

\( I_d \) is the the directionality index (m/s)
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\[ g \] is the acceleration of gravity (9.81 m/s²)

\[ a(t) \] is the acceleration time history of the ground motion

\[ < x > = \begin{cases} x, & \text{if } x > 0 \\ 0, & \text{if } x \leq 0 \end{cases} \]

If \( I_d < 0.0 \), the ground motion drives the system mostly leftwards while if \( I_d > 0.0 \), the ground motion has a rightwards bias. For the acceleration time history (nga1013), the directionality index, \( I_d = -4.92 \) reveals a strong leftwards bias. Indeed, from Figure 4.4 we can see a net horizontal translation of approximately 100mm at mid-height of the wall. The directionality of the seismic input also means that if we apply to the model the reverse ground motion, the system’s response will also reverse.

For Category 3, the average maximum bending moment ranges from \( M_{\text{max}} = 1.0 \text{ MNm/m} \) to \( 1.5 \text{ MNm/m} \) at about mid-height of the wall (i.e. less than the yield resistance of a typical 1m width reinforced concrete diaphragm wall). The ground settlement patterns are similar to Category 01 with slightly smaller settlements behind the wall. The majority of Category 3 earthquakes show that the ground settlement behind each wall has not only a different pattern but also completely different values in contrast to the previous two groups in which the settlement had a similar trend and similar numerical values for the walls in the same analysis (Figures 4.2d and 4.3d).

4.4 Correlations Between Basic Earthquake Characteristics and Performance

Figure 4.5 summarizes correlations between the computed performance and the maximum ground acceleration (a_{max}) and Arias intensity for the complete suite of earthquake motions. Figures 4.5a1 and 4.5a2 show no clear correlations between the horizontal wall deflection (at the strut level, mid-height or toe elevation) in any of the three categories mentioned above (color variation) with the peak ground acceleration or Arias Intensity.

These Figures show significant correlation between how the wall deflects (directly related to the category of the analysis – color variation -) and both earthquake parameters. In Category 2, the wall deflects almost uniformly. As a result, we can see that the different symbols representing the deflection at different wall elevations are very close together. Moreover, for the same category, the wall deflection seems to be somewhat independent of the two earthquake characteristics examined. The wall deflections of Categories 1 and 3 are much larger than the ones of the second category and their values increase as the peak ground acceleration and Arias Intensity of the seismic input increase.

The maximum bending moments (Figures 4.5b1 and 4.5b2) do show clearly that the maximum bending moment correlates with the earthquake characteristics, and show a well-defined trend with Arias intensity for all earthquakes motions. Figure 4.5b1 suggests a linear
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trend between the maximum wall moment and the peak ground acceleration. The lower bending moment values (< 0.8MN/m) are those of Category 2. On the other hand, higher values belong to Category 1 (>1.5MN/m). As we can easily observe, the increase in $a_{\text{max}}$ leads to an increase in the maximum bending moment exerted on the wall. Moreover, it should be stated that a very strong power law correlation is observed between the maximum moment and the Arias Intensity of the earthquake that follows the same logic as described above.

Figures 4.5c1 and 4.5c2 reveal that there is weak correlation between the maximum ground surface settlement and the earthquake’s characteristics. Finally, Figures 4.5d1 and 4.5d2 show that although there are no correlations between the maximum strut loads and the magnitudes of the earthquakes, all cases show average strut load, $N_{av} = 600 \pm 100$ kN/m. However, the scatter in Figures 4.5d1 and 4.5d2 is quite significant for all three categories.

4.5 Conclusions

This chapter summarizes computed results on the performance of an excavation in dry sand supported by diaphragm wall and a single row of struts subject to a suite of base ground motions. Results of these analyses are summarized in:

- Three distinct categories based on the computed permanent horizontal wall deflections. In Category 1, the two walls primarily rotate around a point that is close to the strut level. Category 2 involves small movements while Category 3 shows net translation of the walls due to the directionality in the base motions.

- The chapter investigates potential correlations between a series of computed wall performance parameters and the reference magnitude properties of the ground motions. The results show clear correlations between peak ground acceleration, $a_{\text{max}}$ and Arias intensity with the maximum wall bending moments, but weaker correlations with wall deflections.
Figure 4.5: Correlation charts between the $a_{\text{max}}$, Arias Intensity of the earthquakes and the horizontal deflections, bending moments of the walls, ground settlements and strut's axial load.
5 Concluding Remarks

5.1 Conclusions

This thesis evaluates the use of numerical analysis as a possible way of predicting how an earth retaining system, comprising three reinforced concrete diaphragm walls connected with a single row of struts (1.5m below ground surface), responds under different ground motions. The finite difference code FLAC v7.0 (Itasca) was used for the numerical simulation of the problem. The walls and strut are modeled as beam elements with interfaces between soil and wall.

Soil behavior is simulated using the PB soil model (Papadimitriou et al., 2002) with input parameters corresponding to medium dense Nevada sand. The PB model combines a kinematic-hardening elastoplastic framework to reproduce soil behavior at large cyclic strains with a Ramberg-Osgood type hysteretic formulation for relatively small shear strains.

The problem simulation was performed in two distinct stages: 1) static simulation of staged excavation; 2) seismic response for a suite of thirty-two different seismic motions selected from the PEER database (applied at the base of the model which was assumed to be a rigid bedrock). The validation of the proposed algorithm was assessed in the end of those two phases by comparing its results in terms of maximum bending moments and horizontal deflections to the analysis results performed by Miriano et al. (2015) for the Assisi earthquake.

For each of the applied seismic ground motions, the performance of the earth support system was instigated from the envelopes of maximum and minimum bending moments and earth pressures, together with post-seismic horizontal wall deflections and ground surface settlements. The computed wall deflections of the retaining system were categorized in three categories (illustrate in Figures 4.2-4.4): 1) the two walls rotate around a point located near the strut level; 2) the two walls exhibit small movement with the maximum conditions at mid-height; 3) one wall rotates and the other translates producing a net horizontal movement due to the directionality of the seismic base motion.

Lastly, the computed responses were then correlated with the reference properties of the ground motions, the peak ground acceleration, $a_{\text{max}}$, and Arias intensity. It was observed then, that as far as the horizontal deflection of the walls and the maximum bending moments are
Chapter 5: Concluding Remarks

Concerned, significant correlations exist between those entities and the earthquake characteristics for each of the three categories created. Regarding the maximum settlement behind the retaining structures, weaker correlations exist with the two earthquake’s parameters of interest and it was also observed that in Category 2, the maximum settlement occurs in the vicinity of the wall in contrast to what happens in the other two categories in which the free-field settlement is the maximum one. Finally, the axial strut loads were found to be uncorrelated for every category with the basic seismic parameters.

5.2 Suggested Topics for Future Work

While working on this research, some points of special interest arose that are worthy of further research. The most important ones are:

- The influence of different soil properties (relative soil density, $D_r$, and soil shear strength parameters) affect the system’s response.

- This thesis focused on a single soil profile (25m deep) overlying rock. Further research should investigate effects of larger soil depth.

- The current earth retaining system is in dry sand. More extensive research is required considering the impact of water. In addition, during the cyclic loading of the model, it produces excess pore pressures that may lead to liquefaction related phenomena.
References


Gazetas, G. (2007). Notes in soil dynamics, National Technical University of Athens (NTUA), unpublished


Chapter 6: References


Appendix A

Acceleration Time History and Fourier Spectra of each Ground Motion

Seismic Motion: nga0779_x-component

Seismic Motion: nga0033_x-component
Chapter 8: Appendix B

Seismic Motion: nga0145_x-component

Seismic Motion: nga0150_x-component

Seismic Motion: nga0448_x-component
Chapter 8: Appendix B

Seismic Motion: nga0451_x-component

Seismic Motion: nga0632_x-component

Seismic Motion: nga0648_x-component
Chapter 8: Appendix B

Seismic Motion: nga0649_x-component

Seismic Motion: nga0669_x-component

Seismic Motion: nga0676_x-component

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Seismic Motion: nga0954_x-component

Seismic Motion: nga0969_x-component

Seismic Motion: nga0982_x-component
Seismic Motion: nga0983_x-component

Seismic Motion: nga1012_x-component

Seismic Motion: nga1013_x-component
Seismic Motion: nga1014_x-component

Seismic Motion: nga1023_x-component

Seismic Motion: nga1031_x-component
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Seismic Motion: nga1055_x-component

Seismic Motion: nga1057_x-component

Seismic Motion: nga1086_x-component
Chapter 8: Appendix B

Seismic Motion: nga1642_x-component

Seismic Motion: nga2399_x-component

Seismic Motion: sim0003_x-component
Appendix B

Performance Parameters Results

Seismic Motion: nga0779_x-component
Chapter 8: Appendix B

Seismic Motion: nga0033_x-component
Seismic Motion: nga0145_x-component
Chapter 8: Appendix B

Seismic Motion: nga0150_x-component
Chapter 8: Appendix B

Seismic Motion: nga0448_x-component
Chapter 8: Appendix B

Seismic Motion: nga0451_x-component

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Chapter 8: Appendix B

Seismic Motion: nga0632_x-component
Chapter 8: Appendix B

Seismic Motion: nga0648_x-component
Chapter 8: Appendix B

Seismic Motion: nga0649_x-component

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Chapter 8: Appendix B

Seismic Motion: nga0669_x-component
Seismic Motion: nga0676_x-component
Seismic Motion: nga0684_x-component
Chapter 8: Appendix B

Seismic Motion: nga0791_x-component
Chapter 8: Appendix B

Seismic Motion: nga0802_x-component
Seismic Motion: nga0954_x-component
Seismic Motion: nga0969_x-component

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Chapter 8: Appendix B

Seismic Motion: nga0982_x-component
Seismic Motion: nga0983_x-component
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Seismic Motion: nga1012_x-component
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Chapter 8: Appendix B

Seismic Motion: nga2399_x-component
Chapter 8: Appendix B

Seismic Motion: sim0003_x-component
Chapter 8: Appendix B

Seismic Motion: sim0005_x-component
Chapter 8: Appendix B

Seismic Motion: sim0006_x-component
Chapter 8: Appendix B

Seismic Motion: sim0007_x-component