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by

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The Dissertation of Patrick Richard Wilson is approved, and it is acceptable in quality and form for publication on microfilm and electronically:

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Chair

University of California, San Diego

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ABSTRACT OF THE DISSERTATION

Large Scale Passive Force-Displacement and Dynamic Earth Pressure Experiments and Simulations

by

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During strong earthquakes, foundation structures such as bridge abutments and pile caps mobilize resistance due to passive earth pressure. Dynamic earth pressure can also increase the demand placed on retaining walls during earthquake excitation. Current uncertainty in the passive earth pressure load-displacement behavior and the evaluation of dynamic earth pressure during earthquake excitation motivates the large scale experimental and numerical investigation presented in this dissertation.

In the experimental investigations, a 2.15 m high, 5.6 m long, and 2.9 m wide dense, well-graded silty sand backfill is constructed behind a stiff vertical concrete wall inside a large soil container. First, the passive earth pressure load-displacement curve is recorded in two tests. From those tests, the peak passive resistance compares well with
the theoretical predictions. Using the test data, a calibrated finite element (FE) model is employed to produce additional load-displacement curves for a wider range of practical applications. A spring model is also developed for representing the passive resistance in dynamic simulations.

Next, dynamic earth pressure is measured in 26 events, as the soil container-wall-backfill configuration is subjected to shake table excitations. With peak input accelerations up to about 0.6 g, the earth pressure resultant force remains close to the static level. Small wall movements coupled with the high backfill stiffness and strength contribute to this favorable response. At higher input acceleration levels, the backfill shear strength is further mobilized, resulting in significant dynamic earth pressure increases. FE simulations support and demonstrate the experimental observations. Results show that accurate consideration of the retaining wall-backfill interaction may result in more realistic dynamic earth pressure predictions than the simplified analytical methods which are currently used in design.

The unique combination of laboratory and large scale test data reveals interesting features regarding backfill soil shear strength. For instance, although the tested backfill soil had only 7% silty fines, cohesion contributed significantly to the passive resistance and helped to limit dynamic earth pressure. The backfill friction angle in the plane strain test configuration was also found to be relatively high, contributing favorably to the response under both passive and dynamic earth pressure loading.
Chapter 1  Introduction and Background

Accurate prediction of lateral earth pressure is critical in the design of key infrastructure such as bridges, bulkheads, spillways, ports, culverts, roadways, basements, and subway systems. Considering the limitless possible combinations of backfill soil composition and adjacent supporting structures, significant uncertainty remains in the prediction of earth pressure, particularly with the added complexity of earthquake loading. Specifically, experimental insight is needed in the areas of: i) the passive earth pressure force-displacement relationship, and ii) dynamic earth pressure.

During strong earthquakes, foundation structures such as bridge abutments and pile caps mobilize resistance due to passive earth pressure. Theoretical peak passive pressure predictions such as the Coulomb (1776), Rankine (1857), and Log Spiral (Terzaghi et al. 1996) methods can contradict each other, varying by a factor of two or more (Cole and Rollins 2006). Backfill soil strength is often only roughly characterized (e.g. Caltrans 2004, AASHTO 2007, CBSC 2007), further contributing to the error in predicted passive pressure. Additionally, the above theoretical predictions do not address the load-displacement behavior, which may be needed in order to include the passive resistance in design applications.

Earthquake excitation may also increase loads on retaining walls due to dynamic earth pressure and inertial forces. In some strong earthquakes, retaining walls were heavily damaged (Fang et al. 2003, Gazetas et al. 2004). However, many retaining walls have performed well, even when earthquake loads were not considered in design (Seed and Whitman 1970, Gazetas et al. 2004, Al Atik and Sitar 2008). Additionally,
experimental results and case history analyses often contradict the theoretical dynamic earth pressure theories (Koseki et al. 1998, Gazetas et al. 2004, Nakamura 2006, Al Atik and Sitar 2008). As a result, there is not a strong consensus on retaining wall seismic design methodology.

The remainder of this chapter provides an introduction for the investigation of passive and dynamic earth pressures. Next, the scope and objectives of the performed research are presented. An outline of the dissertation concludes the chapter.

1.1 Passive Earth Pressure

In geotechnical engineering practice, earth pressure is categorized as at-rest, active, or passive (Lambe and Whitman 1969). “At-rest” earth pressure occurs when a stiff, massive or otherwise restricted retaining wall does not move relative to the supported soil (the backfill).

“Active” earth pressure can occur if a wall moves away from the adjacent backfill, decreasing pressure and inducing extensional lateral strain in the soil (Kramer 1996). If a wall moves sufficiently far away, the soil will fail in shear along a surface (the failure plane), and a wedge will slide down and towards the wall. In that limit state, minimum active earth pressure acts on the wall. As such, active earth pressure is lower than the at-rest condition (Figure 1.1).

In contrast, “passive” earth pressure occurs if a wall moves towards the adjacent backfill, increasing pressure and inducing compressive lateral strain (Kramer 1996). If the wall moves sufficiently toward the backfill, the soil will fail in shear along the failure plane, a wedge will slide up that slope, and maximum passive pressure acts on the wall.
As expected, passive pressure is higher than the at-rest condition, and often exceeds the active pressure by a significant factor (Figure 1.1). Mobilization of the maximum passive pressure also typically requires greater displacement than that of the minimum active earth pressure (Figure 1.1).

Passive earth pressure can provide stability in a range of applications. On the shallow side of a retaining wall (Figure 1.2), passive pressure resists sliding and overturning (Lambe and Whitman 1969). A bulkhead or sheet pile (Figure 1.3) may be anchored to a wall which mobilizes passive pressure (Lambe and Whitman 1969). Abutment backfills (Figure 1.4) provide passive resistance to earthquake-induced bridge deck displacements (Caltrans 2004, AASHTO 2007, Shamsabadi et al. 2007, Bozorgzadeh 2008, Lemnitzer et al. 2009). Acting on the cap of a pile group (Figure 1.5), passive pressure also increases the lateral stiffness and capacity (Gadre and Dobry 1998, Cole and Rollins 2006, Rollins and Cole 2006).

Conversely, passive earth pressure may exert detrimental loads. An integral abutment (Figure 1.6) may transmit forces to the bridge deck and foundation from passive pressure due to thermal expansion (Duncan and Mokwa 2001, Shah 2007). Due to lateral offset from a seismic fault rupture, buried tunnels and pipelines may also experience passive pressure loads (Wang and Yeh 1985, Lin et al. 2007, Abdoun et al. 2008).

1.1.1 Passive Earth Pressure Theories

Based on limit equilibrium and simplifying assumptions, the Coulomb, Rankine and Log Spiral methods (Lambe and Whitman 1968, Terzaghi et al. 1996) are commonly
used to predict passive earth pressure. These predictions require accurate values for parameters including the soil friction angle $\phi$, cohesion $c$, soil unit weight $\gamma$, the supported soil height $H$, wall-soil interface friction $\delta$, and other geometric properties of the wall and backfill as discussed below.

1.1.1.1 Rankine Theory

Rankine’s theory (1857) offers a simple prediction of the peak passive earth pressure. This theory applies to smooth ($\delta = 0$) vertical walls, and assumes that the failure plane (mentioned above) forms at the inclination where the Mohr-Coulomb failure criterion (Lambe and Whitman 1969) is met, at an angle of $45 + \phi/2$ from the vertical (Figure 1.7). According to Rankine theory (1857), the earth pressure is assumed to act at an angle $\beta$ from the horizontal, which is equal to the slope of the backfill. Based on these assumptions, the passive pressure resultant force $P_p$ can be estimated using the following equations:

$$P_p = \frac{1}{2} K_p \gamma H^2$$  \hspace{1cm} (1.1)

where $K_p$ is the coefficient of maximum passive earth pressure. The Rankine solution yields:

$$K_p = \cos \beta \frac{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}$$  \hspace{1cm} (1.2)

For cohesive backfills, Eq. (1.1) can be extended to:

$$P_p = \frac{1}{2} K_p \gamma H^2 + 2cH\sqrt{K_p}$$  \hspace{1cm} (1.3)
In most practical cases, the smooth wall ($\delta = 0$) assumption is not accurate, and as a result the Rankine passive pressure prediction tends to under-estimate the actual available resistance (Kramer 1996, Terzaghi et al. 1996). However, due to its simplicity it is often used.

1.1.1.2 Coulomb Theory

Similar to the Rankine theory, Coulomb (1776) assumed that the failure wedge formed along a planar surface, but extended the prediction to include $\delta$, and walls inclined at an angle $\theta$ from the vertical (Figure 1.8). By employing force equilibrium on trial failure wedges (Figure 1.8), and finding the surface that yields the critical (minimum) passive resistance, the Coulomb (1776) method provides the following equation:

$$K_p = \frac{\cos^2 (\phi + \theta)}{\cos^2 \theta \cos(\delta - \theta) \left[1 - \frac{\sin(\phi + \delta) \sin(\phi + \beta)}{\sqrt{\cos(\delta - \theta) \cos(\beta - \theta)}}\right]^2}$$

Equation (1.4) can be used in Eq. (1.1) to calculate the peak passive pressure resultant $P_p$.

The Coulomb Theory provides a convenient equation to account for $\delta$, and inclined walls. However in cases with large $\delta$, a curved portion of the failure surface has been observed after experiments (Cole and Rollins 2006). As a result, the Coulomb prediction tends to over-estimate the passive resistance in such cases (Kramer 1996, Duncan and Mokwa 2001, Cole and Rollins 2006).
1.1.1.3 Log Spiral Theory

For a rough wall ($\delta > 0$), due to the shear stresses on the wall-soil interface, the major principal axis may be shifted, resulting in a curved failure surface near the wall (Kramer 1996). This curvature can be accounted for by using a log spiral function to describe the failure surface shape near the structure (Figure 1.9), with a relatively straight section closer to the backfill surface, depending on the value of $\delta$ (Terzaghi et al. 1996).

By employing a graphical solution (Terzaghi et al. 1996), or a numerical analysis technique such as a spreadsheet (Duncan and Mokwa 2001), the force equilibrium can be solved in order to obtain the critical (minimum) peak passive resistance using such a curved wedge shape. Charts are also available which list $K_p$ values based on this method, which can in turn be used with Eq. (1.1) to compute the peak passive pressure resultant (e.g., Caquot and Kerisel 1948).

As discussed in further detail below, the log spiral method has been shown to provide a more accurate (lower) prediction than the Coulomb theory, particularly when $\delta > \phi/2$ (Kramer 1996, Duncan and Mokwa 2001). However, this method can be much more complex and time consuming to use, and is often avoided for these reasons.

1.1.2 Passive Pressure Mobilization with Displacement

While the Rankine, Coulomb and Log Spiral theories provide estimates of the peak load resistance, no information is provided about the associated force-displacement relationship. In many practical cases, this relationship plays a major role. Examples include bridge abutment deflection (Caltrans 2004, Bozorgzadeh 2007, Shamsabadi et al. ...

Compared with experimental data, hyperbolic models (Figure 1.10) have been shown to provide a good representation of the passive load-deflection behavior up to the peak resistance (Duncan and Mokwa 2001, Shamsabadi et al. 2007). Shamsabadi et al. (2007) proposed a model based on a secant stiffness $K$. Employing the parameters shown in Figure 1.10, the derived Hyperbolic Force-Displacement (HFD) model of Shamsabadi et al. (2007) is described by:

$$F(y) = \frac{F_{ult}(2Ky_{max} - F_{ult})y}{F_{ult}y_{max} + 2(Ky_{max} - F_{ult})y}$$

(1.5)

where $F$ is the resisting force, $y$ is the horizontal displacement, $F_{ult}$ is the maximum passive resistance, and $K$ is the secant stiffness at $F_{ult}/2$ (Figure 1.10). Shamsabadi et al. (2007) also offer a flowchart for an iterative procedure to calculate the force-displacement curve based on soil properties which can be measured in the lab.

Alternatively, Duncan and Mokwa (2001) employed a hyperbolic model (Figure 1.10) defined by the initial stiffness ($K_{max}$) according to the following equation:

$$F(y) = \frac{y}{\frac{1}{K_{max}} + R_f \frac{y}{F_{ult}}}$$

(1.6)

where $R_f$ is a failure ratio (refer to Duncan and Mokwa 2001 and Cole and Rollins 2006 for further description). Duncan and Mokwa (2001) also offer a spreadsheet (PYCAP) which can calculate the force-displacement curve based on soil properties and recommended parameters.
1.1.3 Experimental Studies

Duncan and Mokwa (2001) performed two passive pressure load tests on a 1.1 meter tall, and 1.9 meter long, and 0.9 meter wide anchor block, first with natural sandy silt (ML) and sandy clay (CL) backfill, and next with crusher run aggregate (GW-GM and SW-SM) backfill. From those tests, Duncan and Mokwa (2001) concluded that the Log Spiral predictions, together with the Brinch-Hansen (1966) correction for 3D effects (on the sides of the anchor block where additional shear resistance occurs) provided the best prediction for the peak passive resistance. The hyperbolic model of Eq. (1.6) was found to provide a good approximation of the load-deflection curve (Duncan and Mokwa 2001). Duncan and Mokwa (2001) also reported that as load was applied in those tests, the anchor block moved slightly upwards as it displaced into the adjacent backfill soil, resulting in low mobilized wall-soil friction $\delta_{mob}$ controlled by vertical equilibrium requirements (Figure 1.11).

Cole and Rollins (2006) performed cyclic load tests on a 1.1 meter tall, 5.2 meter long, and 3.1 meter wide pile cap, supported by 12 piles, using four different backfill soils. Tests were conducted with and without soil backfill in front of the pile cap in order to approximate the contribution of lateral resistance which was provided by the passive pressure (Rollins and Cole 2006). From those tests, Cole and Rollins (2006) concluded that the Log Spiral theory with 3D correction (Brinch-Hansen 1966) provided the best estimates of the peak passive pressure and that the hyperbolic model proposed by Duncan and Mokwa (2001) provided the best agreement with the load-deflection behavior for monotonic loadings.
Bozorgzadeh (2007) performed 5 tests on 1.5, 1.7 and 2.3 meter tall and 4.7 and 5.5 meter long bridge abutment walls with silty sand (SM) and clayey sand (SC) backfills. In some of those tests, Bozorgzadeh (2007) constructed the backfill on a cut slope, similar to the construction of a bridge abutment in practice, and found that this method can introduce a weak surface, reducing the peak passive resistance. Bozorgzadeh (2007) also observed significant post-peak strain softening in tests where the wall moved up with the adjacent backfill soil, and proposed a force-displacement model to account for this effect. Similar to the above experimental studies, the Log Spiral method provided good estimates of the peak passive resistance.

In another bridge abutment investigation, Lemnitzer et al. (2009) performed a test on a 1.7 meter tall and 4.6 meter long wall, with well graded silty sand (SW-SM) backfill, by applying lateral and diagonal (downward) loads to provide a purely horizontal wall translation. In that configuration, $\delta_{mob} = 14$ degrees at the instant of the peak passive resistance was determined from the vertical component of the diagonal actuator force (Stewart et al. 2007, Lemnitzer et al. 2009). Friction on the bottom of the wall was also measured by performing a test without backfill soil, in order to remove this portion from the estimated abutment passive resistance (Lemnitzer et al. 2009, Stewart et al. 2007). Lemnitzer et al. (2009) concluded that the peak resistance was well estimated by the Log Spiral method, and the shape of the hyperbolic curves of Eqs. (1.5) and (1.6) provided a good match with the recorded load-displacement behavior.
1.1.4 Numerical Simulations

Martin and Yan (1995) conducted numerical studies using the finite difference code, FLAC, for passive pressure behind bridge abutments. In that study, a soil model using the Mohr-Coulomb failure criterion, and an elastic-perfectly plastic constitutive relationship were employed. Martin and Yan (1995) concluded that the FLAC models provided reasonable results in terms of peak passive resistance when compared with the theoretical predictions.

More recently, a Hardening Soil (HS) model in the FE program Plaxis (2004), has been demonstrated to provide an improved representation of the passive pressure force-displacement relationship up to the peak resistance, compared with experimental results (Shamsabadi and Nordal 2006, Bozorgzadeh 2007). Similar to the Martin and Yan (1995), the HS model employs the Mohr-Coulomb failure criterion, but it improves on the constitutive end by using a hyperbolic stress-strain relationship (Plaxis 2004).

1.1.5 Passive Pressure Summary

Classical passive earth pressure theories allow for estimation of the peak resistance as a wall moves toward the backfill, but those predictions do not include a representation of the load-displacement relationship. Renewed interest in the development of passive pressure with displacement has motivated recent excellent experimental studies. Yet variability of backfill soil shear strength, loading configurations, and wall heights in the field continue to motivate further experimentation.
1.2 Dynamic Earth Pressure and the Retaining Wall-Backfill Response to Earthquake Excitation

When subjected to earthquake excitation, the dynamic backfill soil response, the inertia and motion of the wall, and the interaction between them can be extremely difficult to predict. Inertial forces and motions induced by earthquakes may increase the demand on structures by imposing larger forces compared to the static active or at-rest earth pressure conditions (Kramer 1996). Stability of a retaining wall may also be reduced due to the decrease in resisting passive earth pressure (Kramer 1996).

1.2.1 Dynamic Earth Pressure Theories

As an approach to aid in the design and analysis of retaining structures for seismic loads, two well-known theoretical methods have been in use for predicting dynamic earth pressure. The Mononobe-Okabe (Okabe 1926, Mononobe and Matsuo 1929) equations are the most widely used for so-called “yielding” walls, which are walls that can displace adequately to achieve minimum active or maximum passive conditions. An elastic solution proposed by Wood (1973), considers “non-yielding” walls, such as basement walls, which are assumed to be unable to move significantly to achieve the active or passive state. These two methods for predicting dynamic earth pressure are discussed in detail further below.

Additional analyses have been conducted on dynamic earth pressure, including a pseudo-dynamic approach (Steedman and Zeng, 1990), which can account for phase difference and amplification effects (Kramer 1996). Richard and Elms (1979), and Whitman and Liao (1985) also proposed methods which focus on the retaining wall
displacements. Kramer (1996) provides a good overview of these additional analyses which are much less commonly used than the above Mononobe-Okabe (Okabe 1926, Mononobe and Matsuo 1929) and Wood (1973) predictions.

### 1.2.1.1 The Mononobe-Okabe Equations

As early as the 1920’s, Okabe (1926) and Mononobe and Matsuo (1929) performed dynamic earth pressure experiments and extended the limit-equilibrium analysis of Coulomb, by including pseudo static inertial forces due to ground acceleration (Figure 1.12). Resulting from those studies, the Monobe-Okabe (M-O) method for predicting dynamic active and passive earth pressure remains the most widely known today. The resulting equation for the total active thrust is:

\[
P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)
\]  

(1.7)

with the dynamic active earth pressure coefficient:

\[
K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos \psi \cos^2 \theta \cos(\delta + \theta + \psi) \left[ 1 + \frac{\sin(\delta + \phi) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)} \right]^2}
\]  

(1.8)

The resulting equation for the total passive thrust is:

\[
P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)
\]  

(1.9)

with the dynamic passive earth pressure coefficient:

\[
K_{PE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos \psi \cos^2 \theta \cos(\delta + \theta + \psi) \left[ 1 + \frac{\sin(\delta + \phi) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)} \right]^2}
\]  

(1.10)
where \( \psi = \tan^{-1}\left[\frac{k_h}{1 - k_v}\right] \) and \( k_v \) and \( k_h \) are the backfill vertical and horizontal accelerations, respectively (Figure 1.12).

Based on the M-O method, the inclination of the critical failure plane also depends on the backfill acceleration. For active pressure, the inclination of that plane becomes flatter for larger input accelerations (NCHRP 2008). As a result, \( K_{AE} \) shares a nonlinear relationship with the input acceleration, pointing upwards more steeply at the higher input acceleration levels (NCHRP 2008).

The above M-O method was originally developed for dry cohesionless soils, but since then, the effect of cohesion has also been added (Saran and Prakash 1968, Richards and Shi 1994, NCHRP 2008, and Shukla et al. 2009). From those studies, including the effect of cohesion in the force-equilibrium can greatly reduce or even eliminate the predicted dynamic active earth pressure up to significant levels of earthquake excitation (Figure 1.13).

### 1.2.1.2 The Wood (1973) Elastic Solution for a Rigid Wall

For the case of rigid so-called “non-yielding” walls, which do not meet the assumptions required for application of the M-O method, an extensive study was performed by Wood (1973) to consider a homogeneous elastic soil bounded by rigid walls. The Wood (1973) solution is very simple to apply using the following equation for the dynamic component of earth thrust \( \Delta P_{eq} \):

\[
\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_p
\] (1.11)
where $a_h$ is the amplitude of the base acceleration, $g$ is the gravitational acceleration constant, and $F_p$ is a parameter ranging from about 0.9 to 1.1 depending on Poisson’s ratio $\nu$, $H$, and the distance between the rigid walls which bound the elastic medium (Kramer 1996). In contrast with the Mononobe-Okabe method prediction, Eq. (1.11) produces a linear relationship between the dynamic earth pressure resultant $\Delta P_{eq}$ and the input acceleration $a_h$.

### 1.2.2 Observed Field Performance of Retaining Walls during Earthquakes

While damage to retaining walls has been observed after some earthquakes, it has often involved a weak (for instance liquefiable) underlying layer (Gazetas et al. 2004, Shirato et al. 2006, Al Atik and Sitar 2008). In the absence of such a weak layer, many retaining structures have performed well, even in cases where the seismic load was not explicitly a design consideration (Seed and Whitman 1970, Lew et al. 1995, Gazetas et al. 2004, Al Atik and Sitar 2008).

However in some cases, retaining walls supporting non-saturated backfills have failed or been damaged. For instance after the 1995 Kobe earthquake, Gazetas et al. (2004) reported that several masonry and unreinforced gravity walls were heavily damaged, while reinforced concrete walls experienced relatively little harm. After the 1971 San Fernando earthquake, Clough and Fragaszy (1977) found that U-shaped channel floodway structures designed only for static Rankine (1857) active pressures, performed well with peak excitation up to about 0.5 g, but sustained damage for larger acceleration levels. After the 1999 Chi-Chi earthquake, Fang et al. (2003) also reported on the failure of three gravity walls.
1.2.3 Experimental Studies

Over the years, a wide range of shake table experiments have been performed in order to measure dynamic earth pressure and investigate the retaining wall response (e.g., Mononobe and Matsuo 1929, Sherif et al. 1982, Bolton and Steedman 1982, Steedman and Zeng 1991, Stadler 1996). Many of these tests were performed on a very small scale, and the results have shown varying levels of agreement with the theoretical predictions (Al Atik and Sitar 2008).

Koseki et al. (1998) suggested modifications to the M-O method based on experimental results from shake table excitation and tilting tests performed with a 0.5 meter tall wall. These modifications consider the possibility of a weakened band of backfill material (strain localization) existing along a previously formed active failure wedge. The experimental and analytical results of Koseki et al. (1998) suggest that the plane formed by that initial wedge might control the consecutive mobilization of earth pressure until shaking levels are strong enough to form a new, larger wedge in the stronger surrounding backfill.

Deewoolkar et al. (2001) performed centrifuge dynamic excitation tests with fixed-base cantilever walls supporting saturated, liquefiable, cohesionless backfills. From those experiments, Deewoolkar et al. (2001) concluded that excess pore pressure generation contributed significantly to seismic lateral earth pressure in the saturated backfill. Deewoolkar et al. (2001) also found that the maximum dynamic thrust was proportional to the input base acceleration.
Ling et al. (2005) conducted shake table tests on 2.8 meter high modular-block geosynthetic-reinforced soil walls. From those large scale tests, Ling et al. (2005) found that existing design methods underestimate the capacity of such flexible systems.

Nakumara (2006) and Al Atik and Sitar (2008) recently conducted separate shake table tests using centrifuge facilities, and both separately concluded that the measured earth pressure during shaking was lower than the M-O method predictions. Nakamura (2006) also found that the inertial force was not always transmitted to the wall and backfill simultaneously.

1.2.4 Numerical Studies

Simulations of the dynamic wall-backfill interaction using numerical models have provided additional valuable insights. Alampalli and Elgamal (1990) developed a numerical model based on the compatibility between mode shapes of the wall and the adjacent backfill soil. Using a model consisting of flexible cantilever wall supporting a semi-infinite uniform visco-elastic layer, Veletsos and Younan (1997) concluded that the magnitude and distribution of wall displacement and pressure can be quite sensitive to the flexibilities of the wall and its base. Richards et al. (1999) presented a kinematic model with springs representing the soil and found that the point of action of the dynamic earth pressure resultant varies with different types of wall movement.

Gazetas et al. (2004) performed simulations of L-shaped walls, prestressed-anchored pile walls, and reinforced soil walls, employing both linear and non-linear soil models. Using those models, Gazetas et al. (2004) showed that including realistic effects such as the wall flexibility, foundation soil deformability, material soil yielding and soil-
wall separation and sliding tends to reduce the effects of dynamic excitations on those walls. Gazetas et al. (2004) also used an FE model to simulate a case history in which a retaining wall performed well during an actual earthquake.

Psarropoulos et al. (2005) performed FE simulations of rigid and flexible non-sliding walls and found that the rigid wall case converged to the Wood (1973) analytical solution, and the flexible wall matched the Veletsos and Younan (1997) solution. After achieving these results, Psarropoulos et al. (2005) extended their model to find that adding inhomogeneous soil layers further complicated the response, suggesting that simply adding a rocking spring to the model base may not accurately account for the wave propagation effects from the underlying foundation layer.

Jung and Bobet (2008) added a translational spring to the base of a bending and rotating wall model supporting elastic soil elements, and found that the wall rotational, bending, and translational flexibilities significantly affected the magnitude and distribution of the dynamic pressure. Specifically, Jung and Bobet (2008) found that the dynamic earth pressure behind a rigid wall with a stiff foundation is larger than that for a flexible wall with a soft foundation.

1.2.5 Dynamic Earth Pressure Summary

The limit equilibrium Mononobe-Okabe (1926, 1929) equations for yielding walls and the elastic soil-rigid wall solution of Wood (1973) provide methods for predicting the dynamic earth pressure. However, the observed field retaining wall seismic performance, shake table tests, and numerical investigations have not consistently agreed with those predictions. Rather, they have demonstrated that the complexity of the dynamic retaining
wall-backfill response may require more detailed analysis and further experimentation in order to aid in refining design standards.

1.3 Objectives and Scope

In bridge seismic design, accurate modeling of the passive earth pressure force-displacement resistance at the abutments and pile caps leads to safer and more economic design. However the few available bilinear design models (e.g., Caltrans 2004, AASHTO 2007) and limited data from large scale tests, do not account for the variation in nonlinear passive force-displacement response that occurs in the field. In addition, the existing test data (from experiments performed on static backfill) and design models do not account for potential inertial effects on the structure and backfill which may influence the actual available resistance during shaking.

Significant uncertainty also remains in the evaluation of dynamic earth pressure for retaining wall seismic design. The performance of many walls during earthquakes is not consistent with the predicted response based on the existing theory. Furthermore, very few well-documented case histories and large scale test data are available for validation of improvements to prediction methods.

An excellent opportunity to provide new experimental insight into passive and dynamic earth pressure was provided as a component of a collaborative NEES investigation into the seismic performance of highway bridges (Saiidi 2004). As the primary component of that study, Saiidi (2004) performed ¼ scale 4 span bridge tests on 3 movable shake tables (Figures 1.14 and 1.15) at the University of Nevada at Reno (UNR). Within this collaborative framework, an abutment investigation was also
designated to be performed at UCSD, using a large soil container on the outdoor shake table at the Englekirk Structural Engineering Center (ESEC). The work presented in this dissertation was conceived to maximize this provided abutment investigation research opportunity, by employing a single test configuration, along with numerical models, in order to address the following objectives:

1) Investigate the passive earth pressure force-displacement relationship. First, perform experiments to record this relationship behind a model abutment wall, full scale in height, with dense sand backfill. Next, test our ability to predict the peak passive resistance and the force-displacement relationship using soil strength and stiffness parameters determined from laboratory tests. Finally, use calibrated finite element (FE) model simulations to compare and provide curves for a range of backfill soil types and wall heights.

2) Develop a model which can represent the backfill passive resistance at the abutments using the provided experimental and numerical curves in dynamic bridge simulations.

3) Record dynamic earth pressure on the vertical wall by subjecting the test configuration to shake table excitation. Provide new insight into the recorded levels of earth pressure during shaking, including any interesting effects observed through the experimental data. Test our ability to predict the dynamic earth pressure using existing theory.

4) Develop FE models which can provide reasonable estimates of the recorded dynamic earth pressure, and capture key aspects of the wall-backfill response.
5) Record dynamic passive earth pressure by first mobilizing resistance behind the wall and then subjecting the test configuration to shake table excitation. Draw attention to the potential impact of this often overlooked dynamic passive pressure “backfill inertia effect” on the available force-displacement resistance.

6) Develop a method for including the above backfill inertial effect in dynamic bridge simulations. Demonstrate using FE models.

7) Provide new insight into the overall dense sand backfill performance based on this unprecedented collective set of laboratory and large scale passive and dynamic earth pressure experimental data.

1.4 Dissertation Outline

Chapters 2 and 4 highlight the main experimental contributions regarding passive and dynamic earth pressure. Chapter 2 covers a pair of large scale passive earth pressure load-displacement tests and associated numerical simulations. Chapter 4 presents 10 large scale, shake table, dynamic earth pressure experiments which are of relevance to: i) consideration of the dynamic interaction between so-called “yielding” and “non-yielding” walls and the supported backfill, and ii) the assessment of seismic forces on such walls. Both Chapter 2 and Chapter 4 are written as stand-alone articles, which contain some repeated information about the background, testing configuration and backfill soil. In that format, Chapters 2 and 4 can be read and interpreted independently from the rest of the dissertation. The remainder of dissertation outline is described below.

Table 1.1 presents a list of the experiments performed as a part of this study including the names used to reference them in the dissertation. In all, 28 large scale
experiments are discussed. These tests include (Table 1.1): i) two passive pressure load-displacement tests (Test 1 and Test 2), ii) 10 “at-rest condition” dynamic earth pressure tests, iii) 7 “passive condition” dynamic earth pressure tests, and iv) 8 “passive failure condition” dynamic earth pressure tests. In addition to the experimental investigations, several different numerical simulations were also conducted as discussed below.

As mentioned above, Chapter 2 presents the passive earth pressure load-displacement tests (Test 1, and Test 2) performed on a 1.7 meter tall wall section with dense, well graded, silty sand as the backfill soil. Results are analyzed, and a FE model is developed using Plaxis (2004) in order to simulate the experimental results. Building on the calibrated FE model from Chapter 2, additional passive earth pressure load-displacement simulations are conducted in Chapter 3, in order to consider a wider range of wall heights and backfill soil types. Models are provided to represent the simulated curves in practical applications.

Chapters 4, 5, and 6 describe the “at-rest”, “passive”, and “passive failure condition” shake table excitation tests (Table 1.1), respectively. The above test names (Table 1.1) describe the initial backfill earth pressure condition for each series. In those tests, the soil container-test wall-backfill configuration was subjected to scaled harmonic (HM) and earthquake record (EM) motions (Table 1.1) using the large outdoor shake table at the Englekirk Structural Engineering Center (ESEC). FE model dynamic earth pressure simulations are also presented in Chapter 7, which further illustrate and investigate some of the key aspects of the experimentally observed response.

As a practical application of the experimental and numerical findings, Chapter 8 discusses bridge modeling (in the longitudinal direction) including the effects of earth
pressure at the abutments. A nonlinear cyclic spring model is presented for use in the FE code OpenSEES (Mazzoni et al. 2006, Dryden 2009), which can represent the passive earth pressure load-displacement resistance at the abutments. A spring-mass system is also introduced in order to include the effects of inertial backfill and wall forces. Use of these models is demonstrated through OpenSees dynamic FE bridge model simulations.

Chapter 9 summarizes and concludes the dissertation. Appendices A through E provide additional valuable data and information about the conducted experimental program.

1.5 References


Abutment backwalls, UCLA – SGEL, Report 2007/02, Department of Civil Engineering, University of California, Los Angeles, Los Angeles, CA.


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Figure 1.1: Comparison of wall movements required to achieve active and passive earth pressures (from NCHRP 1991)
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Figure 1.6: Passive earth pressure acting on an integral abutment
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Mohr-Coulomb failure criterion

Failure stress acts on theoretical failure plane at an angle of $45 + \phi/2$ from the vertical plane

(horizontal stress acting on vertical plane)

Figure 1.8: Forces acting on a triangular Coulomb (passive) trial wedge
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Chapter 2  Large Scale Passive Earth Pressure Load-Displacement Tests and Numerical Simulation

2.1  Abstract

Passive earth pressure is recorded in two different tests, using a 6.7 meter long, 2.9 meter wide soil container. In these tests, sand with 7% silt content is densely compacted behind a moveable test wall to a supported height of 1.68 meters (5.5 ft). Lateral load is applied to the vertical reinforced concrete wall section, which displaces freely along with the adjacent backfill in the horizontal and vertical directions. The recorded passive resistance is found to increase until a peak is reached at a horizontal displacement of 2.7% - 3% of the supported backfill height, decreasing thereafter to a residual level. In this test configuration, a triangular failure wedge shape is observed, due to the low mobilized wall-soil friction. Backfill strength parameters are estimated based on this observed failure mechanism. From these estimates, along with triaxial and direct shear test data, theoretical predictions are compared with the measured passive resistance. Using the test data, a calibrated finite element (FE) model is employed to produce additional load-displacement curves for a wider range of practical applications (e.g., potential bridge deck displacement during a strong earthquake). Hyperbolic model approximations of the load-displacement curves are also provided.
2.2 Introduction and Background

Passive earth pressure plays an important role in a wide range of geotechnical and structural engineering problems. Acting on the shallow side of cantilever and gravity retaining walls, it provides a stabilizing force. Walls with an attached anchor block also mobilize additional passive resistance to limit movement and increase stability (Duncan and Mokwa 2001). In the case of shallow foundations, the passive pressure mechanism contributes significantly to the development of bearing capacity (Lambe and Whitman 1969). During an earthquake, bridge abutments may provide passive resistance to excessive displacement of the deck (Caltrans 2004, Bozorgzadeh 2007, Shamsabadi et al. 2007, Stewart et al. 2007, Lemnitzer et al. 2009). Acting on the cap of a pile group, passive pressure also increases lateral stiffness and capacity (Gadre and Dobry 1998, Cole and Rollins 2006, Rollins and Cole 2006).

Conversely, passive earth pressure may exert detrimental loads. A bridge abutment may transmit forces to the deck and foundation from passive pressure due to thermal expansion (Duncan and Mokwa 2001, Shah 2007). Due to lateral offset from a seismic fault rupture, buried tunnels and pipelines may also experience passive pressure loads (Wang and Yeh 1985, Lin et al. 2007, Abdoun et al. 2008).

2.2.1 Passive Earth Pressure Theories

Based on limit equilibrium and simplifying assumptions, the Coulomb, Rankine and Log Spiral methods (Lambe and Whitman 1968, Terzaghi et al. 1996) are commonly used to predict earth pressure. The Rankine theory assumes no friction between the structure and the backfill, while both the Coulomb and the Log-Spiral methods include
the wall-soil interface friction angle $\delta$. The Rankine and Coulomb theories adopt a triangular failure wedge shape, while Log Spiral accounts for the possibility of curvature near the structure with a relatively straight section closer to the backfill surface, depending on the value of $\delta$ (Terzaghi et al. 1996). According to experimental studies, those theories have been shown to provide varying levels of accuracy for different soil-structure boundary conditions as further discussed below.

2.2.2 Recent Experimental Work

Some of the recent large scale experiments which included a passive pressure measurement were performed by Duncan and Mokwa (2001), Fang et al. (2002), Cole and Rollins (2006), Bozorgzadeh (2007), and Lemnitzer et al. (2009). From those studies, conclusions relevant to the current investigation are summarized as follows:

1) The Log-Spiral theory was found to provide good estimates of passive pressure over a wide range of $\delta$ (Duncan and Mokwa 2001, Fang et al. 2002, Cole and Rollins 2006, Bozorgzadeh 2007, Lemnitzer et al. 2009).

2) The Coulomb and Log Spiral theory predictions were very similar in the case of small $\delta$ (Duncan and Mokwa 2001, Fang et al. 2002, Bozorgzadeh 2007), both resulting in an essentially linear failure surface. However for large $\delta$, the Coulomb prediction was found to significantly exceed that of the Log Spiral (and the measured passive resistance), due to the linear failure surface assumption (Cole and Rollins 2006).

3) As expected, the Rankine solution significantly underestimated the measured failure earth pressure for tests with large $\delta$ (ranging from 14 to 30 degrees) in the tests by Cole and Rollins (2006), and Lemnitzer et al. (2009). When compared to results from
experiments with smaller $\delta$ of 4 to 7 degrees (Duncan and Mokwa 2001, and Bozorgzadeh 2007), the Rankine solution continued to under-predict the failure earth pressure, but to a lesser degree.

The above mentioned experiments have provided valuable insights into the mobilization of passive earth pressure within a range of practical situations. Nevertheless, large-scale experiments remain few in comparison to the wide variability in loading conditions, and backfill soil shear stiffness and strength.

2.2.3 Sensitivity to Backfill Strength Parameters

Laboratory experiments on granular materials are often performed under triaxial shearing conditions (Lambe and Whitman 1969). However, the soil stress state behind long retaining walls is closer to plane-strain, dictating a different level of shear strength (Lambe and Whitman 1969, Terzaghi et al. 1996, Alshibli and Sture 2000). Confining stress level, degree of compaction, water content, and aging are also known to considerably affect the soil in-situ shear strength (Holtz and Kovacs 1981, Lambe and Whitman 1969, Schanz and Vermeer 1996, Terzaghi et al. 1996, Hanna 2001, and Alshibli et al. 2003).

Furthermore, engineered sandy backfills are often implicitly characterized in terms of frictional resistance alone (CBSC 2007), yet laboratory tests on such soils compacted to the field conditions can yield a significant cohesion intercept (EMI 2005). In such cases, it may be conservative (possibly resulting in costly over-design) to ignore the cohesion, or the possible increase in shear strength due to plane strain loading.
However, when the passive earth pressure imposes a detrimental load, the opposite can be true.

The mobilized wall-soil interface friction angle $\delta_{mob}$ also depends on several factors including type of backfill soil, as well as surface roughness of the structure, its weight (mainly for light structures such as anchor blocks), and direction of movement (Duncan and Mokwa 2001). Predicting $\delta_{mob}$ at the instant of peak passive resistance ($\delta_{mob} \leq \delta_{max}$, Duncan and Mokwa 2001) can be difficult, and the Coulomb and Log Spiral predictions are sensitive to this input parameter.

2.2.4 Backfill Stiffness and the Force-Displacement Relationship

While the Rankine, Coulomb and Log Spiral theories provide estimates of the peak resistance, no information is provided about the associated force-displacement relationship. In many practical cases, this relationship plays a major role. Examples include bridge abutment deflection (Caltrans 2004, Bozorgzadeh 2007, Shamsabadi et al. 2007, Lemnitzer et al. 2009), and lateral resistance of shallow foundations and pile caps (Gadre and Dobry 1998, Cole and Rollins 2006, Rollins and Cole 2006).

2.2.5 Conducted Studies

In order to provide additional insights, the passive earth pressure force-displacement relationship was documented in two large scale experiments and in finite element (FE) simulations. In the following sections, the experimental configuration is described, followed by presentation and analysis of the test results. Next, a FE model is calibrated based on these results and utilized to generate additional force-displacement
curves. For practical applications, hyperbolic model approximations are provided to represent these force-displacement relationships.

2.3 Passive Pressure Experiments

Two passive earth pressure tests (Test 1 and Test 2) were conducted outdoors in a large soil container (Figures 2.1 and 2.2) at the University of California, San Diego (UCSD). One test (Test 2) was conducted with the backfill close to its placement water content, and the other was performed in a drier condition (20 days after construction, at an average high and low daily temperature of about 31 and 18 degrees Celsius, respectively). The test wall supported a backfill with a height $h$ of 1.68 meters (5.5 feet, Figure 2.1) and a width of 2.87 meters (the width dimension is not shown in the figure).

2.3.1 Test Configuration

In addition to the sections below, an extensive description of the testing configuration is included in Appendix A.

As shown in Figure 2.2a, an available large laminar box was restrained from translation by two steel towers (rigidly anchored to the ground). To help achieve a rigid box configuration, the laminar sections of the container were supported on wood blocks (rather than using the usual roller bearing system). Figure 2.1 shows the inside dimensions of this soil container. To allow unrestricted development of the passive failure surface, the backfill occupied the majority of the container length and extended deeper than the base of the test wall (Figure 2.1). A rectangular hollow wooden box was placed beneath the wall to support this extended backfill depth (Figure 2.1).
Thin plywood lined the sides of the container to provide an even surface along all lateral boundaries. Three layers of smooth low friction plastic (Visqueen) sheeting covered the plywood to minimize friction between the soil and the sides of the container. In a study by Fang et al. (2004), a similar configuration yielded a friction angle of 11.5 to 14 degrees (one thin plastic sheet surrounded by two thick sheets). On this basis, it is estimated that compared to the measured passive force, contribution of friction between the soil and the container sides is relatively small (less than 2%, Appendix C).

2.3.1.1 Test Wall

The heavily reinforced (essentially rigid) concrete test wall was 0.2 m (8 in) thick, 2.74 m (9 ft) wide and 2.13 m (7 ft) tall (Figure 2.2b). Small gaps between the side edges of the wall and container were filled with soft foam (approximately 6.5 cm on each side). For stability during construction and testing, the wall was suspended from a beam which rested on rollers (Figures 2.2b and e) supported on the longer sides of the soil box (for clarity, Figure 2.1 does not show the supporting beam above the wall). A small gap between the bottom of the test wall and the box beneath it (Figure 2.1) was filled with weak, soft foam to prevent soil from spilling through. With this configuration, there was minimal shear resistance between the wall and the box below, allowing isolation of the resistance provided by the backfill. The combined mass of the test wall and supporting beam was about 4500 kg (weight of 15.4 kN per meter of width), equivalent to that of a 0.3 m or 1 ft thick wall of similar width and height.
2.3.1.2 Loading System

Load was applied to the test wall by 4 hydraulic jacks (Figures 2.1 and 2e). These jacks were mounted on concrete filled steel posts (Figure 2.2e) which reacted against the left side of the container (Figure 2.2a). The jacks were connected to a hydraulic pump through a manifold, allowing independent control in order to help maintain a uniform field of lateral wall displacement. Hollow plunger jacks were used instead of long stroke actuators so as to provide maximum space for development of the full passive failure wedge within the soil container. Each jack was fitted with a Teflon pad acting on a greased smooth steel plate (attached to the wall) in order to allow for potential upward wall movements.

2.3.1.3 Backfill Soil

The soil was selected to comply with Caltrans (2006) structure backfill requirements, and purchased from a leading national supplier of construction aggregates (Vulcan Materials, www.vulcanmaterials.com). For both tests, the backfill material (Figure 2.3) consisted of sand with non-plastic silt (about 7%) and fine gravel (less than 7%). The gravel particles were noted to be more angular than round. Sand Equivalent test value was 74 (ASTM D2419-02), and specific gravity was 2.67. The modified proctor (ASTM D1557) dry unit weight was 20.26 kN/m³, and the optimum moisture content was 8.5%.

Results from laboratory direct shear and triaxial compression tests on samples remolded as closely as possible to the field backfill placement conditions are shown in Figures 2.4 through 2.6. Direct shear tests were performed (Earth Mechanics Inc.,
Fountain Valley, CA) on new samples to failure at 5 different normal stresses. The results (Figures 2.4 and 2.5) suggest peak and residual friction angles of $\phi = 48$, and $\phi_r = 35$ degrees, with cohesion intercepts $c = 14$ kPa, and $c_r = 8$ kPa, respectively.

In addition, drained triaxial tests were performed (Leighton and Associates, Irvine, CA) by subjecting three separate specimens to respective cell pressures of 37, 72 and 144 kPa and then shearing to failure by increasing the deviator stress (Figure 2.6). Compared with the direct shear tests (Figures 2.4 and 2.5), the triaxial results suggest a lower peak $\phi = 44$ degrees, slightly higher residual $\phi_r = 36$ degrees, the same $c = 14$ kPa, and slightly lower $c_r = 6$ kPa. From Figure 2.6, the secant modulus to 50% of the peak stress ($E_{50}$) may be estimated to be 16400 kN/m$^2$, 18700 kN/m$^2$, and 48200 kN/m$^2$ for the 37 kPa, 72 kPa, and 144 kPa cell pressure levels, respectively.

The $\phi$ values mentioned above are near the upper end of the range documented from an investigation of actual structural backfills (Earth Mechanics 2005), with $\phi$ of 39 - 49 degrees in direct shear tests and $\phi$ of 24 - 43 degrees in triaxial tests (for sands and silty sands). The above $c$ values are closer to the lower end of the range of $c = 6 - 68$ kPa reported by Earth Mechanics (2005). Estimated from the triaxial stress strain curves, the range of $E_{50}$ from Earth Mechanics (2005) encompasses the values listed above.

2.3.1.4 Backfill Construction

The backfill (Figures 2.1 and 2.2d) was placed in small lifts (each about 0.2 m thick), and compacted using gasoline powered tamper rammers (Figure 2.2c). Compaction verification was done by nuclear gauge measurements. It is noted that using this technique, Noorany (1990) reported that variation in relative compaction of the order
of 10% may still occur. According to the measurements, average compacted dry unit weight $\gamma_d$ was about 19.30 kN/m$^3$ and 19.41 kN/m$^3$ (95.3% and 95.8% of the modified proctor) with average water content at placement of 9.4% and 8.7% for Test 1 and Test 2, respectively.

As mentioned above, time between construction and testing was 20 days for Test 1 and 3 days for Test 2. Additional time was needed for Test 1 in order to hand-drill holes in the backfill for filling with soft breakable foam cores (Figure 2.1) and to configure, prepare and test all instrumentation. Drainage and natural water evaporation from the backfill between construction and testing thus led to a less moist backfill for Test 1.

### 2.3.1.5 Instrumentation

Instrumentation for the passive pressure tests (Figure 2.1) included load cells, displacement transducers, pressure transducers and breakable foam cores. A load cell was installed behind each of the 4 hydraulic jacks (Figures 2.1 and 2.2e) to measure the applied load. String potentiometers recorded the total wall displacement. Linear potentiometers measured the vertical displacement of the wall and heave of the backfill (with the ends resting on small plates on the backfill surface, Figures 2.1 and 2.2d). Six transducers between the wall and backfill recorded the distribution of pressure along the height and width of the wall. Breakable foam cores in the backfill (Test 1 only) assisted in documenting the shape of the low $\delta$ passive failure plane configuration (as discussed below).
2.3.2 Test Procedure

In each test, the hydraulic jacks were used to push the test wall into the backfill and record the load-displacement relationship. In both tests, the wall was permitted to move upwards with the adjacent backfill soil, similar to the experimental anchor block configuration of Duncan and Mokwa (2001). After the first test, the backfill was removed down to the bottom 0.15 meters (6 inches), replaced and re-compacted for the second experiment. During the load application phase, displacements were monitored and the jacks were individually controlled to maintain a vertical wall orientation (within a range of about 1 degree of rotation). Equal jack pressures along the height consistently resulted in higher displacements near the wall top, confirming that the resultant passive force was located below the wall mid-height.

2.3.3 Test Results

2.3.3.1 Load-Deflection Behavior and Peak Passive Resistance

In both experiments, the passive earth resistance increased up to a peak and then decreased to a residual level. Figure 2.7 shows the backbone load (from the load cells) versus horizontal wall displacement (determined from string potentiometer and linear potentiometer readings) in the two tests. The peak loads measured in the two tests are clearly different, but the residual resistance is virtually the same (Figure 2.7). With the mobilized backfill width of 2.87 meters and height \( h \) of 1.68 meters (Figure 2.1), the peak load measured in Test 1 was 1105 kN (251 kips) at a displacement of 46 mm (1.8 in), or about 2.7% of the supported backfill height. Peak load measured in Test 2 was 936 kN (213 kips) at a 51 mm (2 in) displacement, or about 3.0% of the supported height. After
the peak, the resistance decreased until a residual level of about 608 kN (55% and 65% of
the peak from Test 1 and Test 2, respectively) was reached in both tests, at a lateral
displacement of about 8% of the supported backfill height.

2.3.3.2 Failure Surface (Test 1)

During Test 1, the foam cores shown in Figure 2.1 sheared-off along the localized
failure surface (Figure 2.8) that developed as the test wall was displaced into the backfill.
On the left side of Figure 2.8, the solid black line represents the test wall. The right
corner of the black bordered triangle (Figure 2.8) is the location of the surface scarp
(Figure 2.2f) at the far edge of the observed failure wedge. From Figure 2.8, the Test 1
failure wedge shape appeared to be essentially triangular as expected for this low $\delta_{mob}$
scenario.

2.3.3.3 Failure Wedge Geometry

Figures 2.9 and 2.10 show profiles of the deformed backfill shape (Test 1 and
Test 2) with the test wall displaced 76.2 millimeters (3 in) into the backfill (after peak
resistance was observed). Readings from the surface linear potentiometers (Figure 2.1)
were used to map the vertical ground displacements. These readings (Appendix C)
indicated that the failure wedge was essentially fully developed around the instant of
peak passive resistance (slightly after, in terms of lateral displacement of the wall).
Beyond this peak resistance, vertical ground displacement continued to increase, only at
the potentiometer locations within the failure wedge (Figure 2.9 and Figure 2.10). Based
on Figure 2.8, a linear representation of the slip surface is also shown in Figures 2.9 and
2.10.
2.3.4 Comparison of the Deformation Pattern from the Two Tests

The recorded data presented above (Figures 2.7, 2.9 and 2.10) show that the backfill responded differently in the two tests. This difference may be attributed mainly to the time between construction and testing (20 days for Test 1 vs. 3 days for Test 2), resulting in a less moist (drier) backfill in Test 1. Manifestations of the difference in response include:

1) Test 1 load-deflection curve (Figure 2.7) shows a sharp peak followed by a rapid degradation of passive resistance. Test 2 shows a more rounded peak with a gradual reduction in passive resistance.

2) Test 1 yielded an 18% higher passive resistance at failure than Test 2 (Figure 2.7).

3) Peak resistance was measured at a lower displacement level in Test 1 than in Test 2 (Figure 2.7).

4) Backfill heave profiles show that the failure wedge in Test 1 assumed a somewhat rigid-body mode, with vertical displacement nearly the same along the wedge surface and a distinct drop thereafter (Figure 2.9). In Test 2, the backfill appears to have responded in a more diffused failure pattern, with larger vertical displacement near the wall and a noticeable slope in the backfill surface deformation (Figure 2.10).

5) Referring to Figures 2.9 and 2.10, the distances $d$ from the wall to the observed surface scarp (indicating the end of the mobilized failure wedge) were between 4.7 and 4.9 meters in Test 1 and between 4.0 and 4.15 meters in Test 2. These surface lengths correspond to an angle $\alpha$ of 70 to 71 degrees for Test 1 and 67 to 68 degrees for Test 2 (Figures 2.9 and 2.10).
2.3.5 Estimation of Mobilized $\delta$, $\phi$, and $c$ Based on Observed Failure Mechanism

2.3.5.1 Mobilized Wall-Soil Interface Friction at Failure

As the wall moved upward with the adjacent soil, the magnitude of the vertical component of the passive resistance was ultimately equal to the test wall weight. In that vertical uplift configuration, $\delta_{mob}$ was governed by the requirements of vertical equilibrium (Duncan and Mokwa 2001). By imposing those requirements, $\delta_{mob}$ was estimated to be 2.3 and 2.7 degrees for Test 1 and Test 2, respectively. For tests on an anchor block of much larger mass, Duncan and Mokwa (2001) reported $\delta_{mob}$ values of 4 - 6 degrees.

2.3.5.2 Peak Friction Angle

For low $\delta_{mob}$ scenarios, earth pressure theories (e.g., Rankine, Coulomb or Log Spiral) indicate that the passive failure wedge forms essentially on a plane at an angle of $45 + \phi/2$ degrees from the vertical wall (Lambe and Whitman 1969, Terzaghi et al. 1996). The basis for this assumption is that the theoretical failure plane is the plane on which the most unfavorable combination of normal and shear stress develops, and the Mohr-Coulomb failure criterion is met (Lambe and Whitman 1969). By assuming that the experimentally observed failure plane matches the theoretical failure plane for these low $\delta_{mob}$ tests, an estimate can be made of the in-situ plane strain peak friction angles $\phi$. Based on this approach, the above ranges of angle $\alpha$, correspond to $\phi$ of about 50-52 degrees for Test 1 and 44-46 degrees for Test 2.
2.3.5.3 Cohesion

Based on force equilibrium (Figure 2.11) for the mobilized failure wedge (Shamsabadi et al. 2007) and the estimated $\phi$ and $\delta_{mob}$, the two remaining unknowns, $c$ and $R$ can be obtained. On this basis, $c$ is estimated as 10-13 kN/m$^2$ and 13-16 kN/m$^2$ for Test 1 and Test 2, respectively.

Thus, the estimated in-situ shear strength characteristics suggest that the backfill from Test 2 showed a lower $\phi$ and higher $c$ than the drier backfill of Test 1 (Lee et al. 1967, Holtz and Kovacs 1981). In fact, the direct shear tests (Earth Mechanics Inc., Fountain Valley, CA) conducted on the backfill material under inundation conditions (wetter than both Test 1 and Test 2) resulted in $\phi = 44$ (lower) and $c = 19$ kPa (higher), consistent with the trend observed from these passive pressure experiments (Appendix B).

2.3.6 Comparison of Measured Peak Resistance with Earth Pressure Theories

The total force measured by the load cells $P_h$ (Figure 2.11) is purely horizontal, but is nearly equal to the passive force resultant $P_p$ due to the low $\delta_{mob}$ (cosine of 2.7 degrees is 0.9989). This measured peak passive resistance in Test 1 is compared to the Rankine, Coulomb and Log Spiral predictions in Table 2.1. For Test 2, a similar comparison is provided (Table 2.2), with the experimental backfill moisture content closest to the lab test conditions. For these low $\delta_{mob}$ tests, the Coulomb and Log-Spiral predictions are essentially the same. Observations based on Tables 1 and 2 include:
1) Using the parameters estimated from the observed failure mechanism, the Coulomb and Log Spiral predictions provided very good estimates, slightly lower than the peak passive resistance (by 2% and 3% for Test 1 and Test 2, respectively).

2) With the field moisture content close to the lab moisture content (Test 2), the Log Spiral and Coulomb estimates from triaxial test parameters were 8% lower, and the direct shear test value predictions were 9% higher than the measured peak resistance.

3) For the drier backfill of Test 1, the Coulomb and Log Spiral estimates were 23% and 9% lower, based on the triaxial and direct shear parameters, respectively.

4) The Rankine predictions underestimated peak passive pressure in all cases (only by 2% however for the direct shear parameters in Test 2).

2.3.7 Residual Passive Resistance

Significant post-peak strain softening (Terzaghi et al. 1996) was clearly observed in both tests (Figure 2.7). In this regard, estimates of the residual passive resistance are made (Table 2.3) through force equilibrium on the observed passive failure wedge (Figure 2.11) considering $\phi_r$ and $c_r$ from the direct shear and triaxial tests (Figure 2.5). From Table 2.3, this force equilibrium approach provides a reasonable estimate (within 13% and 17% of the measured value using direct shear and triaxial parameters, respectively) of the measured residual resistance. These estimates are closer than what is predicted by employing the Rankine, Coulomb and Log Spiral theories, which implicitly assume a smaller incorrect failure wedge size based on $\phi_r$. 
2.4 FE Simulation of Load-Deflection Response

As in the case of Test 1 and Test 2, $\delta_{mob}$ can be small when relatively light structures (e.g. a sheet pile anchor wall) move upwards during passive loading. However, for a more massive structure, or one supported on piles, potential vertical motion may be restrained (Duncan and Mokwa 2001), resulting in larger $\delta_{mob}$, and affecting the passive force-displacement resistance. For the tested backfill soil, in addition to actual backfill soils (Earth Mechanics 2005), this issue is addressed below, by employing plane-strain (Plaxis 2D) FE simulations.

2.4.1 Configuration of FE Model and Boundary Conditions

Boundary conditions and backfill dimensions of the FE model were first set to match the test configuration (Figure 2.12). The FE model backfill was 5.6 meters long and 2.15 meters tall with 1.68 meters of backfill in contact with the model wall. Width of the plane strain section was 1 m. The far end of the backfill was fixed in the horizontal direction and free in the vertical as was the boundary of the small soil domain beneath the model wall (to mimic the relatively smooth plastic liner interface). To simulate the interface between the backfill and the base of the steel laminar box, interface friction ($\delta$) along the bottom of the model backfill was $0.2\phi$ (McCarthy 2007).

Stiff plate elements (1.68 meters tall) supported the soil laterally (Figure 2.12), with the same weight as the experimental wall and supporting beam (Figure 2.2b). During simulation of the experiments, a friction interface ensured that the wall would move upwards with the adjacent soil elements as the lateral displacement was being applied.
2.4.2 Elements and Mesh

Triangular 15-node elements were used to model the backfill. Simulations were run for this study using a “medium” mesh (Plaxis 2004) as shown in Figure 2.12. Within the imposed loading range, a stable solution with satisfactory convergence was achieved.

2.4.3 Soil Model

In a similar earlier numerical study (using FLAC), Martin and Yan (1995) modeled the passive earth pressure capacity using the Mohr-Coulomb failure criterion, with an elastic-perfectly plastic constitutive model. Similar to Martin and Yan (1995), the employed Plaxis (2004) Hardening Soil (HS) model uses the Mohr-Coulomb failure rule, but improves on the constitutive modeling end by employing a nonlinear hyperbolic stress-strain relationship. However, the HS model does not reproduce the observed dense sand softening behavior at large-strain shear response (Figure 2.6). Therefore, all analyses were restricted to the pre-peak loading range, with numerically stable solutions. Backfill material parameters (Table 2.4) were determined based on direct shear and triaxial tests, and user manual recommendations (Plaxis 2004).

2.4.4 Numerical Simulation of the Experimental Results

The FE simulation of Test 2 was made by prescribing a horizontal displacement boundary condition for the wall plate (left side of Figure 2.12), while allowing free vertical displacement. With this configuration, the wall moves upwards with the adjacent backfill in accordance with the experiments. Figure 2.13 presents a comparison between the measured and simulated load-displacement response from Test 2.
2.4.5 Modification for Additional Simulations

In order to minimize potential interference with the model boundaries, all additional simulations were performed on an extended and deepened backfill domain (Figure 2.12). Horizontal and vertical fixities were applied along the model base, with horizontal fixity along the boundaries at the far end of the backfill and beneath the model wall. In the cases where vertical wall movement was restrained, a $\delta = 0.35 \phi$ interface (Lemnitzer et al. 2009) was provided along the plate-soil boundary. A horizontal displacement was again ascribed to the plate that represented the test wall, with the vertical displacement assigned as zero.

2.4.6 Numerical Simulation for a Range of $\phi$, $c$, and $\delta$

Based on the good match with the experimentally measured behavior (Figure 2.13), FE modeling was used to simulate: i) the low $\delta_{mob}$ case with unrestricted wall movement; and ii) the vertically restrained wall with a high $\delta_{mob} = 0.35 \phi$ as suggested by Lemnitzer et al. (2009). In each case, two additional scenarios were studied (Earth Mechanics 2005), representing relatively soft and stiff actual backfills (soils EM1 and EM2 in Table 2.4). The resulting force-displacement relationships (Figure 2.14) indicate:

1) With vertical wall displacement restricted and $\delta_{mob} = 0.35 \phi$, a significantly greater resistance occurs compared to the vertical uplift scenario. For instance with soil T2, at 0.04 m of horizontal displacement, the restricted wall resistance was greater by more than 30% as compared with the vertical uplift counterpart.

2) As deformations accumulate, differences in resistance of as much as a factor of 2 may occur, depending on the employed backfill soil.
3) For this short wall scenario, a stiff response is observed for the high friction soil T2, and the EM2 soil of low friction and relatively high cohesion (Table 2.4).

2.5 Hyperbolic Models

Hyperbolic models have been employed to represent the passive load-deflection behavior up to peak resistance (Duncan and Mokwa 2001, Cole and Rollins 2006, Shamsabadi et al. 2007). Duncan and Mokwa (2001) employed a model defined by the initial stiffness ($K_{max}$) according to the following equation:

$$F(y) = \frac{y}{1 + \frac{y}{R_f} \frac{F}{F_{ult}}}$$  \hspace{1cm} (1)

where $F$ is the resisting force, $y$ is the horizontal displacement, $F_{ult}$ is the maximum passive resistance, and $R_f$ is a failure ratio. Alternatively, Shamsabadi et al. (2007) proposed a model based on secant stiffness $K$ (at $F_{ult}/2$) described by:

$$F(y) = \frac{F_{ult} (2K_{max} - F_{ult}) y}{F_{ult}y_{max} + 2(K_{max} - F_{ult}) y}$$  \hspace{1cm} (2)

where $y_{max}$ is the displacement when $F = F_{ult}$ (Duncan and Mokwa 2001 or Cole and Rollins 2006).

For use in practical applications, hyperbolic model parameters (Equations 1 and 2) are provided in Tables 2.5 and 2.6, which approximate the curves in Figures 2.14a and b (using recommended parameters where applicable from Duncan and Mokwa 2001 and Shamsabadi et al. 2007). The values listed in Tables 2.5 and 2.6 produce essentially identical curves using either Equation (1) or Equation (2). These models may be limited
at appropriate residual levels for displacements beyond the peak by capping the curve and continuing with a constant resistance thereafter.

2.6 Summary and Conclusions

Two large scale dense sand passive earth pressure experiments were performed under low $\delta_{mob}$ conditions. Test results were presented and analyzed. In addition, these results were complemented with FE simulations to consider different backfill soils and cases of larger $\delta_{mob}$. The results of this study suggest the following observations and conclusions:

1) Passive earth resistance from the dense sand backfill was shown to reach a peak, at a deflection of 2.7 and 3 percent of the wall height, before decreasing to a residual level in the two tests. The passive failure wedge was observed to be fully formed near the peak measured load.

2) Using the parameters estimated from the observed failure mechanism, the Coulomb and Log Spiral predictions provided very good estimates which only slightly underestimated the peak passive resistance (by 2% and 3% for Test 1 and Test 2, respectively).

3) With the field moisture content close to the lab moisture content in Test 2, the Log Spiral and Coulomb estimates from triaxial test parameters were about 10% lower than the measured result, while the direct shear test value predictions were about 10% higher.
4) Solving for force equilibrium on the observed failure wedge using ultimate shear strength parameters ($\phi_r$ and $c_r$) provided a reasonable estimate of the residual passive resistance.

5) With greater time between construction and testing (which allowed for evaporation and drainage to occur), about 20% greater maximum resistance, and more rapid shear strength degradation with displacement beyond the peak, were observed compared with the more moist (as-built) case. Analysis of the observed failure suggested that the drier backfill responded with a higher $\phi$ (about 10%) and lower $c$ (about 20%) than the wetter counterpart. This issue may warrant further investigation in light of similar potential long term variations due to aging effects and change in the placement water content.

6) Using parameters derived from direct shear and triaxial tests, the plane strain FE simulation was in good agreement with the experimental load-deflection response up to about 95% of the peak resistance.

7) Additional FE simulations illustrated trends as to how the strength and stiffness of actual backfills impact the load-displacement resistance.

2.7 Acknowledgements

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2.8 References


Table 2.1: Peak passive force by theoretical predictions for Test 1 (peak measured force = 385 kN/m and $\delta_{\text{mob}} = 2.3$ degrees)

<table>
<thead>
<tr>
<th>Source of shear strength parameters</th>
<th>$\Phi$ (deg)</th>
<th>$c$ (kPa)</th>
<th>Log Spiral/Coulomb</th>
<th>Rankine ($\delta=0$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Value (kN/m)</td>
<td>% of measured</td>
</tr>
<tr>
<td>Estimated from Failure Wedge$^a$</td>
<td>51$^b$</td>
<td>11.5$^b$</td>
<td>378</td>
<td>98</td>
</tr>
<tr>
<td>Triaxial Test</td>
<td>44</td>
<td>14</td>
<td>295</td>
<td>77</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>48</td>
<td>14</td>
<td>350</td>
<td>91</td>
</tr>
</tbody>
</table>

$^a$Field moisture content at time of test lower than the lab condition  
$^b$Average for a range of $\phi = 50$ - 52 and $c = 10$ - 13

Table 2.2: Peak passive force by theoretical predictions for Test 2 (peak measured force = 326 kN/m and $\delta_{\text{mob}} = 2.7$ degrees)

<table>
<thead>
<tr>
<th>Source of shear strength parameters</th>
<th>$\Phi$ (deg)</th>
<th>$c$ (kPa)</th>
<th>Log Spiral/Coulomb</th>
<th>Rankine ($\delta=0$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Value (kN/m)</td>
<td>% of measured</td>
</tr>
<tr>
<td>Estimated from Failure Wedge$^a$</td>
<td>45$^a$</td>
<td>14.5$^a$</td>
<td>315</td>
<td>97</td>
</tr>
<tr>
<td>Triaxial Test</td>
<td>44</td>
<td>14</td>
<td>299</td>
<td>92</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>48</td>
<td>14</td>
<td>355</td>
<td>109</td>
</tr>
</tbody>
</table>

$^a$Average for a range of $\phi = 44$ - 46 and $c = 13$ - 16
Table 2.3: Comparison of measured residual force (212 kN/m in both tests) with failure wedge equilibrium predictions using \( \phi \) and \( c_r \) from the lab tests (\( \delta_{mob} = 4.2 \) based on vertical equilibrium requirements)

<table>
<thead>
<tr>
<th>Source of shear strength parameters</th>
<th>( \Phi_r ) (deg)</th>
<th>( c_r ) (kPa)</th>
<th>Test 1</th>
<th>% of measured</th>
<th>Test 2</th>
<th>% of measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial</td>
<td>36</td>
<td>6</td>
<td>183</td>
<td>86</td>
<td>177</td>
<td>83</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>35</td>
<td>8</td>
<td>192</td>
<td>91</td>
<td>185</td>
<td>87</td>
</tr>
</tbody>
</table>

Table 2.4: FE model backfill parameters and additional soil information (Soils T2, EM1, and EM2)

<table>
<thead>
<tr>
<th>FE model parameter</th>
<th>T2a</th>
<th>EM1b</th>
<th>EM2b</th>
<th>Basis for selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi ) Effective angle of internal friction (deg)</td>
<td>46</td>
<td>34</td>
<td>25</td>
<td>Triaxial and direct shear test data</td>
</tr>
<tr>
<td>( c ) Effective cohesion (kN/m²)</td>
<td>14</td>
<td>15</td>
<td>62</td>
<td>Triaxial and direct shear test data</td>
</tr>
<tr>
<td>( \psi ) Angle of dilatancy (degrees)</td>
<td>16</td>
<td>4</td>
<td>0</td>
<td>( \psi = \phi - 30 ) (Plaxis 2004)</td>
</tr>
<tr>
<td>( p_{ref} ) Reference stress (confined) for stiffnesses (kN/m²)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>Default value (Plaxis 2004)</td>
</tr>
<tr>
<td>( m ) Power for stress-level dependency of stiffness</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>Plaxis (2004)</td>
</tr>
<tr>
<td>( E_{\text{50}}^{\text{ref}} ) Secant stiffness in standard drained triaxial test (kN/m²)</td>
<td>40000</td>
<td>31000</td>
<td>37000</td>
<td>Triaxial test data</td>
</tr>
<tr>
<td>( E_{\text{oed}}^{\text{ref}} ) Tangent stiffness for primary oedometer loading (kN/m²)</td>
<td>40000</td>
<td>31000</td>
<td>37000</td>
<td>Triaxial test data</td>
</tr>
<tr>
<td>( K_0 ) Coefficient of at-rest earth pressure</td>
<td>0.4</td>
<td>0.44</td>
<td>0.58</td>
<td>Default value (Plaxis 2004)</td>
</tr>
<tr>
<td>( \gamma ) Total unit weight (kN/m³)</td>
<td>20.6</td>
<td>17.3</td>
<td>20.9</td>
<td>Measured</td>
</tr>
<tr>
<td>( R_f ) Failure ratio for hyperbolic relation</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>Within recommended range (Duncan and Mokwa 2001)</td>
</tr>
</tbody>
</table>

**Additional backfill soil information**

<table>
<thead>
<tr>
<th>Classification (USCS)</th>
<th>SW-SM</th>
<th>SP-SM</th>
<th>SC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight (kN/m³)</td>
<td>19.3</td>
<td>16.2</td>
<td>18.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% Gravel</th>
<th>1 to 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Sand</td>
<td>86 to 92</td>
</tr>
<tr>
<td>% Fines</td>
<td>7 to 12</td>
</tr>
</tbody>
</table>

\( a \) Test 2, current study  
\( b \) Earth Mechanics Inc. (2005)  
\( c \) Adjusted for compatibility with other soil parameters (Plaxis 2004)
Table 2.5: Hyperbolic model parameters (per meter of width) for a 1.68 m (5.5 ft) tall wall, with $\delta$ governed by vertical equilibrium requirements (vertical uplift condition)

<table>
<thead>
<tr>
<th>Soil</th>
<th>$\phi$ (deg.)</th>
<th>c (kPa)</th>
<th>Equation (1) (Duncan and Mokwa 2001)</th>
<th>Equation (2) (Shamsabadi et al. 2007)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_{max}$ (kN/cm/m)</td>
<td>$R_t$</td>
<td>$F_{ult}$ (kN/m)</td>
<td>$\gamma_{max}$ (cm)</td>
</tr>
<tr>
<td>T2</td>
<td>46</td>
<td>14</td>
<td>250</td>
<td>0.75</td>
</tr>
<tr>
<td>EM1</td>
<td>34</td>
<td>15</td>
<td>220</td>
<td>0.75</td>
</tr>
<tr>
<td>EM2</td>
<td>25</td>
<td>62</td>
<td>260</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Table 2.6: Hyperbolic model parameters (per meter of width) for a 1.68 m (5.5 ft) tall wall, with $\delta = 0.35\phi$

<table>
<thead>
<tr>
<th>Soil</th>
<th>$\phi$ (deg.)</th>
<th>c (kPa)</th>
<th>Equation (1) (Duncan and Mokwa 2001)</th>
<th>Equation (2) (Shamsabadi et al. 2007)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_{max}$ (kN/cm/m)</td>
<td>$R_t$</td>
<td>$F_{ult}$ (kN/m)</td>
<td>$\gamma_{max}$ (cm)</td>
</tr>
<tr>
<td>T2</td>
<td>46</td>
<td>14</td>
<td>270</td>
<td>0.75</td>
</tr>
<tr>
<td>EM1</td>
<td>34</td>
<td>15</td>
<td>250</td>
<td>0.8</td>
</tr>
<tr>
<td>EM2</td>
<td>25</td>
<td>62</td>
<td>270</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Figure 2.1: Schematic elevation view of passive earth pressure experiment with instrumentation layout (foam cores were only used in Test 1)

Figure 2.2: Test configuration photographs
Figure 2.3: Backfill soil grain size distribution range from conducted tests
Figure 2.4: Peak strength parameters from direct shear and drained triaxial test results

Figure 2.5: Residual strength parameters from direct shear and drained triaxial test results
Figure 2.6: Stress versus strain from drained triaxial tests at three cell pressure levels showing the corresponding secant modulus to 50% of the peak stress ($E_{50}$)
Figure 2.7: Total measured load versus wall horizontal displacement backbone relationships for Test 1 and Test 2

Figure 2.8: Broken foam cores showing shape of failure wedge in Test 1 (note: the second core from the right did not fill its hole during construction and should be ignored).
Figure 2.9: Profile of Test 1 passive failure wedge

Figure 2.10: Profile of Test 2 assumed passive failure wedge
Figure 2.11: Forces acting on the observed failure wedge
Figure 2.12: FE model layout (top) and sample deformed mesh (bottom)
Figure 2.13: FE model calibration

Figure 2.14: Load-displacement results from FE passive pressure simulations: a) with $\delta$ governed by vertical equilibrium requirements (vertical uplift condition); and b) with $\delta = 0.35\phi$ (without vertical uplift)
Chapter 3  Passive Pressure Load-Displacement Models for a Range of Backfill Soils and Depths

3.1  Introduction

As explained in Chapters 1 and 2, passive earth pressure provides a mechanism to resist lateral foundation movement, resulting in either an increase or a decrease in the demand placed on the other structural components. While the Log-Spiral method (Terzaghi et al. 1996) has been shown to provide good estimates of experimentally measured peak passive resistance (Duncan and Mokwa 2001, Rollins and Cole 2006, Bozorgzadeh 2007, Lemnitzer et al. 2009), it does not offer information concerning the force-displacement relationship. For such force-displacement response, soil stiffness may substantially depend on depth (Terzaghi et al. 1996), and the different types of soils used in backfills have shown an extensive range of variation in shear stiffness and strength (Earth Mechanics 2005). Recent experiments have provided valuable new insight, but cover only a limited range of backfill soil types and wall heights (up to 2.3 meters).

The satisfactory match with the Test 2 experimental data, shown in Chapter 2 (Figure 2.13), and the need for additional passive force-displacement curves, motivate a continuation of the finite element (FE) investigation. A similar match is obtained between the FE model and the Test 1 force-displacement relationship, up to the peak resistance. Shamsabadi and Nordal (2006) and Bozorgzadeh (2007) also found good

On that basis, Plaxis (2004) plane-strain FE models are employed to consider a wider range of backfill types and wall heights. The resulting curves provide valuable insight into the variation in backfill passive earth pressure load-displacement response that can occur over a range of realistic backfill soils. Hyperbolic model representations of the simulated load-displacement curves are provided for use in practical applications.

3.2 Simulation of Test 1

A FE model simulation was compared with the passive earth pressure load-displacement relationship from Test 1 (Figure 3.1) by modifying the stiffness and strength soil properties of the Test 2 model (described in Chapter 2). All other model parameters remained the same. As shown in Table 3.1, the Test 1 simulation (Soil T1) backfill shear strength parameters were adjusted based on analysis of the observed passive failure wedge discussed in Chapter 2. A larger $E_{50}^{ref}$ also accounted for the experimentally observed higher stiffness compared with Test 2 (Figure 3.1). With those modifications to the FE model, a satisfactory match with the Test 1 data is obtained (Figure 3.1).

3.3 FE Simulations for a Range of Backfill Soils and Depths

FE Simulations are performed considering walls ranging from 1 meter (e.g., a pile cap, Rollins and Cole 2006) to 5 meters (e.g., a tall bridge abutment, Siddharthan et al.
1997) in height. Four different backfill soils (Soils T2, D-S, MD-SM, and MD-SC) are investigated to cover a range of likely backfill soil properties (Table 3.1).

### 3.3.1 Backfill Soil Properties

Soils D-S, MD-SM, and MD-SC (Table 3.1) represent three categories of sandy soils found during an extensive investigation of actual bridge abutment backfills in California (Earth Mechanics 2005). Soil D-S is dense (clean) sand, Soil MD-SM is medium-dense silty sand, and Soil MD-SC is medium-dense clayey sand. Soil T2 is the placement condition backfill from Test 2, as described in Chapter 2 (Table 3.1). In terms of $\phi$, Soil T2 represents an upper bound considering the soils found in the Earth Mechanics (2005) investigation. As such, the higher $\phi$ Soil T1 is not included in the practical simulations below.

The required modeling parameters are listed in Table 3.1 for Soils D-S, MD-SM, and MD-SC. Parameters were determined based on the Earth Mechanics (2005) report and Plaxis (2004) recommendations, following the same guidelines that produced the satisfactory match with Test 2 data in Chapter 2. The soil friction angle $\phi$ and cohesion $c$ were determined from conducted direct shear and triaxial tests (Earth Mechanics 2005). The dilatancy angle is estimated as $\psi = \phi - 30$ (and $\geq 0$, Plaxis 2004).

The stiffness parameter $E_{50,ref}$ (Figure 3.2) was selected at a reference stress $p_{ref} = 100$ kPa from the triaxial test stress-strain data (Earth Mechanics 2005) according to a power law with $m = 0.5$ defined by (Plaxis 2004):

$$E_{50} = \frac{E_{50,ref}^{m}}{\left(\frac{c \cos \phi - \sigma_3 \sin \phi}{c \cos \phi + p_{ref}^{m} \sin \phi}\right)^m} \quad (3.1)$$
where $\sigma_3'$ is the minor principal stress, which is the confining pressure in a triaxial test.

Figure 3.3 demonstrates the determination of $E_{50}$ from triaxial test data. The default values of $E_{oed}^{ref} = E_{50}^{ref}$, and $E_{ur}^{ref} = 3 \times E_{50}^{ref}$ were also taken for the reference oedometer stiffness and the unloading-reloading stiffness, respectively (Plaxis 2004).

Small variations in the coefficient of lateral earth pressure at rest $K_0^{nc}$ were found to have little or no effect on the passive load-displacement response. As such, Plaxis (2004) default values were used (and sometimes adjusted internally by Plaxis for compatibility with the other soil parameters). Soil total unit weight $\gamma$ was specified according to the Earth Mechanics (2005) recommendations. A failure ratio ($R_f = q_f/q_{as}$, Figure 3.2) value of $R_f = 0.75$ was adopted after it was found to work well in simulating Tests 1 and 2.

### 3.3.2 Adjustments to the Model

In order to consider taller wall configurations, and to minimize potential interference with the model boundaries, simulations were performed on extended and deepened backfill domains (e.g., Figure 3.4), with acceptable dimensions determined through a trial and error process. Horizontal and vertical fixities were applied along the model base, with horizontal fixity also along the far end of the backfill and the soil domain beneath the model wall.

Lemnitzer et al. (2009), recently conducted a bridge abutment passive pressure experiment in which the test backwall displaced essentially solely in the horizontal direction. This was done in consideration of the potentially large friction force between the end of the bridge deck and the backwall (Lemnitzer et al. 2009), which may prevent
vertical wall movement. In that configuration, wall-soil interface friction $\delta = 0.35\phi$ was measured at the instant of the peak measured load (Lemnitzer et al. 2009).

In the case of pile caps and integral abutments, a restriction from vertical wall uplift may also be anticipated (Duncan and Mokwa 2001). On that basis, a $\delta = 0.35\phi$ interface $R_{inter}$ (Plaxis 2004) was provided along the plate-soil boundary for the FE simulations. In order to simulate the passive earth pressure force-displacement relationship, a horizontal displacement was ascribed to the plate that represented the wall, but the vertical displacement was assigned as zero (Figure 3.4), in accordance with the Lemnitzer et al. (2009) experiment. Analyses are restricted to the pre-peak loading range, with numerically stable solutions.

### 3.3.3 Simulation Results

Figures 3.5 and 3.6 show the simulated passive force-displacement response, per meter of wall width, for the 4 soils mentioned above, considering wall heights $H$ ranging from 1 to 3 meters. For clarity the axes in Figure 3.5 are scaled differently for each soil. For direct comparison, the curves are also presented in Figure 3.6 with the axes scaled the same for all 4 soils. FE simulations of 4 and 5 meter walls were also conducted (shown further below).

From Figures 3.5 and 3.6, there is clearly a wide range in backfill strength and stiffness, depending on both the soil type and the wall height $H$. According to the FE model simulation with clean sand backfill (Soil D-S), the passive resistance with $H = 3$ reached nearly 10 times that of the $H = 1$ meter case (Figure 3.5b). The stiffness also increased rapidly as the wall became taller for Soil D-S (Figure 3.5b). For instance at a
horizontal wall displacement of 1 centimeter, about 4.4 times the passive resistance was mobilized for $H = 3$ meters, compared with $H = 1$ meter (Figure 3.5b). In contrast, for the high $c$ and lower $\phi$ Soil MD-SC simulation (Figure 3.5d), the passive resistance with $H = 3$ reached only about 4 times that of the $H = 1$ meter case, and the stiffness increase for taller walls was also less pronounced.

The drastic difference in load-displacement response for Soils T2 (Figure 3.6a) and D-S (Figure 3.6b) helps to further illustrate why it can be important to accurately account for the backfill soil strength and stiffness. Soil shear strength is often roughly approximated for earth pressure predictions, sometimes neglecting the cohesion in sandy soils (CSBC 2007). In some cases, it may be conservative (possibly resulting in costly over-design) to neglect cohesion and use a typical dense sand friction angle $\phi = 38$ degrees (Earth Mechanics 2005), such as Soil D-S in Figure 3.6b. However when the passive pressure imposes loads which might damage the structure (e.g., expansion of an integral abutment bridge), the opposite may be true. For instance, if a dense sand backfill similar to Soil T2 ($\phi = 46$ degrees and $c = 14$ kPa) were characterized with the more typical $\phi = 38$ degrees, and $c = 0$ values of Soil D-S, the passive resistance could be underestimated by a factor of 2 or more (Figures 3.6a and b).

Load-displacement simulations of the four backfill soils are also compared for the 1 and 5 meter tall walls in Figures 3.7 and 3.8, respectively. Behind the 1 meter wall of Figure 3.7, the highly cohesive Soil MD-SC was quite strong and preserved its stiffness over a large range of deflection. However, due to the higher confining stress conditions (Terzaghi et al. 1996) for the 5 meter wall of Figure 3.8, the soil with the largest $\phi$ (T2) became the strongest and stiffest by a significant margin. Similarly, the cohesionless Soil
D-S was clearly the weakest for the 1 meter wall (Figure 3.7), but came close to matching Soils MD-SM and MD-SC in Figure 3.8, due to its relatively high $\phi$ and the deeper (5 meter) backfill.

### 3.4 Comparison of the Simulated Curves with Design Models

For use in seismic bridge design, AASHTO (2007) provides bi-linear abutment passive resistance models for “cohesive” and “non-cohesive” soils. Peak passive resistance ($P_p$) and abutment stiffness ($K_{abut}$) for these models are derived from Equations (3.2) and (3.3):

\[
P_p = p_p \times H_w \times w_w \tag{3.2}
\]

\[
K_{abut} = \frac{P_p}{F_w \times H_w} \tag{3.3}
\]

where $p_p$ is the passive lateral earth pressure, $H_w$ is the wall height, $w_w$ is the width, and $F_w$ is a factor ranging from 0.01 for dense sands to 0.05 for compacted clays (Clough and Duncan 1991). For “non-cohesive”, non-plastic backfill (fines content < 30%), AASHTO (2007) provides an estimated $p_p = 100H_w$ kPa. For “cohesive” backfill (clay fraction > 15%), AASHTO provides an estimated $p_p = 240$ kPa.

For a typical abutment sacrificial backwall height of 1.7 meters (Shamsabadi et al. 2007), Figure 3.9 compares the FE curves with the AASHTO (2007) “non-cohesive” backfill model, for the soils that are closest to the criteria described above (T2, D-S, and MD-SM). A similar comparison is shown in Figure 3.10 for a taller (3 meter) abutment wall (Siddharthan et al. 1997). Figures 3.11 (1.7 meter wall) and 3.12 (3 meter tall wall)
compare the AASHTO (2007) “cohesive” backfill model with the Soil MD-SC (which meets the above cohesive backfill criteria) simulated force-displacement curve.

According to the FE simulation results for the 1.7 meter tall wall of Figure 3.9, the AASHTO (2007) “non-cohesive” backfill model with $F_w = 0.015$ provides a good match with Soil MD-SM, but significantly under and over predicts the resistance provided by Soils T2, and D-S, respectively. For the 3 meter wall of Figure 3.10, the AASHTO (2007) model provides a better representation for Soil T2, while over estimating in terms of both stiffness and capacity for Soils D-S and MD-SM.

For the 1.7 meter wall with Soil MD-SC, the AASHTO (2007) “cohesive” soil model with $F_w = 0.025$ provides a satisfactory match in terms of stiffness, but underestimates the capacity (Figure 3.11). With the 3 meter wall, the AASHTO (2007) model underestimates both the stiffness and capacity, compared with the FE model results (Figure 3.12). Based on the comparisons in Figures 3.9 through 3.12, higher order approximations of the passive force-displacement relationship could clearly lead to a safer or more economic design.

3.5 Hyperbolic Load-Displacement Models

As mentioned previously, hyperbolic models have been shown to provide a good representation of the passive load-deflection behavior up to the peak resistance (Duncan and Mokwa 2001, Cole and Rollins 2006, Shamsabadi et al. 2007). Such hyperbolic models can be used as nonlinear springs to represent the passive earth pressure load-displacement resistance. For dynamic simulations, a material (“hyperbolicgapmaterial”) is also available for use as a spring in the finite element code OpenSees (Mazzoni et al.}
The “hyperbolicgapmaterial” (Dryden 2009) implements a backbone curve using the Duncan and Mokwa (2001) hyperbolic model Equation (3.4), as described in the previous chapters, an adjustable expansion gap, and a linear unloading and reloading stiffness approximation (described in further detail and demonstrated in Chapter 8 of the dissertation).

\[
F(y) = \frac{y}{\frac{1}{K_{\text{max}}} + R_{R} \frac{y}{F_{\text{ult}}}}
\]  

Equation (3.4)

Hyperbolic model parameters for Equation (3.4) are provided in Tables 3.2 through 3.5 as approximations of the FE simulation results (Figure 3.13). Due to the limitations imposed by the range of numerically stable FE model solutions, the peak resistance \( F_{\text{ult}} \) was approximated based on the Log Spiral (Terzaghi et al. 1996) prediction. The additional parameters were selected to match the FE load-displacement response pattern (Figure 3.13).

In practical applications, these models can be scaled according to the width of the structure which mobilizes the passive pressure behind it, with an applied 3D correction factor in the case of narrow walls (e.g. Brinch-Hansen 1966). Compared with the bi-linear models shown in Figures 3.9 through 3.12, the hyperbolic curves of Figure 3.13 clearly provide a superior representation of the passive force-displacement relationship.

3.6 Conclusions

On the basis of the good agreement found between experimental data and simulated results, FE models were employed to investigate the passive earth pressure force-displacement relationship considering four different backfills and a range of wall
heights. Results show how the different backfill soils can provide substantially different load-displacement resistance, in terms of both stiffness and strength. It was shown that the increase in supported backfill height and the depth dependent stiffness of the soil contribute to significant variations in the available resistance.

For practical applications, hyperbolic model approximations were provided for 32 different combinations of backfill soil type and wall height. Such hyperbolic models can be used as nonlinear springs to represent the passive earth pressure load-displacement resistance in pushover analyses and dynamic simulations.

3.7 Acknowledgements

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3.8 References


### Table 3.1: FE model soil parameters

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<th>MD-SM&lt;sup&gt;b&lt;/sup&gt;</th>
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<sup>a</sup>Current experimental study  
<sup>b</sup>Earth Mechanics Inc. (2005)

### Table 3.2: Hyperbolic model parameters for Soil T2

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### Table 3.3: Hyperbolic model parameters for Soil D-S

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Table 3.4: Hyperbolic model parameters for Soil MD-SM

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Table 3.5: Hyperbolic model parameters for Soil MD-SC

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Figure 3.1: Backbone force-displacement relationships from Test 1 and Test 2 compared with the FE model predictions.

Figure 3.2: Hyperbolic stress-strain relationship (from Plaxis 2004).
Figure 3.3: Determination of $E_{50}$ from a standard drained triaxial test (from Plaxis 2004)

Figure 3.4: Sample deformed mesh from simulation of a 3.5 meter tall wall with backfill Soil MD-SC
a) Soil T2 (\(\phi = 46\) degrees, \(c = 14\) kPa, \(E_{50}^{\text{ref}} = 40,000\) kN/m\(^2\))

b) Soil D-S (\(\phi = 38\) degrees, \(c = 0\), \(E_{50}^{\text{ref}} = 35,000\) kN/m\(^2\))

c) Soil MD-SM (\(\phi = 33\) degrees, \(c = 24\) kPa, \(E_{50}^{\text{ref}} = 30,000\) kN/m\(^2\))

d) Soil MD-SC (\(\phi = 23\) degrees, \(c = 95\) kPa, \(E_{50}^{\text{ref}} = 30,000\) kN/m\(^2\))

Figure 3.5: FE simulation results
a) Soil T2 ($\phi = 46$ degrees, $c = 14$ kPa, $E_{s0}^{ref} = 40,000$ kN/m$^2$)

b) Soil D-S ($\phi = 38$ degrees, $c = 0$, $E_{s0}^{ref} = 35,000$ kN/m$^2$)

c) Soil MD-SM ($\phi = 33$ degrees, $c = 24$ kPa, $E_{s0}^{ref} = 30,000$ kN/m$^2$)

d) Soil MD-SC ($\phi = 23$ degrees, $c = 95$ kPa, $E_{s0}^{ref} = 30,000$ kN/m$^2$)

Figure 3.6: FE simulation results (same as Figure 3.5, except with the axes scaled for direct comparison between the 4 soils)
Figure 3.7: Comparison of Soils T2, D-S, MD-SM, and MD-SC for a 1 meter tall wall.

Figure 3.8: Comparison of Soils T2, D-S, MD-SM, and MD-SC for a 5 meter tall wall.
Figure 3.9: Soils T2, D-S and MD-SM compared with the AASHTO (2007) bilinear model with $F_w = 0.015$ for non-cohesive soil and a 1.7 meter tall wall

Figure 3.10: Soils T2, D-S and MD-SM compared with the AASHO (2007) bilinear model with $F_w = 0.015$ for non-cohesive soil and a 3 meter tall wall
Figure 3.11: Soil MD-SC compared with the AASHTO (2007) bilinear model with $F_w = 0.025$ for cohesive soil and a 1.7 meter tall wall.

Figure 3.12: Soil MD-SC compared with the AASHTO (2007) bilinear model with $F_w = 0.025$ for cohesive soil and a 3 meter tall wall.
Figure 3.13: FE simulated force-displacement curves for 1, 1.3, 1.7, 2, 2.5, 3, 4 and 5 meter tall walls) and hyperbolic model approximations
Chapter 4  Large Scale Shake Table Dynamic Earth Pressure

Experiments:  At-Rest Condition

4.1  Abstract

Experiments are performed to measure dynamic lateral earth pressure during shake table excitation. A dense well-graded silty sand backfill (2.15 m depth, 2.87 m width, and 5.6 m length) is constructed behind a vertical test wall section (1.7 m high) in a large soil container. Built on an outdoor shake table, the soil container-wall-backfill configuration is subjected to earthquake and harmonic input motions. Acceleration, displacement, force, and pressure measurements are presented from these shake table excitation experiments. With peak input accelerations of up to about 0.65 g, the measured dynamic earth pressure resultant is found to be relatively small. In this input acceleration range, experimental data show that as the top of the wall moves slightly away from the backfill, the relatively strong and stiff soil does not deform adequately to exert large earth pressure increases over the full test wall height. However, at instants of higher acceleration (> 0.65 g), the earth pressure increases simultaneously along the full wall height and the resultant dynamic force becomes significant. The results help to illustrate how accurate consideration of the retaining wall-backfill interaction may result in more realistic dynamic earth pressure predictions than the more common simplified analytical methods.
4.2 Introduction

Reliable retaining wall performance is critical for the stability and safety of critical infrastructure components, such as bridges, roadways, railways, ports, and subway systems. After a major earthquake, these structures must allow continued safe transportation for rescue and repair efforts, and facilitate a rapid recovery. Yet, consensus on the prediction of seismic forces on retaining structures continues to elude practicing engineers. For instance, recent research suggests that assumptions made by analytical solutions of the seismic earth pressure, such as the Mononobe-Okabe equations (Okabe 1926, and Mononobe and Matsuo 1929) and Wood’s elastic solution (1973), may not apply to many practical retaining wall situations (Koseki et al. 1998, Gazetas et al. 2004, Psarropoulos et al. 2005, Nakamura 2006, NCHRP 2008, Al Atik and Sitar 2009, Shukla et al. 2009).

While damage to retaining walls has been observed after some earthquakes, it has often involved a weak (for instance liquefiable) underlying layer (Gazetas et al. 2004, Shirato et al. 2006). In the absence of such a weak layer, some retaining structures have performed well, even in cases where the seismic load was not explicitly a design consideration (Seed and Whitman 1970, Gazetas et al. 2004, Al Atik and Sitar 2009). In this regard, a better understanding of the loads placed on retaining structures during earthquakes would be beneficial for practical seismic design.

Based on the considerations above, there is a substantial demand for further insight into the dynamic earth pressure problem. To that end, an experimental program is presented in the sections below, in which a vertical reinforced concrete test wall section,
and supported dense, well-graded silty sand backfill are subjected to shake table excitation.

A brief review of some notable analytical and experimental studies on dynamic earth pressure is provided first. The test configuration and program are described next. Finally, the recorded output data are presented, analyzed and discussed.

### 4.3 Previous Analytical and Experimental Studies

As early as the 1920’s, Okabe (1926) and Mononobe and Matsuo (1929) performed dynamic earth pressure experiments and extended the limit-equilibrium analysis of Coulomb, to include pseudo static backfill inertial forces (Seed and Whitman 1970, and Kramer 1996). Resulting from those studies, the Monobe-Okabe (M-O) method for predicting dynamic active (and passive) earth pressure is used extensively today (Veletsos and Younan 1997, Richards et al. 1999, Jung and Bobet 2008, Sitar and Al Atik 2009). The M-O method was originally developed based on the assumption of dry cohesionless soil behind and a wall which yields enough to produce minimum active pressure (Seed and Whitman 1970). In that configuration, a soil wedge (with its size depending of the level of input acceleration) is assumed to act on the wall, and to behave as a rigid body (Seed and Whitman 1970).

In consideration of rigid “non-yielding” walls, which do not meet the assumptions required for application of the M-O method, a rigorous analytical study was performed by Wood (1973). Wood’s (1973) extensive work provided an approach for predicting dynamic earth pressure for the idealized case of a perfectly rigid wall connected to an elastic layer of finite length.
Additional analytical studies have been prompted by the apparent favorable performance of retaining walls during earthquakes (Seed and Whitman 1970, Al Atik and Sitar 2008). For instance, Seed and Whitman (1970) showed that walls designed with an adequate factor of safety (1.5 for instance) for the static active earth pressure may be able to safely withstand earthquake shaking at levels of 0.3 g or more. In addition, since the duration of peak ground acceleration instances is very brief, it was suggested to use a reduction factor when considering the M-O earth pressure (Seed and Whitman 1970). Modifications have also been proposed which reduce the M-O dynamic active earth pressure prediction by accounting for soil cohesion (e.g., Saran and Prakash 1968, Richards and Shi 1994, NCHRP 2008, and Shukla et al. 2009).

The above analytical studies have provided a means for predicting earth pressure during an earthquake based on sets of simplifying assumptions. However, in many practical situations, the retaining wall-backfill behavior may not meet either the M-O method or the rigid wall set of assumptions (Veletsos and Younan 1997, Gazetas et al. 2004, Nakamura 2006, Al Atik 2008). In such cases, the wall-soil interaction mechanism might significantly affect the magnitude and distribution of dynamic earth pressure.

For instance, Alampalli and Elgamal (1990), Veletsos and Younan (1997), Gazetas et al. (2004), Psarropoulos et al. 2005, and Jung and Bobet (2008) found through numerical and analytical simulations that a retaining wall might experience lower dynamic earth pressure if it is able to rotate or bend. Richards et al. (1999) used a kinematic solution to show that the location of the dynamic earth pressure resultant may depend on the type of wall movement. Currently, experimental verification of such
interactions would instill further confidence into the observations made based on these analytical studies.

In addition to analytical studies, recent 1 g (Koseki et al. 1998 and Ling et al. 2005) and centrifuge (Nakamura 2006, and Al Atik and Sitar 2009) shake table studies have also provided new insights. Based on experimental evidence, Koseki et al. (1998) suggested modifications to the M-O method considering the possibility of a weakened band of backfill material (strain localization) existing along a previously formed active failure wedge. From large scale test results on reinforced modular block walls, Ling et al. (2005) concluded that existing design methods underestimate the capacity of such flexible systems. Results from two separate centrifuge experiments (Nakamura 2006, and Sitar and Al Atik 2009) suggest that the M-O method predictions are higher than the actual dynamic earth pressure, and that the inertial force is not always transmitted to the wall and backfill simultaneously.

The above experimental investigations have found further inconsistency between the predicted and observed dynamic earth pressure. In the following sections, additional significant contributions are added to the existing knowledge base on dynamic earth pressure through the presentation of new large scale 1 g shake table experiments and related analyses.

### 4.4 Testing Facility

The Englekirk Structural Engineering Center (ESEC) of the University of California San Diego (UCSD) is home to the world’s first outdoor, and currently the largest shake table in the U.S. (UCSD 2003). Experiments for the presented study were
performed on the 12.2 m (40 ft) by 7.6 m (25 ft) outdoor shake table (Figures 4.1 and 4.2). At a location approximately 15 km east (inland) of the UCSD campus, a warm semi-arid climate provides an ideal environment for this outdoor testing facility. Funded by the George E. Brown Jr. Network for Earthquake Engineering Simulation (NEES, http://nees.ucsd.edu), the shake table is currently capable of supporting a vertical payload of 20 MN (UCSD 2003, Restrepo et al. 2005) and producing uni-axial lateral motions of up to 3 g (acceleration), 1.8 m/s (velocity) and 0.75 m (displacement). For future applications, the shake table is designed to be upgradeable to six degrees of freedom (Luco et al. 2004).

4.5 Experimental Configuration

The main components of the test configuration are the same as those presented in Chapter 2. Additional figures describing the testing configuration are included in Appendix A.

A vertical test wall section was placed inside a large soil container, with a dense sand backfill constructed on one side (Figure 4.3). In order to provide stability during shaking under this configuration, the employed laminar box was post-tensioned to the shake table and restrained laterally (Figures 4.1 and 4.2). In place of rollers (designed to facilitate lateral displacement between the laminar box frames), wooden blocks were used to support the frames. Additional restraint was provided by post-tensioning two large stiff steel towers (originally designed to provide stability along the longer sides of a much taller laminar container configuration) to the shake table on the ends of the container
(Figures 4.1 and 4.2). The container ends and towers were also welded together to prevent separation and impacting during shaking.

Figure 4.3 shows the inside dimensions, and the primary test components and instrumentation (additional installed instrumentation is described in Appendix A). The backfill occupied the majority of the container length and extended deeper than the base of the test wall. A rectangular wooden box was placed beneath the wall (with a small gap in between) to support this extended backfill depth.

4.5.1 Test Wall

The reinforced concrete test wall was 0.2 m (8 in) thick, 2.74 m (9 ft) wide and 2.13 m (7 ft) tall (Figure 4.4). Gaps between the side edges of the wall and container were filled with soft foam (approximately 6.5 cm on each side). For stability during construction and testing, the wall was suspended from a beam which rested on rollers (Figure 4.4) supported on the longer sides of the soil box. The combined mass of the test wall and supporting beam was about 4500 kg (equal mass of a 0.3 m thick wall of similar width and height). The depth of backfill supported laterally by the test wall was $H = 1.68$ m (Figure 4.3).

Adjustments to the test wall position were made using 4 hydraulic jacks (Figures 4.3 and 4.4). These jacks were mounted on concrete filled steel posts which reacted through load cells onto the laminar frames and stiff tower on the left side of Figures 4.1 and 4.2. The jacks were connected to a hydraulic pump through a manifold, allowing independent control in order to adjust the wall orientation (in subsequent passive pressure testing). Closing the valves in the manifold locked each individual jack in its respective
position. Through this mechanism, the test wall could be positioned and held at varying levels of lateral displacement into the backfill. The employed short plunger jacks and load cells also helped to maximize the backfill length within the soil container while providing this added source of control and force measurement (Figures 4.3 and 4.4).

While the soil container was restrained as described above, and the test wall was supported from behind by the jack and load cell configuration (Figures 4.3 and 4.4), minor relative displacements of the wall were possible due to inevitable compliance of this testing configuration (Figures 4.1 through 4.4). These small relative test wall displacements are an important consideration in the overall response, as discussed later in further detail.

### 4.5.2 Instrumentation

In addition to the load cells mentioned above, instrumentation (Figure 4.3) included linear potentiometers (LP), pressure transducers (PT) horizontal and vertical accelerometers in the soil (HA, and VA) and accelerometers on the soil container and test wall. The load cells measured the combined test wall inertia and lateral earth pressure resultant force. Linear potentiometers measured the vertical displacement of the wall (Appendix A) and heave or settlement of the backfill. Pairs of transducers at 3 depths between the wall and backfill recorded the distribution of pressure along the height and width of the wall. Through numerical integration and filtering, accelerometer recordings also provided estimates of velocity and displacement at the various installed locations (Figure 4.3).
4.6 Backfill Soil and Construction

Sand with small amounts of gravel and silty fines (Figure 4.5) was placed in small lifts (each about 0.2 m thick), and compacted using gasoline powered tamper rammers (Figure 4.6). Plywood and three layers of smooth plastic sheeting covered the inside walls (Figure 4.6) to minimize friction between the soil and the sides of the container (friction angle of 11.5 to 14 degrees, Fang et al. 2004). The backfill compaction verification was done by nuclear gauge measurements. According to the measurements, compacted dry unit weight $\gamma_d$ was about 19.3 kN per cubic meter (95.5% of the modified proctor, or ASTM Standard Method D1557 Test value) with average water content at placement of 8.6% (optimum value was 8.5%).

Results from laboratory direct shear and triaxial compression tests on samples remolded as closely as possible to the field backfill placement conditions are shown in Figures 4.7 and 4.8. Direct shear tests were performed (Earth Mechanics Inc., Fountain Valley, CA) on new samples to failure at 5 different normal stress levels. The results (Figures 4 and 5) suggest peak and residual friction angles of $\phi = 48$, and $\phi_r = 35$ degrees, with cohesion intercepts $c = 14$ kPa, and $c_r = 8$ kPa, respectively.

Drained triaxial tests were performed (Leighton and Associates, Irvine, CA) by subjecting three separate specimens to respective cell pressures of 37, 72 and 144 kPa and then shearing to failure by increasing the deviator stress. Compared with the direct shear tests (Figures 4.7 and 4.8), the triaxial results suggest a lower peak $\phi = 44$ degrees, slightly higher residual $\phi_r = 36$ degrees, the same $c = 14$ kPa, and slightly lower $c_r = 6$ kPa. The secant modulus to 50% of the peak stress ($E_{50}$) was approximately 16400
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kN/m², 18700 kN/m², and 48200 kN/m² for the 37 kPa, 72 kPa, and 144 kPa cell pressure levels, respectively.

The $\phi$ values mentioned above are near the upper end of the range documented from an investigation of actual structural backfills (Earth Mechanics 2005), with $\phi$ of 39 - 49 degrees in direct shear tests and $\phi$ of 24 - 43 degrees in triaxial tests (for sands and silty sands). The above $c$ values are closer to the lower end of the range of $c = 6 - 68$ kPa reported by Earth Mechanics (2005). Estimated from the triaxial stress strain curves, the range of $E_{50}$ from Earth Mechanics (2005) encompasses the values listed above.

4.7 Input Motions

As mentioned in Chapter 1, these shake table dynamic earth pressure tests were a part of a collaborative investigation into highway bridge seismic performance (Saiidi 2004). Scaled versions of a modified Century City Station record of the 1994 Northridge earthquake (Saiidi 2004) were designated as input excitations for the shake table tests performed in that collaborative study. In addition, a harmonic (sine wave) motion with frequency increasing gradually from 1 to 5 Hz was employed for the tests presented here. Motions were imparted in the direction normal to the test wall face (one dimensional shaking). Recorded acceleration time histories (at the container base, Figure 4.3) from the above mentioned baseline Northridge Earthquake Motion (EM) and harmonic input control motion (HM) with frequency increasing gradually from 1 to 5 Hz are shown in Figures 4.9 through 4.16.
4.8 Initial Conditions and Test Sequence

After the backfill was constructed inside the soil container, the test wall position was adjusted slightly (about 2.5 millimeters of lateral displacement towards the backfill) by extending the jacks (Figures 4.3 and 4.4) until they all engaged the wall, and each of the load cells recorded a nonzero reading. The jacks were locked with the test wall in this near at-rest (close to zero displacement) position. Next a series of shake table tests was performed using scaled versions of the EM and HM (Figures 4.9 through 4.16) with the jacks locked in that position (it also noted that separation between the wall and jacks was not restricted in this configuration). Table 4.1 presents the order of the conducted test series, and the recorded peak base acceleration (A), velocity (V) and displacement (D) from each test (velocity and displacement were estimated through integration of the base acceleration record).

4.9 Test Results

4.9.1 Lateral Force Measurements

Situated behind the test wall, the load cells (Figures 4.3 and 4.4) provide a measurement of the total lateral force. As such, the load cell (LC) force values include the lateral earth thrust plus the test wall inertia during shaking, and the static lateral earth pressure resultant before and after the excitation (when the inertia of the wall and soil is zero). The base acceleration and LC force (per meter of wall width) are presented as time histories from the 1, 2, 3 (two tests), 3.3 (two tests) times the EM tests, and the 1 and 1.5 times the HM tests in Figures 4.9 through 4.16.
During the 1 and 2xEM tests of Figures 4.9 and 4.10, the LC force increased and decreased mildly, in a pattern similar to that of the base acceleration. A similar pattern of response was observed from the 1/3 and 2/3xEM tests (not shown). However in the stronger 3 and 3.3xEM tests (Figures 4.11, 4.12, 4.15, and 4.16), significantly larger brief increases in the LC force occurred near the peak instants of base acceleration.

Similarly, Figure 4.13 (1xHM) and Figure 4.14 (1.5xHM) present the base acceleration and LC force from the harmonic input motion tests. With base acceleration peaks of about 0.6 g (1xHM), Figure 4.13 shows LC force increases on the order of 20 kN/m. With acceleration peaks of about 0.9 g (1.5xHM), Figure 4.14 shows much larger LC force increases, on the order of 80 kN/m.

A summary of the LC force time histories from each of the 10 conducted tests is also provided Table 4.1, including the initial, final, maximum, and minimum value, and the peak increase and decrease from the initial (static) value. From Table 4.1, during the 1/3, 2/3 and 1xEM tests, the LC force did not change significantly from its initial value. In the 2xEM and 1xHM tests, the LC force changes became slightly more significant. In the remaining (very strong excitation) experiments, much larger increases in lateral force were observed. Figure 4.17 graphically illustrates this trend, by comparing the peak LC force increase, per meter of wall width (peak dynamic force), with the corresponding input acceleration from each test. The order of execution of the tests is indicated next to each result.

Additional observations from the presented data include: i) the final static load decreased noticeably from its initial value during some of the stronger excitation tests (Table 4.1), and ii) the peak force increase was significantly larger in the repeated 3 and
3.3xEM tests than during the initial execution of those two input motions (Table 4.1, Figure 4.17). These observations indicate that changes in the backfill condition may have occurred throughout the test series, possibly resulting in a decrease in its ability to resist deformation during shaking.

4.9.2 Acceleration, Displacement, Force and Pressure Time History Results

In Figures 4.18 through 4.23, a detailed set of the recorded experimental data is presented for six representative tests. These figures reveal further insight into the significant difference in LC force response to input motions with peak input acceleration levels below and above about 0.66 g (Figure 4.17). In order to focus on key aspects of the response, these time histories show a 10 second window from the strong shaking phase of the input excitation. Force (F) values are normalized per meter of wall width.

In Figures 4.18 through 4.23, the base (input) and backfill surface (HA 1, Figure 4.3) accelerations are included at the top (a). Next (b), displacement histories are shown by double integrating and filtering (bandpass filter from 0.4 to 120 Hz) accelerometer data. Using that method, the total base (input) displacement is presented on the left, and the displacement near the top and bottom of the test wall (Figure 4.3), relative to the base, are shown on the right. Next, on the left side (c), the total force measured by the load cells (LC force) is presented. Below that, the inertia of the test wall is represented by its mass times its acceleration. To the right (d), the pressure transducer (PT, Figure 4.3) readings from near the middle and top of the supported backfill height (0.45H and 0.82H) are presented.
During the moderate shaking of the 1xEM test (Figure 4.18), the peak base acceleration was 0.33 g, with the peak surface acceleration slightly higher. Peak base displacement was 8 cm. Changes in the LC force were mild, comparable to those of the test wall inertia. The top of the wall moved as much as just over 1 millimeter relative to the container base, with bottom displacement being much smaller. The recorded pressure at the top (0.82H) and middle (0.45H) varied slightly from the static value. Notably, there was an apparent phase difference which resulted in a decrease in the middle pressure at the time of an increase near the top and vice versa (most visible near the peak response cycle from just before to just after 15 seconds).

During the stronger shaking (base acceleration up to 0.66 g, and displacement up to 14 cm) of the 2xEM test (Figure 4.19), the range of LC force was slightly larger than what is accounted for by the test wall inertia. Compared with the 1xEM test with half the peak input acceleration (Figure 4.18), more than double the range of pressures and LC forces were measured during 2xEM (Figure 4.19). The phase difference between the middle and top pressure transducer readings also occurred during 2xEM, similar to the 1xEM test. Displacements at the test wall were as much as 2.5 millimeters near the top, with much smaller movements occurring at the bottom.

With peak base accelerations of 1.06 g and 1.2 g, and displacements of 22 cm and 24 cm, the 3 and 3.3xEM tests (Figures 4.20 and 4.21, respectively) produced a significantly different response than the milder input motions discussed above. The LC force range exceeded the test wall inertia by a significant margin for these extreme shaking levels. Pressure measurements near the top and middle (and bottom, not shown) increased significantly, in phase, near the instant of peak positive acceleration. The
largest recorded pressure increase was near the middle. Displacements of the test wall reached about 7 mm and 10 mm at the top and 3 mm and 5 mm at the bottom in those tests (Figures 4.20b and 4.21b). Similar observations were also made from the repeated 3xEM and 3.3xEM tests (not shown), but with the magnitude of the recorded LC force and pressure changes being somewhat larger.

The harmonic input excitation tests of Figures 4.22 and 4.23 help to further illustrate some of the above observations from the scaled earthquake record tests. The response to 1xHM (Figure 4.22, peak input acceleration of 0.61 g), bears significant similarities to that of the 2xEM test described above, including the phase difference between top and middle pressure readings. The excitation of 1.5xHM (Figure 4.23, peak input acceleration of 0.91 g) produced similar response characteristics to those mentioned above from the stronger 3 and 3.3xEM test, with pressure increasing simultaneously at all measured depths, and the resulting LC force significantly exceeding the test wall inertia.

4.9.3 Summary of the Results

Based on the data presented in this section, there was a distinct difference in the response to input accelerations above and below about 0.66 g. During the tests with peak input accelerations up to 0.66 g (1/3, 2/3, 1 and 2xEM, and 1xHM), the measured increases in total lateral force (LC force) could be mostly accounted for by the test wall inertia. Based on the PT measurements in those tests, the lack of an increase in the earth pressure component of thrust may be due to a decrease in the earth pressure near the top occurring at the same instant as an increase further below, as the test wall moved slightly away from the backfill (up to 2.5 mm near the top of the wall). In the stronger excitation
tests with acceleration peaks above 0.66 g (3 and 3.3xEM, and 1.5xHM), the earth pressure increased simultaneously along the full supported backfill height, resulting in total lateral force (LC force) measurements that clearly exceeded the inertia of the test wall.

4.10 Analysis of the Test Results

4.10.1 Backfill Strength and Stiffness

The dense silty sand backfill of these experiments was rather strong, with a cohesion intercept $c = 14$ kPa and friction angle $\phi = 46$ degrees (average of triaxial and direct shear test peak results, Figure 4.7). With those strength parameters and a total unit weight $\gamma = 20.6$ kN/m$^3$, the backfill soil could stand vertically on its own (without a supporting wall) up to a height $H_c$ of more than 3 meters under static active earth pressure conditions according to (Coduto 1994):

$$H_c = \frac{2c}{\gamma \sqrt{K_a}} \quad (4.1)$$

where $K_a$ is the Rankine active earth pressure coefficient. Such a strong backfill might also be expected to remain intact up to a significant level of shaking.

However, due to inertial forces during shaking, the intact backfill would be expected to deform and exert a pressure increase on the supporting wall. Nevertheless, considering the high compacted backfill stiffness in the conducted tests ($E_{50} = 48200$ kN/m$^2$ from the 144.1 kPa confining stress triaxial compression test), such deformations would be small, until substantial mobilization of the soil shear strength occurs.
4.10.2 Test Wall and Backfill Deformation Compatibility

The test wall movement and its interaction with the backfill may help to explain the recorded dynamic response, particularly in the range of input acceleration up to about 0.66 g. From Figures 4.18b through 4.23b, test wall movements during shaking were larger near the top than at the bottom. On that basis, Figure 4.24 illustrates some of the possible deformation characteristics which could be combined to describe the test wall movements during shaking.

With the frequency of strong excitations much lower than the first natural frequency of the short and stiff soil layer, a typical free-field elastic soil column response (first mode, Kramer 1996) is also included in Figure 4.24. By visual comparison, the compatibility between the different wall movement patterns and the elastic free-field soil column deformation shown in Figure 4.24 could lead to varying pressure distributions along the wall height.

As mentioned previously, numerical analyses have been conducted, which considered the dynamic interaction between flexible walls and supported elastic soil media (Veletsos and Younan 1997, Gazetas et al. 2004, Psarropoulos et al. 2005, and Jung and Bobet 2008). In those studies, when the rotational or bending flexibility was high enough, the earth pressure near the top decreased at instants when it was increasing lower along the wall (e.g., Figure 4.25). That response is similar to what was observed in the experiments with peak acceleration up to 0.66 g. Jung and Bobet (2008) also investigated the case of a translating wall and found that during shaking, earth pressure could be fully relieved near the base of the wall, while increasing near the top (Figure 4.26).
Figure 4.27 presents the recorded change in pressure (from the initial static value) at the three PT depths (Figure 4.3), at the instant when the peak LC force was measured from representative tests. In Figure 4.27a, three of the more moderate input excitation tests (1 and 2xEM and 1xHM) are shown. As in the time histories of Figures 4.18d, 4.19d, and 4.22d, the pressure decreased close to the top while it increased near the middle (Figure 4.27a). The dynamic pressure near the bottom was close to zero (Figure 4.27a). Considering that the wall was in a rotated position with slight horizontal translation at those peak LC force instants (Figures 4.18b, 4.19b, and 4.22b), a combination of the dynamic pressure distributions shown in Figure 4.25 (rotating wall) and Figure 4.26 (translating wall) may help to illustrate the experimentally observed dynamic response.

In the stronger excitation tests (Figure 4.27b), the pressure increased at all three PT depths at the instant of the peak measured load. This larger earth pressure contribution helps to explain how the recorded LC forces began to substantially exceed the test wall inertia in those tests (Figures 4.20, 4.21, and 4.23). At the strong shaking levels, the backfill shear strength was further mobilized, and a larger mass of soil began to act on the wall.

The test wall and backfill deformation compatibility discussed in this section is further investigated using FE simulations in Chapter 7.

4.10.3 Comparison with the Theoretical Predictions

The resultant dynamic force on the wall due to earth pressure, or dynamic earth thrust (ΔP<sub>eq</sub>) can also be estimated at peak instants by subtracting the static force and the
test wall inertia from the LC force (Figures 4.18 through 4.23). That value is compared against the horizontal input acceleration for the peak response of each test in Figure 4.28. The order of execution of the tests is also indicated in Figure 4.28. Estimates for $\Delta P_{eq}$ are compared with the test results in Figure 4.28 based on: i) the Wood (1973) elastic solution (assuming Poisson’s ratio $\nu = 0.3$), ii) the classical M-O active earth pressure predictions (without including the cohesion, Kramer 1996), and iii) modified M-O estimates including the effect of the soil cohesion (supplied by Earth Mechanics, Inc., Fountain Valley, CA based on NCHRP 2008). Figure 4.28 includes predictions considering the peak soil shear strength ($\phi = 46$ degrees and $c = 14$ kPa) as well as the residual ($\phi_r = 35$ degrees and $c_r = 6$ kPa), using a wall-soil friction angle $\delta = 2/3 \phi$.

From Figure 4.28, the pattern of the dynamic earth pressure component versus the input acceleration is not well represented by the linear elastic rigid wall solution of Wood (1973). A better representation is provided by the M-O predictions, which account for an increased soil wedge size with larger acceleration, resulting in a nonlinear force-acceleration relationship (Figure 4.28). However, the classical M-O prediction which neglects the soil cohesion overestimates the measured dynamic force increase from nearly all of the tests (Figure 4.28). For the first four conducted tests, the M-O with peak shear strength values including cohesion predicts a dynamic earth thrust of zero (or a negative value) which agrees well with the experimental results (Figure 4.28). In the remaining tests, that method significantly under-predicts the dynamic earth pressure when considering the peak shear strength parameters, but appears to capture the response pattern more accurately using the residual strength (Figure 4.28).
4.11 Discussion

The results presented here provide experimental evidence which may help to explain why some retaining walls have performed well during earthquakes, and to motivate a more accurate consideration of SSI effects in predicting seismic earth pressure. For instance, in the 2xEM test, with a peak input acceleration of 0.66 g, slight movement of the wall away from the backfill (about 2.5 mm of lateral displacement near the surface) appears to have limited the dynamic earth pressure increase to less than 1/20th of the Wood (1973) rigid wall prediction (Figure 4.28). This observation may be extremely valuable when considering structures which are typically analyzed as rigid or “non-yielding,” such as basement walls, if small displacements (on the order of a few millimeters, for instance) may actually be possible.

As shown in Figure 4.28, the modified M-O limit-equilibrium methods which include the effect of soil cohesion can predict a zero (or negative) active earth pressure coefficient up to a significant level of input acceleration. However, that limit equilibrium solution does not account for any increase in wall pressure which may occur due to the elastic deformation of the backfill (Figures 4.25 and 4.26). Such an increase in pressure may cause a wall to move or experience a bending moment, even when the supported backfill has not reached the plastic limit state.

In contrast, very large dynamic earth pressure thrusts occurred during the extremely strong shaking tests (with peak input accelerations on the order of 0.9 g or more). With a peak base acceleration of 1.2 g, as much as 55% of the supported backfill inertial force (mass of the top 1.7 m of backfill times the backfill acceleration) acted on
the wall. Such forces are significantly greater than static active or at-rest pressures for which retaining walls are typically designed.

An additional observation of interest is that the short duration and quick reversal of the high peak accelerations resulted in only brief instants of lateral force increase on the wall during the earthquake record input tests (e.g. Figures 4.20c and 4.21c). Consequently, if a retaining wall can sustain such a quick jolt by sliding or bending slightly, or even experiencing minor damage without reaching failure, it may be over-conservative to design based on the peak possible thrust.

Limitations imposed by large scale testing constraints such as the boundary conditions, the particular backfill soil type and density, and the limited amount of available data for analysis should also be considered along with the conclusions provided below.

4.12 Summary and Conclusions

Large scale shake table dynamic earth pressure experiments were performed with dense well-graded silty sand backfill. Along with a description of the test configuration, results were presented, analyzed, and discussed, suggesting the following observations and conclusions:

1) In the conducted tests with earthquake record input accelerations up to 0.66 g, the dynamic component of lateral earth pressure was small relative to the test wall inertial force. In that range, the test wall rotated and translated slightly away from the backfill (up to about 2.5 mm of lateral displacement at the top) as the peak force occurred. With the wall in a rotated position, the earth pressure decreased near the
top, and increased below. With that pressure distribution, the resultant lateral force was close to zero. The high stiffness and strength (including the cohesion) of the compacted backfill appear to have contributed to this type of response.

2) At larger input acceleration levels (above 0.9 g), the dynamic earth pressure thrust clearly exceeded the test wall inertial force. In those strong motion instants, the earth pressure increased along the full wall height. The soil shear strength was further mobilized, and a significant portion of the supported backfill inertia acted on the wall (55% for a 1.2 g input acceleration spike).

3) In the majority of the performed tests, the classical M-O (1929) dynamic active earth pressure equations over-predicted the recorded earth thrust. Modified M-O predictions including the effect of cohesion came closer to the measured result in the low acceleration range of earthquake record motions (up to 0.66 g of peak input acceleration) by predicting a dynamic earth thrust of zero (or a negative value).

4) With stronger shaking levels, larger portions of the backfill began to contribute to the wall pressures, resulting in a nonlinear input acceleration-dynamic earth thrust relationship, with increasing slope. By predicting an increasing active wedge size with greater input accelerations, the M-O limit equilibrium analyses account for a similar effect.

5) During some of the very strong tests (peak input acceleration above 0.9 g), a portion of the static lateral stresses was relieved by the end of the shaking. During the repeated very strong tests, larger levels of lateral earth pressure were measured than in their initial counterparts. It appears from these observations that the backfill may
have lost some of its resistance to the dynamic excitations throughout the applied test sequence.

6) With wall movements during shaking as small as 1 mm away from the strong and stiff backfill, the dynamic earth pressure resultant was significantly less than the Wood (1973) rigid wall prediction. This observation may be important to consider for structures which are typically analyzed as rigid or “non-yielding,” such as basement walls, if small displacements may actually be possible (on the order of a few mm).

7) The interaction between the test wall and the supported backfill significantly affected the magnitude and distribution of the earth pressure in the conducted tests. Based on that observation, accurate representation of such effects in analysis and design may result in more realistic predictions of the dynamic earth pressure and the wall response.

4.13 Acknowledgements

This chapter contains material from the paper published in the Bulletin of the New Zealand Society of Earthquake Engineering under the title “Full Scale Shake Table Investigation of Bridge Abutment Lateral Earth Pressure” with co-author Ahmed Elgamal. The dissertation author is the primary author and investigator of this paper.

4.14 References


Luco, J., Conte, J., Moaveni, B., Mendoza, L. and Whang, D. (2004) “Forced vibration tests of the foundation block and surrounding soil at the NEES/UCSD large high-


Table 4.1: Test summary

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<tr>
<th>Test</th>
<th>Input</th>
<th>Peak Input Value</th>
<th>Total Load Cell Force (includes the wall inertia)</th>
<th>Peak Change (from Initial)</th>
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Figure 4.1: Schematic view of soil container and restraining towers on shake table showing overall dimensions

Figure 4.2: Restrained laminar soil container on outdoor shake table at ESEC
Figure 4.3: Schematic elevation view inside the soil container showing test wall (supporting beam not shown), backfill, jacks, load cells, accelerometer locations (black circles), linear potentiometers (LP 1 through 9), and pressure transducers (PT).
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Figure 4.10: Base acceleration (A) and LC force (F) time histories from 2xEM

Figure 4.11: Base acceleration (A) and LC force (F) time histories from 3xEM
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Figure 4.18: Acceleration (A), displacement (D), force (F), and pressure (P) from 1xEM
(a) Acceleration

(b) Displacement

(c) Force

(d) Pressure

Figure 4.19: Acceleration (A), displacement (D), force (F), and pressure (P) from 2xEM
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Figure 4.27: Recorded dynamic component of pressure at the 3 PT depths at the instant of the peak measured LC force from: a) tests with peak input acceleration up to 0.66 g and b) tests with peak input acceleration greater than 0.9 g
Figure 4.28: Comparison of the peak lateral dynamic earth pressure thrust from the ten experiments with the theoretical predictions.
Passive pressure is often relied on as a stabilizing force for retaining walls, or to provide resistance to foundation displacements (refer to Chapters 1 through 3 for further description). Figure 5.1 illustrates how earthquake induced inertial forces in the backfill can cause the total passive resistance to deviate from the static value (Kramer 1996). Instants of decreased passive pressure (Figure 5.1a) could lead to a reduction in stability or resistance.

With that motivation, several analytical investigations have addressed the dynamic passive earth pressure problem (e.g., Okabe 1926, Davies et al. 1986, Morrison and Ebeling 1995, Soubra 2000, Kumar 2001, Choudhury and Nimbalkar 2005, Rao and Choudhury 2005). Due to the lack of experimental data on this topic, validation of these analytical is studies often limited to comparisons with other similar investigations (Figure 5.2), with the goal of obtaining a lower passive pressure prediction (Kumar 2001, Choudhury and Nimbalkar 2005, Rao and Choudhury 2005).

Dynamic passive earth pressure predictions typically correlate the peak resistance with a measure of the input acceleration (e.g., Figure 5.2). However, it would be difficult to perform an experiment where a fixed backfill acceleration level was maintained for long enough to fully mobilize and accurately record the corresponding peak passive resistance. Considering that difficulty and the lack of experimental data, the soil
container-test wall-backfill test configuration of Chapters 2 and 4 was also employed to conduct 2 simplified series of dynamic passive pressure experiments.

Similar to the passive pressure load-displacement tests of Chapter 2, the test wall was pushed into the backfill first, until a portion of the peak resistance was mobilized. The jacks were then locked, and a series of scaled shake table motions was imparted on the system. In that configuration, effects of dynamic excitation on the mobilized passive resistance were recorded. These “passive condition” dynamic tests are presented in the remainder of this chapter. A final series of “passive failure condition” dynamic tests is presented in the following chapter.

5.2 Initial Conditions and Test Sequence

After conducting the “at-rest condition” dynamic tests of Chapter 4, the wall was pushed using the hydraulic jacks (similar to the tests described in Chapter 2 as mentioned above) into the backfill in order to mobilize a portion of the peak passive resistance. Part of that loading process had to be repeated due to an accidental loss of the mobilized pressure. During that re-loading phase, the force-displacement relationship was recorded (Figure 5.3). The jacks were locked with that level (Figure 5.3) of passive resistance initially mobilized behind the test wall.

Next a series of scaled versions of the earthquake record (EM) and harmonic input (HM) motions was performed, while the jacks remained locked as described above. Table 5.1 presents the conducted tests series in the order of execution, and the peak recorded base acceleration(A) in each direction (Figure 5.4), velocity (V) and displacement (D) from each test (velocity and displacement were estimated by integration
of the base acceleration data). Aside from the initial wall displacement and pre-loaded backfill condition (Figure 5.3), the testing configuration was the same as that of the at-rest condition dynamic tests (see Chapter 4).

5.3 Test Results

5.3.1 Lateral Force Measurements

The base acceleration and LC force (total lateral force behind the wall, per meter of width) are presented as time histories from the 1, 2, and 3 times the EM tests, and the 1 and 1.5 times the HM tests in Figures 5.5 through 5.9. As explained in Chapter 4, the load cell (LC) force values represent the lateral earth pressure resultant plus the test wall inertia during shaking, and the static lateral earth pressure resultant before and after the excitation (when the inertia of the wall is zero).

Similar to the at-rest condition tests of Chapter 4, force increases were mild in the 1/3, 2/3 (not shown), 1 (Figure 5.5), and 2xEM (Figure 5.6) tests. During the stronger 3xEM test (peak positive direction input acceleration of 1 g), a much larger force increase occurred, at just after 15 seconds in Figure 5.7.

In contrast to the at-rest condition tests, much greater force decreases occurred in these tests (Figures 5.5 through 5.9), as a portion of the mobilized passive resistance was temporarily relieved at instants during shaking. A permanent LC force reduction also occurred by the end of shaking in each of the passive condition tests, indicating a loss in the static earth pressure level.

Table 5.1 highlights the recorded LC force data from each of the 7 conducted tests, including the initial, final, maximum, and minimum value, and the peak increase
and decrease from the initial (static) value. Figures 5.10 and 5.11 also compare the peak LC force increase and decrease (per meter of wall width) with the corresponding input acceleration from each test, with the order of execution of the tests indicated next to each result.

Similar to the at-rest condition test results of Chapter 4, Figure 5.10 illustrates how the force increases were relatively low in the 1/3, 2/3, and 1xEM tests, but disproportionally larger thrusts were measured during the very strong 3xEM and 1.5xHM tests (with peak positive direction input acceleration of 1 and 0.94 g, respectively). From Figure 5.11, stronger acceleration (in the negative direction according to Figure 5.4) also resulted in greater decreases in LC force.

### 5.3.2 Representative Time-History Results

A detailed set of the experimental data is presented for the 1, 2, and 3xEM, and 1 and 1.5xHM tests in Figures 5.12 through 5.16. In order to focus on key aspects of the response, these time histories contain a shortened section of the tests (10 seconds). Force (F) values are normalized per meter of wall width.

In Figures 5.12 through 5.16, the base (input) and backfill surface (HA 1, Figure 4.3) accelerations are included at the top (a). Next (b), displacements are approximated by double integrating and filtering (bandpass filter from 0.4 to 120 Hz) accelerometer data. Using that method, the total base (input) displacement is presented on the left, and the displacement near the top and bottom of the test wall (Figure 5.4), relative to the base, are shown on the right. Next, on the left side (c), the total force measured by the load cells (LC force) is presented. Below that, the inertia of the test wall is represented by its
mass times its acceleration. On the bottom of the left-hand column, the test wall inertia is removed (subtracted) from the LC force, as an estimate of the lateral earth pressure resultant force. To the right (d), the pressure transducer (PT, Figure 5.4) readings near the top, middle and bottom of the wall height (0.82H, 0.45H, and 0.08H) are presented.

Similar to the at-rest condition test, during the passive condition 1xEM test the wall inertia contributed significantly to the observed force variations (Figure 5.12c). At the peak LC force instant (about 15 seconds, Figure 5.12c) the pressure decreased slightly near the wall top, while increasing below (Figure 5.12d). The top of the wall moved about 1 millimeter relative to the container base, with bottom displacement being much smaller (Figure 5.12b).

In contrast with the at-rest condition 1xEM test, there was a noticeable permanent drop in the total earth pressure (Figure 5.12c and d) during the passive condition test. Pressure decreases near the middle and bottom of the wall were more significant than those near the top (Figure 5.12d).

Most notably from the stronger shaking 2xEM test (base acceleration up to 0.66 g, and displacement up to 14 cm, Figure 5.13), there was a distinct drop in pressure at all three of the instrumented depths near the instant of the peak acceleration in the negative direction (according to Figure 5.4). Static pressure levels also experienced a significant permanent drop close to the bottom of the test wall (Figure 5.13d). Peak wall movements were on the order of 2.5 millimeters at the top and 1 millimeter at the bottom.

With peak base acceleration of 1 g, a much more significant increase in earth pressure occurred in the 3xEM test (Figure 5.14), compared with those presented above.
The largest increase was measured by the bottom pressure sensors, followed by the middle, with the smallest increase near the top (Figure 5.14). A significant pressure decrease also occurred (between 14 and 15 seconds), with the distribution following a similar pattern to that of the increase (Figure 5.14).

Similar to the 2 and 3xEM tests, pressure increases and decreases were in phase at all three instrumented depths during the 1 and 1.5xHM tests of Figures 5.15 and 5.16. Consequently, the earth pressure resultant went through increasing and decreasing cycles throughout the excitation (Figures 5.15c and 5.16c). Pressure increases and decreases were largest near the wall bottom, followed by the middle, with the smallest changes recorded near the top.

5.3.3 Dynamic Pressure Distribution

Figure 5.17a illustrates the distribution of pressure increases (change from the initial value) along the test wall height, at the instant of the peak LC force measurement. Figure 5.17b illustrates the distribution of pressure decreases at the instant of the minimum LC force measurement. The tests which produced the most significant pressure variations are included in these figures. In contrast with the dynamic pressure distribution from the at-rest condition tests (Figure 4.27), in Figure 5.17, both the increases and decreases in pressure show a roughly triangular distribution, with the magnitude of the change increasing with depth.

5.3.4 Backfill Surface Displacements

Backfill surface displacements (adjusted to start from zero for each test) from the 9 linear potentiometers (LP, Figure 5.4) are presented from the 2 and 3xSTM tests in
Figures 5.18 and 5.19, respectively. During those tests, the backfill surface closest to the wall (LP 1 through LP 4) settled, while further from the wall (LP 6 through LP 9), it heaved. Figure 5.20 shows the vertical displacement profile from before and after the full conducted dynamic test series. The pattern of deformation depicted in Figure 5.20 suggests a trend of gradual leveling of the backfill surface throughout the series of shaking events. Backfill surface displacement time histories are also included in Appendix D of this dissertation.

5.3.5 Summary of the Test Results

From the data presented in this section, the relationship between the increase in LC force and the input acceleration (Figure 5.10) is similar to that of the at-rest condition tests. Specifically, the increases were relatively minor up to about 0.6 to 0.7 g (Figures 5.5, 5.6, and 5.8), but with larger acceleration peaks, the LC force increased substantially (Figures 5.7 and 5.9).

In contrast with the at-rest condition tests, significant temporary and permanent decreases in the LC force and the pressure occurred. Earth pressure increases and decreases on the wall were largest closer to the bottom (Figure 5.17), in contrast with the at-rest condition tests, where pressure increases were larger closer to the middle.

Similar test wall movements to those of the at-rest condition tests were observed in the passive condition tests. However the phase-difference effect discussed in Chapter 4 (decreasing pressure near the wall top with increasing pressure below) was only slightly noticeable in the 1xEM (and weaker) tests (e.g., Figure 5.12). In the remaining stronger
tests (Figures 5.13 through 5.16), pressure increases and decreases occurred concurrently at all 3 PT depths (Figure 5.4) near the instant of peak response.

The section of backfill near the test wall, which had experienced the most significant vertical heave during the initial passive pressure mobilization phase (Figure 5.3), settled slightly throughout the strong shaking (Figure 5.20). The section of backfill further from the test wall heaved slightly throughout the strong shaking (Figure 5.20).

5.4 Comparison with the Initial and Peak Passive Forces

Table 5.2 lists the peak increase and decrease in the LC force as a percentage of the initial value from each test and of the peak passive resistance of 326 kN per meter of width measured with a static backfill in Test 2 (Chapter 2). From Figure 4.17, the largest increase in the passive force was 29% of the initial value, and 18% of the peak passive resistance. The largest decreases were 32% of the initial value, and 23% of the peak passive resistance.

5.5 Mononobe-Okabe Prediction

As explained in Chapter 1, the Mononobe-Okabe (M-O) equations (1926, 1929) provide an estimate of the peak dynamic passive pressure based on an extension of the Coulomb (1776) static prediction which includes a pseudo-static backfill inertial force (Kramer 1996). Using Equation (1.9) and (1.10) to predict the peak dynamic passive resistance, and subtracting that prediction from the static value (Coulomb prediction, or M-O with input acceleration $k_h = k_v = 0$), Figure 5.21 presents the predicted change in the
peak passive pressure due to shaking (with friction angle $\phi = 46$ degrees and wall-soil interface friction $\delta = 0$).

In the conducted tests, the peak passive pressure was not mobilized during shaking, so a direct comparison with the M-O peak resistance prediction in Figure 5.21 may not be appropriate. However some qualitative observations can be made from Figure 5.21 as follows: i) by including the inertia of the failure wedge, the M-O prediction results in lower peak passive pressure with increasing magnitude acceleration, and ii) with increasing magnitude acceleration, the M-O equations predict a larger passive failure wedge and results in a nonlinear force reduction-acceleration relationship. Similar patterns of reduction in the mobilized passive resistance occurred in the experiments, as shown in Figure 5.11.

5.6 Effect on the Load-Displacement Response

In addition to the decrease in capacity, the backfill inertial forces may influence the force-displacement behavior, as illustrated in Figure 5.22. The upper and lower bound curves of Figure 5.22 were estimated by imposing the measured changes in mobilized pressure for 1 g of input acceleration over the Test 2 backbone load-displacement curve (Chapter 2). In structures such as bridge abutments, in which passive earth pressure provides resistance to the deck displacement during an earthquake, such a variation in the available resistance may have a substantial impact on the dynamic response. A simplified strategy for modeling this inertial effect is discussed later in Chapter 6.
5.7 Summary and Conclusions

The results of the conducted passive condition dynamic earth pressure experiments with dense, well-graded silty sand backfill suggest the following observations and conclusions:

1) With peak input acceleration levels of up to 0.6 g, increases and decreases of only about 5% of static peak passive resistance occurred. However, increases and decreases of about 30% of the mobilized static passive resistance occurred, with input accelerations of about 1 g.

2) Based on the test results, the instantaneous level of passive resistance available for providing stability (e.g. sheet pile anchor wall, or shallow side of a cantilever wall) and resisting structure movements (e.g. bridge abutments and pile caps) could substantially depend on the magnitude and direction of the excitation at that instant.

3) The dynamic component of earth pressure (change from the initial value) had a roughly triangular distribution, with the magnitude of the change increasing with depth.

4) A portion of the mobilized static passive stress within the backfill was relieved during the strong shaking phase. After the shaking event, the static lateral load decreased by as much as 15%. The permanent passive pressure losses were typically more significant at greater depth.

5) The section of backfill near the test wall, which had experience the most significant vertical heave during the initial passive pressure mobilization phase, settled slightly
throughout the strong shaking. The section of backfill further from the test wall heaved slightly throughout the strong shaking.

6) Additional testing in which the peak passive resistance is mobilized and recorded during shaking would be highly desirable to further quantify the dynamic passive earth pressure and for comparison with the existing analytical theories.

5.8 Acknowledgements

This chapter contains material from the paper published in the Bulletin of the New Zealand Society of Earthquake Engineering under the title “Full Scale Shake Table Investigation of Bridge Abutment Lateral Earth Pressure” with co-author Ahmed Elgamal. The dissertation author is the primary author and investigator of this paper.

5.9 References


Table 5.1: Test summary

<table>
<thead>
<tr>
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Table 5.2: Peak LC force changes compared with the initial mobilized force and the peak passive resistance from Test 2 (Chapter 2)

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Figure 5.1: Effect of the backfill inertia on the passive resistance

Figure 5.2: Comparison of predicted dynamic passive earth pressure coefficient $K_{pe}$ for different backfill acceleration levels $k_h$ and $\phi$ values (from Choudhury and Nimbalkar 2005)
Figure 5.3: Reloading of test wall to mobilize passive pressure

Figure 5.4: Schematic elevation view inside the soil container showing test wall, backfill, jacks, load cells, accelerometer locations (black circles), linear potentiometers (LP 1 through 9), and pressure transducers (PT)
Figure 5.5: Base acceleration (A) and LC force (F) time histories from 1xEM

Figure 5.6: Base acceleration (A) and LC force (F) time histories from 2xEM

Figure 5.7: Base acceleration (A) and LC force (F) time histories from 3xEM
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Figure 5.9: Base acceleration (A) and LC force (F) time histories from 1.5xHM
Figure 5.10: Peak increase in LC force versus input acceleration from the 7 tests (the numbers indicate the order of execution)

Figure 5.11: Peak decrease in LC force versus input acceleration from the 7 tests (the numbers indicate the order of execution)
Figure 5.12: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 1xEM
Figure 5.13: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 2xEM
Figure 5.14: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 3xEM
(a) Acceleration

(b) Displacement

(c) Force

(d) Pressure

Figure 5.15: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 1xHM
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Figure 5.17: Recorded change in pressure (from the initial value) at the 3 PT depths: a) at the instant of the peak measured LC force, and b) at the instant of minimum LC force.
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Chapter 6 Large Scale Shake Table Dynamic Earth Pressure

Experiments: Passive Failure Condition

6.1 Introduction

Shearing of dense sand can result in dilation and a reduction in strength along a localized band, or failure plane (Schanz and Vermeer 1996, Terzaghi et al. 1996, Alshibli and Sture 2000), as demonstrated in the passive pressure experiments of Chapter 2. After formation of a failure plane, the soil within that localized band may have reduced (“residual” or “critical state”) friction angle $\phi_r$ and cohesion $c_r$ values (e.g., Figure 4.8). Meanwhile, the surrounding soil can still be much stronger with peak $\phi$ and $c$ (e.g., Figure 4.7).

Based on that logic, Koseki et al. (1998) suggested that a previously formed active failure wedge might exhibit lower shear resistance than the surrounding backfill soil. In shake table tests, Koseki et al. (1998) found that the plane formed by that initial wedge controlled the consecutive mobilization of dynamic earth pressure until shaking levels were strong enough to form a new, larger wedge in the stronger surrounding backfill (Figure 6.1).

Similarly, if a passive failure wedge (e.g., Figures 2.8 through 2.10) is mobilized in the backfill, the plane formed at the bottom of that wedge (where $\phi = \phi_r$ and $c = c_r$) might dictate the subsequent dynamic response. In consideration of this possible influence on the dynamic passive resistance, a third and final series of shake table tests is presented in this chapter. Similar to the tests presented in Chapter 5, the effects of
dynamic excitation on the mobilized passive resistance were recorded. However in this
test series a passive failure wedge was fully mobilized before the dynamic excitations
were executed as described below. These “passive failure condition” dynamic tests are
presented and discussed in the remainder of this chapter.

6.2 Initial Conditions and Test Sequence

First, the hydraulic jacks were used to push the test wall into the compacted
backfill beyond the level of peak passive resistance (Figure 6.2). That initial loading
phase is also the passive pressure load-displacement Test 2 of Chapter 2. As such,
backfill construction details (dense well-graded silty sand backfill at 95% relative
compaction) are described in Chapter 2.

With the test wall displaced laterally about 120 mm into the backfill (Figure 6.2),
a crack had formed on the surface (Figure 6.3) indicating the end of the mobilized failure
wedge shown in Figure 6.4. The jacks were locked with the test wall in that position.
Figure 6.4 also shows the backfill horizontal (HA) and vertical (VA) accelerometer
layout employed for the passive failure condition test series.

Next a series of scaled earthquake record (EM) and harmonic input (HM) motions
was performed while the jacks remained locked as described above. Table 6.1 presents
the order of the conducted test series, and the peak recorded base acceleration (A) in the
positive and negative directions (Figure 6.4), velocity (V) and displacement (D) from
each test (velocity and displacement were estimated by integration of the base
acceleration data).
6.3 Test Results

6.3.1 Lateral Force Measurements

The base acceleration and load cell (LC) force (total lateral force measured behind the test wall, per meter of wall width) time histories are presented from the 1, 2, 3, 3.3, and 3.5xEM, and the 1 and 1.5xHM tests in Figures 6.5 through 6.11. During the 1xEM test of Figure 6.5 (peak input acceleration of about +/- 0.35 g) the LC force increased and decreased mildly, in a pattern similar to that of the base acceleration. A similar pattern of response was observed in the weaker 1/3 and 2/3xEM tests (not shown).

Similar to the passive condition dynamic tests, significant temporary drops in the passive resistance occurred as the input motions became stronger (Figures 6.6 through 6.11). Large permanent reductions in the LC force were also observed during strong shaking. Permanent reductions were most noticeable in the tests with strong shaking and higher initial stress conditions (e.g. Figures 6.6 and 6.7).

Highlights of the LC force histories from each of the 9 conducted tests are summarized in Table 6.1, including the initial, final, maximum, and minimum value, and the peak increase and decrease from the initial (static) value. Figures 6.12 and 6.13 also compare the peak LC force increase and decrease (per meter of wall width), respectively, with the corresponding input acceleration from each test. The order of execution of the tests is also indicated in Figures 6.12 and 6.13.

The trend in Figure 6.12 indicates that the peak LC force increase was generally larger for greater input acceleration levels. Similar to the at-rest and passive condition test results, the relationship between the increase in LC force and the input acceleration
also became steeper at the stronger input levels. Similarly, the trend in Figure 6.13 indicates that the peak LC force decrease was generally larger for greater input acceleration levels. However from Table 6.1 (and Figure 6.13), the largest measured LC force decrease occurred in the 3xEM test, which had a higher initial lateral stress condition, but did not have the strongest input motion.

### 6.4 Comparison with the Initial and Peak Passive Forces

Table 6.2 lists the peak increase and decrease in the LC force as a percentage of the initial value from each test, and of the peak passive resistance of 326 kN per meter of width measured with a static backfill in Test 2 (Chapter 2). From Table 6.2, the largest increase in the passive force was 106% of the initial value, and 26% of the peak passive resistance. The largest decreases were 54% of the initial value, and 16% of the peak passive resistance.

### 6.5 Backfill Dynamic Response

In order to consider the effect of the existing localized shear band with reduced strength parameters $\phi = \phi_r$ and $c = c_r$ (Figure 6.4), the backfill dynamic response is compared at locations inside and outside of the passive failure wedge in Figures 6.14 through 6.20. The purpose of this comparison is to determine if the pre-existing weak failure plane influenced the dynamic response during these tests, similar to the Koseki et al. (1998) observations.

In Figures 6.14 through 6.20, acceleration (a) and displacement (b) measurements are compared at locations inside (left hand column) and outside (right hand column) of
the passive failure wedge described in Figure 6.4. With reasonable consistency from those figures, horizontal accelerations were slightly larger, vertical accelerations were significantly larger, and vertical backfill surface displacements were larger inside than outside the failure wedge. For instance, the peak vertical accelerations in Figures 6.14a through 6.20a at VA1 (within the failure wedge as shown in Figure 6.4), were on the order of 2 to 10 times those at VA5 (outside the failure wedge as shown in Figure 6.4).

As an additional illustration of the difference in response, sudden vertical backfill displacements on the order of 3 millimeters occurred within the wedge (LP2 and LP5, Figure 6.4), compared with movements of less than 1 millimeter outside the wedge (LP8 and LP9, Figure 6.4) during the 3xEM test of Figure 6.16b.

From Figures 6.14b through 6.20b, noticeable settlements also occurred inside the failure wedge, while outside that region they did not. For instance in Figure 6.16b, almost 4 millimeters of settlement occurred at LP2 and LP5 (within the failure wedge as shown in Figure 6.4). In contrast, no significant backfill settlement was observed at LP8 and LP9 (outside the failure wedge as shown in Figure 4.3).

It is also noted from the HM tests with sweeping frequency (Figures 6.19a and 6.20a) that the backfill vertical acceleration appears to have been significantly amplified at around 18 seconds, particularly at VA1.

From the comparison presented in Figures 6.14 through 6.20, the backfill dynamic response was much more noticeable within the failure wedge of Figure 6.4, than outside that region. These results indicate that the weakened band of soil ($\phi = \phi_r$ and $c = c_r$), at bottom of the previously mobilized passive failure wedge contributed to the increased response as indicated schematically in Figure 6.21.
Considering the interaction depicted in Figure 6.21, the change in passive resistance might be controlled by the inertia of the fixed-size failure wedge within a range of input acceleration. In Figure 6.22, the reduction in horizontal passive resistance due to the inertial force in the 70 kN/m (per meter of width) failure wedge is compared with the Mononobe Okabe prediction from Chapter 5. The fixed-size wedge would result in a linear correlation between the reduction in passive resistance and the backfill acceleration as illustrated in Figure 6.22.

This differs from the Mononobe-Okabe prediction, which considers the static passive wedge shape for the input acceleration \( k_h = 0 \) (static backfill) case, but accounts for a larger failure wedge with increasing backfill acceleration, resulting in a nonlinear passive force reduction-input acceleration relationship (Figure 6.22). However, the predicted reduction in passive resistance for the fixed-wedge size is relatively close to the M-O prediction up to a significant level of backfill acceleration (Figure 6.22).

### 6.6 Comparison with the Passive Condition Test Results

Figures 6.23 and 6.24 compare the peak increase and decrease in LC force measurements, respectively, versus input acceleration, from the passive and the passive failure condition dynamic tests. In Figure 6.23, the LC force increase-input acceleration relationship is similar from both test series, with increasing slope at stronger shaking levels.

However notable differences in the LC force decrease-input acceleration relationship from the two test series include: i) during the tests with the strongest input acceleration levels (magnitude greater than about 0.65 g), noticeably greater force
decreases occurred in the passive condition tests than in the passive failure condition
tests, and ii) a straight line would more closely account for the force decrease-input
acceleration relationship for the passive failure condition tests, whereas a curved line
would better represent that of the passive condition tests. As such, the passive failure
condition test results more closely resemble the force decrease according to the fixed-size
wedge inertia in Figure 6.22, while the passive condition test results are in better
agreement with the curved shape of the M-O prediction.

6.7 Spring-Mass Model Representation

In the case of a fixed-size failure wedge, a simple spring-mass model might
account for the effect of the failure wedge inertia in addition to the static passive force-
displacement resistance (Figure 6.25). Figure 6.25 describes how the static backfill
passive force-displacement resistance could be accounted for using a nonlinear spring,
such as the hyperbolic model curves presented in Chapters 2 and 3. The backfill inertial
force (and possibly also the wall inertia) could be approximated by applying the ground
acceleration to a connected mass (Figure 6.25), determined according to the expected
passive failure wedge geometry.

6.8 Backfill Surface Displacements

Figure 6.26 shows the backfill vertical displacement profile before and after the
conducted dynamic test series. From Figure 6.26, the backfill surface settlement was
clearly greater within the passive failure wedge region, closer to the test wall. A
photograph of the backfill surface (Figure 6.27) was also taken after the dynamic test
series was completed. From a comparison of Figures 6.3 and 6.27, the backfill surface crack indicating the end of the passive failure wedge before the start of the shaking events remained visible at the end of the dynamic test series.

### 6.9 Passive Resistance from Static Backfill after Dynamic Test Series

After completion of the series of dynamic excitations, the hydraulic jacks were used to push the test wall further into the static backfill. The recorded load-displacement behavior from that phase is presented in Figure 6.28. From Figure 6.28, the passive resistance increased before leveling off at the residual resistance, without experiencing any post-peak strain softening behavior. Figure 6.29 shows the enlarged surface crack at the end of the initial passive wedge (Figure 6.4), after the test wall was pushed further into the static backfill (Figure 6.28). These test results indicate that upon re-loading, the initially mobilized passive failure wedge continued to slide up the weak, already softened ($\phi = \phi_r$ and $c = c_r$) shear band (Figure 6.21).

### 6.10 Summary and Conclusions

The results of the conducted passive failure condition dynamic earth pressure experiments with dense, well-graded silty sand backfill suggest the following observations and conclusions:

1) With peak input acceleration levels of up to 0.6 g, increases and decreases of only about 5% of the static peak passive resistance occurred. However, with peak input acceleration of 1.3 g, increases and decreases of as much as 100% and 50% (respectively) of the initially mobilized static passive resistance were recorded.
2) The recorded motions within the previously mobilized passive failure wedge were significantly greater than those outside that region.

3) When lateral load was re-applied to the static backfill after the end of dynamic testing, the previously mobilized wedge resumed sliding up the pre-existing weakened failure surface.

4) The above response indicates that the weak passive failure plane with $\phi = \phi_r$ and $c = c_r$ remained weaker than the surrounding backfill soil throughout the test series, and influenced the response to dynamic excitation.

5) In order to approximate the effects of the backfill inertia on the passive force-displacement resistance, a spring-mass model could be employed. In that model, the static backfill passive force-displacement resistance is accounted for using a nonlinear backbone curve (e.g., Chapters 2 and 3), and the backfill inertia effect is applied by assigning a ground motion history to a connected mass representing the mass of the contributing backfill soil (and possibly also the wall).

6) In strong aftershock events, a reduced passive resistance may also occur due to a previously mobilized wedge in the backfill.

6.11 References


*Geotechnique*, 46(1), 145-151.

### Table 6.1: Test summary

<table>
<thead>
<tr>
<th>Test Input</th>
<th>Peak Input Value</th>
<th>Total Load Cell Force (includes the wall inertia)</th>
<th>Peak Change (from Initial)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A (positive) g</td>
<td>A (negative) g</td>
<td>V m/s</td>
</tr>
<tr>
<td>1.08 m/s</td>
<td>-1.18 g</td>
<td>0.12 g</td>
<td>0.02</td>
</tr>
<tr>
<td>1.75 m/s</td>
<td>-2.06 g</td>
<td>0.21 g</td>
<td>0.04</td>
</tr>
<tr>
<td>2.96 m/s</td>
<td>-3.63 g</td>
<td>0.37 g</td>
<td>0.07</td>
</tr>
<tr>
<td>7.03 m/s</td>
<td>-6.77 g</td>
<td>0.79 g</td>
<td>0.15</td>
</tr>
<tr>
<td>10.20 m/s</td>
<td>-10.50 g</td>
<td>1.16 g</td>
<td>0.22</td>
</tr>
<tr>
<td>6.41 m/s</td>
<td>-6.28 g</td>
<td>0.77 g</td>
<td>0.14</td>
</tr>
<tr>
<td>9.20 m/s</td>
<td>-9.81 g</td>
<td>1.13 g</td>
<td>0.20</td>
</tr>
<tr>
<td>8.58 m/s</td>
<td>-11.77 g</td>
<td>1.27 g</td>
<td>0.24</td>
</tr>
<tr>
<td>12.31 m/s</td>
<td>-12.75 g</td>
<td>1.34 g</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### Table 6.2: Peak LC force changes compared with the initial mobilized force and the peak passive resistance from Test 2 (Chapter 2)

<table>
<thead>
<tr>
<th>Test Input</th>
<th>Initial LC Force (kN/m)</th>
<th>Peak Acc. (+) (g)</th>
<th>Peak Acc. (-) (g)</th>
<th>% Change in LC Force Compared with Initial Value</th>
<th>% Change in LC Force Compared with Peak Passive Resistance (326 kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/3 x EM</td>
<td>147.5</td>
<td>0.11</td>
<td>-0.12</td>
<td>1.9 Increase 1.4 Decrease</td>
<td>0.9 Increase 0.6 Decrease</td>
</tr>
<tr>
<td>2/3 x EM</td>
<td>147.5</td>
<td>0.18</td>
<td>-0.21</td>
<td>3.1 Increase 2.8 Decrease</td>
<td>1.4 Increase 1.3 Decrease</td>
</tr>
<tr>
<td>1 x EM</td>
<td>147.5</td>
<td>0.30</td>
<td>-0.37</td>
<td>4.6 Increase 6.8 Decrease</td>
<td>2.1 Increase 3.1 Decrease</td>
</tr>
<tr>
<td>2 x EM</td>
<td>145.9</td>
<td>0.72</td>
<td>-0.69</td>
<td>11.4 Increase 24.6 Decrease</td>
<td>5.1 Increase 11.0 Decrease</td>
</tr>
<tr>
<td>3 x EM</td>
<td>126.7</td>
<td>1.04</td>
<td>-1.07</td>
<td>32.9 Increase 41.8 Decrease</td>
<td>12.8 Increase 16.2 Decrease</td>
</tr>
<tr>
<td>1 x HM</td>
<td>101.2</td>
<td>0.65</td>
<td>-0.64</td>
<td>26.3 Increase 20.0 Decrease</td>
<td>8.2 Increase 6.2 Decrease</td>
</tr>
<tr>
<td>1.5 x HM</td>
<td>96.1</td>
<td>0.94</td>
<td>-1.00</td>
<td>46.4 Increase 35.7 Decrease</td>
<td>13.7 Increase 10.5 Decrease</td>
</tr>
<tr>
<td>3.3 x EM</td>
<td>85.8</td>
<td>1.13</td>
<td>-1.20</td>
<td>82.6 Increase 45.7 Decrease</td>
<td>21.7 Increase 12.0 Decrease</td>
</tr>
<tr>
<td>3.5 x EM</td>
<td>80.9</td>
<td>1.25</td>
<td>-1.30</td>
<td>106.3 Increase 53.6 Decrease</td>
<td>26.4 Increase 13.3 Decrease</td>
</tr>
</tbody>
</table>
Figure 6.1: Initial active failure wedge formed with ground acceleration $k_h = 0$ (static active wedge) and secondary wedge formed with $k_h = 0.62$ g (from Koseki et al. 1998)

Figure 6.2: Backbone passive pressure force-displacement curve (from Test 2, Chapter 2) showing the initial condition for the passive failure condition tests
Figure 6.3: Side view photograph showing location of backfill surface crack indicating the end of the mobilized passive failure wedge (taken after the wall was pushed into the backfill and before the dynamic excitation tests)
Figure 6.4: Schematic elevation view inside the soil container showing estimated passive failure wedge shape, accelerometer locations (black circles) and linear potentiometer locations (LP 2 through 9)
Figure 6.5: Base acceleration (A) and LC force (F) time histories from 1xEM

Figure 6.6: Base acceleration (A) and LC force (F) time histories from 2xEM

Figure 6.7: Base acceleration (A) and LC force (F) time histories from 3xEM
Figure 6.8: Base acceleration (A) and LC force (F) time histories from 1xHM

Figure 6.9: Base acceleration (A) and LC force (F) time histories from 1.5xHM
Figure 6.10: Base acceleration (A) and LC force (F) time histories from 3.3xEM

Figure 6.11: Base acceleration (A) and LC force (F) time histories from 3.5xEM
Figure 6.12: Peak increase in LC force versus input acceleration from the 7 tests (the numbers indicate the order of execution)

Figure 6.13: Peak decrease in LC force versus input acceleration from the 7 tests (the numbers indicate the order of execution)
Inside the Failure Wedge  

Outside the Failure Wedge

(a) Acceleration

(b) Displacement

Figure 6.14: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 1xEM
Inside the Failure Wedge  

Outside the Failure Wedge

(a) Acceleration

(b) Displacement

Figure 6.15: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 2xEM
Figure 6.16: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 3xEM
Inside the Failure Wedge

Outside the Failure Wedge

(a) Acceleration

(b) Displacement

Figure 6.17: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 3.3xEM
Figure 6.18: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 3.5xEM
Inside the Failure Wedge  

Outside the Failure Wedge

(a) Acceleration

(b) Displacement

Figure 6.19: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 1xHM
Inside the Failure Wedge    Outside the Failure Wedge

(a) Acceleration

(b) Displacement

Figure 6.20: Acceleration (A), displacement (D), force (F), and pressure (P) measurements from 1.5xHM
Figure 6.21: Schematic view of backfill inside soil container showing possible influence of weakened band of soil on the dynamic response

Figure 6.22: Mononobe Okabe predicted reduction in the peak passive pressure compared with the inertia of the experimental mobilized failure wedge
Figure 6.23: Comparison of the peak increase in LC force versus input acceleration from the passive and the passive failure condition dynamic tests

Figure 6.24: Comparison of the peak decrease in LC force versus input acceleration from the passive and the passive failure condition dynamic tests
Figure 6.25: Spring-mass model schematic to account for static passive force-displacement resistance in addition to failure wedge inertia

Figure 6.26: Backfill surface displacement profiles before and after conducted shake table tests
Figure 6.27: Photograph showing the location of the backfill surface crack after completion of the dynamic test series

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Figure 6.28: Passive earth pressure force-displacement data from final push into static backfill after completion of the dynamic test series
Figure 6.29: Photograph showing surface crack after final push into static backfill
Chapter 7  Dynamic Earth Pressure Simulations

7.1  Introduction

Finite element (FE) simulations offer a valuable method for investigating dynamic earth pressure and the retaining wall-backfill interaction. In contrast with large scale experiments, FE simulations can easily be repeated, with minor or major changes to the configuration in order to highlight important aspects of the response. In this chapter, key observations from the presented large scale experiments (Chapters 4 through 6) are further demonstrated and verified using FE model simulations.

From the at-rest condition dynamic earth pressure experiments of Chapter 4, distinct differences in response occurred with input acceleration levels above and below about 0.7 g. With input excitations below that level, the response was heavily influenced by the wall-soil interaction. At the instants of the peak load cell force increase from those tests, the wall moved away from the backfill by rotating and slightly translating, and the pressure decreased near the top while increasing below (Figures 4.18, 4.19, and 4.22). Additionally, due to the high shear strength including soil cohesion, the backfill essentially remained intact during those tests.

Numerical investigations using models with linear elastic soil domains have shown that including the wall rotation, bending, or translation (e.g., Figures 7.1 through 7.5) can significantly affect the magnitude and distribution of dynamic earth pressure (Veletsos and Younan 1997, Gazetas et al. 2004, Psarrapolous et al. 2005, Jung and Bobet 2008). Similar to the above numerical studies, elastic soil model FE simulations are conducted first, focusing on the effect of various wall movements (Figure 7.1) on the
magnitude and distribution of the simulated dynamic earth pressure. Results from the FE simulations are compared with the experimental data from tests with peak input acceleration up to 0.7 g.

During the at-rest condition experiments (Chapter 4) with peak input acceleration above 0.7 g, the backfill shear strength was further mobilized, and much larger dynamic earth pressure increases occurred (Figures 4.20, 4.21, and 4.23). In those tests, the nonlinear soil shear behavior appears to have played an increasingly dominant role. In order to further investigate that response, FE simulations are conducted with input accelerations up to 1.2 g, using a nonlinear elastic-plastic soil material model. The magnitude and distribution of the simulated dynamic earth pressure are compared with the experimental data.

Finally, in the dynamic tests conducted with a portion of the peak passive resistance mobilized behind the wall (Chapters 5 and 6), significant instantaneous pressure decreases occurred, in addition to noticeable permanent reductions. Nonlinear elastic-plastic soil model FE simulations are performed with passive pressure first mobilized in the backfill mesh, in order to verify these response characteristics.

### 7.2 Employed Software Tools

The FE modeling and computations were performed using the Pacific Earthquake Engineering Research Center (PEER) FE analysis code, Open System for Earthquake Engineering Simulations (OpenSees, http://opensees.berkeley.edu). OpenSees is an open source, object-oriented nonlinear FE analysis framework, which allows the integration of new components in addition to the existing libraries without modifying existing code.
Three additional software packages were used in the FE study. GiD (http://gid.cimne.upc.es/) assisted in creating the mesh coordinates. Tcl (http://www.tcl.tk/) scripts were written to assign the model geometry, boundary conditions, response recorders, loading and excitation patterns, analysis procedures, and to initiate the FE analysis calculations. Recorded output data files were processed using Matlab (http://www.mathworks.com/) code, in order to interpret and create graphical representations of the results.

7.3 FE Model Configuration

7.3.1 Mesh and Boundary Conditions

A 130 element mesh of 2D plane strain (1 meter wide section) four-node quadrilateral elements represented the backfill soil, with dimensions set to match the experimental backfill (Figure 7.6). Elastic beam elements were used to model the test wall (Figure 7.7). Horizontal and vertical fixities were applied along the base (Figure 7.7), and the right boundary and the small soil domain beneath the model wall on the left side were also fixed in the horizontal direction.

7.3.2 Model Wall

Restrictions to the wall movement were simplified in the model using a rotational and a translational spring at the base of the beam elements (Figure 7.7). Using this simple model, rigid, bending, rotating, and translating walls can all be investigated, similar to Veletsos and Younan (1997), Gazetas et al. (2004), Psarrapolous et al. (2005), and Jung and Bobet (2008). For instance, by assigning an extremely high bending
stiffness to the wall beam elements (Figure 7.7), and also a very high value to the translational spring stiffness, and much lower rotational spring stiffness, the model wall will respond to increased lateral soil pressure by rotating. Assigning extremely high stiffness values to the bending beam, rotational spring, and translational spring will essentially result in a rigid wall response, and so on. Combinations of the different wall deformation modes can also be investigated with this model.

7.3.3 Wall-Soil Interface

Zero length elements (Mazzoni et al. 2006) lined the left and right hand vertical backfill domain boundaries (Figure 7.7). Parallel “elastic-no-tension” and “Viscous” materials (Mazzoni et al. 2006) were assigned to those elements during some simulations (as discussed further below) in order to allow potential separation between the soil domain and the supporting wall (and the far boundary, Figure 7.7), and to allow radiation damping of the impacts from soil on those boundaries. In that case, the elastic-no-tension compression stiffness was set much higher than that of the adjacent soil elements in order to prevent noticeable compressive strains in the interface elements. The viscous material damping parameters were selected through a trial and error process in order to eliminate unrealistic force spikes without affecting the key aspects of the response.

In order to consider a more basic model, similar to Veletsos and Younan (1997) and Psarrapoulos (2005), an elastic material could also be applied to the zero length interface elements (Figure 7.7). In that configuration, a very high stiffness could be applied, bonding the soil elements to the supporting wall and far boundary, as if the interface elements were not there.
7.4 Linear-Elastic Soil Element Dynamic Earth Pressure Simulations

First, the quadrilateral backfill elements (Figure 7.6) were set to behave as linear elastic soil, but with shear modulus $G$ and bulk modulus $B$ as a function of initial confinement. This was achieved by employing the “PressureDependMultiYield” (PDMY) material in OpenSees (Elgamal et al. 2002, 2003, Yang et al. 2003) and switching to material stage 2 (Mazzoni et al. 2006). When using material stage 2, stress strain behavior is linear-elastic, but the required stiffness parameters are controlled based on the following equations:

\[
G = G_r \left( \frac{p'}{p_r'} \right)^d \tag{7.1}
\]

\[
B = B_r \left( \frac{p'}{p_r'} \right)^d \tag{7.2}
\]

where $G_r$ and $B_r$ are the shear and bulk modulus, respectively, at the reference confinement $p_r$, $p'$ is the initial confinement, and $d$ is a positive constant defining the variations.

7.4.1 Elastic Soil Model Parameters

The backfill element density was set to match the experimental soil density of 2.1 Mg/m$^3$. The reference shear and bulk moduli $G_r$ and $B_r$ were approximated based on the following relationships from elasticity (Lambe and Whitman 1969):

\[
G = \frac{E}{2(1 + \nu)} \tag{7.3}
\]

\[
B = \frac{E}{3(1 - 2\nu)} \tag{7.4}
\]
with Young’s modulus $E = 60,000$ kPa at 1 atm ($p_r' = 100$ kPa) and poisson’s ratio $\nu = 0.3$, based on recommendations for dense sand by Lambe and Whitman (1969) and Duncan and Mokwa (2001). The constant $d$ was set at a default value of 0.5 (Mazzoni et al. 2006). In the linear elastic model, small viscous damping was also included. Rayleigh stiffness and mass proportional constants were applied to provide damping of 5% at 1 and 6 Hz.

### 7.4.2 Effect of Wall Movement on the Dynamic Earth Pressure

Using the elastic soil models, a demonstration of the effect of various wall movements on the magnitude and distribution of dynamic earth pressure is presented in this section. In these simulations, the soil elements were allowed to separate from the model wall as discussed above (Figure 7.7). By allowing this separation, the wall position and the deformed adjacent backfill shape (at the instant of the peak pressure) are able to schematically assist in demonstrating the wall-soil interaction effects.

Similar to Veletsos and Younan (1997) and Jung and Bobet (2008), a harmonic input motion was used in order generate the dynamic increase in pressure (Figure 7.8). Using linear elastic soil models, Wood (1973) showed that dynamic amplification was negligible for input motions at less than half the fundamental frequency $f_0$ of the unrestrained backfill (Kramer 1996). The first natural frequency ($f_0 = V_s / 4H$) of the model soil layer is about 10 Hz (shear wave velocity of 90 m/s). A much lower excitation frequency of 1 Hz (Figure 7.8) was adopted in this study.
7.4.2.1 Rigid Wall

A rigid wall configuration was investigated first. Figure 7.8 presents the input acceleration and the total force exerted on the wall (earth pressure resultant force) during the excitation. At the instant of the peak recorded earth pressure resultant force (about 4.3 seconds in Figure 7.8), Figure 7.9a shows the relative displacement of the wall and the adjacent column of backfill soil elements (this is obvious for the rigid wall, but more revealing for the cases below). At the same instant, Figure 7.9b presents the pressure distribution along the height of the wall. From Figure 7.9, the adjacent soil elements are in contact with the wall, and exerting pressure along its full height.

7.4.2.2 Rotating Wall

In this section, the model was configured to consider a wall which could rotate about its base. Through a trial and error process, a rotational spring stiffness (Figure 7.7) was applied which resulted in a small level of wall rotation under the applied harmonic input excitation. Figure 7.10 illustrates the potential effect of wall rotation on the pressure distribution, at the peak force instant (similar to above). As the wall rotates away from the adjacent backfill (Figure 7.10a), the soil elements near the base deform enough to exert pressure (Figure 7.10b). In contrast, the soil elements near the backfill surface do not deform adequately (Figure 7.10a), a gap forms, and the resulting earth pressure near the wall top is zero (Figure 7.10b). A similar result was reported by Jung and Bobet (2008) with $d_R = 50$, as shown in Figure 7.3.
7.4.2.3 Bending Wall

Next, the model was configured to consider a bending wall. Through a trial and error process, a wall beam element stiffness (Figure 7.7) was applied which resulted in a small level of bending under the applied harmonic input excitation. Figure 7.10 illustrates the effect of bending on the dynamic pressure distribution, at the peak force instant. Similar the above rotating wall case, as the wall is bent away from the adjacent backfill (Figure 7.11a), the soil elements near the base deform enough to exert pressure (Figure 7.11b). The soil elements near the backfill surface do not deform adequately (Figure 7.11a), a gap forms, and the resulting earth pressure near the wall top is zero (Figure 7.11b). The shape of the pressure distribution of Figure 7.11b also agrees well with that of Jung and Bobet (2008) with $d_B = 50$, as shown in Figure 7.4.

7.4.2.4 Translating Wall

Finally, the model was configured to consider a translating wall. Through a trial and error process, a horizontal spring stiffness (Figure 7.7) was applied which resulted in a small level of lateral translation under the applied harmonic input excitation. Figure 7.12 illustrates the effect of the wall translation on the dynamic pressure distribution. In contrast with the above rotating and bending wall cases, as the wall is pushed away from the adjacent backfill in pure lateral translation, (Figure 7.12a) the pressure is zero near the base, but increases higher along the wall (Figure 7.12b). A similar result was reported by Jung and Bobet (2008) with $d_H = 0.1$, as shown in Figure 7.5.
7.4.3 Simulations for Comparison with the Experimental Results

In this section, comparisons are made between the dynamic earth pressure experiments presented in chapter 4 (with peak input accelerations up to 0.7 g), and the FE model simulations. Two important factors about the experiments are accounted for in the FE simulations as follows (refer to Chapter 4):

1) Slight test wall movements away from the backfill were larger near the top and smaller near the bottom of the very stiff wall. This response suggests that the wall translated slightly, and rotated somewhat more.

2) The test wall was pushed about 2.5 mm into the backfill before the start of the dynamic test series resulting in a higher lateral stress condition than that achieved by applying own-weight to the FE model backfill. As a result of that higher initial stress condition, the pressure did not reach zero near the middle and top of the test wall in the experiments, indicating that full separation (gapping) between the wall and backfill did not occur (in contrast with the result shown in Figure 7.10).

Modeling all of the intricacies in the exact test configuration (refer to Chapter 4) including the soil container, restraining towers, hanging wall, jacks and load cells mounted on steel posts behind the wall, etc. is beyond the desired scope of this investigation. However, by addressing the 2 issues (above), a useful comparison is made between the experimental and simulated results. Points 1 and 2 are addressed in the model as follows:

1) In order to include the effect of the observed test wall movements, rigid wall, rotating wall, and combined rotating and translating wall simulations are compared with the experimental results first. For the rotating and the combined wall motion cases,
spring stiffness values (Figure 7.7) were adjusted in the model until the simulations resulted in similar movements to those observed during the experiments.

2) For the rotating and combined rotation and translation cases, a constant horizontal load was applied to the left side of the wall (Figure 7.7). By applying that load, the initial lateral stress condition in the backfill was raised to a level comparable to that of the experiments, while the model wall remained able to move during the dynamic excitation (as opposed to the effect of assigning a fixed displacement to the wall in order to achieve the higher initial stress condition).

7.4.3.1 Simulation Results

Figures 7.13, 7.14a, and 7.14b show the distribution of dynamic pressure (change from the initial static pressure), at the instant of the peak earth pressure resultant force, along the wall height for the 3 investigated cases described above. As such, positive dynamic pressure in Figures 7.13, 7.14a, and 7.14b indicates an increase from the static value. Negative dynamic pressure indicates a decrease from the static value.

As expected for the rigid wall (Figure 7.13), the pressure increased simultaneously along the full rigid wall height as the peak lateral force occurred. The rotating wall (Figure 7.14a) experienced a decrease in pressure near the top, at the same time as an increase below. Finally, Figure 7.14b presents the dynamic pressure distribution on the wall which translated slightly (about half a millimeter) and rotated somewhat more (about 3 mm of displacement near the top). In that case, the pressure also decreased near the wall top and increased below (Figure 7.14b). However, a pressure increase did not occur near the wall base (Figure 7.14b).
7.4.3.2 Comparison with the Experimental Results

Integrating the dynamic earth pressure distribution (Figures 7.13 and 7.14) along the wall height gives the resultant lateral force increase, or dynamic earth thrust. From the rigid, rotating, and combined rigid and translating wall FE simulations, the dynamic earth thrust is compared with experimental results in Figure 7.15, over a range of input acceleration. From Figure 7.15, including the observed (small) wall rotation and translation in the FE simulation provides the best estimation of the resultant earth pressure force, as compared with the simplified rigid wall approximation.

It is noted that in Figure 7.15, one experimental data point is in apparent disagreement with the observed trend. That data point corresponds to the 1xHM test (refer to Chapter 4) which occurred after the very strong 3 and 3.3xEM tests were conducted with peak acceleration of 1.2 g. Those strong shaking tests may have resulted in a weaker backfill condition during the 1xHM test (as discussed in Chapter 4), contributing to the difference in response.

Figure 7.16 compares the dynamic pressure distribution from the 1 and 2xEM tests (Chapter 4) with results from the combined rotating-translating wall FE simulations. The general trend of decreasing pressure near the wall top, increasing pressure near the mid-height, and small changes in pressure near the base, is consistent between the experiments and the FE model simulations (Figure 7.16).

Based on the satisfactory match in terms of both peak force (Figure 7.15) and pressure distribution (Figure 7.16), the combined rotating-translating wall model was also subjected to the recorded base accelerations from the 1 and 2xEM at-rest condition tests of Chapter 4. Figures 7.17 and 7.18 compare the change in recorded pressure from the
tests with the FE simulation results. From Figures 7.17 and 7.18, the FE model with elastic soil elements was able to produce similar dynamic pressure time histories to those recorded in the experiments.

7.5 **Nonlinear Elastic-Plastic Soil Element Dynamic Earth Pressure Simulations**

In this section, simulations are conducted using a nonlinear elastic-plastic material model to represent the backfill soil. Simulations are conducted in consideration of the at-rest (Chapter 4) and passive (Chapters 5 and 6) condition dynamic tests.

7.5.1 **Soil Model**

The elastic-plastic PDMY (PressureDependMultiYield) material mentioned above was employed for the nonlinear soil simulations. This material was implemented in OpenSees (http://cyclic.ucsd.edu/opensees, Mazzoni et al. 2006) for simulating the essential response characteristics of pressure sensitive soil (e.g., sand). In addition to the pressure sensitive shear stiffness and strength; the PDMY model simulates nonlinear stress-strain behavior, by including a number of yield surfaces sharing a common apex (Figure 7.19), with the outermost surface acting as the failure envelope (Elgamal et al. 2002). The involved parameters are described in Table 7.1. Refer to Yang (2000), Elgamal et al. (2002, 2003), or Yang et al. (2003) for further description of the PDMY material model.

The recommended dense sand parameters listed in Table 7.1 were used in the simulations with the following exceptions: i) the friction angle was set as 46 degrees based on the conducted triaxial and direct shear tests performed on the backfill soil (refer
to Chapter 2 or Chapter 4), ii) the reference shear and bulk moduli $G_r$ and $B_r$ were set to match those used in the elastic models described above, iii) the phase transformation angle was set at a high value (85 degrees), and iv) the contraction and dilatancy parameters were all set as zero. The reason for both iii) and iv) is that those parameters are intended to account for pore pressure generation and liquefaction behavior in saturated soil, which are not the intended purpose of the dynamic earth pressure model.

7.5.2 Simulations for Comparison with the At-Rest Condition Tests

FE rigid, rotating, and combined rotating-translating wall simulations with the PDMY soil model are performed first using the harmonic motion (e.g., Figure 7.20) to record the dynamic pressure distribution and the peak dynamic earth thrust.

7.5.2.1 Simulation Results

Similar to the above elastic soil element models, the earth pressure increased along the full wall height (Figure 7.21) at the instant of the peak resultant force (Figure 7.20) in the rigid wall-nonlinear soil simulation. The earth pressure decreased only slightly near the top of the rotating wall height using the PDMY soil (Figure 7.22a). Wall translation resulted in a reduction in earth pressure near the bottom of the wall, at the same time as an increase further above (Figure 7.22b).

7.5.2.2 Comparison with the Experimental Results

Figure 7.23 presents the dynamic earth pressure resultant force (dynamic earth thrust) and corresponding input acceleration from the experiments and simulations. In contrast with the elastic model results of Figure 7.15, the PDMY soil simulations produce
a nonlinear input acceleration-dynamic earth thrust relationship (Figure 7.23).
Additionally, the rigid wall came closer to predicting the peak dynamic earth thrust from most of the experiments with peak input acceleration above 0.9 g, than the models which include the wall movement (Figure 7.23).

Due to the better agreement in terms of dynamic earth thrust shown in Figure 7.23, the rigid wall FE model peak dynamic pressure distributions are compared with the experimental results from the 3 and 3.3xEM tests in Figure 7.24. The general trend of increasing pressure along the full wall height was consistent between the experiments and the FE model rigid wall simulations (Figure 7.24). However, the FE model significantly over-predicted the peak dynamic earth pressure from the 3xEM (Figure 7.24) test.

The PDMY soil-rigid wall model was also subjected to the recorded base acceleration from the second run of 3xEM and the first run of 3.3xEM dynamic tests (refer to Chapter 4). From those simulations, the earth pressure resultant force is compared to the experimental result (LC force minus the wall inertia, Chapter 4) in Figures 7.25 and 7.26. In both simulations, the peak model force was close to the experimental result. However, the FE model over-predicted the response at instances of lower level acceleration input peaks (Figures 7.25 and 7.26).

Achieving a better match with the experimental data may be possible by including a more sophisticated mechanism to simulate the test wall movement, for instance by modeling the detailed soil container configuration of Chapter 4. However such a detailed model is outside the desired scope of this investigation.
7.5.3 Effect of Cohesion

A cohesion intercept can also be included in the PDMY soil model. The effect of including cohesion $c = 14$ kPa (based on the laboratory triaxial and direct shear tests performed on the employed backfill soil) is illustrated in Figure 7.27. From Figure 7.27, including the cohesion brings the rotating and translating wall FE simulation results closer to the experimental dynamic earth thrust at the lower level input accelerations (up to 0.7 g). Including the cohesion behind the rigid wall model does change the response noticeably until input accelerations exceed about 0.6 g.

7.5.4 Simulations with Passive Pressure Initially Mobilized in the Backfill

This section investigates the effects of dynamic excitation on the model, after passive pressure has been mobilized in the backfill. The wall beam elements and translational and rotational springs (Figure 7.7) were removed from the model for these simulations. Instead, a lateral displacement was first applied to the soil element nodes along the wall-backfill boundary (Figure 7.28) in order to mobilize a portion of the peak passive resistance. Similar to the experiments (Chapters 5 and 6), the earthquake record motion was executed in the model after the lateral displacement was applied.

During the passive (Chapter 5) and passive failure (Chapter 6) condition dynamic tests, a portion of the initial (static) passive pressure was permanently relieved during the strong shaking phase. In order to subject the FE model backfill to a comparable excitation for comparison of the total decrease in the initial passive pressure, the full earthquake record input motions were used (Figures 7.28 through 7.30).
7.5.4.1 Simulation Results

Similar to Figures 7.25 and 7.26, Figures 7.28 through 7.30 present the earth pressure resultant force from simulations using the base acceleration recorded during the 1, 2, and 3xEM tests. From Figures 7.28 through 7.30, the level of mobilized passive pressure increased and decreased throughout the earthquake record excitations. A portion of the initial (static) passive pressure was also lost during the strong shaking phase (Figures 7.28 through 7.30). These observations are in general agreement with the experimental results of Chapters 5 and 6.

7.6 Conclusions

Conclusions based on the conducted elastic and nonlinear elastic-plastic dynamic earth pressure simulations can be summarized as follows:

1) Retaining wall movements had a significant impact on both the dynamic earth pressure distribution and the resultant force in both the linear elastic and nonlinear elastic-plastic model simulations.

2) Specifically, bending and rotating walls may experience lower dynamic earth pressure near the top, as compared with rigid walls. Walls which can translate may experience lower dynamic earth pressure near the bottom as compared with rigid walls.

3) Compared with the at-rest condition shake table dynamic earth pressure experiments with peak input acceleration up to about 0.7 g, the FE simulations which account for the wall rotation and translation provided the best match in terms of the dynamic earth pressure distribution and the resultant force. With the nonlinear elastic-plastic
soil model, including soil cohesion also improved the accuracy of the prediction in this range.

4) Compared with the peak response from the at-rest condition shake table dynamic earth pressure experiments with peak input acceleration above about 0.9 g, the nonlinear soil model FE simulations of a rigid wall provided the best match in terms of the dynamic earth pressure distribution and the resultant force. These results may reflect limitations to the wall compliance in the employed test configuration (stiffening effect with increasing wall displacement). A more complex FE model representation of the experimental configuration may be able to better capture the overall response at these extreme shaking levels.

5) Compared with the passive and passive failure condition shake table dynamic earth pressure experiments the following response characteristics were also captured in the nonlinear soil model simulations: i) the level of initially locked in passive pressure increased and decreased throughout the excitations, and ii) the static passive pressure level experienced a permanent decrease by the end of the strong shaking.

7.7 References


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<td>$B_r$ (kPa)</td>
<td>Reference bulk modulus</td>
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Figure 7.1: Possible deformation characteristics of a retaining wall and a typical free-field dynamic response of an elastic soil column (first mode)

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(from Jung and Bobet 2008)

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Figure 7.8: FE elastic soil model rigid wall simulation
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Chapter 8  Bridge Abutment Dynamic Interaction Modeling

8.1 Introduction

As mentioned in previous chapters, strong ground shaking can cause a bridge superstructure (deck) to impact the abutments during an earthquake (Figure 8.1). In seismic bridge design, a seat-type abutment is often constructed with a sacrificial backwall, designed to shear off if impacted by the bridge (Figure 8.1). This system (Figure 8.1) aims to protect the abutment foundation (ATC/MCEER 2001, Caltrans 2004, AASHTO 2007). Once the backwall shears off, resistance to superstructure impact is provided by the development of passive earth pressure in the backfill (Shamsabadi et al. 2005). This resistance will thus reduce the demand on other elements of the bridge earthquake resisting system (AASHTO 2007). Accurate representation of maximum passive earth pressure and the load-deflection relationship is consequently vital for predicting the overall bridge lateral response.

A new material model has been implemented in the open source finite element analysis framework OpenSees (OpenSees, http://opensees.berkeley.edu) for the purpose of modeling the abutment passive pressure contribution to longitudinal bridge deck resistance. The model was developed based on observations and data from the experimental and analytical work performed as a part of this dissertation. The model is described in this chapter and its use is demonstrated in simple 2D bridge model dynamic simulations using OpenSees.
In addition to the static backfill passive force-displacement resistance, data presented in Chapters 5 and 6 illustrate how backfill and backwall inertial forces may also affect the dynamic response. A spring mass model is also introduced and demonstrated in order to simulate this inertial effect.

### 8.2 Cyclic Loading Passive Resistance Spring Model

For application of the hyperbolic model representations of the passive earth pressure force-displacement curves of Chapters 2 and 3 in dynamic bridge simulations, a cyclic loading model is presented in this section. Within the overall collaborative framework of the investigation into highway bridge seismic performance (Saiidi 2004), this cyclic model has recently been implemented and is available for use in the FE platform OpenSees (Mazzoni et al. 2006). For that purpose, a “HyperbolicGapMaterial,” or HGM (Figure 8.2) was developed for OpenSees with the help of Dr. Matthew Dryden of the University of California at Berkeley, as a part of his PhD study under the supervision of Professor Gregory Fenves (Dryden 2009).

The model follows a hyperbolic backbone curve for virgin loading (Figure 8.3 and Figure 8.4) controlled by the following equation (Duncan and Mokwa 2001):

\[
F = \frac{y}{\frac{1}{K_{\text{max}}} + R_f \frac{y}{F_{\text{ult}}}}
\]

(8.1)

where \(F\) is the resisting force, \(y\) is the horizontal displacement, \(F_{\text{ult}}\) is the peak resisting force, \(K_{\text{max}}\) is the initial tangent stiffness and \(R_f\) is the failure ratio (refer to Chapter 1 for a more detailed description of the hyperbolic curve).
In addition, the HGM can account for the expansion gap which may exist between the end of the bridge deck and the abutment wall (Figures 8.1 through 8.4). An unloading and reloading stiffness $K_{ur}$ is also included for subsequent response cycles (Figures 8.2 through 8.4). The model assumes that if the bridge deck pushes the abutment backwall into the backfill and then retreats, the wall essentially remains at its furthest penetration (the small soil cohesion helps the deformed backfill to retain its shape). On subsequent loading cycles (Figure 8.4), it is assumed that the abutment loses its resisting capacity up to the point of prior unloading. For the implemented model, $K_{ur} = K_{max}$ (the initial stiffness) may be adopted as the unloading and reloading stiffness. Cole and Rollins (2006) and Lemnitzer et al. (2009) provide additional insight on the passive pressure unloading and reloading stiffness for a wider range of backfill soils.

### 8.3 2D Bridge Model Demonstration

In this section, a simple 2D (longitudinal direction) 2 span elastic bridge model is developed in order to first demonstrate the use of the abutment spring (Figure 8.5). Bridge dimensions and associated model parameters (Table 8.1) were set to approximately match the “Route 14 bridge (R14)” highway overpass in California, as reported by Aviram et al. (2008). For simplicity, the cap beam (Aviram et al. 2008) was not included in the longitudinal response analysis.

The bridge columns were modeled as fixed at the base and at the connection to the bridge deck (deforming in double-curvature). With that configuration, the first mode in the longitudinal direction had a natural period of about 0.7 seconds.
The modified 1994 Northridge earthquake record motion (1xEM) from the dynamic earth pressure experiments (Chapters 4 through 6) was used to demonstrate the model. Five cases were considered in this section as follows (Table 8.2):

1) Neglecting the longitudinal abutment passive earth pressure resistance (with rollers in the longitudinal direction at the bridge deck ends); and including the abutment passive resistance (for a 1.7 m backwall height) by using the HGM according to:

2) The dense well-graded silty sand hyperbolic passive force-displacement model (Soil T2, Chapter 3);

3) The dense clean sand hyperbolic passive force-displacement model (Soil D-S, Chapter 3);

4) The medium-dense silty sand hyperbolic passive force-displacement model (Soil MD-SM, Chapter 3); and

5) The medium-dense clayey sand hyperbolic passive force-displacement model (Soil MD-SC, Chapter 3).

The maximum column deformation (and the related ductility demand) is often a critical consideration in seismic bridge design. As such, the displacement at the connection between the top of the column and the bridge deck, relative to the base, is used as a basis for comparison of the model response. Figure 8.6 shows the input acceleration and displacement at the column-bridge deck connection for Case 1 (no abutment contribution). From Figure 8.6, with a peak input acceleration of about 0.35 g, the columns displaced as much as 14 cm, relative to the base.
Figure 8.7 presents the input acceleration and column displacement for Case 2, and Figure 8.8 demonstrates the HGM cyclic force-displacement behavior at the abutments. Compared with Figure 8.6, with the abutment passive resistance included in the model in Figure 8.7, the column deflection is noticeably reduced. Similar results are included for Cases 3 through 5, in Figures 8.9 through 8.14. The peak column displacement from each case is also listed in Table 8.2.

From Table 8.2, including the abutment contribution considering the backfill soil used in the Chapter 2 passive pressure tests (Soil T2) resulted in a 40% reduction in the column displacement demand. Including a weaker backfill (Soil D-S) reduced the demand by 20%. The range of backfill soil force-displacement resistance models (Cases 2 through 5) resulted variations in the maximum column deflection of up to 3 cm, from the 1xEM input with peak acceleration of about 0.35 g.

8.4 Backfill Inertial Effect

As mentioned above, it was shown by the passive and passive failure condition dynamic tests (Chapters 5 and 6) that the level of resistance may also depend on the ground shaking. The HGM model presented above does not directly account for such effects.

In order to include an approximation of the observed backfill inertial effects from the dynamic tests, a lumped mass is added to the bridge model in this section (Figure 8.15). The participating lumped backfill mass (Figure 8.15) is determined based on the size of the passive failure wedge (Chapter 6) and the backwall mass. The required additional spring stiffness (Figure 8.15) is estimated based on the shear stiffness of the
backfill. The force exerted by the mass on the bridge deck was first validated against the recorded changes in passive thrust from the dynamic tests.

The abutment force-displacement response from bridge simulations using 2xEM (peak input acceleration of 0.66 g) as the input record is shown in Figure 8.16 (including the lumped mass) and Figure 8.17 (without the lumped mass). This model adds the effect of the backfill inertia which causes the passive force-displacement relationship to diverge (Figure 8.16) from the static resistance behavior (Figure 8.17).

A final comparison of the bridge model response is presented in terms of the column deformation for two earthquakes in Figures 8.18 (2xEM) and 8.19 (Rinaldi record of the 1994 Northridge earthquake) using three different abutment models: i) no resistance provided by the abutments (using roller supports without any springs), ii) considering only the static backfill passive force-displacement (spring), and iii) including both the static force-displacement resistance and the backfill inertia effects (spring-mass).

Similar to the above section, by including the abutment resistance (as opposed to using only rollers), the predicted column displacement demand is reduced substantially (Figures 8.18 and 8.19). In Figure 8.18, the difference in column displacement response obtained by including (spring-mass model), and not including (spring only model) the effect of the backfill inertia is relatively small. This is partly due to the fact that the backfill inertial force is changing direction rapidly, causing both increases and decreases in the available resistance.

During the Rinaldi input record of Figure 8.19, influence of the backfill inertia is more pronounced. In Figure 8.19, the sudden, strong acceleration jolt at about 2.5 seconds, results in a larger displacement (about 20% more) when the backfill inertial
effect is included. At such instances, the backfill inertial force reduces the available abutment resistance (detrimental effect). In some other cycles (e.g. around 5 seconds), the backfill inertia increases the abutment resistance, and reduces the corresponding bridge displacement (beneficial effect).

8.5 Conclusions

1) A Hyperbolic Gap Material (HGM) spring model was implemented in the FE code OpenSees, to represent the static passive force-displacement in dynamic bridge simulations. The model backbone curves are based on the measured and simulated passive resistance relationships of Chapters 2 and 3.

2) The implemented model was demonstrated using linear elastic bridge simulations in OpenSees. Results from this demonstration suggest that including the abutment passive force-displacement resistance in design may significantly reduce the projected column displacement demand, with substantial dependence on the backfill soil type.

3) An added lumped mass to simulate the effects of inertial forces in the backfill, caused instants of both increased and decreased abutment resistance during shaking. Depending on the bridge and input ground motion configurations, the overall influence of backfill inertia may be small. In such cases, simply including the static backfill passive force-displacement resistance would be sufficient. However, additional research to further investigate the effects of backfill shaking on the abutment passive force-displacement is needed, possibly within a performance-based earthquake engineering framework.
4) More sophisticated nonlinear bridge models, and investigations considering a wider range of earthquake ground motions, could be performed using the implemented models to further investigate the effect of backfill soil type and the backfill and backwall inertia on the overall response.

8.6 References


Table 8.1: Bridge model specifications

<table>
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<tr>
<th>Specification</th>
<th>Value</th>
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<tbody>
<tr>
<td>Span 1 length (m)</td>
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</tr>
<tr>
<td>Span 2 length (m)</td>
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</tr>
<tr>
<td>Superstructure width (m)</td>
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<tr>
<td>Superstructure depth (m)</td>
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</tr>
<tr>
<td>Number of columns</td>
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<tr>
<td>Column diameter (m)</td>
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</tr>
<tr>
<td>Area A of each column (m$^2$)</td>
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<tr>
<td>Young's Modulus E of the columns (GPa)</td>
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<tr>
<td>Moment of inertia I per column (m$^4$)$^a$</td>
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$^a$Reduced to consider effect of cracked section

Table 8.2: Comparison of bridge model response to 1xEM (modified Northridge 1994 Century City Station earthquake record)

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<th>Case</th>
<th>Abutment backfill soil type</th>
<th>Spring model parameters</th>
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<td>$K_{\text{max}}$ (kN/m/m)</td>
<td>$F_{\text{ult}}$ (kN/m)</td>
<td>$R_t$</td>
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<td>None</td>
<td>0</td>
<td>0</td>
<td>~</td>
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<tr>
<td>2</td>
<td>T2 (dense well-graded silty sand)</td>
<td>24000</td>
<td>550</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>D-S (dense clean sand)</td>
<td>24000</td>
<td>190</td>
<td>0.8</td>
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<tr>
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<td>MD-SM (medium-dense silty sand)</td>
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<tr>
<td>5</td>
<td>MD-SC (medium-dense clayey sand)</td>
<td>15000</td>
<td>650</td>
<td>0.75</td>
</tr>
</tbody>
</table>
Hyperbolic Gap Material

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Patrick Wilson: prwilson@ucsd.edu

This command is used to construct a hyperbolic gap material object.

```
uniaxialMaterial HyperbolicGapMaterial $matTag $Kmax $Kur $Rf $Fult $gap
```

$matTag: unique material object integer tag
$Kmax: initial stiffness
$Kur: unloading/reloading stiffness
$Rf: failure ratio
$Fult: ultimate (maximum) passive resistance
$gap: initial gap

*NOTE: This material is implemented as a compression-only gap material. $Fult and $gap should be input as negative values.

Figure 8.1: Schematic elevation view of a seat-type bridge abutment

Figure 8.2: OpenSees Command Language Manual description of Hyperbolic Gap Material (Mazzoni et al. 2006)
Figure 8.3: Representative first loading cycle using abutment HGM model

Figure 8.4: Representative subsequent loading cycle using abutment HGM model
Figure 8.5: FE model used to demonstrate the abutment HGM model

Figure 8.6: FE simulation results without abutment spring (Case 1)
Figure 8.7: FE simulation results from Case 2: HGM with dense well-graded silty sand backfill model

Figure 8.8: FE simulation results from Case 2: HGM with dense well-graded silty sand backfill model
Figure 8.9: FE simulation results from Case 3: HGM with dense clean sand backfill model

Figure 8.10: FE simulation results from Case 3: HGM with dense clean sand backfill model
Figure 8.11: FE simulation results from Case 4: HGM with medium-dense silty sand backfill model

Figure 8.12: FE simulation results from Case 4: HGM with medium-dense silty sand backfill model
Figure 8.13: FE simulation results from Case 5: HGM with medium-dense clayey sand backfill model

Figure 8.14: FE simulation results from Case 5: HGM with medium-dense clayey sand backfill model
Figure 8.15: Bridge model implementation of the backfill inertia effect using a lumped mass

Figure 8.16: Force-displacement response at the bridge deck-abutment interface including the backfill inertial effect during simulation of the (2xEM) Century City Station earthquake record (peak input acceleration of 0.66 g)
Figure 8.17: Force-displacement response at the bridge deck-abutment interface without including the backfill inertial effect during simulation of the (2xEM) Century City Station earthquake record (peak input acceleration of 0.66 g)
Figure 8.18: Bridge displacement response to Century City Station earthquake record (2xEM)
Figure 8.19: Bridge displacement response to Rinaldi earthquake record
Chapter 9  Summary and Conclusions

9.1  Summary

Two main topics related to lateral earth pressure were investigated in order to improve our understanding and practice: i) the passive earth pressure force-displacement relationship, and ii) dynamic earth pressure. Passive earth pressure provides resistance to foundation displacement behind bridge abutments, pile caps and shallow foundations. In order to include this resistance in analysis and design, the force-displacement relationship is needed. However, passive earth pressure theories do not provide information on the force-displacement relationship, and existing experimental data and models do not account for the range of resistance from backfill soil types and wall heights used in the field.

To that end, passive earth pressure tests were performed in a large soil container by pushing a test wall into dense well-graded silty sand backfill, while recording the force-displacement relationship. Finite element (FE) models simulated the experimental response, and provided additional force-displacement curves for a range of practical backfill types and wall heights. A spring model was implemented in the open-source FE code OpenSees, which employs the experimentally and numerically derived force-displacement curves to represent the passive earth pressure resistance at the abutments in dynamic bridge simulations (as well as in other applications in which passive earth pressure provides resistance).
The seismic response of retaining walls and supported soil backfill is another complicated soil-structure interaction problem which is not well understood. Experimental data and case histories of field retaining wall performance have often contradicted the available dynamic earth pressure predictions. Consensus among researchers and practitioners on an appropriate approach to retaining wall seismic design is yet to be achieved. As such, dynamic earth pressure remains a problem which requires significant further research before definitive improvements can be made.

For that reason, dynamic earth pressure experiments were also performed by subjecting the above soil container-test wall-backfill configuration to shake table excitation. Force, pressure, acceleration and displacement were recorded throughout these experiments. Dynamic earth pressure was found to be very low at significant shaking levels (up to 0.66 g). Analysis of the test data illustrated the significant effects of slight wall movements on the magnitude and distribution of dynamic earth pressure. This new experimental evidence helps to provide a basis for needed improvements to the methods which are currently used to predict dynamic loads on retaining structures.

Additional dynamic earth pressure experiments were performed after passive earth pressure was mobilized behind the test wall. These tests demonstrated how the instantaneous level of passive resistance can also depend on the ground motion, particularly during very strong shaking. FE model simulations provided additional insights and confirmed key aspects of the dynamic response.

The data from all of the performed large scale tests have also been uploaded for public use on the NEESit data archive (https://central.nees.org/). Detailed conclusions
from the conducted studies are presented in two separate sections below regarding: i) passive earth pressure, and ii) dynamic earth pressure.

**9.2 Passive Earth Pressure**

From the two conducted large scale tests with dense well graded silty sand backfill, the passive earth pressure force-displacement relationship was found to be highly nonlinear, reaching a peak at a deflection of 2.7 and 3 percent of the wall height (1.7 meters), before decreasing to a residual level. In the plane strain passive earth pressure loading configuration, the employed backfill soil was found to be very strong in terms of friction, with a considerable contribution also from cohesion.

The allowed vertical wall movement in these tests resulted in a low $\delta_{mob}$, and a triangular failure wedge shape. The low $\delta_{mob}$ and triangular failure wedge shape contributed to the observed high degree of post-peak strain softening.

For the test conducted with the experimental backfill moisture content close to the lab moisture content (Test 2), the Log Spiral and Coulomb peak passive resistance estimates (equal to each other due to the low $\delta_{mob}$) using triaxial test parameters were about 10% lower than the measured result, while the direct shear test value predictions were about 10% higher. The average of the triaxial and direct shear test results provided the closest estimate of the experimental backfill shear strength (under plane strain loading). Solving for force equilibrium on the observed failure wedge using ultimate shear strength parameters ($\phi_r$ and $c_r$) provided a reasonable estimate of the residual passive resistance.
With greater time between construction and testing (which allowed for evaporation and drainage to occur), about 20% greater maximum resistance, and more rapid shear strength degradation with displacement beyond the peak, were observed compared with the more moist (as-built) case. Analysis of the observed failure suggested that the drier backfill responded with a higher $\phi$ (about 10%) and lower $c$ (about 20%) than the wetter counterpart. This issue may warrant further investigation in light of similar potential long term variations due to aging effects and change in the placement water content.

Using parameters derived from direct shear and triaxial tests (for the moist backfill of Test 2), and analysis of the observed failure (for the drier backfill of Test 1), plane strain FE simulations were in good agreement with the experimental load-deflection response up to about 95% of the peak resistance. On that basis, the calibrated FE models were employed to investigate the passive earth pressure force-displacement relationship considering four different backfill soils and a range of wall heights. Results show how the different backfill soils provide substantially different load-displacement resistance, in terms of both stiffness and strength. It was shown that the increase in supported backfill height and the depth dependent stiffness of the soil also contributed to significant variations in the force-displacement relationship.

For practical applications, hyperbolic model approximations were provided for 32 different combinations of backfill soil type and wall height. Such hyperbolic models are a significant improvement from the available bilinear design models. A FE model material was also implemented in the FE code OpenSees, which employs the above hyperbolic models as springs to represent the passive earth pressure load-displacement
resistance in pushover analyses and dynamic simulations. The use of the passive force-displacement spring model was demonstrated at the abutments in dynamic FE model bridge simulations.

For use in applications in which large displacements are anticipated (post-peak softening), the above models can also be capped at the expected residual resistance level, based on the ultimate shear strength of the backfill soil.

9.3 Dynamic Earth Pressure

Dynamic earth pressure experiments were performed considering 3 different initial backfill earth pressure conditions: i) at-rest, with only a very small (about 2.5 mm) wall displacement into the backfill before testing, ii) passive, with a significant level of the pre-peak passive resistance mobilized by first pushing the wall into the backfill, and iii) passive failure, with a passive failure wedge fully mobilized in the backfill by first pushing the wall beyond the level of peak passive resistance. Conclusions from the at-rest condition tests provide valuable insight pertaining to earth pressure during earthquakes on retaining walls which are subjected to at-rest or active earth pressure under static conditions (e.g. cantilever walls, gravity walls, basement walls, etc.). Conclusions from the passive and passive failure condition tests are of relevance to retaining walls or foundation structures subjected to earthquake loading, which rely on resistance or stability provided by passive earth pressure.
9.3.1 At-Rest Initial Condition

In the conducted tests with dense well-graded silty sand backfill, and earthquake record input accelerations up to about 0.66 g, the resultant dynamic component of lateral earth thrust was close to zero. The inertia of the wall was much higher than the dynamic earth pressure in that input range. Factors contributing to the low levels of measured dynamic earth thrust include: i) the high backfill soil stiffness and strength including cohesion, ii) the relatively short supported backfill height (1.7 meters), iii) the ability of the wall to rotate and translate slightly away from the backfill (up to 2.5 mm of displacement near the top), and iv) deformation compatibility along the wall-backfill interface (tendency of the free-field soil to deform like a shear beam), which resulted in instants of counter-acting pressure decrease near the wall top and increase below. Based on the above factors, different combinations of backfill soil type and wall configuration may also experience low dynamic earth pressure up to a significant level of shaking.

At larger input acceleration levels (near 1 g), the dynamic earth thrust became very large (several times the static earth pressure at peak instants), and clearly exceeded the test wall inertial force. During those brief instants, the earth pressure increased simultaneously along the full wall height, as the soil shear strength was further mobilized. In the employed configuration, the test wall also became more rigid as it was pushed larger distances away from the backfill (stiffening effect due to limited compliance in the test configuration). This effect of the wall yielding slightly and then becoming stiffer contributed to the high dynamic pressures recorded during strong shaking.

FE simulations further illustrated the significant effects of wall rigidity, rotation, bending, and translation on the magnitude and distribution of dynamic earth pressure.
Good agreement with experiments having peak input acceleration up to 0.66 g was achieved when the FE model was able to account for the observed wall rotation and translation, and the high shear stiffness and strength including cohesion.

The theoretical dynamic earth pressure predictions provided varying levels of agreement with the recorded values. With wall movements during shaking as small as 1 mm away from the strong and stiff backfill, the dynamic earth pressure resultant was significantly less than prediction based on a rigid wall configuration. This observation is important to consider for structures which are typically analyzed as rigid or “non-yielding,” such as basement walls, if small displacements (on the order of a few mm) may actually be possible. The modified Mononobe-Okabe active earth pressure prediction including cohesion provided the best prediction (zero) of the recorded dynamic thrusts with input acceleration up to 0.66 g, but underestimated the recorded pressure at stronger input levels. None of the theoretical predictions which were considered in this analysis were able to accurately predict the dynamic earth pressure resultant force over the full range of tested input acceleration levels.

Based on the presented tests, simulations, literature review and careful consideration of the dynamic earth pressure problem, it is recommended that efforts to improve methods for predicting seismic loads on retaining walls should consider: i) accurate representation of the backfill stiffness and strength, possibly including plane strain friction angles and cohesion in sandy backfills, ii) the potential for reduction in dynamic earth pressure levels due to small wall movements, iii) the deformation compatibility along the wall-backfill interface, iv) the short duration of peak dynamic
earth thrusts, v) inertial forces from the retaining wall mass, and vi) the wall and supported backfill height including phase and amplification effects (for taller walls).

9.3.2 Passive Pressure Initial Condition

In the conducted tests with dense well-graded silty sand backfill, and input acceleration levels of up to 0.6 g, increases and decreases of only about 5% of static peak passive resistance occurred. However, increases and decreases of about 30% of the mobilized static passive resistance occurred with input accelerations of about 1 g. Based on those results, the level of passive resistance available for providing stability (e.g. sheet pile anchor wall, or shallow side of a cantilever wall) and resisting structure movements (e.g. bridge abutments and pile caps) may depend on the magnitude and direction of the excitation, particularly at high input acceleration levels.

A portion of the mobilized static passive stress within the backfill was also relieved during the strong shaking phase (as much as 15%). This observed lateral stress reduction due to shaking may warrant further investigation for applications where mobilized passive pressure or initially high confining stresses are relied upon for stability, such as retaining walls, micro piles and compaction grouting.

Considering the case of dynamic excitation performed with passive pressure mobilized in the backfill, the following response characteristics were also captured in FE model simulations using a nonlinear soil model: i) the level of initially locked in passive pressure increased and decreased throughout the excitations, and ii) the static passive pressure level experienced a permanent decrease by the end of strong shaking.
9.3.3 Passive Failure Initial Condition

With peak input acceleration levels of up to 0.6 g, increases and decreases of only about 5% of the static peak passive resistance were recorded. However, with peak input acceleration of 1.3 g, increases and decreases of as much as 100% and 50% (respectively) of the initially mobilized static passive resistance occurred. Similar to above, these results suggest that the level of passive resistance may depend on the magnitude and direction of the excitation, particularly at high input acceleration levels.

The recorded motions within the previously mobilized passive failure wedge were significantly greater than those outside that region. When lateral load was re-applied to the static backfill after the end of dynamic testing, the previously mobilized wedge resumed sliding up the pre-existing weakened failure surface. This response indicates that the pre-existing weak passive failure plane with $\phi = \phi_r$ and $c = c_r$ remained weaker than the surrounding backfill soil throughout the test series, and influenced the response to dynamic excitation. Based on that result, a pre-existing shear band or one that forms during an earthquake (or in an aftershock event) could influence the dynamic response.

As a simplified method to approximate the effects of the backfill inertia, a spring-mass model was added to the static passive force-displacement spring at the abutments in FE model bridge simulations. In those simulations, the added mass to simulate the effects of inertial forces in the backfill caused instants of both increased and decreased abutment resistance during shaking. Depending on the bridge and input ground motion configurations, the overall influence of backfill inertia may be small. In such cases, simply including the static backfill passive force-displacement resistance would be
sufficient. However, additional research to further investigate the effects of backfill shaking on the abutment passive force-displacement is needed.

9.4 Recommendations for Future Research

1) A series of separate passive earth pressure load-displacement experiments in which different levels of mobilized wall-soil friction are achieved, with all other aspects of the test held constant, would be highly desirable. Comparison of the backfill failure mechanism from such experiments would also provide valuable insight. The effect of mobilized wall-soil friction on the response would be valuable in terms of the stiffness (force-displacement relationship), maximum capacity, and residual resistance level.

2) A series of separate passive earth pressure load-displacement experiments in which different wall heights are tested, with all other aspects of the test held constant, would also be highly desirable. The effect of wall height on the response would be valuable in terms of the stiffness (force-displacement relationship), maximum capacity, and residual resistance level.

3) In order to consider the effect of backfill shaking on the passive earth pressure force-displacement relationship, two tests could be conducted. A baseline force-displacement curve could be recorded first using a static backfill. Next, passive earth pressure could be mobilized during shake table excitation, with all other aspects of the test held constant.
4) Shake table bridge model testing with sacrificial backwall seat-type abutments and soil backfills would provide valuable new insight into the soil-structure-interaction effects.

5) More sophisticated dynamic FE bridge model simulations could be performed to consider the effect of different soil types on the response. The effect of backfill inertia (dynamic passive pressure) could also be investigated by including backfill soil domains in the FE bridge models.

6) Any additional retaining wall shake table testing with careful observation of the wall-backfill interaction and resulting pressure distribution and magnitude would make an extremely valuable contribution.

7) Dynamic retaining wall experiments using backfill soil with and without cohesion behind a yielding wall are needed for validation of new dynamic active earth pressure prediction methods which include the effect of soil cohesion.
Appendix A  Test Configuration

In addition to those provided in the previous chapters of this dissertation, photographs and drawings of the test configuration are included in this appendix.

Figure A.1: Restrained laminar soil container on shake table
Figure A.2: Transportation of the soil container laminar frames

Figure A.3: Soil container base on shake table
Figure A.4: Stool to allow hydraulic jack to fit over the soil container feet in order to post-tension to shake table

Figure A.5: Container base and frames assembled on the shake table
Figure A.6: Plywood lining inside the soil container

Figure A.7: Steel posts with attached rods for mounting jacks and load cells (top); supporting beam with attached channel sections (middle); reinforced concrete test wall section (bottom)
Figure A.8: Test wall suspended from supporting beam

Figure A.9: Inserting the test wall
Figure A.10: Installing mounting hardware on concrete-filled steel posts for attachment to the soil container

Figure A.11: Loading mechanism behind the test wall showing 1 of 4 jack and load cell stacks mounted on concrete-filled steel posts
Figure A.12: Overhead view of loading mechanism behind the test wall

Figure A.13: Hydraulic pump and manifold to control the jacks
Figure A.14: Plastic lining inside the soil container

Figure A.15: Test wall suspended above wooden box inside soil container
Figure A.16: Backfill soil delivery

Figure A.17: Adjusting soil moisture content

Figure A.18: Backfilling
Figure A.19: Backfilling and increasing moisture content for compaction

Figure A.20: Backfill compaction
Figure A.21: Verification of the achieved soil density using nuclear gauge

Figure A.22: Filled soil container
Figure A.23: Installing the backfill surface linear potentiometer (stick-pot) frame

Figure A.24: Post-tensioning reaction/restraining tower to shake table
Figure A.25: Post-tensioning reaction/restraining tower to the shake table (close-up view)

Figure A.26: Backfill surface overhead view showing installed linear potentiometer frame and foam cores (before Test 1)
Figure A.27: Mounting video camera on restraining tower

Figure A.28: Excavation of soil container between test series
Figure A.29: Excavation of soil container

Figure A.30: Close-up view of soil container excavation
Figure A.31: Schematic of restrained empty laminar soil container on shake table jack mounting post schematic (units: inches)

Figure A.32: Elevation view sketch showing instrumentation labels
Figure A.33: Schematic elevation view showing dimensions (units: feet and inches)

Figure A.34: Plan view schematic showing dimensions (units: feet and inches)
Figure A.35: Left side view sketch of test wall showing dimensions (units: feet and inches)

Figure A.36: Schematic elevation view showing approximate accelerometer locations (HA = horizontal acceleration, VA = vertical acceleration)
Figure A.37: Schematic plan view showing approximation accelerometer locations

Figure A.38: Left side view showing accelerometers attached to the test wall
Figure A.39: Test wall side view (left) and front view (right) fabrication drawings showing installed reinforcement and ducts for suspension rods (units: inches)
Figure A.40: Supporting beam fabrication drawing 1 (units: inches)
Figure A.41: Supporting beam fabrication drawing 2 (units: inches)
Figure A.42: Fabrication drawing for steel posts for mounting load cells and hydraulic jacks
Appendix B  Backfill Specifications and Laboratory Testing

This appendix contains information about the backfill soil used in the presented experiments (in addition to the information provided in the previous chapters).

Table B.1: Specifications from the backfill soil supplier

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<th>Supplier's Data</th>
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| Supplier        | Vulcan Materials Company  
| Plant           | Mission Valley, San Diego  
| Material name   | MV Sand  
| Date            | Jun-06  

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<tr>
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</tr>
<tr>
<td>Silica alkali reactivity</td>
</tr>
<tr>
<td>Unit weight loose (pcf)</td>
</tr>
<tr>
<td>Unit weight rodded (pcf)</td>
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<tr>
<td>% Clay lumps, chert, friables, coal, lignite</td>
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<td>Moisture (%)</td>
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### Table B.2: Summary of conducted nuclear gauge backfill density test measurements

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Figure B.1: Direct shear data provide by Earth Mechanics, Inc. from test conducted on backfill soil at optimum moisture and 95% compaction.
Figure B.2: Direct shear data provide by Earth Mechanics, Inc. from test conducted on inundated backfill soil

STRENGTH PARAMETERS
PEAK $C = 0.41$ ksf $\phi = 43.5^\circ$
ULTIMATE $C = 0.14$ ksf $\phi = 31.7^\circ$
Figure B.3: Triaxial data provided by Leighton and Associates from tests conducted on backfill soil at optimum moisture and maximum achievable lab density.
Figure B.4: Photographs of sheared backfill soil samples after triaxial compression tests (provided by Leighton and Associates)
Figure B.5: Additional direct shear results from tests performed on backfill soil at optimum moisture and 95% compaction showing higher friction angle under lower stress condition.

Figure B.6: Estimated stress range (based on assumed linear distribution with depth) for passive pressure tests used to determine stress range for laboratory shear testing.
Appendix C  Passive Pressure Load-Displacement Test Details

Details about the passive earth pressure load-displacement tests which are not included in Chapter 2 are included in this appendix.

The contribution of friction along the sides of the container to the overall measured resistance is estimated first. As mentioned in Chapter 2, the three layer plastic system used to line the inside of the soil container yields a friction angle of about 12 degrees. The area $A$ of the passive failure wedge in contact with the side walls was about 7.65 m (including the contribution from both sides). The at-rest lateral earth pressure coefficient $K_0$ for the compacted soil is estimated at 0.5. With that $K_0$ value, the average lateral stress over the triangular area in contact with the wall is about 5.8 kPa. The resultant force normal to the side walls within the failure wedge region is 44 kN. Based on those estimates, about 10 kN of force would be required to overcome the side friction, which is about 1% of the peak resistance measured in Test 1 and Test 2. As such, it is estimated that the side friction resistance accounts for less than 2% of the peak measured resistance from Test 1 and Test 2.

During Test 1, a problem with the data acquisition system led to a loss of significant portions of data (Figure C.1). Additionally, the method of load application (independent control of the 4 hydraulic jacks) led to small wall rotations, resulting in an irregular recorded load-displacement pattern (Figure C.1 and Figure C.2). Smooth backbone curves were developed in order to highlight the overall response, as shown in Figure C.1 and Figure C.2.
Figures C.3 and C.4 show the measured backfill heave from the surface linear potentiometers, versus the lateral wall displacement. Figures C.5 through C.12 include photographs describing the passive failure wedge.
Figure C.1: Test 1 backbone curve representation of the load cell force (per meter of wall width) versus lateral displacement data

Figure C.2: Test 2 backbone curve representation of the load cell force (per meter of wall width) versus lateral displacement data
Figure C.3: Backfill vertical displacement with lateral wall movement from Test 1 (LP 1 malfunctioned after reaching about 10 mm of vertical soil displacement)

Figure C.4: Backfill vertical displacement with lateral wall movement from Test 1 (LP 1 malfunctioned after reaching more than 70 mm of vertical soil displacement)
Figure C.5: Backfill surface crack indicating the end of the passive failure wedge from Test 1

Figure C.6: Foam cores: a) before excavation, and b) during excavation
Figure C.7: Excavation of foam cores

Figure C.8: Excavated foam cores
Figure C.9: Broken foam cores showing the shape of the failure wedge (Test 1)

Figure C.10: Broken foam cores with linear approximation of failure surface (Test 1)

Figure C.11: Backfill surface after Test 2 (before passive failure condition dynamic tests)
Figure C.12: Backfill surface crack indicating the end of the passive failure wedge from Test 2 (before passive failure condition dynamic tests)
Appendix D  Dynamic Tests: Backfill Vertical Displacement

Vertical displacement measurements from the at-rest, passive, and passive failure condition dynamic tests are presented in this appendix. Figure D.1 shows the locations of the linear potentiometers (LP) which measured these displacements. Figures D.2 through D.18 include LP 1 through LP 9, and the base acceleration (as a reference). Figures D.19 through D.27 include LP 2 through LP 9 (LP 1 was damaged), and the base acceleration and load cell force (as a reference).
Figure D.1: Test configuration, sensor layout and sign convention for the at-rest and passive condition dynamic tests
Figure D.2: Backfill vertical displacement during the 1/3xEM at-rest condition test
Figure D.3: Backfill vertical displacement during the 2/3xEM at-rest condition test
Figure D.4: Backfill vertical displacement during the 1xEM at-rest condition test
Figure D.5: Backfill vertical displacement during the 2xEM at-rest condition test
Figure D.6: Backfill vertical displacement during the 3xEM at-rest condition test
Figure D.7: Backfill vertical displacement during the 3.3xEM at-rest condition test
Figure D.8: Backfill vertical displacement during 1xHM test
Figure D.9: Backfill vertical displacement during 2xHM at-rest condition test
Figure D.10: Backfill vertical displacement during the second 3.3xEM at-rest condition test
Figure D.11: Backfill vertical displacement during the second 3xEM at-rest condition test
Figure D.12: Backfill vertical displacement during the 1/3xEM passive condition test
Figure D.13: Backfill vertical displacement during the 2/3xEM passive condition test
Figure D.14: Backfill vertical displacement during the 1xEM passive condition test
Figure D.15: Backfill vertical displacement during the 2xEM passive condition test
Figure D.16: Backfill vertical displacement during the 3xEM passive condition test
Figure D.17: Backfill vertical displacement during 1xHM passive condition test
Figure D.18: Backfill vertical displacement during 1.5xHM passive condition test
Figure D.19: Backfill vertical displacement during the 1/3xEM passive failure condition test
Figure D.20: Backfill vertical displacement during the 2/3xEM passive failure condition test
Figure D.21: Backfill vertical displacement during the 1xEM passive failure condition test
Figure D.22: Backfill vertical displacement during the 2xEM passive failure condition test
Figure D.23: Backfill vertical displacement during the 3xEM passive failure condition test
Figure D.24: Backfill vertical displacement during 1xHM passive failure condition test
Figure D.25: Backfill vertical displacement during 1.5xHM passive failure condition test
Figure D.26: Backfill vertical displacement during the 3.3xEM passive failure condition test
Figure D.27: Backfill vertical displacement during the 3.5xEM passive failure condition test
Appendix E  Dynamic Tests: Backfill Deformation

Backfill deformation characteristics from the at-rest and passive condition dynamic tests are presented in the form of relative displacement time histories in this appendix. Figure E.1 shows the locations of accelerometers (represented by small circles) throughout the backfill and test configuration. Displacement at the accelerometer locations was estimated by double integrating and filtering (bandpass filter from 0.4 to 120 Hz) the recorded acceleration data.

As a basis for comparison, Figures E.2 through E.27 also include the base acceleration, velocity and displacement, the total force measured by the load cells (per meter of width), and the recorded pressure at the three instrumented depths (Figure E.1).

Relative displacements between different backfill and test configuration locations are presented as time histories in Figures E.2 through E.27. The relative displacements are labeled as the difference between to accelerometer locations according to Figure E.1. For instance, “D1 – Dbase” (Figures E.2 through E.27) is the displacement calculated from the HA 1 accelerometer data minus the displacement calculated from the container base accelerometer data; and “D1 – D3” is the displacement calculated from the HA 1 accelerometer data minus the displacement calculated from the HA 3 accelerometer data, and so on.

The relative displacement time histories presented in this appendix can provide valuable insight into the backfill deformation characteristics from the dynamic excitation tests. However, possible sources of error (e.g., numerical integration, filtering, and experimental error) should also be considered in the interpretation of these results.
Figure E.1: Test configuration, sensor layout and sign convention for the at-rest and passive condition dynamic tests
Figure E.2: 1xEM at-rest condition
Figure E.3: 1xE at-rest condition
Figure E.4: 2xEM at-rest condition
Figure E.5: 2xEM at-rest condition
Figure E.6: 3xEM at-rest condition
Figure E.7: 3xEM at-rest condition
Figure E.8: 3.3xEM at-rest condition
Figure E.9: 3.3xEM at-rest condition
Figure E.10: 1xHM at-rest condition
Figure E.11: 1xHM at-rest condition
Figure E.12: 1.5xHM at-rest condition
Figure E.13: 1.5xHM at-rest condition
Figure E.14: 3.3xEM (repeat) at-rest condition
Figure E.15: 3.3xEM (repeat) at-rest condition
Figure E.16: 3xEM (repeat) at-rest condition
Figure E.17: 3xEM (repeat) at-rest condition
Figure E.18: 1xEM passive condition
Figure E.19: 1xEM passive condition
Figure E.20: 2xEM passive condition
Figure E.21: 2xEM passive condition
Figure E.22: 3xEM passive condition
Figure E.23: 3xEM passive condition
Figure E.24: 1xHM passive condition
Figure E.25: 1xHM passive condition
Figure E.26: 1.5xHM passive condition
Figure E.27: 1.5xHM passive condition