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Nonlinear Seismic Behavior of Retaining Wall – Soil Systems

by

Noritake Inada

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE Master of Science

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ABSTRACT

Nonlinear Seismic Behavior of Retaining Wall-Soil Systems

By
Noritake Inada

The prediction of the seismic behavior of waterfront structures has been considered as a challenging problem and attracted significant research interest, especially after the severe damage of such structures in Niigata, Japan, during the 1964 Niigata Earthquake.

The objective of the present study is to improve our understanding of the effects of the backfill material and the wall-soil interface on the seismic behavior and safety performance of retaining walls. The nonlinear analyses are conducted by using an explicit finite-difference formulation for large-deformation analysis of soil-structure systems subjected to seismic excitation.

The effects of separation at the wall-soil interface are investigated assuming a fix-based rigid wall and a linearly elastic backfill material. These effects are found to be significant, resulting to wall forces and moments that may be, respectively, 25 and 40% larger than those based on the assumption of no separation.

The effects of nonlinearity of a typical saturated backfill soil in a waterfront structure are investigated by considering three different materials, namely, loose sand, medium sand and dense sand. The study examines the effects of relative density, intensity of base excitation, frequency of base excitation and number of cycles of loading on the wall pressures, forces, moments, displacements and rotations. The results show the dramatic effect of the excess pore-water pressure buildup that may lead to liquefaction or cyclic mobility.
ACKNOWLEDGEMENT

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I dedicate this thesis to everyone who has supported me throughout my life.
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<th>Description</th>
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<tr>
<td>AF</td>
<td>Amplification Function</td>
</tr>
<tr>
<td>c</td>
<td>damping coefficient</td>
</tr>
<tr>
<td>C</td>
<td>maximum propagating wave speed for FLAC</td>
</tr>
<tr>
<td>$D_r$</td>
<td>relative density of sand</td>
</tr>
<tr>
<td>e</td>
<td>void ratio</td>
</tr>
<tr>
<td>$\bar{E}_v$</td>
<td>tangent modulus of the one-dimensional curve at a point corresponding to the vertical effective stress</td>
</tr>
<tr>
<td>F</td>
<td>non-dimensional parameter for the moment in dynamic condition</td>
</tr>
<tr>
<td>$F_{st}$</td>
<td>dimensionless static stress</td>
</tr>
<tr>
<td>$F_{dv}$</td>
<td>base shear in dynamic condition</td>
</tr>
<tr>
<td>$F_i$</td>
<td>force vector for node i</td>
</tr>
<tr>
<td>$F_x$</td>
<td>force along the x-coordinate</td>
</tr>
<tr>
<td>$F_y$</td>
<td>force along the y-coordinate</td>
</tr>
<tr>
<td>f</td>
<td>frequency</td>
</tr>
<tr>
<td>$f_i$</td>
<td>fundamental natural frequency</td>
</tr>
<tr>
<td>G</td>
<td>shear modulus of elasticity</td>
</tr>
<tr>
<td>$G_{m0}$</td>
<td>initial shear modulus</td>
</tr>
<tr>
<td>$G_{mt}$</td>
<td>shear modulus at time t</td>
</tr>
<tr>
<td>$G_s$</td>
<td>specific gravity</td>
</tr>
<tr>
<td>H</td>
<td>thickness of the soil layer, the height of the wall</td>
</tr>
<tr>
<td>h</td>
<td>dimensionless height of the wall</td>
</tr>
<tr>
<td>$K_0$</td>
<td>coefficient of lateral stress</td>
</tr>
<tr>
<td>$K_a$</td>
<td>coefficient of active soil pressure</td>
</tr>
<tr>
<td>$K_w$</td>
<td>bulk modulus of water</td>
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L  width of the rigid base
M_a  dimensionless static moment
M_d  moment in dynamic condition
M  non-dimensional parameter for the moment in dynamic condition
n_e  porosity of the sand
n_i^{(1)}, n_i^{(2)}  nodal forces with two different arc 1, 2 for node I
P_{n,m}  static-one-g model participation factor for mode n, m
PA_{n,m}  pseudo acceleration response for mode n, m
Q  non-dimensional parameter for the base shear in dynamic condition
Q_{n,m}(t)  participation coefficient for mode n, m
q  dimensionless stress against the wall
r_d  pore-water pressure ratio
s_1^{(1)}, s_1^{(2)}  arc lengths 1, 2 for the triangle described in Figure 2.1
u  hydrostatic pressure
u(z)  total displacement at depth z in the soil
\ddot{u}(z)  total velocity at depth z in the soil
\dddot{u}(z)  total acceleration at depth z in the soil
\dddot{u}_b  peak ground acceleration in x-direction
\dddot{u}_q^{(a)}, \dddot{u}_q^{(b)}  velocity vector for node i with both side grids a, b
u_{n,m}(x, y)  displacement component in x-direction of mode n, m
V_1^*  complex-valued shear wave velocity of the soil
V_s  shear wave velocity of the soil
V_d  dilatational wave speed
\beta  hysteretic damping ratio
\gamma  seismic shear strain
\( \gamma_d \)  
Dry unit weight of sand

\( \gamma_{sat} \)  
Saturated unit weight of sand

\( \delta \)  
Material damping factor

\( \Delta t \)  
Smallest timestep for FLAC to keep stability

\( \Delta x \)  
Smallest size of element for FLAC to keep stability

\( \Delta \varepsilon_{vd} \)  
Volumetric strain change

\( \varepsilon_{vd} \)  
Volumetric strain

\( \phi \)  
Friction angle of the sand

\( \phi_{n,m}(t) \)  
Mode shape for mode n, m

\( \nu \)  
Poisson's ratio

\( \rho \)  
Mass density

\( \sigma_{ij} \)  
Stress components for node i, j

\( \sigma_{h0} \)  
Initial effective stress in vertical direction

\( \sigma_x \)  
Normal stress along the x-coordinate

\( \sigma_y \)  
Normal stress along the y-coordinate

\( \sigma_v \)  
Effective stress in vertical direction

\( \sigma_{v0} \)  
Initial effective stress in vertical direction

\( \tau \)  
Current value of shear stress

\( \tau_{xy} \)  
Shear stress in the x-y plane

\( \tau_{m0} \)  
Initial shear strength

\( \tau_{mi} \)  
Shear strength at time t

\( \omega \)  
Circular frequency of excitation

\( \omega_{n,m} \)  
Natural angular mode frequency for mode n, m

\( \zeta_{n,m}(t) \)  
Fraction of critical damping in mode n, m
Chapter 1

Introduction

1.1 Objectives

The prediction of the seismic response of waterfront structures has been considered as a challenging problem since the severe damage of waterfront structures in Niigata, Japan, in the 1964 Niigata Earthquake. Following the Hansin Great Earthquake in 1995, which caused severe damage on the retaining walls of the ports of Kobe, Japan, and the surrounding area, significant progress has been made in understanding the dynamic characteristics and seismic behavior of retaining including the effects of liquefaction. It is obvious that reliable prediction of their behavior and better understanding of the seismic response characteristics of retaining walls can prevent such severe failures in existing unsafely designed retaining walls. Despite the significant progress, still, the effects of nonlinearity of the materials and the nonlinearity of the interface between the wall and the retained soil layer are yet to be fully understood. The objective of the present study is to improve our understanding of the effects of the material and interface nonlinearity on the seismic behavior and safety of retaining walls.

To this end, the present work investigates first the effect of interface nonlinearity on a retaining wall supporting a backfill layer that consists of a linearly elastic soil. Subsequently, the study examines the effect of material nonlinearity by considering the behavior of three different backfill materials, including saturated loose sand, medium
sand, and dense sand, and the associated effects of excess pore-water pressure buildup and liquefaction.

1.2 Background

Retaining walls in port facilities are highly vulnerable to strong earthquake shaking. Failures of such facilities may result in major disruptions of critical commercial operations and have serious economic consequences for the stricken regions. A dramatic recent reminder of the potential for significant damage was the Hansin Great Earthquake, which destroyed or severely damaged 95% of all waterfront facilities in the port of Kobe with repair cost exceeding the amount of $11 billion.

Typical waterfront structures include gravity walls, sheet pile walls and pile supported wharves. The response of such structures to strong ground shaking depends on the complex soil-structure-foundation interaction and the undrained behavior of the foundation and backfill soils. Current philosophy in the design of such systems aims at limiting deformations of the supported structures to acceptable levels, depending on their importance and function. A practical approach in the design of such structures is to allow: (a) limited deformation without loss of serviceability for the design earthquake; and (b) repairable damage for the maximum credible earthquake. New design methods and novel ground improvement techniques allow significant control over the expected level of damage. The success of certain ground improvement techniques has been clearly demonstrated by the satisfactory performance of several previously treated sites subjected to the Great Hansin Earthquake (JGS 1996). Presently, there is considerable need for further investigation of the performance and optimization of these techniques for reliable
and cost effective design or improvement of waterfront structures. Moreover, there is need to establish practical analytical methods for seismic evaluation of the effectiveness of such systems.

The evaluation of the seismic pressure induced on the back of a retaining structure is an important factor in the development of sound principles and guidelines for the design of such structures. To this end, numerous studies have been conducted by various investigators over several decades. These studies may be divided in three categories: (a) limit analysis studies in which the backfill soil is assumed to reach the failure state and the wall is moving or yielding; (b) linear elastic analysis studies in which the backfill soil is assumed to behave as an elastic material and the wall responds as a rigid or elastic body; (c) finite element studies in which the stress-strain behavior of the backfill material is modeled with a nonlinear model. Detailed accounts of representative studies have been presented by Nazarian and Hadian (1979), Whitman (1991), Veletsos and Younan (1995).

Based on limited analyses, the method proposed by Mononobe-Okabe (Mononobe and Matsuo 1929, Okabe 1924) has been widely used all over the world as a simple means of computing seismic pressures using a pseudostatic extension of the Coulomb static method. Subsequent studies by Seed and Whitman (1970), Sherif et al. (1982, 1984) offered modifications and improvements of the Mononobe-Okabe method. By utilizing the Mononobe-Okabe earth pressures, the permanent displacement of a retaining wall can be evaluated based on a Newmark-type analysis (Newmark 1965, Franklin and Chang 1977, Richards and Elms 1979, Nadim and Whitman 1983, Tawhata and Islam 1987). In
a similar way, tilt may also be evaluated by extending the simplified method for rotation (Nadim and Whitman 1983, Whitman and Liao 1984, Steedman and Zeng 1976).

For a wall that moves rigidly with the underlying base, the seismic pressures would be larger than those predicted by the Mononobe-Okabe theory that assumes active failure conditions. Considering a homogeneous linearly elastic layer confined by two vertical rigid boundaries, Wood (1973, 1975) provided exact analytical solutions and comprehensive results of earth pressures, base shear and moment applied on the wall.

To investigate the effects of the flexibility of the wall itself and of its elastic support, Veletsos and his co-workers developed a series of analytical solutions for rigid and flexible walls that are elastically constrained against rotation (Veletsos and Younan 1997, 1994a, 1994b, Veletsos et al. 1995, Younan and Veletsos 1999). The results of these studies have shown that for realistic wall flexibilities the maximum seismic pressures are significantly lower than those obtained for fixed-based rigid walls and of the same order of magnitude as those computed by the Mononobe-Okabe method.

For submerged soils encountered in waterfront structures, the assumptions of simplified approaches are questionable. During cyclic shear loading, depending on its density and confining stress, soil may dilate or contract, causing a change in the pore-water pressure. Thus, loose sands may experience liquefaction and dense sands may develop negative excess pore-water pressure due to their dilatant nature. The nonlinearity of the soil behavior and the complexity of the soil-structure interaction phenomena associated with waterfront structures require the use of numerical methods for the study of their seismic behavior. Detailed accounts of recent research efforts using advanced numerical methods may be found in an excellent state-of-the-art paper by Iai (1998).
1.3 Scope of work

The first part of the study describes the key features of the numerical formulation that is used to analyze the response of the wall-soil system subjected to a strong shaking and examines its accuracy.

The second part of the study investigates the response of the retaining wall supporting a backfill soil that consists of a linearly elastic material. Two types of interface behavior have been considered: the first allows sliding but no separation during the seismic shaking, whereas the second allows both sliding and separation. An exact analytical solution derived by Wood (1973), is utilized to verify the results of the numerical formulation and to investigate the effects of separation at the soil-wall interface on the dynamic pressures exerted on the wall.

The third part of the study investigates the response of the walls retaining three different backfill soils, loose, medium and dense sands, which experience liquefaction flow or cyclic mobility during a strong seismic shaking. The effect of excess pore-water pressure buildup on the behavior of the wall is analyzed and the permanent displacements and rotation angles of the wall for different loading condition are investigated. For the cases studied, design graphs for estimating the permanent displacement and rotation of the wall are presented. Additional parametric studies are necessary to develop a more general, simplified method for estimating the permanent displacements of the wall.
Chapter 2

Numerical Formulation for
Large Deformation Analysis

2.1 Introduction

The nonlinear behavior of waterfront structures is studied utilizing an efficient numerical formulation named 'Fast Lagrangian Analysis of Continua' (FLAC) developed by Itasca Consulting Group, Inc (1995). This chapter examines the key features of the numerical formulation, the method of analysis and its accuracy.

FLAC is a 2D or 3D explicit finite difference program developed for numerical solutions of problems in engineering mechanics, geomechanics, mining, etc. This program simulates the behavior of structures built of soil, rock, or other materials that may undergo plastic flow when their yield limits are reached. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or non-linear stress-strain law in response to the applied forces or boundary restraints. The material can yield and flow, and the grid can deform and move with the material that is represented. The explicit calculation scheme, explained in Section 2.2, and the mixed-zoning technique used in FLAC ensure that plastic collapse and flow are modeled very accurately.
FLAC contains many useful capabilities that are essential for realistic nonlinear soil-structure interaction problems under both static and dynamic loading. The most important of them are:

1. Groundwater and consolidation (fully coupled) model
2. Interface elements to simulate distinct planes along which slip and/or separation can occur
3. Constitutive models for soils subjected to both static and dynamic loading
4. Capability for user-defined models that allows incorporation of any elastoplastic model for static and dynamic analysis

2.2 Explicit finite difference method

In the finite difference method, the derivatives contained in the governing equations are replaced with algebraic equations written in terms of field variables (such as stress, displacement, pore-water pressure, etc) at discrete points. Even though the set of algebraic equations is derived in a different way than that in the finite element method, the two sets of equations are identical.

The difference between the two methods is that finite element methods often combine the element matrices into a large global stiffness matrix, whereas this is not normally done with finite differences because it is relatively efficient to regenerate the finite difference equations at each step.

FLAC uses an 'explicit time-marching method' to solve the algebraic equation. The general calculation sequence embodied in FLAC is illustrated in Figure 2.1. This procedure first invokes the equations of motion to derive new velocities and
displacements from stresses and forces. Then, strain rates are derived from velocities, and new stresses from strain rates. This cycle around the loop takes place in one time step. Each box in Figure 2.1 updates all of its grid variables from known values that remain fixed while control is within the box. The accumulation of these cycles of the loop is considered as the "dynamic time".

As mentioned earlier, there are several advantages of this method and, most importantly, no iteration process is necessary when using this method, even if the constitutive law is widely nonlinear. On the other hand, the disadvantage of the explicit method is the small timestep, which means that large numbers of steps must be taken. To acquire accurate results, the stability condition must be satisfied, which for an elastic solid discretized into elements of size $\Delta x$ is given by:

$$\Delta t < \frac{\Delta x}{C}$$ (2.1)

where $C$ is the maximum speed at which information can propagate, $\Delta t$ is the smallest time step for this method. If the model requires a fine mesh, $\Delta x$, and therefore, $\Delta t$ must be small, leading to a large number of steps.

The solid body can be divided into a finite difference mesh composed of quadrilateral elements to accommodate any geometry. Internally, FLAC subdivides each element into two overlayed sets of constant-strain triangular elements as shown in Figure 2.2(a). The four triangular sub-elements are termed a, b, c and d. The stress components of each triangle are maintained independently. The force vector exerted on each node is taken to be the mean of two force vectors exerted by two overlaid quadrilaterals. Each side of a triangle has two tractions that are divided into two equal forces at the two end nodes.
Each triangle corner has two force contributions from the two adjoining sides, which are added as follows:

\[ F_i = \frac{1}{2} \sigma_{ij} \left( n_j^{(1)} s^{(1)} + n_j^{(2)} s^{(2)} \right) \]  
(2.2)

where \( F_i \) is the force vector at the node, \( \sigma_{ij} \) is the stress component for each node, \( n_j^{(1)}, n_j^{(2)} \) are the normal vector on the triangle side, \( s^{(1)}, s^{(2)} \) are the lengths for each triangle side.

In this way, the response of the composite element is symmetric, for symmetric loading. If one pair of triangles becomes badly distorted by the external force, then the corresponding quadrilateral is not used and only nodal forces from the other quadrilateral are used. That is the method of 'mixed discretization' that allows the model to deform more naturally and accurately. More importantly, using this method, boundaries can be of any shape and any elements can have any property value.
Fig 2.1 Basic explicit calculation cycle

Fig 2.2 The grid for numerical formulation; (a) Overlaid quadrilateral elements used in FLAC; (b) Typical triangular element with velocity vectors; (c) Nodal force vector
\section*{2.3 Verification Example: Response of a Homogeneous Soil Layer}

The numerical formulation has been systematically verified in its various modes of analysis by its developer. As an illustrative example of the accuracy of the numerical solution, as well as the proper use of the model, a simple case of one-dimensional wave propagation is considered in this section, and the results are compared to those derived from the exact analytical solution.

The model consists of a homogeneous, infinitely long elastic soil layer on a rigid base, subjected to harmonic horizontal motion (Figure 2.3(a)). The response parameter examined is the amplification of the motion evaluated at the surface of the layer. The amplification function (AF) is defined as:

\[ AF = \frac{u(z)}{u(H)} = \frac{\ddot{u}(z)}{\ddot{u}(H)} = \frac{\dddot{u}(z)}{\dddot{u}(H)} \quad (2.3) \]

where \( u(z) \), \( \ddot{u}(z) \) and \( \dddot{u}(z) \) are the total displacement, velocity and acceleration at depth \( z \) and \( H \) is the thickness of the soil layer. The analytical solution of AF is given by:

\[ AF = \frac{1}{\cos(\omega H / V_s)} \quad (2.4) \]

where \( \omega \) is the circular frequency of excitation, \( H \) is the thickness of elastic soil layer, and \( V_s \) is the complex-valued shear wave velocity of the soil, given by:

\[ V_s = V_s \sqrt{1 + 2i\beta} \quad (2.5) \]

in which \( V_s \) is the shear velocity of the soil, \( \beta \) is the hysteretic damping ratio of the soil and \( i = \sqrt{-1} \).
For the example examined here, a layer of thickness \( H = 10 \text{ m} \) is considered, having an uniform shear wave velocity of \( V_s = 200 \text{ m/s} \), unit weight \( \gamma = 20 \text{ kN/m}^3 \), Poisson’s ratio \( \nu = 0.3 \), and hysteretic damping ratio \( \beta = 0.05 \). The fundamental natural frequency of the layer is given by:

\[
 f_1 = \frac{V_s}{4H} = \frac{200 \ (\text{m} / \text{s})}{4 \times 10 \ (\text{m})} = 5 \text{ Hz}
\]  

(2.6)

A discretization of the geometry of the problem is shown in Figure 2.3(b). Note that free-field conditions apply at the two vertical soil columns attached to the two boundaries of the finite-difference grid. The grid nodes at the two vertical boundaries are connected to the free-field segments with appropriate dashpots that dissipate any difference in the energy between the free-field region and the grid boundaries.

Figures 2.4 to 2.7. plot the AF versus time until steady-state harmonic response is reached for four frequencies namely \( f/f_1 = 0.9, 1.0, 1.1, 3.0 \).

The steady-state values of AF from several analyses are plotted in Figure 2.8 along with the analytical solution from Equation (2.4) versus a dimensionless frequency \( f/f_1 \).

It is evident from Figure 2.8 that there is very good agreement between the exact analytical solution and the numerical solution by FLAC. A comprehensive verification study presented by the developer (Itasca, 1995) demonstrate that FLAC is an accurate and efficient numerical method to analyze the behavior of a soil-structure system under seismic conditions.
Fig 2.3  Homogeneous elastic soil layer subjected to horizontal harmonic motion:
(a) Soil layer and material properties
(b) Finite difference discretization and boundary conditions
Fig 2.4 Amplification Function versus time: $f / f_1 = 0.9$

Fig 2.5 Amplification Function versus time: $f / f_1 = 1.0$
Fig 2.6  Amplification Function versus time: $f / f_1 = 1.1$

Fig 2.7  Amplification Function versus time: $f / f_1 = 3.0$
Fig 2.8 Amplification Function at the surface of the layer versus dimensionless frequency
Chapter 3

Dynamic earth pressures on a rigid retaining wall

3.1 Introduction

It is of interest to examine the dynamic earth pressures on a rigid retaining wall imposed by a homogeneous elastic soil layer subjected to harmonic horizontal motion. The study presented in this chapter consists of the following parts:

1. First, the exact analytical solution of the elastic response of the soil-wall system subjected to a static horizontal loading is examined in Section 3.2. The exact solution developed by Wood (1973) is used for analyzing the response of the retaining wall to a uniform body force imposed by the elastic material toward the wall. Also, the results for the numerical formulation are presented and compared to those of the exact solution, for verification of the proper use of the model. Understanding of the seismic behavior of this simple soil-wall system is very essential and constitutes the first step in a broader study that would include the effects of the interface of the wall-soil system and material nonlinearity, presented in Sections 3.3, 3.4 and Chapter 4.

2. Secondly, the exact solution of the elastic response of the soil-wall system subjected to base acceleration is examined in section 3.3. In this section, the exact solution developed by Wood (1973) is used for analyzing the response of the retaining wall subjected to a harmonic horizontal ground excitation. Similar results obtained from FLAC are compared to the exact solution for verification of the numerical formulation.
3. Thirdly, the effects on the response of the soil-wall interface nonlinearities are examined by using the numerical formulation. For this purpose the same model as in Section 3.3 is used, except that separation is allowed between the wall and the soil layer. The wall-soil interface separation can be modeled properly by using the numerical formulation. By comparing the results with those developed by Section 3.3, the effects of interface nonlinearity are considered.

3.2 Layer-wall system under horizontal static load

3.2.1 System considered

The system investigated is shown in Figure 3.1. It is a uniform elastic soil layer of width $L$ and height $H$, which is free at its upper surface, is fully bonded to a rigid base, and is retained along its vertical boundaries by rigid walls. The contact between the homogeneous linearly elastic soil and the wall is assumed to be free from shear stresses and does not allow any separation. The lower horizontal boundary represents a rigid layer on which no relative displacement is permitted. A uniform horizontal body force is assumed to act throughout the soil layer. For convenience, the magnitude of this body force is taken as $\gamma$, the unit weight of the soil, and so it is equivalent to the effect of a static horizontal ground acceleration of one $g$.

The properties of the layer are defined by its mass density $\rho$, shear modulus of elasticity $G$ and Poisson's ratio $\nu$. 
3.2.2 The exact analytical solution

The exact analytical solution of the elastic response of the soil-wall system under static condition by Wood (1973) is presented briefly in the following.

Under the assumption of the system mentioned in the previous section, the equilibrium equation for a homogeneous, linearly elastic are

\[
\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + F_x = 0
\] (3.1)

\[
\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + F_y = 0
\] (3.2)

in which

\(\sigma_x, \sigma_y\) = normal stresses along the x- and y-coordinates, respectively

\(\tau_{xy}\) = shearing stress in the x-y plane

\(F_x, F_y\) = body forces per unit volume along the x- and y-coordinates, respectively

The stress-strain relations may be expressed as

\[
\frac{\sigma_x}{G} = K \frac{\partial u}{\partial x} + (K - 2) \frac{\partial v}{\partial y}
\] (3.3)

\[
\frac{\sigma_y}{G} = (K - 2) \frac{\partial u}{\partial x} + K \frac{\partial v}{\partial y}
\] (3.4)

\[
\frac{\tau_{xy}}{G} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}
\] (3.5)

in which

\(u, v\) = displacements in the x and y directions, respectively

\(G\) = shear modulus
\[ K = \frac{V_s^2}{V_d^2} = \frac{2(1 - \nu)}{1 - 2\nu} \]

\[ V_d = \text{dilatational wave velocity} \]

\[ V_s = \text{shear wave velocity} \]

\[ \nu = \text{Poisson's ratio} \]

Substitution of Equations (3.3), (3.4) and (3.5) into Equations (3.1) and (3.2) gives the equilibrium equations for the problem in terms of the displacements. For the cases of no vertical body force these equations are

\[ K \frac{\partial^2 u}{\partial x^2} + (K - 1) \frac{\partial^2 v}{\partial x \partial y} + \frac{\partial^2 u}{\partial y^2} = \frac{\gamma}{G} \quad (3.6) \]

\[ K \frac{\partial^2 v}{\partial y^2} + (K - 1) \frac{\partial^2 u}{\partial x \partial y} + \frac{\partial^2 v}{\partial x^2} = 0 \quad (3.7) \]

for \( 0 < x < L; \quad 0 < y < H \)

The constant forcing term \( \frac{\gamma}{G} \) can be expanded in a Fourier series to give

\[ \frac{\gamma}{G} = \sum_{n=1,3,5,\ldots} a_n \sin \frac{n \pi x}{L} \quad (3.8) \]

\[ \frac{\gamma}{G} = \frac{4}{\pi} \left( \sin \frac{\pi x}{L} + \frac{1}{3} \sin \frac{3\pi x}{L} + \frac{1}{5} \sin \frac{5\pi x}{L} + \ldots \right) \quad (3.9) \]

The solution of Equations (3.6) and (3.7) can be expressed as

\[ u(x, y) = \sum_{n=1,3,5,\ldots} \tilde{u}_n(y) \sin n \pi x \quad (3.10) \]

\[ v(x, y) = \sum_{n=1,3,5,\ldots} \tilde{v}_n(y) \cos n \pi x \quad (3.11) \]
in which
\[ r = \frac{n\pi}{L}, \quad n=1, 3, 5 \ldots \]

A solution of this form clearly satisfies the boundary condition at \( x = 0 \), independently of the functions \( \bar{u}_n(y), \bar{v}_n(y) \).

Substitution of Equations (3.9) to (3.11) into (3.6) and (3.7) gives a coupled pair of equations for each of the terms in the series form of the solution. The equations for the \( n \)-th term are
\[ \frac{\partial^2 \bar{u}_n}{\partial y^2} - r^2 K \bar{u}_n - (k-1) r \frac{\partial \bar{v}_n}{\partial y} = \frac{4\gamma}{n\pi g} \] 
(3.12)
\[ \frac{\partial^2 \bar{v}_n}{\partial y^2} - \frac{r^2 \bar{v}_n}{K} - \frac{(k-1)}{K} r \frac{\partial \bar{u}_n}{\partial y} = 0 \] 
(3.13)

It can be shown by substitution that Equations (3.12) and (3.13) are satisfied by the displacement functions
\[ \bar{u}_n(y) = \frac{a_n}{r^2 K} \left\{ B_n \cosh(ry) + C_n (ry + k)e^{\gamma} + D_n (ry - k)e^{-\gamma} - 1 \right\} \] 
(3.14)
\[ \bar{v}_n(y) = \frac{a_n}{r^2 K} \left\{ -B_n \sinh(ry) - C_n rye^{\gamma} + D_n rye^{-\gamma} \right\} \] 
(3.15)
in which
\[ k = 3 - 4\nu \]
\[ B_n, C_n, D_n = \text{constants that can be determined by satisfying the boundary conditions at } y = 0 \]

The condition that \( \bar{v}_n = 0 \) at \( y = 0 \) is satisfied directly by the second equation. The condition \( \bar{u}_n = 0 \) at \( y = 0 \) gives
\[ B_n = k(D_n - C_n) + 1 \]  
\[ \text{(3.16)} \]

The boundary conditions at \( y = H \) are \( \tau_{xy} = 0 \), \( \sigma_y = 0 \). From Equations (3.3) to (3.11), it can be shown that this stress-free surface condition is satisfied if

\[ \frac{\partial u_n}{\partial y} - r \frac{v_n}{y} = 0 \quad \text{(at } y = H) \]  
\[ \text{(3.17)} \]

\[ (K - 2) r u_n + K \frac{\partial v_n}{\partial y} = 0 \quad \text{(at } y = H) \]  
\[ \text{(3.18)} \]

Substitution of Equations (3.14), (3.15) and (3.16) into (3.17) and (3.18) gives the following linear algebraic equations for the constants \( C_n, D_n \).

\[ C_n \left\{ (2rH + k + 1)e^{rh} - 2k \sinh(rH) \right\} + \]
\[ D_n \left\{ (-2rH + k + 1)e^{-rh} + 2k \sinh(rH) \right\} = -2k \sinh(rH) \]  
\[ \text{(3.19)} \]

\[ C_n \left\{ (2rH + k - 1)e^{rh} - 2k \cosh(rH) \right\} + \]
\[ D_n \left\{ (2rH - k + 1)e^{-rh} + 2k \cosh(rH) \right\} = -(K - 2 + 2 \cosh(rH)) \]  
\[ \text{(3.20)} \]

Solving for \( C_n, D_n \) gives

\[ C_n = -\frac{1}{\Delta} \left[ e^{rh} \left\{ (2rH - k - 1)(K - 2 + 2 \cosh(rH) \right) \right. \]
\[ + (2rH - k + 1)(2 \sinh(rH)) \} - 2(K - 2)k \sinh(rH) \]  
\[ \text{(3.21)} \]

\[ D_n = -\frac{1}{\Delta} \left[ e^{-rh} \left\{ (2rH + k + 1)(K - 2 + 2 \cosh(rH) \right) \right. \]
\[ - (2rH - k + 1)(2 \sinh(rH)) \} - 2(K - 2)k \sinh(rH) \]  
\[ \text{(3.22)} \]

in which

\[ \Delta = 2 \left\{ 1 + (2rH)^2 + k^2 + 2k \left( \sinh^2(rH) + \cosh^2(rH) \right) \right\} \]  
\[ \text{(3.23)} \]

Hence from expression (3.14)

\[ B_n = -\frac{2}{\Delta} \left[ (K - 2)k \left\{ (2rH \sinh(rH) + (k + 1) \cosh(rH) \right\} \right. \]
\[ - \left\{ 1 + (2rH)^2 - K \right\} \]  
\[ \text{(3.24)} \]
The complete displacement solution can be expressed in the following dimensionless form

\[
\frac{\mu G}{\gamma H^2} = \frac{4}{\pi^2 k^2} \left( \frac{L}{H} \right)^2 \sum_{n=1,3,5} \frac{1}{n^2} \left\{ B_n \cosh(ry) + C_n (ry + k) e^{-n} + D_n (ry - k) e^{-n} - 1 \right\} \times \sin(ry)
\]

(3.25)

\[
\frac{v G}{\gamma H^2} = \frac{4}{\pi^2 k^2} \left( \frac{L}{H} \right)^2 \sum_{n=1,3,5} \frac{1}{n^2} \left\{ -B_n \sinh(ry) - C_n ry e^{-n} + D_n ry e^{-n} - 1 \right\} \times \cos(ry)
\]

(3.26)

in which

\[ r = \frac{n \pi}{L}, \quad n=1,3,5, \ldots \]

By substituting Equations (3.25), (3.26) into (3.3), the complete solution for horizontal dimensionless stresses is

\[
\frac{\sigma_r}{\gamma H} = \frac{4}{\pi^2 k^2} \frac{L}{H} \sum_{n=1,3,5} \frac{1}{n^2} \left\{ 2B_n \cosh(ry) + C_n (2ry + k + 3) e^{-n} + D_n (2ry - k - 3) e^{-n} - K \right\} \times \cos(rx)
\]

(3.27)

By using Equation (3.27), the wall pressure distributions for \( v \) equals to 0.2, 0.3, 0.4 and 0.5 are shown in Figures 3.2 and 3.3, for aspect ratios \( L/H = 5 \) and 50, respectively.

The two figures plot a dimensionless stress, \( q \), given by

\[
q = \frac{\sigma_r}{\gamma H}
\]

(3.28)

versus a dimensionless height, \( h \), equal to

\[
h = \frac{y}{H}
\]

(3.29)
3.2.3 Numerical solution

The elastic response of the soil-wall system under static conditions is also computed using the numerical formulation. The finite grid discretization for the case of $L/H = 5$ is shown in Figure 3.4.

Figure 3.5 presents the normalized pressure distribution at the back of the wall for aspect ratio $L/H = 5$ and Poisson’s ratios $v=0.3$ and 0.4. The results represented with open circles correspond to the numerical solution, whereas the results represented by the solid curves correspond to the exact solution given by Wood. It is evident that the exact solution and the numerical solution are in very good agreement.
Fig 3.1 Soil-wall system subjected to uniform horizontal body force, $\gamma$: Base and soil-wall interfaces do not allow separation.
Fig 3.2 Pressure distributions on rigid wall for static horizontal body force $\gamma$ and $L/H = 5$, derived from Wood's solution.

Fig 3.3 Pressure distributions on rigid wall for static horizontal body force $\gamma$ and $L/H = 5$, derived from Wood's solution.
Fig 3.4  Finite difference discretization of layer-wall system (L/H=5).
Fig 3.5 Comparison between pressures from Wood's solution and FLAC solution for (a) $\nu = 0.3$ and (b) $\nu = 0.4$ ($L/H = 5$).
3.3 Layer-wall system subjected to base acceleration

3.3.1 System considered

The system investigated is shown in Figure 3.6. It is a uniform elastic soil layer of width \( L \) and height \( H \), which is free at its upper surface, is fully bonded to a rigid base, and, is retained along its vertical boundaries by rigid walls. In this section, the base of the layer and the wall are presumed to be excited by a horizontal motion, the acceleration of which at any time \( t \) is given by \( a = \ddot{u}_b \sin \omega t \).

The properties of the layer are defined by its mass density \( \rho \), shear modulus of elasticity \( G \), Poisson’s ratio \( \nu \), and the material damping ratio \( \beta \).

3.3.2 Exact analytical solution

The exact analytical solution of the elastic response of the soil-wall system subjected to a base acceleration given by Wood (1973) is presented briefly in the following. The displacement equation of the motion for the homogeneous linearly elastic layer can be readily derived from the equilibrium equations and the stress-strain relations (3.1) to (3.5) by the addition of appropriate inertia terms to the equilibrium equations.

The equations of motion for this problem are shown as

\[
G \left\{ \frac{\partial^2 u(x, y, t)}{\partial x^2} + (K - 1) \frac{\partial^2 v(x, y, t)}{\partial x \partial y} + \frac{\partial^2 u(x, y, t)}{\partial y^2} \right\} = \rho \frac{\partial^2 u(x, y, t)}{\partial t^2} + \frac{\partial^2 u_b(t)}{\partial t^2}
\]

(3.30)
\[ G \left\{ \frac{\partial^2 v(x, y, t)}{\partial x^2} + (K - 1) \frac{\partial^2 u(x, y, t)}{\partial x \partial y} + K \frac{\partial^2 v(x, y, t)}{\partial y^2} \right\} = \rho \frac{\partial^2 v(x, y, t)}{\partial t^2} \]  

(3.31)

in which

\[ \rho \] = mass density of soil

\[ u_b(t) \] = displacement in x-direction of rigid boundary

The effect of dissipation within the system is approximated by the addition of viscous damping terms in the analysis. With these terms, the equations of motion may be written in vector form as

\[ L \ddot{u}(x, y, t) = \rho \ddot{u}(x, y, t) + c \ddot{u}(x, y, t) + \rho \ddot{u}_b(t) \]  

(3.32)

in which

\[ L \] = linear operator with respect to the special coordinates

\[ \ddot{u}(x, y, t) = \begin{bmatrix} u(x, y, t) \\ \dot{v}(x, y, t) \end{bmatrix} \] = vector of the displacement components \( u \) and \( v \)

\[ u_b(t) = \begin{bmatrix} u_b(t) \\ 0 \end{bmatrix} \] = vector of displacements on the rigid boundary

\[ c \] = damping coefficient

The dots above the symbols means that differentiation with respect to time.

Equation (3.32) may be solved by the standard normal mode expansion technique. Because the normal modes form a complete set of functions, the solution can be expressed as

\[ u(x, y, t) = \sum_{n=1}^{\infty} Q_{n,m}(t) \phi_{n,m}(x, y) \]  

(3.33)

in which
\[ Q_{n,m}(t) = \text{participation coefficient for mode } n,m \]

\[ \phi_{n,m}(x, y) = \text{mode shape for mode } n,m \]

\[ n = 1, 2, 3, \ldots \]

\[ m = 1, 2, 3, \ldots \]

Substitution of expression (3.33) into (3.32) gives

\[ \sum_{n=1}^{\infty} Q_{n,m}(t) L \phi_{n,m}(x, y) = \sum_{n=1}^{\infty} \left\{ \rho \ddot{\phi}_{n,m}(t) + c \dot{\phi}_{n,m}(t) \right\} \times \phi_{n,m}(x, y) + \rho \dddot{u}_b(t) \tag{3.34} \]

From the equations of motion for free undamped vibrations

\[ L \phi_{n,m}(x, y) = -\rho \omega_{n,m}^2 \phi_{n,m}(x, y) \tag{3.35} \]

For convenience let the damping coefficient be expressed as

\[ c = 2 \rho \omega_{n,m} \zeta_{n,m} \tag{3.36} \]

in which

\[ \omega_{n,m} : \text{natural angular frequency of mode } n,m \]

\[ \zeta_{n,m} : \text{fraction of critical damping in mode } n,m \]

Substitution of Equation (3.35) and (3.36) into (3.34) gives

\[ -\rho \sum_{n=1}^{\infty} \left\{ \dot{\phi}_{n,m}(t) + 2\omega_{n,m} \zeta_{n,m}(x, y) \phi_{n,m}(x, y) \dot{Q}_{n,m}(t) + \omega_{n,m}^2 Q_{n,m}(t) \right\} \times \phi_{n,m}(x, y) \]

\[ = \rho \dddot{u}_b(t) \tag{3.37} \]

The uncoupled equation of motion for each of the model participation coefficients can be derived by forming the dot product of equation (3.37) with a particular mode,
investigating over the volume and employing the orthogonal property of the modes. On the assumption of uniform soil density these equations of motion can be expressed as

\[ \ddot{Q}_{n,m}(t) + 2\omega_{n,m} \zeta_{n,m} \dot{Q}_{n,m}(t) + \omega^2_{n,m} Q_{n,m}(t) = \frac{-\ddot{u}_b(t) \int_V u_{n,m}(x,y) dV}{\int_V \phi_{n,m}(x,y) \cdot \phi_{n,m}(x,y) dV} \quad (3.38) \]

in which

\[ u_{n,m}(x,y) = \text{displacement component in x-direction of mode } \phi_{n,m} \]

\[ V = \text{volume} \]

The solution of equation (3.38) for zero initial conditions is given by

\[ Q_{n,m}(t) = \frac{\int_V u_{n,m}(x,y) dV}{\int_V \phi_{n,m} \cdot \phi_{n,m} dV} \times \int_0^t \ddot{u}_b(\tau) e^{-\zeta_{n,m} \omega_{n,m} (t-\tau)} \sin \omega_{n,m} \sqrt{1 - \zeta^2_{n,m}} (t-\tau) d\tau \frac{1}{\omega_{n,m} \sqrt{1 - \zeta^2_{n,m}}} \quad (3.39) \]

Let

\[ P_{n,m} = \frac{-g}{\omega^2_{n,m}} \frac{\int_V u_{n,m} dV}{\int_V \phi_{n,m} \cdot \phi_{n,m} dV} \quad (3.40) \]

\[ P_{n,m} \] is called the static-one-g model participation factor for mode n,m. Let

\[ PA_{n,m}(t) = \omega^2_{n,m} D_{n,m}(t) \quad (3.41) \]

in which

\[ D_{n,m}(t) = \frac{1}{\omega_{n,m} \sqrt{1 - \zeta^2_{n,m}}} \int_0^t \ddot{u}_b(\tau) e^{-\zeta_{n,m} \omega_{n,m} (t-\tau)} \times \sin \omega_{n,m} \sqrt{1 - \zeta^2_{n,m}} (t-\tau) d\tau \quad (3.42) \]
the relative displacement response of a single degree of freedom oscillator, with undamped natural angular frequency \( \omega_{n,m} \), and fraction of critical damping \( \zeta_{n,m} \), to a base excitation \( \ddot{u}_s(t)/g \).

\( PA_{n,m}(t) \) is the pseudo-acceleration response for mode \( n,m \). Thus, the complete solution of Equation (3.33) can be expressed as

\[
\ddot{u}(x, y, t) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} P_{n,m} PA_{n,m}(t) \phi_{n,m}(x, y)
\]  

(3.43)

From the stress-strain relations in Equation (3.3),(3.4) and (3.5), the normal stress on the wall can be expresses as

\[
\sigma_x(0, y, t) = L_p \ddot{u}(x, y, t)
\]  

(3.44)

in which

\[ L_p = \text{a linear operator with respect to the spatial coordinates} \]

Thus the normal pressure distribution on the wall, resulting from an arbitrarily time-varying horizontal acceleration of the rigid boundaries, can be expressed as

\[
\sigma_x(0, y, t) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} P_{n,m}(y) PA_{n,m}(t)
\]  

(3.45)

in which

\[ P_{n,m}(y) = P_{n,m} L_{n,m}(x, y) \]  

(3.46)

The horizontal base shear and base moment are expressed as

\[
F_{dx} = \int_0^h \sigma_x(0, y, t) dy
\]  

(3.47)
\[ M_{d_v} = \int_0^H y \sigma_y(0, y, t) dy \]  

(3.48)

It is convenient to normalize \( F_{d_v} \) and \( M_{d_v} \) using the following non-dimensional parameters

\[ Q = \frac{F_{d_v}}{\rho \ddot{u}_b H^2} \]  

(3.49)

\[ M = \frac{M_{d_v}}{\rho \ddot{u}_b H^3} \]  

(3.50)

in which \( \rho \) is the unit weight of the soil, \( \ddot{u}_b \) is the maximum ground acceleration.

Figure 3.7 plots the normalized amplitude of base shear and moment per unit length for a harmonically excited system with a fixed-base wall using Wood’s exact solution described above. The solution is given for Poisson’s ratio \( \nu = 0.3 \) and material damping ratio \( \beta = 0.05 \).

In addition to the exact solution, the solution derived by the numerical formulation is also plotted with open circles. The results from the exact solution and the numerical solution are again in good agreement.

Figures 3.8 to 3.10 plot time histories of normalized base shear and moment for three characteristic frequency ratios, \( f/f_1 = 0.5, 1.0 \) and \( 2.0 \).
Fig 3.6 Soil-wall system subjected to harmonic transient horizontal excitation; Base and soil-wall interface allows sliding without separation.
Fig 3.7 Amplitude of (a) base shear and (b) moment per unit length for harmonically excited system with fixed-based wall; sliding with no separation is allowed between the wall and the soil layer: $v = 0.3$, $\beta = 0.05$, $f_i = 5$ Hz.
Fig 3.8  Dimensionless (a) base shear and (b) moment against the wall per unit length for harmonically excited system with fixed-based wall; sliding with no separation not allowed between the wall and the soil layer; $f = 2.5$ Hz, $v = 0.3$, $\beta = 0.05$. 
Fig 3.9 Dimensionless (a) base shear and (b) moment against the wall per unit length for harmonically excited system with fixed-based wall; sliding with no separation not allowed between the wall and the soil layer; 
$f = 5 \text{ Hz}, v = 0.3, \beta = 0.05$. 
Fig 3.10  Dimensionless (a) base shear and (b) moment against the wall per unit length for harmonically excited system with fixed-based wall; sliding with no separation not allowed between the wall and the soil layer; 
\( f = 10 \text{ Hz}, \nu = 0.3, \beta = 0.05. \)
3.4 Effect of interface nonlinearity

3.4.1 System considered

In the previous section, the interface between wall and soil layer was considered smooth but it did not allow horizontal separation of the two bodies. This represents the idealized conditions assumed in the derivation of Wood's exact solution.

In this section, it is of interest to examine the effect of nonlinearity, associated with a wall-soil interface allowing both sliding and separation. The system investigated in this section is shown in Fig 3.11.

3.4.2 The experimental results

The system consists of a fixed-base rigid wall supporting a semi-infinite, homogeneous, linearly elastic layer of soil. The soil layer has thickness $H$, unit weight $\gamma$, shear wave velocity $V_s$, Poisson's ratio $\nu$, and material damping ratio $\beta$.

The soil-wall interface is smooth ($\tau_{xy} = 0$) and allows separation of the two bodies during application of tensile stresses. Finally, the soil layer is fixed at its bottom to a rigid base.

To be able to account for the separation at the soil-wall interface, the numerical formulation is utilized. The discretized model consists of a layer of thickness $H = 10$ m, unit weight $\gamma = 20$ kN/m$^3$, shear wave velocity $V_s = 200$ m/s, Poisson's ratio $\nu = 0.3$, and material damping ratio $\beta = 0.05$. A Rayleigh-type material damping has been used in the analysis.
A harmonic input excitation is applied at the base of the soil-wall system, and the free-field response at the “infinite” end of the soil layer. This is achieved by discretizing only a portion of the soil layer having aspect ratio $L/H = 5$, and connecting the distant vertical boundary of the soil layer to a free-field soil column with appropriate dashpots.

A parametric study is undertaken to compute the dynamic pressures, base shear and base moment exerted on the wall for a wide range of frequencies of the input excitation.

To investigate the effect of interface separation during dynamic loading, both dynamic and static gravity induced active soil pressures must be considered. The associated static forces and moment for active conditions, $F_a$ and $M_a$, respectively, are given by

$$F_a = \frac{1}{2} \gamma H^2 K_a$$  \hspace{1cm} (3.51)

$$M_a = F_a \cdot \frac{H}{3}$$  \hspace{1cm} (3.52)

in which $K_a$ is the coefficient of active soil pressure,

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

in which $\phi$ is a friction angle.

In this study, friction angle $\phi = 35^\circ$ is applied, for which $F_a$ is 270 kN/m and $M_a$ is 900 kN-m/m. In a dimensionless form, these values are

$$Q_{\text{static}} = \frac{F_a}{\rho u_b H^2} = 0.675$$  \hspace{1cm} (3.53)

$$M_{\text{static}} = \frac{M_a}{\rho u_b H^3} = 0.225$$  \hspace{1cm} (3.54)
in which $\ddot{u}_b$ is the amplitude of the base acceleration, which in this study is taken equal to 0.2 g.

These values are added to the dimensionless dynamic values of $Q$ and $M$, respectively, and the total values are compared to the total values computed in Section 3.3, where no separation was allowed at the soil-wall interface.

Figure 3.12(a) and (b) plot the dimensionless total force $Q$ and moment $M$, respectively, versus the frequency ratio $f/f_1$, where $f_1$ is the fundamental frequency of the soil layer ($f_1 = 5$ Hz). For comparison, the two figures also plot the corresponding total values of $Q$ and $M$ for the case of a soil-wall interface that does not allow separation, derived from both the exact and the numerical solution in Section 3.3.

The results in Figure 3.12 demonstrate the effect of the separation at the soil-wall interface. Indeed, separation at the interface results to higher maximum forces and moments between $f/f_1 = 0.5$ and $f/f_1 = 2.5$ than those computed assuming no separation. For $f/f_1 = 1$, $Q$ and $M$ are 15 % and 23 % higher, respectively, and, for $f/f_1 = 2$, they are 20 % and 40 % higher, respectively.

These higher values are attributed to the fact that, during separation from the wall, the soil layer accumulates more kinetic energy than that of a soil mass bonded to the wall. During movement of the backfill soil away from the fixed-based rigid wall, the soil is not constrained by the wall and can move freely. Upon rebound, this kinetic energy is transformed into elastic strain energy resulting to higher pressures acted on the wall, with maximum increase of $Q$ and $M$ at $f/f_1 = 1$. On the other hand, for $f/f_1$ is less than 0.5 and $f/f_1$ is larger than 2.5, no difference is observed for both forces and moments.
Figures 3.13 (a) and (b) plot the maximum dynamic pressure distribution against the wall for the cases of bonded and unbonded (separation allowed) interfaces for frequency ratios $f/f_1 = 0.5$, 1 and 2, respectively. Similarly, Figures 3.14 (a) and (b) plot the total pressure distribution, consisting of the active pressures at static condition and the dynamic pressure for frequency ratios $f/f_1 = 0.5$, 1 and 2, respectively.

Figures 3.15 (a), (b) and (c) plot the pressure distribution against the wall for the cases of unbonded interfaces for frequency ratio $f/f_1 = 1$ at the top, middle and bottom of the wall. The results of Figures 3.15 show that, for all 3 cases, there is no tensile stress between the wall and the soil layer because of the interface nonlinearity. It is interesting to note that, at the top and middle parts of the wall, there are few moments that stress becomes zero, which clearly indicates the separation.

Figures 3.16 to 3.18 plot time histories of normalized total base shear and moment for three characteristic frequency ratios $f/f_1 = 0.5$, 1.0 and 2.0.
Fig 3.11 Soil-wall system subjected to harmonic transient horizontal excitation: The soil-wall interface allows separation and sliding.
Fig 3.12 Comparison of (a) base shear and (b) moment from the nonlinear and linear interface for peak ground acceleration = 0.2g; \( v = 0.3, \beta = 0.05, f_1 = 5 \text{ Hz} \).
Fig 3.13  Maximum dynamic pressure distribution against the wall for (a) linear interface and (b) nonlinear interface for peak ground acceleration = 0.2g.
Fig 3.14 Maximum total pressure distribution against the wall for (a) linear interface and (b) nonlinear interface for peak ground acceleration = 0.2g.
Fig 3.15 Pressure time history at the (a) top, (b) middle and (c) bottom of the wall; separation and sliding are allowed between the wall and the soil layer; for peak acceleration = 0.2g, f = 5 Hz, ν = 0.3, β = 0.05.
Fig 3.16 Dimensionless base shear and moment against the wall per unit length for harmonically excited system with fixed-based wall considering both dynamic and static stress; sliding and separation are allowed between the wall and the soil layer; \( f = 2.5 \text{ Hz} \), \( v = 0.3 \), \( \beta = 0.05 \).
Fig 3.17 Dimensionless base shear and moment against the wall per unit length for harmonically excited system with fixed-based wall considering both dynamic and static stress; sliding and separation are allowed between the wall and the soil layer; $f = 5 \text{ Hz}$, $\nu = 0.3$, $\beta = 0.05$. 
Fig 3.18 Dimensionless base shear and moment against the wall per unit length for harmonically excited system with fixed-based wall considering both dynamic and static stress; sliding and separation are allowed between the wall and the soil layer; $f = 10 \, \text{Hz}$, $\nu = 0.3$, $\beta = 0.05$. 
3.5 Conclusions

The conclusions of this study are summarized as follows:

1. For the static one-g horizontal loading presented in section 3.2, the results in Figure 3.5 show a very good agreement between Wood's exact solution and the numerical solution.

2. For the case of harmonic base excitation, the results in Fig 3.7 show good agreement between Wood's exact solution and the numerical solution. The maximum difference is less than 10% in all the considered cases. These minor differences are attributed to differences in the calculation methods of two solutions. Wood's solution for a semi-infinite layer (L/H = ∞) is computed by considering L/H = 50, since for higher values of L/H, the evaluation of the solution leads to numerical difficulties. The numerical solution by FLAC involves errors due to discretization and probably the use of Rayleigh damping, which may vary slightly with frequency, as opposed to the viscous damping used in Wood's solution.

3. The effect of soil-wall separation on the values of the induced pressures, forces and movements of the wall is found to be significant. For the case for which separation is allowed, the values of maximum forces and moments may be respectively about 25 and 40% higher than those obtained assuming no separation. It is concluded that whenever separation is possible, its effects should be properly accounted for in the computation of forces and moments acting on the wall.
Chapter 4

Nonlinear inelastic seismic response of a retaining wall-backfill soil system

4.1 Introduction

This chapter examines the seismic behavior of a waterfront retaining wall supporting a cohesionless backfill-soil that behaves in a nonlinear manner. The system considered, shown in Figure 4.1, consists of a rigid retaining wall supporting a saturated sand layer resting on a rigid base. Three different relative densities for the sand material are considered, namely, loose sand, medium sand and dense sand, which behave in a distinctively different manner under undrained cyclic loading conditions. The soil-wall-water system in Figure 4.1 is subjected to a harmonic horizontal base excitation, and the nonlinear response and permanent deformation of the wall and backfill are examined.

4.2 Background

4.2.1 Behavior of undrained saturated sand under cyclic loading

Experimental data show that dry loose sand subjected to either monotonic or cyclic shear tends to contract. On the other hand, dry dense sand subjected to small-strain cyclic shear tends to contract too, but under large strains, either monotonic or cyclic, it tends to dilate.
When the sand is fully saturated and drainage is prohibited, the tendency for contraction or dilation of the sand skeleton is causing an increase in the pressure of the relatively incompressible water filling the voids. The accumulated excess pore-water pressure after a number of cycles is causing a decrease of the inter-particle contact stresses that leads to a reduction of the strength and stiffness of the sand.

Figure 4.2 shows an element of soil with initial effective stresses in the vertical and horizontal direction given by

\[ \sigma_{\text{vo}} = \sigma_{\text{vo}} - u \]  

\[ \sigma_{\text{ho}} = K_0 \sigma_{\text{vo}} \]  

in which \( \sigma_{\text{vo}} \) is the total stress in vertical direction; \( \sigma_{\text{vo}} \) is the effective stress in vertical direction; \( \sigma_{\text{ho}} \) is the effective stress in horizontal direction; \( u \) is the hydrostatic pressure and \( K_0 \) is the coefficient of lateral stress at rest. As the excess pore-water pressure increases during cyclic shear loading, the value of \( u \) increases and the effective stress \( \sigma_{\text{vo}} \) decreases. At the ultimate case, the vertical effective stress \( \sigma_{\text{vo}} \) may become zero, suggesting zero strength for the saturated sand. This condition is called initial liquefaction and is encountered frequently in level soil sites containing layers of loose saturated sand.

The presence of initial static shear stresses, such as those beneath the foundation of a building, retaining wall, dam, etc. may lead to an unstable condition, as the excess pore-water pressure builds up during several cycles of shear loading. In this case, the saturated loose sands (and the supported structures) may experience a sudden liquefaction flow failure.
Examples of such liquefaction flow failures are abundant, the most dramatic recent ones being the liquefaction of the waterfront structures in Kobe, Japan, in the Great Hansin Earthquake.

When the saturated soil is dense sand, then the accumulation of large strains causes a tendency for dilation, which in turn curbs the movement of the soil and prevents a liquefaction flow failure. However, dense sand under cyclic shear load may still accumulate gradually very large deformations that may be catastrophic for the structure. This phenomenon of accumulation of large deformation in saturated dense sands subjected to cyclic shear loading is called cyclic mobility.

It should be noted that the phenomena of initial liquefaction, liquefaction flow failure, and cyclic mobility are different types of liquefaction. The physics and mechanics of liquefaction has been the subject of several experimental and analytical studies over the past 30 years. Detailed accounts of recent and past contributions may be found in NRC (1985), Kramer (1996), Dakoulas, et al. (1998a), Lade and Yamamoto (1999), etc.

4.2.2 Modeling of the undrained behavior of sand

To be able to predict the behavior of a mass of saturated soil under cyclic loading, a realistic constitutive model for the soil is required. Such a model should be able to capture with sufficient accuracy the stress-strain behavior and the coupling of the shear and volumetric strain that is fundamental to the representation of the excess pore-water pressure buildup.

To this end, a large number of constitutive models for the drained-undrained behavior of sand have been proposed by various investigators that range from simple,
physically motivated models to advanced models representing several aspects of soil behavior and requiring many soil parameters.

This study is based on a simple model developed by Finn et al. (1977) that has been used extensively for the last twenty years for solving 1-D and 2-D problems. It is an effective stress model for the monotonic and cyclic behavior of sands that is based on few, physically meaningful, soil properties. Among these properties are the initial in-situ shear modulus at small strains $G_{m_0}$ and the initial shear strength of the soil $\tau_{m_0}$.

The modulus $G_{m_0}$ can be obtained from in-situ measurements of the shear wave velocity, $V_s$, from the expression

$$G_{m_0} = V_s^2 \cdot \rho$$

(4.3)

in which $\rho$ = mass density. Alternatively, it may be estimated by the experimental expression

$$G_{m_0} = 14760 \frac{(2.973 - e)^2}{1 + e} \left(1 + \frac{2K_0}{3}\right)^{1/2} \sqrt{\sigma_v}$$

(4.4)

in which $e$ = void ratio, $K_0$ = coefficient of lateral stress, $\sigma_v$ = vertical effective stress ($\sigma_v$, $G_{m_0}$ in pounds per square foot).

The initial shear strength $\tau_{m_0}$ can be computed from

$$\tau_{m_0} = \left[\left(\frac{1 + K_0}{2} \sin \phi \right)^2 - \left(\frac{1 - K_0}{2} \right)^2\right]^{1/2} \sigma_v$$

(4.5)

in which $\phi$ = the effective friction angle of the sand.

The stress-strain relationship during the initial monotonic loading stage is expressed with a hyperbolic model given by
\[
\tau = \frac{G_{mo} \gamma}{1 + \frac{G_{mo} \gamma}{\tau_{mo}}} \quad (4.6)
\]

in which \( \tau \) = seismic shear stress, \( \gamma \) = seismic shear strain, \( G_{mo} \) = initial shear modulus and \( \tau_{mo} \) = initial shear strength. Figure 4.3 plots the hyperbolic curve forming the so-called backbone curve used for monotonic loading.

For the unloading and reloading stages, the backbone curve from the initial loading stage is modified using the Masing rule that is consistent with experimental evidence. According to this rule, the unloading curve is similar to the backbone curve, but its origin is translated to the unloading point, the curve is rotated by 180° and both scales of stress and strain are multiplied by 2. Thus, the unloading curve becomes

\[
\frac{\tau - \tau_1}{2} = \frac{G_{mo} (\gamma - \gamma_1)}{2} \left/ \left\{ 1 + \frac{G_{mo} (\gamma - \gamma_1)}{2 \tau_{mo}} \right\} \right.
\]

(4.7)
in which \( \tau \) and \( \gamma \) are the current values of shear stress and strain, and, \( \tau_1 \) and \( \gamma_1 \) are the values at the point where unloading starts.

Similarly, the reloading curve is written as

\[
\frac{\tau + \tau_2}{2} = \frac{G_{mo} (\gamma - \gamma_2)}{2} \left/ \left\{ 1 + \frac{G_{mo} (\gamma - \gamma_2)}{2 \tau_{mo}} \right\} \right.
\]

(4.8)
in which \( \tau_2 \) and \( \gamma_2 \) are the values at the point where reloading starts.

Fig 4.4 plots the backbone, unloading and reloading curves at time t, using the hyperbolic relationship and the Masing criterion. Notice that the maximum shear modulus at time t, \( G_{mo} \), and the shear strength \( \tau_{mo} \) are different than those at time t = 0.
This is due to the degradation of the shear modulus and shear strength with increasing excess pore-water pressure. The values of $G_{mr}$ and $\tau_{mr}$ at time $t$ are

$$G_{mr} = G_{mr0} \left( \frac{P_0^*}{P_0} \right)^{1/2} \quad (4.9)$$

$$\tau_{mr} = \tau_{mr0} \left( \frac{P_0^*}{P_0} \right) \quad (4.10)$$

in which $P_0^* = (\sigma_1^* + \sigma_2^* + \sigma_3^*)/3$ at time $t$ and $P_0^* = (\sigma_{10}^* + \sigma_{20}^* + \sigma_{30}^*)/3$ at time $t = 0$.

It can be shown that (Finn et al. 1977) the excess pore-water pressure during one cycle of loading is given by

$$u_{\text{excess}} = \frac{\Delta \varepsilon_{vd}}{\frac{1}{E_r} + \frac{n_e}{K_w}} \quad (4.11)$$

in which $E_r$ = the tangent modulus of the one-dimensional unloading curve at a point corresponding to the vertical effective stress, $n_e =$ porosity of soil; $K_w = $ bulk modulus of water, and $\Delta \varepsilon_{vd} = $ the volumetric strain change during one cycle of loading.

For saturated sands, water may be assumed to be effectively incompressible. By assuming $K_w \gg E_r$, Equation (4.11) becomes

$$\Delta u = \frac{\Delta \varepsilon_{vd}}{E_r} \quad (4.12)$$

The volumetric strain change $\Delta \varepsilon_{vd}$ can be expressed as a function of the amplitude of cyclic shear $\gamma$, and the accumulated volumetric strain $\varepsilon_{vd}$.

Based on the experimental results by Martin et al. (1975), $\Delta \varepsilon_{vd}$ is expressed as

$$\Delta \varepsilon_{vd} = C_1(\gamma - C_2\varepsilon_{vd}) + \frac{C_3 \varepsilon_{vd}^2}{\gamma + C_4 \varepsilon_{vd}} \quad (4.13)$$
in which $C_1$, $C_2$, $C_3$, and $C_4$ are constants that depend on the sand type and relative density and $\varepsilon_{uv}$ is the accumulated shear strain.

As an illustrative example, Figure 4.5 plots the increase of pore-water pressure and the decrease of vertical effective stress versus the number of cycles for an element of sand having initial vertical stress $\sigma_{v0} = 325$ kPa and relative density $D_r = 45\%$. Notice that at 5 cycles of loading the vertical effective stress $\sigma_{v0}$ has been reduced to zero, reaching the state of initial liquefaction.
Fig 4.1  Idealized waterfront structure subjected to base excitation. The wall-soil interface allows sliding but no separation. At the wall-rigid base interface both sliding and separation are allowed.
Fig 4.2 Basic model of cyclic loading

Fig 4.3 The hyperbolic stress-strain relationship
Fig 4.4  Loading, unloading, and reloading at time $t$ using the hyperbolic stress-strain relationship and the Masing Criteria.
Fig 4.5 Effective stress and pore-water pressure under cyclic loading:
Relative density 45%; Initial vertical effective stress = 325 kPa;
Peak ground acceleration = 0.5g.
4.3 Nonlinear seismic behavior of a wall-soil system

It is of interest to examine the behavior of the system in Figure 4.1 under harmonic base excitation. The concrete wall in Figure 4.1 has a height of 10 m, width of 2 m and is resting on a rigid base. The wall is supporting a cohesionless backfill material having thickness $H = 10$ m and extending to infinity. Three different soils are examined as backfill material: (a) loose sand with relative density $D_r = 45\%$, (b) medium sand with relative density $D_r = 60\%$ and, (c) dense sand with relative density $D_r = 80\%$. It is reminded that the loose sand displays a contracting behavior leading to liquefaction whereas the dense sand displays dilative behavior leading to cyclic mobility. The material properties for the three sands used as backfill soil, including the parameters for Finn’s model, are given in Table 4.1. The wall has been proportioned so that it has a factor of safety against overturning failure $FS = 2.31$ using the Mononobe-Okabe pseudostatic method for $\ddot{u}_b = 0.2g$. The wall may slide and separate at its base, but it can only slide without separation at its interface with the backfill soil. The latter assumption of no separation at the wall-backfill interface is justified by the fact that when the wall tends to separate during an earthquake shaking, the stress reduction will soften the sand that will deform quickly to fill any gap between wall and backfill. In addition, the incompressibility of the water and its (finite) resistance in tension most likely will not allow any significant separation between the wall and backfill material. Sliding however is expected to occur during movements of the wall and backfill, and it is modeled properly. It should be noted that in case of very strong ground shaking, the assumption of no separation may not be appropriate.
Figure 4.6 presents the finite-difference discretization of the wall-soil system. Only a finite length equal to \( L = 5 \cdot H \) of the backfill is discretized, whereas the rest of the backfill is represented with a column of soil at free-field conditions, connected to the end of the discretized boundary of the backfill with appropriate dashpots.

The hydrodynamic effects of the sea water are modeled approximately by considering the water as an elastic material having the bulk modulus and unit weight of the water, and a very low value of shear modulus. Although the shear modulus of water is of course zero, the small value used here is necessary for numerical stability and has very small influence on the results.

The friction angle at the wall-soil interface is taken equal to \( \phi_w = 30^\circ \) and the wall-base \( \phi_s = 30^\circ \) for all analyses.

A parametric study was conducted to examine the effects of the following parameters on the seismic behavior of the wall:

(a) the relative density of backfill (\( D_r = 45, 60, 80 \% \)).

(b) the amplitude of harmonic base acceleration. Three values of \( \ddot{u}_b \) are considered, equal to 0.2g, 0.35g and 0.5g.

(c) the frequency of harmonic base excitation. Three frequency ratios are considered, namely, \( f/f_1 = 0.5, 1 \) and 1.5, where \( f_1 \) is the natural frequency of the soil layer.

(d) the number of cycles of base excitation. The number of cycles considered ranges from 1 to 20.
The response parameters evaluated in each analysis are

(a) the distribution of dynamic pressures at the back of the wall
(b) the base shear
(c) the overturning moment
(d) the wall displacements
(e) the wall rotation
(f) the excess pore pressure
### Properties of Crystal Silica Sand No. 20

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Table 4.1 Properties of the three sands used as backfill soil
Fig 4.6 Finite-difference discretization of water-wall-layer system; D = 20m, B = 2m, L = 50m and H=10m
4.4 Results and discussion

Figures 4.7 to 4.16 plot the horizontal pressures, forces, moments acting on the wall, displacements of the wall and displacement vectors for the case of loose sand (Sand A) and dense sand (Sand C), subjected to a harmonic base excitation with peak ground acceleration \( a_g = 0.2 \) g at frequency ratio \( f/f_1 = 0.5 \).

Figures 4.7(a) and (b) plot total horizontal stresses near the top, at the mid-height and at the base of the wall versus time for Sand A (relative density = 45%) and Sand C (relative density = 80%). The corresponding effective horizontal stresses and pore-water pressures for the three points are plotted in Figures 4.8 and 4.9, respectively.

As shown in Figure 4.9, the response of the loose sand to this excitation is characterized by a quick increase of the pore-water pressure even within one cycle of loading, which results in an immediate drop of the strength and stiffness of the loose sand. This results in relatively small oscillation of stresses, after the significant reduction in stiffness and the associated filtering of strains and accelerations. On the other hand, the response of the dense sand to this excitation is characterized by a slower increase of the pore-water pressure even after five cycles of loading, which results in a smaller decrease of the strength and stiffness of the dense sand compared to that of the loose sand. It is interesting to note that although the effective horizontal stresses and the pore-water pressures appear different for the two sands, the average total horizontal stress applied on the wall is not very different (see Figures 4.7 (a) and (b)).

Figures 4.10 and 4.11 plot the total force and total moment acting against the wall, as well as the contributions due to the pore-water and sand particles, versus time. The results
demonstrate clearly the significant contribution of the pore-water pressure on the total force and moment which is about 80 to 90% for both sands.

Figures 4.12(a) and (b) plot the total horizontal displacement at the bottom of the wall, during the first 8 cycles, whereas Figures 4.13(a) and (b) plot the corresponding relative displacements. The relative displacements show clearly the sliding history of the wall on the rigid base during shaking. The results show much smaller sliding in Sand C compared to Sand A.

Figures 4.14(a) and (b) plot the total displacements at the top of the wall, reaching the value of 1 m for Sand A and 80 cm for Sand C.

Figures 4.15 and 4.16 plot the vectors of displacement around the wall, indicating that the predominant mode of deformation of the wall is rotation around its toe, whereas sliding is comparatively small.

Figures 4.17(a) and (b) plot the dynamic force and moment respectively, exerted on the retaining wall by the backfill soil, for the three sands, A, B and C and for peak ground acceleration $\ddot{u}_g = 0.2g$ and frequency ratio $f/f_1 = 0.5$. The results are plotted versus the number of cycles of loading.

The results in Figure 4.17 show an increased force and moment during the first cycle of loading, with a significant drop immediately afterwards for the case of loose sand A. As shown earlier, this indicates that during the first cycle of loading, significant pore-water pressures have developed in the loose sand, which decrease both the stiffness and strength of the soil. This is indeed confirmed in Figure 4.22(a) which plots the pore-water pressure versus the number of cycles for this case. The figure shows that the pore-water pressure jumps from the hydrostatic value of 490 kPa to a value of 740 kPa in one cycle,
and continues to increase at a much slower rate at subsequent cycles. Figure 4.22(b) plots the pore-water pressure ratio, defined as

\[ r_d = \frac{u_c}{\sigma_{v0}} \]  \hspace{1cm} (4.14)

in which \( u_c \) = the excess pore-water pressure developed during the earthquake shaking and \( \sigma_{v0} \) = the initial vertical effective stress before the earthquake. When \( r_d \) approaches one, the current effective stress approaches zero, and, therefore, liquefaction takes place. The ratio \( r_d \) for sand A in Figure 4.22(b) becomes about 0.77 in one cycle and approaches rapidly the value of one.

By contrast, the dense sand C in Figure 4.17 induces significantly smaller force and moment on the wall at first cycle, which continue to increase with increasing number of cycles. This behavior is attributed to the higher strength and stiffness of this sand and the much smaller excess pore-water pressures developed, as shown in Figures 4.22(a) and (b).

As expected, the response of the medium-dense sand (B) yields values that are between those from sand A and sand C (see Figure 4.17 and 4.22).

The results discussed above are given for a frequency ratio \( f/f_1 = 0.5 \). Similar results for \( f/f_1 = 1 \) are presented in Figure 4.18 and 4.23, and for \( f/f_1 = 1.5 \) in Figure 4.19 and 4.24.

It is of interest to note that the maximum response in terms of force, moment and excess pore pressures is observed at \( f/f_1 = 1.5 \). This is attributed to the fact that, as degradation due to straining and excess pore-water pressure generation begins, the initial fundamental frequency of the soil layer decreases from \( f_1 \) to \( f'_1 \) and the ratio \( f/ f'_1 \) may
approach the value of one, resulting to maximum response. Obviously, more analyses with different frequencies near resonance are needed to provide a more clear representation of the effect of frequency on the system response. However, this is beyond the scope of this study.

The effect of the intensity of the ground excitation is shown in Figures 4.17, 4.20 and 4.21, which plot the dynamic forces and moment for frequency ratio \( f/f_1 = 0.5 \) and peak ground acceleration \( \ddot{u}_b = 0.2g, 0.35g, \) and \( 0.5g, \) respectively.

The results show increasing values of the dynamic force and moments with increasing base acceleration. They also show, that as \( \ddot{u}_b \) increases, a significant reduction occurs in the forces in moments after the first cycle, due to the substantial increase of excess pore water pressure in all sands, regardless of relative density. This is indeed shown in Figure 4.25 and 4.26 for \( \ddot{u}_b = 0.35g, \) and \( 0.5g, \) respectively.

Figures 4.27(a) and (b) plot the horizontal displacement and rotation angle towards the sea for the peak ground acceleration \( \ddot{u}_b = 0.2g \) and frequency ratio \( f/f_1 = 0.5 \) versus the number of cycles.

The results in Figure 4.27(a) show a relatively small displacement at the base of the wall up to 5 cycles of loading, followed by a steady rapid increase for sands A and B, and a slower increase for the denser sand C. Rotations shown in Figure 4.27(b) increase with the number of cycles but after 10 cycles, rotation values become more or less constant. Similar trends for displacements are observed in Figure 4.28(a) and 4.29(a), corresponding to peak ground acceleration \( \ddot{u}_b = 0.35g \) and \( 0.5g, \) respectively, although the values are substantially higher than those of Figure 4.27(a).
The associated values of rotation for $\ddot{u}_b = 0.35g$ and $0.5g$ do not seem higher than those for $\ddot{u}_b = 0.2g$ due to sliding at the wall base. The relatively small rotations are also attributed to the presence of the rigid base, which does not allow any yielding of the foundation of the wall that could increase significantly the wall rotation. Moreover, as the wall tends to tilt, pore-water pressures and stresses behind the wall would tend to decrease, causing a decrease in the overturning moment.

Figures 4.30 to 4.32 and Figures 4.33 to 4.35 plot similar results on displacements and rotations for the frequency ratios $f/f_1 = 1$ and 1.5, respectively. Similar trends are observed in the results for the two higher frequencies.

To assist in a simple estimation of the permanent deformation of the system considered, the displacement and rotation results are interpolated for more values of peak ground acceleration and plotted in Figures 4.36 to 4.44.
Fig 4.7 Total horizontal pressure at the top, middle and bottom of the wall for (a) Sand A, (b) Sand C; Frequency = 1.35 Hz. Peak ground acceleration = 0.2g.
Fig 4.8 Effective horizontal pressure at the top, middle and bottom of the Wall for (a) Sand A, (b) Sand C: Frequency = 1.35 Hz. Peak ground acceleration = 0.2g.
Fig 4.9 Pore-water pressure at the top, middle and bottom of the wall
For (a) Sand A, (b) Sand C; Frequency = 1.35 Hz.
Peak ground acceleration = 0.2g.
Fig 4.10 Total, effective and pore-water force against the wall for (a) Sand A, (b) Sand C; Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.11 Total, effective and pore-water moment against the wall for (a) Sand A, (b) Sand C; Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.12 Total displacement at the bottom of the wall for
(a) Sand A, (b) Sand C; Frequency = 1.35 Hz,
Peak ground acceleration = 0.2g.
Fig 4.13  Relative displacement at the bottom of the wall between wall and base for (a) Sand A, (b) Sand C;
Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.14 Displacement at the top of the wall for (a) Sand A, (b) Sand C:
Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.15 Displacement vectors of the model for Sand A after 10 cycles; Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.16 Displacement vectors of the model for Sand C after 10 cycles; Frequency = 1.35 Hz, Peak ground acceleration = 0.2g.
Fig 4.17 Dynamic (a) force and (b) moment against the wall from the sand layer; Frequency = 1.35Hz. Peak ground acceleration = 0.2g.
Fig 4.18 Dynamic (a) force and (b) moment against the wall from the sand layer; Frequency = 2.7 Hz, Peak ground acceleration = 0.2 g.
Fig 4.19 Dynamic (a) force and (b) moment against the wall from the sand layer; Frequency = 4.05Hz, Peak ground acceleration = 0.2g.
Fig 4.20 Dynamic (a) force and (b) moment against the wall from the sand layer:
Frequency = 1.35Hz. Peak ground acceleration = 0.35g.
Fig 4.21  Dynamic force and moment against the wall from the sand layer; Frequency = 1.35 Hz, Peak ground acceleration = 0.5 g.
Fig 4.22  (a) Pore-water pressure at the bottom of the sand layer and (b) Pore-water pressure ratio, \( r_u \); Frequency = 1.35 Hz, Peak ground acceleration = 0.2 g.
Fig 4.23  (a) Pore-water pressure at the bottom of the sand layer and (b) Pore-water pressure ratio, $\frac{p}{\sigma_s}$; Frequency = 2.7Hz, Peak ground acceleration = 0.2g.
Fig 4.24 (a) Pore-water pressure at the bottom of the sand layer and (b) Pore-water pressure ratio.  
Frequency = 4.05 Hz, Peak ground acceleration = 0.2 g.
Fig 4.25  (a) Pore-water pressure at the bottom of the sand layer and (b) Pore-water pressure ratio, $r_0$; Frequency = 1.35Hz, Peak ground acceleration = 0.35g.
Fig 4.26 (a) Pore-water pressure at the bottom of the sand layer and (b) Pore-water pressure ratio. Freq: Frequency = 1.35Hz, Peak ground acceleration = 0.5g.
Fig 4.27 (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 1.35 Hz, Peak ground acceleration = 0.2 g.
Fig 4.28  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 1.35Hz, Peak ground acceleration = 0.35g.
Fig 4.29  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 1.35Hz, Peak ground acceleration = 0.5g.
Fig 4.30  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 2.7Hz, Peak ground acceleration = 0.2g.
Fig 4.31 (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 2.7 Hz, Peak ground acceleration = 0.35 g.
Fig 4.32 (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 2.7Hz, Peak ground acceleration = 0.5g.
Fig 4.33  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 4.05Hz, Peak ground acceleration = 0.2g.
Fig 4.34  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles: Frequency = 4.05 Hz, Peak ground acceleration = 0.35 g.
Fig 4.35  (a) Sliding distance and (b) rotation angle of the wall for the number of cycles; Frequency = 4.05 Hz, Peak ground acceleration = 0.5 g.
Fig 4.36  Estimation of (a) permanent wall displacement and (b) rotation.  
Frequency ratio = 0.5, Sand A (relative density = 45 %)  
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.37 Estimation of (a) permanent wall displacement and (b) rotation. Frequency ratio = 1. Sand A (relative density = 45%) Peak ground acceleration = 0.2 to 0.5g.
Fig 4.38  Estimation of (a) permanent wall displacement and (b) rotation. Frequency ratio = 1.5. Sand A (relative density = 45 %)
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.39  Estimation of (a) permanent wall displacement and (b) rotation.
Frequency ratio = 0.5 , Sand B (relative density = 60 % )
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.40  Estimation of (a) permanent wall displacement and (b) rotation.
Frequency ratio = 1. Sand B (relative density = 60%)
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.41  Estimation of (a) permanent wall displacement and (b) rotation.  
Frequency ratio = 1.5 , Sand  B (relative density = 60 % )
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.42  Estimation of (a) permanent wall displacement and (b) rotation. Frequency ratio = 0.5 , Sand C (relative density = 80 % ) Peak ground acceleration = 0.2 to 0.5g.
Fig 4.43  Estimation of (a) permanent wall displacement and (b) rotation.  
Frequency ratio = 1, Sand  C (relative density = 80 %)  
Peak ground acceleration = 0.2 to 0.5g.
Fig 4.44  Estimation of (a) permanent wall displacement and (b) rotation.
Frequency ratio = 1.5, Sand C (relative density = 80 % )
Peak ground acceleration = 0.2 to 0.5g.
4.5 Conclusions

The results of the analyses in Chapter 4 lead to the following conclusions.

1. The development of excess pore-water pressures has a dramatic effect on the response of the wall-soil system. After a few cycles of loading, the effective stress behind the wall may decrease substantially for loose sands during moderate shaking and for all sands during strong shaking.

2. Although excess pore-water pressures increase significantly and the effective stresses approach in many cases zero, liquefaction flow failure is not observed despite the high intensity of the excitation and the large number of cycles. This is attributed mainly to the presence of a rigid base beneath the wall. In actual waterfront structures, the foundation layer is most likely to be a medium-dense sand layer and, in some cases, a loose sand layer. Degradation of the foundation soil in addition to the pressures from the backfill soil during the earthquake has caused liquefaction flow failure of many waterfront retaining walls with movements of several meters. A typical example of this behavior was observed at Kobe during the Great Hansin Earthquake in 1995.

3. The predominant form of deformation occurred in the form of lateral seaward displacement. For the loose sand (A), subjected to 10 cycles of loading at \( f/f_1 = 0.5\), the computed permanent sliding at the base of wall is about 5 cm for \( \ddot{u}_b = 0.2g \) and 50 cm for \( \ddot{u}_b = 0.5g \).

4. Rotations of the wall were limited to values of about 5° or less.

5. Proper modeling of the interfaces at the back and at the base of the wall has a significant effect on the behavior of the soil-wall system.
6. The results of the numerical analyses for the highly idealized wall-soil system appear to be reasonable. However, further detailed analyses of actual case histories are required to confirm the computed prediction with actual earthquake damage observations.
References


