SHALLOW FOUNDATION SYSTEMS RESPONSE TO BLAST LOADING

A Thesis Presented to

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Master of Science

by

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CHAPTER 1

INTRODUCTION

1.1 MOTIVATION FOR PERFORMING BLAST LOADING RESEARCH

Underground infrastructure such as foundation systems and utilities such as waterlines, gas lines and electrical conduit are vital lifelines to the people of the United States. Events such as the bombing of the Alfred P. Murrah Building in Oklahoma City demonstrate the seriousness of the threat of terrorist attacks on United States soil. With the new emerging threat of terrorist attacks on our nation, new steps must be taken in protecting our critical infrastructures and key assets from further terrorist exploitation.

In February 2003, the Department of Homeland Security unveiled a national strategy for protecting the vast network of roads, industrial plants, and energy systems that make up the country's critical infrastructure from possible terrorist attack. "[The strategy] provides a unifying structure, defines rules and responsibilities, and identifies major initiatives that will drive our near-term protection priorities," said President Bush in a foreward to the document.

However, if a building is not destroyed by a blast, there could be different responses of the building foundation system to the blast event. Severely damaging or destroying a foundation element could result in horizontal and vertical settlement or distortion settlement and tilt of the building. Long-term geotechnical effects, such as localized
blast-induced liquefaction or reduction in soil shear strength around a foundation element could lead to excessive settlements, footing rotation and bearing capacity failure.

1.2 RELEVENCE OF RESEARCH

Movement of a foundation element can result in many undesirable responses to the superstructure. Some of these responses include: distortion and racking of the structural frame, breaks in critical utilities entering into the building, localized failure of the building some period of time after the blast event or possible progressive collapse of the building. Even if a building is not completely destroyed as a result of a blast event, it could be rendered unusable for an extended period of time. These types of damage can cause major disruptions in terms of cost, repair and inconvenience to local citizens.

Any critical structure that is supported on a foundation system, such as bridges, power transmission towers, piers and wharfs and national monuments is vulnerable to blast loading. Many studies have been performed regarding underground infrastructure and the response to blast events. However, these past studies primarily deal with blasts that are below any magnitude that would cause major damage to nearby buried structures. They are more concerned with blasting in a controlled manner, for the purposes of mining or excavations. But with the new threat of terrorism, when a blast could be induced with the intent to cause damage, a new threat is realized. The magnitude of a blast of this nature would surely be large enough to destroy anything in its path. In this uncertain time, it is
vital to have a better understanding of this in order to protect existing underground infrastructure and to prevent massive damage to future underground infrastructure.

1.3 OBJECTIVES OF RESEARCH

The objectives of this research are listed below:

- Develop a finite element model to evaluate a foundation system response to blast loading
- Evaluate the “pseudo-static” foundation system response to blast loading
- Compare the long-term changes in the surrounding soil stress state with the soil stress state that existed before the blast loading
- Determine if the “pseudo-static” response is critical to possible foundation failure

1.4 OVERVIEW OF THESIS

Chapter 2 presents technical background information of ground shock and wave propagation theory. Empirical equations and figures are presented that can be used to relate dynamic properties of soils to ground shock prediction. Shear strength of soil for drained and undrained loading conditions is discussed and related to excess pore water pressures that are developed as a result of the blast loading. Lastly, a literature review is presented to describe the previous criterion that has been developed for blast-induced damage to structures.
Chapter 3 presents the methods that were used in the assembly of the finite element model, using the finite element software Plaxis. This includes a description of the boundary conditions used and the generation of the finite element mesh. Background information of two different soil models which were used in the analysis is also presented.

Chapter 4 provides the results of an evaluation of a free-field blast model. Included in this chapter is a presentation of two case histories that were used as a means of model calibration. Both case histories were modeled in Plaxis and the results then compared with the case history data. Parametric studies are presented which were used for the calibration of the Plaxis model. A discussion is included which compares the differences between the two soil models and the selection of the soil material model that will be used in the shallow foundation analysis.

Chapter 5 presents an evaluation of blast response of a foundation system. A static analysis of the foundation system was first performed to ascertain the ground deformations and footing movements due to a static column load. These deformations were then used as a reference for comparison with long-term deformations. The dynamic displacement response of the foundation system was then analyzed. Excess pore water pressures that were generated during the blast loading were analyzed as a way to predict blast-induced liquefaction around the foundation elements. Finally, a drained and undrained analysis was performed to study long-term changes in soil shear strength.
Chapter 6 presents a technical analysis of the research. A preliminary damage threshold was established based on the work of the U.S. Bureau of Mines. It is used to help the researchers estimate scaled distances in which varying levels of damage to a buried structure can be expected as a result of a blast loading. Pore pressure relations are also presented which can be used as a prediction tool for blast-induced liquefaction. Data from the current research is compared with data from previous studies.

Chapter 7 presents the conclusions of this research.
CHAPTER 2

TECHNICAL BACKGROUND

2.1 GROUND SHOCK AND WAVE PROPAGATION THEORY

When an explosion occurs on or below the ground surface, a phenomenon known as ground shock immediately follows. Ground shock is the propagation of blast-induced waves through earth media. Ground shock propagation in earth media is a complex function of the dynamic constitutive properties of the soil, the detonation products, and the geometry of the explosion. Due to the complexity of this type of problem, predicting ground and structure response from a blast detonation can be a very difficult task. Depending on the magnitude of explosion, ground shock has the capability to cause significant deformation of the soil matrix and subsequently cause varying levels of damage to surrounding infrastructure.

Of particular importance to ground shock is the blast-induced waves which propagate through the earth media. There are two major categories of waves that are generated as a result of an explosive detonation. According to Dowding (1985), the two main wave types can be divided into two varieties: body waves, which propagate through the body of the rock or soil, and surface waves, which are transmitted along the surface (usually the upper ground surface). The most important surface wave is the Rayleigh wave, denoted
as R-waves. Body waves can be further subdivided into compressive waves, denoted as P-Waves, and shear waves, denoted as S-Waves, as shown in Figure 2-1.

These three wave types produce drastically different patterns of motion in soil or rock particles as they pass through. Therefore, structures that are built on or in soil or rock will respond differently to each wave type. The particle motions of each wave type are shown in Figure 2-2. The compressive wave produces particle motions parallel to its direction of propagation, while the shear wave produces particle motions perpendicular to its direction of propagation. The Rayleigh wave produces particle motions both in the vertical direction and parallel to its direction of propagation.
Figure 2-2: Particle Motion of Blast Waves a) P wave, b) S wave, c) R wave

(Dowding, 1985)
Explosions produce predominately body waves at small distances. These body waves propagate outward in a spherical manner until they intersect a boundary such as another rock or soil layer, or the ground surface. At this intersection, surface waves are produced. Rayleigh surface waves become more important at larger transmission distances. At small distances, all three wave types will arrive together and greatly complicate wave-type identification, whereas at large distances, the more slowly moving shear and surface waves begin to separate from the compressive wave, Dowding (1985). Another type of surface wave is the Love wave. Love waves produce particle motions perpendicular to its direction of propagation, on the ground surface. The Love wave is less important than the Rayleigh wave and is generally associated more with earthquakes and seismic activity, rather than blasting.

2.2 BLAST RESPONSE IN FREE-FIELD CONDITIONS

If no structure comes into contact with ground shock waves as they transmit through a soil medium, the resulting condition is referred to as the free-field condition. Prediction of particle displacement, velocity, acceleration, pressure and other parameters in the earth media resulting from an explosive detonation is a complicated and difficult task. Primary guidance on this topic is from the United States Department of the Army Technical Manual “Fundamentals of Protective Design for Conventional Weapons”, TM 5-855-1 (1986). Other guidance is given by Bulson (1997), which provides empirical formulae and relationships to describe and predict these parameters. TM 5-855-1 (1986) and
Bulson (1997) are generally complimentary to one another and are typically used together to predict blast response and blast-induced parameters.

2.2.1 DYNAMIC MATERIAL PROPERTIES

Seismic velocity is often used as a rough index of soil or rock properties for ground shock prediction. Seismic velocity, or compressive wave velocity, provides a measure of the stiffness and density of the soil through which the ground shock propagates. Seismic velocity is given as,

\[ c = \frac{\sqrt{M}}{\rho_o} \]  

(2-1)

where, \( c \) = Seismic velocity  
\( M \) = Stiffness modulus of soil (Constrained modulus)  
\( \rho_o \) = Mass density of soil

The constrained modulus, \( M \), can be defined in terms of elastic soil parameters using the relationship,

\[ M = \frac{(1-\nu)E}{(1+\nu)(1-2\nu)} \]  

(2-2)
where, \( E \) = Modulus of elasticity of soil

\( v \) = Poisson’s ratio of soil

A similar expression can be defined for the shear wave velocity,

\[
c_s = \frac{G}{\sqrt{\rho_o}} \quad (2-3)
\]

where, \( G \) = Shear modulus of soil

The compressive waves travel at velocities much higher than shear waves. Shear waves and Rayleigh waves travel at approximately the same velocity. The soil parameters used in the wave velocity equations can be obtained from lab and field data of the given soil type, or from tables providing typical values of these parameters for various soils.

2.2.2 DYNAMIC SOIL RESPONSE

There are many forms in which soils can be loaded dynamically. A few of these include construction vibrations, wave propagation due to earthquakes and wave propagation due to explosive detonations. When a blast occurs, the resulting internal stress and particle velocity of a soil can be defined by exponential type time-histories. These values of stress and velocity quickly rise to a peak value and then exponentially decay to nearly zero over
time. The time of arrival of a blast, $t_a$, is the elapsed time from the instant of detonation to the time at which the ground shock arrives at a given location. Time of arrival is given as,

$$t_a = \frac{R}{c}$$  \hspace{1cm} (2-4)

where,

- $R =$ Absolute distance from the explosion
- $c =$ Seismic velocity of media

Figure 2-3 shows a typical time-history diagram for pressure or velocity.
After the ground shock has arrived at a given location, the stress and velocity begin to decay according to the following equations,

\[ P(t) = P_0 e^{-\alpha t} \]  

\[ V(t) = V_0 (1 - \beta t/t_a) e^{-\beta t/t_a} \]
where, \[ P(t) = \text{Shock stress} \]

\[ V(t) = \text{Particle velocity} \]

\[ P_0 = \text{Peak stress} \]

\[ V_0 = \text{Peak velocity} \]

\[ t = \text{Time} \]

\[ \alpha, \beta = \text{Time Constants} \]

These time constants generally vary with specific site conditions, however, as per guidance given in TM 5-855-1 (1986), they can be taken as \( \beta = 0.4 \) and \( \alpha = 1.0 \). Bulson (1997) also reports these same values for \( \alpha \) and \( \beta \). They have also been confirmed from experiments performed by the U. S. Army.

The peak values of free-field stress and velocity, which are used in Equations 2-5 and 2-6, are given as,

\[
P_0 (\text{psi}) = f (\rho c) 160 \left( \frac{R}{W^{1/3}} \right)^{-\alpha} \quad (2-7)
\]

\[
V_0 (\text{fps}) = f 160 \left( \frac{R}{W^{1/3}} \right)^{-\alpha} \quad (2-8)
\]

where, \( P_0 = \text{Peak pressure (psi)} \)

\( V_0 = \text{Peak particle velocity (fps)} \)
\[ pc = \text{Acoustic impedance (psi / fps)} \]
\[ R = \text{Distance to the explosion (ft)} \]
\[ W = \text{Charge weight (lb)} \]
\[ n = \text{Attenuation coefficient} \]
\[ f = \text{Ground coupling factor} \]

It should be noted that Equations 2-7 and 2-8 are empirical and thus the parameters must be given in the units shown. The attenuation coefficient, seismic velocity and acoustic impedance can be estimated based on the soil type and properties. Both TM 5-855-1 (1986) and Bulson (1997) present ranges of values for these ground shock parameters based on measurements collected during various explosion tests. Figures 2-4 and 2-5 present the data reported in TM 5-855-1 (1986).

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Seismic Velocity c (fps)</th>
<th>Acoustic Impedance (pc) (psi/fps)</th>
<th>Attenuation Coefficient n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose, dry sands and gravels with low relative density</td>
<td>600</td>
<td>12</td>
<td>3–3.25</td>
</tr>
<tr>
<td>Sandy loam, loess, dry sands, and backfill</td>
<td>1,000</td>
<td>22</td>
<td>2.75</td>
</tr>
<tr>
<td>Dense sand with high relative density</td>
<td>1,000</td>
<td>44</td>
<td>2.5</td>
</tr>
<tr>
<td>Wet sandy clay with air voids (greater than 4 percent)</td>
<td>1,800</td>
<td>48</td>
<td>2.5</td>
</tr>
<tr>
<td>Saturated sandy clays and sands with small amount of air voids (less than 1 percent)</td>
<td>5,000</td>
<td>130</td>
<td>2.25–2.5</td>
</tr>
<tr>
<td>Heavy saturated clays and clay shales</td>
<td>&gt;5,000</td>
<td>150–180</td>
<td>1.5</td>
</tr>
</tbody>
</table>

*Figure 2-4: Soil Properties for Ground Shock Parameters (TM 5-855-1, 1986)*
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Dry Unit Weight $\gamma_{\text{dry}}$ pcf</th>
<th>Total Unit Weight $\gamma$ pcf</th>
<th>Air-Filled Voids %</th>
<th>Seismic Velocity c fps</th>
<th>Acoustic Impedance $\rho c$ psi/fps</th>
<th>Attenuation Coefficient $n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry desert alluvium and playa, partially cemented</td>
<td>87</td>
<td>93-100</td>
<td>&gt;25</td>
<td>2,100-4,200$^a$</td>
<td>40</td>
<td>3-3.25</td>
</tr>
<tr>
<td>Loose, dry poorly graded sand</td>
<td>80</td>
<td>90</td>
<td>&gt;30</td>
<td>600</td>
<td>11.8</td>
<td>3-3.5</td>
</tr>
<tr>
<td>Loose, wet, poorly graded sand with free-standing water</td>
<td>97</td>
<td>116</td>
<td>10</td>
<td>500-600</td>
<td>12.5-15</td>
<td></td>
</tr>
<tr>
<td>Dense dry sand, poorly graded</td>
<td>99</td>
<td>104</td>
<td>32</td>
<td>900-1,300</td>
<td>25</td>
<td>2.5-2.75</td>
</tr>
<tr>
<td>Dense wet sand, poorly graded, with free-standing water</td>
<td>109</td>
<td>124</td>
<td>9</td>
<td>1,000</td>
<td>22</td>
<td>2.75</td>
</tr>
<tr>
<td>Very dense dry sand, relative density $\geq$100%</td>
<td>105</td>
<td>109</td>
<td>30</td>
<td>1,600</td>
<td>44</td>
<td>2.5</td>
</tr>
<tr>
<td>Silty-clay, wet</td>
<td>95-100</td>
<td>120-125</td>
<td>9</td>
<td>700-900</td>
<td>18.25</td>
<td>2.75-3</td>
</tr>
<tr>
<td>Moist loose, clayey sand</td>
<td>100</td>
<td>122</td>
<td>5-10</td>
<td>1,000</td>
<td>28</td>
<td>2.75-3</td>
</tr>
<tr>
<td>Very wet sandy clay, above water table</td>
<td>95</td>
<td>120-125</td>
<td>4</td>
<td>1,800</td>
<td>48</td>
<td>2.5</td>
</tr>
<tr>
<td>Saturated sand, below water table in marsh</td>
<td>--</td>
<td>--</td>
<td>1-4$^b$</td>
<td>4,900</td>
<td>125</td>
<td>2.25-2.5</td>
</tr>
<tr>
<td>Saturated sandy clay, below water table</td>
<td>78-100</td>
<td>110-124</td>
<td>1-2</td>
<td>5,000-6,000</td>
<td>130</td>
<td>2-2.5</td>
</tr>
<tr>
<td>Saturated sandy clay, below water table</td>
<td>100</td>
<td>125</td>
<td>&lt;1</td>
<td>5,000-6,000</td>
<td>130-180</td>
<td>1.5</td>
</tr>
<tr>
<td>Saturated stiff clay, saturated clay-shale</td>
<td>--</td>
<td>120-130</td>
<td>0</td>
<td>&gt;5,000</td>
<td>135</td>
<td>1.5</td>
</tr>
</tbody>
</table>

$^a$ High because of cementation.

$^b$ Estimated.

Figure 2-5: Soil Properties From Explosion Tests (TM 5-855-1, 1986)
Figure 2-6 presents values for the ground shock coupling factor as a function of media type (i.e. soil, concrete, air) and scaled depth of burst. The ground shock coupling factor, $f$, describes how the magnitude of the stress and ground motions will be greatly enhanced as the weapon penetrates further into the soil before it detonates, and is introduced to correct for this effect, TM 5-855-1 (1986). A ground shock coupling factor of one or greater indicates that a given blast is buried deep enough that all of the blast energy is released into the surrounding earth media and none is lost to the atmosphere. The depth of burial when the ground shock coupling factor is equal to one is called the optimum depth of burial (ODB).
2.2.3 DYNAMIC PORE PRESSURE RESPONSE

An explosive detonation is characteristic of undrained conditions. Upon blast detonation, the soil experiences an instantaneous applied external stress. This causes an instantaneous increase in excess pore pressure that is not able to drain quickly. Three states that exist during blast loading of a soil were described by Shim (1996) in Figure 2-7. These states are the static condition, the transient or dynamic condition and the residual or pseudo-static condition.

Figure 2-7: Excess Pore Water Pressure Measurement (Shim, 1996)
In many circumstances it is important to know the strength and deformation characteristics of soils under transient loading. However, the current research is more concerned with long term or residual changes to soil strength. This occurs after the transient loading, but a build up of excess pore pressures still exist. These residual excess pore pressures could still be high enough that the soil may be susceptible to failure. As previously stated, for conditions in which excess pore water pressures do not dissipate with time, the effective stresses are reduced. This ultimately leads to a permanent reduction of soil shear strength, which could cause a localized failure of a buried structure.

Most research involving blast-induced liquefaction has resulted in empirical relationships for prediction purposes. The pore water pressure increases in soil, including liquefaction potential can be estimated by using the pore pressure ratio (PPR) (Veyera et al., 2002). The pore pressure ratio is defined as,

$$PPR = \frac{u_{\text{res}}}{\sigma'_{\text{o}}}$$  \hspace{1cm} (2-12)

where,

- $u_{\text{res}} = \text{Residual excess pore water pressure}$
- $\sigma'_{\text{o}} = \text{Initial effective stress}$
2.2.4 SHEAR STRENGTH OF SOIL

The failure mode of all soils is shearing. Therefore, the most important property of soils is shear strength. The shear strength of a soil is the shear stress of the soil on its failure plane at the time of failure. The failure plane of an individual soil particle is the shearing surface on which the soil fails, for a given loading. Drainage conditions heavily influence the shear strength of a soil. Drained conditions occur when excess pore pressure developed during loading is allowed to dissipate and the excess pore pressure goes to zero. Undrained conditions occur when excess pore pressures do not dissipate immediately. It is noted that for conditions in which excess pore pressures do not dissipate with time, this condition is referred to as residual pore pressure. The existence of these conditions is a function of soil type, geologic formation and the rate of loading. Because shear strength is a function of effective stress, undrained loading conditions are the most critical for soils. From the Mohr-Coulomb relationship, shear strength can be written as,

\[ S = c + \sigma' \tan \varphi = c + (\sigma - u) \tan \varphi \]  \hspace{1cm} (2-9)

where, \( S \) = Shear strength
\( \sigma' \) = Effective stress
\( \sigma \) = Total stress
\( u \) = Total pore pressure
\[ c = \text{Cohesion intercept} \]

\[ \phi = \text{Phi angle} \]

2.2.4.1 DRAINED AND UNDRAINED STRENGTH

For undrained conditions, total pore pressure and total stress are defined as,

\[ u = u_o + u_e \]  \hspace{1cm} (2-10)

\[ \sigma = \sigma_o + \Delta \sigma \]  \hspace{1cm} (2-11)

where,

- \( u_o \) = Initial pore pressure
- \( u_e \) = Excess pore pressure
- \( \sigma_o \) = Initial stress
- \( \Delta \sigma \) = Applied external stress

Based on these basic relationships, it can be seen that as the excess pore pressure increases, the effective stress decreases, which moves the soil closer to its failure plane. This causes a loss in soil shear strength. This concept can be illustrated by the Mohr’s Circle shown in Figure 2-8.
A blast event is characteristic of undrained loading conditions because it is a very quick and rapid loading event. As a result of the blast load, permanent changes in pore pressure have caused permanent changes in the undrained shear strength of the soil. In saturated normally-consolidated soil, the cohesion is approximately zero. The shear strength of the soil for undrained loading conditions can then be calculated using Equation 2-12.

\[
S = \frac{\sin \phi' [k_o + A_F (1 - k_o)] \sigma'}{1 + (2A_F - 1) \sin \phi'}
\]

(2-12)

where, \( S = \) Shear strength
\[ \sigma' = \text{Effective stress} \]
\[ \varphi' = \text{Effective phi angle} \]
\[ k_o = \text{Lateral earth coefficient at rest} \]
\[ A_F = \text{Skempton pore pressure parameter} \]

The long-term changes in soil shear strength can be analyzed by studying how the final stress state has changed the drained shear strength of the soil. Drained conditions are characteristic of long-term changes to the soil matrix because the longer amount of time allows for the excess pore water pressures to dissipate. The shear strength of the soil for drained loading conditions can be calculated using Equation 2-13.

\[ S = \sigma' \tan \varphi \]  \hspace{1cm} \text{(2-13)}

2.2.4.2 STRESS PATHS

A good way to analyze the long-term changes in shear strength is through the use of a stress path. A stress path is a line that connects a series of points of maximum shear stress throughout a loading sequence. Lambe (1964) suggested a type of stress path that plots \( q \) against \( p \), where \( q \) and \( p \) are the coordinates of the top of a Mohr's circle. For undrained conditions, the top of the Mohr's circle is the shear strength. This can be explained with the aid of Figure 2-9. The coordinates of the Mohr's circle are given in Equations 2-14 to 2-16.
Most situations in geotechnical engineering involve a static ground water table; thus an initial pore water pressure, $u_o$, acts on a given element. There are three stress paths that
are considered, the effective stress path (ESP), the total stress path (TSP), and the 
\((T - u_o)\)SP. These three stress paths are shown in Figure 2-10 for a normally consolidated 
clay with an initial pore water pressure \(u_o\) undergoing axial compression loading. It can 
be noted that as long as the ground water table remains at the same elevation, \(u_o\) does not 
affect either the ESP or the conditions at failure (Holtz and Kovacs, 1981). The \(k_o\) line 
shown in Figure 2-10 represents constant ratios of stress.

*Figure 2-10: ESP, TSP, and \((T - u_o)\)SP for a Normally Consolidated Clay (After Holtz and 
Kovacs, 1981)*
2.2.5 CUBE AND SQUARE ROOT SCALING

Scaling of distance is necessary to predict peak particle velocities when both the charge weight per delay, $W$, and the distance, $R$, vary. The two most popular approaches are square root scaling and cube root scaling, given in Equations 2-17 and 2-18, respectively.

\[
\frac{R}{W^{1/2}} \tag{2-17}
\]

\[
\frac{R}{W^{1/3}} \tag{2-18}
\]

Both approaches are employed to compare field data and to predict the attenuation or decay of peak particle velocity. Cube root scaling typically matches results from deeply buried point charges and was developed in connection with small-scale modeling of nuclear blasts. The fundamental assumption of cube root scaling is that the variation in $W$ and $R$ is much more significant than that of $\rho$ and $c$ for a given soil mass. Despite large variations of $W$, peak particle velocity values within a given soil mass display a consistent relationship with the scaled range, $R/W^{1/3}$ (Dowding, 1985).

Square root scaling is the distance, $R$, divided by the square root of the charge weight, $R/W^{1/2}$ and is more traditional than cube root scaling. Square root scaling is based on the observation that the charge is distributed in a long cylinder or blast hole and typically
matches results from row charges, line charges, and near-surface charges generating surface waves (Charlie et al., 2001). Therefore, the diameter of the hole is proportional to the square root of the charge weight per unit length of hole, if the density is constant. The ratio of \( \frac{R}{W^{1/2}} \) is more or less the ratio between the distance to the blast and the radius of the blast hole that is proportional to \( W^{1/2} \) (Dowding, 1985).

Slight differences occur between the use of cube and square root scaling, which can be attributed to the principles in which each scaling method was developed. Despite these differences, both can be used for prediction of peak particle velocity with good accuracy.

2.2.6 RAYLEIGH DAMPING

Natural dynamic systems show some degree of damping when subjected to dynamic loads. In soils, damping is mainly due to loss of energy resulting from internal friction in the material and viscous properties. A damping term known as Rayleigh damping can be determined to account for natural damping and will be used in the finite element analysis for this research.

The basic equation of motion of a body under the influence of a dynamic load is,

\[
[M] \ddot{u} + [C] \dot{u} + [K] u = F
\]  
(2-19)
where,

\[ [M] = \text{Mass matrix} \]

\[ [C] = \text{Damping matrix} \]

\[ [K] = \text{Stiffness matrix} \]

\[ \ddot{\mathbf{u}} = \text{Acceleration vector} \]

\[ \dot{\mathbf{u}} = \text{Velocity vector} \]

\[ \mathbf{u} = \text{Displacement vector} \]

\[ \{F\} = \text{Load vector} \]

The mass and stiffness matrices are determined by numerical integration and are given as,

\[
[M] = \int \rho [N]^T [N] dV \tag{2-20}
\]

\[
[K] = \int [B]^T [D] [B] dV \tag{2-21}
\]

where,

\[ [N] = \text{Interpolation function} \]

\[ [B] = \text{Strain-displacement relationship} \]

\[ [D] = \text{Elastic material stiffness matrix representing Hooke’s Law} \]

\[ \rho = \text{Material density} \]

\[ V = \text{Volume} \]
The matrix $C$ represents the material damping of the materials. In reality, material damping is caused by friction or irreversible deformations (plasticity or viscosity). With more viscosity or more plasticity, more vibration energy can be dissipated. If elasticity is assumed, damping can still be taken into account using the damping matrix, $C$. However, other parameters are necessary in order to determine the damping matrix. In finite element formulations, the matrix $C$ is often formulated as a function of the mass and stiffness matrices, and is known as Rayleigh damping (Plaxis, 2002).

The damping matrix is then given as,

$$[C] = \alpha_R[M] + \beta_R[K]$$  \hspace{1cm} (2-22)

where, $\alpha_R, \beta_R$ = Rayleigh damping coefficients

The Rayleigh damping coefficients are used to determine the damping matrix. However, the Rayleigh damping coefficients are difficult to determine. According to Plaxis (2002), the Rayleigh damping coefficients $\alpha_R$ and $\beta_R$ can be determined from damping ratios that correspond to natural frequencies of vibration. The relationship between these parameters is,

$$\alpha_R + \beta_R \omega^2 = 2\omega \zeta$$  \hspace{1cm} (2-23)
whee, \[ \zeta = \text{Damping ratio} \]
\[ \omega = \text{Natural frequency of vibration} \]

The damping ratios and natural frequencies can be determined experimentally by tests performed on the given material, such as the resonant column test.

2.3 DYNAMIC SOIL-STRUCTURE INTERACTION

2.3.1 DYNAMIC RESPONSE OF BURIED STRUCTURES

The dynamic response of buried structures is dependent of the characteristics of the blast and blast geometry, the properties of the surrounding soil, and the properties of the structure itself. There are several ways in which a buried structure can be loaded by detonated explosions. The structure may be buried at a sufficient depth beneath the surface to escape the effects of the blast-induced pressures resulting from an above ground explosion. Structural loading can then result from air-induced ground shock which arises from the passage of the shock front across the surface of the soil above the structure. If a large explosion occurs on the surface of the soil, or below the surface of the soil, the ground shock can travel directly through the soil on its way to the structure. Underground structural loading is also influenced by the flexibility of the structure in relation to the properties of the surrounding soil and the soil-structure interaction effects that occur as the structure begins to deflect under shock pressures. The structure and its
surrounding soil form a composite body. The loading can then be influenced by soil-arching and the disturbance to the natural properties of the soil (Bulson, 1997).

### 2.3.2 DAMAGE TO BURIED STRUCTURES FROM BLAST WAVE

One of the ways in which blast-induced damage to a buried structure can occur is by the ground shock traveling through the soil and impacting the structure. The blast-induced loading can cause varying levels of damage from minor cracking to catastrophic failure. It should be noted here that the current research is not concerned with damage that could occur to the superstructure. It is more focused on substructures.

A dynamic soil-structure interaction problem was studied by Ann et al. (2002) and involved modeling of a reinforced soil wall subject to a blast load. Explosive testing of a geotextile reinforced soil wall was carried out in field experiments. The experiments were performed in order to study the dynamic behavior of the reinforced soil wall subject to blast loading. Ann et al. (2002) modeled the problem using the finite element software PLAXIS version 7.12. Results from the finite element model were then compared with the results from the field experiments.

A Mohr-Coulomb soil model was used in the finite element analysis. Two different blast events were applied to the reinforced soil wall. One had a peak pressure near 180 kPa and the other near 130 kPa. The loads were applied as a blast pressure in the form of a
uniformly distributed load over the length of the reinforced soil wall. The load
instantaneously rose to its peak value and then decayed to zero over time.

The results of the finite element analysis compared very well to the field data. Ann et al.
(2002) noted that the Plaxis results show there is a residual stress in the soil after blasting,
which is what would be expected. These results prove that PLAXIS can be used to
correctly model a dynamic soil-structure interaction problem using the Mohr-Coulomb
soil model. Upon this validation, the current researchers chose to use PLAXIS for finite
element modeling of the current research.

2.3.3 BLAST-INDUCED LIQUEFACTION

In addition to damage to buried structures due to blast-induced ground shock, damage can
also happen as a result of bearing capacity failure or excessive settlements of the
foundation systems. Excessive settlements and bearing capacity failures can be attributed
to the loss of shear strength or liquefaction of the surrounding soil. When a blast occurs
in soil, the soil matrix itself is deformed and pore water pressures instantaneously
increase. As previously described, this results in a reduction in strength due to the
decrease in the effective stress of the surrounding soil.

Liquefaction occurs in a soil body when the effective stresses go to zero. Consequently
the shear strength of the soil also dissipates and the soil loses its capacity to provide
lateral support for any type of buried structure. In effect, the soil behaves as a liquid without any shear strength. This can lead to damage to buried structures.

While blast-induced liquefaction may not necessarily damage a facility structurally, it may render it unusable. Blast-induced liquefaction can cause late-time decreases in the soil’s strength that results in damage disproportional to the amount of explosive used. Late-time implies loss of shear strength at a time after the transient blast event has passed. Late-time liquefaction can lead to catastrophic consequences including landslides, foundation failure, settlement of piles, flotation of buried buoyant structures and ground subsidence.

Veyera et al., (2002) studied the response of shock-induced pore pressure in saturated carbonate sand. The study was experimentally carried out to evaluate the transient and residual pore water pressure responses of Enewetak coral sand, for undrained uniaxial confined compressive shock-loading conditions. A blast event was simulated by impacting the specimens with compressive shock loads. The goal of the research was to study the influence of effective stress and relative density on liquefaction potential of the sand. According to Veyera et al. (2002), liquefaction occurs when the effective stress in a water-saturated cohesionless soil reaches zero due to increases in the residual excess pore water pressure. This liquefied state could last for a significant amount of time depending on drainage conditions. Based on the experimental results, Veyera et al. (2002) determined that liquefaction of Enewetak coral sand can be induced by dynamic, one-
dimensional, confined compressive loading under undrained conditions. Veyera et al. (2002) also states that little information is available about liquefaction induced by dynamic compressive stresses from sources such as explosions, pile driving, impact loading and dynamic compaction.

The pore pressure ratio was used to study liquefaction potential of the Enewetak coral sand. Veyera et al. (2002) report that a pore pressure ratio of unity indicates that a soil has been liquefied. A pore pressure ratio between 0.6 and 1.0 indicates that a soil is liable for liquefaction. A pore pressure ratio less than 0.5 indicates that a soil is safe from liquefaction. A pore water pressure ratio of zero indicates that no residual excess pore water pressure was developed by the shock loading.

Empirical equations were also presented to predict pore pressure ratio values for coral sand and Monterey No. 0/30 sand. These empirical relationships are presented in Equations 2-24 and 2-25 for the two types of sand, respectively.

\[ PPR = (5.81)(\Sigma e_{pk})^{429}(\sigma'_{o})^{-176}(D_r)^{-022} \quad (2-24) \]

\[ PPR = (16.3)(\Sigma e_{pk})^{331}(\sigma'_{o})^{-308}(D_r)^{-179} \quad (2-25) \]

where, \[ \Sigma e_{pk} = \text{Cumulative peak compressive strain from successive impacts} \]
\[ D_r = \text{Relative density} \]
Charlie et al. (2001) investigated blast-induced pore pressure in a sandfill dam.

According to Charlie et al. (2001), blasting near earthfill dams has the potential to increase residual pore pressure, reduce the dam's stability, induce settlements, or cause other damage. This research was primarily concerned with the potential for blast-induced residual pore pressure increases to reduce the shear strength of the soil long enough to allow a gravity caused slope failure of earth structures. Settlement may also result due to these blast-induced vibrations.

A prototype earthfill dam was constructed of Lytle sand. Parameters measured during testing were residual pore pressure, peak water pressure during the passage of the compressive stress wave, and particle velocity. Explosives were placed 0.8m below the water level and 2.7m from the toe of the dam. Eight tests were carried out with charge weight ranging from 0.44 g to 1.5 kg.

Charlie et al. (2001) offer empirical relationships between peak particle velocity and pore pressure ratio, based on the results of their experiments. These relationships are given in Equation 2-26. Charlie et al. (2001) also report that excess pore pressure in saturated soil can be generated by nearby blasting, which can lead to settlements and other damage.
\[ PPR = 4,025 \left( \frac{R}{W^{1/3}} \right)^{2.08} (\sigma_0)^{1.56} = 520,000 (V_{\text{peak}})^{3.01} (\sigma_0)^{1.56} \]  

(2-26)

where,  
\[ \frac{R}{W^{1/3}} = \text{Cube root scaled distance} \]

\[ V_{\text{peak}} = \text{Peak particle velocity} \]

### 2.3.4 DAMAGE TO FOUNDATION SYSTEMS

Research involving the response of pile foundations in saturated soil under blast loading was studied by Shim (1996). Damage to piles was assessed using centrifuge model testing of aluminum piles. A field test was also carried out on prototype prestressed concrete piles. Shim (1996) states that there are several modes in which piles in saturated soil and subject to blast loading may fail. These include bending failure, compressive failure of concrete, spalling of concrete, crushing of hollow pipes, shear failure, punching failure, excessive settlement and buckling. Buckling, shear failure, punching failure and settlement all can be directly related to liquefaction. As soil reaches the point of liquefaction, the soil strength dissipates to zero. Therefore, the soil can no longer provide lateral support to a pile or a shallow foundation. Failure of a pile or a foundation system can lead to detrimental effects for the superstructure that is being supported.

Shim (1996) presented the response of piles subjected to buried blasts in the form of time-history data of bending moments. Lateral deflections and bending moments induced in the pile resulting from a blast were used to determine how a pile will respond.
Assumptions were made to determine any damage that may occur to a pile based on the results of the tests. According to Shim (1996), bending stiffness, EI, is the dominant parameter in determining the development of bending moments on a pile. Bending moments and deflections on the pile can be compared to the ultimate moment capacity and deflection criteria of the pile. If these ultimate values are exceeded, damage will occur. All bending moment data exhibited a rapid build-up followed by a slow decay. Data from strain gages at different depths on the same pile showed a strong consistency. Therefore, a detailed analysis of the bending moment profile along the pile could be performed. Figure 2-11 shows the deformed piles after testing.
Most non-military research conducted regarding blast-induced damage to structures was performed by the U.S. Bureau of Mines. Studies were conducted in order to find a limiting criterion for blast-induced ground vibrations that cause damage to residential structures, including basements. Many mining and blast operations were occurring next to residential neighborhoods, and blasting criteria needed to be established to avoid

Figure 2-11: Deformed Piles After Testing (After Shim, 1996)
annoyance to neighbors and damage to residential property. Bureau of Mines studies Bulletin 656 performed by Nicholls et al. (1971), RI 8507 performed by Siskind et al. (1980) and RI 8969 performed by Siskind et al. (1985) presented criteria for blasting near residential structures. One of the most important conclusions coming from the U.S. Bureau of Mines studies was the use of peak particle velocity as the best index of measure of blast-induced damage potential. Nicholls et al. (1971) and Siskind et al. (1980) report that safe levels of ground vibration range from 0.5 to 2.0 in/sec peak particle velocity to prevent cracking for residential structures.

According to Dowding (1985), nearby structures can be cracked during blasting through four mechanisms: 1) Permanent ground distortion created by gas pressures 2) Vibratory settlement of foundation materials 3) Fly rock impact 4) Vibratory cracking from ground vibration or air blast. Cracks can also be induced due to differential foundation settlement. Dowding (1985) presented levels for classifying damage for residential structures, as follows,

- **Threshold**: Opening of old cracks, formation of new plaster cracks
- **Minor**: Not affecting structural strength (e.g. loosened or fallen plaster), cracks in masonry.
- **Major**: Serious weakening of the structure (e.g. large cracks, shifting of foundations or bearing walls, major settlement resulting in distortion or weakening of the superstructure.
Some blast vibration studies are presented in Figure 2-12, which show damage to residential structures, including basements. Damage is classified as previously discussed. Figures 2-13 and 2-14 present damage from blasting and safe levels of peak particle velocity. As stated before, this research concerns blast-induced damage to residential structures. However, this criterion can be used as a beginning point for the current research to investigate blast-induced damage to other buried structures or foundation systems.

Other research has been performed to establish blasting criterion for other types of structures as well. Pal Roy (1998) states that in India, a long-term research program was undertaken by the Central Mining Research Institute (CMRI) for the determination of vibration threshold values of typical Indian structures in the vicinity of operating mines. Natural frequencies of structures and cracking levels due to repeated blasting effects were studied. Tables 2-15 and 2-16 show results of some of the studied performed by CMRI. The Pal Roy (1998) criteria compares well to the Siskind et al. (1980) criteria for appearance of cracks at peak particle velocity values of approximately 2 in/sec (50 mm/sec).

An investigation by Lindsey (1989) examined the need for determining safe vibration levels for highway structures in the vicinity of blasting operations. The measured response of a highway bridge to blasting vibrations was taken on a bridge deck and piers. Lindsey (1989) report that because highway bridges are engineered structures, the
<table>
<thead>
<tr>
<th>Study</th>
<th>Damage classifications</th>
<th>Types of damage</th>
<th>Overburden type</th>
<th>Structures studied</th>
<th>Distances to shot, ft</th>
<th>Shot sizes, half decay</th>
<th>Frequency range, Hz</th>
<th>Total shots</th>
<th>Non-damage</th>
<th>Threshold</th>
<th>Minor</th>
<th>Major</th>
<th>Instrumentation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thoenen and Widos, Bureau of Mines (31)</td>
<td>Threshold and minor.</td>
<td>Plaster cracks and fall of plaster.</td>
<td>None</td>
<td>6 frame, brick, and stone, 1 to 3 story.</td>
<td>None</td>
<td>None</td>
<td>4-40</td>
<td>1165</td>
<td>105</td>
<td>26</td>
<td>34</td>
<td>0</td>
<td>Displacement.</td>
</tr>
<tr>
<td>Langevin, Westergren, and Kahn (50)</td>
<td>Minor and major.</td>
<td>Rock</td>
<td>NA</td>
<td>NA</td>
<td>46-120</td>
<td>105</td>
<td>22</td>
<td>16</td>
<td>5</td>
<td>32</td>
<td>16</td>
<td>NA</td>
<td>NA.</td>
</tr>
<tr>
<td>Edwards and Northwood (16)</td>
<td>Threshold, minor, and major.</td>
<td>Cracks in masonry, bricks, or stone basement walls.</td>
<td>Solid, wet sand with clay 30 ft down, and well-consolidated glacial till.</td>
<td>50-200</td>
<td>67-750</td>
<td>22</td>
<td>6</td>
<td>8</td>
<td>5</td>
<td>4</td>
<td>5</td>
<td>Displacement and acceleration measured on basement walls.</td>
<td></td>
</tr>
<tr>
<td>Northwood, Crawford, and Edwards (34)</td>
<td></td>
<td>Basement wall damage close in, and superstructure plus basement damage far out.</td>
<td>Glacial till and limestone overlain by thin till layer.</td>
<td>3-500</td>
<td>0.5-1,000</td>
<td>60</td>
<td>10</td>
<td>4</td>
<td>5</td>
<td>0</td>
<td>0</td>
<td>Velocity, MB-120 gage, measured on basement wall.</td>
<td></td>
</tr>
<tr>
<td>Thoenen and Widos, Bureau of Mines (31)</td>
<td>Threshold and minor.</td>
<td>None</td>
<td>10 quarries</td>
<td>713-2,500</td>
<td>350-1,200</td>
<td>14 total</td>
<td>45</td>
<td>11</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Displacement and acceleration.</td>
<td></td>
</tr>
<tr>
<td>Morris and Westover (31)</td>
<td>Threshold</td>
<td>Plaster and partition cracks.</td>
<td>1 quarry and 1 surface coal mine.</td>
<td>113-320</td>
<td>200-14,000</td>
<td>3.7-3.7</td>
<td>7</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Displacement.</td>
<td></td>
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<td>Dvorak (15)</td>
<td>Threshold, minor, and major.</td>
<td>Plaster and masonry cracks.</td>
<td>4 brick and masonry.</td>
<td>50-164</td>
<td>2.3-4</td>
<td>1.5-15</td>
<td>7</td>
<td>25</td>
<td>15</td>
<td>11</td>
<td>0</td>
<td>Do.</td>
<td></td>
</tr>
<tr>
<td>Wis and Nichols (37)</td>
<td>Minor</td>
<td>Drywall cracks</td>
<td>Glacial till</td>
<td>55-200</td>
<td>1.85</td>
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<td>9</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>Velocity, MB-120 gage.</td>
<td></td>
</tr>
<tr>
<td>Jensen and Riesman (37)</td>
<td></td>
<td>Rock with 0 to 7 ft of soil overburden.</td>
<td>18 frame structures.</td>
<td>50-183</td>
<td>1.75-12.75</td>
<td>111-126</td>
<td>29</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>Do.</td>
<td></td>
</tr>
<tr>
<td>Bureau of Mines new data</td>
<td>Threshold and minor.</td>
<td>Plaster, Drywall, and masonry cracks.</td>
<td>Various, usually with soil overburden.</td>
<td>14-2,500</td>
<td>18-2,000</td>
<td>6.5-71</td>
<td>225</td>
<td>37</td>
<td>23</td>
<td>5</td>
<td>0</td>
<td>Do.</td>
<td></td>
</tr>
</tbody>
</table>

1 Shaker uses.
2 Excavation in rock, small shots.
3 Predominately 10 to 25 Hz for damage data.
4 Plus 1 to 5 Hz.
5 Mostly >30 Hz.
6 NA = Not available.

Figure 2-12: Studies of Damage to Residences From Blasting Vibration (Siskind et al., 1980)
Table 2-1

<table>
<thead>
<tr>
<th>Damage effects</th>
<th>Peak particle velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sand, gravel, clay below water level; c = 1,000 - 1,500 m/sec(^1)</td>
</tr>
<tr>
<td></td>
<td>mm/sec</td>
</tr>
<tr>
<td>No noticeable crack formation</td>
<td>18</td>
</tr>
<tr>
<td>Fine cracks and failure peak threshold</td>
<td>30</td>
</tr>
<tr>
<td>Crack formation</td>
<td>40</td>
</tr>
<tr>
<td>Severe cracks</td>
<td>60</td>
</tr>
</tbody>
</table>

\(^1\) Propagation velocity in m/s is given by c.

Figure 2-13: Damage Levels From Blasting (Siskind et al., 1980)

Table 2-2

<table>
<thead>
<tr>
<th>Type of construction</th>
<th>Peak particle velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Incoherent loose soils, soft cohesive soils, rubble mixtures: c = 1,000 m/sec(^1)</td>
</tr>
<tr>
<td></td>
<td>mm/sec</td>
</tr>
<tr>
<td>Special care, historical monuments, hospitals, and very tall buildings</td>
<td>2.5</td>
</tr>
<tr>
<td>Current construction</td>
<td>5</td>
</tr>
<tr>
<td>Reinforced construction, e.g., earthquake resistant</td>
<td>15</td>
</tr>
</tbody>
</table>

\(^1\) Propagation velocity in m/s is given by c.

Figure 2-14: Limiting Safe Vibration Values of Peak Particle Velocity (Siskind et al., 1980)
<table>
<thead>
<tr>
<th>PPV (mm/s)</th>
<th>Place of measurement</th>
<th>Type of structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appearance of first crack</td>
<td>50.0</td>
<td>Ground floor</td>
</tr>
<tr>
<td>Appearance of first crack</td>
<td>76.0</td>
<td>First floor</td>
</tr>
<tr>
<td>Appearance of first crack</td>
<td>400.0</td>
<td>Side walls</td>
</tr>
<tr>
<td>Widening of existing cracks</td>
<td>90.0</td>
<td>Ground floor</td>
</tr>
<tr>
<td>Deep cracks</td>
<td>240.0</td>
<td>x</td>
</tr>
<tr>
<td>Falling of plaster</td>
<td>280.0</td>
<td>x</td>
</tr>
<tr>
<td>Severe cracks</td>
<td>192.0</td>
<td>Side walls and corners</td>
</tr>
</tbody>
</table>

Figure 2-15: Minimum Values of Vibration at which Damage Occurred (Pal Roy, 1998)

<table>
<thead>
<tr>
<th>Type No.</th>
<th>Structural specification</th>
<th>Threshold value of PPV (mm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>Domestic houses, dry wall interior, structures with plaster, bridges</td>
<td>≤ 24 Hz</td>
</tr>
<tr>
<td>(b)</td>
<td>Industrial buildings, steel or reinforced concrete structures</td>
<td>5.0</td>
</tr>
<tr>
<td>(c)</td>
<td>Objects of historical importance, very sensitive structures, more than 50 years old, structures with poor state of condition and unrepaired</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0</td>
</tr>
</tbody>
</table>

Figure 2-16: CMRI-Established threshold Values of Ground Vibration at the Foundation Level (Pal Roy, 1998)
damage potential is small if the ground vibration limits of 2.0 in/sec recommended by the U.S. Bureau of Mines are employed. Although the U.S. Bureau of Mines studies were related to mine blasting and damage potential to residential structures, Lindsey (1989) applied this criteria to highway bridges. The U.S. Bureau of Mines blasting criteria can be used as a platform to investigate different types of blast events and the response to different types of structures.
CHAPTER 3

FINITE ELEMENT MODEL ASSEMBLY

3.1 BOUNDARY CONDITIONS

Soil and structures are not always subject to static loads, but often dynamic loads such as earthquakes, winds or blasting. The finite element program Plaxis 2D – Version 8 (2002) allows for analysis of the dynamic effects of ground vibrations in soil. One of the most important parameters of a finite element model is the boundary conditions that are imposed on the model. Correct boundary conditions are essential to ensure that the data provided by the model is accurate. Plaxis provides built in boundary conditions for use in a dynamic analysis. These boundary conditions are referred to in Plaxis as Standard Fixities and Standard Absorbent Boundaries. Standard Fixities imposes a set of general boundary conditions which apply roller supports to the horizontal boundary sides of the model and fixed supports along the bottom vertical boundary of the model.

3.1.1 ABSORBENT BOUNDARIES

According to Plaxis (2002), an absorbent boundary is required to absorb the increments of stresses on the boundaries caused by dynamic loading, that otherwise would be reflected inside the soil body. When performing a dynamic analysis in Plaxis, absorbent boundaries must be included. For an axisymmetric two-dimensional model, standard
absorbent boundaries are generated by Plaxis at the bottom and right-hand boundaries of the model. The left-hand boundary is the axis of symmetry and the top boundary represents the ground surface. Therefore, absorbent boundaries are not applied to these two sides of the model. When performing a dynamic analysis, it is also important to choose a model size having dimensions a significant distance away from the vibration source. This helps to avoid unwanted and unrealistic reflection of ground shock waves (Yang, 1997).

3.1.2 RELAXATION COEFFICIENTS

Along with the absorbent boundaries, Plaxis provides relaxation coefficients $C_1$ and $C_2$, which are used to help improve the wave absorption on the absorbent boundaries. Dissipation of waves in the direction normal to the boundary is corrected by $C_1$, while $C_2$ corrects for wave dissipation in the tangential direction.

The absorbent boundaries used in Plaxis are viscous boundaries, or dampers. According to Waterman (2002), viscous boundaries are dashpots that dissipate the stress increase due to reflection at the boundaries. Plaxis (2002) defines the stress components in the dashpot in Equations 3-1 and 3-2.

$$\sigma = C_1 \cdot \rho \cdot V_p \cdot V_x$$ (3-1)
where, \[\sigma = \text{Normal stress}\]
\[\tau = \text{Shear stress}\]
\[\rho = \text{Material density}\]
\[V_p = \text{Compressive wave velocity}\]
\[V_s = \text{Shear wave velocity}\]
\[V_x = \text{Horizontal velocity}\]
\[V_y = \text{Vertical velocity}\]

Plaxis (2002) recommends using values of \(C_1 = 1\) and \(C_2 = 0.25\) for all practical applications. These factors are based on a comparison of calculation results to theoretical formulas for a cantilever beam under dynamic load. The best fit between calculation results and theoretical solutions was reached for \(C_1 = 1\) and \(C_2 = 0.25\). These other values for relaxation coefficients should be used for more theoretical cases when it is absolutely certain that only compression waves occur.

3.2 MESH GENERATION

Proper construction of the finite element mesh is necessary in order to accurately represent the behavior and response of a given problem. Plaxis has a built in finite element mesh generator, which develops the finite element mesh.
3.2.1 GLOBAL COARSENESS

The global coarseness represents the size of the individual elements that make up the finite element mesh. As the element size becomes smaller, the accuracy of the predicted results increases. The element size that is ultimately used is a function of the problem that is being modeled. In areas of a model where absolute representation of material behavior is necessary, a smaller element size should be used. In areas where absolute representation is not the case, larger element sizes can be used. According to Plaxis (2002), the mesh generator uses a general meshing parameter which represents the average element size, $l_c$. In Plaxis, this parameter is calculated from the outer geometry dimensions and a global coarseness setting and is given in Equation 3-3.

$$l_c = \sqrt{\frac{(x_{\text{max}} - x_{\text{min}})(y_{\text{max}} - y_{\text{min}})}{n_c}}$$  \hspace{1cm} (3-3)

where,

- $l_c = \text{Average element size}$
- $x_{\text{max}} = \text{Maximum horizontal geometry dimension}$
- $x_{\text{min}} = \text{Minimum horizontal geometry dimension}$
- $y_{\text{min}} = \text{Minimum vertical geometry dimension}$
- $y_{\text{max}} = \text{Maximum vertical geometry dimension}$
- $n_c = \text{Element size factor}$
The average element size and the number of elements generated depend on this global coarseness setting. There are five levels of global coarseness available in Plaxis. A rough estimate, based on a generation without local refinement, is given in Table 3-1.

<table>
<thead>
<tr>
<th>Mesh Coarseness</th>
<th>$n_e$</th>
<th>Approximate No. of Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Coarse</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>Coarse</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>Medium</td>
<td>100</td>
<td>250</td>
</tr>
<tr>
<td>Fine</td>
<td>200</td>
<td>500</td>
</tr>
<tr>
<td>Very Fine</td>
<td>400</td>
<td>1000</td>
</tr>
</tbody>
</table>

### 3.2.2 LOCAL COARSENESS

The local coarseness is a parameter that allows for a more accurate mesh in certain areas of the geometry. This may be necessary if large deformations are expected in particular areas, while other parts of the geometry may not be as important to the overall analysis. It allows for more accurate results in specified areas of the geometry by making the finite element mesh finer. The finer mesh provides smaller element sizes, which increase accuracy. In Plaxis, local coarseness is defined by the Local Element Size Factor (LESF). The LESF gives an indication of the relative element size with respect to the average element size previously defined by the global coarseness setting. By default, the LESF is set to a value of 1.0 at all geometry points. This indicates that the local element size is
equal to the global element size. The LESF can be changed at any point within the geometry. But, values must be in the range of 0.05 to 5.0.

The LESF can be adjusted and changed through the use of mesh refinement. Each time the finite element mesh is refined, the LESF becomes one-half of its previous value. The finite element mesh can be refined repeatedly as long as the LESF is within its given range.

3.3 SOIL MODELS

Proper selection of a soil material model that will accurately predict expected soil deformations is important for any analysis. The soil material models offered by Plaxis include the Mohr-Coulomb model and the Jointed Rock model, and advanced soil models including the Hardening-Soil model, Soft-Soil model, Soft-Soil-Creep model or a user-defined soil model.

3.3.1 MOHR-COULOMB MODEL

The Mohr-Coulomb soil model is a linear elastic, perfectly-plastic model. A perfectly-plastic model is defined as a constitutive model with a fixed yield surface, i.e. a yield surface that is fully defined by model parameters and not affected by plastic straining. The Mohr-Coulomb yield surface is shown in Figure 3-1. For stress states represented by
points within the yield surface, the behavior is purely elastic and all strains are reversible. The basic principle of elastoplasticity is that strains and strain rates are decomposed into two parts, elastic and plastic. Through Hooke’s Law, stress rates can be related to elastic strain rates (Plaxis, 2002). Figure 3-2 illustrates the concept of linear elastic, perfectly-plastic behavior.

![Mohr-Coulomb Yield Surface](Plaxis, 2002)

*Figure 3-1: Mohr-Coulomb Yield Surface (Plaxis, 2002)*
The Mohr-Coulomb strength equation is given in Equation 3-4.

\[ \tau_f = c + \sigma_f \tan \varphi \]  

(3-4)

where,

\( \tau_f \) = Shear stress on the failure plane at failure

\( \sigma_f \) = Normal stress on the failure plane

\( c \) = Cohesion intercept

\( \varphi \) = Angle of internal friction

Figure 3-2: Linear-Elastic, Perfectly-Plastic Model
The angle of internal friction, $\phi$, and cohesion, $c$, are referred to as strength parameters. Young’s Modulus of elasticity and Poisson’s ratio are the basic stiffness parameters used for the elastic Mohr-Coulomb model. The input parameters for the Mohr-Coulomb soil model are given in Table 3-2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>Phi Angle</td>
<td>Degree</td>
</tr>
<tr>
<td>$c$</td>
<td>Cohesion</td>
<td>kPa</td>
</tr>
<tr>
<td>$E$</td>
<td>Young’s Modulus</td>
<td>kPa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s Ratio</td>
<td>-</td>
</tr>
</tbody>
</table>

### 3.3.2 HARDENING-SOIL MODEL

The yield surface of a hardening plasticity model is not fixed in principal stress space, like that of a linear elastic, perfectly-plastic model. It can expand as a result of plastic straining, and is shown in Figure 3-3. There are two main types of hardening: shear hardening and compression hardening. Irreversible strains due to primary deviatoric loading are modeled with strain hardening. Irreversible plastic strains due to primary compression in oedometer and isotropic loading are modeled with compression hardening.
When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. The Hardening-Soil model involves a relationship between rates of plastic strain. For the special case of the drained triaxial test, the relationship between the axial strain and the deviatoric stress can be approximated by a hyperbola. Figure 3-4 shows these relationships for the hyperbolic representation of the Hardening-Soil model. By using the theory of plasticity, in lieu of the theory of elasticity, by including soil dilatancy and introducing a yield cap, the Hardening-Soil model provides an advanced model for simulating soil behavior (Plaxis, 2002).
As previously stated, the formulation of the Hardening-Soil model is the hyperbolic relationship between the vertical strain, \( \varepsilon \), and the deviatoric stress, \( q \), in primary triaxial loading. Equations 3-5 to 3-7 describe relationships of the Hardening-Soil model, Plaxis (2002). Standard drained triaxial tests tend to provide curves described by,

\[
\varepsilon_1 = \frac{1}{2E_{s0}} \cdot \frac{q}{1 - q/q_a}
\]  

(3-5)

where,

- \( \varepsilon_1 \) = Vertical strain
- \( q \) = Deviatoric stress
- \( q_a \) = Asymptotic value of the shear strength
- \( E_{s0} \) = Confining stress dependent stiffness modulus for primary loading
The parameter $E_{50}$ is given by,

$$
E_{50} = E_{50}^{\text{ref}} \cdot \left( \frac{c \cos \phi - \sigma'_3 \sin \phi}{c \cos \phi + p^{\text{ref}} \sin \phi} \right)^m
$$

(3-6)

where,

- $E_{50}^{\text{ref}}$ = Secant stiffness modulus
- $p^{\text{ref}}$ = Reference confining pressure
- $m$ = Power for stress-level dependency of stiffness
- $\phi$ = Angle of internal friction
- $c$ = Cohesion

For unloading and reloading, another stress-dependent stiffness modulus is used,

$$
E_{ur} = E_{ur}^{\text{ref}} \cdot \left( \frac{c \cos \phi - \sigma'_3 \sin \phi}{c \cos \phi + p^{\text{ref}} \sin \phi} \right)^m
$$

(3-7)

where,

- $E_{ur}^{\text{ref}}$ = Reference Young’s modulus for unloading and reloading

The advantage of the Hardening-Soil model over the Mohr-Coulomb model is not only the use of a hyperbolic stress-strain curve instead of a bi-linear curve, but also the control of stress level dependency (Plaxis, 2002). This is advantageous because in the Mohr-Coulomb model, a fixed value of Young’s modulus is used, where in real soil this stiffness depends on the stress level. Table 3-3 provides the parameters that make up the
Hardening-Soil model. Some of the parameters are those from the Mohr-Coulomb model, after Plaxis (2002).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Failure Parameters as in Mohr-Coulomb Model</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Phi Angle</td>
<td>Degree</td>
</tr>
<tr>
<td>c</td>
<td>Cohesion</td>
<td>kPa</td>
</tr>
<tr>
<td><strong>Basic Parameters for Soil Stiffness</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_{50\text{ ref}}$</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>kPa</td>
</tr>
<tr>
<td>$E_{oed\text{ ref}}$</td>
<td>Tangent stiffness for primary oedometer loading</td>
<td>kPa</td>
</tr>
<tr>
<td>m</td>
<td>Power for stress-level dependency of stiffness</td>
<td>-</td>
</tr>
<tr>
<td><strong>Advanced Parameters</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_{ur\text{ ref}}$</td>
<td>Unloading / reloading stiffness</td>
<td>kPa</td>
</tr>
<tr>
<td>$\nu_{ur}$</td>
<td>Poisson's ratio for unloading / reloading</td>
<td>-</td>
</tr>
<tr>
<td>$P_{ref}$</td>
<td>Reference stress for stiffness</td>
<td>kPa</td>
</tr>
<tr>
<td>$k_{o\text{ nc}}$</td>
<td>$k_o$ value for normal consolidation</td>
<td>-</td>
</tr>
<tr>
<td>$R_{t}$</td>
<td>Failure ratio $q_t / q_a$</td>
<td>-</td>
</tr>
<tr>
<td>$q_t$</td>
<td>Ultimate deviatoric stress</td>
<td>kPa</td>
</tr>
<tr>
<td>$q_a$</td>
<td>Asymptotic value of shear strength</td>
<td>kPa</td>
</tr>
</tbody>
</table>
3.4 MODEL ASSEMBLY

The model used in this research utilized the principles that have just been discussed. A two-dimensional axisymmetric model with standard absorbent boundaries was used for the dynamic analysis. Parametric studies were completed for various values of the relaxation coefficients, which help improve wave absorption on the absorbent boundaries. No noticeable difference was noted in calculation results, from changes made to the relaxation coefficients. Based on the results of this parametric study and knowing that shear waves occur during an explosive detonation, the recommended values for the relaxation coefficients were used.

The concept of mesh refinement and local and global coarseness used for the model is shown in Figure 3-5, for a finite element mesh generated by Plaxis. The blast location can be seen in the upper left corner of the geometry. Because it is important to have a finer mesh near the area of the blast event, the finite element mesh was locally refined in this area, as can be seen in the blown-up view window. The remaining area of the finite element mesh was not refined. It should be noted that the global coarseness is the same for the entire mesh. However, by locally refining the mesh around the blast event, the LESF has decreased and made the average element size smaller.
The soil model that was initially chosen to model the soil in the current analysis was the Mohr-Coulomb model. The Mohr-Coulomb is an elastic soil model that allowed for an initial investigation into the problem at hand. The Hardening-Soil model was then used to model the soil in order to make comparisons between the two different soil model types.

Figure 3-5: Effects of Mesh Refinement
CHAPTER 4

EVALUATION OF FREE-FIELD MODEL

4.1 MODEL CALIBRATION

The first step in the modeling process was to verify that a blast simulation could be correctly and successfully modeled using Plaxis. This step was accomplished by modeling free-field conditions (i.e. no buried structures in the soil mass) presented in case histories. Two case histories for free-field blasting were modeled. During the modeling process, the parameters of the finite element model were calibrated by performing a variety of parametric studies. The calculation results of the Plaxis models were then compared to case history data and empirical relationships given by TM 5-855-1 (1986). Upon calibrating the Plaxis models to the case history data and empirical relationships, the research could move forward and model other blasting situations with confidence that the Plaxis model would yield accurate results.

4.1.1 YANG (1997) CASE HISTORY

Yang (1997) performed research using the commercial finite element software ABAQUS to predict the response of buried shelters to blast loadings. Part of the research involved a free-field analysis, which was investigated by using a 2-D finite element analysis in which a viscoelastic soil model was used. Yang (1997) then compared free-field
pressures generated in the finite element analysis to those calculated from the empirical relationships given in TM 5-855-1 (1986).

In the analysis, the detonation of the blast was represented by a pressure load that was applied to the circumference of a circle with a given radius, whose center coincides with that of the explosive charge. The pressure loading on the circumference of the circle was calculated using equations given in TM 5-855-1 (1986). The soil properties used in the Yang (1997) analysis are given in Table 4-1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>Modulus of elasticity</td>
<td>3.45E+04</td>
<td>kPa</td>
</tr>
<tr>
<td>ν</td>
<td>Poisson's ratio</td>
<td>0.3</td>
<td>-</td>
</tr>
<tr>
<td>ρ</td>
<td>Mass density</td>
<td>1.70E+03</td>
<td>kg/m³</td>
</tr>
<tr>
<td>ρc</td>
<td>Acoustic impedance</td>
<td>5.00E+05</td>
<td>Pa·sec/m</td>
</tr>
<tr>
<td>W</td>
<td>Charge weight</td>
<td>60</td>
<td>N</td>
</tr>
<tr>
<td>H</td>
<td>Charge depth</td>
<td>10</td>
<td>m</td>
</tr>
<tr>
<td>f</td>
<td>Ground shock coupling factor</td>
<td>1</td>
<td>-</td>
</tr>
</tbody>
</table>

Using this data, Yang (1997) produced the Figures 4-1 and 4-2, which plots free-field pressure versus time at two different locations from the blast, for two different values of seismic velocity. The points of interest are located at 7.33 m and 9.81 m from the blast. The seismic velocity of 165 m/s is calculated from the elastic properties of the soil. The other value of 300 m/s is a commonly used value for soils, according to Yang (1997).
Figure 4-1: Free-Field Pressure for $R = 7.33$ m (Yang, 1997)
These figures prove the validity of the equations presented in TM 5-855-1 (1986) by observing that the finite element results compare well to the empirical equations. The present research also verified these equations by hand calculations using the soil properties given by Yang (1997), which were then compared to the finite element results.
Research performed by Rosengren et al. (1999) modeled the propagation of ground shock waves as well as the interaction between soil and structures. A numerical model was calibrated against free-field data for a buried charge. The computer code FLAC was used for the calibration modeling. The Mohr-Coulomb soil model was one of the models used in the free-field research. The computer program CONWEP, which is based on field data compiled by the U.S. Army was also used to estimate explosion. The CONWEP program provides peak particle velocity data at selected points outside the blast crater, for a given scenario. Soil properties chosen by Rosengren et al. (1999) characterized a moraine. The soil properties provided are given in Table 4-2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>ρ</td>
<td>Mass density</td>
<td>1.90E+03</td>
<td>kg/m³</td>
</tr>
<tr>
<td>c</td>
<td>Seismic velocity</td>
<td>1000</td>
<td>m/s</td>
</tr>
<tr>
<td>ρc</td>
<td>Acoustic impedance</td>
<td>1.90E+06</td>
<td>Pa·sec/m</td>
</tr>
<tr>
<td>φ</td>
<td>Phi Angle</td>
<td>30</td>
<td>Degree</td>
</tr>
<tr>
<td>c</td>
<td>Cohesion</td>
<td>0</td>
<td>kPa</td>
</tr>
<tr>
<td>W</td>
<td>Charge weight</td>
<td>245</td>
<td>N</td>
</tr>
<tr>
<td>H</td>
<td>Charge depth</td>
<td>0.61</td>
<td>m</td>
</tr>
<tr>
<td>f</td>
<td>Ground shock coupling factor</td>
<td>0.9</td>
<td>-</td>
</tr>
</tbody>
</table>

*Table 4-2. Soil Properties Provided by Rosengren et al. (1999)*
Rosengren et al. (1999) modeled the blast detonation using the same methods as Yang (1997). A dynamic load was applied on a previously determined crater boundary. The dynamic load was represented by a uniformly distributed pressure acting normal to the crater boundary. Figure 4-3 illustrates this concept, as well as boundary conditions used. Viscous boundaries at a reasonable distance were applied to the bottom and right vertical boundary. The left vertical boundary coincides with the axis of symmetry and has fixed normal displacements.

*Figure 4-3: Blast Pressure and Boundary Conditions*
Figure 4-4 was developed for the free-field analysis for two different points located at distances of 4.0 m and 8.0 m directly beneath the blast.

![Graph showing particle velocity over time for Points P1 and P2](image)

*Figure 4-4: Particle Velocity for Points P1 and P2 (Rosengren et al., 1999)*

4.1.3 MODELING METHODS

The Plaxis model used 15-node triangular elements to model the soil. The 15-node triangular element provides a fourth order interpolation of displacements and the numerical integration involves 12 Gauss points (stress points). The geometry was simulated by means of an axisymmetric model in which the blast crater is positioned along the axis of symmetry. The deformation and stress state are assumed to be identical
in any radial direction. The axisymmetric model results in a 2D finite element model with two translational degrees of freedom per node.

The blast detonation was modeled in the same manner as that of Yang (1997) and Rosengren et al. (1999). A dynamic load in the form of a uniformly distributed pressure load was applied to the boundary of the crater. The dynamic pressure load was made to instantaneously increase in magnitude to its peak value and then decay to zero after certain duration. The dynamic pressure-time history load was input into Plaxis by means of a user-defined ASCII file. The pressure-time history load was calculated using equations given in TM 5–855-1 (1986). Peak pressures were found to be 10 MPa for the Yang (1997) case history and 20 MPa for the Rosengren et al. (1999) case history. Figure 4-5 shows the pressure-time histories used for the blast events in modeling the Yang (1997) and Rosengren et al. (1999) case histories.
Figure 4-5: Pressure-Time Histories

4.2 RESULTS AND COMPARISONS

4.2.1 YANG (1997) – MOHR-COULOMB MODEL

The first case history to be modeled was that of Yang (1997). The Mohr-Coulomb soil model was used along with the soil and blast properties that were provided by Yang (1997). Data was recorded by Yang (1997) at two points located at horizontal distances of 7.33 m and 9.81 m away from the blast detonation, as show in Figure 4-6.
The model calibration began by choosing an appropriate domain for the finite element model, as was discussed in Section 3.1.1. For the case of a blasting situation, the boundary should be set at a sufficient distance away from the blast detonation to simulate an infinite boundary. The point under consideration for the calibration study is Point 1 ($R = 7.33$ m). Figure 4-7 plots particle velocity against time and shows the effects of the model domain on peak particle velocity. It can be seen that as the model domain increases, the peak particle velocities decreases and becomes closer to the empirical value. Table 4-3 provides the data from Figure 4-7.
Figure 4-7: Effect of Model Domain Size on PPV
By studying Figure 4-7, it can be seen that a substantial amount of wave reflection is occurring over time. The smooth exponential decay is not evident, as predicted by the empirical equation. It is apparent that the peak particle velocity decreases as the domain size increases, however, increasing the model domain does not seem to dampen out the noticeable wave reflection. This wave reflection is most likely due to the lack of material damping in the system. In order to eliminate this, damping was introduced into the soil material model by means of Rayleigh damping. Plaxis allows the use of Rayleigh damping by the $\alpha$ and $\beta$ damping parameters. Figures 4-8 to 4-10 plots particle velocity against time and shows the effects of Rayleigh damping on the system. Due to the difficulty in determining damping coefficients, the best possible fit for the Rayleigh $\alpha$ and $\beta$ was determined by a trial and error calibration method. This was done by holding the $\alpha$ value constant for three different values and then varying the $\beta$ value over a given range. It can be seen from Figures 4-8 to 4-10 that there is little to no difference when the $\alpha$ value is changed. Hence, it was adopted that the best fit to the empirical value of peak particle velocity occurred for $\alpha = 0.001$ and $\beta = 0.01$. 

<table>
<thead>
<tr>
<th>Domain Size (m)</th>
<th>PPV (m/sec)</th>
<th>Arrival Time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 x 20</td>
<td>0.592</td>
<td>0.043</td>
</tr>
<tr>
<td>50 x 50</td>
<td>0.364</td>
<td>0.042</td>
</tr>
<tr>
<td>100 x 100</td>
<td>0.325</td>
<td>0.042</td>
</tr>
<tr>
<td>200 x 200</td>
<td>0.313</td>
<td>0.042</td>
</tr>
<tr>
<td>500 x 500</td>
<td>0.218</td>
<td>0.048</td>
</tr>
</tbody>
</table>
Figure 4-8: Effect of Rayleigh Damping on PPV ($\alpha = 0.001$)
Figure 4-9: Effect of Rayleigh Damping on PPV ($\alpha = 0.01$)
By introducing damping into the system, the decay of the particle velocity curve is a much better match to the empirical relationship. This can be seen by the absence of wave reflection and by the smoother decay of the curve from the peak value. However, by introducing damping into the system, the arrival time of the blast to the specified point changed drastically. This can easily be seen when comparing the arrival time values of the curves in Figure 4-7 to those in Figures 4-8 to 4-10. This was a trend also observed by Yang (1997), which also used damping in the free-field analysis. Yang (1997) compared
results of the viscoelastic soil model with damping to two seismic velocity values. One value was 165 m/s, calculated directly from the elastic constants of the soil, while the other was 300 m/s, a value from tables given in TM 5-855-1 (1986) and commonly used for soils. Yang (1997) noted that for the viscoelastic model, the finite element results were observed to compare much better to the seismic velocity of 300 m/s. It was concluded that calculations directly from the elastic properties of a given soil is not the most accurate method to determine seismic velocity. It can be used as an acceptable estimate, but may not compare well to field conditions. The current research shows similar results with regards to the seismic velocity for the Mohr-Coulomb model. These observations agree with TM 5-855-1 (1986), which states that seismic velocity provides a crude index of soil and rock properties for ground shock prediction purposes.

The results of the finite element analysis compare well to the empirical results when a seismic velocity of 300 m/s is used, in lieu of 165 m/s. Finite element results obtained from the current analysis are compared to the same two values of seismic velocity in Figure 4-11. In Figure 4-11, the seismic velocity of 165 m/s is calculated directly from the elastic properties of the soil given by Yang (1997). The value of 300 m/s is taken from TM 5-855-1 (1986) for dry dense sand. Results from the Yang (1997) analysis and the current analysis show that regardless of the soil’s elastic properties, values for seismic velocity and other parameters given in TM 5-855-1 (1986) and Bulson (1997) can be used in the empirical equations. This phenomenon occurs because the values presented in
TM 5-855-1 (1986) and Bulson (1997) are based on results from actual field tests of blasting in soils. They are not based on the principles of elastic theory.

![Graph showing Plaxis Particle Velocity vs. $c = 165$ m/sec and $c = 300$ m/sec](image)

*Figure 4-11: Plaxis Particle Velocity vs. $c = 165$ m/sec and $c = 300$ m/sec*

The final parametric study performed for model calibration involved changing the global coarseness, as well as localized refinement of the finite element mesh. As previously discussed, changing the global coarseness of the finite element mesh as well as the local refinement provides a smaller average element size. Smaller element sizes will result in a more accurate representation of the soil behavior. Figure 4-12 plots particle velocity
against time and shows the effects of changing the global coarseness (i.e. changing the average element size). It should be noted that there is no localized mesh refinement for the curves in Figure 4-12. The curves presented in Figure 4-13 do take mesh refinement into account and shows the effects of local mesh refinement. This shows that local refinement of the fine mesh in the area around the blast detonation yields the best results. Results from these studies provided a basis for localized mesh refinement only around the blast area.

\[
\text{Medium Mesh} = \text{Approx. 250 Elements} \\
\text{Fine Mesh} = \text{Approx. 500 Elements}
\]

*Figure 4-12: Effects of Global Mesh Coarseness on Particle Velocity*
Upon completion of the parametric studies, the best possible model parameters were chosen for modeling the Yang (1997) case history. The results of these models are presented in Figures 4-14 to 4-16 and show that the Plaxis model was able to replicate the free-field analysis performed by Yang (1997) reasonably well. The Plaxis model also matched the empirical equations presented in TM 5-855-1 (1986) with acceptable accuracy by matching the peak particle velocity values and arrival times.
Figure 4-14: Plaxis vs. Empirical Particle Velocity for Points 1 and 2
Figure 4-15: Plaxis vs. Yang (1997) vs. Empirical Pressure for Point 1
The Rosengren et al. (1999) case history was chosen to further verify the sufficiency of Plaxis to model blast response. Rosengren et al. (1999) performed a free-field analysis on a moraine soil, using the Mohr-Coulomb model. There are a couple reasons why the Rosengren et al. (1999) case history was chosen for modeling. The first being that Rosengren et al. (1999) chose soil properties that characterized a moraine, as opposed to
generic soil properties characterizing a sand as in the Yang (1997) case history. This would enable the current researcher to establish that different soil conditions could be successfully modeled. The second reason for modeling was that Rosengren et al. (1999) used the Mohr-Coulomb soil model. This would allow for a direct comparison of results from two analyses that both utilized the Mohr-Coulomb model.

Model parameters such as domain size, Rayleigh damping coefficients, mesh coarseness and mesh refinement had previously been determined during the calibration of the Yang (1997) case history. It was assumed that these same parameters could be adopted throughout the research and would be used for all models. By successfully modeling the Rosengren et al. (1999) case history, this assumption could be verified.

Soil and blast properties provided by Rosengren et al. (1999) and model parameters obtained from the Yang (1997) case history calibration were used for modeling. Rosengren et al. (1999) provided peak particle velocity data for two points located at vertical distances of 4.0 m and 8.0 m below the explosive charge, as shown in Figure 4-17. The peak particle velocity data was obtained from the Mohr-Coulomb analysis and from the computer program CONWEP. Figures 4-18 to 4-19 and Table 4-4 show the results of the Plaxis model compared to Rosengren et al. (1999), CONWEP and empirical relationships given in TM 5-855-1 (1986).
Figure 4-17: Rosengren et al. (1999) Blast Schematic
Figure 4-18: Plaxis vs. Rosengren et al. (1999) vs. Empirical Particle Velocity for Point 1
Figure 4-19: Plaxis vs. Rosengren et al. (1999) vs. Empirical Particle Velocity for Point 2
Table 4-4. Comparison of Peak Particle Velocity

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.93</td>
<td>3.93</td>
<td>2.65</td>
<td>2.57</td>
</tr>
<tr>
<td>3</td>
<td>0.68</td>
<td>0.62</td>
<td>1</td>
<td>0.78</td>
</tr>
</tbody>
</table>

From Figures 4-18 to 4-19 and Table 4-4, it can be seen that the results obtained from the Plaxis analysis are in good agreement with the results presented by Rosengren et al. (1999). This establishes that blast events can be accurately modeled in Plaxis using a Mohr-Coulomb soil model.

4.2.3 YANG (1997) – HARDENING-SOIL MODEL

The Yang (1997) case history was modeled again, but with a more advanced soil model. The Hardening-Soil model was chosen as the soil model to use for this advanced analysis. By modeling with the Hardening-Soil model, differences could be explored between a linear-elastic, perfectly-plastic soil model and a plasticity model.

Soil and blast properties provided by Yang (1997) were again used for modeling. Guidance for the selection of the Hardening-Soil model advanced parameters not given by Yang (1997) was provided by Plaxis and work done by Roboski (2001). Roboski (2001) provided typical values of the Hardening-Soil model advanced parameters for various soil types. Parameters were chosen for the soil representing the soil that was
modeled by Yang (1997). From Roboski (2001) and several parametric studies, the parameters that resulted in the best fit of the Plaxis model data to that of Yang (1997) data were determined. These parameters are listed in Table 4-5.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{so}$</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>5,400</td>
<td>kPa</td>
</tr>
<tr>
<td>$E_{sod}$</td>
<td>Tangent stiffness for primary oedometer loading</td>
<td>4,264</td>
<td>kPa</td>
</tr>
<tr>
<td>$E_{ur}$</td>
<td>Unloading / reloading stiffness</td>
<td>11,710</td>
<td>kPa</td>
</tr>
<tr>
<td>$m$</td>
<td>Power for stress-level dependency of stiffness</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>$P_{ref}$</td>
<td>Reference stress for stiffness</td>
<td>100</td>
<td>kPa</td>
</tr>
<tr>
<td>$R_f$</td>
<td>Failure ratio $q_f / q_a$</td>
<td>0.7</td>
<td>-</td>
</tr>
</tbody>
</table>

Model parameters such as domain size, Rayleigh damping coefficients and mesh coarseness and refinement were the same as in the Mohr-Coulomb case histories. Upon completion of the Hardening-Soil model analysis, it became evident that the values of the Rayleigh damping coefficients were not adequate. A trial and error method was again used to determine the best fit for the Rayleigh damping coefficients. From the results of the Rayleigh damping parametric study, the best fit to the empirical curve occurs for $\alpha = 0.035$ and $\beta = 0.025$. 

With the final parameters of the Hardening-Soil model in place, a direct comparison could then be made to the Mohr-Coulomb model results. Figures 4-20 and 4-21 show comparisons of particle velocity and pressure obtained from the Hardening-Soil model to empirical equations.

Figure 4-20: Plaxis vs. Empirical Particle Velocity for Point 1
Figure 4-21: Plaxis vs. Empirical Pressure for Point 1
A comparison can be made between the Mohr-Coulomb model and the Hardening-Soil model results for the Yang (1997) case history. It can be seen from Figures 4-22 and 4-23 that for both particle velocity and pressure, the Mohr-Coulomb soil model yields reasonably good results. Peak particle velocity, pressure and arrival times are matched closer to the empirical equations by use of the Mohr-Coulomb model. The velocity curve decay of the Hardening-Soil model appears to match the empirical curve better than the Mohr-Coulomb model. However, the peak particle velocity and arrival time values are not as accurate as in the Mohr-Coulomb model. The pressure curve produced from the Hardening-Soil model is also not as close to the empirical curve, when compared to the Mohr-Coulomb model results.
Figure 4-22: Mohr-Coulomb vs. Hardening-Soil vs. Empirical Particle Velocity for Point 1
One of the goals of the current research is to make a preliminary investigation whether blast-induced ground vibrations will cause long-term changes to a soil matrix, therefore affecting any structural elements supported by the soil. Because this is a preliminary investigation into this problem, absolute representation of soil behavior is not possible. The analysis using the Hardening-Soil model was performed as an investigation into whether the use of a more advanced soil model would immediately yield results that were noticeably more accurate than those of the Mohr-Coulomb model. As this was not the
case, it was decided the Hardening-Soil model would not be used. The difficulty of obtaining input parameters for the Hardening-Soil model is a further reason not to use it. Based on the results of this evaluation, the Mohr-Coulomb soil model was chosen for the remainder of the analysis. It was concluded that the Mohr-Coulomb soil model and the parameters used are the best fit for this project. The Mohr-Coulomb soil model will sufficiently reflect soil behavior for the purposes of this project.
5.0 EVALUATION OF BLAST RESPONSE OF A FOUNDATION SYSTEM

5.1 FOUNDATION SYSTEM

The type of structures modeled in the analysis was shallow square footings. In the literature review of this project, little to no existing research could be found regarding blast effects on shallow foundations. Some research has been performed on this problem by the U.S. Army, but is not available for civilian use. Therefore, shallow foundations were chosen as the structure type for analysis. The general schematic that was followed for the analysis of the shallow foundations is shown in Figure 5-1.
**Figure 5-1: Schematic for Shallow Foundation Analysis**
5.1.1 FOOTING PROPERTIES

Three different square footing sizes were chosen for analysis, with the purpose of analyzing blast effects over a range of footing sizes. The footing sizes chosen were 1.5 m, 3.0 m and 4.5 m. The thickness for all footings was assumed to be 0.457 m. A typical square shallow foundation is shown in Figure 5-2. The footings were assumed to be made entirely of concrete, having a compressive strength, $f_c = 20680$ kPa. Because Plaxis does not allow modeling of steel reinforcing bar in a structural element, the concrete weight was assumed to be 2400 kg/m$^3$. This would include the weight of any reinforcing bars that would be used in construction of the shallow foundations. Section 8.5.1 ACI 318-02 (2002) defines the modulus of elasticity for concrete in Equation 4.1. Equation 4.1 is acceptable for values of $w_c$ between 1400 and 2500 kg/m$^3$.

$$E_c = 0.043 w_c^{1.5} \cdot \sqrt{f_c}$$

(4.1)

where, \( w_c \) = Weight of concrete (kg/m$^3$)

\( f_c \) = Compressive strength of concrete (N/mm$^2$)

The shallow foundations in this analysis were modeled by means of the plate element provided by Plaxis. The plate element is suitable for modeling structural elements (Plaxis, 2002). For elastic behavior, an axial stiffness, $E_A$, and flexural rigidity, $E_I$, were specified as material properties. According to Plaxis (2002), for an axisymmetric model,
the values of $EA$ and $EI$ relate to a stiffness per unit width in the out-of-plane direction (i.e. the longitudinal direction). The properties for all footings are listed in Table 5-1.

$DF = \text{Depth from ground surface to bottom of footing}$

$B = \text{Square footing dimension}$

$T = \text{Thickness of footing}$

**Table 5-1. Footing Properties**

<table>
<thead>
<tr>
<th>Footing Size (m)</th>
<th>Ec (kPa)</th>
<th>A (m$^2$)</th>
<th>I (m$^4$)</th>
<th>EA (kN/m)</th>
<th>EI ((kN*m$^2$)/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>2.29E+07</td>
<td>2.25</td>
<td>0.01195</td>
<td>5.15E+07</td>
<td>2.74E+05</td>
</tr>
<tr>
<td>3</td>
<td>2.29E+07</td>
<td>9</td>
<td>0.0239</td>
<td>2.06E+08</td>
<td>5.47E+05</td>
</tr>
<tr>
<td>4.5</td>
<td>2.29E+07</td>
<td>20.25</td>
<td>0.0358</td>
<td>4.64E+08</td>
<td>8.20E+05</td>
</tr>
</tbody>
</table>
5.1.2 FOOTING LOCATIONS

The footings were placed at a specified location from the blast detonation point. Previous research performed for the U.S Bureau of Mines, which linked distance from blast to expected structure damage, was used as a guideline in selecting the structure location. The footing location was chosen where major to severe cracking of the structure could be expected. Locations where severe cracking of the structure could be expected were analyzed for long-term soil strength effects. It was concluded that if no evidence of soil strength loss existed for the most severe case, it would not be expected for cases in which the structure was significantly farther away from the blast detonation point. The depths of the footings were also varied during the analysis. The depth of burial of the footings $D_F$, were $B/2$, $B$, and $2B$.

5.1.3 SOIL PROPERTIES

Soil properties were chosen to represent both sand and clay. Two different groundwater table locations were also analyzed. For the dry condition, the groundwater table was set at a considerable depth, at the bottom of the model geometry. For the fully saturated condition, the groundwater table was placed at the top of the model geometry. Isotropic and homogeneous soil behavior was assumed for both soil types. Table 5-2 lists the soil properties used in the analysis.
5.2 BLAST PROPERTIES

All of the blast properties given in Table 4-1, which were provided by Yang (1997) and used in the model calibration, were used throughout the remainder of the analysis.

Therefore, only one charge weight was used. However, the blast depth was varied during the analysis. This enabled relationships to be developed based on blast depth. The blast depth varied from the ground surface, to the footing depth B, and to twice the footing depth 2B. Blasts occurring at the ground surface were not at optimum depth of burial.

This means that a significant amount of the blast energy is released into the atmosphere, not the soil body. Again, for calculation of the pressure-time histories, soil parameters given in TM 5-855-1 (1986) were used.
5.3 STATIC ANALYSIS OF THE FOUNDATION SYSTEM

A static analysis was performed to ascertain the ground deformations and footing movements due to a static column load. These deformations were then used as a reference for comparison with long-term deformations. Allowable column loads that could be supported by the shallow foundations were also determined during the static analysis. The foundation system was analyzed as a concentrically loaded footing. Column loads representing probable design loads were required in order to simulate field conditions. Load-deflection curves were produced for each footing and for each soil condition. These probable column loads were determined by finding the load which gave a settlement of 25 mm for a given shallow foundation. An allowable settlement of one inch is typical design settlement criterion for shallow foundations. Figures 5-3 to 5-6 show the load-deflection curves for the different soil conditions. The footings were loaded under undrained conditions. A blast detonation is a rapid loading event that is characteristic of undrained conditions; therefore, the static and dynamic analyses were performed under undrained conditions. From the static analysis, a critical load for each footing was determined. In many cases, due to the critical drainage conditions, the supporting soil failed in shear before 25 mm of settlement occurred. Therefore, 25 mm of deflection or soil failure was used as criteria to determine the critical column loads.
Figure 5-3: Dry Sand Load-Deflection Curves
Figure 5-4: Wet Sand Load-Deflection Curves
Figure 5-5: Dry Clay Load-Deflection Curves
Figure 5-6: Wet Clay Load-Deflection Curves
The column loads were then determined from the load-deflection curves. A column load of 50% critical load was chosen for use in the analyses. A column load of 50% critical load will provide a load that is within the linear region of the load-deformation curves. This is important because it stays consistent with the Mohr-Coulomb soil model, which also assumes linear-elastic behavior. These loads are also less than the allowable soil strength and thus the soil will not fail in shear during the static analysis.

5.4 DYNAMIC DISPLACEMENT RESPONSE OF FOOTING

Using the results obtained from the static analysis, the dynamic analysis of the blast detonation was then completed. The footings were placed at a distance from the detonation point where major to severe damage could be expected. The blast and footing depths were then varied as previously described in Sections 4.1.2 and 4.2. The footings were loaded with 50% of the maximum column load to allow an initial stress state to develop. After the initial conditions were established, the blast was detonated. Figures 5-7 to 5-9 show the results of the analysis. Results are presented for a point on the footing nearest to the blast detonation. This point was assumed to be the most critical point on the footing because it was closest to the blast detonation. Displacements in sand resulting from a blast detonation are generally larger than those in clays (TM 5-855-1, 1986). Therefore, peak displacements resulting from the blast detonation for the wet sand condition are presented in Figures 5-7 to 5-9. Results from the dry sand and wet and dry clay conditions produced curves very similar to those of the wet sand.
Figure 5-7: Normalized Peak Displacements in Saturated Sand (Blast @ Surface)
Figure 5-8: Normalized Peak Displacements in Saturated Sand (Blast @ B)
In Figures 5-7 to 5-9, peak displacements and footing size have been normalized to the footing depth. This allows for an easier comparison of data. It can be seen that when the blast is at the ground surface, the energy released upon detonation contacts the footing from above. This results in a downward deflection of the footing. When the blast is located at a depth of 2B, the blast energy contacts the footing from below and results in an upward deflection of the footing. For cases when the blast depth is at B, the footing response depends upon its location relative to the blast (i.e. above or below the blast). However, for each case, it can be seen that the 1.5 m footing experiences the most...
deflection and the 4.5 m footing experiences the least deflection. This trend shows that for the same charge weight, larger footing sizes are capable of withstanding more blast energy, with less disturbance than smaller footing sizes.

In Figure 5-9, it can be seen that there is a noticeable break in slope in the normalized displacement curves. For each case, this point occurs at ratios of Footing Size/Footing Depth equal to one. The break in slope indicates a critical point. In Figure 5-9, the slope of the curves for ratios of Footing Size/Footing Depth between 0.5 and 1.0 has a steeper slope than Footing Size/Footing Depth ratios between 1.0 and 2.0. For values between 0.5 and 1.0, the footing is closer to the blast location and absorbs more of the energy dissipated from the blast, resulting in a steeper slope of the curve. For values between 1.0 and 2.0, the footing gradually moves further away from the blast and absorbs less energy.

Another way in which the effects of the blast event to the footings can be seen, is to view the surrounding soil deformations. This can be done by looking at the deformed finite element mesh before and after the blast loading. Figure 5-10 and 5-11 present the state of the finite element mesh for the loading sequences for sand and clay, respectively. It should be noted that the finite element mesh deformations have been scaled up 200 times. It can be seen how the surrounding soil matrix has changed as a result of the blast loading.
Figure 5-10: Deformed FE mesh used for sand a) undeformed, b) after static load, c) after blast load
Figure 5-11: Deformed FE mesh used for clay a) undeformed, b) after static load, c) after blast load
The immediate response to the blast load was observed by means of displacements of the footing and of the soil matrix. However, the long-term ground response to the blast load is assumed to also be critical to the stability of the foundation system. The ground response was analyzed by means of excess pore water pressures that developed during the blast detonation. High excess pore water pressures could lead to instabilities of the surrounding soil matrix, as well as immediate and long-term reductions in shear strength. These excess pore water pressure changes could also result in soil liquefaction. The soil liquefaction potential was determined through the use of the pore pressure ratio (PPR), as defined in Section 2.2.3.2. The pseudo-static response was analyzed using residual excess pore water pressures. The residual excess pore water pressures take a much longer time to dissipate and may in some cases result in a permanent change in the insitu stress state. It is this permanent change in the stress state that is a great cause of concern to engineers. Permanent increases in pore pressure result in a permanent decrease in effective stress and therefore a permanent decrease in soil strength. This can possibly cause significant problems to shallow foundations after a blast event. As before, results were obtained for ground response of the footing for a point nearest to the blast detonation.
Loose, saturated sand is the most critical soil condition for liquefaction to occur. Figures 5-12 to 5-14 plot pore pressure ratio against normalized footing depth for wet sand for different blast depths for the pseudo-static response. According to Veyera et al. (2002), a pore pressure ratio greater than one indicates that liquefaction has occurred. It can be seen that many of the pore pressure ratio values are greater than one. Some of the values are now less than one, but are still within ranges where the soil is still susceptible to liquefaction, per criteria given in Section 2.2.3.2. In cases where the values are greater than one, the soil remained in a liquefied state after the passing of the transient event. Again, changes in slope of the curves are evident, as was before with the normalized displacement curves. The change in slope occurs at the point where the normalized footing depth is equal to one, and represents a critical point. In every case, the highest values of pore pressure ratio occur for the closest distance of the footing to the blast location, and the lowest values occur for the farthest distance of the footing to the blast location.
Figure 5-12: Liquefaction Potential for Residual Response (Blast @ Surface)
Figure 5-13: Liquefaction Potential for Residual Response (Blast @ B)
The results obtained from the residual excess pore water pressures are the most important aspect of the analysis. This shows that regardless of damage to the structure itself, the supporting soil is still susceptible to liquefaction or has liquefied in some cases. This can lead to possible future damage to the substructure as well as to the supported superstructure.
A blast event is characteristic of undrained loading conditions because it is a very quick and rapid loading event. Figures 5-15 and 5-16 present the changes in undrained shear strength for saturated sand and clay, respectively, as a result of the blast detonation. The top curve represents the shear strength of the soil before the blast occurred. The bottom curve represents the shear strength of the soil immediately after the blast. The soil shear strength was calculated using the provided equation. It can be seen that the undrained shear strength of the soils have decreased as a result of the blast loading.

$$S = \frac{\sin \phi' [k_o + A_F (1 - k_o)]t'}{1 + (2A_F - 1)\sin \phi'}$$
The long-term changes in soil shear strength can be analyzed by studying how the final stress state has changed the drained shear strength of the soil. Drained conditions are characteristic of long-term changes to the soil matrix. Figures 5-17 and 5-18 present the long-term changes in drained shear strength for saturated sand and clay, respectively. The top curve represents the shear strength of the soil before the blast occurred. The bottom curve represents the shear strength of the soil a long time after the blast. The soil shear strength was calculated using the provided equation. It is observed that there is a long-

\[
S = \frac{\sin\phi [k_n + A_f (1 - k_n)] \sigma'}{1 + (2A_f - 1) \sin\phi'}
\]
term loss in soil shear strength for both soil types. This long-term loss in soil shear strength could lead to a possible failure of the foundation system.

$S = \sigma \tan \phi$

Figure 5-17: Changes in drained shear strength of saturated sand
Figure 5-18: Changes in drained shear strength of saturated clay

\[ S = \sigma \tan \phi \]
5.6 LONG-TERM SOIL STRENGTH CHANGES

Based on the results obtained from the excess pore water pressures, it is evident that the surrounding soil matrix has changed drastically as a result of the blast detonation. In many cases the soil is still susceptible to liquefaction or has liquefied. Because the soil is still prone to liquefaction, a loss of soil shear strength is evident.

By drawing a stress path for the blast loading, the shear strength of the soil can be analyzed. Stress paths were constructed for two conditions; a static condition before the blast detonation and a residual condition after the blast detonation. Stress paths were constructed for total and effective stress states for saturated clay and saturated sand. Figures 5-19 and 5-20 present the changes in shear strength of the sand and clay, respectively. The $k_o$ line, which represents a constant ratio of stress, is also plotted on the figures. The stress paths presented in Figures 5-19 and 5-20 are representative of compression loading, and is a typical loading condition for shallow foundations. During undrained loading, the total stress path is not equal to the effective stress path because of the development of excess pore water pressures. For compression loading, a positive excess pore water pressure develops and the effective stress path lies to the left of the total stress path. These relationships can be seen in the figures. It can be seen that from before the blast detonation to after the blast detonation, there is a noticeable loss of shear strength for both soil types. This clearly shows that long-term losses in shear strength occur after a blast detonation. This then becomes the critical period of time in which
shallow foundations are susceptible to failure. The allowable soil strength has been greatly decreased and thus the soil may no longer be able to support the necessary loads of the superstructure. This could result in a local bearing capacity failure of the shallow foundation and could seriously weaken the superstructure.

Figure 5-19: Long-Term Effects of Shear Strength for Sand
Figure 5-20: Long-Term Effects of Shear Strength for Clay
CHAPTER 6

TECHNICAL ANALYSIS

6.1 DAMAGE THRESHOLD

Damage to buried structures can be broken down into two categories. One in which damage to the structure is immediately noticeable and is a direct result of the dynamic event of the ground shock waves slamming into the buried structure. The other in which the blast waves have passed by the structure and damage may occur as a result of the pseudo-static event. The pseudo-static event is defined as the long-term or permanent change to the surrounding soil matrix, pore pressures and soil body geometry resulting from the dynamic event of the explosive detonation. As stated before, these long-term effects can lead to excessive settlements, shear failure and buckling of the buried structure.

A preliminary damage threshold was established based on the work by the U. S. Bureau of Mines. It is used to help the researchers to initially estimate scaled distances in which the pseudo-static event will control the damage. Based on previous research by the U. S. Bureau of Mines involving damage to structures resulting from blasting, the preliminary damage threshold can be established.
6.1.1 PEAK PARTICLE VELOCITY

After studying the existing vibration standards and criteria to prevent damage to structures resulting from surface or shallow blasting depths, preliminary design charts were constructed. According to Dowding (1996), several square-root attenuation relationships are employed in the United States for blasting. One of which is the bounds of experience developed by Hendron and Oriard (1972). For preliminary estimates in blasting projects, the upper bound curve of Hendron and Oriard (1972) should be used (Dowding, 1996). This will give blasters a general idea of a scaled distance factor to use in order to prevent ground vibrations from exceeding a certain specified value.

Siskind et al. (1980) presents information regarding existing standards to prevent damage. It is reported that low-velocity materials will have higher ground strains and therefore potentials for failure, for a given particle velocity. Data from research performed by Siskind et al. (1980) is presented in Figures 6-1 to 6-3. Different levels of damage which are presented in Siskind et al. (1980) are included in Figures 6-1 to 6-3. The figures represent damage criteria for different ranges of seismic velocity from 1,000 to 1,500 m/s, 2,000 to 3,000 m/s, and 4,500 to 6,000 m/s. These seismic velocities correspond to different soil types. It can be noted that as the seismic velocity increases, so does the allowable particle velocity that will preclude damage.
Sand, Gravel, Clay Below Water Level
$c = 1,000$ to $1,500$ m/s

**Figure 6-1: Damage Criteria for Sand, Gravel and Clay**

- **SEVERE CRACKS**
  - $PPV = 2.4$
- **CRACK FORMATION**
  - $PPV = 1.6$
- **FINE CRACKS / FALLING PLASTER**
  - $PPV = 1.2$
- **NO NOTICEABLE CRACK FORMATION**
  - $PPV = 0.71$

**NOTES:**
- $SD = (RW^2)$
  - Assuming a shallow buried blast.
- Bounds of Experience From Hendron and Oriard (1972).
- PPV and Cracking Data From Siskind et al. (1980).

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**Assuming a shallow buried blast.**

**Oriard Upper Bound**

**Oriard Lower Bound**

**Average Bound**
Moraine, Slate, or Soft Limestone

$c = 2,000$ to $3,000$ m/s

**Figure 6-2: Damage Criteria for Moraine, Slate or Soft Limestone**

NOTES:
- $SD = (R/W)^{0.5}$
- Assuming a shallow buried blast.
- Bounds of Experience From Hendron and Oriard (1972).
- PPV and Cracking Data From Siskind et al. (1980).
Granite, Hard Limestone, or Diabase

c = 4,500 to 6,000 m/s

Figure 6-3: Damage Criteria for Granite, Hard Limestone or Diabase

Dowding (1996) presented information from research performed by Edwards and Northwood (1960) and Northwood et al. (1963) of observed response for old plaster-and-lath-walled homes. These levels of observed damage are included in Figure 6-4. Note the higher levels of allowable PPV to cause major damage to structures.
The U.S. Bureau of Mines set a safe level of ground vibration of 2 in/sec. This is the threshold peak particle velocity (PPV) value that prevents cosmetic cracking of above-ground structures subjected to buried blasts and governs most blasting operations in the United States. The current research did not further investigate ground vibrations that cause cosmetic cracking. The current research typically involved ground vibrations that cause serious weakening of the structure, and shifting or settlement of foundations. This
level of damage is classified as major or severe damage, and as can be seen from previous criteria, will occur at PPV values higher than 2 in/sec.

The current research investigated blasts occurring in sands and clays. Figure 6-5 presents PPV versus cubed-root scaled distance for various soils, as given by TM 5-855-1 (1986). Also plotted on the figure are the damage criteria given by Siskind et al. (1980) and Dowding (1996). Figure 6-5 can be used to estimate PPV values for different soil types. These PPV values can then be correlated to damage that can be expected to occur to a structure. Thus, PPV and possible structure damage can be estimated from Figure 6-5.
6.1.2 BLAST-INDUCED LIQUEFACTION

Figure 6-6 presents pore pressure ratio (PPR) versus inverse cubed-root scaled distance for different studies involving liquefaction potential of soils. Data from the current
research is plotted along with data from studies presented by Shim (1996). It can be seen that the current research matches well to predicted PPR values and results presented by Shim (1996). It can be said that the PPR is a useful tool for estimating blast-induced liquefaction potential.

Figure 6-6: Pore Pressure Ratio Relations
CHAPTER 7

CONCLUSIONS

7.1 CONCLUDING REMARKS

After completion of the study, there are several important conclusions that can be made regarding foundation element response to blast loading. It was determined that Plaxis software is capable of modeling blast events and provides a useful tool for blast prediction purposes.

- Two case histories were modeled and yielded acceptable results
- Plaxis results compared well to results provided by the case histories and to empirical relationships provided by TM 5-855-1 (1986)

After analyzing the data obtained during the study it is evident that blast loadings are capable of producing residual excess pore water pressures that result in blast-induced liquefaction.

- Blast-induced liquefaction potential was shown through the use of the pore pressure ratio (PPR)
- In most cases, the soil surrounding the shallow foundation elements remained in a liquefied state (PPR > 1.0), or was susceptible to liquefaction (0.5 < PPR < 1.0) after the dynamic event of the blast detonation had passed.
The soil surrounding a shallow foundation element is at risk for potential liquefaction for long-term or pseudo-static conditions resulting from blast loading.

It also became evident that a significant loss of soil shear strength resulted from the blast event.

- Undrained shear strength decreased by an average of about 6,000 Pa
- Drained shear strength decreased by an average of about 10,000 Pa
- This long-term loss of soil shear strength shows that the initial stress state of the soil matrix has changed drastically as a result of the blast event
- The pseudo-static condition, which exists for a significant amount of time after the blast detonation, should be of great concern to engineers
- Long-term failures of the foundation element could possibly occur due to the pseudo-static blast event

It was also observed that residual excess pore water pressures remain as a result of a blast event.

- Residual excess pore water pressures could lead to possible failure of a foundation element
7.2 RECOMMENDATIONS FOR FUTURE RESEARCH

It has been concluded that the pseudo-static event that results from a blast detonation is the most critical condition for shallow foundation elements. The pseudo-static event is a great cause of concern for engineers and needs to be investigated further. Actual field testing and centrifuge modeling should be completed to investigate this problem further.

- Perform tests using a range of charge weights
- Use different ground water table locations

Another finite element model should be developed and analyzed that utilizes an advanced soil model. The Mohr-Coulomb soil model that was used for this research is a linear-elastic, perfectly-plastic soil model.

- Because blast events are not a truly linear-elastic phenomenon, a plastic soil model could possible yield more accurate results.

It is also evident that excess pore water pressures have a direct impact on the shear strength of soil.

- Research should be performed to relate the pore pressure ratio (PPR) directly to shear strength of soil
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APPENDIX
Figure 1: Normalized Peak Displacements in Dry Sand (Blast @ Surface)
Figure 2: Normalized Peak Displacements in Dry Sand (Blast @ B)
Figure 3: Normalized Peak Displacements in Dry Sand (Blast @ 2B)
Figure 4: Normalized Peak Displacements in Saturated Clay (Blast @ Surface)
Figure 5: Normalized Peak Displacements in Saturated Clay (Blast @ B)
Figure 6: Normalized Peak Displacements in Saturated Clay (Blast @ 2B)
Figure 7: Normalized Peak Displacements in Dry Clay (Blast @ Surface)
Figure 8: Normalized Peak Displacements in Dry Clay (Blast @ B)
Figure 9: Normalized Peak Displacements in Dry Clay (Blast @ 2B)