Measurement and modelling of ground response due to dynamic loading

Anand, S

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Measurement and Modelling of Ground Response due to Dynamic Loading

S Anand

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SUMMARY

Shock wave propagation through soil due to a buried explosive loading is a complex function of the dynamic soil properties, explosion products, and geometry of the explosion. The soil properties and composition of soil are a reflection of their geology and mechanical effects that operated on them in the past. A buried explosion in soil usually results in a single, prominent pulse in the time-history record that lasts a very short duration (typically 0.02 seconds), and is sometimes referred to as a transient dynamic loading. Assessment of shock wave propagation in soils is of great importance to engineers, as they dictate the magnitude of load that will impinge on structures for a given source, and thus determine the basis for structural design.

This study has several primary objectives: (1) to investigate blast wave propagation and attenuation characteristics in Singapore soils, (2) to develop a reliable numerical method to simulate buried explosions in soil, and (3) to derive suitable relations for the prediction of free-field and reflected ground shock parameters in Singapore soils. To serve these three objectives, field explosion tests, numerical simulation of underground explosions, and data analyses were carried out. Field explosion tests are challenging to conduct and are multi-faceted in nature. Test sites selected for investigation have to be reasonably representative of the geological formation being studied. The test set-up, type and shape of explosive have to be considered for meaningful data to be obtained. In addition, a rigorous instrumentation system has to be designed to enable the transient data from the explosion to be captured. Data acquisition is particularly tricky in the near field, i.e. at distances close to the explosion where the transducers and the cabling system have to withstand the harsh environment. Scaled-model field explosion tests undertaken in Singapore geology are described.

Results from two series of field explosion tests conducted in 1996 and 2002 are presented in this thesis. Site characterization works were also undertaken as part of the field explosion tests to determine the dynamic properties of the in-situ soil. Field tests were undertaken in various soil types including sands, clays, and residual soils. A

variety of buried targets were included used in the field explosion tests to investigate the amplification effects on stress waves caused by these buried targets. The buried targets ranged from 1 m reinforced concrete cube targets to smaller devices such as the In Situ Dynamic Measurement Unit (or IDMU). The IDMU was modified and enhanced under this research to make it a more versatile measurement device for use in field explosion tests.

Results from the site characterization works and the field explosion tests were used to calibrate a numerical model. The 2-D and 3-D numerical modelling were undertaken using LS-DYNA, a commercial finite element program. It was found that an elasto-plastic soil model was able to simulate the behaviour of soil subject to explosion loading adequately. Input data for the soil model was obtained from site characterization works. An anisotropic brittle damage model was used to define the concrete buried target and solid finite elements were used to define all the other targets. The explosion was simulated using the Jones-Wilkins-Lee (JWL) explosion model in LS-DYNA. Relationships for predicting free-field and reflected response of Singapore soils were developed using field test results, numerical results and published literature.

Given the large number of factors that affect shock wave propagation in soils, the US Army Corps of Engineers (USACE) developed TM5-855-1 (1986) which provides quasi-empirical equations for computing ground response as well as design guidelines for underground structures subjected to buried explosion loading. In order to compare data from various explosion tests, an energy scaling method is commonly used. Two of the most popular approaches are the cube-root ($R/E^{1/3}$) scaling and the square-root ($R/E^{1/2}$) scaling methods, where R is the horizontal distance from the source, and E is the equivalent TNT explosive energy. TM5-855-1 (1986) suggests a cube-root scaling method for buried explosions. These equations are an oversimplification of the true ground and buried structure responses but are widely used as the computations are simple and the results are deemed adequate for preliminary analysis. The scaling method used and the predictive equations are further investigated in this study to see if

they are applicable in local geology and if the form of equation can be improved for easier application.

The predictive accuracy of the TM5-855-1 (1986) equations was found to be good if the soil density, ρ , seismic velocity, c , and the attenuation coefficient, n , were determined directly from field tests. Improvement in predictive accuracy of the TM5-855-1 (1986) equations was found to be better in partially saturated soils than in wet soils as the attenuation coefficient and seismic velocity in wet soils did not vary significantly. However, the accuracy of the predicted values was only about $\pm 35\%$ of the field test values. Sensitivity of the TM5-855-1 (1986) equations to the attenuation coefficient appears to be the main cause for the poor predictive accuracy of the TM5-855-1 (1986) equations. Field explosion test results and finite element analyses were considered with a view of establishing a better form of expression for the attenuation coefficient and to improve the predictive accuracy for free-field stresses and motions.

The analyses suggest that the attenuation of stress waves in soils can be alternatively expressed as a function of Poisson's ratio of soil. With this finding, the equations for free-field stresses and motions were re-written using dimensionless parameters and Poisson's ratio. Equations proposed in this thesis are dimensionless and avoid problems relating to units and conversions, which are often encountered when using TM5-855-1 (1986) equations. It was also found from the analyses that the free-field stresses and motions were affected by material damping. Charts were thus developed, as a function Poisson's ratio and material damping. The proposed equations based on the Poisson's ratio and damping ratio provided a more objective estimate of the free-field stresses and motions.

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LIST OF SYMBOLS

Generally, symbols will be defined where they occur in the report. A list of commonly used symbols are given below:

ν	Poisson's ratio
V_r	Phase velocity
V_s	S-wave velocity
G	Shear modulus
G_{max}	Low-strain shear modulus
c_u	Undrained shear strength
λ	Rayleigh wavelength
Φ	Phase difference
f	Frequency
z	Depth
R	Horizontal distance (explosion analyses)
W	TNT equivalent charge weight
ρ	Mass density of the soil
P_{ff}	Incident free-field pressure
P_r	Reflected pressure
P_i	Measured pressure from sensor
P_o	Peak pressure in soil
V_o	Peak particle velocity
c_c	Average propagation velocity of ground shock
u_g	Velocity of soil particles
u_k	Velocity of the sensor
e	Void ratio
σ_0	Average effective confining pressure
σ_l	Effective vertical stress
ϕ'	Drained friction angle
OCR	Overconsolidation ratio.

CHAPTER 1

INTRODUCTION

1.1 Background

Predicting ground vibrations induced by blasting is becoming increasingly important. Ground vibrations resulting from blasting are a serious problem in that they are a cause of annoyance, complaints and possible structural damage due to their detrimental effects. In a densely populated and urbanised country like Singapore, the effects of ground vibrations due to buried explosions are particularly important for the survivability of infrastructure and people. Many ground shock studies have been undertaken by other countries, with charge quantity and distance being the most dominant concern of the investigations (Taqieddin 1982). There have been very few carefully controlled investigations in ground shock that specifically determine the way in which each design factor has an influence on ground shock propagation in Singapore soils.

An important reason for increased interest in ground vibrations is rapid advancements in technology including high-speed computers. Analytical and construction techniques facilitate the design and construction of more efficient structures. Extensive research has been performed to improve identification and characterization of soils using a variety of field and laboratory investigations. However, there is still a considerable gap between the state-of-the-art and state-of-practice for ground shock prediction and analysis.

Singapore has been largely shaped by its tropical climate. About two-thirds of the land area is covered by residual soils, the products of intensive in-situ mechanical and chemical weathering. The groundwater table in these deposits is usually very deep

which means that the surfacial soils are usually unsaturated. The remaining land area of Singapore is covered largely by alluvial soils and marine clay. Although Singapore occupies a small land area of about 707 km², the variety of soils encountered over short distances poses great challenge to the design of structures subjected to dynamic loadings. Dynamic loadings in soils may arise from vibratory sources (such as reciprocating machines), detonation of explosives or earthquake effects. There is concern in Singapore about how structures and soils will react when subjected to near surface explosions in local geology.

Dynamic loadings on or within the ground surface cause stress waves to be generated in the soil. Stress wave propagation in soils is generally governed by the soil properties and the magnitude of the energy source. For a vibratory source applied at the ground surface, almost two-thirds of the applied energy will dissipate in the form of surface waves (Henrych 1979). Rayleigh wave is the most persistent stress wave that develops in the ground. Rayleigh waves carry important information about the ground through which they propagate. Early research on Rayleigh waves focused mainly on transforming the information carried by these waves into a form that is useful for engineering applications (Thomson 1950).

Stress waves due to explosions are more complex due to the composition of the explosive, the rupture mechanism and coupling effects at the depth of detonation. Explosions are usually accompanied by a violent release of energy and the shock waves are usually characterised as high frequency and short duration loadings. The behaviour of soils subject to such loadings is complex, especially in the near field. Soil inhomogeneity and degree of saturation can introduce further complexities as the transient waves reflect and refract off the ground surface and other subsurface layer boundaries. In practice, it would not be possible to account for all these effects in the soil media. As such, an empirical approach to the measurement and subsequent idealisation of the explosive source is usually adopted in practice (TM5-855-1 1986). Earthquakes can inflict substantial damage when they occur. However, they are generally lower frequency energy sources and last much longer durations (Pan 1995)

compared to explosive sources. For this research, the focus is on characterisation of wave propagation in local soils for the assessment of blast loading on buried structures built for civilian or military purposes.

Typically, hardened blast shelters are constructed of cast-in-place or precast reinforced concrete. The most common configurations used in personnel and equipment shelter construction is the flat-roof box and the arch-shape box. The flat-roof box type shelter can be constructed as a simple single bay or consist of multiple bays and multiple stories depending on the space requirements. These structures are usually built under a shallow soil cover. Any attempt to model such buried structures should thus be undertaken using hollow box structures for realism.

Blast loads on buried structures can result from detonation of a nuclear device or a conventional bomb containing high explosives. A nuclear detonation is characterized as a very rapid release of energy within a limited space that causes a large increase in pressure and temperature. Materials near the explosion are converted to hot gases that expand rapidly and initiate a shock wave in the surrounding media. About fifty percent of the energy released during a nuclear detonation is in the form of blast and shock. The remaining fifty percent of the explosion energy is released in the form of thermal and nuclear radiation. Conventional detonations are similar to nuclear detonations, except the energy density of the weapon is lower and no nuclear radiation is released.

Burying a structure below the ground also takes advantage of soil-structure interaction effects to reduce the structural loads on the buried structure. The design and analysis of shallow-buried, flat-roof shelters are very complicated. The main reason for this complication is the uncertainty in loading. The loading resulting from aboveground detonations (air bursts) is primarily due to airblast-induced ground shock (Figure 1-1). For subsurface detonations, the loading is a result of airblast-induced ground shock and direct-induced ground shock (Figure 1-2). Typically, the structural design for buried structures is based on the airblast-induced component in isolation, rather than the coupled loading components. For a more accurate analysis, geological data such as the

depth to bedrock, the depth of the ground water table, and soil stratification must be known for the evaluation of the direct-induced ground shock and airblast-induced ground shock effects. Adverse site conditions (shallow depth to bedrock or high ground water table) can greatly affect ground response and in-structure shock. Unless comprehensive geological information is known, the structure is generally assumed to be embedded in a homogeneous half-space. The effects of nuclear detonations will not be addressed in this research.

Figure 1-1. Airblast induced ground shock problem

Figure 1-2. Direct induced ground shock problem

Wave propagation in soils can be studied using simplified plane wave equations. However, such analyses have limited applications as they are an oversimplification of the complex three-dimensional nature of stress wave propagation. Another important aspect of stress wave propagation is its interaction with buried structures. Past research has shown that embedded structures can lead to wave reflections, thereby increasing the magnitude of free-field quantities (Dongen and Weerheijm 1992). It is thus important to undertake accurate measurements of shock wave propagation and understand the mechanisms involved in soil-structure interaction. A good understanding of soil-structure interaction is also a prerequisite for the development of a damage index for buried structures. It has been a normal practice particularly in rock engineering to use peak particle velocity as a damage index (Dowding 1996). However, current understanding is that peak particle velocity is not a good measure of damage (Woods and Jedelee 1985). A complete description of wave propagation through soils can be achieved if the time history of the wave can be measured in three orthogonal directions. Experimental data in this respect for ground motions from blast loading are lacking. There are also very limited data on the dynamic properties of Singapore soils, which are crucial for ground response studies.

Seismic tests can be used to determine the dynamic properties of soils and rocks. From a practical viewpoint, seismic tests enable scientists and engineers to peek into the ground and gather information without the need to extract samples from the ground for test. Seismic tests are based on elastic wave propagation theory and have been successfully used for many engineering applications. A widespread application of seismic tests is in the petroleum industry where surface waves are used for oil prospecting (Bedford 1994). In civil engineering, seismic tests have been used to target and map sub-surface features (Butcher and Tam 1994). More recently, seismic tests have been used for small-strain shear modulus profiling of soil deposits (Clayton et al. 1995). The shear modulus is a key soil parameter that is routinely required for geotechnical design. This recent application of seismic tests holds great benefits for geotechnical engineers, as the small-strain shear modulus can now be determined at the

in-situ state of the soil, without incurring sampling disturbances as part of the sample recovery and testing method.

Installation of sensors is an important aspect in the conduct of field explosion tests. In this respect, the in-situ dynamic measurement unit (IDMU) has shown promising performance in past field explosive tests for the installation of sensors although it is only a uniaxial system (Leong et al. 1996). Since the wave propagation problem is typically three-dimensional, a triaxial measurement system will be required to study wave propagation and attenuation in soils. The IDMU can be improved to allow measurements to be undertaken in three orthogonal directions. Improvements to the installation, cabling and mechanical handling would make the IDMU more compact and sturdy for field applications. The dynamic properties of Singapore soils can be obtained through in-situ measurements using a surface wave technique, as well as through dynamic laboratory tests. The field explosion test results obtained can be combined with available soil data and other information to develop and calibrate a numerical model for dynamic problems using commercially available finite element (FE) programs like LS-DYNA, AUTODYN, etc. Further analyses can be undertaken based on the numerical model to study the effects of various parameters on shock wave propagation in soils. The field test results and numerical analyses can also offer an insight into the damage mechanisms that occur in buried structures subjected to dynamic loading.

1.2 Objectives and Scope

The main objectives of this project are to characterise the response of Singapore soils subjected to transient dynamic loadings, and to assess the effects of such loadings on buried structures.

Scope of the research includes the following:

- Using the continuous surface wave system (CSWS) for shear modulus profiling at various sites.

- Processing of CSWS test results and study various methods of inversion for surface wave data.
- Characterisation of the Singapore soils using data obtained from CSWS.
- Development of a triaxial measurement system for characterising ground motion due to dynamic loading.
- Study ground motion due to dynamic loading by conducting field explosion tests.
- Numerical modelling of field explosion tests.
- Parametric study of ground response due to dynamic loading.
- Examine and evaluate current design codes, namely TM5-855-1 (1986), for buried structures subjected to dynamic loading.

1.3 Need for Research

Past research have shown that the effects of explosions can be accurately predicted if numerical models were complemented with independently determined soil properties (Ghosh 1983). The soil properties, particularly the shear modulus, have to be determined over the full strain range that would be anticipated in the field. Soil stiffness at various strain levels can be measured using various approaches. The small-strain stiffness of soils is generally very difficult to measure due to the need for mechanisms that could apply such infinitesimal strains, typically in the range of 0.0001%. Specialised laboratory investigations or field seismic methods can be used to determine the small-strain stiffnesses in soils. The intermediate strain levels of soils can be determined using cyclic triaxial, cyclic simple shear or torsional shear apparatuses. The large strain levels can be obtained using conventional laboratory apparatuses.

The quantification of the dynamic properties of Singapore soils has been relatively piecemeal with fragmented data established under various research programmes. However, very little has been done to obtain the stiffness-strain relationships of the soil over a large range of strain level. The effect of the non-linearity of the stiffness-strain relationship on free-field wave propagation in soils and the behaviour of shallow-buried structures subjected to blast loading have not been investigated. Furthermore, the

applicability of TM5-855-1 (1986) design parameters for local soil conditions have not been fully verified.

1.4 Organisation of Thesis

The thesis is organised into seven chapters. Chapter 1 provides an introduction to the measurement and modelling of ground response when subjected to dynamic loading, the objectives and the scope of this research. Chapter 2 reviews elastic wave propagation in soils, Singapore geology, seismic tests, explosion tests and numerical modelling. Chapter 3 outlines the continuous surface wave system (CSWS), which has been used in several field tests to determine the steady-state response of Singapore soils. Chapter 4 describes field explosion tests undertaken in Singapore geology and highlights some results from the field tests. Chapter 5 discusses the numerical modeling procedure, and the numerical model that was used to model the field explosion tests. Chapter 6 discusses the field test and numerical results and presents some local relations for determining free-field and reflected parameters in Singapore soils. Chapter 7 presents the conclusion of the study and recommends future work that would be needed in this area of research.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter reviews the propagation of elastic waves in soils, Singapore geology, seismic tests, explosion tests and numerical modelling. The modes of wave propagation in soils and the nature of dynamic loading are first discussed. This is followed by a review of the Singapore geology and selected dynamic properties of Singapore soils. As properties of Singapore soils will be used in the numerical modelling, a large part of this chapter is devoted to dynamic behaviour and determination of dynamic properties. Field seismic tests are commonly used to determine dynamic properties of soils. An important consideration in most field seismic tests is the inversion of field dispersion curves. Inversion techniques enable the field seismic data to be interpreted and hence the seismic properties of the soil to be determined. With increasing computing power and versatility of numerical modelling programs, explosion modelling is increasingly undertaken using numerical models. The effects of the type and shape of explosive, the depth of embedment and other considerations are discussed in this chapter. The proposed methodology to be adopted for this research is presented at the end of this chapter.

2.2 Elastic Waves: Definition, Terminology and Characteristics

Ground motion induced by underground explosions involves two typical domains of mechanical response of the soil to energy released: hydrodynamic regime in the near field and solid wave regime at further distances from the explosion source (ISEE 2000). When the wave propagates in the solid wave regime, there are two types of waves, compressive P-wave and shear S-wave. When these body waves come to a free surface

or interface, a surface wave will be generated on the surface. The most prevalent surface wave is the Rayleigh wave.

Different wave types have different velocities. Body waves such as P and S waves propagate fastest. Figures 2-1(a) and (b) illustrate P and S waves, respectively. P-waves or primary waves have the greatest velocity. They propagate in the same direction as the direction of particle motion. Due to their compressive nature, P-waves can propagate through solids, liquids and gases. The distortional S-waves or secondary waves propagate in the direction where the direction of motion is perpendicular. As fluids have no capability to resist shear force, S-waves cannot propagate in fluids or gases.

Figure 2-1. Particle motion variation with wave type: (a) P-wave, (b) S-wave (SH), (c) Rayleigh wave (from Dowding 1996)

Published literature supports the existence of four types of surface waves: Rayleigh wave, Love wave, Scholte wave and Stonely wave (Thomson 1950). As discussed previously, only P and S waves exist in a uniform infinite medium. The surface waves are formed due to the introduction of one or more boundaries across which there are differences in elastic properties (e.g. in layered strata). The earth's surface is a significant boundary, from which most observations of waves are made. Rayleigh waves are the most important for studying ground dynamic response, and are named after Lord Rayleigh (1885), an English physicist. Rayleigh waves have retrograde and elliptical particle motion; similar to the waveforms caused when a stone is dropped into a pond. The vertical axes of the ellipses are about 1.5 times the horizontal and at the highest points of the ellipses, the particle motion is opposite, as shown in Figure 2-1(c). Rayleigh waves are believed to arise as a result of interfering longitudinal (P) waves and shear vertical (SV) waves (Bollinger 1970). Love waves are similar to Rayleigh waves but their particle motion acts in a horizontal plane. Scholte waves are generated at solid-water interfaces (e.g. seabed conditions) and Stonely waves propagate in a fluid filled cavity (Henrych 1979). Surface waves travel slower than S-waves (which are slower than P-waves) and so in earthquake studies, surface waves tend to arrive at the seismograph just after the S-waves and P-waves.

Surface waves have two main characteristics: The first is that the waves are 'guided' by the surface on which they propagate, and the second is that they decrease exponentially in particle motion with increasing distance from boundaries (and thus a surface phenomenon). The exponential decay pattern also makes it very difficult to separate and detect the surface waves individually within a short distance from a source. As distances increase, the waveforms are more clearly discernible.

Figure 2-2 illustrates the pulse fronts of the P, S and Rayleigh waves due to an oscillating footing on an elastic half space. The body waves propagate radially outward from the source along a hemispherical wave front. The wave magnitude will decrease with the distance. The decay law is usually $\frac{1}{r^n}$ ($n = 1, 2, 3$) where r is the radial distance

from the source and n is a dimension-dependent constant. This phenomenon is called geometrical damping. Material damping also exists if the viscosity of material is not ignored. Rayleigh waves propagate radially outward along a cylindrical wave front. All of the waves encounter an increasingly larger volume of soil as they travel outwards. Miller and Pursey (1955) calculated the distribution of total energy from a vertically oscillating point source on the surface of a homogeneous, isotropic and elastic half-space among the different wave types (Figure 2-2). The distribution is:

Compression (P) wave	=	7%
Shear (S) wave	=	26%
Rayleigh (R) wave	=	67%

About two-thirds of the total energy goes into Rayleigh wave. Also, Rayleigh waves decay much more slowly than the body waves, therefore at large distances from the source, Rayleigh waves will be predominant in the wave train. The persistence of the Rayleigh waves at large distances are frequently encountered in earthquakes, where the approaching Rayleigh waves are felt as a rolling or rocking motion and are sometimes called ‘ground-roll’ (EERC 2000).

Figure 2-2. Wave propagation from a circular footing in homogeneous, isotropic, elastic half-space (from Woods 1968)

Rayleigh waves originate on the surface at some distance from the source. Theoretically, a Rayleigh wave will develop at a distance from the point source at a depth, d , as shown in Figure 2-3. In Figure 2-3, V_r and V_p are the Rayleigh and compression wave velocity, respectively. A long cylindrical column of explosives can be idealized as an infinite number of point sources, and there will therefore be a zone where Rayleigh waves will develop.

Figure 2-3. Minimum distance for formation of Rayleigh wave (from Ewing et al. 1957)

2.2.1 Attenuation of amplitude in the elastic zone

The amplitude of the seismic waves decreases as the waves propagate. Two mechanisms are responsible for the decay of amplitude: geometric spreading and inelastic attenuation or damping. Both mechanisms should be considered when formulating a model for prediction of ground vibration.

As the waves propagate they encounter an increasing volume of the medium for both cylindrical and spherical sources. Even assuming that there is no other loss of energy, the energy density will decrease with increasing distance from the source. The decay of amplitude is called attenuation due to geometric spreading or geometric damping.

In an ideal elastic half-space, the attenuation of amplitude of different types of waves are summarized below (Richart et al. 1970):

Body waves propagating along the surface:	Amplitude $\propto r^{-2}$
Body waves propagating through the medium:	Amplitude $\propto r^{-1}$
Rayleigh waves:	Amplitude $\propto r^{-0.5}$

where r is the radial distance from the source.

Field and laboratory measurements show that the actual amplitude decay is much more than that predicted by geometric spreading alone (Ghosh and Daemen 1983). Inelastic attenuation is more frequently encountered in rocks than in soils (Wiss and Linehan 1978). Inelastic attenuation is a very complex phenomenon. Numerous mechanisms have been cited as possible causes for the inelastic attenuation (Ghosh 1983):

- matrix inelasticity, including dissipation due to relative motions at grain boundaries and across crack surfaces;
- attenuation due to fluid-flow, including relaxation because of shear motions at pore-fluid boundaries;
- dissipation in a fully saturated rock because of relative motion of the frame with respect to fluid inclusions;
- shearing flow of the fluid layer;
- partial saturation effects, such as gas-pocket squeezing;
- enhanced inter-crack flow;
- stress induced diffusion of absorbed volatiles;
- energy absorbed in a system undergoing phase changes;
- a large category of geometrical effects, including scattering off small pores and large irregularities and selective reflection from thin beds.

2.3 Nature of Dynamic Loading

2.3.1 Difference between static and dynamic loading conditions

The method of load application in soils can be classified generally into three types, namely, slow static loading, rapid transient loading, and repetitive loading as illustrated in Figure 2-4. The main factors that distinguish between static and dynamic problems are the speed of loading and effects of load repetition (Figure 2-5).

Figure 2-4. Types of loading (from Ishihara 1995)

Figure 2-5. Classification of dynamic problems (from Ishihara 1995)

Under static loading conditions, the load is applied monotonically with time, such as in a staged construction. Rapid transient loading involves the application of one single impulse such as dropping of bombs or blasting of explosives. Investigations by researchers (e.g. Tabatabai 1987) have consistently shown that the soil under transient loading conditions exhibits greater modulus and strength compared to static tests. Repetitive loading is usually of interest in vibration analysis, where the load is applied several times at some frequency (Ishihara 1995).

There are several sources of dynamic loading. Drop-weights used in dynamic compaction to improve the properties of underlying soil by dynamically transferring energy to the ground. An explosion, whether on the ground surface or buried, also creates dynamic loading. A surface explosion creates an air-blast effect, which is characterized by an almost instantaneous rise from the ambient pressure to a peak value. A buried explosion produces ground shock which is transmitted through the soil media.

Stresses from ground shock are usually greater in magnitude and last longer compared to a corresponding blast in air. The extent of damage caused to buried structures is dependent on the amount and type of explosive detonated and the properties of the transmission medium. Dynamic loading from explosion has been advantageously used for densifying loose saturated sands by creating a state of liquefaction, which subsequently results in the formation of a much denser soil matrix (Henrych 1979). Earthquakes and nuclear explosions are clear illustrations of massive dynamic loading that can cause widespread damage to both buried and surface infrastructure.

Some of the characteristics that distinguish different types of dynamic loading are as follow:

- Explosions transmit energy at higher frequencies compared to nuclear or earthquake sources and have very short periods. Most explosive sources are compact, having a small origination point (focus) and are usually detonated close to the earth's surface. As explosive pressures are developed very quickly, some of the energy is transmitted through the surrounding ground in the form of seismic strain energy. The actual strains encountered by soils during an explosion are known to be very small. However, it is this seismic strain energy that is propagated through the ground and is responsible for the ground vibrations that accompany an explosion (see Taqieddin 1982).
- Earthquakes are caused by the constant motion of the earth's surface. The earth's rock layer is broken into large pieces. From time to time, the pieces may lock together and the energy that accumulates between the pieces may be suddenly released. The energy that is released travels through Earth in the form of waves. Earthquakes take more time to complete the rupture process, and occur along long faults rather than a single point. They usually originate quite deep in the crust, and radiate shear or S-waves more in some directions than others (EERC 2000).

- Given the very short duration and accompanying small displacements, explosive sources carry negligible energy compared to nuclear or earthquake sources. The difference in energy source is even smaller in the case of construction activities (e.g. pile driving machines) or surface wave vibratory sources. Surface wave sources are hence considered to induce very small strains in the medium they propagate. Wiss (1980) illustrated that 0.5 kg of commercial dynamite can produce the same amount of energy as a 7-ton drop-weight used for dynamic compaction.
- The range of principal frequencies for earthquake and nuclear motions are greater, compared to explosive and vibratory sources (Dowding 1996). For instance, a single pulse from a drop-weight has a single dominant frequency and can be computed using wave theory. In buried explosions, the reflection of shock waves at soil interfaces can produce several peaks in a time-history record. In earthquakes, several peaks can occur at different frequencies due to the transmission of waves in soil medium. This can result in a range of principal frequencies and a response spectrum will generally be required to address such problems (EERC 2000).

For this research, the main interest is to measure and model ground response when subjected to conventional explosive sources (i.e. non-nuclear). Surface wave (Rayleigh wave) propagation due to vibratory sources is also important, as the surface wave technique can be used to determine the small-strain properties of the soil.

2.3.2 Shear and elastic moduli

The dynamic (or seismic) properties of Singapore soils are important prerequisites for studying the ground response subjected to dynamic loading. The most important seismic properties of soil that are routinely required in dynamic design are the shear wave (S-wave) velocity, V_s , and the compression wave (P-wave) velocity, V_p .

The S-wave velocity, V_s , can be used to determine the small-strain shear modulus (or G_{max}), if the bulk density, ρ , of the soil is known:

$$V_s = \sqrt{\frac{G_{\max}}{\rho}} \quad (2.1)$$

The maximum elastic modulus of soil, E_0 , can be determined from $G_{\max}$ and Poisson's ratio, ν , using the theory of elasticity:

$$E_0 = 2G_{\max} (1 + \nu) \quad (2.2)$$

The P-wave velocity, V_p , can be obtained if E_0 is known:

$$V_p = \sqrt{\frac{E_0}{\rho}} \quad (2.3)$$

These soil properties are essential for any numerical model that is adopted for ground dynamic response study. The dynamic properties of the soil can be obtained from laboratory tests or field tests. The small-strain shear modulus is a key parameter to be determined, but is difficult to measure in the laboratory. A better understanding of soil behaviour has mooted a shift towards seismic test methods for the determination of small-strain shear modulus of soils (Matthews et al. 1996).

Figure 2-6 shows some commonly used field seismic tests for determination of soil dynamic properties. For this research, the steady-state ground response of soil is obtained using a continuous surface wave system (CSWS), which is discussed in detail in Chapter 3.

Figure 2-6. Commonly used field seismic test methods
(modified from GDS Instruments Ltd 1998)

2.3.3 Stiffness-strain behaviour of soil

As mentioned above, researchers in the early 1950s were aware of the differences in elastic modulus and strength response when soil was subjected to a static or dynamic load. In addition, they were also intrigued by differences in shear modulus (or stiffness) values obtained from laboratory measurements and those obtained using seismic methods. Differences were also noted among laboratory tests, with increasing stiffness being observed from ‘dynamic’ laboratory configurations (e.g. resonant column test) and much lower stiffness from conventional triaxial tests. Results using cross-hole surveys sometimes also differed from those obtained using the surface wave techniques.

In some cases, the reported variation between field and laboratory determined shear modulus was 50% or more (Cunny and Fry 1973).

Anderson and Woods (1975) undertook several tests to compare values of V_s obtained using the resonant column test and cross-hole surveys. They noted that laboratory measured V_s in saturated clays showed a primary and a secondary response with time and the average effective confining pressure. For a given average effective confining pressure, the laboratory value of V_s depends on the length of time that the test has been running when V_s is measured. The comparison between laboratory and field V_s improved as laboratory test time increased, i.e. as more secondary increase in the shear wave velocity is included. Hence, when resonant column tests are used to predict V_s values, it is necessary to consider secondary behaviour. Based on the test results, Anderson and Woods (1975) suggested that a 20-year extrapolation of data obtained from laboratory tests could give fairly good estimates of V_s . Anderson and Woods (1975) also noted that cohesionless soils were less influenced by secondary time effects. These findings added difficulties in determining the 'correct' V_s value from laboratory tests.

In the early 1970s, Seed and Idriss (1970) provided a better understanding of soil behaviour. Seed and Idriss (1970) expressed the shear modulus for sands as a function of their relative density and shear strain. For saturated clays, they plotted G/c_u (where G is the shear modulus and c_u is shear strength) against shear strain as shown in Figure 2-7. They also suggested that sands and clays show a strain-dependent behaviour. When small-strain ($\approx 0.01\%$) measurements were undertaken in the triaxial test using local strain transducers inside the triaxial cell, the readings showed that the small-strain shear modulus was much greater than the large-strain shear modulus obtained from conventional measurements (Jardine et al. 1986). The higher G values from resonant column tests could also be reconciled as the apparatus applied very small strains on the samples. Researchers then realized that the 'dynamic' and 'static' shear moduli are both soil responses to very small strain and large strain conditions, respectively. Most soils behave elastically at very small strains (i.e. $<0.001\%$), giving rise to a constant stiffness

as shown in Figure 2-8. At large strains ($>0.1\%$), plastic behaviour dominates and the stiffness approaches a minimum value, G_{min} . Therefore G_{max} is a fundamental parameter in defining the stiffness-strain curve of Figure 2-8.

Figure 2-7. In-situ shear modulus for saturated clays (from Seed and Idriss 1970)

2.3.4 Drawbacks of laboratory testing for determination of G_{max}

Figure 2-8 illustrates that the bender element and resonant column tests can be used to determine the upper bound of soil stiffness in the laboratory. However, laboratory tests are often burdened by the problems of sampling disturbances and unrepresentative samples. Even if sampling effects such as structural reorientation, stress relief and specimen preparation are ignored, the volume of soil tested in the laboratory is generally much smaller compared to the volume of the ground. There is also a need to re-create the field environment (e.g. confining stresses, etc) in the laboratory so that the field conditions are simulated as closely as possible. Anderson and Woods (1975) suggested that an error of about 12% in laboratory measured G_{max} is common as the in-situ confining stress is usually poorly estimated in laboratory tests.

Figure 2-8. Typical stiffness-strain behaviour of soils

Soil disturbances also make it extremely difficult to measure G_{max} for cohesionless soils (e.g. gravels, sands), which are representative of in-situ conditions. The conventional triaxial test is commonly used in laboratory tests to measure the shear strength of soils. However, the limited stress configuration and strain measurement range in such tests coupled with bedding errors, etc., exaggerate the apparent strain and hence underestimate the shear modulus of soil samples.

2.4 Singapore Geology

Singapore consists of one main island and 58 islets, with the main island covering an area of 540 km², and islets covering a further area of 44 km² (PWD 1976). The physical land area of Singapore is also increasing with ongoing reclamation works being undertaken at several parts of the island. The current land area of Singapore is about 707

km² (Mapping Unit 2004). Singapore is a moderately low relief island (PWD 1976, Pan 1995) with about 10% of the island over 30m high and most of the remaining parts are less than 15m high. The highest land can be found in the Bukit Timah area. The Public Works Department (PWD 1976) has divided Singapore into nine Formations: Sajahat Formation, Gombak Norite, Paleozoic Volcanic, Bukit Timah Granite, Jurong Formation, Old Alluvium, Huat Choe Formation, Tekong Formation and Kallang Formation. A simplified geological map of Singapore is shown in Figure 2-9.

Figure 2-9. Simplified geological map of Singapore (from PWD 1976)

The definition, classification and formation of residual soils are first discussed to provide some insight into the type of soil in which field explosion tests were undertaken (Chapter 4). This is followed by a discussion of the Bukit Timah Granite, Jurong

2.4.1 Definition, formation and classification of residual soils

In Singapore, residual soils occupy almost two-third of the Singapore Island. They are mainly derived from sedimentary rocks of Jurong Formation in the western part of Singapore and the Bukit Timah Granite in the central part of Singapore. Soils, except organic soils, are formed from the weathering of rocks through physical, chemical and biological processes. The type of weathering process, the degree of weathering and the weathering products depend on the mineralogy of the bedrock, the topography and the climatic conditions during the weathering processes.

Soils can be either residual or transported in origin. Residual soils develop in situ, and their characteristics depend on the kind of bedrock from which they are derived. Transported soils are surficial deposits which accumulate due to erosion, transportation and deposition of weathered residual soil or bedrock (Perry 1986). The definition of residual soils varies from country to country. For the context of this research, a general definition suggested by Blight (1997) is adopted:

'A residual soil is a soil-like material derived from in situ weathering and decomposition of rock which has not been transported from its original location.'

Residual soils have distinct characteristics compared to transported soils in terms of engineering properties. According to Blight (1997), the conventional concepts of soil grain and permeability characteristics of transported soils are inapplicable to residual soils. Particles of residual soil often consist of aggregates or crystal that may easily break down and become progressively finer when the soil is reconstituted. The process of formation of residual soil profile is complex, difficult to understand and to generalise. However, it was generally agreed that the formation of residual soil is largely dependent on the climatic and topographical conditions.

It has been suggested (Uehara 1982) that tropical soils around the earth's Equator, including the residual soils of Singapore, contain a large proportion of low activity clays

such as kaolin, oxides and hydrated oxides of iron and aluminum. Due to the abundant rainfall in countries like Singapore, the top surface of the soil might undergo laterization, forming a type of soil which is usually termed as laterites or lateritic soils. The laterites or lateritic soils are largely distributed around the world especially in tropical humid areas.

The degree of weathering of residual soils is important in the study of their dynamic properties. Macari and Hoyos (1996) show that the degree of weathering was found to be an influencing factor on the dynamic properties of residual soils. An understanding on the weathering profiles of the residual soils is thus essential for dynamic applications.

The earliest rational system (a four-category system) for the classification of degree of rock weathering was proposed by Vargas and Pichler (1957). Little (1969) proposed a six-grade rock weathering system, shown in Figure 2-10, with grade I representing fresh rock and grade VI representing the residual soils. The scale of weathering of rock mass is shown in Table 2-1.

2.4.2 Bukit Timah Granite

The Bukit Timah Granite occupies about one-third of the main island's land area, and lies in the central part of Singapore. The granite has been a source of construction aggregates. The Bukit Timah granite consists of igneous rock from the Triassic Period (195 to 225 million years ago). The top portion underlying the rock has been transformed to residual soil by effect of rapid weathering under the humid tropical conditions in Singapore (Thum and Yong 1987).

Three distinct zones have been observed in this residual soil zone: the upper, intermediate and lower zones. The upper zone, about 6m thick, consists of reddish-brown sandy silty clay wherein laterisation has taken place.

Figure 2-10. Classification of a weathered rock mass profile (after Little 1969)

The laterisation process results in the formation of reddish-brown ferruginous soil of clayey texture, and which also has a concretionary appearance. The intermediate zone consists of 10m to 25m of yellowish-brown sandy clayey silt. The lower zone is greyish to whitish-grey sandy silt or silty sand. This layer may comprise of soil from weathering zone V (Thum and Yong 1987).

Table 2-1. Scale of weathering of rock mass (from Little 1969)

Grade or Zone	Degree of Weathering	Description	Remarks
VI	Residual soil	All rock material is converted to soil. Soil has not been significantly transported	SPT N value: 0 to 30
V	Completely weathered	All rock material is completely decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.	SPT N value: 30 to 100
VI	Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh rock may be present as discontinuous framework or as core stones.	SPT N value: over 100
III	Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh rock may be present as discontinuous framework or as core stones.	
II	Slightly weathered	Distinctly weathered through much of the rock fabric with some discontinuous surfaces.	
I	Fresh rock	No visible sign of rock material weathering. Perhaps slight discolouration of major discontinuity surfaces.	

2.4.2.1 Geotechnical and seismic properties

A comprehensive review of the geotechnical properties of Singapore soils and rocks is not an easy task, and is also outside the scope of this research. For this research, the main focus is on geotechnical properties related to dynamic soil behaviour, such as

density, shear modulus, shear wave velocity, etc. The geotechnical properties can also be used in empirical equations to obtain an estimate for the small-strain shear modulus (G_{max}) and for comparison with field measurements. The study of dynamic response of soil inevitably requires determination of their dynamic properties. However, there is limited data on dynamic properties of Singapore soils. Typical engineering and mechanical properties of Bukit Timah Granite are shown in Table 2-2.

Table 2-2. Typical engineering and mechanical properties of the Bukit Timah Granite (from Zhou et al. 1993)

Properties	Min. – Max.	Average
Density, Mg/m ³	2.43 – 2.65	2.61
Uniaxial Compressive Strength, MPa	158.9 – 232.2	185.7
Young’s Modulus, GPa	66.7 – 131.4	83.6
Poisson’s Ratio	0.05 – 0.31	0.2
Point Load Index, MPa	7.46 – 10.66	8.89
Brazil Tensile Strength, MPa	9.94 – 11.74	11.37
Four-Point Flexure, MPa	14.6 – 18.56	16.57
Sonic Velocity (V_p), 1000 m/s	5.49 – 6.27	5.79
Schmidt Hammer	60.4 – 67.6	64.5

PWD undertook a broad measurement of the seismic velocities of Singapore soils at seven sites (Veijayaratnam et al. 1993). In most of the seven sites, the average value of the S-wave velocity, V_s , was found to be constant in each soil layer. This is unlike the P-wave velocity, V_p , which progressively increases with depth. The concept of a constant V_s within a soil layer is an important observation. Published literature (e.g. Clayton et al. 1995) and analyses of seismic data also supported this observation. Table 2-3 provides a summary of the seismic properties of residual soils from the Bukit Timah granite. Lee et al. (1993) found the average V_s of the highly weathered granite and residual soils to be

about 300 m/s, substantially smaller than the average V_s of the underlying fresh granite which is about 2700 m/s (Zhou et al. 1993).

Table 2-3. Summary of properties of Bukit Timah Granite residual soils
(data from Veijayaratnam et al. 1993)

Soil Type	Shear strength (kN/m ²)	Bulk density (Mg/m ³)	P-wave velocity (m/s)	S-wave velocity (m/s)	Shear modulus G_{max} (MPa)	Poisson's ratio, ν	Elastic modulus, E_0 (MPa)
Residual soil	38	1.80	1650	200	72.0	0.4925	214.92

2.4.3 Jurong Formation

Jurong Formation is the bearing stratum for foundations in southern and western Singapore and consists of a variety of sedimentary rocks like mudstone, siltstone, sandstone, conglomerate and limestone. These sedimentary deposits have been subjected to different modes of weathering and resulted in the decomposition of the rock (PWD 1976).

The variety of stratified sedimentary rocks is formed in the late Triassic, and Lower to Middle Jurassic (approx. 135 to 225 million years ago). The Jurong Formation is severely folded and faulted. In low-lying areas such as Jurong River Basin, peaty clay or marine clay usually overlies the Jurong Formation. Based on the grain size, composition and hardness, six sedimentary facies are recognized: Queenstown facies, Jong facies, Ayer Chawan facies, Rimau facies, St John facies and Tengah facies. Under the action of weathering, most of these facies have been reduced to similar end products (PWD 1976). The geological formation at NTU, which is based on the Jurong Formation, is complicated and has been subjected to intrusions and faults in the past. PWD (1976) has suggested that the Tengah facies and Queenstown facies still exist within the site. The Tengah facies is typically a muddy fine to medium grained poorly lithified sandstone,

whereas the Queenstown facies consists of sandstone, clay, clayey sand or a mixture of two or more.

Given the diversity of sedimentary rocks and the complex geological operations on them, the characterization of soil in the Jurong Formation is not an easy task. Several borehole logs have been sunk around NTU and borehole logs at many locations show that the residual soils are of a wide array of colours. According to British Standards Code, BS 5930: 1999 – Code of practice for site investigations, only two dominant colours should be used to describe the soil type encountered. This has added some difficulty in comprehensively describing the residual soil and most soil investigation companies resort to lengthy descriptions to describe the residual soils.

2.4.3.1 Geotechnical and seismic properties

Thum and Yong (1987) produced several soil profiles that are typically encountered in Singapore geology. The profiles were intended to be used as a guide when interpreting field test results. Figure 2-11 shows a typical soil profile at the Jurong River Basin. The undrained shear strength and SPT N values shown in Figure 2-11 can be used to obtain an estimate for G_{max} using empirical correlations (see section 2.5.5). In the Jurong Formation, the residual soils in the Tengah and Queenstown facies were found to have a V_s value of 280 m/s and 210 m/s, respectively, as shown in Table 2-4.

2.4.4 Kallang Formation

The Kallang Formation is found around Sungei Kallang, Sungei Jurong, Sungei Serangoon and Sungei Seletar (PWD 1976). Five members are recognized in the Kallang Formation: marine member, alluvial member, littoral member, and transitional member and reef member. These members are identified based on the mode of sedimentation. Pockets of peat, shells and lithic fragments can also be found in this Formation (PWD 1976, Gupta 1986).

Depth (m)	Description	c_u (kPa)	γ (kN/m ³)	SPT
0	Fill Medium Stiff Sandy Silty Clay	50	18	10
2	∇ GWT Soft Peaty Clay	15	15	2
8	Medium Stiff Reddish Silty Sand	50	18	10
12	Loose Grayish Silty Sand	-	17	10
15	Medium Dense Dark Grey Silty Sand (Residual Soil)	70	19	-
18	Light Grayish Highly Weathered Sandstone	110	19	100
22	Weathered Sands	-	-	100

Figure 2-11. Typical soil profile and properties at Jurong River Basin (Jurong Formation) (data from Thum and Yong 1987)

Table 2-4. Summary of properties of Jurong Formation residual soils (data from Veijayaratnam et al. 1993)

Soil Type	Shear strength (kN/m ²)	Bulk density (Mg/m ³)	P-wave velocity (m/s)	S-wave velocity (m/s)	Shear modulus, G_{max} (MPa)	Poisson's ratio, ν	Elastic modulus, E_0 (MPa)
Residual soil of Tengah facies	69	1.83	1600	280	143.5	0.48	425.9
Residual soil of Queenstown facies	162	2.10	1750	210	92.6	0.49	276.5

The widespread marine clays usually consist of an upper marine clay and a lower marine clay separated by a weathered crust of the lower marine clay layer. The lower marine clay was deposited unconformably and was in turn overlain by unconsolidated peat, beach and young fluvial deposits. The separation layer between the upper and lower marine clay represents a significant boundary and can be 1m to 2.5m thick of stiff reddish brown or grey silty clay. These boundaries lie between 5m and 20m depth (Lim 1982). Although deposited at different times, the upper and lower marine clays share similarities in appearance and touch. Both members are essentially a kaolinite-rich clay deposit.

2.4.4.1 Geotechnical and seismic properties

An extensive study on the geotechnical properties of Singapore marine clay was undertaken by Lim (1982). He concluded that the marine clay in the Kallang Formation is normally consolidated, and that both the upper and lower layers do not differ significantly in engineering behaviour. Table 2-5 provides a summary of the geotechnical and seismic properties of peaty clay and marine clay of the Kallang Formation.

Table 2-5. Summary of properties of peaty clay and marine clay in the Kallang Formation (data from Veijayaratnam et al. 1993)

Soil Type	Shear strength (kN/m ²)	Bulk density (Mg/m ³)	P-wave velocity (m/s)	S-wave velocity (m/s)	Shear modulus, G_{max} (MPa)	Poisson's ratio, ν	Elastic modulus, E_0 (MPa)
Peaty clay	20	1.42	1350	60	5.1	0.499	15.3
Marine clay	13	1.51	1450	70	7.4	0.499	22.2

2.5 Determination of G_{max} using a Continuous Surface Wave (CSW) Method

2.5.1 Background

The measurement of in-situ ground stiffness using seismic methods (Figure 2-5) relies on the propagation of elastic waves (particularly the Rayleigh waves) through the ground. These methods are based on the fact that the velocity of wave propagation or impulse in an elastic body is a function of the modulus of elasticity, Poisson's ratio and the density of the material (Hvorslev 1949). As early as the 1940s, soil mechanics pioneers like Terzaghi (1943) and Hvorslev (1949) presented reviews on surface-wave testing. However, the work was hampered by the state of technology at that time, and involved tedious manual work. Similarly, Jones (1958) who used surface wave geophysics to assess materials under road pavement at the Transport Research Laboratory (UK) found the task hampered and formidable using an oscilloscope. The advent of spectral analyses and portable computers which enable online application of Fourier transforms, coupled with a better understanding of soil behaviour over the last two decades has given surface wave geophysics a boost in popularity.

The magnitude of shear strains induced by the propagation of Rayleigh waves in seismic tests is very small, typically 0.001% (Nazarian and Stokoe 1984). Geophysics thus offers an attractive method for small-strain shear modulus measurement by considering the propagation of elastic waves through an elastic medium, thereby enabling the mathematical formulation to be based on the familiar theory of elasticity.

In-situ seismic tests, such as the surface wave techniques, are non-intrusive and test a large zone of the ground at its natural stress and saturation level. The results obtained from such tests are thus more representative of the ground, compared to laboratory tests.

2.5.2 Seismic methods and their differences

Most seismic methods induce shear strains lower than 0.001% and the shear wave velocity, V_s can be used to compute G_{max} . The measured shear wave velocity is generally considered the most reliable means to obtain G_{max} for a soil deposit. These methods involve the creation of a transient or steady-state stress wave (source) and the interpretation of the arrival time and spectral response at one or more locations (receivers). The generation of the impulse wave by the source can vary from a sledgehammer blow at ground surface, to a buried explosive charge or to an active varied frequency source vibrator. There are different methods for creation of impulse waves. These sources generate P-waves, S-waves and surface waves at different relative amplitudes depending on the dominant wave in the method used.

2.5.3 Rayleigh wave dispersion

Bedford (1994) presented a compilation of previously published literature, which shows a relationship between Rayleigh wave and S-wave velocities. Bedford showed that the ratio of Rayleigh waves to S-waves depended only on the Poisson's ratio in a homogeneous, isotropic solid. Figure 2-12 illustrates the relationship between Poisson's ratio and velocities of wave propagation in a semi-infinite elastic medium.

The S-wave velocity can be obtained from the Rayleigh wave velocity (Aki and Richards 1980):

$$\frac{V_r}{V_s} = 0.9194 + 0.1612\nu \left(= \frac{1}{p} \right) \quad (2.4)$$

where V_r is the Rayleigh wave velocity, V_s is the S-wave velocity, ν is the Poisson's ratio, and p is a constant. When $\nu \rightarrow 0$, Equation 2.4 can be written as

$$V_s = 1.088V_r \quad (2.5)$$

Figure 2-12. Relation between Poisson's ratio, ν , and velocities of propagation of compression (P), shear (S) and Rayleigh (R) waves in a semi-infinite elastic medium (from Richart et al. 1970)

Subsequent research on the determination of Poisson's ratio suggested that a 25% error in estimating the Poisson's ratio only resulted in less than 10% change in S-wave velocity. In most cases Poisson's ratio can be measured with better accuracy than the assumed error (Carmichael 1989).

In stratified grounds, it is important to obtain the dynamic properties of each soil stratum. In a non-uniform or bounded media, Rayleigh waves are known to travel at a velocity depending on their wavelength. Therefore surface waves of different wavelengths propagate at different velocities because they penetrate the ground to different depths. In geophysics, this phenomenon is described as 'dispersive'. For a given mode, Rayleigh waves with longer wavelength (lower frequency) would penetrate deeper than those with shorter wavelength (higher frequency).

A simple analogy for this dispersive property can be found in optics: the ground separates or decomposes the applied source according to wavelength, like a prism which separates white light to different colors according to their wavelengths. Rayleigh waves are non-dispersive in a solid homogeneous half-space condition, something that does not occur in field applications. In practice, the majority of Rayleigh wave energy is contained within a zone that extends to a depth of approximately one wavelength (Tokimatsu et al. 1991). Rayleigh wave dispersion is a key concept in subsurface geophysics and can be exploited to determine soil stratification and the associated dynamic properties.

2.5.4 Inversion of field dispersion curve

Inversion is an important technique to understand subsurface profile of soil masses through surface wave test. Inversion refers to the process of converting field dispersion curve to a Rayleigh wave velocity-depth profile through wave propagation theory.

Inverse problem in geotechnical problem is complex. Rayleigh modes of wave propagation make the inversion analysis more complex. For this reason, most inversion approaches do not consider Rayleigh wave propagation modes in their inversion algorithm. Furthermore, soil masses are usually stratified. Material properties (λ , G , ρ , etc) may vary across layer interfaces. This introduces conditions that stress and displacement fields are continuous across these interfaces. These interfaces reflect the wave and make the Rayleigh wave modes complex (Gucunski and Wood 1992). However, the Rayleigh waves in layered media can be solved through an eigenvalue problem. Lai and Rix (1998) developed a linear differential eigenvalue problem for Rayleigh wave propagation in a vertically heterogeneous medium. Their work is the further development of Aki and Richards' work (1980). Their eigenvalue problem is comprised of displacement and stress eigenfunctions. After applying non-trivial solution and appropriate boundary conditions, the linear eigenvalue problem can be solved.

Rayleigh wave modes depend on soil type, soil layer and frequency of excitation. In general several modes of wave propagation exist at any excitation frequency and these modes depend on layer depth and the frequency of excitation (Lai and Rix 1998). Ewing et al. (1957) proved that in a medium composed of a finite number of homogeneous layers overlying a homogeneous half-space; the total number of Rayleigh wave propagation modes is always finite. These behaviours of Rayleigh waves have also been observed by many authors (e.g. Tokimatsu et al. 1991). Tokimatsu et al. (1991) also suggested that the existence of different modes at a given frequency could be explained by the constructive interference occurring among waves. It should be noted that Rayleigh wave modal behaviour is different from 'double back' encountered in surface wave testing (Kalinski 1998). When a 'double back' occurs, a phase spectrum (obtained by Fourier transformation of time domain data) can have different frequencies for the same phase. Since each phase difference defines a wavelength, the wavelength appears to exist at three frequencies. This can occur when the time domain data contains a significant amount of energy other than arrival of surface wave energy.

Up to now, three principal surface wave inversion methods are widely used in geotechnical engineering. They are:

- Factored wavelength method (also called wavelength-depth method)
- Linear models
- Finite element (FE) approach

2.5.4.1 Factored wavelength method (or wavelength-depth method)

The factored wavelength method is the simplest practical tool. It is widely used in surface wave tests as it offers a relatively quick method to process and present data profile on site, thereby facilitating a preliminary site assessment. Furthermore, this method can also provide an initial estimate for the phase velocity-depth profile to other complex inversion algorithms such as FE inversion method.

The principle of factored wavelength method is very simple. A representative depth is a fraction of the wavelength, λ . That is, λ/z is a constant for surface wave, where z is the depth. This constant may vary with soil types and site conditions (Jones 1958, Ballard and Maclean 1975, Abbis 1981, Matthews et al. 2000). For example, Gazetas (1982) recommended $\lambda/z = 4$ for sites where stiffness increases significantly with depth, and Jones (1958), Ballard and McLean (1975), Abbis (1981) recommended $\lambda/z = 2$ for more homogeneous sites. Gazetas (1982) recommended that $\lambda/z = 3$ is applicable for most test sites, obtained through a regression analysis on field-test results and the average scatter of the test data.

Rayleigh wave theory can roughly give the theoretical background of this method. It is known that Rayleigh waves can penetrate to a depth of approximately one wavelength. If a triangular Rayleigh wave is distributed over an effective depth of one wavelength, the centroid of this triangle is at one-third depth of the wavelength (Butcher and Tam 1994). This explanation is perfect but only a few literatures adopt it to support the $\lambda/z = 3$ for inversion of field dispersion curve. Matthews and Clayton (1993) conducted a series of surface wave tests on UK soils and found that the simple inversion can be successfully applied to highly compressible soils.

The following example will illustrate this simple and widely used inversion method. Figure 2-13(a) is a field dispersion curve. If the phase velocity is known, the small-strain shear modulus, G_{max} , can be calculated by $G_{max} = \rho p^2 V_r^2 n$ where ρ and p ($=V_s/V_r$) values are known. The depth which corresponds to G_{max} is calculated as $z = \lambda/3$. Hence this simple inversion procedure can be used to obtain shear modulus profile from the field dispersion curve at any site. The derived shear modulus profile is shown in Figure 2-13(b).

The main weakness of the factored wavelength method is its highly arbitrary nature, its inability to address modal propagation of Rayleigh waves, and its uncertainty in the assignment of shear modulus when vertically heterogeneous grounds (Ballard and McLean 1975) are considered. Clayton et al. (1995) compared FEM solutions of a

construction site near Sweden and found that this simple inversion technique underestimates the shear modulus when the stiffness changes rapidly with depth. However, the ratio may be variable with soil conditions such as $\lambda/z=2$ in soft grounds and $\lambda/z=4$ in stiff grounds. This makes the factored wavelength method difficult to use in practice because the soil profile is unknown for a site.

Figure 2-13. Inversion of field dispersion curve using factored wavelength method ($\lambda/z = 3$)

2.5.4.2 Linear models

Linear models have attracted much research due to their versatility and ability to deal with the various aspects of Rayleigh wave propagation. Because Rayleigh wave velocity is related to the S-wave velocity, S-wave velocities can be determined adequately from Rayleigh wave velocities (Carmichael 1989).

Linear model is based on linear elastic wave theory. Because the strain in wave problem is usually small, linear elasticity is reasonable. When a wave propagates in an elastic solid, Navier's equation is the equation of motion (Ewing et al. 1957). If the solid media is vertically heterogeneous and body force is ignored, the fundamental equation is given as follows:

Navier's equation of motion:

$$G\nabla^2 \mathbf{u} + (\lambda + G)\nabla(\text{div} \mathbf{u}) + \mathbf{e}_y \frac{d\lambda}{dy} \text{div} \mathbf{u} + \frac{dG}{dy} \left(\mathbf{e}_y \times \text{curl} \mathbf{u} + 2 \frac{\partial \mathbf{u}}{\partial y} \right) = \rho \frac{\partial^2 \mathbf{u}}{\partial t^2} \quad (2.6)$$

with boundary conditions

$$\begin{aligned} \sigma(r, y) \cdot \mathbf{n} &= 0 & \text{at } y &= 0 \\ u(r, y) &\rightarrow 0, \quad \sigma(r, y) \cdot \mathbf{n} \rightarrow 0 & \text{at } y &\rightarrow \infty \end{aligned} \quad (2.7)$$

and interface condition

$$\begin{aligned} \mathbf{u}(r, y^+) &= \mathbf{u}(r, y^-) \\ \sigma(r, y^+) \cdot \mathbf{n} &= \sigma(r, y^-) \cdot \mathbf{n} \end{aligned} \quad (2.8)$$

Such a problem has the solution with the following form:

$$\mathbf{u} = \left\{ u_1 = r_1(y, k, \omega) \cdot e^{i(\omega t - kr)}, \quad u_2 = 0, \quad u_3 = ir_2(y, k, \omega) \cdot e^{i(\omega t - kr)} \right\} \quad (2.9)$$

where G is shear modulus and λ is a Lamé constant.

$\nabla = i \frac{\partial}{\partial x} + j \frac{\partial}{\partial y} + k \frac{\partial}{\partial z}$ is an operator, $\nabla^2 = \nabla \cdot \nabla = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$ is Laplace operator.

$$\text{Curl } \mathbf{u} = \nabla \times \mathbf{u}, \text{ div } \mathbf{u} = \frac{\partial u_x}{\partial x} + \frac{\partial u_y}{\partial y} + \frac{\partial u_z}{\partial z}.$$

Frequently, Rayleigh wave propagation in an elastic and vertically heterogeneous medium is described mathematically in the form of a linear eigenvalue problem:

$$\frac{df(y)}{dy} = A(y) \cdot f(y) \quad (2.10)$$

where $f(y)$ is a 4-element column vector consisting of the horizontal and vertical components of the displacement and stress eigenfunctions, $A(y)$ is a 4x4 matrix whose elements are functions of the wavenumber, mass density, and elastic moduli of a soil deposit. Two approaches can be used in the solution of the Rayleigh wave propagation problem. The first is a forward problem, where given the medium parameters (e.g. ρ , V_s , etc), the dispersion curve is computed. The second approach is the inverse or back analysis problem, where the Rayleigh wave velocity and associated frequency are known, and the medium parameters (especially shear wave velocity and depth) are calculated. The eigenvalues of the eigenproblem can be obtained by applying conditions for a non-trivial solution and inserting the appropriate boundary conditions. The boundary conditions to be used for Equation (2.10) are vanishing stress at the free surface of the half-space, and the radiation condition as y tends to infinity (Rix and Lai 1998).

The forward problem has not attracted much interest, as the shear wave velocity of the soil medium is usually not known, and field seismic test results would result in a field dispersion curve, from which the shear wave velocity and associated depth are to be determined. As such, there has been keen interest in the second approach (i.e. the inverse or back analysis problem), where the shear wave velocity and depth are inferred

from a given dispersion curve. However, Rayleigh wave propagation inverse problems are known to be notoriously ill-posed. Groetsch (1993) indicated that a well-posed problem should have a unique solution and that the solution should be insensitive to small changes in the observed values (i.e. stable). In practice ill-posedness can be a problem as the inverted solution may correspond to more than one experimental dispersion curve, and hence lead to uncertainty in analyses. Groetsch (1993) suggested that non-uniqueness in the solution could be caused by a lack of sufficient information to constrain the solution, as well as the magnitude and extent of loading (which is often very small compared to the area involved). Therefore, one way to avoid ill-posedness is to impose additional information (called a-prior information) on the problem, e.g. many algorithms also include the P-wave velocity or material damping ratio in the characteristic equation.

The computational effort developed for the solution of Rayleigh eigenvalue inverse problem is devoted to two tasks. The first task is to develop an algorithm which can construct a theoretical dispersion curve based on the properties of an assumed profile, and the second task is to develop an algorithm to minimize the error between the theoretical and experimental dispersion curves. Researchers have proposed several algorithms for minimising the error between the theoretical and experimental dispersion curves, so as to make the computational task more efficient and enable a solution to be obtained more quickly (Constable et al. 1987, Parker 1994).

Thomson (1950) and Haskell (1953) proposed a matrix transfer method, which was very popular in the early days as this method was conceptually simple and could be easily implemented in a computer. However, this method suffered numerical instability, especially at high frequencies. It has been modified and improved by numerous researchers (e.g. Harvey 1981). Kenneth and Kerry (1979) introduced a new class of algorithm for solving eigenvalue problems. This method is based on reflection and transmission coefficients. Reflection and transmission matrices are developed for a stratified medium. In all these cases, the assumption of a linear elastic constitutive model will generally lead to acceptable results, provided that only the shear wave

velocity profile is of interest. Rix and Lai (1998) suggested that if damping is also required, a visco-elastic constitutive model should be used in conjunction with a coupled linear inversion scheme. However, such models inevitably require the solution of complex-valued parameters.

Xia et al. (1999) discussed a simple and efficient algorithm for Rayleigh wave inversion. They undertook sensitivity analyses for input parameters (density, strata thickness, S-wave velocity, P-wave velocity) and concluded that V_s is a dominant parameter influencing changes in the phase velocities, thereby simplifying the resulting inverse problem. The least-square minimization technique is widely applied to the convergence of solution. Although the formulation only considered the dominant propagation mode by Rayleigh waves, the results presented are in close agreement with model test results. Xia et al. also showed that their algorithm provided reasonable results when compared with seismic field test results.

Honjo et al. (1998) demonstrated that whilst the least-square method was a popular minimization tool, the use of an extended Bayesian method was much better for dealing with ill-posed solution. The main features of this method are the introduction of scalars, which automatically adjust the relative weights of the observed data (field dispersion), and the a-prior information, thereby constraining the solution. Ganji et al. (1998) developed an automated inversion procedure for the spectral analysis of surface wave (SASW) incorporating a non-linear minimization technique. This algorithm is elegant as it can handle all the Rayleigh waves and their associated modes.

A linear inversion approach was also applied in this research. This is an elastic, uncoupled, surface wave inversion program developed by Lai and Rix (1998) and was obtained from the internet website of Prof Glenn J. Rix at <http://www.ce.gatech.edu/~grix/>. The Matlab script files for the inversion program was installed on a personal computer. The program requires Compaq Visual Fortran run-time libraries to be installed in the computer before the program can be executed in the Matlab environment.

The program solves the nonlinear inverse problem of estimating the shear wave velocity profile given a surface wave dispersion curve (i.e., phase velocity vs. frequency) and associated uncertainties. The user may choose to perform an inversion based only on the fundamental mode of surface wave propagation or to perform an inversion using the effective velocity, which is velocity of several superimposed modes of propagation. The medium is assumed to be elastic and the solution of the inverse problem is uncoupled from the related problem of estimating the shear damping ratio profile from surface wave attenuation data. The input parameters for the program consists of vectors of frequencies, experimental phase velocities, standard errors of experimental phase velocities, layer thicknesses (not including half space), layer mass densities, initial shear wave velocities and Poisson's ratios. The output from the program provides the final shear wave velocities (empty if no convergence), vector of rms errors for each iteration, matrix of theoretical phase velocities and vector of standard deviations of final shear wave velocities.

The strategy of the algorithm is as follows: given an experimental dispersion curve and associated uncertainties, find the smoothest shear wave velocity profile subject to the constraint of a specified misfit between experimental and theoretical data. Convergence is defined as achieving a root-mean-square (rms) error of 5.0 or less (i.e., the specified misfit) or a negligible change in the shear wave velocity profile from one iteration to the next. A maximum of 20 iterations was specified in the program. The user may change these parameters as desired. The inversion program is based on the algorithm presented in Constable et al. (1987). A detailed description of the inversion algorithm can be found in Lai and Rix (1998).

2.5.4.3 Finite element approach

Finite element approach can be used to solve the inverse problem of wave propagation through a synthetic dispersion curve. Figure 2-14 shows the flow chart for the application of FEM to the inverse problem.

Figure 2-14. Inversion procedure by FE approach

There are two steps in the FEM analysis: solving the forward problem and parameter optimization. If a problem is well-posed, its solution is unique. This is a forward approach. An inversion problem is the optimization of many forward problems under an optimization function. Therefore, FEM will solve a series of well-posed forward problems to achieve the optimized match of synthetic dispersion curve and field dispersion curve. The adjustable parameters may be shear modulus or thickness of soil layers or both. This method is regarded as exact and robust (Lai and Rix 1998) as the higher modes of wave propagation are naturally accounted for. Because FEM is a powerful tool, complex constitutive laws, boundary condition and geometrical conditions may be involved in the inverse problems. The best match function can be selected and guided by least square method or genetic algorithm (Liu et al. 2002). In order to determine the body wave contribution in the near field, Green's function or

semi-analytical solutions (Nogami and Chen 2003, Wolf 2003) can be applied in some zones of problem domains.

Finite element method has its own demerits. For example, mesh size, time step size and numerical damping are three major issues to be solved in the numerical algorithm (Gucunski et al. 1996). The size of the elements is determined by minimizing numerical errors in the phase velocity for the wavelength of interest. The error is described as a function of a dimensionless parameter $\frac{\lambda}{2\pi\Delta s}$, where λ is the wavelength and Δs is the element size. The larger the parameter, the smaller is the error, as shown in Figure 2-15.

Figure 2-15. Error in phase velocity due to discretization of continua
(from Gucunski et al. 1996)

The time step should be small enough to capture all of the desired frequency accurately, and also to provide a numerically stable integration over time for transient dynamic analysis. The stability limit is equal to approximately the time needed for a wave to cross the smallest element in the model. The third issue is numerical damping, which can reduce the magnitude of the response for high frequency components faster than for

low frequency components. In order to capture the high frequency components, the meshless method may be useful because of its high order of approximation and element-free nature (Liu and Chen 2001, Wang et al. 2002).

The finite element method is computationally more tedious and involves an iterative forward modelling because of iterations. As will be shown in Chapter 3, the nodal information (time domain) is Fourier transformed to decompose the continuous system response to its harmonic wave components (frequency domain). The gradient of the plot of phase angle against distance from the source can be used to obtain the Rayleigh wavelength for that frequency. This process can be repeated for various frequencies (e.g. 5 Hz to 100 Hz), thereby yielding a synthetic dispersion curve (Groetsch 1993).

As a summary, the following points must be considered when an appropriate FE scheme is selected for the inverse problem of surface wave tests:

Firstly, a finite element model should be suitable for the particular problem. FE analysis is used to reproduce the field dispersion curve and hence determine the upper-bound shear modulus profile. The source frequency and boundary conditions should be considered. As discussed in the previous section, the source frequency determines the element size and time step size. If the high frequency components are to be captured, fine elements and high-order approximation are preferable.

Secondly, constitutive law of soils should be suitable to describe the stress-deformation behaviour. In the iterations, properties of the soils are important to the convergence of the numerical algorithm. In seismic tests, wave propagation in most soils is elastic as the strains are small. Therefore, the propagation of elastic waves during a seismic test can be simulated by an elastic media.

Lastly, the finite element model domain size and boundary conditions determine computation cost and efficiency. FEM is a domain-based problem and ground is an infinite medium. The larger the finite element model, the larger will be demand on

computational resources. If geometry and load conditions are symmetrical, only one-half of the geometry has to be modeled. Because of truncation of computational domain, far field responses should be approximated through boundary conditions such as transmission and reflection boundaries (Zienkiewicz et al. 1998).

Based on the above considerations, inversion of the CSWS dispersion curve was performed using a two-dimensional axisymmetric finite element mesh incorporating non-reflecting boundaries. The commercial FE program LS-DYNA was used. The initial soil properties and stratification were estimated with the help of the factored wavelength method. The results will be discussed in Chapter 3.

2.5.5 Appraisal and Applications of CSWS

CSWS is non-intrusive and tests a large volume of undisturbed ground at its natural in-situ stress and saturation level. Given these considerations, the results are more representative of the ground compared to small volume of samples recovered and tested in the laboratory. The CSWS is highly portable and is complemented by a portable generator for ease of deployment in the field. It takes about 20 minutes to set-up the CSWS on site and once this is done, the CSWS can provide about 100 stiffness measurements in about 2 hours. The depth of stiffness measurements is dependent on the soil type.

Rayleigh waves have depths of penetration depending on their wavelength. As the amplitude of Rayleigh waves diminishes exponentially with depth, there is a limitation on the maximum depth of measurement using the CSWS method. The CSWS is thus suited to near surface investigation only. The system can provide stiffness profiles in soft ground up to about 10m, and can reach a maximum depth of 40m in hard soils and rocks.

It has also been suggested that anisotropy in soils can affect the results obtained from different seismic tests, e.g. due to the propagation of horizontally polarised shear wave

(or SH-waves) in cross-hole tests (Matthews et al. 1996). White (1983) proposed that seismic techniques can be used to determine anisotropy of the ground. For surface waves tests, it has been suggested that the S-wave velocity in anisotropic soils affects the relationship between V_r and V_s and can be accounted for by varying ' p ' in Equation (3.4) (White 1983). The presence of nearby vibration sources or environmental conditions (e.g. vehicular traffic) can affect the performance of the CSWS.

Some of the common applications of the surface wave method are listed below:

- Shear Modulus Profiling: Gordon et al. (1995) and Anand et al. (2000) have demonstrated how the CSWS method can be used for in-situ characterisation of soil.
- Assessment of shear damping ratio (Lai and Rix 1998). The damping ratio is of interest in dynamic analyses of structures (e.g. machine foundations, earthquake, etc.)
- Assessment of liquefaction potential (Stokoe and Nazarian 1985). An algorithm has been proposed for determination of liquefaction potential in cohesionless deposits.
- Prediction of ground deformation (Matthews and Clayton 1993). The CSWS can provide much better predictions for ground deformation, as the system measures the very small strain ground stiffnesses. The results have been supported by finite element back analyses.
- Quality control for ground improvement (Cuellar et al. 1995): The surface wave method is ideal for assessing ground improvement (e.g. use of stone columns) and for monitoring changes in site characteristics.
- Determination of landfill thickness (Butcher and Tam 1994): The surface wave method allows a non-intrusive method of determining the stiffness of the landfill and associated depths, without having to come in physical contact with the underlying contaminants.

2.5.6 Comparison of $G_{\max}$ obtained from empirical methods and published literature

Apart from laboratory and field seismic methods, researchers have also suggested empirical equations for shear modulus or S-wave velocity over the past thirty years. There is a number of correlations ranging from simple relationships for estimation of shear modulus with minimum soil information to more complex relationships that take into account soil behaviour, field and laboratory data.

Ashford and Jakrapiyanun (1999) compared the estimated and measured small-strain shear wave velocity (V_s) for Bangkok soils. A huge archive of geotechnical data (from 44 boreholes) was available and a best-estimate V_s profile was developed for the Bangkok soils up to a depth of 80m. The selection of appropriate empirical formulas for V_s is a complicated task, given the availability of many empirical formulae. In practice, available soil information could influence to some extent the selection of the empirical formula for V_s . Ashford and Jakrapiyanun (1999) were concerned with the empirical nature of relationships used and they confirmed the selected formula by undertaking in-situ V_s measurements using downhole surveys at five locations. Cased boreholes were used at all five locations. Reasonable agreement between the estimated and measured shear wave velocity profiles was obtained. The depth of V_s profile was estimated using a combination of the downhole test results and changes in V_s values, which suggested a change in stratigraphy. As mentioned in Section 2.4.1.1, the average value of V_s in each layer was found to be constant, and changes in V_s values can be attributed to a change in stratification. However Ashford and Jakrapiyanun (1999) did not address the possible effects on S-wave propagation such as wave reflection at strata interfaces (Kalinski 1998). Based on the field tests, they suggested that the shear wave velocity for soft Bangkok clays can be estimated using:

$$V_s = 68.7c_u^{0.475} \quad (2.11)$$

where V_s is the shear wave velocity in m/s, and c_u is the undrained shear modulus of the soil in kPa.

Seed and Idriss (1970) proposed one of the initial formulae, sometimes called the normalized modulus (also see Figure 2-7):

$$G_{\max} = 2200c_u \quad (2.12)$$

Subsequent researchers however found that the normalized modulus is not a constant but is dependent on the overconsolidation ratio, plasticity index and effective confining stress of the soil (Ashford and Jakrapiyanun 1999). Seed and Idriss (1970) also suggested the following relationship for sand and silty sand using regression analyses, based solely on the Standard Penetration Test (SPT) index, N :

$$V_s = 56.388\sqrt{N} \quad (m/s) \quad (2.13)$$

A summary of the proposed empirical equations for $G_{\max}$ is listed in Table 2-6. These empirical equations can be used with available site information, but should be calibrated using results obtained from field and laboratory tests.

2.5.7 Developing the stiffness-strain curve using $G_{\max}$

$G_{\max}$ is an important parameter to define the stiffness-strain curve in Figure 2-8, and the complete shear modulus curve will be required for numerical modelling of ground response subjected to dynamic loads. This is important as dynamic loads impose a varying degree of strain on soils at various distances, and this effect has to be accounted for in the numerical analyses.

Table 2-6. Empirical equations for determination of G_{max}

Reference	Empirical equation	Remarks and applicability
Hardin and Black (1968)	$G_{max} = \frac{6908(2.17 - e)^2}{1 + e} \sigma_0^{1/2}$ $\sigma_0 = 1/3(\sigma_1 + \sigma_2 + \sigma_3)$ $\sigma_1 = \sigma_v$ $\sigma_2 = \sigma_3 = K_o \sigma_v$ $K_o \approx 1 - \sin(\phi')$	<ul style="list-style-type: none">• Applicable for round-grained sandy material.• e = void ratio• σ_0 = average effective confining pressure• σ_1 = effective vertical stress• ϕ' = drained friction angle
Hardin and Black (1968)	$G_{max} = \frac{3230(2.97 - e)^2}{1 + e} \sigma_0^{1/2}$	<ul style="list-style-type: none">• Applicable for angular grained sandy material.
Seed and Idriss (1970)	$G_{max} = 1000 K_{2(max)} \sigma_0^{1/2}$	<ul style="list-style-type: none">• Applicable for sandy soils.• $K_{2(max)}$ is a constant that varies with sand relative density.
Hardin and Drnevich (1972)	$G_{max} = \frac{3230(2.97 - e)^2}{1 + e} (OCR)^k \sigma_0^{1/2}$	<ul style="list-style-type: none">• Applicable for clays of moderate sensitivity.• OCR is the over-consolidation ratio.• K is a constant that varies with soil plasticity index.

The shear modulus-strain curve can be defined by conducting field seismic tests to determine G_{max} and laboratory tests at various strain levels to develop the lower portion of the curve. In practice, it would not be possible to undertake field tests at all the test sites, nor undertake laboratory tests at all the possible strain levels given the limited resources and costs involved. An attractive alternative is to use established shear modulus ratios for Singapore soils to factor down the G_{max} values at a given strain level.

Kramer (1996) provided some guidelines for scaling down G_{max} values for use in normal engineering applications. This is illustrated in Figure 2-16.

Figure 2-16. Stress-strain curve with variation of shear modulus and modulus reduction curve (from Kramer 1996).

The shear modulus degradation with cyclic strain is represented by means of the modulus reduction curve. The modulus reduction curve normalizes the shear modulus (G) with respect to the maximum shear modulus (G_{max}) and is commonly referred to as the modulus ratio. Figure 2-16 shows a schematic of the typical cyclic behaviour of soils. As the soil element loses stiffness with the amplitude of strain, its ability to dampen dynamic forces increases. This is due to the energy dissipated in the soil by friction, heat or plastic yielding. Normalized relationships, such as Figure 2-16, allow the engineer to use well-established degradation curves and scale them to the measured in-situ value of G_{max} . For example, Kramer suggests that the stiffness at operational strain levels around proposed construction works is usually between 40% to 80% of the maximum stiffness level. Given the stiffness degradation curve and the expected field strain level, it would be possible to estimate the field shear modulus value from field seismic tests. In the absence of extensive dynamic soil testing at all ranges of shear strain, G_{max} can be used as the scaling parameter.

Hardin and Drnevich (1972) suggested a hyperbolic relationship between shear modulus and shear strain for clays which is given by:

$$G = \frac{G_{\max}}{(1 + \gamma'_h)} \quad (2.14)$$

where γ'_h is the hyperbolic strain given by

$$\gamma'_h = \left(\frac{\gamma}{\gamma_r} \right) \left[1 + ae^{-b \left(\frac{\gamma}{\gamma_r} \right)} \right] \quad (2.15)$$

where γ is the shear strain, γ_r is a reference strain, and a and b are constants.

Equations (2.14) and (2.15) can be used to develop a shear modulus-strain curve by incorporating shear moduli obtained from field seismic tests (i.e. $G_{\max}$) and those obtained from laboratory cyclic triaxial tests at various strain levels. Leong and Pan (1998) used the Hardin and Drnevich (1972) relationship to develop shear modulus-strain curves for Singapore marine clay using seismic down-hole and cross-hole surveys, and dynamic triaxial test results. They established the $G_{0.01}/G_{\max}$ ratios for upper, intermediate and lower marine clay layers, where $G_{0.01}$ is the shear modulus at 0.01% strain. Their results also suggested that the Singapore marine clay is not sensitive to $G/G_{\max}$ ratios for shear strains greater than 0.01%, as shown in Figure 2-17.

Recently, Leong et al. (2003) assessed the stiffness-strain relationships of Singapore residual soils from cyclic triaxial tests. They found that the suggested stiffness-strain relationships for saturated sands and clays in the literature overestimated the shear moduli of Singapore residual soils. Leong et al. (2003) suggested that the following formula proposed by Borden et al. (1996) is more suitable for Singapore residual soils:

$$\frac{G}{G_{\max}} = \frac{1}{(1 + a\gamma^b)^c} \quad (2.16)$$

where a , b and c are constants.

Figure 2-17. Shear modulus versus shear strain at a soft soil site in Singapore
(from Leong and Pan 1998)

2.6 Field explosion tests

The prediction of ground vibration from blasting has gained attention for a long time. Efforts were made to determine a safe level of vibration for buildings and other structures (e.g Northwood et al. 1963). It is very difficult to predict the magnitude, frequency and duration of ground vibrations as they are affected by many variables. Undoubtedly, the level of ground vibration will depend on the maximum explosive charge and the distance between the blast and measurement point. But other variables, such as soil type, topography, design of detonation system and efficiency, coupling of the explosive to the surrounding media, etc., may significantly influence the characteristics of the ground motion generated. It is thus not possible, in common blasting situations, to take into account all these factors into the elasto-dynamic equations of motion and solve for the unknown particle velocity.

Therefore field test have been used extensively for investigating ground vibrations due to explosions (Ingram 1977). Field tests are monitored at different distances from the

blast source and data from blasts in the same or similar geologic medium are used. The data obtained are fitted to some model with a few empirical constants. The values of these constants are valid for a particular site and for a particular kind of blasting. The fitted data are usually further complemented by numerical modelling.

2.6.1 Importance of determining local geology and soil characteristics

Explosive pressures are developed very quickly and applied dynamically. Generally some of the energy is transmitted through the surrounding ground in the form of seismic strain energy (Tabatabai 1987). It is the seismic strain energy propagating through the ground that is responsible for the ground vibrations that accompany blasting. The amount of seismic energy generated will be greater if the charge weight is higher or if displacement of the surrounding ground is restricted.

The local geology and soil characteristics have moderate to significant influence on the ground vibration characteristics (Wiss and Linehan 1978). Local soil characteristics should therefore be determined for any dynamic application. Presence of overburden (soil overlying basic soil structures) generally does not affect the magnitude of particle velocity. Also, since soil has a lower modulus compared to rock, the propagation velocity of waves, V , decreases faster in soil compared to rock. The frequency of vibration, f , decreases but the displacement, A , increases significantly with the increase in overburden thickness. For sinusoidal excitation, $V = 2\pi fA$ (Nicholls et al. 1971, Ladegaard-Pedersen and Dally 1975).

An additional effect of overburden on ground vibration is that as displacement increases, more energy is required for vibrating the particles at higher amplitude, and to overcome the friction between adjacent particles. Therefore, the ground vibration will lose energy at a high rate as the waves spread out. The magnitude of vibration, at large distances, will decrease rapidly if a thick layer of overburden is present.

2.6.2 Significant properties of commercial explosives and loading characteristics

Commercial explosive products are mixtures of two or more chemical compounds, formulated in such a way that the overall effects produced by the rearrangements of the various component atoms contained within the molecular constituents will react exothermally (ISEE 2000). Mixtures usually can contain one or more oxygen suppliers and one or more reducers. For practical purposes, this chemical reaction is considered simply as being a very rapid oxidation in the form of a combustion, with the development of high pressures produced by the highly heated gaseous products (ISEE 2000). To start an explosion, an initiation whose function is to provide the proper pressure and temperature conditions necessary for the chemical ingredients to react efficiently, is required. A primer, or an explosive charge containing the initiating device, is generally employed at one end of a blasthole's explosive column to ensure that the entire charge will fire and that the reaction direction is as desired.

Industrial explosives have a rather limited range of densities, from about 0.6 to 1.7 g/cm³, and designed reaction velocities that may be as low as 1520 m/s (5,000 fps) up to 7620 m/s (25,000 fps) (Taiquideen 1982). However, velocities from about 2740 m/s to 4570 m/s (9,000 to 15,000 fps) are the most common for the products typically used for the vast majority of industrial blasts. Potential energy content on the basis of mechanical work equivalent for the most frequently used products are from about 152,440 kg-m/kg (500,000 ft-lb/lb) up to as much as six times that amount, depending on the specific components used.

An explosive reaction may either be a deflagration or a detonation (ISEE 2000). A deflagration is a rapid burning at a velocity less than the sonic velocity of the unreacted explosive. A detonation is also a rapid reaction, but proceeds at speed greater than the speed of sound in the unreacted explosive. This condition occurs where a compressional wave or shock front is produced with a finite pressure discontinuity formed at the front of the combustion zone, the shock front moving through the unreacted explosive at a supersonic rate. As a consequence there are two independent pressures generated, one

initially from the detonation front passing in advance through the explosive and one produced by the gaseous reactions products remaining in the hole, Figure 2-18.

Figure 2-18. Characteristics of detonation reaction for explosives

Conditions must be favorable for a detonation to occur (ISEE 2000). The explosive’s chemical composition, physical makeup, and intimacy and uniformity of ingredient mixing must be proper. There also must be adequate initiating energy and sufficient confinement provided. Because reaction products expand laterally as they are formed and the reaction progresses longitudinally along an explosive column, the charge diameter and its length must both be large enough not to restrict the reaction’s movement. Each explosive mixture, therefore, will have its specific minimum diameter and length below which a reaction will begin to slow down. If a reduced reaction rate

occurs, only a low order detonation or just a deflagration will develop. The diameter below which no complete reaction can occur is called the explosive's critical diameter.

The loading actions or effects of the detonation and borehole pressures on the ground surrounding a typical blasthole differ. The detonation pressure is directed primarily axially away from the explosive's initiation point through the charge column's centre where the reaction velocity is greatest. Many authors attribute the shock produced by an explosive's detonation to be the cause that generates ground vibrations (Tadieddin 1982) when considering how the detonation pressure is applied. The proportion of the total energy contained in an explosive's shock front is small, and both slowly reacting explosives and non-detonating black powder can all produce ground vibrations when employed for blasting. Since the borehole pressure during its initial buildup loads the surrounding ground dynamically, it seems much more likely than the detonation pressure to be the primary source for the seismic energy generation. This would be consistent with the literature in which a number of investigators have stated that the amount of explosive energy and the manner in which it is applied rather than the explosive's reaction rate, have the greater influence on ground vibration intensity.

Explosives vary in a number of ways that determine their effectiveness, including not only the detonating rate or velocity, but also other characteristics such as density and heat production. Various explosives are compared by their various characteristics for demolition charges or by their blast pressure and impulses for bombs and shells (TM5-855-1 1986). TNT is used as a standard or reference explosive for equating the effects of various explosives. The free-air equivalent weight of a particular explosive is the weight of the standard explosive TNT required to produce a selected shock wave parameter of magnitude equal to that produced by a unit weight of the explosive in question (TM5-855-1 1986). The equivalent weight of an explosive based on blast pressure or impulse varies at different pressure levels. Generally, an average value of equivalency is used. Average free-air equivalent weights for some commonly used explosives are given in Table 2-7 (TM5-855-1 1986, ISEE 2000).

Table 2-7. Average free-air equivalent weights based on blast pressure and impulse
(data from TM5-855-1 1986 and ISEE 2000)

Explosive	Equivalent weight Pressure	Equivalent weight Impulse	Pressure range, psi
ANFO	0.82	-	1-100
Composition C-4	1.37	1.19	10-100
PETN	1.27	-	5-100
Pentolite	1.42	1.00	5-100
TNT	1.00	1.00	Standard for pressure ranges shown

In free-air, the detonation of an explosive is characterised by a violent release of energy in a gaseous medium, giving rise to a sudden increase in pressure. The pressure disturbance, termed blast wave, is characterised by an almost instantaneous rise from ambient pressure to a peak incident pressure. After the arrival of the shock wave, and after the rise to the peak value, the incident pressure disturbance decays to an ambient value, and is termed as the positive phase duration. This is followed by a negative phase duration, which is longer than the positive phase duration and characterised by a pressure below the ambient pressure at detonation (i.e. suction effect). The negative phase is usually less important in design than the positive phase.

In the case of a confined explosion, the loading from the explosion can be characterised as consisting of two phases. The first phase is the reflected blast loading consisting of an internal high-pressure, short duration reflected wave, and the second loading phase is a quasi-static pressure pulse caused by the overpressure settling to a slowly decaying level. The negative phase does not develop in confined explosions unless an opening is present to allow the overpressure to dissipate swiftly. Studies have also been conducted on airblast transmission through tunnels and ducts (TM5-855-1 1986).

When explosives are detonated underwater, water is displaced and pressurised both by burning and detonation within the water column. Water is somewhat compressible in the near-blast region by extremely elevated pressure due to the explosion. The water column depth and work accomplished by the blasting (due to its confinement) are

significant conditions in determination of the explosion's effects. Besides the compression waves produced by the explosion, other products could include noise, projectiles and gaseous chemical products, which are vented to the water or air. Water is nearly incompressible at standard temperature and pressure and cannot support shear waves. The extreme pressures and temperatures of explosives' detonation and its interaction with water complicate the analysis of underwater explosions (Keevin and Hempen 1997). Cole (1965) suggests that the peak pressure, P_0 in bar, for an underwater explosion can be computed using the following equation:

$$P_0 = 674.2 \left(\frac{W^{1/3}}{R} \right) \quad (2.17)$$

where W is the TNT equivalent weight of the explosive in kg, and R is the distance in m from the explosion source.

In addition to free-air explosions and underwater explosion, explosives can also be detonated at or near the ground surface. When an explosive is detonated at or below the ground surface, the stresses from these explosions are usually greater in magnitude and of much longer duration than corresponding explosions in air, and is commonly called ground shock. Significant enhancement of stresses and ground motions will occur if the explosion occurs deeper in the soil layer. Other visible effects of buried explosions are the formation of crater and ejecta (i.e. debris from the explosion). Soil saturation can have a profound influence on ground shock propagation, and increase the ground shock magnitude significantly.

2.6.3 Effects of charge coupling, cratering and embedment depth

The direction of blast progression may affect both magnitude and frequency of ground vibration. Wiss and Linehan (1978) observed that the amplitude of ground vibration differs significantly at different angles from the direction of blast progression. Due to the Doppler effect, the frequency of seismic waves in the direction of blast progression

might be expected to be higher than that in the opposite direction. Wiss and Linehan (1978) showed that the frequency content of the seismic waves in both directions was the same. Probably the blast parameters used for the study produced very little change in frequency due to the Doppler effect only. Other factors such as depth of charge embedment and soil type had much more influence on the frequency content of seismic waves. Therefore, the small change in frequency was not detectable.

The magnitude of the stress and ground motions will be greatly enhanced as the explosive penetrates more deeply into the soil before it detonates. The concept of an equivalent-effect coupling factor is introduced to account for this effect on the ground shock parameters (TM5-855-1 1986). The coupling factor, f , is defined as the ratio of the ground shock magnitude from a partially buried or a shallow-buried explosion (near surface explosion), to that from a fully buried explosion (contained explosion) in the same medium:

$$f = \frac{(P, V, d, I, a) \text{ near surface}}{(P, V, d, I, a) \text{ contained}} \quad (2.18)$$

where P is the penetration of the explosive charge (inches), V is the velocity at which the explosive penetrates into the medium (fps), d is the diameter of the explosive charge (ft), I is the average impulse transmitted to the structure (ib-sec/in²) and a is the width of the area over which the load is applied (ft).

A single coupling factor, f , is applicable for all ground shock parameters that depend upon the depth of explosion (measured to the centre of the weapon) and the medium in which the detonation occurs, i.e. soil, concrete, or air (TM5-855-1 1986). It is important to note that the coupling factor concept does not indicate the size of an equivalent charge, but is a reduction factor applied to the ground shock computed for a contained explosion to account for the effects of a shallow weapon detonation depth.

Coupling factors are different for explosions in air, soil, and concrete, and depend upon the scaled depth of explosion of the weapon (TM5-855-1 1986). The scaled depth of explosion is defined in TM5-855-1 (1986) as $D/W^{1/3}$, where D is the depth to the centroid of the explosive (ft), and W is the charge weight (lb). The coupling factor for a detonation in air does not vary with weapon position, but has a constant value:

$$f = 0.14 \quad (2.19)$$

For explosions in contact with concrete at the ground surface, the recommended coupling factor, f , is about 0.64, and is considerably higher than explosions in free-air. For soils, experiments undertaken by US Army Corps of Engineers (Ingram 1977) suggest that the coupling factor, f , for explosions in contact with soils is about 0.18. TM5-855-1 (1986) conservatively recommends the use of $f=0.14$ for all contact explosions in air, regardless of the underlying medium. Ingram (1977) has also found that the underlying soil type does not significantly affect the coupling factor, f .

Apart from ground shock enhancements due to the charge coupling, explosions at or below the ground surface can also result in the formation of a crater. A crater is defined as a hole in the ground formed by an explosion (Perret et al. 1963).

Figure 2-19 illustrates the characteristics of a crater, each of which may have some significance in various situations. General references to a crater normally mean the apparent crater, or the visible hole left after the explosion. The true crater, is the hole actually excavated by the explosion, but normally masked by the fallback, i.e., the dirt or debris that falls back into the true crater. If the explosion occurs deep enough to be completely contained below the surface, the true crater will consist of a cavity called a camouflet (Bull and Woodford 1999), shown in Figure 2-20.

The rupture zone is a region immediately surrounding the true crater (or camouflet) in which the earth material remains in place, but has been severely disturbed by the force of the explosion. In turn, the rupture zone is surrounded by a larger region of lesser

Figure 2-19. Half-crater profile taken about a vertical centreline of an explosion
(from TM5-855-1 1986)

disturbance known as the plastic zone. On the surface, the rupture and plastic zones form a region of surface displacement, or upthrust. The formation of an upthrust is often encountered even if the explosion is fully contained in the ground, and a camouflet is formed. A crater is also accompanied by ejecta, i.e., material thrown out of the crater by the explosion. The crater lip is the deposit of ejecta, which completely covers the ground surface beyond the crater edge. Sometimes, pieces of ejecta are thrown beyond the outer edge of the lip and are of sufficient size to present an impact hazard to personnel and structures, and are called fragmentation effects.

2.6.4 Effects of charge shape and direction of blast progression

If a perfectly spherical explosive charge is detonated in a homogeneous, isotropically stressed soil, the only displacement would be radial. The radial displacement of the soil will create compressive radial and tensile compressive strains. As such, shearing of the soil skeleton occurs (Perret and Bass 1975).

Figure 2-20. Variation in crater size and shape with depth of explosion
(from TM5-855-1 1986)

Generally, the explosive is cylindrical in shape, which detonates at one of the ends or at the centre of the charge. The detonation proceeds along the charge axis at the explosive's detonation velocity, V_d . Hence the wavefront is neither spherical nor

cylindrically expanding. Rather, the wavefront expands conically (Roman 1986). The wave propagates spherically only below the charge.

It has also been found that the shear strain in saturated sands is inversely proportional to the detonation velocity, when a long, cylindrical charge is used. A long charge has a length to diameter (L/D) aspect ratio of at least five. The cylindrical charge apparently produces better shearing of wet sands at a lower detonation velocity. However, this would also suggest that the dynamic pressures and particle velocities are reduced. As such, the relationship between propagation velocity, V , and detonation velocity, V_d , should be investigated further (ISEE 2000). Finally, it should be noted that soil is neither homogeneous nor isotropic. As such, numerous other sources of shearing strains will develop. Shear waves will develop at interfaces of dissimilar media and propagate through soil. Most notably, waves from deeper, higher impedance materials will be reflected back into the near surface layers (Roman 1986).

2.6.5 Generation of ground vibration and types of seismic waves developed

When an explosive charge is detonated in a drill hole, very rapid decomposition of the charge takes place, forming gases at very high temperatures and pressures. The shock wave crushes the soil structure up to about five drill-hole radius (Ivanov 1967). High tangential stresses and strains are developed beyond this zone, resulting in radial cracks propagating away from the drill hole. This causes permanent distortion of the soil beyond the initially crushed zone (Atchison 1972, Langefors and Kihlstrom 1978, Du Pont Handbook 1977, Siskind et al. 1973). The intensity of the shock wave attenuates very rapidly as large amounts of energy are utilized in deformation and shearing of soil. At some distance from the hole, the intensity drops to a level such that there will be no permanent deformation. The remaining energy goes directly into the surrounding soil as seismic waves. The waves propagate elastically.

Two types of surface waves usually are produced from conventional explosions, Rayleigh (R) waves and Love (Q) waves. Other types of surface waves include Coupled

(C) waves, Hydrodynamic (H) waves, Stonely waves, and Channel waves (Bath 1979). Although some investigators (Howell 1949, Keller 1955) have reported the presence of coupled and hydrodynamic waves in the seismograms from small explosive shots, these surface waves usually are not detected in typical blasts.

2.7 Prediction and Modelling of Buried Explosions

2.7.1 Reflected parameters due to buried explosions

Pressure amplifications can occur when stress waves impinge on a rigid surface, such as a concrete wall or when they are reflected from stiffer underlying layers (e.g. bedrock), and thus increase the total load experienced by the buried structure. In the case of a buried explosion, the incident stress waves move radially outwards from the centre of the explosion until they come in contact with a rigid surface and are reflected. The reflection reinforces the incident stress waves and amplifies the incident pressure (Figure 2-21). The reflection coefficient or factor is defined as the ratio of incident pressure to the free-field pressure (TM5-855-1 1986). The reflection coefficient is an important parameter for the structural design of buried structures. Transient dynamic loadings usually last very short durations and the reflected pressure is experienced almost instantaneously by the buried structure.

Figure 2-21. Amplification of blast pressure due to buried structure

Experimental study of ground shock commonly involves the measurement of free-field stresses and reflected stresses due to a buried explosion. Compared to free-field ground motion measurements (accelerations, velocities and displacements) during ground shock, measurement of free-field soil pressure is more difficult as there must be good contact between the sensing face of the pressure transducer with the soil. Any air gap or poor contact will inevitably lead to lower than actual soil pressure being measured. Furthermore, the sensors that are used for the measurements have a finite size and mass, and therefore constitute a mini-structure in the free-field (Dongren and Weerheijm 1992). It is quite common to find in the literature, measurements of peak accelerations, velocities and displacements without going into the specifics as to the validity of the measurements. A sensor, embedded in the ground, to measure free-field response is a soil-structure interaction problem. In practice, it is very difficult to position sensors in the ground to measure ground response due to explosives. This usually involves some disturbance of the original ground condition and the degree of difficulty increases rapidly as the sensor depth increases.

2.7.2 Free-field and reflected loads from field explosion test

Hardened design of structures for conventional weapons effects is currently accomplished by following the guidance given in TM5-855-1 (1986) and the Air Force Protective Construction Design Manual (AFPCDM 1989). Both manuals use single degree of freedom (SDOF) models as the primary design tool but allow finite element analysis; however, they do not cite acceptable finite element codes. The first step in design or analysis is to determine the free-field loads. After the free-field loads are known, the structural loads are determined and the structural analysis is performed on a trial section. If deformations exceed the design limits a new section is selected for analysis. The TM and AFPCDM use nearly identical procedures for calculating the free-field loads and structural resistance, but they differ in predictions of loads on structures. The TM uses empirical factors to modify the free-field stress to calculate the structural loads. The AFPCDM provides tabulated values of multipliers for given geometrical shapes and free-field stress to calculate the structural loads.

TM5-855-1 (1986) uses an empirical approach to determine load on structure, i.e. the computed peak free-field stress is multiplied by a reflection coefficient to determine the load on the buried structure. The AFPCDM model is an improvement over the TM5-855-1 (1986) load on structures method and can be used in conjunction with nuclear free-field ground shock predictors to improve structural response predictions.

2.7.3 Current practices for predictions of free-field loads

The free-field predictions for both TM5-855-1 and AFPCDM manuals are based on fits to peak free-field stress, particle velocity, and displacement. The free-field peak stress and peak velocity consist of a linear rise to peak and an exponential decay over the duration of loading as expressed below (Note: the unit on the equations are US Customary as given in the references):

$$P_0 = f \cdot \rho c \cdot 160 \cdot \left(\frac{R}{W^{1/3}} \right)^{-n} \quad (2.20)$$

$$P_0 = \rho \cdot c \cdot V_0 \quad (2.21)$$

where P_0 is peak pressure in psi, f is a coupling factor and is dependent on the scaled depth of the explosion given by $d/W^{1/3}$, d is the depth to the centroid of the explosive charge, ρc is acoustic impedance in psi/fps, c is seismic velocity in fps, R is distance from source in feet, W is charge weight in lb, V_0 is peak particle velocity in fps, and n is an attenuation coefficient.

The propagation or seismic velocity, c , will be routinely required for most dynamic computations. This parameter can be determined using soil tests or obtained from tabulated suggestions such as TM5-855-1 (1986). Table 2-8 gives the propagation velocity for a variety of soil media, and may be used with Equations (2.20) and (2.21) to determine the free-field pressure.

Table 2-8. Soil properties for calculating ground shock parameters
(from TM5-855-1 1986)

S/No	Material description	Seismic velocity, c (fps)	Acoustic impedance, ρc (psi/fps)	Attenuation coefficient, n
1.	Loose dry sands and gravels with low relative density	600	12	3 - 3.25
2.	Sandy loam, loess, dry sands, and backfill	1000	22	2.75
3.	Dense sand with high relative density	1600	44	2.5
4.	Wet sandy clay with air voids (greater than 4%)	1800	48	2.5
5.	Saturated sandy clays and sands with small amount of air voids (less than 1%)	5000	130	2.25 - 2.5
6.	Heavy saturated clays and clay shales	>5000	150-180	1.5

Some researchers have cited that peak velocity comparisons with data are much better than stress comparisons, and attribute the effect to the attenuation coefficient, n . The selected n value matches the velocity attenuation but not the stress attenuation. This has been identified as a flaw with the current free-field predictor (Ghosh 1983). The predicted peak stress and waveforms do not match the data very well. The velocity comparison is better than the stress comparison, but the character of the predicted time history is different from the data.

2.7.4 Current practices for structural analysis

The TM5-855-1 (1986) allows the user to account for compressive membrane enhancement of the ultimate resistance by reducing the load. Some researchers feel that this is not correct and should not be employed (Ghosh 1983). Instead, a resistance based on moment capacity at one-half balanced thrust is suggested for compressive membrane enhancement. More exact determination of the resistance function, considering effects of compressive membrane action coupled with external wall thrust due to loads on the

roof and floor slabs, are beyond the practical scope of SDOF models. It has also been suggested that the TM5-855-1 (1986) prediction of wall response is much lower than the observed response even though TM5-855-1 (1986) loading is much higher than the actual field values. The observed response is predominantly rigid body motion whereas the TM5-855-1 (1986) prediction is for wall deformation only (Slawson 1995).

Soil-structure interaction due to rigid body motion of the structure reduces the observed wall loading, but the reduction for a very stiff (almost nondeforming) wall is not as dramatic as for a more flexible wall (Drake et al. 1987). This is due to the response times associated with wall deformation versus rigid body motion. If only minor cracking was observed in the test structure, then the TM5-855-1 (1986) prediction is probably acceptable. For relatively stiff structures, the TM5-855-1 (1986) method may be acceptable for predicting deformation provided rigid body motion does not play a dominant role in the total response of the wall. Use of the TM5-855-1 (1986) method usually results in a conservative design. The only cases where the TM5-855-1 (1986) method provides a reasonable prediction is when very stiff structures with nearly elastic responses are involved (Slawson 1995).

These conclusions summarise the current practice in predicting the response of shallow-buried structures to conventional weapons effects. Most designs start at this level, and many designs are finalized at this level of sophistication. Both the TM5-855-1 (1986) and SDOF/AFPCDM methods have limitations. The TM5-855-1 (1986) loads on structures are empirical rather than founded on a physical model. Neither methods consider the large spatial gradients in wall loading in the out-of-plane dimension of the structure. Current research also suggests that the free-field prediction model does not reproduce the character of the free-field stress and velocity waveforms, and in some cases the peaks are not predicted correctly. The basic SDOF idealization of the responding wall is not valid for structures that are subject to significant rigid body motion. Supplementing code predictions with finite element analysis can offer better insight to free-field predictions and amplifications due to buried structures.

2.7.5 Shortcomings of analytical methods and use of finite element method and finite difference method for explosion modelling

Modelling field explosion tests have preoccupied researchers for several decades. This section briefly reviews analytical methods used to model buried explosions followed by issues in using numerical methods to model buried explosions.

Prior to the popularity of finite element and finite difference methods, attempts were made to solve explosion problems using analytical methods. Henrych (1979) provided a comprehensive discussion of the mechanisms of explosion and their interaction with the surrounding environment. Soils behave in a complex manner when they are subjected to an explosion. Published literature provides several analytical methods for dealing with soils under a transient dynamic loading. One approach considers soil as discrete 'blocks' of a three-phase medium consisting of solid particles, air and water. The blocks are linked by an elastobrittle linkage between the blocks (Henrych 1979). The solution involves the determination of displacements and stresses in the linkages. Poruchikov (1993) presented a systematic discussion of analytical methods of elastodynamics for use with dynamic problems, using the linear theory of elasticity and continuum mechanics. Both methods are, however, tedious for solutions to be obtained by manual calculations.

If there is a buried structure in the soil media, a dynamic soil-structure interaction occurs between the buried explosion and the buried structure. The physical processes in soil-structure interaction are very complex and often highly non-linear. Some phenomena observed are: wave attenuation, reflection and refraction, interfacial effects such as slippage, debonding and rebonding (Stevens and Krauthammer 1989a). Highly idealized situations enable closed-form solutions to be derived for the response of soil and structure. These situations include linear elastic materials, small deformations and simple geometry. The majority of cases require numerical simulations because the soil-structure responses are highly non-linear and geometry is complicated.

Numerical modelling of buried explosions in soil presents a multi-faceted and often challenging physical problem. Firstly, configuration of the charge in terms of type of explosive, shape, weight and density affects the characteristics of the explosion. Different types of explosive have different energy yield and detonation energy. As mentioned in section 2.6.2, it is customary to normalize charges with reference to TNT and thereby enable the effects of the different type of explosives to be compared in terms of a TNT equivalent charge (TM5-855-1 1986). Secondly, soil conditions such as soil type, density and degree of saturation can significantly affect the manner in which ground shock propagates through the ground (Ingram 1977). Thirdly, there are special difficulties such as large deformations, moving material interfaces, deformable boundaries and free surfaces exist (Stevens and Krauthammer 1989b). These considerations are discussed further in Chapter 5.

Compared to earthquake and vibration source, blasting-induced ground motion consists of short-duration and large-amplitude excitation of high-frequency which may cause damage to nearby surface or subsurface structures (Dowding 1996). In order to simulate the propagation of this shock wave in soil and rock masses, finite element method (FEM), finite difference method (FDM) and meshless method (smooth particle hydrodynamics or SPH) are effective numerical tools. As discussed in Section 2.5.4.3, the mesh size depends on the velocity of ground shock if the corresponding frequency is to be captured in the computation.

Three popular commercial software, LS-DYNA (LSTC 1999), AUTODYN (Century Dynamics 1997) and SHALE (Liu et al. 2003), are widely used for numerical simulations of underground explosion. LS-DYNA and AUTODYN are FEM-based general software which are designed with an explicit nonlinear dynamic relaxation technology, while SHALE is FDM-based general software for hydrodynamics. Because the solid deformation is usually described by Lagrange method and the flow in hydrodynamic regime is described by the Euler method, the interface needs a data exchanger when both Lagrangian and Eulerian processors are coupled together in the same problem. In the implementation, the Eulerian processor (mesh) is used to deal with

regions where large deformations exist, while the Lagrangian processor (mesh) is used to track material interfaces. The Lagrangian–Eulerian coupling technique (ALE) allows the implementation of an efficient algorithm to carry out the finite element analysis.

Several distinctively different approaches have been proposed for modelling dynamic behaviour of soils (Chen and Baladi 1985). Given the highly non-linear behaviour of soils under dynamic loading, a rate-dependent model is usually adopted, as it is well accepted that the strain rate strongly affects the response of most soils. The shear modulus-strain curve can be incorporated in the soil model to define their strain dependency. Several material models are available for reinforced concrete in commercial FE program libraries. Literatures also suggest that a rate-dependent model is preferred especially for near-field dynamic applications involving regular geometries. In buried RC arches, it is preferable to model the RC structure by considering the plain concrete material and steel reinforcements separately, and defining an interaction between them (Stevens and Krauthammer 1989b).

2.8 Research Methodology

Based on the discussions presented above, the following methodology is used for the measurement and modelling of ground response due to dynamic loading:

- The measurement of dynamic soil properties, particularly the shear modulus versus strain response of soil, is required before any attempt can be made to model the ground response subjected to dynamic loadings. The maximum shear modulus, G_{max} is a fundamental parameter for defining the peak point of the shear modulus versus strain curve. Given the shortcomings of conventional laboratory tests, the determination of G_{max} should be based on field seismic tests. The continuous surface wave system (CSWS), presented in Chapter 3, was used to obtain G_{max} at test sites and defines a single point in the shear modulus versus strain curve. Several field tests have been undertaken as part of this research to determine G_{max} with a view to develop the shear modulus versus strain curves at the field test sites. The results from a few selected test sites are presented in Chapter 3.

- Although accurate determination of G_{max} is important to define the peak point in the shear modulus-strain curve, it is also necessary to develop the complete shear modulus-strain curve, as soils exhibit a strain-dependant behaviour. Thus, at locations close to the explosive source, the shear strains can be expected to be much higher, than at locations further away. This shear modulus versus strain response has to be incorporated in the finite element analyses in order to model the explosion wave propagation through the soil medium. Leong and Pan (1998) have used the Hardin and Drnevich (1972) relationship to develop shear modulus versus strain curves for Singapore marine clay. Recently, Leong et al. (2003) also suggested that a reasonable shear modulus versus strain envelope could be obtained using Borden et al.'s equation (1996). Soil samples recovered from selected CSWS test sites were subjected to laboratory investigation and the shear modulus values at a few strain levels were obtained. These shear modulus values can be used in conjunction with the G_{max} values obtained from CSWS and fitted using Borden et al.'s equation (1996) as suggested by Leong et al. (2002). The points used to fit the shear modulus versus strain curve should ideally encompass shear modulus and strain values from the G_{max} region (from CSWS), the G_{min} region (from conventional triaxial tests) and the intermediate region (using resonant column or special triaxial apparatus).
- Field explosion tests were conducted in local residual soils. The aims of the field explosion tests are to study near-field shock wave propagation in residual soils and the interaction of shock waves with buried targets in the near-field. The field tests were planned such that a range of explosive weights was detonated below the ground. The shock wave propagation at various radial distances was monitored using transducers mounted centrally on the buried targets and the In-Situ Dynamic Measurement Unit (IDMU). Each set of instrumentation consisted of a pressure transducer and accelerometers in three orthogonal directions. Past research has shown that in the far-field, or at distances far away from an explosion source, the shock waves can be approximated using one-dimensional wave theory. However, for close-in blasting, the assumption may not be valid and had to be investigated further for Singapore residual soils. The IDMU was selected as it had performed

satisfactorily in previous field explosion tests, and was enhanced to a three-dimensional measurement device in this research. The raw data from the field tests were processed and examined. The processed results enable the scaling relations of the ground shock parameters to be examined, and to determine if the cube-root scaling law applies to the Singapore residual soils. The results also enable the attenuation properties of the residual soils to be examined.

- The processed test results from the field explosion tests were also used to calibrate a numerical model, based on independently obtained soil data from site characterisation works undertaken at the test site. The finite element model was developed using LS-DYNA. LS-DYNA is a comprehensive program as it incorporated an explosion model, several soil models, and has been used in the past to model buried explosion effects (e.g. Johnson and Bammann 1984). The explosive was modelled using a JWL equation of state, as traditionally used in most explosive simulations, and the soil was defined using an elastoplastic soil model incorporating the shear modulus versus strain curve for the test site. The mesh was generated, ensuring the geometry of the mesh and the associated time step did not exceed critical values for the explosion simulation. The boundaries of the finite element mesh were also treated such that the effects of simulated wave reflections did not affect the numerical results. The calibration of the finite element model was undertaken by initially considering shear modulus versus strain curve obtained for the test site. The calibration involves adjusting the shear modulus versus strain curve until a reasonable match was achieved with most of the processed time-history results. In reality, it is not possible to obtain an exact match with all the test results, in view of the recording accuracy and other field errors.
- Buried targets were subsequently introduced in the numerical model, with an attempt to calibrate the numerical model to simulate the amplification effects caused by the buried targets. The reinforced concrete (RC) box targets were defined using an anisotropic brittle damage model available in LS-DYNA, and the steel targets

were defined using solid finite elements. The numerical results and the processed field test results were compared.

- With a calibrated numerical model, it is possible to undertake parametric studies by varying the soil parameters and the geometry of the buried targets. The results from the parametric analyses and the field tests were used to develop design aids and guidelines for free-field stresses and reflected stresses due to buried explosions in soils.

2.9 Conclusions

The characterisation of surface waves is important for any ground response study. But it is also not an easy task as surface waves, when measured using the CSWS can be sensitive to ground motions from nearby dynamic sources. For ground shock studies, obtaining surface wave properties using the CSWS is preferable compared to laboratory investigations as the volume of soil tested is larger and provides a better estimate of the average soil properties. NTU has an ongoing research to establish a database of dynamic properties of Singapore soils. The results from this research could contribute data on dynamic properties of Singapore residual soils, which would be useful for studying other dynamic problems such as earthquakes. There has also been keen interest in development of linear inversion models for interpretation of surface wave data. More sophisticated algorithms have been proposed and efforts have been directed towards the development of a compact computational scheme with faster convergence of solutions. Field explosion testing forms the backbone for any good ground shock research. However, field testing can also be very demanding, particularly instrumentation and data acquisition. The use of additional data monitoring points and backup power and data acquisition systems could safeguard against unforeseen data loss. Field explosion tests can also be used to calibrate finite element models, and enables the blast environment to be parametrically studied in a computer environment. FE analyses are complex and require experience and judgement to understand and analyse the results.

CHAPTER 3

SITE CHARACTERISATION USING THE CONTINUOUS SURFACE WAVE SYSTEM

3.1 Introduction

A continuous surface wave system (CSWS) developed by GDS (GDS Instruments Ltd 1998) is described in this chapter. As mentioned in Chapter 2, the field dispersion curve obtained using CSWS can be inverted using an inversion method to determine the small-strain shear modulus (G_{max}). The initial CSWS field tests were confined to the campus of Nanyang Technological University (NTU), to familiarise with the equipment and to obtain a working knowledge of the GDS CSWS. This was followed by CSWS field tests at various sites of different geological formations. Results of the field tests from soft soil sites and residual soil sites are illustrated. The field test results are interpreted using a finite element inversion method. The linear inversion method will not be discussed here. This chapter concludes with a discussion of the results obtained from the field tests and analyses.

3.2 Surface Wave Sources

Two surface wave sources are popularly used today: impact sources, such as a falling hammer or a surface explosion, and continuous sources such as mass vibrators. Impact sources produce responses in the form of a single elastic impulse, whereas continuous surface sources produce a continuous supply of harmonic impulses. The source type can affect the way in which the field data are acquired and subsequently processed. Impact sources are widely used in America and have typically been processed using spectral analyses of surface waves (SASW) (Stokoe and Nazarian 1985). Vibrator sources have been widely adopted in UK (Butcher and Tam 1994) and Japan (Tokimatsu 1991) for

use with the Continuous Surface Wave System (CSWS) method. For this research, the continuous surface wave technique was used.

The depths to which Rayleigh waves (generated by the surface wave sources) penetrate depend on the Rayleigh wavelengths. In practice, the majority of wave energy is contained within a zone that extends to a depth of approximately one wavelength. The frequency content of an impact source depends on its shape and mass. In general, heavier weights generate signals dominated by lower frequencies and thus penetrate deeper into the ground. In order to survey the ground over a zone of interest, a suite of different impact sources will be required to produce Rayleigh waves encompassing a range of wavelengths. The lack of precise control over the wave frequency generated in impact sources is a disadvantage for field applications.

Electromechanical vibration sources are better suited for field tests as they can be programmed to generate a range of frequencies. However, light vibrators do not provide good quality waveforms at low frequencies (about 7 Hz and below) and thus affect the maximum depth of investigation (Butcher and Tam 1994). This problem can be overcome by selecting a heavier vibrator. However, such vibrators may limit the portability for field usage. A frequency range of 5 Hz to 50 Hz is adequate for most field applications and also enables the vibrator to be kept portable.

3.3 Overview of the Continuous Surface Wave System (CSWS)

3.3.1 Introduction to GDS CSWS

A continuous surface wave system (CSWS) developed by GDS Instruments Ltd (1998) was used for the shear modulus profiling in this study. A schematic set-up of the CSWS is shown in Figure 3-1.

A computer controlled inertial vibrator (mass of 64 kg) applies a regulated and predominantly vertically polarised disturbance to the ground surface. This generates

surface waves with frequencies from 5 Hz to 100 Hz which are detected by a line of geophones (either 2 Hz or 4.5 Hz) collinear with the vibrator. A cooling fan is connected to the vibrator to prevent it from overheating. The frequency of the vibrator and data acquisition is controlled by the PC. Once the vibrator and geophones have been positioned and the details of their layout are entered into the computer program, the vibrator is excited from 5 Hz to 100 Hz in 5 Hz increments, and the resulting shear modulus profile based on the factored wavelength method can be viewed on the computer display. Based on this preliminary survey, sometimes called the coarse run (Addo and Robertson 1992), the user can select a number of frequency ranges and assign suitable frequency increments to provide a more refined shear modulus profile. This stage is important as it enables the user to appraise the quality of the raw data, and select missing frequency blocks to ensure continuity of data in the shear modulus profile. By progressively varying the frequency of disturbance, velocity measurements can be made over a range of depths.

Figure 3-1. Schematic equipment set-up for the GDS continuous surface wave system (CSWS)

Generally, the main interest is in the lower frequency runs, which also dictate the maximum depth of profiling that can be obtained using the CSWS. The signals from the geophones (time-domain) are fed back to the computer, which analyses the phase relationships and computes the velocity of surface waves. The computer decomposes the continuous signals into an equivalent summation of infinite series of harmonics, using Fourier transform (frequency domain). The raw data can be retrieved from the computer for further analyses. Figures 3-2 and 3-3 show a typical test set-up of the CSWS and a close-up view of the vibrator and geophones, respectively.

Figure 3-2. CSWS test data acquisition and control system, and vibrator.

3.3.2 Principle of continuous surface wave

Consider a simple case in which a continuous vibratory source of surface wave is placed on the ground and driven at a known frequency, f . Two geophones, positioned at distance, d , apart will experience a phase difference, Φ (i.e. $\Phi_1 - \Phi_2$, in radians), due to a train of steady state signals that pass between them, as shown in Figure 3-4.

Figure 3-3. Vibrator and geophones spaced at 0.5 m apart

Figure 3-4. Illustration of phase difference, Φ .

If d is less than the wavelength, λ , then by proportion,

$$\lambda = 2\pi d / \Phi \quad (3.1)$$

If d is greater than λ , then

$$\lambda = 2\pi d / (2\pi n + \Phi) \quad (3.2)$$

where n is an integer. For most surface wave applications, Equation 3.2 will hardly be required as the geophones are generally placed at distances typically ranging from 0.5m to 1.0m apart.

The velocity of the Rayleigh wave, V_r , of frequency, f , at the site is given by the familiar relationship,

$$V_r = f\lambda \quad (3.3)$$

A plot of V_r against λ for various frequencies is called the field dispersion curve. According to elastic theory, the velocity of shear wave propagation V_s is related to V_r by

$$V_s = pV_r \quad (3.4)$$

where p is a function of the Poisson's ratio, ν , (see Equation 2.4) and varies from 1.088 and 1.047 for $\nu = 0$ and 0.5, respectively. The shear modulus, G_{max} , can now be related to the shear wave velocity by

$$G_{max} = \rho V_s^2 \quad (3.5)$$

where ρ is the bulk density of the soil. By combining Equations (3.4) and (3.5) above, the following equation is obtained:

$$G_{max} = \rho p^2 V_r^2 \quad (3.6)$$

Equations (3.1) to (3.6) illustrate the principle of using surface wave (Rayleigh wave) to obtain G_{max} .

3.3.3 Obtaining the field dispersion curve from CSWS data file

The field dispersion curve is an important starting point for any inversion to be undertaken, and can be obtained from the CSWS data files. An example of a CSWS data file is shown in Table 3-1. The column data in Table 3-1 provides the following information:

- Frequency – applied by the vibrator.
- Phase angle – as calculated by the least squares best-fit approach (degrees).
- Y-deviation – i.e. the maximum deviation of the geophones from the least square best-fit line (degrees). This can be used to obtain an initial indication of the quality of the raw data.
- Stiffness (MPa) – as calculated using Equations (3.1) to (3.6).
- Depth – as calculated using the factored wavelength approach. For the GDS system, the Rayleigh wavelength, λ , can be taken as $\lambda=3z$, where z is the depth.
- Individual phase angles – for each of the six geophones used in the field test.

From the above information, it is possible to construct field dispersion curve, as the phase velocity (V_r) can be back-calculated using Equation (3.6), and the corresponding Rayleigh wavelength, λ , can be obtained from the depth. Figure 3-5 shows a schematic diagram of the stages involved in the development of a field dispersion curve using the CSWS.

Table 3-1. Typical CSWS data file

wt = 1.9 pois = 0.25 01-18-2000,																
frequency	Phase Angle	Y deviation	Stiffness	Depth	0.5m						2.0m		2.5m		3m	
					Geo1 ph	Geo2 ph	Geo3 ph	Geo4 ph	Geo5 ph	Geo6 ph	Geo4 ph	Geo5 ph	Geo6 ph	Geo5 ph	Geo6 ph	
5	288.7	167.91	0.09		0.42	-153.55	-154.56	-155.07	-154.46	-161.11	-152.18					
6	293.37	166.59	0.12		0.41	-166.83	-170.83	-174.49	-172.91	-182.59	-178.9					
7	193.79	132.55	0.38		0.62	-91.32	-97.4	-101.92	-101.84	-112.49	-112.54					
8	10.17	3.67	180.34		11.8	-81.32	-87.7	-92.04	-93.03	-103.85	-107.02					
9	10.33	2.9	221.28		11.62	102.74	97.64	92.77	90.6	80.69	77.21					
10	12.55	3.26	184.88		9.56	-100.95	-105.13	-110.77	-114.72	-125.72	-131.74					
11	12.7	3.1	218.66		9.45	121.46	118.37	112.46	107.94	97.27	90.59					
12	16.18	3.51	160.11		7.41	-57.56	-60.95	-67.97	-74.93	-87.26	-97.03					
13	16.77	4.23	174.93		7.15	-55.53	-57.51	-65.54	-73.64	-85.62	-95.75					
14	16.14	4.2	219.19		7.44	-55.91	-57.46	-65.35	-73.37	-84.87	-94.34					
15	20.23	6.03	160.04		5.93	39.52	38.87	29	18.33	4.07	-8.29					
16	19.36	5.83	198.83		6.2	53.89	53.09	43.99	34.01	20.27	7.81					
17	125.29	163.97	5.36		0.96	8.31	8.92	-2.17	-14.32	-29.92	-44.46					
18	126.04	163.25	5.94		0.95	117.83	120.02	107.47	94.18	78.33	64.36					
19	129.03	162.42	6.32		0.93	62.13	64	50.31	35.2	17.36	1.54					
20	132.34	161.13	6.65		0.91	-92.2	-90.18	-105.02	-122.1	-142.35	-160.67					
21	131.91	160.96	7.38		0.91	-2.45	0.51	-14.88	-31.92	-51.57	-69.47					
22	133.01	160.27	7.97		0.9	-146.97	-143.35	-159.34	-177.34	162.28	143.72					
23	136.65	159.75	8.25		0.88	55.08	58.66	39.82	18.68	-3.27	-21.8					
24	140.76	157.23	8.47		0.85	45.25	49.59	30.69	7.27	-18.56	-41.85					
25	140.31	156.75	9.25		0.86	-165.03	-159.69	-178.65	158.46	133.21	108.74

Figure 3-5. Schematic diagram showing the steps involved in the determination of a dispersion curve using the CSWS (from Matthews et al. 1996)

3.4 Familiarisation tests using CSWS

This section presents the CSWS field test results and analyses of field data. CSWS field tests were undertaken at several sites. The initial field tests were confined to the campus of Nanyang Technological University (NTU), to familiarise with the equipment and to obtain a working knowledge of the GDS CSWS. More tests were undertaken in various geological formations in Singapore, and are discussed later in this Chapter.

Field tests were undertaken at two different locations in NTU. The first site (site NTU-1) is at the far end of Block N1 (School of Civil and Environmental Engineering), whereas the second site (site NTU-2) is near the Jalan Bahar NTU entrance. A good archive of records is available for site NTU-1, as several borehole investigations have been undertaken at this site as part of past research and in-house practical training program for undergraduates. As for site NTU-2, Cone Penetration Test (CPT) results are available and can be used to estimate the site stratigraphy. Figure 3-6 shows a map of NTU and the locations of the two sites (site NTU-1 and site NTU-2).

Site NTU-1 was bounded by a slope on one side and a concrete footpath on the side, as shown in Figure 3-7. The site is ideal for studying the effects of boundary conditions provided by the slope and concrete footpath on surface wave measurements. A total of 54 CSWS tests were conducted at site NTU-1, as shown in Table 3-2.

Although these test results may be only applicable for a similar residual soil site, the results provided some insights on the following issues:

- Optimal geophone spacing, including the effects of non-uniformly spaced geophones. The geophones were spaced at distances varying from 0.5 m apart to 1.5 m apart, at equal intervals. Further tests were undertaken with geophones spaced at varying distances. For instance, some tests were conducted with geophones spaced at 0.5 m apart equally for the first three geophones, followed by a spacing of 1m apart equally for the remaining three geophones.

Figure 3-6. Map showing the field test locations within NTU

Figure 3-7. View of site NTU-1 with part of the slope and footway shown.

Table 3-2. Summary of CSWS tests conducted at NTU campus

Test number and description	Remarks
Tests 1 to 3: Setting up and familiarisation with CSWS	To understand the various features of GDS software, equipment installation procedure and the output data files.
Tests 4 to 18: Optimal geophone spacing and geophone types.	The 2 Hz and 4.5 Hz geophones were used in the tests. Geophone spacing was varied from 0.5 m, 1.0 m, and 1.5 m apart.
Tests 19 to 25: Effects of geophone array orientation with respect to the vibrator	<p>The geophone array was shifted at various radial angles with respect to the vibrator.</p> <p>The vibrator position was also shifted to the end of the array to study the effects on the CSWS results.</p>
Tests 25 to 30: Repeatability of tests	The tests were repeated at the same location using 4.5 Hz and 2 Hz geophones. The geophones were spaced at 0.5 m, 1.0 m, and 1.5 m apart.
Tests 31 to 38: Disabled geophones in the array	Some of the geophones were disabled before the CSWS tests were undertaken. 4.5 Hz geophones were used for these tests at various spacing.
Tests 39 to 46: Effects of slope	To investigate the effect of a slope on CSWS results. The geophones and vibrator were offset at various distances, parallel to the slope. 4.5 Hz geophones were used for these tests.
Tests 47 to 54: Effects of footway	Similar to tests on effects of slope (Tests 39 to 46). 4.5 Hz geophones were used for these tests.

- Optimal geophone type, 2 Hz or 4.5 Hz. The CSWS comes with 2 types of geophones. The 2 Hz geophones are more sensitive than the 4.5 Hz geophones.
- Repeatability of tests, i.e. if repeat tests yield similar results.
- Usefulness of pre-programmed and non-editable 'template' runs for residual soils. There are three template runs to choose from, varying from soft soils, medium stiff soils, and stiff soils. These template runs were created by GDS.
- Effects of disabled geophones in the array.
- Effects caused by boundary conditions.

Based on the CSWS tests conducted at site NTU-1, the following observations were noted:

- Optimal geophone response was obtained for this residual soil site when the geophones were spaced at 0.5 m apart at equal intervals. Field tests conducted at soft soil sites outside NTU also supported this observation. In determining the optimal geophone spacing, attention should be paid to the measured phase angles from the array of geophones and the spectral response of the geophones. Phase angles that fall naturally into a reasonable straight-line fit indicate well-defined surface waveforms. The online spectral amplitude vs. frequency plot for each geophone should each show a sharp pronounced peak. If this peak is not present, it could suggest the presence of high background noise around the frequency at which the vibrator is operating. Geophones spaced 1m apart or more resulted in phase angles that were usually out of line. As the geophones were brought closer, the phase angles lined up and provided a better straight-line fit. No significant improvement in the phase shift was observed by spacing the geophones closer than 0.5m apart. The advantage of using several geophones is that a best-line fit can be drawn through the phase angle-distance plot, and thus averaging the individual errors (or deviation) in

the field measurement. In cases where the CSWS shear modulus profile appears erratic or unreasonable, the user can decide immediately if additional tests are required.

- The 4.5 Hz geophones were found to produce reasonable results. The 2 Hz geophones were unable to provide the shear modulus profile for a depth deeper than that provided by the 4.5 Hz geophones, suggesting that only higher frequency surface waves were being detected by the geophones. The 2 Hz geophones are very sensitive to background noise, and the lower frequency (long wavelength) signals were not detected. The field test results from the soft soil sites using the 2 Hz geophones supported the earlier observation that background noise (such as vehicular traffic) affects the performance of 2 Hz geophones, and the 2 Hz geophones should be used in a relatively quiescent environment. Field tests should also be undertaken using geophones that are of the same type, i.e. a mixture of 2 Hz and 4.5 Hz geophones should not be used in surface wave tests due to their difference in measurement sensitivity.
- The effect of geophone array at various radial angles (with respect to the vibrator) was found to produce similar results for a given geophone spacing, suggesting that the underlying soil is homogeneous. The depth of CSWS investigation was observed to reduce as the geophones were spaced further apart.
- The CSWS results are repeatable as shown in Figure 3-8. Field tests conducted a few hours apart produce similar CSWS shear modulus profile, hence suggesting that the system and its interaction with the ground are consistent and dependable as a source of raw data.
- The GDS CSWS offers three pre-programmed 'standard' template runs for quick collection of data in typical geological conditions. The usual procedure is to perform a 10 Hz to 100 Hz frequency run, at 5 Hz increments. This 'coarse' frequency run takes about 5 minutes. Based on the results of this test, a smaller frequency block

Figure 3-8. Repeatability of CSWS field tests.

(e.g. 5 Hz to 25 Hz) and frequency increment (e.g. 1 Hz) can be chosen, to refine the test results. The missing frequency blocks can be entered in the GDS software. This procedure was observed to produce a more detailed shear stiffness profile with more data points, than the template runs. This method also allows the user to obtain an intuitive feeling of the applied frequencies and the depth of penetration of surface waves.

- A geophone could be disabled by physically removing the geophone connection or by specifying this condition in the GDS software. The main difference is that in the former, no measurements will be taken from the disabled geophone as no signals

can be transmitted to the system. In the latter, data will still be acquired from the geophone but will be subsequently omitted from the linear regression analyses of the phase angles. The former procedure is particularly helpful if a geophone is consistently out of phase with the rest of the geophones, e.g. due to localised disturbances, poor contact with ground. Reducing the number of geophones reduced the maximum depth of shear stiffness profiling as the monitoring extent (in terms of radial distance) was reduced. For field tests, six geophones should be used.

- The field tests also suggest that boundary conditions (slope and concrete slabs e.g. footpath or rigid inclusions) can affect the estimated shear modulus profiles at the site. Figure 3-9 illustrates typical results obtained from CSWS measurements at various distances from a slope. Although these results may not be applicable at all sites, they do offer some insight on how the surface waves behave when near such features. These conclusions were also (conservatively) extended to areas affected by large root zones. Rayleigh waves produced from a source may reflect off ground surfaces and produce tensile waves (Dowding 1996). Similarly, secondary compression waves are known to develop, when the waves are reflected off boundaries containing higher impedance material. These waves when combined with the incident waves can either reduce (tensile waves) or increase (compression waves) the total energy content of the propagated wave. These in turn affect the strength (or amplitude) of the signals and affect the field results. Given the small distances at which the geophones are placed and transit time of surface waves, it has been suggested that (secondary) tensile and compression waves can affect results of lower frequency runs. There is, however, limited literature on this matter and published works rely to a large extent on classical Snell's law (analogous to optics) for the mathematical formulation and evaluation of secondary wave components (e.g. Achenbach 1973).

Figure 3-9. Shear modulus profiles adjacent to slope at site NTU-1.

With the above observations, CSWS field tests were conducted at several locations using six 4.5 Hz geophones spaced 0.5 m apart, and placed collinear with the vibrator. The vibrator was programmed to propagate surface waves of frequency 10 Hz to 100 Hz at 10 Hz increment initially, so as to establish the average stiffness of the ground. The frequency runs were subsequently varied from 5 Hz to 100 Hz at smaller increments to obtain a more accurate shear modulus profile of the ground. The presence of nearby vibration sources or environmental conditions (e.g. vehicular traffic) are sometimes unavoidable and can limit the maximum depth over which stiffness profile is obtained.

3.5 Finite element analysis of continuous surface wave results

This section provides the formulation for inversion using the finite element method. Published literature (e.g. Clayton et al. 1995) suggested that the finite element method could accurately model the near-field surface vibrations experienced by the underlying soil in a surface wave test. The continuous surface wave test can be modelled as a two-dimensional axisymmetric elastic wave propagation problem.

An initial assumption of the soil stratification (number of soil layers and thickness of each soil layer) and soil properties ($G_{\max}$, ν , and ρ) for each layer is made. To keep the finite element mesh small, non-reflecting boundaries can be adopted at the bottom and side boundaries of the mesh. The governing equation for the problem is the equation of motion given by:

$$\begin{aligned}\rho \frac{\partial^2 u}{\partial t^2} &= (\lambda^* + G) \frac{\partial \Theta}{\partial x} + G \nabla^2 u \\ \rho \frac{\partial^2 v}{\partial t^2} &= (\lambda^* + G) \frac{\partial \Theta}{\partial y} + G \nabla^2 v\end{aligned}\tag{3.7}$$

where $\Theta = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y}$, $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$, $\lambda^* = \frac{E\nu}{(1-2\nu)(1+\nu)}$, u and v are displacements in the x -direction and y -direction, respectively, λ^* is Lamé's parameter, E is elastic modulus, ν is Poisson's ratio, ρ is density of soil mass and t is time.

The finite element analyses were performed using LS-DYNA (LSTC 1999). LS-DYNA uses explicit central-difference scheme to integrate the equations of motion. Equation (3.7) can be expressed as a semi-discrete equation of motion at time, t , as shown below:

$$Ma^t = P^t - F^t + H^t\tag{3.8}$$

In Equation (3.8), M is the diagonal mass matrix, P' accounts for external and body force loads, F' is the stress divergence vector, and H' is the hourglass resistance. The time-step to be used in the finite element analysis is bounded by the highest natural frequency, $\omega_{\max}$, of any individual element in the finite element mesh. The critical time-step size can be decided only after the mesh size is determined. The critical time-step size (LSTC 1999) is obtained using:

$$\Delta t \leq \frac{2}{\omega_{\max}} \quad (3.9)$$

Once the critical time-step size is determined, LS-DYNA automatically compares the input time-step and selects the time-step size that satisfies Equation (3.9).

Wave propagation problems often involve domains which have finite thicknesses, but are laterally extensive. The general approach in numerical simulation is to select a finite element scheme such that the effects of reflected waves do not affect the accuracy of the results. This could involve truncating the remote boundary at a large distance from the zone of interest and then imposing free or fixed boundary conditions, so that the wave reflection effects are negligible. A large finite element mesh will involve high computational cost, storage and time penalties. In the past, several approaches have been proposed to deal with unbounded domains (Khalili et al. 1999). The viscous boundary method was originally proposed by Lysmer and Kuhlemeyer (1969) and is widely used. In the viscous boundary method, the infinite domain is truncated at an arbitrary location and replaced with distributed dampers, which absorb almost all the energy in the waves that impinge on it and minimizes reflections from the boundary. The normal stress, σ_n , and shear stress, τ , of the waves absorbed by a damper are such that:

$$\begin{aligned} \sigma_n &= -c_1 \rho V_p \dot{u}_x \\ \tau &= -c_2 \rho V_s \dot{u}_y \end{aligned} \quad (3.10)$$

where ρ is the density of material, V_p , V_s are the compression and shear wave velocities, respectively, and c_1 , c_2 are relaxation coefficients. The use of $c_1 = 1$ and $c_2 = 0.25$ will result in reasonable absorption of waves at the boundary in the dynamic finite element analyses (LSTC 1999). The transmitting or silent boundary is defined by providing a complete list of boundary segments and associated material properties during initialization of the finite element analysis.

The nature of stress wave propagation in layered systems is complex. The selection of a suitable boundary can also have significant effects on the shear modulus profile from the finite element analysis, especially at high frequencies. Ideally, the selected boundary should be able to reproduce the wave behaviour encountered in the field. When a dynamic source is applied on the ground surface, the stress waves from the dynamic source can be reflected by discontinuities in the ground, such as the ground surface and buried obstructions. Reflections from the ground surface will produce tensile waves that can combine with the incident waves to reduce the shear modulus on the upper portions of the soil deposit (Reinhart 1975, LSTC 1999). Reflections from deeper layers will produce secondary compression waves, which can combine with the incident waves to increase the measured shear modulus. Limited results are available on the effects of discontinuities on Rayleigh wave propagation in CSWS measurements. Some studies have been undertaken on the effects of discontinuities in SASW tests (Sheu et al. 1989). Recently, Gucunski et al. (1996) analysed the results obtained from finite element model incorporating a variety of rigid obstacles and cavities. The results revealed that the fluctuations caused by the obstructions could result in misinterpretation of the shear modulus profile.

To illustrate the application of the finite element method in the interpretation of the CSWS measurements, consider the finite element model shown in Figure 3-10. The finite element model consists of a homogeneous soil layer with $G_{max} = 50$ MPa. The vibrating source in the CSWS field test (typically from 5 Hz to 100 Hz) is applied at the surface node as:

$$\begin{aligned} F &= \hat{F} \sin(\omega t + \phi_0) \\ \omega &= 2\pi f \end{aligned} \tag{3.11}$$

where f is frequency, $\hat{F}$ is maximum variation of the harmonic loading and ϕ_0 is initial phase angle. The vertical displacement versus time histories at geophones 1 and 6 for a 10 Hz frequency run are shown in Figure 3-11. The corresponding phase angle plots from the Fourier transform analysis are shown in Figures 3-12 and 3-13 for geophones 1 and 6, respectively. The gradient of the phase angle versus distance plot (Figure 3-14) can be used to obtain the wavelength for the applied frequency (see Equation 3.1). By repeating this procedure for each applied frequency, a synthetic dispersion curve of wavelength versus phase velocity is obtained (Figure 3-15). Also shown in Figure 3-15 is the effect of the top reflecting boundary compared with the case where the top boundary is non-reflecting. The use of reflecting boundary at the top of the finite element model (i.e. ground surface) results in tensile waves. These tensile waves “decrease” the actual G_{max} values. In practice, the phase velocity would be expected to deviate from the true phase velocity due to the top reflecting boundary. If the factored wavelength method is used to interpret the dispersion curve given in Figure 3-15 and setting $\lambda/z = 3$, the G_{max} profile in Figure 3-16 is obtained.

Figure 3-10. Hypothetical axisymmetric finite element model (single homogeneous soil layer)

Figure 3-11. Vertical displacement versus time curves for geophones 1 and 6 (at 10 Hz)

Figure 3-12. Phase angle versus frequency plot for geophone 1 (0.5 m from source)

Figure 3-13. Phase angle versus frequency plot for geophone 6 (3 m from source)

Figure 3-14. Phase angle versus distance plot (for 10 Hz frequency finite element run)

Figure 3-15. Synthetic dispersion curve for finite element model shown in Figure 3-10

Figure 3-16. Shear modulus profile from factored wavelength method (using $\lambda/z = 3$)

The model shown in Figure 3-10 was extended to investigate the effects of a two-layered soil profile, each 3 m thick, having $G_{max} = 50$ MPa and $G_{max} = 30$ MPa, respectively (Figure 3-17). The synthetic dispersion curve from this analysis is shown in Figure 3-18, and suggests that the weaker soil layer was detected in the finite element analysis at a wavelength of about 8.6m, or a factored wavelength (λ/z) of about $(8.6/3) = 2.9$. In the case where the interface between the soil layers 1 and 2 is inclined at a 10° slope at the central mid-depth of the model (Figure 3-17), the synthetic dispersion curves for the left-hand and right-hand side of the model are shown in Figure 3-19. The presence of a sloping soil profile affects the finite element analysis and correctly detects the softer soil layer at a shallower depth on the right side and deeper on the left side. If the λ/z ratio computed from the finite element model in Figure 3-18 was used, i.e. 2.9, it is possible to determine the depth at which the weaker soil layer was encountered for the left and right sides. The results suggest that the soft soil layers were detected in the analysis at about 2.37 m and 3.24 m for the right and left sides of the model, respectively. Since the factored wavelength method uses an averaging technique, the depth at which the soft soil layer is detected is not always at the centre of the soil profile under consideration. The above illustrations show the versatility of the finite element method in interpreting the CSWS measurements for varying soil profiles.

Figure 3-17. Hypothetical finite element model (two-layered soil profile)

Figure 3-18. Synthetic dispersion curve (two horizontal soil layers)

Figure 3-19. Synthetic dispersion curves from finite element analysis to the right-hand side and left-hand side of Figure 3-17

When applying the finite element method to interpret the CSWS measurements, an estimate of the soil profile and respective G_{max} of the soil layers are needed. The synthetic dispersion curve from the finite element analysis is then compared with the field dispersion curve. By adjusting the soil layer thickness and G_{max} , the match between the synthetic dispersion curve and field dispersion curve is improved. In most cases, the discrete data points from the synthetic dispersion curve and the field dispersion curve do not occur at the same wavelength. The relative error between the field dispersion curve and the synthetic dispersion curve can be obtained by fitting a polynomial to the field dispersion curve, and computing the error to the polynomial. The relative error in phase velocity between the field and synthetic dispersion curves is expressed as a root mean square (RMS) error:

$$RMS = \sqrt{\frac{\sum_{i=1}^n (v_{ti} - v_{mi})^2}{n}} \quad (3.12)$$

where v_{ti} and v_{mi} are the phase velocities of the synthetic and field dispersion curves, respectively, at the i th point and n is the total number of points. A smaller RMS error indicates that the numerical results matched more closely to the measured value.

Several techniques are available to iteratively adjust the soil stratification and soil properties to obtain a better match of the synthetic dispersion curve with the field dispersion curve (Gucunski and Woods 1992, Ganji et al. 1998). For the purpose of the CSWS analysis, the factored wavelength method is used to provide an estimate of the initial soil profile and the respective G_{max} of the soil layers. An iterative procedure was then used to match the synthetic dispersion curve to the field dispersion curve. It will be illustrated below that a good match of the synthetic and field dispersion curves can be obtained within 7 to 12 iterations.

3.6 Field Tests at Residual Soil Sites

The results of field tests undertaken at two residual soil sites are illustrated in this section. The surface wave tests were undertaken in the residual soils of the Bukit Timah Granite and Jurong Formation, and are identified as site BT and site JF in this section. The field tests were carried out using six 4.5 Hz geophones spaced 0.5 m apart and placed collinear with the vibrator. The vibrator was programmed to propagate surface waves of frequency 10 Hz to 100 Hz at 10 Hz increments initially to establish the general stiffness of the ground, and subsequently from 5 Hz to 30 Hz at 1 Hz increments to obtain more data points in the field dispersion curve from the deeper deposits. The CSWS measurements were also undertaken at four orthogonal directions with respect to the vibration source to observe if there are inclined soil layers. No significant difference in the field dispersion curves was observed, thus suggesting that the underlying soil layers were horizontal.

The shear modulus profile for site BT using the factored wavelength method is shown in Figure 3-20. Figure 3-20 shows a linearly increasing shear modulus profile. The depth of 8.5 m is the maximum penetration depth of the CSWS system. The initial stratification for the finite element model was based on Figure 3-20. A change in the gradient of the shear modulus profile was taken as a new layer and was assigned a strata depth corresponding to the depth over which the gradient of the shear modulus profile was constant (Figure 3-20). The initial shear modulus input for the finite element model is taken as the average shear modulus over the assumed depth. The finite element model and the two-dimensional axisymmetric mesh that was used in the analysis are shown in Figures 3-21 and 3-22, respectively. The synthetic dispersion curve from the initial finite element analysis is used to adjust the soil stratification and properties for subsequent analysis.

Figure 3-20. Factored wavelength method shear modulus profile of site BT

Figure 3-21. Finite element model and starting soil profile

Figure 3-22. Axisymmetric finite element mesh

If the phase velocity in the field dispersion curve is higher than the synthetic dispersion curve, the corresponding assumed G_{max} in the finite element model is too low, and would thus have to be increased. Similarly, if the phase velocity in the field dispersion curve is lower than the synthetic dispersion curve, the corresponding assumed G_{max} in the finite element model is too high, and would thus have to be decreased. The adjustment to the shear modulus value is implemented by considering the average difference in shear moduli computed from the synthetic and field dispersion curves over the strata depth:

$$\Delta G_{max} = \sum_{i=1}^n \frac{1}{n} [\rho p^2 (V_{ti}^2 - V_{mi}^2)] \frac{Z_2}{Z_1} \quad (3.13)$$

$$\left. \begin{aligned} G_{new} &= G_{previous} + 0.5 \cdot \Delta G_{max} \text{ if } V_{ti} - V_{mi} < 0 \\ G_{new} &= G_{previous} - 0.5 \cdot \Delta G_{max} \text{ if } V_{ti} - V_{mi} > 0 \end{aligned} \right\} \quad (3.14)$$

where v_{ii} and v_{mi} are the phase velocities from the synthetic and field dispersion curves between depths z_1 and z_2 , respectively, n is the total number of points, p is a function of the Poisson's ratio (ν), ρ is the bulk density of soil, G_{new} is the revised shear modulus for the next finite element iteration and ΔG_{max} is the computed change in shear modulus using Equation 3.13.

Thus, for the first layer in Figure 3-21, Equation 3.13 yields $\Delta G_{max} = 6$ MPa. The shear modulus for this layer should now be updated to $G_{new} = (8 + 0.5 \cdot 6) = 11$ MPa before the next finite element iteration is carried out.

In most instances, only a few points from the synthetic dispersion curve will match with the field dispersion curve, and it will be necessary to adjust the assumed soil layer thickness or increase the number of soil strata until there is a reasonable match between the curves. If the original assumed stratification consisted of more soil layers than required, the convergence will result with similar G_{max} values in some of the soil layers. These soil layers could then be combined into a single stratum. The adjustment to the depth of soil layers can be implemented by comparing the gradient of the synthetic dispersion curve and the field dispersion curve. If the gradients of the synthetic and field dispersion curves are similar, the assumed stratification is correct, and the solution can be achieved by updating the shear moduli of the soil layers. If the gradients are not the same, the thicknesses of the soil layers should be adjusted, starting from the first layer (i.e. the soil layer closest to the ground surface). Once the field and synthetic dispersion curves for the first layer is matched, the next layer is adjusted.

It is important to follow a systematic approach starting from the first layer to the last layer in order to minimize the number of finite element iterations before a solution is obtained. As a guide, it will be adequate to adjust the layer thicknesses by about 10% of their existing thickness for subsequent analysis. Any deviation or convergence of synthetic dispersion curve can be noticed with a 10% change to the layer thickness, and acts as a guide for the next iteration. Once the solution appears to converge, the layer thickness increments can be reduced to 5% of the existing thickness to refine the search

for the final solution. The synthetic and field dispersion curves for site BT at the beginning and the end of seven iterations are shown in Figure 3-23 (b). The starting and final shear modulus profiles for site BT are shown together with the borehole log for site BT in Figure 3-24.

Figure 3-23. Synthetic and field dispersion curves for site BT

The borehole log in Figure 3-24, shows that the site consists of 1 m of very loose fill material, 5 m of loose clayey silt with some traces of laterite, and a loose clayey silt with some quartz gravel beyond 6 m. Soil investigation works for site BT was terminated at about 7.5 m at a SPT N value of 100 in the dense sandy material. There is good correspondence between the soil stratification from finite element analysis and the borehole log.

The shear modulus profile for site JF using the factored wavelength method ($\lambda/z = 3$) is shown in Figure 3-25 (a). Figure 3-25 (a) shows a linearly increasing shear modulus profile until about 12 m depth, when a significant increase in the shear modulus profile was observed. The field dispersion curve and synthetic dispersion curves for site JF are shown in Figure 3-25(b). The initial and final soil profiles at the end of finite element

analysis and a schematic cross-section of two boreholes (BH1 and BH2) at site JF are shown in Figure 3-26.

Figure 3-24. Borehole log and shear modulus profiles of site BT

The borehole log in Figure 3-26 show that the site consists of 5.5 m of very loose clayey silt, followed by 5.3 m to 6.2 m of loose clayey silt with some quartz gravels. Both boreholes were terminated upon encountering granitic material at a depth of about 11m. The final shear modulus profile from finite element analysis shows discrete soil strata at about 1.8 m, 5.8 m and 11.5 m depths. The finite element stratifications at 5.8 m and 11.5 m depths correspond well with the borehole log stratification. The presence of a softer soil layer at the top surface may not have been evident during the soil investigation stage as the soil stratification in the borehole log depends largely on visual observations. Such visual observations may not reflect the soil stiffness profile accurately.

Figure 3-25. Factored wavelength method shear modulus profile and dispersion curves of site JF

Figure 3-26. Borehole log and shear modulus profiles of site JF

The RMS errors versus iteration number for the two sites are shown in Figure 3-27. Figure 3-27 shows that the iteration number generally increases with the number of soil layers. Convergence is obtained between 7 to 12 iterations for the two residual soil sites. The results were considered acceptable when relative error between the field dispersion curve and the synthetic dispersion curve was within 3%. Figure 3-28 presents a flowchart of the finite element approach for inversion of field test data.

Figure 3-27. RMS error versus iteration number

3.7 Comparison of Shear Wave Velocity from CSWS and Cross-Hole Survey

It is useful to compare test results from CSWS with other geophysical methods. The cross-hole seismic survey is regarded as the most reliable of the borehole methods (Kramer 1996) and would serve as a good comparison.

Crosshole seismic surveys (Figure 2-6) involve measurement of the travel time of seismic energy transmitted between two or more boreholes in order to derive information on the elastic properties of the intervening materials. One hole is used to deploy the source whilst the other hole is used to detect the arrival of the seismic

Figure 3-28. Inversion procedure by FE approach

energy. The travel times of the seismic waves are used with the known distance between the shot and receiver boreholes to calculate the compression and shear wave velocities at a given depth.

CSWS field tests were undertaken at a test site where cross-hole data was available and the shear wave velocities were compared. The subsoil within this site consisted of fill and the Bukit Timah Granite. Borehole investigations at the site suggested that the soil strata consisted of a 3.5 m fill underlain by a 3 m black organic clay, followed by marine clay layers and sand, Figure 3-29. The clay and sand layers are part of the Alluvium deposits, extending about 21 m below the fill layer. The Alluvium deposits are underlain by residual soils of the Bukit Timah granite.

Figure 3-29. Comparison of shear wave velocities using CSWS and cross-hole survey

The borehole log for the site is shown alongside with the shear wave velocities obtained using the CSWS and cross-hole method [Figure 3-29(b)]. Figure 3-29(b) shows that the CSWS was able to penetrate to about 8 m depth only. This is generally the maximum

depth typically achievable using the CSWS in very soft soils. A 2 Hz geophone was used at this site in a relatively quiet environment during the CSWS tests. The shear wave velocities suggested the presence of a softer stratum, as a change of shear wave velocities was observed at about 3.5 m deep, which was also observed in the borehole log as a change from the fill to the organic clay.

The field test results in Figure 3-29 showed that a reasonable match was obtained between the shear wave velocity data from the CSWS and the cross-hole survey. This suggests that G_{max} could be estimated reasonably well using either CSWS results or cross-hole data (if available).

3.8 Discussions

Soil stratification from CSWS field tests using the factored wavelength method is arbitrary and introduces a shear modulus profile that increases with depth (Figure 3-20). Field seismic tests using cross-hole measurement and published literature (Veijayaratnam et al. 1993) suggest that the average value of the S-wave velocity, V_s , is relatively constant in each of the soil layer, except where changes in the stratigraphy are encountered. The shear modulus values from the factored wavelength method with $\lambda/z = 3$ were compared with results from FE analyses at the two test sites in Table 3-3. In Table 3-3, the G_{max} value and depth of each soil layer from the finite element results are used as a basis to compare with results from the factored wavelength method.

Table 3-3 suggests that the factored wavelength method with $\lambda/z = 3$ may not be able to adequately recover the shear modulus profile where there are rapid increases in stiffness with depth, such as in residual soils. The average shear modulus values from the factored wavelength method are consistently lower than the shear modulus values obtained from the FE analyses. The average shear moduli from the factored wavelength method vary significantly, ranging from about 30% to 90% of the shear modulus values from FE analyses. This wide variation adds uncertainty in using the factored wavelength method to determine the shear modulus values for the soil layers. The field test results

were also used to investigate the extent to which shear modulus profile using the factored wavelength method could assist with soil stratification and the results compared with the FE analyses in Table 3-4.

Table 3-3. Comparison of G_{max} from finite element analysis and factored wavelength method

Site	Finite element		Factored wavelength method, $\lambda/z = 3$		
	Depth (m)	G_{max} (MPa) (1)	G_{max} (MPa)	Average G_{max} (MPa), (2)	Difference (2)/(1) %
BT	0 to 0.9	14.5	0 to 15.8	7.9	54.4
	0.9 to 5.3	58.0	15.8 to 60.0	37.9	65.3
	5.3 to 9.0	91.0	60.0 to 97.5	78.8	86.6
JF	0 to 1.7	55.5	0 to 34	17.0	30.6
	1.7 to 5.8	135.0	34 to 144.5	89.3	66.2
	5.8 to 11.4	265.0	144.5 to 321.5	233.0	88.0
	11.4 to 16.0	535.0	321.5 to 590.0	455.8	85.2

Table 3-4. Summary of soil stratifications and G_{max} values from finite element analysis and factored wavelength method

Site	Soil strata and G_{max} from finite element analysis		Soil strata and average G_{max} from factored wavelength method, $\lambda/z = 3$	
	Depth (m)	G_{max} (MPa)	Depth (m)	Average G_{max} (MPa)
BT	0 to 0.9	14.5	0 to 0.8	8.0
	0.9 to 5.3	58.0	0.8 to 2.3	25.0
	5.3 to 9.0	91.0	2.3 to 9.0	60.0
JF	0 to 1.7	55.5	0 to 11.8	160.0
	1.7 to 5.8	135.0	11.8 to 16.0	460.1
	5.8 to 11.4	265.0		
	11.4 to 16.0	535.0		

Table 3-4 suggests that the factored wavelength method using $\lambda/z = 3$ may not be able to detect discrete soil strata at the test site unless significant changes in shear modulus are encountered. For test sites BT and JF, the shear modulus of the second soil stratum was about 2.5 times the shear modulus of the first soil stratum. The CSWS field test results for site JF suggests that some soil stratifications may not appear in the shear modulus

profile from the factored wavelength method if there is no appreciable increase in the stiffness of the underlying soil strata. The finite element results for site JF shows a total of four distinct soil layers. However, the factored wavelength method only identified two soil layers. The finite element results also (Figure 3-18) suggests that the finite element method can identify weak soil layers underlying stiffer deposits even if the change in shear modulus value was not substantial (i.e. approximately two times G_{max} of subsequent layer).

The results from Tables 3-3 and 3-4 suggest that even if the shear modulus values are considered acceptable by applying a suitable factor in the factored wavelength method (e.g. $\lambda/z = 3$), the method may not be able to provide the correct soil stratification at the test site. More accurate soil stratifications and shear modulus values can be obtained using the FE approach. It has been suggested by Matthews and Clayton (1993) that the factored wavelength method can be successfully used to recover the shear modulus profile in soft or compressible soils. However, there is limited literature on this point and it is unclear if this is generally applicable for all soft or compressible soils.

Despite the problems above, the factored wavelength method is still useful for preliminary assessment of soil stratification, and provides an idea of the likely shear modulus values and approximate soil stratification at the test site. The shear modulus values for the soil strata also appear to lie within a band generated using the factored wavelength method with $\lambda/z = 2$ and $\lambda/z = 4$. The results suggest that the use of factored wavelength method with $\lambda/z = 3$ is a reasonable compromise for the two residual soil test sites presented in this paper.

The FE analyses for sites BT and JF show a reasonable agreement with the borehole log stratifications. For test site BT, the FE analysis yielded the same number of soil strata as the borehole log. The analysis suggests that the third soil strata comprising of loose clayey silty material commenced at a depth of 5.3 m and extended to a depth of 9.0 m, which was also the maximum depth to which the CSWS measurements were able to penetrate. The results from the analysis are slightly different from the borehole log,

which suggests that the third soil strata commenced at 6.0 m. For test site JF, the FE analysis detected the presence of the granitic rock at a depth of 11.4 m. The borehole log supports this observation, as granitic rock was encountered at an average depth of 11.3 m. Although the borehole log was terminated at 11.7 m, the FE results suggest that the granitic rock is homogeneous and extends to a depth of 16.0 m.

Based on the above discussions, the following conclusions can be drawn:

- Laboratory test results could sometimes be affected by soil sampling and disturbances, and the representativeness of soil sample in relation to the site. G_{max} is thus difficult to measure in the laboratory, but is a fundamental parameter in the shear modulus versus strain curve (Section 2.3.3). The use of field seismic tests, such as the CSWS is attractive as the soil is tested in-situ at the natural stress and saturation state. The CSWS is also an effective tool for determining the field dispersion curve and is obtained using an inherently averaging technique, testing a large zone of soil in the field.
- For dynamic modelling, the complete shear modulus versus strain curve will be required. It may not be always possible to obtain the shear modulus at various strain values. However the guidelines by Kramer (1996) can be applied to develop the shear modulus versus strain curve using G_{max} . The use of the shear modulus versus strain curve in finite element modelling is further discussed in Chapter 5.
- The field dispersion curves from CSWS field tests, when subjected to different inversion approaches will provide different solutions (i.e. different G_{max} and associated stratum thickness). The factored wavelength method and the finite element method have been illustrated in this Chapter. The results show that the factored wavelength method cannot adequately recover the shear modulus profile when there are rapid increases in stiffness with depth (such as in residual soil sites). Subsequent field tests in other soft soil sites in Singapore also suggest the same conclusion, i.e. the soft soil layers are not generally detected unless the change in

stiffness is significant. However, the factored wavelength method can be used as an initial estimate for shear modulus and layer thickness for more sophisticated inversion analysis such as the finite element method. The finite element inversion method was shown to be able to replicate the field dispersion curves reasonably well, and is versatile to handle mixed combinations of soft and stiff soil strata in a given soil deposit. The finite element method is computationally more demanding. However, the number of iterations required for convergence can be reduced if the starting G_{max} and soil stratum thickness were obtained from the factored wavelength method.

CHAPTER 4

FIELD EXPLOSION TESTS IN SINGAPORE GEOLOGY

4.1 Introduction

Field explosion tests were undertaken in Singapore geology to study stress wave propagation characteristics in wet soils and unsaturated soils. Two test sites, Site A and Site B, are described in this chapter. Site A involved four field explosion tests, with a common charge weight of 4 kg PETN in each test. The PETN used had a TNT equivalence of 1.088. The geology of the test site consisted of relatively uniform alluvial deposit. The charges were detonated under four different soil conditions: dry clayey soil, wet clayey soil, dry sand and wet sand. Site B consisted of four clusters, with charges varying from 1 kg to 10 kg PETN. The tests were conducted in residual soils of the Bukit Timah Granite. Tests at Site A and Site B were scaled model tests, and involved a variety of buried targets and instrumentation. The main aim of the tests was to investigate the free-field and reflected stresses resulting from the buried detonations.

4.2 Benefits of small-scale field explosion tests

Explosion engineering problems are commonly studied by means of prototype model testing (Skjeltnor et al. 1977). While such field tests undeniably provide the most realistic modelling to a problem, the preparation and testing often incur large investments in cost, effort and time. Moreover, the results of prototype model tests are heavily dependent on site geology. In most cases, characterization of test sites is seldom performed. It is also difficult, if not impossible to conduct parametric studies using

prototype model testing due to the cost associated with the prototype model tests. Scaled model tests offer a much more cost-effective and faster way to parametrically examine a problem, although some assumptions are required.

There are several important factors to be considered in scaled model tests. Firstly, the development of model materials with properties similar to the prototype is an important consideration. The ability to build models with acceptable tolerance is another concern (Tabatabai 1987). Secondly, the development and proper understanding of the scaling relationships are essential. A properly scaled model will enable the observed response on the model to be extrapolated to predict the response of the prototype structure. Thirdly, the development of instrumentation methods and devices for the measurement of response parameters such as soil pressure and acceleration on the scaled model is important to ensure that the instrumentation methods and devices do not distort the stress field.

Complete adherence to the scaling relationships developed on the basis of the laws of similitude may require some form of dead load compensation to properly account for the effect of gravity stresses. However, researchers suggest that the effects of gravity on the response of buried structures can be ignored since gravity stresses for buried structures subjected to blast loading are much lower than the blast-induced stresses (Kutter et al. 1988, Baird 1985). Hader (1983) presented a summary of extensive investigations concerning the local effects of bare and cased explosives on reinforced concrete walls. The investigation included scaled model tests involving charges ranging from 6.8 kg TNT to 10 kg TNT. The results were compared with test results from published literature involving full-scale tests of up to 227 kg TNT. He concluded that scaled model test results scaled reasonably well with the full-scale test results without dead load compensation.

In view of the above discussions, scaled model tests enable a finer characterization of the test site and a larger number of tests to be undertaken to provide additional parameters that were not possible to be conclusively studied in prototype model tests.

Thus scaled model tests enable a more thorough understanding of the ground shock problem. The benefits of scaled model test may be considered to outweigh the minor assumptions that may be required to scale the scaled model test results to the prototype condition. In this research, scaled model tests were performed.

4.3 Field explosion tests at Site A

4.3.1 Test set-up and objectives

Shock wave propagation in soils and their interaction with buried structures were investigated by undertaking field explosion tests in soils using three different structures: a reinforced concrete box structure (RC structure), the in-situ dynamic measurement unit (IDMU) and an aluminium box. The charge was placed in a pre-drilled borehole and the targets were placed in line with the explosion source (Section 4.3.3). The instrumentation consisted of a pressure transducer placed in contact with the soil in the radial direction and an accelerometer mounted centrally behind the pressure transducer (Section 4.3.4). The pressure and accelerometer were mounted together to enable recovery of the free-field pressure using a one-dimensional plane wave theory (see Section 6.2.3). The dimensions of the buried structures and the test layout are shown in Figure 4-1(a).

The objectives of the field explosion tests at site A were: First, to investigate the influence of two soil types (i.e. sand and clay) on free-field stresses and motions when subjected to buried explosion loadings. Second, to investigate the effects of degree of saturation on the two soil types used. The sand and clay were artificially wetted and subjected to buried explosion loadings to determine the effects of moisture on free-field stresses and motions. Third, to investigate the stress wave amplification effects caused by interference of buried targets with free-field wave propagation. All the buried targets used were hollow and had a square face area. The buried targets were used to investigate the effect of size of target face area on stress wave amplification. Since two

Figure 4-1. Layout of field explosion test and isometric view of structures used in test

different soil types with varying degrees of saturation were used, the tests were also used to investigate if the soil types and degrees of saturation affected the magnitude of stress wave amplification caused by buried targets.

The IDMU is a pneumatic system with the accelerometer and pressure transducer attached to the fixed end (Figure 4-1c). The IDMU is lowered into a pre-drilled and cased borehole, and is pneumatically extended until it is in firm contact with the ground.

The mass of the three structures together with the sensors are approximately 2257 kg, 1.33 kg and 1 kg, respectively.

The IDMU had a target face area of 64 mm x 64 mm. Leong et al. (1996) reported that the signals recorded by accelerometers mounted radially on IDMUs and accelerometers buried within soil at the same radial distance were almost identical. The presence of the open borehole in which the IDMU was used did not appear to modify the motion field significantly (Leong et al. 1996). Soil pressure transducers were not used in Leong et al.'s (1996) field study. The 90 mm x 90 mm aluminum target face area was double the IDMU target face area, and was selected to investigate if there would be any change in acceleration and pressure signals due to a marginal increase in the target face area. The 1 m x 1 m reinforced concrete target face area was selected to study the reflection behaviour of a rigid structure and the concrete failure mechanism due to near-field explosion loadings.

4.3.2 Test site characterisation

The in-situ soil found at the test site consisted of 7% sand and 93% clay. The soil is classified as clay of high plasticity (CH) under the Unified Soil Classification System (ASTM 1997). Beach sand from a nearby shore, consisting of 89% sand and 11% silt were also used in some of the tests. In these tests, the test pit was backfilled with beach sand instead of the in-situ soils (see Section 4.3.3 for details). The sand is classified as silty sand, poorly graded (SM-P) under the Unified Soil Classification System. Tests were conducted under wet and dry soil conditions designated as wet soil and dry soil tests for the in-situ soil, and wet sand and dry sand tests for the beach sand. Soil samples were also recovered at a depth of 1 m below the ground surface for cyclic triaxial tests to determine the shear modulus versus strain behaviour (Anand et al. 2001). Details of the soils' degree of saturation, moisture content and bulk density are shown in Table 4-1.

Table 4-1. Properties of soils used at test site A

	Degree of Saturation (%)	Moisture content (%)	Bulk density (kg/m ³)
Wet soil	99.1	30.2	1900
Dry soil	32.3	17.2	1650
Wet sand	99.3	29.3	1970
Dry sand	25.6	12.7	1780

4.3.3 Test site preparation, target placement and charge detonation

At the test site, a pit 3.4 m deep and 10 m by 10 m at the bottom was first created with side slopes of 1:1, Figure 4-1(d). The soil is then backfilled back into the pit in several lifts. After each lift, the soil was compacted to ensure uniformity. The RC structure was placed when the backfill reached the designated level. Soil was carefully compacted around the structure. The IDMU and aluminium box was not placed using this procedure as the compaction and backfilling process may shift their positions. Instead when the pit has been completely backfilled, boreholes were drilled to the required depths. For the IDMU, the borehole was left open and partially cased with mild steel casing to prevent borehole collapse during the test as recommended by Leong et al. (1996). For the aluminium box, it was positioned at the base of the borehole and soil was backfilled into the borehole. During the backfilling of the borehole, the soil was compacted to the same density as previously.

The dry soil test was performed in the natural saturation state of the soil (i.e. degree of saturation of 32.3%). Soil samples were recovered at 1 m depth prior to the tests. For the wet soil test, the soil was inundated with water after the pit was completely backfilled, such that the degree of saturation of the soil is close to 100%, i.e. volume of air is less than 1% (Table 4-1). The installation of the IDMU and aluminium boxes for the wet soil tests were done after the soil was inundated with water and became

uniformly wet. This was to minimize any damage to the sensors and cables that might occur during the inundation of water.

The explosive, a 4-kg PETN charge, was moulded into a sphere. The centroid of the placed charge was 1 m below the soil surface. The pressure transducers and accelerometers, located centrally on the target faces of the structures, were at the same level as the centroid of the explosive and arranged in radial lines away from the charge. Three radial distances of 1 m, 1.5 m and 3.3 m were used in the test.

4.3.4 Instrumentation system

The accelerometers used were piezoelectric PCB Series 353 quartz-shear ICP accelerometers with a range of ± 500 -g peak. These accelerometers have essentially a flat amplitude response up to 2000 Hz. The pressure transducers used were Kulite LQ-080U series of soil stress gauges with ranges of 0-1000 psi and 0-5000 psi. The acceleration and pressure were sampled at 100 MHz using a PSO 9080 data acquisition system. The recorder permitted 32 channels of signals to be recorded during each test. The sampling rate for the data acquisition was set to 200 kHz with a recording duration of about 2 s. BNC coaxial cables and 4-core shielded cables were used to transmit the signals from the accelerometers and pressure transducers, respectively, to the data acquisition system.

4.4 Typical Site A test results

Not all the recorded signals are of high quality. The signals for the RC box structure at 1.0 m radial distance, and aluminium structure at 1.5 m radial distance had to be discarded for the wet soil and wet sand tests as the peak pressure and peak acceleration could not be captured. The signals at radial distances of 3.3 m for all the tests were of higher quality than those at radial distances of 1.0 m. The IDMU showed better overall performance at all the three radial distances compared to the buried targets.

Typical field test results from the dry soil, wet soil, dry sand and wet sand tests are shown in Figures 4-2 to 4-6. Figure 4-2 shows the reflected pressure measured by the RC box structure and the aluminum structure. The RC structure appears to attract a higher reflected pressure than the aluminum structure. The results similarly suggest higher values for radial acceleration for the RC structure compared to the aluminum structure (Figure 4-3). Figure 4-4 compares IDMU pressures obtained from the wet sand test and dry sand test, and shows that the degree of saturation has a profound effect on the incident pressure. For the same radial distance of 1 m, the magnitude of incident pressure in the wet sand is about ten times greater than that in the dry sand.

The field test results can be used to determine the seismic velocity of the soil and the attenuation coefficient. The wave arrival time shown in the time-history can be used to determine the average propagation velocity (or the seismic velocity) of the compression waves in the soil medium. In Figure 4-5(a), a linear regression technique is applied to the field test data from the wet soil test and wet sand test. The results suggest that the seismic velocity in the wet soil and wet sand is similar, estimated to be about 1380 m/s. The results also suggest that the seismic velocity in the dry soil and dry sand are slightly different, Figure 4-5(b). The seismic velocity for the dry sand was found to be about 205 m/s while the seismic velocity for the dry soil was found to be about 190 m/s.

The attenuation coefficient of the soil is represented by the slope of the regression line in a semi-log plot of free-field pressure versus distance (Ingram 1977). Figure 4-6(a) shows that the attenuation coefficient, n , for the wet soil and wet sand is about 2.53. Figure 4-6(b) shows that the attenuation coefficient for dry sand is 3.05, higher than the attenuation coefficient for the dry soil, that is 2.71. The soil properties, attenuation coefficient, n , and the seismic velocity, c , can be used to estimate the free-field parameters using TM5-855-1. This is discussed further in Chapter 6.

(a) RC box structure

(b) Aluminium structure

Figure 4-2. Reflected pressure for RC box structure and Aluminium structure at 3.3m radial distance

(a) RC box structure

(b) Aluminium structure

Figure 4-3. Acceleration for RC box structure and Aluminium structure at 3.3m radial distance

(a) Wet sand

(b) Dry sand

Figure 4-4 IDMU results on wet and dry sands at 1 m radial distance

(a) Wet soil and wet sand

(b) Dry soil and dry sand

Figure 4-5 Seismic velocities from wet and dry soil tests

(a) Wet soil and wet sand

(a) Dry soil and dry sand

Figure 4-6. Attenuation coefficients from wet and dry soil tests

4.5 Lessons learnt from field explosion tests at Site A

Results from the field explosion tests at Site A were encouraging. However, a number of shortcomings associated with the field tests required to be addressed. Firstly, the shear wave velocities of the soils were not available. The pressure and acceleration measurement points were positioned at distances ($R/W^{1/3} \leq 2.02 \text{ m/kg}^{1/3}$) close to the explosion source. This scaled range was insufficient to detect the separation of the compression waves from the slower-moving shear waves and Rayleigh waves (Dowding 1996). If pressure and acceleration measurements were available at larger scaled ranges, the slower moving shear waves and Rayleigh waves could have been detected as a delayed peak in the pressure and acceleration time-histories. The shear wave velocity can provide an estimate of the G_{max} of the soil. The GDS CSWS was not available during these tests. Secondly, there were insufficient data for reliable conclusions to be drawn on the free-field pressures and the reflection coefficients. In some instances, poor contact between the soil and the pressure transducers led to very low stresses being measured. The attenuation coefficients and seismic velocity were determined from very limited test data. It was difficult to ascertain the relationship of attenuation of pressure and acceleration with distance. Thirdly, there was also a need to protect the ground from rainfall, a common occurrence in Singapore. Soil saturation after a period of rainfall can alter the in-situ soil conditions, and thus affect the predicted measurements and instrumentation settings. Some of the dry soil tests had to be delayed due to rainfall. Fourthly, the embedment depth of the charge in the wet soil was inadequate for full containment of the explosion and resulted in the formation of craters upon detonation. Fifthly, the use of a common charge weight of 4 kg PETN was not adequate for extrapolation of results to other charge weights. Tests with different charge weights would also enable scaling relationships to be examined. Finally, the instrumentation system and data requirement had to be reviewed to determine the optimal placement of the sensors and their survivability during the test. Almost half the coaxial cables at 1 m radial distance from the charge were damaged during the tests. The data records suggested that the coaxial cables had sheared due to ground movements before the data could be conveyed to the data acquisition system.

4.6 Field explosion tests at Site B

4.6.1 Objectives of field explosion test

The field explosion tests at Site B were designed based on the experiences drawn from the tests at Site A. The test program was developed to address some of the shortcomings and data gaps that were identified in the field tests at Site A. The field explosion tests at Site B were conducted in June 2002 in the Mandai Area, Singapore. The test area geometry, site characterization, targets and charge weights used, accelerometer and pressure transducer locations, were designed to study the following:

- a. Dynamic soil pressures and their attenuation with distance.
- b. Subsurface ground motions and their attenuation with distance.
- c. Three-dimensional wave propagation.
- d. Reflection characteristics of buried targets.
- e. Scaling relationships with variation in charge weights.

Section 4.7.2 will discuss the site selection, and the test team organization structure. Section 4.7.3 will cover the site characterization works done at Site B. Section 4.7.4 covers the field explosion test configuration and monitoring points. Section 4.7.5 describes the site works to implement the field test program and safety considerations during the test. Section 4.7.6. outlines the improvements to the IDMU. Section 4.7.7 describes the design of the buried targets used in the tests. Section 4.7.8 provides details of charge preparation, placement and detonation. Section 4.7.9 provides details of the instrumentation system used to capture the field test data. Selected field test results are shown in Section 4.8.

4.6.2 Test site selection and test team

As mentioned in Chapter 2, about two-thirds of the land area of Singapore is covered by residual soils. In these deposits, the ground water table is deep with the result that the

soils are usually in an unsaturated condition. There are few published literature on the behaviour of residual soils when subjected to buried explosive loading. One reported field explosion test in residual soil discussed the behaviour of piled foundations when subjected to buried blast loading (Leong and Pan 1993). The test was undertaken in the Mandai area. Another investigation in 1997 reported the behaviour of granitic rock when subjected to a buried blast loading in rock (Chong and Zhou 1999). Lee et al. (1999) obtained instrumented measurements of soil pressure and acceleration at a few elevations in a 1 m high earth berm constructed directly above a granitic rock surface. The 1 m high earth berm was constructed using residual soils from the test site, and the instrumentation monitoring points were at the rock-soil interface, at mid-depth of the earth berm and at the surface of the earth berm. The field explosion test was aimed at studying blast propagation from the rock-soil interface and the blast propagation in the residual soils (Lee et al. 1999). Field tests in residual soils are desirable due to their extensive occurrence in Singapore geology.

Other considerations for the test site were:

- a. Geology: A deep and uniform deposit of residual soils would be desirable. In field explosion tests, the presence of mixed geological media can lead to blast wave reflections when a buried charge is detonated. Wave reflections from underlying stiff or soft soil deposits can reinforce or reduce the incident waves and thus increase or reduce the free-field stresses and motions. This makes interpretation of the test results difficult. A deep and uniform deposit of residual soils would also make the site characterisation task easier. The site characterisation and field test results can be used to calibrate a numerical model. The field tests were designed such that the explosion will be contained within the ground. Since the guidelines (TM5-855-1 1986) for determination of containment depth are for a particular soil type only, it would be desirable to perform the tests in a deep and uniform deposit of residual soils. The containment of explosions also results in maximum free-field

stresses and motions in soils and loss of energy due to ejecta and cratering are avoided.

- b. Timing and schedule: The tests should preferably be conducted during the drier periods of the year, i.e. between May and July. This will minimise interruptions due to inclement weather. Dry weather conditions also allow tests to be set up more readily. Furthermore, these tests are to be conducted in unsaturated soil conditions.
- c. Site topography: A road access or temporary access to the site is necessary for movement of personnel and equipment to the site as well as for safety reasons.
- d. Area of site: Sufficient safety stand-off distance to allow large charges of up to 10 kg PETN (i.e. 10.88 kg TNT) to be detonated below the ground, without causing injuries to people or damage to nearby properties during the explosion. The safety distance (in meters) can be computed using an inhabited building distance (IBD) of $22W^{1/3}$, where W is the charge weight in kg TNT (ESTC 1990). For a 10 kg PETN charge, the minimum safety distance is 48.6 m, or approximately 50 m. The ESTC formulation is conservative as the IBD is based on free-air blast overpressure from a charge detonating at the ground surface.

Based on the above considerations, a test site in Mandai was found suitable for the field explosion tests.

The conduct of a field explosion test involves strict adherence to safety and regulations and a team of trained personnel are required to ensure that important aspects of the field explosion tests are properly managed. For this, the author is grateful for the support of DSTA. The organisation of the test team is shown in Figure 4-7. The field explosion test was supervised by a Test Commander (the author). Four ICs (In-Charge) were also

assigned to oversee and manage tasks in four important areas, namely construction, ordnance, safety, and instrumentation. The author was assisted by DSTA on the ordnance matters and by SembCorp Engineers and Constructors Pte Ltd on safety matters.

The construction works included site clearance, soil drilling, site office provision and site reinstatement after the tests. The ordnance works included collection of explosive charge, preparation of charge, installation and detonation of the charge during the field tests. Safety works include the provision of telecommunication devices, manning of roadblocks, and ensuring the site was safe for detonation. Instrumentation is a crucial part of any field explosion tests. The instrumentation works included calibration of sensors and other devices for capturing data, laying of cables and data recording.

Figure 4-7. Test team organisation structure

4.6.3 Site characterisation

The Bukit Timah Granite has been extensively investigated in the feasibility study of underground space utilization (Zhao and Choa 1993). Field and laboratory investigations were carried out in several phases, including geological mapping of

granite outcrops, surface geophysical surveys using seismic refraction and electrical resistivity methods, deep borehole drilling, and comprehensive laboratory tests. There were no past records of field or laboratory investigations at Site B. However, borehole logs and surface geophysical data at radial distances of 70 m and 85 m from Site B were available. The borehole log and surface geophysical information are shown in Figure 4-8.

Figure 4-8 – Borehole logs at 70 m and 85 m radial distances from Site B.

Figure 4-8 suggests that the residual soil deposit in the vicinity of the test site varies from 19 m to 28 m. The residual soil is described consistently in the borehole logs as yellowish brown clayey silt with sand for the shallow deposits. The borehole log data agrees with previous studies in the Bukit Timah Granite (PWD 1976). Three distinct zones have been observed in the residual soil zone: the upper, intermediate and lower zones. The upper zone, about 6m thick, consists of reddish-brown sandy silty clay wherein laterisation has taken place. The laterisation process results in the formation of reddish-brown ferruginous soil of clayey texture, and which also has a concretionary appearance (Thum and Yong 1987). The intermediate zone consists of 10 m to 25 m of yellowish-brown sandy clayey silt. The lower zone is greyish to whitish-grey sandy silt or silty sand. This layer may comprise of soil from weathering zone V (Thum and Yong 1987).

A continuous surface wave system (CSWS) was used to determine the shear modulus profile at Site B. CSW tests were undertaken at eight different locations within Site B to determine the stratification of the test area. The site vegetation consisted of thick undergrowth as shown in Figure 4-9.

Figure 4-9. CSWS set-up at location 1, Site B.

For the CSW tests, the vegetation was manually removed using spades and changkol and the ground surface was then levelled. The ground elevations of the levelled area was then obtained using a Sokia SDL30 digital leveling instrument. Given the undulating terrain of the site, it is important to determine the ground elevation at the various CSW test locations so that the shear modulus profile of the site can be compared. The initial CSW tests provided a good idea of the underlying soil at the test site. The initial shear modulus profile was obtained using the factored wavelength method. The initial test observations are summarized in Table 4-2. The shear modulus profiles from the CSW tests (i.e. using a factored wavelength method, $\lambda/z = 3$) for the eight test locations are shown in Figure 4-10.

Table 4-2. Observations during CSW tests at Site B

Test Location	Test description	Test observations (ground level at 0 m)
1	<div>4.5 Hz geophones spaced at 0.5 m centres. All tests performed in 4 orthogonal direction, with vibrator position fixed.</div> <div> </div>	Change in G_{max} at -1.5 m and -16 m
2		Change in G_{max} at -1.0 m and -15.5 m
3		Change in G_{max} at -1.1 m and -15 m
4		Change in G_{max} at -1.2 m and -15.4 m
5		Change in G_{max} at -1.9 m and -15.4 m
6		Change in G_{max} at -0.8 m and -16 m
7		Change in G_{max} at -1.7 m and -15.6 m
8		Change in G_{max} at -1.2 m and -15.8 m

Figure 4-10. Shear modulus profile from CSW tests (factored wavelength method)

The CSW tests at Site B consistently showed change in stiffness at average depths of 1.3 m and 15.5 m. The observed G_{max} values varied from 45 MPa at the ground surface to 27 MPa at about 1.3 m depth. The shear modulus profile was observed to increase gradually until a depth of about 16 m where the shear modulus was observed to change significantly, until about 18 m depth, the maximum depth of penetration of the CSW tests. The observed G_{max} at 18 m depth was about 170 MPa. The observed G_{max} values suggested that the site was underlain by about 16 m of residual soils, followed by a stiffer deposit of residual soils. This observation concurred with known borehole log data in the vicinity (e.g. Figure 4-8) and published literature in this deposit (e.g. Zhou et al. 1993).

In most residual soils, the repeated wetting and drying of soils due to the tropical environment advances the weathering process and can be observed as a stiff crust formation near the ground surface. Borden et al. (1996) considered the weathering profile of igneous rocks and found that the upper zone of residual soils was completely weathered with well-developed pedologic horizons. The weathering is most advanced at the ground surface and decreases with increasing depth. The initial change in shear modulus stiffness suggested by the CSW results could thus be explained on the basis of advanced weathering resulting in the formation of a denser material (stiff crust) near the ground surface, which was also detected in the CSW field tests (Figure 4-10). The CSW field test results can thus be better interpreted with an understanding of the site geology.

The field dispersion curve from the CSW tests was inverted using a finite element method. Since the field dispersion curves at the eight CSW test locations (Table 4-2) did not vary significantly, an average field dispersion curve was used. The shear modulus profiles obtained using the factored wavelength method and the finite element method are shown in Figure 4-11.

Figure 4-11 – Average shear modulus profile at Site B.

The finite element inversion method (Figure 4-11) also detected the presence of a thin layer of stiff deposit near the ground surface that extended to a depth of 1m. The observed G_{max} for this layer was about 37 MPa. The shear modulus profile was observed to reduce slightly below the 1 m depth, and remained constant with a G_{max} of 31 MPa from a depth of 1 m to 9 m. At 9 m, the G_{max} increased from 31 MPa to 46.5 MPa. The shear modulus remained constant from a depth of 9 m to 16.5 m. At 16.5 m, the G_{max} increased significantly from 46.5 MPa to 348 MPa, suggesting that a hard stratum exists at about 16.5 m depth.

The soil at the test site was known to have a bulk density, ρ , of 1650 kg/m³ and Poisson's ratio, ν , of 0.3. Given these soil properties, the compressive wave velocity, V_p , for the hard stratum at about 16.5 m depth can be computed using Equation 2.2 to be 740 m/s. The V_p computed using the finite element method is significantly greater than the V_p computed on the basis of the factored wavelength method, i.e. 518 m/s. The compressive wave velocity computed from the finite element results appears to be more reasonable when considering the reported V_p values in Figure 4-10. As mentioned in Chapter 3 (Section 3.6), the factored wavelength method may not be able to adequately recover the shear modulus profile where there are rapid increases in stiffness with depth, such as in residual soils. The CSW test results should be used in conjunction with finite element analysis and other field information to obtain a more accurate characterisation of the test site.

The first 1.5 m depth of the soil at Site B consisted of heavy root growth due to the thick vegetation, and had to be excavated as part of the preparation works for the field explosion tests. During the field explosion tests, soil samples were recovered for laboratory soil testing. The soil samples were obtained from Site B with the aid of a mechanical excavator during the field explosion tests. The excavator was fitted with an extended arm, and was able to excavate soil up to a depth of 4m. Block soil samples measuring about 0.5 m cubes were carved out. The soil samples were extracted at depths of 2 m, 3 m and 4 m. The soil samples were trimmed and quickly wrapped in cling wrap and protective foil and sealed to prevent loss of moisture from the soil sample. The soil samples were transported and kept in a moist room in the laboratory until they were ready for soil testing. Further laboratory tests were also conducted on residual soil samples extracted from the test site. The typical grain size distribution curve of the soil is presented in Figure 4-12

Figure 4-12 suggests that the residual soils at Site B is coarse to medium sand with traces of silt. The Coefficient of Uniformity and Coefficient of Curvature of the soil were about 5.4 and 1.45, respectively, and the average D_{50} of the soil was about 2.2 mm. The soil may be classified as poorly graded sand with little or no fines, designated as SP

under the Unified Soil Classification System. The bulk density of the soil was 1650 kg/m³ and the moisture content was about 19%. The Atterberg’s limits, density and moisture content are shown in Figure 4-12.

Figure 4-12 – Grain size distribution of residual soil at Site B.

4.6.4 Field explosion test configuration

The free-field and reflected stresses due to buried explosions in the residual soil were investigated by placing pressure and acceleration transducers at various radial distances from the buried explosion source. The test configuration consisted of three clusters, with buried charges of 1 kg PETN, 4 kg PETN and 10 kg PETN at the centre of each cluster, and measurement points at various radial distances from the charge as shown in Figure

4-13. The three clusters were spaced such that the effects of underground explosion from adjacent clusters did not significantly affect the other subsequent explosion tests.

The explosion tests were performed in sequence, with the 1 kg PETN charge detonated first, followed by the 4 kg PETN charge and the 10 kg PETN charge. A fourth test with a charge weight of 6 kg PETN was also detonated. The charge was positioned such that selected measurement from the 4 kg PETN and 10 kg PETN clusters could be obtained to fill in the gaps of data from the previous explosion tests. The test configuration for the 6 kg PETN charge test is shown in Figure 4-14.

Figure 4-13. Field explosion test configuration for 1 kg, 4 kg and 10 kg PETN charge

Figure 4-14. Field explosion test configuration for 6 kg PETN charge

The centroid of the 1 kg PETN charge was 2 m below the ground surface. For the 4 kg, 6 kg and 10 kg PETN charges, the centroid of the charge was 2.5 m below the ground surface (Figure 4-15). The tests were designed such that the depth of charge burial would enable full containment of the explosion. Since no local data on charge containment depth in Singapore residual soils were available, charge containment depth values estimated by TM5-855-1 (1986) were used. TM5-855-1 (1986) uses the explosive weight, depth of detonation and type of soil to determine if a crater will be caused by the detonation. Table 4-3 summarises the computed containment depth values using TM5-855-1 (1986).

Figure 4-15. Schematic sectional elevation view of field explosion test (not to scale)

Table 4-3. Depth of burial for containment of charge

Test Number	Charge weight (PETN)	Containment depth (m) based on TM5-855-1 (1986)	Burial depth (m) used in field explosion test
1	1 kg	1.2	2.0
2	4 kg	1.9	2.5
3	10 kg	2.4	2.5
4	6 kg	2.1	2.5

The pressure and acceleration transducers were mounted on IDMUs and buried targets (details of the IDMU and buried targets are presented in Sections 4.6.6 and 4.6.7, respectively). For reliable pressure measurement, there must be a good contact condition between the pressure gauge and the soil. For the IDMU, contact between the soil and pressure gauge was by application of pneumatic pressure to a double acting cylinder (see Section 4.7.6). For the buried targets, the site was excavated down to placement level and the target was positioned. Backfilling was done manually till above the target and the area between the pressure gauge and soil was manually compacted such that a good contact was created (Section 4.7.7). The accelerometers were mounted on triaxial mounting blocks placed directly behind and central to the pressure gauge position. The buried targets consisted of two sizes, 300 mm x 300 mm x 300 mm cuboidal box and 600 mm x 300 mm x 300 mm rectangular box targets. All of the measurement points were installed at the same level as the buried charge, with the exception of two IDMU measurement points. The depths of the IDMUs at these two locations were 4 m (in Cluster 2) and 4.5 m (in Cluster 3). These measurement points were used to study the characteristics of explosion wave propagation with depth. The field explosion test sequence and measurement points are summarised in Table 4-4.

4.6.5 Site preparation and safety considerations

The field explosion test site was cleared of vegetation and topsoil covering an area 50 m long by 30 m wide. The site was initially surveyed for shallow obstructions by considering an area 75 m long and 50 m wide, and dividing this area into grids of about 25 m squares each. A pit of 1 m x 1 m x 0.6 m depth was manually excavated at each grid intersection location to check for shallow obstructions. Once the site was cleared of any shallow obstructions, site clearance works proceeded. The site equipment consisted of a bulldozer, a front loader, an excavator and a reliable mechanical hand-held compactor. The bulldozer was used to reduce the existing ground level by about 1.5 m. This levelling removes the stiff zone at ground surface that was detected during the CSW tests. The excavated material showed that this is the root zone of the site.

Table 4-4. Summary of field explosion tests

Test No.	Charge weight	Measurement Points	Remarks
1	1 kg PETN Cluster 1	<u>IDMUs at:</u> 1m, 1m, 2m, 3m, 4m, 5m, 6m, and 10m. <u>Buried targets at:</u> 1m, 2m and 3m.	<ul style="list-style-type: none">• Two IDMUs at 1m radial distance• IDMU at 10m radial distance located in cluster 2.
2	4 kg PETN Cluster 2	<u>IDMUs at:</u> 1.59m, 1.59m, 3.17m, 4.76m, 6.35m, 7.94m. 7.94m and 15m. <u>Buried targets at:</u> 1.59m, 3.17m and 4.76m	<ul style="list-style-type: none">• Two IDMUs at 1.59m radial distance• One of the IDMUs at 7.94m radial distance was at 4m depth• IDMU at 15m radial distance was located in cluster 3
3	10 kg PETN Cluster 3	<u>IDMUs at:</u> 2.15m, 2.15m, 4.31m, 6.46m, 8.62m, 8.62m and 10.77m. <u>Buried targets at:</u> 2.15m, 4.31m, 4.31m, 6.46m	<ul style="list-style-type: none">• Two IDMUs at 2.15m and 8.62m radial distances• IDMUs at 10.77m radial distance was at 4.5m depth• Rectangular buried targets at 4.31m and 6.46m.
4	6 kg PETN	<u>IDMUs at:</u> 3.60m, 5.45m, 8.20m, 8.80m, 9.60m, 10.80m.	<ul style="list-style-type: none">• No buried targets in this test.

The excavated soil from the 1.5 m cut was collected by a front loader and used to form a 0.5 m high embankment around the 50 m x 30 m area to demarcate the test site. The excavator was used to shape the embankment and was compacted in-situ using a mechanical hand-held compactor. The excavator and the hand-held compactor were

retained during the full duration of the field tests to assist with site works. Tarpaulin sheets were also acquired and kept at the test site. The site was covered with the tarpaulin sheet at the end of each workday to protect the ground against rainfall. A temporary benchmark was also established to determine the ground level at the test site. A view of the test site is shown in Figure 4-16.

Figure 4-16. View of test site

Once the site had been cleared, the next stage was to drill the boreholes for installing the IDMUs. The charge hole and associated monitoring points at various radial distances were marked out and the ground levels were determined using a Sokia SDL30 digital leveling instrument. The soil boring was undertaken using a dry pneumatic percussion drilling method. The boring rig was mobile and was able to move around the site on crawler tracks, Figure 4-17. The depth of boring for each of the IDMU boreholes was computed with respect to the centroid of the charge for each cluster. A 175 mm drill bit was used to form the boreholes for the IDMU installation.

Figure 4-17. Borehole drilling for charge and IDMU installation

The IDMU holes were lined using 200 mm internal diameter mild steel pipes (with 3-mm wall thickness) to prevent the boreholes from collapsing during detonation, as well as to prevent deformation of the pipe during the explosion (Figure 4-18). If the pipes were deformed by the detonation, it would make recovery of the IDMUs difficult, and the pipe will have to be excavated to withdraw the IDMU. The pipes were cut to lengths of 2.3 m, and were installed to a depth of about 300 mm from the bottom of the borehole. The 300 mm unlined portion of the borehole was for IDMU installation. The pipe lengths were determined such that there was an adequate length of exposed soil below the steel pipes for the IDMU installation, as well as a short projection above the ground for easy identification. The pipes were covered at the top to prevent debris or water from entering the borehole. Each cased borehole was tested for verticality using a surveying staff held against the vertical wall of the steel pipe and the digital levelling instrument. The IDMU device was also tested by lowering it into every cased borehole to ensure that there will be no difficulties in installing the IDMU during the test.

Figure 4-18. IDMU borehole and mild steel pipe liner.

The schematic layout of the test site is shown in Figure 4-19. The safety stand-off distances for each charge estimated using ESTC (1990) guidelines are also indicated in Figure 4-19. An ISO 20 ft x 8 ft container cabin was located at the minimum safety stand-off distance (i.e. 50 m). The container housed the data acquisition system as well as the firing system to detonate the buried charge. The container was sited such that there was a clear view of the test site. A portable generator was used to provide electrical supply to the container cabin.

The safety of the field explosion tests was stringently enforced by setting up signage, flags, barriers and sentries at the three access roads leading to the test site (Figure 4-19). Communications by sentries at these three locations were by portable, battery-powered military telephones, called telejays, which were connected by telephone cables to a master telejay unit in the container. The siting of the safety container and the sentries were such that there was a clear field of vision and eye contact between all the sentries and the test area. This facilitated hand signals to be used in the event the direct lines were not working. The area surrounding the test site was checked carefully before detonation to ensure that there were no unauthorised personnel in the vicinity. Prior to a detonation, a red flag was raised and a siren was sounded to signal that an explosive is to be detonated. After detonation, the red flag was replaced with a white flag and the siren was sounded again to signal that the area is safe again.

Figure 4-19. Field test layout and control points manned during charge detonation

During the transportation and placement of charges, all telecommunication devices were switched off to prevent accidental detonation of the electric detonators. Before arming the explosive, all non-essential personnel were withdrawn to the container office or outside the test site. Only selected personnel were allowed to stay on site to minimise any risks. The traffic along the three access roads was also completely stopped when the explosive was ready to be armed. This is to avoid any driving mishaps when the charge was detonated.

All personnel on site were accounted for before arming and detonation of the explosive. Once the explosive was armed, a siren was sounded continuously for 1 minute as a warning before detonation. The charge was detonated using a firing device inside the container. Upon detonation, the test site was inspected by the Test Commander and Ordnance IC to determine if the detonation was complete. Field test results from the data acquisition system were also reviewed to see if there was any anomaly in the recorded measurements which might suggest incomplete detonation. The explosion effects were also recorded using digital photos, video camera and other means before any other works were allowed to proceed.

4.6.6 In-situ dynamic unit (IDMU), modifications and instrumentation

The IDMU is a pneumatic system that was developed by Leong et al. (1996) and is an attractive alternative to buried sensors as it avoids problems relating to sensor orientation, sensor contact with the soil and recovery in buried explosion tests. The device is lowered in a predrilled cased borehole and pneumatically extended until the target face is in firm contact with the soil (Figure 4-20). Several field tests have been undertaken using the IDMU. Leong et al. (1996) have also reported that the unit has been used successfully up to a depth of 10m for monitoring ground motion due to a buried explosion.

The operation of the IDMU for sub-surface measurement is summarised below:

Figure 4-20. Schematic view of IDMU installed in a borehole (from Leong et al. 1996)

- Prior to the installation of the IDMU, a 200 mm diameter borehole has to be drilled to the required depth. As a precaution, a mild steel pipe liner may be introduced to prevent soil from collapsing, leaving a short unlined portion for the IDMU installation (Figure 4-20). Leong et al. (1996) observed that mild steel pipe liners were more suitable for field explosion tests, as PVC liners tended to distort when they are pushed into the borehole and thus led to installation difficulties for the IDMU.
- The IDMU consists of a pneumatic unit capable of exerting a maximum pressure of 1 MPa and has two end plates. One of the end plates is fixed while the other end plate is moveable, as it is connected to the piston of a pneumatic unit. There are two air lines leading into the pneumatic unit consisting of a double-acting cylinder. When pressure is applied through the air lines, the IDMU can be extended or retracted, as shown in Figure 4-21. The amount of pressure applied to the lines can be monitored by installing a pressure gauge with the air lines. The air pressure for operating the double-acting cylinder is provided from carbon dioxide cylinders.
- The IDMU is lowered with the aid of a placement tool to the unlined bottom portion of the borehole and aligned to the source. The placement tool consists of a series of

1m long aluminium pipes that can be joined together to lower the IDMU to the desired depth. Upon establishing the correct alignment, the piston is extended and the IDMU is wedged firmly to the exposed soil in the borehole. The pressure in the air line has to be maintained during the field explosion test. Leong et al. (1996) performed several field experiments and concluded that the applied pressure to the IDMU cylinders should be held at about 50 psi (345 kPa) for optimal results. The IDMU has a 3 mm diameter steel cable which is secured at the top of the borehole after installation. The steel cable is used to retrieve the IDMU from the borehole after the field explosion tests.

Figure 4-21. IDMU moveable plate retracted and extended.

- The original IDMU was fitted with a pressure sensor for measuring the incident pressure and accelerometer for the radial acceleration. The sensor cables were connected to a data acquisition system for measuring the stress waves. The original IDMU was 145 mm in length when fully retracted and measured 177 mm with the piston fully extended. The weight of the IDMU with sensors attached is about 1.0 kg.

The main improvements made to the IDMU in this study are the placement system, the air pressurization system, and instrumentation system. Figure 4-22 shows a schematic view of the IDMU and some of the enhancements.

Figure 4-22. Overall assembly consisting of moveable end plate, compact cylinder, fixed end plate, galvanised steel cover plate, connector and holder unit (not to scale)

Figure 4-22 shows that a new galvanized cover plate has been added to the original IDMU for additional protection against debris which may prevent the IDMU from retracting and making recovery difficult. The cover plate was designed such that it would sit into the recess of the double acting cylinder. The cover plate was also secured using four screws on both sides of the IDMU to hold the plate firmly. In addition, a 14-pin military style connector was incorporated to consolidate the instrumentation signal cables from the IDMU. Holes were also provided at the top of the cover plate to allow the passage of the pneumatic air lines to the double-acting cylinder. Additional fabrication details of the galvanized plate and mountings are shown in Figure 4-23.

Figure 4-23. Details of base unit and galvanised steel cover plate (not to scale)

The placement system consisted of a holder unit and a connector unit (Figure 4-24). The connector unit was machined to fit a rectangular guide slot in the cover plate (Figure 4-22). The connector was positioned such that it was at the centre of gravity of the total assembled unit. The purpose of the connector unit was to provide a means for applying suction pressure. The suction pressure was applied using a hand-held vacuum pump capable of providing a maximum vacuum of 50 psi, with the air line connected to a 4 mm straight valve that led into the connector unit (Figure 4-24). The suction pressure was sufficient to lift the assembled IDMU and position it in the borehole. The contact areas between the placement system and connector unit were machined to ensure a good fit, and an O-ring was used to seal the suction pressure against leakage. The connector unit was threaded into the holder unit by a threaded dowel. The holder unit is modular, with each module measuring 1 m length. Overlapping of holder units at the joints, of about 600 mm, and a latch for securing the holder units together ensure that the holder units act as a rigid rod.

(a) Details of holder unit

(b) Details of connector to IDMU

Figure 4-24. Details of (a) holder unit and (b) connector (not to scale)

Standard 4 mm diameter nylon tubes and 4 mm quick connectors were used in the IDMU. The air lines were pressurized using carbon dioxide cylinders. Carbon dioxide was selected as it does not support combustion, was safe to handle and was safe to be used in field explosion tests. The air pressure was applied through a 3-way centre close valve (lever type) which came with a 4 mm quick connector and the air pressure was monitored using a pressure gauge attached to the air lines. The applied pressure was locked in at 50 psi and the carbon dioxide cylinder was removed from the site as an added safety precaution (Figure 4-25).

Figure 4-25. Checking applied air pressure in installed IDMU

The IDMU was fitted with a pressure transducer and three accelerometers in three orthogonal directions (Figure 4-26). The pressure transducer was installed in a machined groove whereas the accelerometers were mounted on a 10 mm triaxial mounting cube. The 10 mm triaxial mounting cube was fastened at the rear of the pressure transducer (Figure 4-26). The connectors for the pressure transducers and accelerometers were soldered to a 14-pin military-style connector at the top of the IDMU. A 7 m long shielded cable, with a torsional strength of 1.5 kN-m, was used to

Figure 4-26. Location of Kulite pressure transducers and PCB accelerometers

carry the instrumentation signals from the IDMU to a junction box. At the junction box, the various transducers were separated and individual cables were used to carry the signals to a data acquisition system (Figure 4-27). Two types of junction box were fabricated for the field tests to provide flexibility in the test configurations. A 4-channel junction box consisting of 3 BNC cable adapters and a single 7-pin DIN plug adapter enabled the shielded cable to be separated into a pressure measurement and triaxial acceleration measurement, and allowed a single IDMU or a buried target to be monitored. In view of the distances between measurement points, it was also possible for a few shielded cables to be gathered at a common location. In this case, a 12-channel junction box was used to allow better collation of instrumentation cables at the test site.

(a) 4-channel junction box and IDMU

(b) 12-channel junction box

Figure 4-27. IDMU and junction boxes in field explosion test

Data acquisition in field explosion tests can be a difficult task, especially in the near-field. The instrumentation and data acquisition system has to be able to capture the high frequency ground motion in a short duration typically lasting only about 2 seconds. The instrumentation should not be loaded beyond its intended range and the data acquisition system (DAS) has to be triggered at the right time so as to capture an enormous amount of data within a short duration for future analyses. These requirements have led to the development of dedicated instruments and appropriate data acquisition system for use in blast vibration analyses. Given the harsh operating climate and the risk of damage, not all monitoring points can be captured and it is thus normal for some redundancy to be built in as part of the overall field test layout. A basic field portable vibration recording system generally consists of three important elements: the transducers, the cabling system, and the data acquisition system as shown in Figure 4-28.

Figure 4-28. Components of an ideal field portable vibration recording system

Transducers are usually the weakest link of the overall system (Riesler et al. 1993). An unsuitable or out-of-range sensor can ruin the whole effort in the data acquisition process, and should thus be selected carefully. Sensors basically operate by converting the physical quantity measured (e.g. pressure) to an electric current. The main considerations for selection of transducers are sensitivity, resolution, frequency,

measuring range, cross-axis sensitivity and durability (Harris and Crede 2000). For most shock and vibration studies, pressure and acceleration are important parameters for analyses and modelling, and can be obtained using pressure transducers and accelerometers, respectively. Cross-axis sensitivity is an important criteria in triaxial measurement systems, where mechanical disturbances are measured in the three orthogonal directions. For proper interpretation of results, a disturbance in one axis should leave the other two axes relatively unaffected. The mass and size of sensors can affect the sensitivity of sensors. Furthermore, adding mass (e.g. an accelerometer) to a vibrating structure can alter the frequency of vibration sometimes also referred to as mass loading (Harris and Crede 2000). For accurate measurement, the transducer has to be properly coupled to the test object by means of a firm mounting over the frequency range of interest. Periodic recalibration of sensors is important as they provide a check on the reference for measurement.

Cables transmit the electric signal from the sensors to a DAS. Ideally, cables and connections should be electrically 'invisible', i.e. they should not cause a change to the signal. Unfortunately, if precautions are not taken, this can happen and the output signals differ from those measured by the sensors. This leads to uncertainty in the results obtained. Coaxial cables incorporating a conducting shield have been shown to be effective in reducing background signals or noise (Klipec 1977). In instrumentation for explosion tests, ground loops can be generated if the sensors or amplifiers are separately grounded. To avoid this problem, the whole system should be continuous and there should only be one point of grounding, usually at the DAS. Data affected by ground loops can be salvaged to a limited extent using digital filters (e.g. frequency rejection filters proposed by Carleton (1974) during the signal processing stage).

Modern data acquisition systems (DAS) are highly sophisticated. The DAS is a collective term comprising the preamplifier, amplifier, permanent recorders, etc. in a single console. Most DAS are straightforward to use: the user plugs in the sensor cable at the channel port of the DAS and makes appropriate settings to define the data to be captured. The ease of operating these systems and their typically robust design make

them attractive for use in sophisticated monitoring situations. A portable generator is usually adequate for most field tests. An uninterruptible power supply (UPS) should additionally be installed at the DAS, to allow data acquisition to continue should there be a power failure during the field test.

The above guidelines were applied in selecting and operating the instrumentation system for the field tests. The field instrumentation for the explosion tests comprised accelerometers in three orthogonal directions, and a pressure transducer in the radial direction. The accelerometers and pressure sensors were each mounted centrally on in-situ dynamic measurement units (IDMUs) and buried targets. The IDMU and buried targets were placed in line with the explosion source, except for two of the monitoring points, which were at 4 m depth (Cluster 2, 4 kg PETN test) and 4.5 m depth (Cluster 3, 10 kg PETN test).

The accelerometers used were piezoelectric PCB Model 353 and 350 quartz-shear ICP accelerometers, with ranges of $\pm 50g$, $\pm 250g$, $\pm 500g$, $\pm 1000g$ and $\pm 5000g$. These accelerometers were used in previous field tests and have performed well for near-field explosion measurements. The characteristics of the accelerometers used in the field tests are shown in Table 4-5. These accelerometers have an essentially flat amplitude response up to about 4000 Hz. The cross-axis sensitivity for the accelerometers ranged from 5% to 7%, which suggests that the effects on orthogonal acceleration measurements were negligible when used in field explosion tests. The PCB accelerometers are made out of quartz, which is a natural piezoelectric material. As it has no tendency to relax to an alternative state, quartz is considered to be the most stable of all piezoelectric materials, thus providing quartz accelerometers with long-term stability and repeatability. This is important in the field explosion tests since the same accelerometers are repeatedly deployed for several field tests. Also, quartz has virtually no pyroelectric effect (output due to temperature change), which provides stability in thermally active environments such as buried explosions. Because quartz has a low capacitance value, the voltage sensitivity is relatively high compared to most ceramic materials, making it ideal for use in voltage-amplified systems.

Table 4-5. Characteristics of PCB ICP accelerometers used in tests at site B.

Parameter	353B34	353B02	353B16	353B14	350B04
Measurement Range	±50 g	±250 g	±500 g	±1000 g	±5000 g
Nominal sensitivity (±5 %)	100 mV/g	20 mV/g	10 mV/g	5 mV/g	1 mV/g
Cross-axis sensitivity	≤ 5 %	≤ 5 %	≤ 5 %	≤ 5 %	≤ 7 %
Frequency range (±5 %), Hz	1 to 4000	1 to 7000	1 to 10000	1 to 10000	0.4 to 10000
Weight, g	27	10	2.7	3.0	4.5
Size, hex x ht (mm x mm)	19.1 x 31.2	12.7 x 31.5	9.5 x 20.6	9.5 x 22.4	19.1 x 31.2
Mounting thread	10-32	10-32	5-40	5-40	¼-28

The PCB Model 353 and 350 quartz-shear accelerometers incorporate built-in signal-conditioning electronics. The built-in electronics convert the high-impedance charge signal that is generated by the piezoelectric sensing element into a usable low-impedance voltage signal that can be readily transmitted, over ordinary two-wire or coaxial cables, to any voltage readout or recording device. The low-impedance signal can be transmitted over long cable distances and used in rugged field environment with little degradation. The electronics within ICP accelerometers require excitation power from a constant-current regulated DC voltage source and was supplied using a PCB Model 483B17 signal conditioner. The PCB signal conditioner was able to condition up to 12-input channels simultaneously, and enabled selectable incremental gain control (up to x200) for normalising input signals. The unit was AC powered and supplied a 24 VDC sensor excitation to the PCB ICP accelerometers. The frequency response of the signal conditioner ranged from 0.09 Hz to 90000 Hz and was adequate for the range of frequency response that expected from Model 350 and 353 PCB ICP accelerometers. The system was ideal for use as a front-end signal conditioner to data acquisition

systems. The input to the signal conditioner (from the accelerometer) and output from the signal conditioner to the data acquisition system were both by means of BNC jacks.

The pressure transducers used were Kulite LQ-080U series of soil pressure sensors with ranges of 0-200 psi, 0-500 psi, 0-1000 psi and 0-2000 psi. The LQ-080U series of soil pressure sensors are designed for accurate measurement of dynamic pressures and have performed well in previous field explosion tests. Insertion of a sensor in soil disrupts the stress field and induces either stress concentrations or reliefs depending on the sensor thickness. This stress-transfer phenomenon can seriously affect sensor accuracy. To overcome this problem, the LQ-080U employs a pair of extremely stiff diaphragms with a diameter-to-thickness ratio of greater than 5 and a diameter-to-deflection ratio of greater than 2000. This design, together with good contact between the soil and pressure sensor, ensure accuracy and repeatability of field test results. The DC energized sensing element of the LQ-080U comprises 4-active semiconductor strain gauges directly bonded to the measuring diaphragms. The rated electrical excitation of the Kulite sensor is 10 VDC (12 VDC maximum) and has a response time of less than 6 μ s, making its resolution infinitesimal. The thermal sensitivity and thermal shift is also negligible when operated at a temperature range of -40°C to 93°C and a relative humidity of 100%. The combined non-linearity hysteresis and repeatability is $\pm 0.5\%$ of the full-scale output (FSO). The acceleration sensitivity of soil pressure sensors is also important, as the vibration due to the buried explosion should not affect the pressure measurements significantly. Acceleration sensitivity is the maximum difference at any measured value between the output with and without the application of a specified constant acceleration. For the Kulite sensors, the acceleration sensitivity is less than 0.03%/g. These features make the Kulite LQ-080U soil pressure sensors suitable for use in field explosion tests. The pressure sensor has a nominal weight of 250 grams and the output may be conveniently monitored on most data acquisition systems. For the field tests, an Entran multi-channel signal conditioner (type MSC12) was used for conditioning signals from the Kulite sensors. Entran MSC12 enables 12 channels of signals to be conditioned simultaneously. The excitation to the Kulite sensors is switchable from 5 V to 15 V, and

allows the sensors to be balanced using a continuously adjustable internal calibrator with a voltage range of ± 1 mV to ± 10 V.

BNC coaxial cables, reeled into lengths of 100m each (Figure 4-29a), were used to transmit the acceleration signals from the accelerometers in the IDMU and buried targets. Four-core shielded cables were used for the pressure transducer. The connection of the sensor's cable to the main signal cable was achieved using junction boxes (Figure 4-29b). The signal conditioners, amplifiers and the data acquisition system were housed inside the container cabin (Figure 4-29c).

The 32-channel Eckelmann PSO 9080, incorporating ADD 2012 PSO/TRB Multichannel Analog Input Modules, and 24-channel Racal StorePlus VL 24 data acquisition systems were used to record acceleration data from the PCB signal conditioner and soil pressure data from Entran multi-channel signal conditioner. The sampling rate for the data acquisition system was set to 500 kHz with a recording duration of about 2 s. An uninterruptible power supply (UPS) was also connected to the power supply to ensure that field test data will still be recorded should the main power supply fail during the data recording process. The input voltage range for the PSO and Racal data acquisition systems was set according to the expected values for the field tests. Before each detonation, all channels were checked, balanced and tested to ensure integrity of the whole measurement chain, from the sensors to data acquisition system. A firing cable, which was connected to a firing device placed in the container cabin, was used to supply voltage to an electric detonator to detonate the buried charge (see Section 4.6.8). A signal was tapped out from the firing box for synchronization of the two data acquisition systems and for triggering. In addition, a breakwire, connected to one of the data acquisition channel, was also attached to the charge to provide the actual detonation time. A video camera was also installed at the top of the container cabin to capture the detonation process. The video camera was monitored and operated from a remote control in the container cabin.

(a) Cable reels outside data acquisition cabin

(b) Connection of sensor's cables to main signal cable through junction boxes.

(b) Data acquisition system.

Figure 4-29. Instrumentation system used in field test

4.6.7 Buried target design and installation

The buried targets consisted of mild steel box structures, measuring 300 mm x 300 mm x 300 mm and 600 mm x 300 mm x 300 mm. The targets sizes and material used were selected to ensure that a target introduced a rigid inclusion in the soil mass which was sufficiently large to attract reflection of free-field blast waves from the buried explosion. In scaled-model tests, buried structures are usually scaled linearly (Timoshenko and Goodier 1970) whereas explosives are scaled using the cube-root scaling method (Reinhart 1975). Thus, for a given prototype and explosive loading, the explosive used in the scaled-model tests will be significantly less than the prototype as the explosive energy would have to be scaled by a cubic relationship. In this regard, it is desirable to make the buried targets stiff (i.e. non-compliant) and place the targets at close stand-off distances from the explosive source. At larger stand-off distances, the free-field stresses and motions decay rapidly and results in a plane-wave expansion. The mild steel targets were used in view of the stiffness of the box targets and ease of fabrication of the targets using built up welding or other means. It was also relatively easy to machine instrumentation recesses and cable slots in mild steel. The targets were economical as they were fabricated for a one-time use in the field explosion tests.

The target sizes were selected to complement the data from field explosion tests at Site A. In Site A, there were some results for reflected pressure and acceleration in targets with a face area of 1 m square (RC structure), 90 mm square (aluminum block) and 65 mm square (IDMU). There was no local data for reflected pressure and acceleration for target face areas between 90 mm square and 1 m square. As such, target face areas of 300 mm square and 300 mm x 600 mm were selected. The targets were built using mild steel plates and welding the plates at each junction. The target sizes also dictated the ease with which the targets could be handled and installed on site, and the weights were calculated such that two persons could carry the targets over short distances and install them on site. The use of two different target face areas also enabled the effect of face area on the reflected pressures and acceleration to be investigated. Fabrication details of the targets are shown in Figure 4-30.

Figure 4-30. Buried target (300 mm or 600 mm long)

The top of the target was provided with a circular opening to enable transducers and cabling works to be undertaken (Figure 4-31). The target face was provided with a machined slot for mounting the Kulite pressure transducer. The pressure cable from the target face was guided into the box target by way of a stainless steel dowel at the top corner of the target face. A groove was also provided for the pressure cable to be seated until it reached the dowel. The dowel enabled the connector head of the pressure sensor

(a) Circular opening for cables and arrangement of pressure transducer and cable on target face

(b) Military style connector for pressure transducer and accelerometers

Figure 4-31. 300 mm x 300 mm buried targets used in field explosion tests

to be inserted through an annular slot in the dowel. The space occupied by the pressure cable was subsequently injected with silicone sealant (Figure 4-31a). This design provided protection for the cable against the blast effects. The acceleration of the buried target was also measured in three orthogonal directions. The accelerometers were mounted on a 10 mm triaxial mounting block at the rear centre of the pressure transducer. Similar to the IDMU, the connectors for the pressure transducers and accelerometers were soldered to a military style connector which was placed inside the box target (Figure 4-31b). A shielded cable was used to carry the instrumentation signals from the box target to a junction box at the ground surface.

The buried targets were installed with the help of an excavator. When the excavation reached the correct level, the ground was levelled and the target positioned. The distance of the buried target from the charge hole and the elevation of the target were checked to ensure that the target face orientation and level were correct before backfilling. Good contact between the pressure transducer and soil was ensured by manual backfilling of the soil around the target. Before backfilling the excavated space,

the shielded cable from the buried target was connected to the junction box and checked to ensure that there was electrical connectivity in the transducer connection. Once the instrumentation check was completed, the soil was backfilled and the location of the buried target was marked using wooden pegs at the ground surface. This marking facilitates the retrieval of the buried targets after the test.

4.6.8 Charge preparation, placement and detonation

The PETN explosive charge used was manufactured by Bofors Explosives and the charge composition consisted of 85% PETN and 15% oil. The PETN charge was supplied in cartridges of 125 g each, with a manufacturer rated TNT equivalence of 1.088 (i.e. $\text{TNT} = 1.088 \text{ PETN}$). The charge had a yellow to yellow brown appearance and was workable at room temperature. A cylindrical mould was prepared by cutting a length of PVC pipe and sealing the base of the pipe using a PVC sheet cut out to a circular plate of the pipe's diameter and pasting it at the base of the pipe. A 50 m long detonation cord was knotted at one end through a small hole at the base of the mould (Figure 4-32a). The mould was designed such that the aspect ratio of the charge (i.e. diameter to height ratio) was approximately unity. The space was then carefully hand compacted with the supplied PETN tubes, maintaining the detonation cord at the centre of the charge. A booster was included in the centre of the 4 kg, 6 kg and 10 kg charges to ensure complete detonation of the charge. The top of the charge was then sealed, and the open end of the detonation cord was enclosed through a 25 mm PVC tube to prevent damage while backfilling the earth (Figure 4-32b). A 15 mm diameter nylon rope was secured around the charge to facilitate lowering of the charge and to recover the lowered charge, should there be a misfire (Figure 4-32c). The charge was placed at the base of an unlined borehole and the borehole was backfilled (Figure 4-32d). The backfill soil was compacted at intervals during the backfilling. Details of the explosive charges used for the field tests are shown in Table 4-6. The explosive charge was detonated using a firing device in the container cabin. A firing cable was used to set off an electronic detonator which was attached to the detonation cord (Figure 4-33).

(a) Charge mould

(b) Charge prior to closing top cover

(c) Lowering of buried charge

(d) Charge being positioned

Figure 4-32. Charge preparation

Table 4-6. Materials used for detonation of charges

	Test 1 (1 kg PETN)	Test 2 (4 kg PETN)	Test 3 (10 kg PETN)	Test 4 (6 kg PETN)
Electric detonator No.8	2 nos	2 nos	2 nos	2 nos
Detonating cord	50 m	50 m	50 m	50 m
Plastic explosives (125 g cartridges)	8 nos	32 nos.	80 nos	48 nos
Booster	Nil	1 no.	1 no.	1 no.
Overall Charge dimension (diameter x length)	104.6 mm x 118 mm	156.2 mm x 155 mm	209.8 mm x 212 mm	156.2 mm x 225 mm

Figure 4-33. Detonation of buried charge

4.7 Typical Site B test results and explosion effects

Selected time-histories from the 1 kg, 4 kg, 6 kg, and 10 kg PETN field explosion tests are shown in Figures 4-34 to 4-43. The pressure and acceleration time histories for all the field explosion tests at Site B are shown in **Appendix A**.

4.7.1 1 kg PETN post test condition

The explosion of the 1 kg PETN test was fully contained in the ground. Ground cracks were just above the charge hole which extended to 1 m radial distance (Figure 4-44). The IDMUs at the 1 m radial distance could not be retrieved as the soil below the mild steel pipe had collapsed. All the data in the test was captured and there were no damage to the sensors and the IDMU. No damage was also noted on the buried targets. The instrumentation system was able to withstand the blast environment.

4.7.2 4 kg PETN post test condition

The buried explosion of the 4 kg PETN test was contained in the ground. There was also about 150 mm of ground heave just above the charge hole which extended to 1.57 m radial distance (Figure 4-45a). The IDMUs at the 1.57 m radial distance could not be retrieved as the soil below the mild steel pipe had collapsed. The mild steel pipe was also displaced laterally. On excavation, a camouflet of 2 m diameter was found in the location of the buried charge (Figure 4-45b). All the data in the test was captured and there was no damage to the sensors and the IDMU (Figure 4-45c). No damage was also noted on the buried targets (Figure 4-45d).

(a) R = 1 m

(b) R = 1.59 m

Figure 4-34. Incident pressure on IDMU: (a) 1kg PETN, $R/W^{1/3} = 0.972 \text{ m/kg}^{1/3}$
(b) 4kg PETN, $R/W^{1/3} = 0.974 \text{ m/kg}^{1/3}$

(a) $R = 3.60\text{ m}$

(b) $R = 2.15\text{ m}$

Figure 4-35. Incident pressure on IDMU: (a) 6 kg PETN, $R/W^{1/3} = 1.926\text{ m/kg}^{1/3}$
(b) 10 kg PETN, $R/W^{1/3} = 0.970\text{ m/kg}^{1/3}$

(a) $R = 1$ m

(b) $R = 1.59$ m

Figure 4-36. Radial acceleration of IDMU: (a) 1 kg PETN, $R/W^{1/3} = 0.972$ m/kg^{1/3}
(b) 4 kg PETN, $R/W^{1/3} = 0.974$ m/kg^{1/3}

(a) $R = 3.60 \text{ m}$

(b) $R = 2.15 \text{ m}$

Figure 4-37. Radial acceleration of IDMU: (a) 6 kg PETN, $R/W^{1/3} = 1.926 \text{ m/kg}^{1/3}$
(b) 10 kg PETN, $R/W^{1/3} = 0.970 \text{ m/kg}^{1/3}$

(a) $R = 1$ m

(b) $R = 1.59$ m

Figure 4-38. Transverse acceleration of IDMU: (a) 1 kg PETN, $R/W^{1/3} = 0.972$ m/kg^{1/3}
(b) 4 kg PETN, $R/W^{1/3} = 0.974$ m/kg^{1/3}

(a) R = 3.60 m

(b) R = 2.15 m

Figure 4-39. Transverse acceleration of IDMU: (a) 6 kg PETN, $R/W^{1/3} = 1.926 \text{ m/kg}^{1/3}$
(b) 10 kg PETN, $R/W^{1/3} = 0.970 \text{ m/kg}^{1/3}$

(a) $R = 3.60\text{ m}$

(b) $R = 2.15\text{ m}$

Figure 4-41. Vertical acceleration of IDMU: (a) 6 kg PETN, $R/W^{1/3} = 1.926\text{ m/kg}^{1/3}$
(b) 10 kg PETN, $R/W^{1/3} = 0.970\text{ m/kg}^{1/3}$

(a) $R = 1.59\text{ m}$

(b) $R = 1.59\text{ m}$

Figure 4-42. 4kg PETN buried target (0.3m cube), $R/W^{1/3} = 0.974\text{ m/kg}^{1/3}$
(a) Incident pressure (b) Radial acceleration.

(a) $R = 1.59\text{ m}$

(b) $R = 1.59\text{ m}$

Figure 4-43. 4kg PETN buried target (0.3m cube), $R/W^{1/3} = 0.974\text{ m/kg}^{1/3}$
(a) Transverse acceleration (b) Vertical acceleration.

Figure 4-44. Explosion effect of 1 kg PETN charge

4.7.3 10 kg PETN post test condition

The buried explosion in the 10 kg PETN test resulted in a crater being formed on the ground surface. This crater was not expected as the 10 kg PETN test was designed to be fully contained in the ground (Table 4-3). Figure 4-46a shows the crater formed by the 10 kg PETN charge. The apparent crater dimension was 2.9 m and the depth of the crater was 1.1 m. Concentric ground cracks were also observed at a radial distance of 8.7 m from the charge hole. The crater also displaced the mild steel pipe at 2.15 m radial distance, but there was no damage to the IDMU nor the steel pipe (Figure 4-46b). All the data in the test was captured and there was no damage to the sensors and the IDMU (Figure 4-46c). No damage was also noted on the buried targets (Figure 4-46d).

(a) Ground heave above detonation

(b) 2 m diameter camouflet

(c) IDMU condition after test

(d) Buried target condition after test

Figure 4-45. Explosion effects of 4 kg PETN charge

(a) Crater at ground surface

(b) Displaced steel pipe with IDMU in place at 2.15 m radial distance

(c) IDMU condition after test

(d) Rectangular buried target condition after test

Figure 4-46. Explosion effects of 10 kg PETN charge

4.7.4 6 kg PETN post test condition

The buried explosion in the 6 kg PETN test was contained in the ground. Parts of the ground above the charge had heaved due to the explosion, but the effect was only localized (Figure 4-47a). All the IDMUs used to monitor this test could be retrieved easily. A camouflet was also formed at the location of the buried charge (Figure 4-47b). The camouflet was spherical in shape and measured about 2.4 m diameter. All the data in the test was captured and there was no damage to the sensors and the IDMU (Figure 4-47c). There were no buried targets in the 6 kg PETN test.

4.7.5 Determination of propagation velocity

The incident pressure and acceleration time-histories for the three orthogonal directions, for the 4.0 kg PETN (or 4.352 kg TNT) test, at a radial distance of 4.76 m are shown in Figure 4-48. In terms of scaled range, this can be expressed as $R/W^{1/3} = 2.92 \text{ m/kg}^{1/3}$.

Both vertical and transverse accelerometers showed two distinct peaks in the acceleration-time histories (Figure 4-48). The first peak was registered at about 22 ms, and the second and larger peak occurred at about 30 ms. The radial accelerometer, however, registered only a single peak signal at about 22 ms. Beyond the hydrodynamic zone, the hydrodynamic waves are replaced by elastic waves (compressive P-wave). Shear waves and surface waves also develop as the stress waves propagate through the soil media, and interact with medium of different elastic properties. At small distances, all the three waves (i.e. body waves and surface waves) will arrive together and greatly complicate wave identification. As the distances increase, the more slowly moving shear and surface waves begin to separate from the compressive wave and allow identification. This can be seen in the transverse and vertical acceleration time-history in Figure 4-48, where the slower moving shear waves show up distinctly in the field data. The first arrival can be used to determine the average propagation velocity (also called the seismic velocity) of the compressive waves in the soil medium. The time of arrival associated with the second peak can also be used to determine the average shear

(a) Ground heave above detonation

(b) 2.4 m diameter camouflet

(c) IDMU condition after test

Figure 4-47. Explosion effects of 6 kg PETN charge

Figure 4-48. 4 kg PETN field test at 4.76 m radial distance

wave velocity, V_s , of the soil medium. Figure 4-49 shows a linear regression technique applied to all the field test data.

The first arrival of the accelerometers and the pressure sensors are plotted against the distance from the explosion source. The gradient of the line gives the seismic velocity, c . Figure 4-49 suggests that the average seismic velocity, c , is 225 m/s. The average shear wave velocity, V_s , for the site can similarly be obtained from time-history records showing a distinct second peak. Figure 4-49 suggests that the average shear wave velocity, V_s , is 159 m/s.

Figure. 4-49. Regression analysis on field data

4.7.6 Determination of attenuation coefficient

The attenuation coefficient of the soil is represented by the slope of the best-fit line in a logarithmic plot of free-field pressure versus distance (Ingram 1977). In Figure 4-50, the slope of the line (or attenuation coefficient) for the three explosion clusters (1 kg, 4 kg and 10 kg PETN) ranged from 2.92 to 3.02.

Figure. 4-50. Determination of attenuation coefficient

CHAPTER 5

FINITE ELEMENT MODELLING OF FIELD EXPLOSION TESTS

5.1 Introduction

A closed-form solution is desirable for any problem in engineering and science. However, closed-form solutions are only available for simple problems with simple domain, loading and boundary conditions. An alternative approach is to use a numerical method to determine solutions for complex problems, such as explosion simulation. Among numerical methods, the finite element (FE) method and the finite difference method are two widely used schemes for the solution of partial differential equations. Both methods have been successfully applied to explosion simulation problems using programs such as LS-DYNA (LSTC 1999) and AUTODYN (Century Dynamics 1997). Finite element analysis can easily deal with complicated domain and boundary condition and is usually applied to spatial domains. Also, in finite element analyses, the global error is controllable but local error is not easily controlled, as the formulation is based on the weak form of the partial differential equation over the whole domain (Wang et al. 2002). Finite difference method can be used in both spatial and temporal domains and is based on the local error control through directly discretizing partial differential equations. However, it is more difficult to treat the boundary conditions in the finite difference method (Slawson 1995). This chapter describes the use of a commercial finite element program, LS-DYNA, to simulate the field explosion tests described in Chapter 4. LS-DYNA is widely used by engineers and scientists to analyze

large deformation response of inelastic solids and structures in a number of industries including automotive, aerospace, metal manufacturing, plastics, and defence. As mentioned in Section 2.7.5, LS-DYNA has been successfully used to model underground explosions in soils. The program incorporates an explosion model and is convenient for explosion modelling analysis.

5.2 Finite element modelling stages

Finite element (FE) analysis is a standard procedure and involves two main steps. The first step is to construct a numerical model to simulate the physical problem. This step is sometimes called design stage or mathematical model. The second step includes FE analysis and result presentation. In the design stage, the objective and expectation of numerical analysis and the necessary conditions for analysis (e.g. prediction, parametric study, etc) should be emphasized. The following items should be included in the first stage: finite element mesh, initial conditions boundary conditions, soil model and model parameters.

Soil model is a key issue in the numerical analysis. The soil model selected should be able to predict the soil behaviour in the field and to reproduce the relevant mechanisms such as large deformation, strain rate effect and wave propagation. The selection of soil model is difficult as many constitutive soil models are available, starting from simple Mohr-Coulomb linear elastic analyses to more complex Cam-clay (isotropic model) or MIT-E3 (anisotropic) models (LSTC 1999). A good soil model will aim to reproduce the mechanisms operating on the ground and simulate them in a numerical environment, and the model does not oversimplify nor overcomplicate the problem (Hight and Higgins 1994).

The second stage is to analyse the model by formulating and solving the finite element equations. Any commercially available FE software can be used to accomplish this stage. The results obtained from the FE analyses depend largely on the model used. One important rule to follow in FE modelling is to progress from the simple to the complex

model. Moving from the simple to the complex model makes it easier to pinpoint difficulties with the model when results of the analyses are unrealistic. Conversely, determining what causes unrealistic results can be difficult if all the possible complexities are included at the start of the problem analysis.

As FE analyses can be time consuming, it is important that the computation time is put to good use, and a consistent set of metric units are suggested for ease of interpretation of results. The amount of computer memory storage and processing time required to solve the finite element equations is proportional to the number of nodes in the problem, the node numbering of each element and the integration order. The accuracy of the results obtained is affected to some extent by the shape of the elements and mixing of the element types. LS-DYNA (LSTC 1999) suggests that the use of uniformly shaped quadrilateral elements will provide reasonable accuracy for most modelling applications. Other element shapes such as triangles should be introduced where the shapes of the object to be modelled is irregular (e.g. jagged edges).

5.3 Governing equations for finite element method using LS-DYNA

5.3.1 Finite element formulation

As discussed in the previous section, the wave propagation in wave regime can be described by a set of partial differential equations describing the conservation of mass, momentum and energy. These, together with material model and initial and boundary conditions, define the problem. In the wave regime (elastic media), the equation of motion can be expressed as follows:

$$\begin{aligned}\rho \frac{\partial^2 u}{\partial t^2} &= (\lambda^* + G) \frac{\partial \Theta}{\partial x} + G \nabla^2 u \\ \rho \frac{\partial^2 v}{\partial t^2} &= (\lambda^* + G) \frac{\partial \Theta}{\partial y} + G \nabla^2 v \\ \rho \frac{\partial^2 w}{\partial t^2} &= (\lambda^* + G) \frac{\partial \Theta}{\partial z} + G \nabla^2 w\end{aligned}\tag{5.1}$$

where $\Theta = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z}$, $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$, $\lambda^* = \frac{E\nu}{(1-2\nu)(1+\nu)}$, u , v and w are displacements in the x -direction, y -direction and z -direction, respectively, λ^* , G are Lamé's parameters, E is elastic modulus, ν is Poisson's ratio, ρ is density of soil mass and t is time. Note that λ^* is undefined when $\nu=0.5$.

As the deformation caused by an explosion is usually large and rate-dependent, the formulation should include a time-dependent deformation. In Lagrangian coordinates, a point Ω initially at X_a ($a=1,2,3$) in a fixed rectangular Cartesian coordinate system moves to a point x_i ($i=1,2,3$) in the same coordinate system. Since a Lagrangian formulation is considered, the deformation can be expressed in terms of the converted coordinates X_a , and time t

$$x_i = x_i(X_a, t) \quad (5.2)$$

At time $t = 0$, the initial conditions are

$$\begin{aligned} x_i(X_a, 0) &= X_a \\ \dot{x}_i(X_a, 0) &= V_i(X_a) \end{aligned} \quad (5.3)$$

where V_i defines the initial velocities.

5.3.2 Weak form of equilibrium equation and spatial discretisation

The weak form of the equilibrium equation used in finite element modelling can be obtained from a virtual work principle:

$$\int_V (\rho \ddot{x} - \sigma_{ij,j} - \rho f_i) \delta x_i dv + \int_{\partial\Omega_1} (\sigma_{ij} n_j - t_i) \delta x_i ds + \int_{\partial\Omega_3} (\sigma_{ij}^+ - \sigma_{ij}^-) n_i \delta x_i ds = 0 \quad (5.4)$$

In Equation 5.4, ρ is the current density, $\ddot{x}$ is acceleration, σ_{ij} is the Cauchy stress, f is the unit body force along an interior boundary, $\partial\Omega_3$ when $x_i^+ = x_i^-$. Here, the

comma denotes covariant differentiation, and n_i is a unit outward normal to a boundary element of $\partial\Omega$. $\partial\Omega_1$ is the traction boundary and $\partial\Omega_2$ is the displacement boundary. In the finite element formulation, it is assumed that δx_i satisfies all boundary conditions on $\partial\Omega_2$, and the integration is over the current geometry. Application of the divergence theorem gives:

$$\int_V (\sigma_{ij} \delta x_i)_{,j} dv = \int_{\partial\Omega_1} \sigma_{ij} n_j \delta x_i ds + \int_{\partial\Omega_2} (\sigma_{ij}^+ - \sigma_{ij}^-) n_j \delta x_i ds \quad (5.5)$$

and noting that:

$$(\sigma_{ij} \delta x_i)_{,j} - \sigma_{ij,j} \delta x_i = \sigma_{ij} \delta x_{i,j} \quad (5.6)$$

leads to the weak form of the equilibrium equations:

$$\int_V \rho \ddot{x}_i \delta x_i dv + \int_V \sigma_{ij} \delta x_{i,j} dv - \int_V \rho f_i \delta x_i dv - \int_{\partial\Omega_1} t_i \delta x_i ds = 0 \quad (5.7)$$

The spatial discretization can be represented as:

$$\{x\} = [N] \{x\}^e \quad (5.8)$$

where the $\{x\}^e$ is the nodal value and the $[N]$ is shape function. The shape function has to be determined for different element types (Zienkiewicz et al. 1998). Shape functions for all the element types found in the LS-DYNA preprocessor have been incorporated with the finite element program, and need not be independently determined for analysis.

After the discretization of Equation 5.8, the weak form of Equation 5.7 can be expressed as:

$$\sum_{m=1}^n \delta x^{eT} \left[\int_{V_m} \rho N^T N dv \ddot{x}^e + \int_{V_m} B^T \sigma dv - \int_{V_m} \rho N^T f dv - \int_{S_m^1} N^T t ds \right] = 0 \quad (5.9)$$

where the Cauchy stress $\sigma^T = [\sigma_x \sigma_y \sigma_z \sigma_{xy} \sigma_{yz} \sigma_{zx}]$, $f^T = [f_1 f_2 f_3]$, $t^T = [t_1 t_2 t_3]$, and

$$[B] = \begin{bmatrix} \frac{\partial}{\partial x_1} & 0 & 0 \\ 0 & \frac{\partial}{\partial x_2} & 0 \\ 0 & 0 & \frac{\partial}{\partial x_3} \\ \frac{\partial}{\partial x_2} & \frac{\partial}{\partial x_1} & 0 \\ 0 & \frac{\partial}{\partial x_3} & \frac{\partial}{\partial x_2} \\ \frac{\partial}{\partial x_3} & 0 & \frac{\partial}{\partial x_1} \end{bmatrix} [N] \quad (5.10)$$

If the mass matrix of an element is expressed as:

$$m = \int_{V_m} \rho N^T N dv \quad (5.11)$$

A system equation can be obtained from Eq.(5.9), such that:

$$M\ddot{x}(t) = P(x, t) - F(x, \dot{x}) \quad (5.12)$$

where P is the external load vector and F is the stress divergence, given by:

$$F = \sum_{m=1}^n \int_{V_m} B^T \sigma dv \quad (5.13)$$

This procedure can be applied to spatially discretize finite element problem.

5.3.3 Decomposition of domain and coupling methodology

LS-DYNA allows subdomains to be use individual finite element processors. For example, structures are usually modelled in Lagrangian coordinates while fluids and gases are modelled in Eulerian coordinates. LS-DYNA then couples these different processors together in space and time to provide an optimized solution. This capability makes LS-DYNA suitable for the study of interaction problems involving multiple systems of structures, fluids and gases. The numerical processors of LS-DYNA and their applicable ranges are listed as follows:

- Lagrange processor for modelling solid continua and structures.
- Euler processors for modelling fluids, gases, and large distortion. These processors include first-order and second-order accurate schemes.
- ALE (Arbitrary Lagrange Euler) processor for the data exchange at the interface of Lagrange and Euler processor.
- Shell processor for modelling thin structural elements.

In LS-DYNA, air and explosive (TNT) are simulated by Euler processor, and are assumed to satisfy their respective equation of state (EOS), i.e. of ideal gas and the explosive model, respectively. Soil is assumed as an elastoplastic material and is simulated by the Lagrange processor. The explosive-soil interface is simulated using the ALE coupling feature in LS-DYNA. Arbitrary Lagrangian-Eulerian (ALE) formulations may be thought of as an algorithm that performs automatic rezoning. Without the ALE formulation, it will be necessary to perform manual rezoning by stopping the calculation when the mesh is distorted, smoothing the mesh and, remapping the solution from the distorted mesh to the smooth mesh. The primary advantage of a simplified formulation is its reduced cost per time step and execution efficiency. The ALE formulation is widely used for modelling fluid-structure and gas-structure interaction problems. This extends to blast and explosion effects in the near-field, and interaction on structures.

5.3.4 One-point integration and artificial viscosity

In order to speed up the computational efficiency, one-point integration is used for the element stiffness and load vector. However, because of the non-linearity of flow equations, shocks can form even though the initial conditions are smooth. In order to handle the discontinuities in the flow variables, it has been a standard practice in finite element formulations to introduce extra viscous terms (Wilkins 1980) into the formulation, and is called viscous hourglass control. LS-DYNA recommends a viscous hourglass control for problems with high mesh deformation. An artificial viscosity term is added to Equation 5.12 to enable a viscous hourglass control in the finite element analysis, Equation 5.14:

$$M\ddot{x}(t) = P(x, t) - F(x, \dot{x}) + H \quad (5.14)$$

where H is the hourglass resistance, which is automatically calculated by LS-DYNA and is added to the physical stresses at the local element level.

For explosive applications, a bulk viscosity, q , is also automatically introduced in the finite element analysis to capture the shock wave front caused by the detonation:

$$q = \begin{cases} \rho l (c_0 l |\dot{\epsilon}_{kk}|^2 - c_1 a |\dot{\epsilon}_{kk}|) & \dot{\epsilon}_{kk} < 0 \\ 0 & \dot{\epsilon}_{kk} \geq 0 \end{cases} \quad (5.15)$$

where $l = \sqrt[3]{V}$ is the characteristic length (or the 3D local sound speed), a is local detonation velocity, and constants $c_0 = 1.5$ and $c_1 = 0.06$ are default values.

5.3.5 Time integration and time step size

The explicit central-difference scheme is used for time domain in Equation 5.14:

$$\ddot{x}(t_n) = M^{-1} [P(t_n) - F(t_n) + H(t_n)] \quad (5.16)$$

$$\begin{aligned}
\dot{x}\left(t_{n+\frac{1}{2}}\right) &= \dot{x}\left(t_{n-\frac{1}{2}}\right) + \frac{1}{2}(\Delta t_{n-1} + \Delta t_n) \ddot{x}(t_n) \\
x(t_{n+1}) &= x(t_n) + \Delta t_n \dot{x}\left(t_{n+\frac{1}{2}}\right)
\end{aligned} \tag{5.17}$$

where

$$\begin{aligned}
t_{n-\frac{1}{2}} &= \frac{1}{2}(t_n + t_{n-1}), & t_{n+\frac{1}{2}} &= \frac{1}{2}(t_{n+1} + t_n) \\
\Delta t_{n-1} &= (t_n - t_{n-1}), & \Delta t_n &= (t_{n+1} - t_n)
\end{aligned} \tag{5.18}$$

Since LS-DYNA uses an explicit scheme for time domain, a maximum time step of integration is necessary to make the numerical solution stable. The value of this time step depends on several parameters of the numerical method and solution, and so the local time step ensuring stability is calculated for each mesh point. The minimum of these values is multiplied by a factor (currently a default value of 2/3 is built into LS-DYNA code) and is chosen as the time step for the next update. The time step to be used in the FE analysis is bounded by the highest natural frequency, $\omega_{\max}$, of any individual element in the FE mesh. The highest natural frequency, $\omega_{\max}$, is obtained by dividing twice the plane-stress speed of sound by the characteristic length of the element. Three user options exist in LS-DYNA for choosing the characteristic length of the element. By default, LS-DYNA determines the characteristic length by dividing the area of a quadrilateral element by the maximum length of the sides defining the element. The initial time step size can be decided only after the mesh size is generated and the geometry defined. The initial time step size is obtained using:

$$\Delta t \leq \frac{2}{\omega_{\max}} \tag{5.19}$$

Once the initial time step size is determined, LS-DYNA automatically compares the input time step and selects the time step size that satisfies Equation 5.19, and applies a factor (i.e. 2/3 by default). For explosion applications, the ALE performs automatic

rezoning when the mesh is distorted, smoothing the mesh and, remapping the solution from the distorted mesh to a smooth mesh before the next time-step is performed. After each iteration, the time-step is recomputed for each mesh point and this value is compared with the critical time-step (Equation 5.19). The lower of these values is multiplied by 2/3 and the next iteration commenced. As a result, the time-step used in the finite element analysis may not always be constant, and will be revised depending on the geometry of the rezoned mesh in the analysis.

5.3.6 Transmitting boundaries

Buried explosion problems often involve semi-infinite domains. The general approach in numerical simulation is to select a FE scheme such that the effects of reflected waves do not affect the accuracy of the results. This could involve truncating the remote boundary at a large distance from the zone of interest and then imposing free or fixed boundary conditions, so that the wave reflection effects are negligible. A large FE mesh will involve high computational cost, storage and time penalties. In the past, several approaches have been proposed to deal with unbounded domains (Khalili et al. 1999, Cohen and Jennings 1983). The viscous boundary method was originally proposed by Lysmer and Kuhlemeyer (1969) and is widely used. In the viscous boundary method, the infinite domain is truncated at an arbitrary location and replaced with distributed dampers, which absorb almost all the energy in the waves that impinge on it and minimizes reflections from the boundary. The normal stress, σ_n , and shear stress, τ , of the waves absorbed by a damper are such that:

$$\begin{aligned}\sigma_n &= -c_1 \rho V_p \dot{u}_x \\ \tau &= -c_2 \rho V_s \dot{u}_y\end{aligned}\tag{5.20}$$

where ρ is the density of material, V_p , V_s are the compression and shear wave velocities, respectively, and c_1 , c_2 are relaxation coefficients. The use of $c_1 = 1$ and $c_2 = 0.25$ will result in reasonable absorption of waves at the boundary in the dynamic FE analyses (LSTC 1999). These default values were used in the finite element analysis

for the field explosion test modelling. The transmitting or silent boundary is defined by providing a complete list of boundary segments and associated material during initialization of the finite element analysis.

5.3.7 Considerations in numerical modelling

Balance between accuracy and speed of computations is very important to the success of a numerical algorithm. The classical displacement approach of FEM is related to propagation of short wavelength. The errors in the velocity of propagation of these waves together with numerical damping make displacement-based finite elements unsuitable for shock propagation problems. One way is to use adaptable mesh in shock wave fronts (LSTC 1999). In order to reduce the computational time, two approaches can be used. The time step size is automatically selected to reduce the repetitive computation and one-point Gaussian integration is implemented to reduce the computational effort in LS-DYNA. Hourglass mode may occur in the element using one-point Gaussian integration, and thus an artificial viscosity, F , is introduced (Equation 5.14)

There are numerous publications dealing with contact problems involving well-defined surfaces (Shanaa 1991, Slawson 1995). Research has shown that under dynamic loading, the response of the interface between soil and concrete do not involve discrete jumps in tangential displacements, but instead the deformation localizes into a narrow shear band at a small distance away from the interface (Desai et al. 1985). Laboratory tests undertaken by Desai et al. (1985) suggest that the strength properties of the interface are similar to those of soil at failure and as such, no special interface element is required and the same soil model can be used at the interface (see Stevens and Krauthammer 1989a). The versatility of modern finite element programs have also contributed to the success of interface treatment by Stevens and Krauthammer's (1989a) approach, and no special interface element is required in the finite element analysis.

Several researchers have successfully used the finite element method for explosion applications. Pullen et al. (1989) demonstrated how DYNA (an earlier version of LS-DYNA) could be used to determine the near-field response of soil when subjected to a conventional explosive load using a two-dimensional model. Their results suggested that a simple elastoplastic model for soil available within DYNA could be used to simulate complex one-dimensional wave propagation behaviour. Quek et al. (1999) used AUTODYN to study the response of multi-storey buildings due to ground shock caused by an underground explosion. These investigations, when used in conjunction with field test data can provide greater insight on the damage mechanism for surface and near-surface structures subjected to buried explosions.

As discussed in Chapter 2 (Section 2.5.4.3), the numerical algorithm for dynamic problems should solve two issues; mesh size and time step size. Nonlinear dynamic analysis is time-consuming. The mesh size should be small enough to capture the main frequency concerned and the time step size should be carefully selected to guarantee the numerical stability and time saving.

5.4 Numerical modelling of field explosion tests

5.4.1 Introduction

The field explosion tests were modelled using LS-DYNA available at the Centre for Advanced Numerical Engineering Simulation (CANES), NTU. LS-DYNA uses the Silicon Graphics Inc. (SGI) Origin 3400 server as a computational platform, which is equivalent to 32 CPUs and has a system bandwidth of 44.8 GB/s. This set-up is adequate for complex three-dimensional problems to be solved using LS-DYNA. Storage allocation in LS-DYNA is dynamic and can be controlled during execution. The size of the finite element model is only limited by the size of the computational platform. CANES permitted up to 150,000 elements to be specified for the finite element model, and are adequate for full-scale three-dimensional modelling problems.

The FEMB (LSTC 1999) pre-processor was used to prepare the finite element model. The results were viewed using LS-POST (LSTC 1999), a post-processor in LS-DYNA. FEMB currently contains more than one hundred material types, and additional material types and properties can be added to the library. User-defined material properties can also be developed for use with the FE model. There are also varieties of element types such as beams, shells, solids, that can be selected for various applications.

The information provided in the pre-processor is converted to a keyword file by LS-DYNA, in preparation for analysis. The keyword file is a logically organized database that is simple to understand. Similar functions are grouped together under the same keyword. For example, under the keyword ELEMENT, are solid beam, shell element, spring elements, discrete dampers, seat belts and lumped masses. This format makes it convenient to locate, view and change material properties for the finite element analysis without the need to invoke the pre-processor. For explosion modelling, LS-DYNA's restart capability is very useful, as it allows the analyses to be broken down into stages. Upon the completion of each stage, a dump file is written, which can be checked using a post-processor. In this way, any computer time on incorrect analysis can be reduced.

5.4.2 Model geometry and boundary conditions

The schematic of the explosion tests is shown in Figure 5-1. The purpose of the finite element analysis was to obtain two important results: the soil free-field response and the amplification effects due to the buried targets. The finite element analysis can be treated as a three-dimensional (3D) problem with the shock waves propagating along three orthogonal directions (vertical, radial and transverse). The numerical model used to define the field explosion tests was axisymmetric about a vertical line through the centroid of the charge, with reflecting boundary condition at the ground surface and transmitting boundary condition at the circumference and bottom boundaries. For free-field parameters, the problem can be modelled as an axisymmetric problem. For buried targets, it is more appropriate to model the problem in 3D. Figure 5-2 shows the FE model and the FE meshes used in the analyses.

Figure 5-1. Schematic of explosion test

Figure 5-2. (a) FE model (b) 2D FE mesh (c) 3D FE mesh

The size of the finite element model was determined based on the maximum distance of monitoring points in the field explosion test, and making some allowance before the transmitting boundaries are introduced. Khalili et al. (1999) suggested that introducing the transmitting boundary with an additional allowance of about 5% of the length

involved is preferred, i.e. the transmitting boundary is offset by 1.05 times the distance where it is supposed to occur. This is not a requirement in finite element modelling as the mathematical formulation (Equation 5.20) can adequately deal with the boundaries without any offset. In the present work, the transmitting boundaries were offset by 1.05 times the distance as recommended by Khalili et al. (1999). The charge depth varied between 2.0 m to 2.5 m from the top of the finite element mesh. Four finite element models were developed for the four field explosion tests, involving 1kg, 4kg, 6kg and 10 kg PETN charges. A 60° axisymmetric sector of the finite element model was used for the 3D analysis. This geometry size was found to be adequate to accommodate the free-field monitoring points and the buried targets used in the field explosion tests. However, the finite element analyses for the free-field response and the buried targets response were undertaken separately for easier collation of data. The overall size of all the finite element models was kept constant, with a radius of 17 m and a height of 6.5 m for both the 2D and 3D finite element models.

FEMB's automeshing capabilities can help reduce time to create a finite element mesh. There is, however, little editing control once the mesh has been generated. Automeshing using FEMB can result in undesirable element shapes at the corners and edges of finite element model. Manual meshing option in FEMB is straightforward to use but the task becomes tedious for 3D models consisting of several thousand elements. Each group of elements have to be defined and the material properties associated with that group have to be specified. For 2D applications, manual meshing can be conveniently undertaken. However, with 3D applications, the selection of element groups becomes more tedious and the finite element model has to be manipulated or rotated until the correct group of elements can be seen on the screen, before selecting and assigning properties to them.

Fortunately, LS-DYNA permits some flexibility in importing finite element models, and enables data to be read from computer aided design (CAD) software or other finite element pre- and post-processors, such as HYPERMESH. HYPERMESH is a finite element pre- and post-processing software developed by Altair Engineering (2001) to generate the finite element mesh. HYPERMESH enables more control on the

automeshing feature and can be used to avoid undesirable mesh elements at the corners and edges of the finite element model. The desired finite element model geometry, element shape and element groups are specified prior to the automeshing process. HYPERMESH will first output a preliminary mesh for the user to decide if the mesh could be accepted. The program also prompts warnings of any potential problems with the geometry of the finite element model and suggests changes that can be undertaken to overcome these warnings. Once the preliminary mesh is accepted, HYPERMESH proceeds with the 3D meshing task and the result is output as a data file. The finite element mesh generated using HYPERMESH for the field explosion test was read directly by FEMB for analysis using LS-DYNA. The boundary conditions and output controls were defined in FEMB for onward analysis in LS-DYNA.

A fine mesh, measuring 4 m radially and 2 m vertically from the centroid of the charge, was introduced in the 3D finite element model to account for the intense activity in the hydrodynamic zone. A coarse mesh was adopted beyond the 4 m radial distance and 2 m vertical distance occupied by the fine mesh zone. The finite element analysis was repeated with a fine mesh adopted for incrementally larger radial and vertical distances. The analysis did not show any appreciable difference in results, observed at the same monitoring points. For the 2D analysis, a 2 m horizontal band of fine mesh was used over the full width of the model. The FEMB finite element geometry checker found the adopted mesh size and extent to be adequate for the explosion analysis. The sizes of the 2D and 3D models used are tabulated in Table 5-1.

Table 5-1. Finite element mesh size

FE Model	Geometry size radius x height	Location of charge (from top)	Number of elements (2D)	Number of elements (3D)
1 kg PETN	17 m x 6.5 m	2.0 m	Total =19,608	Total =87,538
4 kg PETN	17 m x 6.5 m	2.5 m		
6 kg PETN	17 m x 6.5 m	2.5 m		
10 kg PETN	17 m x 6.5 m	2.5 m		

An initial time-step of 4.5×10^{-4} s was specified for the finite element analysis. This time step was compared with the critical time step by LS-DYNA (Equation 5.19), and if necessary LS-DYNA will update the time step to a new value to ensure stability of the current finite element run. The computed time-step was used for all zones found in the finite element model for the current iteration. The time step was recomputed and revised, if necessary, at the end of the current iteration. The new time-step was then used for the next time step of integration. For the field explosion tests, the initial time-step of 4.5×10^{-4} s was maintained throughout the finite element iterations, suggesting that the time step selected was lower than the critical time step for the finite element model.

For the transmitting boundaries, $c_1 = 1$ and $c_2 = 0.25$ were used for the relaxation coefficients (Equation 5.20) as suggested in the LS-DYNA users manual (LSTC 1999). The extent of the transmitting boundaries was defined by providing a complete list of boundary segments and associated material during initialization of the finite element analysis. A few results from finite element nodes adjacent to the transmitting boundaries were selected and compared with the results from nodes at the transmitting boundaries to ensure that there was no unusual changes to the stress and displacement fields due to the transmitting boundaries. The results from these comparisons were reasonable and the assumptions used for the transmitting boundaries were deemed reasonable.

Before the finite element analysis can be undertaken, the material models representing the soil medium and the explosive material will have to be defined. The explosive material can be represented using a JWL equation of state, while soil medium is usually represented using an elastoplastic soil model.

5.4.3 Equation of state and explosive source modelling

An equation of state (EOS) is defined as the relation between the hydrostatic pressure, the local density (or specific volume) and local specific energy (or temperature). The equation of state can be determined from knowledge of the thermodynamic properties

of the material and ideally does not require dynamic data to establish the relationship. However, in practice, the only practical way of obtaining data on the behaviour of a material at high strain rates is to carry out well-characterised dynamic experiments.

The PETN explosive charge is converted to equivalent TNT charge by applying a factor suggested by the explosive manufacturer (i.e. 1.088) for the finite element analysis. This conversion is necessary as most explosion models are calibrated using TNT as a reference charge. The equivalent TNT charge is assumed cylindrical with a density of 1630 kg/m³. The explosive charge size was computed by determining the equivalent TNT charge weight and assuming a 1:1 diameter to height aspect ratio. The charge sizes used in the finite element analysis are shown in Table 5-2 and are determined based on the known density of TNT (i.e. 1630 kg/m³) and the geometry of charge (i.e. 1:1). The charge sizes used in the finite element analysis differed slightly from the actual charge sizes used in the field explosion tests. The slight difference was due to the difference in density between PETN and TNT. The charge sizes used in the field explosion tests were not prepared exactly to a 1:1 aspect ratio (Table 4-6) given the limited sizes of PVC pipes available to make the mould. The PETN charge used in the field tests also had a slightly lower density of 1605 kg/m³. The difference in the charge sizes was deemed marginal.

Table 5-2. Determination of charge size used in finite element analysis

Charge used in field explosion test	Equivalent TNT charge (Col 1) x 1.088	Charge volume (m ³) $V=M/\rho$ (Col 2)/1630	Diameter or Height (mm) (Col 3*4/ π) ^{1/3}
1 kg PETN	1.088 kg	6.675x10 ⁻⁴	94.7
4 kg PETN	4.352 kg	2.670x10 ⁻³	150.4
6 kg PETN	6.528 kg	4.005x10 ⁻³	172.1
10 kg PETN	10.88 kg	6.675x10 ⁻³	204.1

The Jones-Wilkins-Lee (JWL) high explosive model (Equation of State Form 2, LSTC 1999) was used to define the pressure-volume-energy behaviour or equation of state for the TNT charge. The model assumes that the TNT charge detonates with a detonation velocity of 6930 m/s and develops a Chapman-Jouguet (CJ) pressure of 21 GPa. The JWL model defines pressure as a function of relative volume, V , and internal energy per initial volume, E :

$$p = A \left(1 - \frac{\omega}{R_1 V} \right) e^{-R_1 V} + B \left(1 - \frac{\omega}{R_2 V} \right) e^{-R_2 V} + \frac{\omega E}{V} \quad (5.21)$$

where A , B , R_1 , R_2 and ω are fitting coefficients for explosives. These coefficients are determined from dynamic experiments and are available for a variety of high explosive materials in LS-DYNA's material library, which were based on compiled test results from Dobratz (1981).

The JWL equation of state for the explosives has to be input in LS-DYNA with three other information: the explosive burn material (Material Type 8, LSTC 1999), the initial detonation, and the control explosive shadow. The explosive burn material allows the detonation of high explosives to be modelled in LS-DYNA, when used with the JWL equation of state. The essence of the explosive burn material is the burn fractions, F , which controls the release of chemical energy for simulating detonations. In the initialization phase of the finite element analysis, a lighting time, t_L , is computed for each element in the explosive zone by dividing the distance from the detonation point to the centre of the element, L_D , by the detonation velocity, D , i.e.:

$$t_L = \frac{L_D}{D} \quad (5.22)$$

After each iteration, the burn fractions, F , increases and it usually require several time steps for F to reach unity, thereby propagating the blast waves over several elements. After reaching unity, F is held constant and the analyses continued. LS-DYNA assumes

that the explosive burn material behaves as an elastic perfectly plastic material before detonation. Upon detonation, the explosive burn material behaves like a gas, following the JWL equation of state. The burn fractions also enable explosion simulation when the charge is not fully detonated, e.g. if only part of the explosive charge was consumed in the detonation. For the field explosion tests, default values were used for the explosive burn material. The default values assume that the charge was completely detonated, and the lighting times are computed automatically from the explosive zone defined during the meshing stage.

The initial detonation input is used to define the points to initiate the high explosive detonations which use explosive burn material (Material Type 8). Several points and lighting times can be selected, particularly for delay explosions. For the field explosion tests, the centroid of the charge was defined as the initiation point for all the finite element models. The control explosive shadow input is used to compute detonation times in explosive elements for which there is no direct line of sight. This can occur if an inclusion is modelled within the explosive material, thereby interfering with the explosion process. Since this was not an issue in the field explosion tests, default values were used and the lighting time for the explosive element was computed using the distance from the centre of the element to the nearest detonation point (i.e. Equation 5.22).

For the JWL model using TNT charge, the information supplied for the finite element analysis is shown in Table 5-3

5.4.4 Elastoplastic soil model and interface modelling

The behaviour of soils subjected to a buried explosion is complex. The material behaviour depends both on the type of material and the rate at which the load is applied. In the case of shock wave propagation, Mabssout et al. (2003) suggest that an elastoplastic model could account for the soil behaviour in a natural way. LS-DYNA includes nearly one hundred constitutive models for metals and non-metallic materials.

Table 5-3. Properties of TNT explosive from LSTC (1999)

Description	Input
Strength criterion	None (hydro)
Erosion	None
Failure model	None
Reference density	1630 kg/m ³
Parameter <i>A</i>	371.2 GPa
Parameter <i>B</i>	3.23 GPa
Parameter <i>R</i> ₁	4.15
Parameter <i>R</i> ₂	0.950
Parameter <i>ω</i>	0.35
Chapman-Jouguet detonation velocity (<i>D</i>)	6930 m/s
Chapman-Jouguet pressure	21 GPa
Burn on compression fraction (default)	0
Pre-burn bulk modulus (default)	0 Pa
Adiabatic constant (default)	0 Pa
Detonation energy, <i>E</i> ₀ (default)	4.25 MJ/kg

A series of material failure models, such as Bulk Strain, Ultimate Stress/Strain, Effective Plastic Strain and Principal Strain models are also available in LS-DYNA. The failure models can be specified with any plastic flow model and in all types of subgrid (Lagrange, ALE, etc). When a specified value exceeds a critical value, the material will fail and lose capacity of bearing load. A constitutive model will provide a stress-strain relation as follows:

$$\{d\sigma\} = [D]_{ep} \{d\varepsilon\} \tag{5.23}$$

where elastoplastic matrix $[D]_{ep}$ is given by:

$$[D]_{ep} = [D] - \frac{[D] \left\{ \frac{\partial g}{\partial \sigma} \right\} \left\{ \frac{\partial f}{\partial \sigma} \right\}^T [D]}{A + \left\{ \frac{\partial f}{\partial \sigma} \right\}^T [D] \left\{ \frac{\partial g}{\partial \sigma} \right\}} \quad (5.24)$$

and $[D]$ is the elastic matrix which is expressed as:

$$[D] = G \begin{bmatrix} \alpha & \beta & \beta & 0 & 0 & 0 \\ \beta & \alpha & \beta & 0 & 0 & 0 \\ \beta & \beta & \alpha & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (5.25)$$

where $\alpha = \frac{K}{G} + \frac{4}{3}$, $\beta = \frac{K}{G} - \frac{2}{3}$. g, f are potential and yielding functions, respectively, and K is the volumetric elastic modulus.

Krieg (1972) provided a simple model (Material Model 5) for foams and soils whose material properties are characterised by shear deformation. This model simulates crushing through the shear deformations of the soil. A pressure-dependent flow rule governs the deviatoric behavior of the model. The Von Mises yielding function, f^* , is expressed by:

$$f^* = \frac{1}{2} S_{ij} S_{ij} - (a_0 + a_1 p + a_2 p^2) \quad (5.26)$$

where a_0, a_1, a_2 are material constants which define a non-linear failure envelope. $S_{ij} = \sigma_{ij} - \delta_{ij} p$ is the deviatoric stress, where σ_{ij} is the stress tensor and p is the hydrostatic pressure:

$$p = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) = \frac{1}{3} (\sigma_x + \sigma_y + \sigma_z), \quad \delta_{ij} = \begin{cases} 1 & i = j \\ 0 & i \neq j \end{cases} \quad (5.27)$$

For simplicity, only linear failure envelope (Mohr-Coulomb criterion) is considered. The yielding function can be expressed in the Drucker-Prager form as:

$$f = \beta p + \sqrt{\frac{1}{2} S_{ij} S_{ij}} = k_f \quad (5.28)$$

where

$$\beta = \frac{\sqrt{3} \sin \phi}{3\sqrt{3 + \sin^2 \phi}}, \quad k_f = \frac{\sqrt{3} c \cos \phi}{\sqrt{3 + \sin^2 \phi}} \quad (5.29)$$

The model uses an associated flow rule, i.e. $g = f$. Implementation of this model is straightforward. One history variable, the maximum volumetric strain in compression, is stored. If the new compressive volumetric strain exceeds the stored value, loading is indicated. When the yield condition is violated, the updated trial stresses, S_{ij} , are scaled back using a simple radial return algorithm:

$$S_{ij}^{n+1} = \left[\frac{a_0 + a_1 p + a_2 p^2}{\frac{1}{2} S_{ij} S_{ij}} \right]^{\frac{1}{2}} S_{ij}^* \quad (5.30)$$

Table 5-4 shows the parameters used for the elastoplastic soil model. Krieg (1972) recommended that for stability of finite element analysis, a small amount of effective cohesion (e.g. 3 kPa) should be specified in the finite element input. The effective friction angle of the soil (28°) was determined from laboratory tests and was specified in the finite element analysis (Table 5-4). The finite element analysis showed that the use of a higher friction angle did not affect the finite element results as the soil outside the hydrodynamic zone was not in a state of failure, i.e. the specified failure envelope was not reached during the finite element analysis.

Table 5-4. Properties of soil

Description	Input
Mass density	1650 kg/m ³
Poisson's ratio	0.30
Bulk Modulus for unloading (default)	Hyperbolic function
Effective cohesion	3 kPa
Effective friction angle	28°
Volumetric crushing option	Off
Reference geometry to initialize pressure	Gravity load
Shear strain values (natural logarithmic values)	10 values maximum
Shear modulus corresponding to shear strain	10 values maximum

5.4.5 Buried structure modelling

Published literature suggests the use of a curve beam or a frame to simulate buried structure like tunnels or concrete boxes, respectively (Mazars 1986, Ma et al. 1999, Bull and Woodford 1999). The analysis can be simulated through two-steps. The first step is to obtain the response at the surface of the buried structure, and the second step is to analyze the buried structure to get the detailed responses within internal points. The shock wave-induced structural analysis usually refers to the second step (Hao et al. 1999). This thesis will only carry out the first step analysis, since the objective of the research is to investigate wave propagation in soil and amplification effects due to buried targets.

Various buried structures were used in the field explosion tests. All the buried structures used in the field tests were rectangular box sections, and were fabricated using concrete, mild steel, and aluminum. The buried structures were modelled as a frame using a brittle damage model for concrete and solid FE elements for the mild steel and aluminum

structures. The air void in the buried target was defined using an equation of state for ideal gas.

There are several equation of state for ideal gas in LS-DYNA. One of the simplest forms of equation of state is that for an ideal polytropic gas which may be used in many applications in LS-DYNA to model the motion of gases, Equation of State Form 1 (LS-DYNA user’s manual 1999). This EOS was derived from the laws of Boyle and Gay-Lussac (1972). Equation of State Form 1 (LSTC 1999) was used to model the air void in the regular box sections in the field explosion test model. The properties of ideal gas used in the finite element model are given in Table 5-5.

Table 5-5. Properties of air (from LSTC 1999)

Description	Input
Strength criterion	None (hydro)
Erosion	None
Failure model	None
Adiabatic exponent	1.4
Reference density	1.225 kg/m ³
Pressure shift	0 Pa
Reference temperature	288.2 K
Specific heat	717.3 J/kgK

An anisotropic brittle damage model (Material model 96) in LS-DYNA was assigned to the reinforced concrete box structures. The model is particularly useful for explosion applications as it enables progressive degradation of tensile and shear strengths across smeared cracks that are initiated under tensile loadings. The degradation is implemented by reducing the material’s modulus normal to the smeared crack plane according to a maximum dissipation law that incorporates exponential softening. The properties of the reinforced concrete are shown in Table 5-6.

Table 5-6. Properties of reinforced concrete (from LSTC 1999)

Description	Input
Young’s modulus	21.7 GPa
Poisson’s ratio	0.2
Initial principal tensile strength	3.1 MPa
Viscosity	500 kPa-s
Fracture toughness of concrete	0.14 kg/cm
Uniaxial compressive yield stress	29 MPa

The buried mild steel and aluminum box structures were modelled as solid FE elements. The densities of the solid element used for the FE analysis was taken as 7850 kg/m³ for the mild steel targets and 3900 kg/m³ for the aluminum targets. The air void, defined using Equation of State Form 1 (LSTC 1999) was coupled with the buried structure in the finite element analysis to make them behave together.

5.5 Implementation and verification of finite element model

The finite element model was implemented and verified using results from the field explosion tests. Finite element modelling on wet soils encountered at Site A was undertaken initially. The model was calibrated using the field test results (Chapter 4). This calibrated model provided a better handle of the finite element modelling for site B. The finite element analysis for simulating the buried explosion had four important components: the model geometry, the explosion model, the soil model and the buried target model. For calibration purposes, the monitoring points selected for the finite element model were similar to the field monitoring points. The horizontal distance to monitoring points was consistently defined as the distance from the centre of the charge to the vertical sensing face (i.e. external face of Kulite pressure sensor), Figure 5-3.

Figure 5-3. Determination of distances to monitoring points

The distance to the edge of the monitoring boreholes, at the ground surface, was taken as the distance to the sensing face. This assumption was reasonable as the boreholes were checked for verticality during drilling. For the buried targets, a vertical plumb bob was hung from the ground surface and was used to determine the horizontal distance to the buried targets.

5.5.1 Verification of finite element model for free-field test results

The soil model used in the finite element program requires input of the stiffness-strain curve. This information is input as co-ordinates, and a maximum of ten pairs of co-ordinates (shear strain, shear modulus) were allowed in LS-DYNA. For Site A, the stiffness-strain curve was developed using cyclic triaxial tests on soil samples recovered at about 1 m below the ground surface by a commercial soil testing laboratory. The maximum shear modulus, $G_{\max}$, cannot be directly obtained from cyclic triaxial tests in view of the limited strain configuration of the cyclic triaxial apparatus. However, $G_{\max}$ can be estimated using a stiffness-strain relationship suggested by Borden et al. (1996):

$$\frac{G}{G_{\max}} = \frac{1}{(1 + a\gamma^b)^c} \tag{5.31}$$

where a and b are constants, and γ is the shear strain. Equation 5.31 is a more general form of the stiffness-strain relationship suggested by Hardin and Drnevich (1972). Borden et al. (1996) found Equation 5.31 to fit normalized shear modulus data for Piedmont residual soils reasonably well. Soga et al. (1995) found Equation 5.31 with $c=1$ fitted the normalized shear modulus of clays very well. Equation 5.31 was used to normalize the shear modulus of the wet soil at Site A, with $c=1$ and c as a variable. It was found that using $c=1$ gives a smaller residual error for the curve fit for the soil at Site A. For Site A, $G_{\max}$ was not determined using the CSWS as the CSWS was not available during the tests at Site A, but was recovered using Equation 5.31 by curve fitting. The fitted curve, together with the respective a and b values are shown in Figure 5-4.

Figure 5-4. Fitted stiffness-strain curve for Site A using Equation 5.31 with $c=1$

Soil samples were also recovered from Site B at depths ranging from 1.5 m to 4 m below the ground surface. Laboratory tests such as P&S wave test and cyclic simple shear test were undertaken by Yeo (2003a, b) on the recovered samples. The stiffness-strain relationship for the soil at Site B at 100 kPa, 200 kPa and 400 kPa confining pressures is shown in Figure 5-5. The maximum shear modulus, $G_{\max}$, was also

determined using the CSWS. The $G_{\max}$ value determined from the CSW tests was found to be about 31.8 MPa and was constant up to a depth of 9 m from the ground surface.

Figure 5-5. Normalised stiffness-strain curve for Site B from P & S wave tests and fitted data from Figure 5-4

Figure 5-5 suggests that the normalised shear modulus, $G/G_{\max}$, reduces with increasing confining pressure. However, the curves for the 100 kPa, 200 kPa and 400 kPa effective confining pressures appear to collapse to a narrow band after a strain level of about 0.1%. The normalised shear modulus curve equation for the wet soil at Site A (Figure 5-4) is also shown in Figure 5-5. Figure 5-5 was developed using soil samples recovered at about 1 m below the ground surface, and the effective confining stress would be approximately 10 kPa. The effect of the lower confining pressure on the normalised shear modulus can be seen in Figure 5-5. The data points from Figure 5-4 plot above the P & S wave data for an effective confining stress of 100 kPa, and appear reasonable.

The observations from Figure 5-5 can be used to obtain an initial stiffness-strain relationship for Site B or other similar residual soil deposit. Based on the results of the CSWS tests and cyclic tests, the field and laboratory tests appear to be reasonable in reproducing the shear modulus versus strain curve for the test site.

For Site B, the stiffness-strain curve was developed from laboratory test results and from CSW tests (Section 4.6.3). The laboratory tests consist of P&S wave tests and cyclic simple shear tests on recovered samples from Site B undertaken by Yeo (2003). The $G_{\max}$ value from the P&S wave tests also provided a check on the $G_{\max}$ value obtained from the CSW tests. The stiffness-strain curve for Site B is shown with tabulated values of the data points in Figure 5-6.

Figure 5-6. Stiffness-strain curve for Site B

The initial finite element analyses were undertaken to verify the numerical model. The aim was to determine if the field test geometry, material models and other assumptions used to develop the finite element model was able to reasonably replicate the field explosion test results at Site B. The stiffness-strain curve (Figure 5-6) is an important

input for the numerical model. If the finite element model can replicate the field explosion test results, which encompasses a range of charge weights, the model can be used for parametric analyses.

A typical time-history from the field explosion test for a 1 kg PETN charge detonated at 1 m radial distance, and associated numerical results are shown in Figure 5-7. Figure 5-7 suggests that the first arrival time for pressure time-history from the finite element analysis is about 4.7 ms. For a radial distance of 1 m, the seismic velocity can be computed as 213 m/s. The results from the field explosion tests suggest that the average seismic velocity in the field was about 225 m/s (Figure 4-49). The numerical results incorporating the stiffness-strain profile shown in Figure 5-6 provided a reasonable match with the results obtained from the field explosion tests. Apart from the peak pressure and arrival time, the waveform from the numerical results also appear to match reasonably well with the field test results.

Figure 5-7. Incident pressure time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results.

The corresponding acceleration time-histories for the field explosion tests at Site B (1 kg PETN at 1 m radial distance) and numerical results are shown in Figures 5-8 to 5-10.

Figure 5-8. Radial acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results

Figure 5-9. Transverse acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results

Figure 5-10. Vertical acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results

Figures 5-8 to 5-10 suggest that the numerical model is able to replicate the field test data reasonably well. The numerical results using the stiffness-strain (Figure 5-6) were substantially within 10% of the first peak that occurred in the time-histories of the field test results, and were considered acceptable. However some of the subsequent peaks and waveforms did not match exactly. This is evident in the acceleration results where the number of cycles in the numerical results is always more than the field test result. The effects of material damping was investigated in the finite element analyses with a view of determining if the inclusion of material damping would provide a better match between the numerical results and the field test results.

5.5.2 Effect of damping ratio on free-field results

The results presented above were obtained without considering material damping, i.e. damping ratio, $\xi=0$. The numerical analyses were again undertaken using several damping ratios, ranging from 0.1 % to 1 %. When a damping ratio of 0.1% was used in the finite element analysis, no significant difference was found in the results compared

to case where there was no damping ratio, i.e. $\xi=0$. When a 1 % was used, the numerical results showed a reduction of the first peak and the arrival time was slightly delayed. The numerical results were also unable to simulate the subsequent peaks in the acceleration time-histories. It was found that when a damping ratio of 0.3 % was used with stiffness-strain curve for Site B (Figure 5-6), the numerical results provided a better match between the field test results and the numerical results compared to the case where there was no damping ratio (Figures 5-11 to 5-14). Figure 5-11 suggests that the match between the numerical results and the field explosion test results improved when a damping ratio of 0.3 % was used. This is evident in Figure 5-11 between 10 ms to 20 ms interval of the pressure time-history, compared to same interval in Figure 5-7 (with $\xi=0$). Figure 5-12 also shows that the 0.3 % damping ratio provided a better match to the field test results in the 10 ms to 20 ms interval of the radial acceleration time-history, compared to the numerical results without any damping ratio (Figure 5-8). The radial acceleration was also found to be relatively constant in the 20 to 50 ms interval with the damping ratio included. Figure 5-13 shows that the 0.3 % damping ratio reduced the peak transverse acceleration by about 5% compared to the numerical results without damping (Figure 5-9). The number of cycles in the transverse acceleration time-history was also reduced from four to three, and the numerical results were found to be relatively constant in the 30 to 50 ms interval with the damping ratio included. Figure 5-14 shows that the 0.3 % damping ratio provided a better match with the field test results, especially at about 10 ms location in the vertical acceleration time-history. There was no other significant improvement to the vertical acceleration time-history.

5.5.3 Effect of stiffness-strain curves

Section 5.5.1 suggests that the finite element model can replicate the field explosion tests at Site B reasonably well. The use of a small amount of damping ratio was also shown to improve the numerical results marginally. Given the importance of the stiffness-strain curve in the finite element model, additional finite element analyses were undertaken to investigate the sensitivity of the stiffness-strain curve on the numerical results.

Figure 5-11. Incident pressure time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results with 0.3% damping.

Figure 5-12. Radial acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results with 0.3% damping

Figure 5-13. Transverse acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results with 0.3% damping

Figure 5-14. Vertical acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results with 0.3% damping

Three parts of the stiffness-strain curve were considered: The small-strain shear modulus or $G_{\max}$, the slope or gradient of the stiffness-strain curve and the large-strain shear modulus or $G_{\min}$ (Figure 5-15). Each of these parts was varied to determine the effect of the stiffness-strain curve on the numerical results. $G_{\max}$ was selected for the sensitivity analyses as $G_{\max}$ is a difficult parameter to measure in the laboratory and the field and often there are reported variations in $G_{\max}$ values when different methods are used. These differences exist not just for laboratory determined $G_{\max}$ values (Cunny and Fry 1973), but also for field determined values (e.g. Matthews et.al 2000). By studying the effect of inaccurate estimation of $G_{\max}$ on numerical results, the associated error in the free-field stresses and motions can be identified and an appropriate factor of safety

Figure 5-15. Stiffness-strain curves for sensitivity analyses of finite element model

can be applied to the free-field stresses and motions where uncertainties in the determination of $G_{\max}$ exist. Previous numerical simulations of field explosion tests at Site A (Anand et al. 2001) also suggested that the gradient of the stiffness-strain curve (i.e. the intermediate portion in Figure 5-15) is sensitive to the numerical results. However, the field explosion tests at Site A only consisted of 4 kg PETN charges. The finite element model was verified using the Site A field test results and it was uncertain if the results from this finite element analysis could be used for parametric analysis when the charge weight differed. Finite element analyses were thus undertaken to investigate the sensitivity of the stiffness-strain curve using the Site B numerical model which was verified for charge weights ranging from 1 kg PETN to 10 kg PETN. The $G_{\min}$ value was also varied to investigate if the numerical results were sensitive to $G_{\min}$ variations.

The stiffness-strain curves for the sensitivity analyses were developed using the stiffness-strain curve for Site B (Figure 5-6) as a reference and is shown as Profile 1 in Figure 5-15. Since the effect of the damping ratio on the numerical results was only marginal, the finite element analyses for the sensitivity study were undertaken without consideration of damping ratio (i.e. $\xi=0$). The initial sensitivity analyses were undertaken by varying the $G_{\max}$ values from about 28.5 MPa (Profile 2) to 42 MPa (Profile 3). This was followed by sensitivity analyses of the gradient of stiffness-strain curve. The $G_{\max}$ and $G_{\min}$ values of Profiles 2 and 5 were made approximately similar. The intermediate portion between Profiles 2 and 5 was used to demarcate the lower and upper bounds, respectively, of the gradients of stiffness-strain curves that were investigated in the sensitivity analysis. The final investigation was on the sensitivity of $G_{\min}$ on the numerical results, which was varied from about 1.7 MPa to 4.8 MPa. Table 5-7 shows the $G_{\max}$ and $G_{\min}$ values for the profiles used in the field test.

The results of the sensitivity analysis, when $G_{\max}$ was raised from 31.8 MPa (Profile 1) to 42 MPa (Profile 3) are shown in Figure 5-16. Figure 5-16 suggests that the increase in peak pressure was about 10% with respect to peak pressure using Profile 1. The first arrival time for pressure time-history using Profile 1 and Profile 3 were 4.7 ms and 3 ms

Table 5-7. Stiffness-strain curves

Profile number	$G_{\max}$ (MPa)	$G_{\min}$ (MPa)	Numerical results compared with field test results
1	31.8	1.7	Calibrate model
2	28.2	4.8	Overestimate
3	42	4.8	Overestimate
4	38	3.8	Overestimate
5	29.8	4.8	Gradient changed. Underestimate
6	35	3.2	Underestimate
7	29.8	3.2	Slightly underestimate

Figure 5-16. Incident pressure time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results.

respectively. For a radial distance of 1 m, the seismic velocity can be computed as 213 m/s (using Profile 1) and 270 m/s (using Profile 3). The field explosion tests suggest that the average seismic velocity was about 225 m/s (Figure 4-53). The higher seismic velocity using Profile 3 can be explained on the basis that the soil in Profile 3 would be expected to be stiffer compared to the soil in Profile 1. For the acceleration, the increase in peak radial acceleration when $G_{\max}$ was raised from 31 MPa (Profile 1) to 42 MPa (Profile 3), was about 8% higher with respect to the peak radial acceleration using Profile 1.

The effect of the gradient of the stiffness strain curve between $G_{\max}$ and $G_{\min}$ was also investigated. The intermediate portion bounded by Profiles 2 and 5 was shifted such that $G_{\max}$ varied between 9.1 MPa and 17 MPa at about a strain level of 0.03 %. The range was selected considering the usual bounds of stiffness-strain curves commonly encountered in local geology (Tou 2001). The numerical results showed that when $G_{\max}$ was varied from 9.1 MPa (Profile 5) to 17 MPa (Profile 2) at a shear strain level of about 0.03 %, the increase in peak pressure was about 5 % with respect to peak pressure using Profile 5. For the acceleration, the associated increase in peak radial acceleration was about 6 % with respect to the peak radial acceleration using Profile 5.

The final investigation was on the effect of $G_{\min}$ variations on the finite element simulation. The $G_{\min}$ value was varied from about 1.7 MPa to 4.8 MPa. It was observed that the $G_{\min}$ variations had only marginal impact on the finite element results, compared to variations in $G_{\max}$ and the intermediate portion of the stiffness-strain curve. The numerical results showed that when $G_{\max}$ was varied from 1 MPa to 5 MPa, the increase in peak pressure and peak radial acceleration was about 2 % with respect to lower peak pressure and peak radial acceleration. The effect of the stiffness-strain curve on the ground response is discussed in Chapter 6.

5.5.4 Verification of finite element model for amplification effects

The soil model and results presented above only considered free-field stresses due to the explosion loading. The stiffness-strain relationship established above was again used to verify the amplification effects of buried structures. A damping ratio of 0.3 % was also specified with stiffness-strain Profile 1 (Figure 5-7) in the finite element analysis. The mild steel buried targets at Site B (Chapter 4) comprised two sizes, 300 mm cuboidal and 600 mm x 300 mm x 300 mm rectangular targets, and were modelled using solid finite elements with a density of 7850 kg/m³. The air void in the target was coupled using an ideal gas equation of state with the solid elements to make them behave together.

The numerical results using stiffness-strain Profile 1 (Table 5-7) were substantially within 10% of the first peak that occurred in the time-histories of the field test results, and were considered acceptable. Figures 5-17 to 5-20 show the time histories for the incident pressure, radial acceleration, transverse acceleration and vertical acceleration, respectively, for a 0.3 m cuboidal buried target subjected to a 1 kg PETN charge at 1 m radial distance. These field and numerical results were selected for convenience of comparison with the free-field results presented above.

Figure 5-17. Incident pressure time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-18. Radial acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-19. Transverse acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-20. Vertical acceleration time-histories from field explosion test (1 kg PETN, 1 m radial distance) and numerical results for 300 mm cuboidal buried targets

The finite element model was also checked for consistency using test results from all the buried targets used in the field explosion tests, i.e. 1 kg, 4 kg and 10 kg PETN. The pressure-time histories for some of the buried targets used in 4 kg and 10 kg PETN charges are shown in Figures 5-21 to 5-24. As with the 1 kg PETN charge, the numerical results for the incident pressure showed a better match with the field test results compared to the acceleration results from the numerical analysis. The mismatch in time-histories was regarded as field and numerical errors, and the numerical results were accepted as the rise times and first peak values from the numerical analysis were substantially within 10 % of the field test values. The results from the finite element model were considered acceptable, and the numerical model was considered calibrated. The numerical model was used to undertake parametric analysis and the results are presented in Chapter 6.

Figure 5-21. Incident pressure time-histories from field explosion test (4 kg PETN, 1.57 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-22. Incident pressure time-histories from field explosion test (4 kg PETN, 3.17 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-23. Incident pressure time-histories from field explosion test (10 kg PETN, 2.15 m radial distance) and numerical results for 300 mm cuboidal buried targets.

Figure 5-24. Incident pressure time-histories from field explosion test (10 kg PETN, 4.31 m radial distance) and numerical results for 300 mm cuboidal buried targets.

5.5.5 Perceived importance of FE modelling components

The finite element analyses suggest that commercial FE programs such as LS-DYNA can be used to replicate field explosion tests and facilitate parametric studies. Although the 3D FE modelling required substantially more preparation and computing time, it was more appropriate to analyse the explosion problem using a 3D model since there was also a need to obtain amplification effects due to buried targets with varying geometry. The use of transmitting boundaries provides economy in mesh extent and computing time. LS-DYNA provides the mathematical background for the transmitting boundaries in their program manual and has suggested relaxation coefficients for various types of problems. It is important to undertake preliminary FE analyses with and without transmitting boundaries and assess if the reflected or transmitted waves appear reasonable. PETN explosive charge was used in the field tests and was converted to equivalent TNT charge by applying a factor. The JWL model is a popular high explosive model used to simulate TNT charge. In the case of PETN, there are well-established data to convert the PETN charge to equivalent TNT charge. Caution has to be exercised where other types of explosives, such as enhanced explosives are used, as the JWL model may not be applicable. The elastoplastic soil model appeared to be reasonably representative of the soils encountered in the field. The FE analyses were sensitive to the input Poisson's ratio in the soil model. The analyses were less sensitive to the input stiffness-strain values, especially at large shear strain values (G_{min}). Variations on the G_{min} had only marginal impact on the FE results. The effects of damping ratio (ξ) were also studied. Error compensation within the overall framework of the FE analyses could arise from a variety of factors. The Poisson's ratio and G_{max} values if poorly estimated can have a significant impact on the numerical results. Thus, if the Poisson's ratio had been slightly underestimated and G_{max} had been overestimated, the overall errors would be compensating. The errors resulting from a poor estimation of damping ratio (ξ) and G_{min} would have less impact on the numerical results.