Response of sand column under impact loading

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RESPONSE OF SAND COLUMN UNDER IMPACT LOADING

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SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING

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Abstract

Dynamic compaction (DC) has been used widely as an effective method in improving the mechanical properties of loose sand deposits often found in reclaimed land. The process involves dropping a large pounder onto the ground surface, causing a densification of the soil as the impact stress wave propagates downward. In the case of near-saturation condition in reclaimed land, this impact stress might even produce a possible liquefaction of the sand. As not much research have been done to observe the pore-water pressure response of nearly saturated sand undergoing impact loading, the occurrence of such a liquefaction phenomena remains difficult to be firmly established.

Therefore, this new experimental apparatus was designed to investigate the transient pore-water pressure response of nearly saturated Ottawa sands for any possibility of liquefaction and its dynamic properties during impact loadings. The specimens were dynamically loaded within a 1050 mm rigid steel pipe of 150 mm diameter by dropping a steel pounder of 110 mm diameter on it at varying heights of 0.3 to 0.5 m. Boundary conditions were for one-dimensional confined compressive loadings with drainage. The new design was found to be adequate in capturing the immediate and residual pore-water pressure with millisecond rise times to peak stress. From the stress-strain relationships of nearly saturated sand, we observed a non-linear behavior, deviating from Ginsburg's model. A high risk of liquefaction was also observed from its residual pore-water pressure at high loads.

The sand's drainage length, initial relative density of sand, number of impact drops and energy input are found to be significant factors that determine the degree of liquefaction. The magnitude of the peak pore-water pressure was observed to have an increasing trend in accordance with any increase in the above four parameters. The magnitude and duration of the residual pore-water pressure was however found to follow an inverse relationship to the increase of drainage length, relative density of sand and the number of drops. Only the increase of energy input causes an increase in the magnitude and duration of the residual pore-water pressure.

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Chapter 1 Introduction

1.1 Background

Loose nearly saturated sands are frequently found in mine tailings and reclaimed sites using hydraulic fill. They can also be formed under natural conditions. Such soils have low bearing capacity and especially vulnerable in earthquake prone areas, where they are susceptible to the risk of liquefaction. Ground improvement is thus introduced to improve the engineering characteristics of the soil.

There are several methods which have been found to be effective in improving the mechanical properties of loose sand deposits. Some of the established ways include vibrocompaction, vibroflotation, sand-column placement, the use of admixtures and dynamic compaction. Among these, the most common method adopted in Singapore's land reclamation projects is the dynamic compaction (DC).

In dynamic compaction, the ground is compacted by the repetitive impact of the soil by dropping a heavy weight from a certain height. Typically, the pounder may range from 5 to 40 tonnes and the drop height can vary from 10 to 30 m. It has been found that DC is an economical method for the improvement of a large area with the tolerance that the vibration caused by the pounder does not pose severe repercussions to its surroundings.

Heavy tamping as some described the dynamic compaction process is usually carried out in a few phases. The impacts are carried out at a predetermined grid pattern for a specified number of blows. The crater created by the dropped weight is then backfilled before the next phase. Subsequent passes are performed on progressively closer spacing with an ironing pass to conclude the final phase. This closing pass which is usually of a lower energy is to enable the compaction of soil closer to the surface.

The operating principle of DC in dry deposits is fairly straight forward and well-understood. The falling impact produces compressive and shear waves that are sufficiently large to overcome the intergranular locking stresses within the loose strata. As a result, the particles are rearranged into a denser configuration. However, the mechanism of densification for a nearly saturated granular deposit is quite different. The compressive stresses induced by the DC impact result in a large zone of pore water pressure increase

which may progress towards a state of liquefaction as observed by some researchers. Rearrangement of particles packing and decreased void volume will then occur during, and immediately after the dissipation of the excess pore water pressure resulting in a denser soil.

There is therefore a need to understand the effect of various impact parameters on its soil dynamic response. Presently, a rational or well-established design procedure is not available to take these into account. As a result of this lack of understanding, the ability of the chosen operational parameters to meet the specification has to be determined by a trial run on the actual site. This time consuming and expensive procedure might result in undue delay to the project causing even more waste of resources.

It is thus believed that a deeper understanding of the response of nearly saturated sand to dynamic, high intensity impact will be of much value to an efficient design of future dynamic compaction projects. The possible occurrence of liquefaction under such dynamic loading is not only of interest to such a soil improvement method but also enables one to investigate further the immediate and residual pore-water pressure response of sands under dynamic shock loadings in general.

A preliminary assessment reveals that a general three-dimensional response of sand to impact would be complex and not easily amenable to experimental investigation. Laboratory and field data have also shown that the compacted soil mass is confined predominantly to an area directly below the pounder, with a smaller degree of improvement laterally. Furthermore, with the high impact energy, the column of soil underneath of pounder has a tendency to punch through the adjacent soil. For simplicity, the compaction of soil can be considered to be largely one dimensional. Hence, an understanding of the behavior of nearly saturated sands under one-dimensional impact may prove to be more valuable on the compaction process.

1.2 Objectives of the study

In an effort to understand the mechanism of dynamic compaction in nearly saturated sands, this experimental study was initiated with the following four objectives:

1. To design a new pore-water pressure measuring system that can filter off the sand horizontal pressure from its pore-water pressure in order to measure the pore-water pressure response of nearly saturated sands under an impact loading.
2. To study the effects of various impact parameters like the relative density of sand, number of impact drops, energy input and sand's drainage length, on the sand's immediate and residual pore-water pressure response.
3. To derive the stress-strain relationship of nearly saturated sand column during an impact loading.
4. To investigate the possibility of liquefaction occurrence in the nearly saturated sand column due to impact loading.

1.3 Scope of the Study

The scope of this research is limited to the following four main constraints:

1. The response of the sand column under impact loadings was investigated based on a one-dimensional study rather than an ideal 3-dimensional full analysis.
2. The type of sand specimen used was restricted to nearly saturated Ottawa sand. Hence the conclusion of any results in this study might not be fully applicable to every other sand sample.
3. The impact loadings simulated were concentrated on relatively low impact energy. A broader spectrum of possible impact energy on the field was not fully stipulated in this research.
4. A partially-drained condition was assumed at the surface of the sand column. Total undrained analysis was not considered in this study.

1.4 Outline of Report

This chapter gives a brief introduction of the project. A critical review of the literature on relevant work in the area of the DC is provided in Chapter 2. Special emphasis is given to the design and analysis aspects of DC. Several case histories of laboratory and field tests are summarised and the major findings are compared and discussed. A discussion on the pore-water pressure response from past researchers will also be included with a focus on the liquefaction phenomena. In Chapter 3, the design of the test equipment and the testing procedure are discussed. The various instrumentation for dynamic measurement are described in detail. A new pore-water pressure measuring system is presented. The basic characteristics of the sand used in the test are also discussed. Chapter 4 presents the results of the impact stress at the surface of the sand column derived from its deceleration records. The stress-strain relationship of a nearly saturated sand will be presented here. Chapters 5 and 6 goes on to investigate on the immediate and residual pore-water pressure response of the sand column and the effects of various impact parameters. Then, in Chapter 7, we present the results of the total horizontal stress during impacts and derived the coefficient of dynamic lateral earth pressure. An additional test was also performed to study the effect of column length of the pore-water pressure response as reported in Chapter 8. Finally, the conclusions developed from the research are presented along with recommendations for additional research in Chapter 9.

Chapter 2 Literature Review

Dynamic loadings on soil can be produced from several different sources. The most common place of occurrence in civil works can be seen in dynamic compaction where steel weights are dropped from a height to improve the properties of underlying soil. An explosion, whether on the ground surface or buried, also creates dynamic loading. The surface explosion actually 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. Although potentially dangerous, 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). Another major source of dynamic loadings comes from earthquakes. The cyclic shear stress transmitted from the plate movements of the earth crust can cause great damage directly to overlying structures. It also resulted in an increase in the pore-water pressure of the soil that cause further liquefaction hazards to the foundation.

In view of the limited understanding of the effects of dynamic loadings on geotechnical engineering, Woods (2000) proposed that the major soil properties and characteristics to be focused in soil dynamics, are:

- a. Dynamic moduli, which include Young's modulus, shear modulus, bulk modulus and constrained modulus.
- b. Poisson's ratio.
- c. Damping and attenuation.
- d. Liquefaction parameters, which include cyclic shearing stress ratio, cyclic deformation and pore-water pressure response.
- e. Shear strength in terms of strain-rate-effects

Particular to the study of our research, an emphasis will be given in investigating the pore-water pressure response.

2.1 Some differences among sources of dynamic loading

There are a few characteristics that can be used to distinguish the various types of dynamic loadings:

- The constant motion of the earth's surface causes earthquakes. The earth's rock layer is broken into large pieces. From time to time, the pieces may lock together and 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 (Earthquake Engineering Research Centre, EERC, 2000). On the other hand, shock loadings like dynamic compaction produce mainly P-waves which are compressional in nature.
- Given the very short duration and accompanying small displacements, explosive sources carry negligible energy compared to nuclear or earthquakes 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 produced in dynamic compaction has a single dominant frequency and can be computed using wave theory. In buried explosions, the reflections 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 earth 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, we will be focusing on shock loadings as one of the sources of dynamic loadings. We will therefore single out dynamic compaction as our principal study source of such impact loads.

2.2 Dynamic Compaction

The history of using drop weight to compact soil dates back to antiquity. The dynamic impact method has been used as a technique for compacting thin layers of compressible soils. It was not a common practice initially to treat soils of large area and great depth. It is only with the development of modern engineering plant, that the method has become practical. The major applications of dynamic compaction (DC) undertaken in recent years include the soil improvement scheme of hydraulic fill prior to the construction of Changi Airport in Singapore (Choa et al. 1979) and Nice Airport (Gambin, 1983) in France.

Ultimately, the objective of DC in a construction site is to improve the geotechnical properties of loose soil and other deposits. Lukas (1992) listed the following results achieved by dynamic compaction:

- a) an increase in bearing capacity and a reduction in compressibility of various types of soil deposits beneath buildings, slab-on-grades, or embankments.
- b) a reduction in liquefaction potential of sands or silty sand deposits beneath earth dams.
- c) densification of surface mine spoil or former landfills to allow for construction of highways or buildings over these deposits.
- d) densification of sinkhole and karst deposits to reduce the risk of large cavities in foundation soil.
- e) densification of collapsible soils beneath highway pavements to reduce pavement irregularity.
- f) densification of nuclear or hazardous waste deposits to reduce the risk of future subsidence of the surface cap.

2.2.1 Factors governing the effectiveness of dynamic compaction

It has been suggested that the most important factor governing the effectiveness of DC is the rate of excess pore water pressure dissipation (Lukas, 1986). During dynamic compaction, the soil particles are displaced into a tighter state of packing and if water is present in the voids of the soil, it is necessary for the excess pore water pressure to dissipate before additional densification under repeated blows can take place. If pore water pressure dissipation does not occur, then repeated blows from the tamper impacting the ground will only cause displacement of the ground and not densification.

Soil grain-size distribution, degree of saturation, permeability and distance of drainage paths are some of the other factors that affect the effectiveness of DC. Figure 2.1 and Table 2.1 show the range of deposits suitable for DC. During compaction, it is generally accepted that compaction is only possible if the excess pore pressure dissipates. Hence, coarse-grained materials in zone 1 with high permeability appear to be suitable for compaction by DC. The coefficient of permeability in this zone is above 1×10^{-3} cm/s. These deposits range from boulders to fine sandy silts or silts with no clay size particles, and has a plasticity index of zero. Uniform sands and silty sands have been the main soil types used in DC. However, necessary delay time must be introduced between passes in conjunction with the increase of silt content, to allow for the dissipation of excess pore water pressure (Menard and Broise, 1975).

On the other hand, relatively impervious fine-grained soils such as clays and saturated organic deposits are not easily compacted by DC. When impacted, the soils will develop excess pore pressure quickly and considerable duration is required for dissipation of pore pressure. As these soils have low strength when saturated, tamping will only cause large shear displacement without densification.

2.2.2 Theoretical principle of dynamic compaction

Although the operation of DC seems to be simple, the theoretical principle behind DC is actually very complex. Van Impe (1989) pointed out that the impact of a falling weight on ground surface induced shock waves in the subsoil (Figure 2.2). Out of which, the three most important elastic waves produced are compression, (or longitudinal), shear (or transversal) waves and Rayleigh waves.

Compression waves or P-waves (Figure 2-3a) have the greatest velocity where the directions of motion of the particles are in the direction of propagation. Shear (S) waves (Figure 2-3b) are distortional traverse waves with the direction of motion of particles perpendicular to the direction of propagation. P-waves can propagate through all medium, solids, liquids and gases but S-waves cannot propagate in fluids, as fluids have no resistance to shear. In a uniform infinite medium, only P and S waves appear. However, near the surface of a homogeneous stratum there arise also mixed compression-shear waves known as Rayleigh waves (Figure 2-3c). Rayleigh waves have retrograde and elliptical particle motion, similar to the waveforms produced when a stone is dropped into a pond.

In an attempt to explain the mechanism of DC, Scott and Pearce (1975) working in particular to coarser-grained soils above ground water table, proposed that the compaction under impact could be caused by the overcoming of inter-granular stresses by shock wave and the inertia forces of the pounder. The consequence is the closure of voids between particles. For saturated coarse-grained soils, the water can be expelled from the voids allowing the repacking of soil particles into a denser state.

On the other hand, with regard to dynamic consolidation of fine grained soils, Menard and Broise (1975) presented a model which differs from Terzaghi's model (Figure 2.4). They suggested that the compaction of saturated soils is due to:

- 1) the compressibility of saturated soils arising from the presence of micro-bubbles which constitutes approximately 1-4% of the total soil volume.
- 2) The gradual liquefaction under repeated impact after the gas is displaced.
- 3) The increase in permeability due to the formation of fissures and the state of near liquefaction.

4) Thixotropic recovery.

Later, Gambin (1979) explained that in relatively impervious soils, the compression stress wave travels mostly through the water phase and shakes the soil skeleton by successively increasing and decreasing the pore water pressure until this skeleton dislocates. The shear wave and Rayleigh wave which travel at a lower velocity subsequently pack the particles into a denser state. As the pore water pressure dissipates, the particles move closer together under the overburden confining pressure. This increases the compactness of soil and is known as thixotropic recovery. Van Impe (1989) supported this hypothesis later by pointing out that compression waves are mainly responsible for the rupture of the existing soil skeleton structure and the break up at the surface of the soil layer. The transverse waves then make for easier compaction of the soil in the deeper layer.

The research of Greenwood and Kirsch (1984) revealed further insights that DC induces punching shear and compaction displacement in the surface soils as the momentum of the falling weight decays. There is also an element of impact and shock, but the waves are of low frequency (up to 10 Hz) and are rapidly damped in saturated soil.

2.2.3 Depth of influence

Dynamic compaction is a kind of deep compaction method where the influence depth is a primary concern. Upon the drop of the pounder, the magnitude of the impact stress attenuates rapidly from the impact point. Henceforth, the ground improvement on the site increases with depth.

Forssblad (1963) stated that as the soil near the surface is compacted, the propagation of stress wave becomes more effective and therefore extends the compacted zone to greater depth. However, as the soil layer closer to the surface is compacted, the side restraint may become more dominant. This side restraint could possibly be more significant than the frontal and pounder inertia force. As a result, the dissipation of energy increases and limits the effective compaction depth.

In an effort to investigate further the effectiveness of dynamic compaction, Lukas (1980) did a study on the measured lateral deformation of soil during DC. The data indicated that the first 2 to 5 drops significantly extend the depth of improvement. Subsequent drops

were found to improve the effective depth slightly, with the compaction zone spreading laterally. Considering this, it is advantageous to employ a greater drop height for the initial pass to extend the compaction depth.

With the widespread use of dynamic compaction, it becomes increasingly important to predict the depth of improvement in a DC project. In response to this need, Menard and Broise (1975) came up with an empirical equation:

$$D = \alpha(WH)^{1/2} \quad (2.1)$$

where D is the improvement depth (m), W is the pounder weight (tonne), H is the drop height (m) and α is the reduction coefficient which is equal to one, for a weight release in free fall.

Several researchers adopted this empirical equation by introducing different values of α . Leonards et al. (1980) compared the relationship between the energy per drop and the depth to which significant densification took place and reported that a coefficient of 1.0 overestimated the effective depth. He found that a α of 0.5 gives a better representation of the field condition (Figure 2.5). In response, Lukas (1980) estimated that α ranges from 0.65 to 0.80 based on his experience on densification of free drainage deposits above ground table. In another separate research, Mayne et al. (1984) collected site measurements from over 120 sites and proposed that α falls within 0.3 to 0.8 (Figure 2.6). Therefore, a coefficient of 0.5 can be used conservatively as an estimate for the influence depth.

Looking closer at Equation 2.1, it was noted that the author made an assumption of the influence depth as primarily a function of the impact energy. This equation ignores the effects of other important factors such as soil type, soil properties, pounder dimension, the presence of soil layer and the ground water table. The α values proposed by others also indicated that they were simply selected to match site data. The definition of influence depth is also far from rigorous. The improvement at the bottom of the influence depth, whether zero improvement or a certain degree of compactness, is usually not accounted for.

Other than site experience, attempts have also been made to assess the problem based on laboratory tests. Fang and Ellis (1983), and Ellis (1986) went out to introduce a new procedure to determine the effective depth of compaction. They carried out laboratory studies of DC on several fill materials under dry and saturated conditions. The test set-up included a compacted silty clay base and fill material of varying depth contained in a metal drum. The ground response was measured with the pressure cell installed on the top of the clay base and the crater profiles being measured after each drop. A relationship was then deduced from laboratory tests and the effective depth was given by,

$$D = \Psi(WH)^{1/2} \quad (2.2)$$

where Ψ is the coefficient of effective depth. The effective depth is defined as the depth which corresponds to a critical ground response. The critical ground response was defined as one in which a reduction in the response will not cause significant differences in D , while an increase in the value will result in an overly conservative D .

Ψ values were plotted against the ratio of crater radius depth, r_c/δ_c as shown in Figure 2.7. For saturated condition, D is defined as the crater depth because the effect of pore water pressure limits the compaction depth. The parallel lines are drawn in between the two extremes to approximate the effect of various degrees of saturation.

Several numerical models have also been proposed to compute the effective depth of compaction. Scott and Pearce (1975) suggested that the impact momentum in DC is generally high and the shear restraint acting on the soil flanking the compacted zone is insignificant when compared to the frontal force. Therefore the dominant mechanism is a punch-through failure mode, involving a block of soil directly under the pounder.

2.3 The effect of impact parameters

Extensive experimental studies have been conducted in an attempt to understand the impact mechanism. Thus naturally, the various factors governing the soil compaction are vital for determining the optimum parameters for compaction. This section presents some of the findings of past researchers in this area.

2.3.1 Three-dimensional model tests

Orrjie and Broms (1970) reported a laboratory test of DC. The effects of drop height, pounder diameter and pounder mass were investigated. They found that the peak deceleration of pounder increased proportionally to the dry unit weight of sand. It was reported that the dynamic loads were proportional to pounder diameters and pounder mass, and increased with the increase of drop height. For comparison, static plate load tests were also conducted. It was found that the bearing capacity factors from dynamic tests are higher than the factors from static plate load tests.

Jessberger and Beine (1981) developed special laboratory apparatus which can be used to measure dynamic stresses in the soil after impact. The relationships between pounder sizes, pounder mass, drop heights and densification depths were investigated. Jessberger and Beine (1981) then proceed to estimate the stress distribution under dynamic loading using the two equations from Fröhlich (1934) and Köger & Scheidig (1948). The dynamic stress at depth z to break down the soil structure $\sigma_{z,dyn}$ (in kN/m^2) is given:

$$\sigma_{z,dyn} = \alpha_a \cdot \frac{m}{A} \sqrt{2gH} \left(1 - \frac{z}{\sqrt{z^2 + r^2}} \right)^\eta \quad 7 < \eta < 15 \quad (2.3a)$$

$$\sigma_{z,dyn} = \alpha_a \cdot \frac{m}{A} \sqrt{2gH} \left(1 - \frac{z}{r} \tan \theta_0 \right)^{-2} \quad 15^\circ < \theta_0 < 25^\circ \quad (2.3b)$$

where

m = mass of the pounder (kg),

A = base area of the pounder (m^2),

H = drop height (m),

r = the radius of the loaded area (m),

α_a = proportionality constant,

g = the acceleration of gravity (m/s^2),

z = depth (m),

θ_0 = angle of load distribution,

η = factor of concentration.

A design chart was given for selecting of the drop height, pounder mass and base area of pounder to obtain optimal treatment of a certain compaction project.

Beine (1983) conducted laboratory tests to investigate the stress distribution in sand under impact loading. A test procedure was developed to estimate the effects of the mass of the pounder, drop height, distance of drop units and numbers of passes and blows on the influence depth. The test device, which was used to investigate the stress distribution in sand compacted by dynamic compaction, is shown in Figure 2.8. The soil used in the test was fine sand, and was rained into the cylindrical steel container of 63 cm in diameter. Stress transducers were installed in different depths and distances from the impact axis.

The total static and dynamic stresses were measured. The soil was loaded by impacts of a free falling pounder with variable diameter and mass. During impact, the deceleration of the pounder was also measured. The stress at the soil surface can be calculated from measured deceleration, mass and base area of the pounder using Newton's law.

The isobars of vertical dynamic stress σ_z related to static stress at the soil surface σ_0 are shown in Figure 2.9. On the left side of the center line the isobars are plotted for static stresses caused by the weight of the pounder, whereas at the right side of the isobars for the measured dynamic stresses at time of impact are drawn in relation to the static pressure at the soil surface. It can be seen that the impact load of the tamper reaches a considerable greater depth than a static load with the same load area, but is restricted to a limited region near the load axis.

Fang and Ellis (1983) presented results of a laboratory test of DC on three types of fill material, namely clay, sand and fly ash. The effects of the pounder weight and area, drop height, and number of drops were studied. They indicated that the ground response is higher for a thin deposit, and the pounder impact area has significant influence on ground response for thin soil layers where the smaller area results in higher peak ground accelerations. In contrast to this, for thicker layers, the pounder size has little effect. Ground response is proportional to impact energy for all pounder sizes. However, the ground response is more sensitive to impact energy for all the smaller pounder sizes. It

was found that with the increase of number of drops, the deformation – drop height ratio increases.

Heh (1991) conducted a series of three-dimensional model tests in which a circular steel pounder was repeatedly dropped on dry sand. The dry sand was contained in a 1.22x1.22x1.22 m cubic steel tank. The measurements included the deceleration of the pounder, soil pressure at three levels of the sand, the tamping settlement and soil strain. At the end of each test, the densities of the sand were measured using a nuclear density instrument for every 15 cm thick layer. The effects of drop height, pounder weight, and contact area of pounder were investigated. A total of 21 DC tests were carried out and the program is shown in Table 2.2.

Heh in 1991, introduced the new concept of dynamic settlement modulus (DSM). The DSM was defined as the slope of the tangent of the loading portion of the impact stress-strain curve, and stated as:

$$DSM = \frac{\Delta p}{\Delta \left(\frac{w}{d} \right)} \quad (2.4a)$$

where

p = the impact stress (N/m²),

w = the displacement of pounder (m),

d = the diameter of pounder (m).

The range of elastic properties for this sand from static triaxial and oedometer test results are shown in Table 2.3 (Poran, 1992). DSM values showed good correlation to soil densities and corresponding elastic moduli obtained from laboratory tests for loose sand. However the results showed discrepancies for dense sand.

Heh (1991) computed and plotted density contours. The density contour could be approximated as an upside-down prolate spheroid with dimension of a and b for horizontal and vertical radii, respectively, as shown in Figure 2.10.

The relationship between the normalized total specific energy nWH/A and the normalized ratios of b/d and a/d for different relative density contours are shown in Figure 2.11 and 2.12, respectively. Two best linear fit curves was determined as:

$$\frac{b}{d} = j + k \log\left(\frac{nWH}{A}\right) \quad (2.4b)$$

$$\frac{a}{d} = l + m \log\left(\frac{nWH}{A}\right) \quad (2.4c)$$

where

n = the number of drops,

W = the pounder weight (kN),

H = the drop height (m),

d = the diameter of pounder (m),

A = the area of the contact surface (m^2),

b = vertical radii of upside-down prolate spheroid (m),

a = horizontal radii of upside-down prolate spheroid (m),

j, k, l , and m are linear regression curve fitting parameters shown in Table 2.11.

Equations 2.4b and 2.4c were used to establish design charts in which “least”, “average” or “most” improvement were proposed.

Yin (1998) conducted a small-scale 3D model test and discovered that as compared to one-dimensional confined sand column test, lateral confinement of the latter affected the depth of crater and the degree of improvement of sand by dynamic compaction. It was also discovered that the peak value of the deceleration of pounder after impact tended to increase with increasing drop number, while the duration of dynamic interaction decreased. The behavior of saturated sand under impact also was found to show some similarity to those in dry sand.

2.3.2 Centrifuge model test

Mikasa et al. (1988, 1989) carried out centrifuge tests to simulate the soil response under different tamping conditions. The ground was modeled to be a semi-cylindrical column, the center of which was tamped repeatedly through a semi-cylindrical rod by a ram which was automatically operated. A glass plate for the observation of the ground deformation was installed on the vertical front face of the model ground.

Table 2.4 shows the test results while Figure 2.13 illustrates the ram penetration (S_p) against the number of drops (N) under different kinetic energies. The intersection of N - S_p lines with $S_p=0$, was taken to correspond to the condition before compaction starts. This figure indicates that the ram penetration has a linear relationship with $\sqrt{N}$. Increasing the ram momentum twice reduced the required number of blows by a quarter, suggesting that the value of $mV\sqrt{N}$ is always constant. Table 2.4 indicates that the product of the ram momentum per unit area and the square root of blows always shows the same value of 485, with a few exceptions. Combining these results with the linear relation between N - S_p yields,

$$S_p = \gamma \frac{mV}{A} \sqrt{N} \quad (2.5)$$

where γ is a constant depending on the ground conditions. This equation shows that ram mass and impact velocity are important factors in ram penetration. Figure 2.14 indicates that the volume reduction is largely influenced by the tamping energy. Figure 2.15 shows the total energy required for a ram penetration of 3 m. It is discovered that the required energy level did not change a great deal for a particular ram mass indicating that the N value needed to get 3 m of penetration is almost inversely proportional to the drop height. Mikasa et al. (1988) also found that the compact zone was concentrated at the area underlying the poulder. This could mean that the lateral spreading effect of the impact was negligible due to the predominant punch-through mechanism.

Takada and Oshima (1994) conducted a centrifuge model test under an acceleration field of 50 g. An actual test was carried out to investigate ground deformation and earth pressure beneath the tamping hole. The field test was done on a loose and homogeneous

sandy fill, 10 m x 10 m in area and 6.5 m in thickness. The pounder mass was 20 tons, base area of pounder was 4 m², drop height was 20 m, and number of blows was 20. The measurements included ground deformation, maximum vertical and horizontal stress.

The set-up for the centrifuge tests were similar to that reported by Mikasa et al. (1988). The model ground was an axi-symmetric cylindrical column 30cm in diameter. The ram was a semi-cylindrical wooden column positioned to strike the center of the model ground. A 13 cm thick model ground was prepared which was used to simulate a prototype fill 15 m in diameter and 6.5 m thick. After tamping, cone resistance at various distances from the center was measured using a special device.

The ground deformation in the field tests of Takada and Oshima (1994) is shown in Figure 2.16. It was found that the ground deformation occurred mainly beneath the tamping hole. Special load cells were used to measure the maximum earth pressure occurred during tamping. The maximum vertical earth pressure recorded at 4 m from impact center increased with depth, while the maximum horizontal earth pressure and the horizontal displacement were the greatest at depth of 5 m. The maximum vertical earth pressure at the center is shown in Figure 2.17; it decreased rapidly with depth.

A comparison of ground deformation in the field test and in the model is shown in Figure 2.18. Both tests showed similar ground deformation. It was found that the relative ram penetration in the centrifuge model was slightly greater than that in the field and the distribution of volumetric strain was similar to that of the rate of increase of the cone point resistance as seen in the comparison between Figure 2.19 and Figure 2.20.

2.3.3 Confined sand column test

Law (1996) studied the behavior of confined dry sand column under the impact of falling weight. Figure 2.21a shows the set-up of the experiment. The sand was contained within an instrumented hollow steel pipe measuring 150mm in diameter. Three horizontal pressure transducers were installed along the wall of the steel pipe. Another pressure transducer was installed at the base of the steel pipe to measure the vertical stress. A dynamic impact was triggered by dropping a pounder from a pre-set height through the use of an electromagnetic mount. The progressive changes in sand density due to repeated impact and the effect of impact parameters on the changes in sand density were investigated. Chow et al.'s (1992a & b) one-dimensional wave equation model was modified, and the experimental results were back analyzed using this model. His work reveals the following observations:

- The peak deceleration upon impact increases as the number of blow increases.
- The increase in peak deceleration is accompanied by a shortening of the deceleration pulse duration.
- The permanent settlement of the sand column due to each drop of the pounder reduces as the number of drop increases.
- When the sand column is loose, the stress waves are found to attenuate rapidly. As the sand stiffens under repeated impact, the peak stresses monitored at different locations increased correspondingly
- The degree of compaction along the sand column varies from dense at the top to loose at the bottom.
- Although there is a close relationship between the total amount of input energy and densification of the sand column, there exists a saturation energy level beyond which no densification can be achieved with further pounder impact.
- For a given impact momentum, a higher impact velocity with a corresponding lower pounder mass, produces a greater compaction at deeper depths.
- It was found that the calculated results agreed reasonably well with the measured ones (Fig 2.21b).

Law's 150mm diameter vertical stainless steel pipe's design provided a good starting point for a proposed experimental design described later in this thesis. Maintaining a vertical

impact through the 150mm wide steel pipe, the adapters for the pressure transducers were re-designed to accommodate the measurements of pore-water pressures in a wet sand column. The locations of the pressure transducers were also re-located to capture the crucial pore-water pressure response of the sand layers.

2.4 The Liquefaction Phenomenon

The effects of liquefaction have traditionally been made in connection with earthquake loadings of soils. A devastating earthquake in 1964 fell on Valdez, Alaska, destroying harbor facilities, roadways, buildings and many homes. After investigation, the cause of accident was attributed in part to a large flow of liquefied soil as suggested by Seed (1968). Occurring in that same year, another major earthquake occurred in Niigata, Japan, causing extensive structural damage as a result of bearing capacity failures. Many buildings sank into the ground, causing even a sewage treatment tank to float up above the ground. An apartment building was seen to be rotated intact almost 80 degrees (Seed and Idriss, 1967). Following this traumatic experience, an interest was started among many researchers like Seed (1968, 1979), Green and Ferguson (1971), and Gilbert (1976) to further study the earthquake-induced liquefaction phenomena.

After recognising the catastrophic consequences associated with seismic ground motions from earthquake loadings, the scope of research begins to expand to investigate any likely hidden danger that could also occur from shock loadings. Indirect observations of construction, military and small-scale field explosive events formed the immediate sources of information in this area. It was discovered that where liquefaction is evidenced, water spouts, sand boils, ground subsidence, and flow of soils also followed the liquefaction explosion (Gilbert, 1976; Charlie, 1978a,b; Melzer, 1978; Benson, 1983; Blouin and Kim, 1983). Gilbert (1976), and Blouin and Kim (1983) believed that the unusually broad, flat crater shapes observed are the result of surface and near-surface detonations in saturated soils, and this may be an indication that liquefaction could occur even in explosive loadings. Explosive loadings thus became a subject of importance as it poses a twofold potential for damage, including the direct effects of the explosion, as well as the additional consequences of soil liquefaction. Works by Gilbert (1976), Rischbieter (1977), and Charlie *et al.* (1980, 1985b) further support the occurrence of this explosive-induced liquefaction phenomena.

The effects of liquefaction from explosion-induced ground motions are not restricted to surface damage as both above ground and buried structures are subjected to potential collapse. Therefore, the possibility of explosive-induced liquefaction also becomes a significant consideration for nearby structures. The resulting damage primarily involves bearing capacity failures which include excessive settlements, upward movements and

rotations. While such occurrences may not necessarily damage a facility structurally, as observed during the Niigata earthquake, they may render it unusable from a practical viewpoint.

2.4.1 Definition of Liquefaction

The term liquefaction is generally associated with the response of loose, saturated sands to loading conditions that induce large strains resulting in flow slides; or to describe a state of stress existing where the effective stress approaches zero; or when the reduction in soil shear strength is substantial enough that the soil behaves like a liquid.

Casagrande (1936) made one of the first attempts to qualify this phenomenon of liquefaction. Since then, there have been increased awareness and interest in liquefaction, particularly by researchers investigating seismic soil response. Whitman (1969), and Finn (1972, 1981) provided subsequent reviews of test procedures for dynamically loading soils and liquefaction analyses. From these investigations, a number of definitions and terms associated with liquefaction have been developed (Seed, 1976; Committee on Soil Dynamics, 1978).

Several definitions of liquefaction and related observations are given as follows:

Liquefaction (Seed, 1976):

“Denotes a condition where a soil will undergo continued deformation at a low residual stress or which no residual resistance due to the build-up of high pore-water pressures which reduce the effective confining pressure to a very low value; pore pressure build-up leading to true liquefaction of this type may be due to either static or cyclic stress application.”

Initial Liquefaction (Seed, 1976):

“Denotes a condition where, during the course of cyclic stress application, the residual pore-water pressure on completion of any full stress cycle becomes equal to the applied confining pressure; the development of initial liquefaction has no implications concerning the magnitude of the deformation which the soil might subsequently undergo; however it defines a condition which is a useful basis for assessing various possible forms of subsequent soil behavior.”

Cyclic Liquefaction (Seed, 1976):

“Denotes a condition in which cyclic stress applications develop a condition of initial liquefaction and subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil stabilizes under the applied loads.”

Liquefaction (Committee on Soil Dynamics, 1978):

“The act or process of transforming any substance into a liquid. In the cohesionless soil, the transformation is from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress.... Liquefaction is thus defined as a changing of states, which is independent of initiating disturbance that could be static, vibratory, sea wave, or shock loading, or a change in groundwater pressure. The definition also is independent of deformation or ground failure movement that might follow the transformation. Liquefaction always produces a transient loss of shear resistance but does not always produce a longer-term reduction of shear strength.”

The research presented herein investigates the phenomenon of liquefaction, as a result of impact loadings, in terms of the pore-water pressure response relative to the initial effective stress and initial soil density, definition by Seed (1976) for ‘initial liquefaction’ and that for ‘liquefaction’ by the Committee on Soil Dynamics (1978) are adopted. As such, liquefaction is evaluated by comparing the amount of pore-water pressure increase above the hydrostatic pressure, remaining after the dissipation of the compressive stress wave, and is also referred to as the ‘residual excess pore-water pressure.’

2.4.2 Factors Influencing Liquefaction

The majority of studies to date on the liquefaction potential of sand were considered mainly under cyclic loading conditions. Little information is available about liquefaction induced by dynamic compressive stresses from sources such as explosions, pile driving, impact loadings and dynamic compaction. To expound our knowledge of liquefaction under such shock loadings, we present a list of critical parameters that were found to affect the liquefaction potential of sand based on a limited research done in this area.

2.4.2.1 Energy Input

The first most obvious factor in determining the liquefaction potential is the amount of disintegrating energy discharged into the sand during an impact. Veyera (1985) has found that the peak compressive strain produced during a shock loading provided consistency and reliability to represent the amount of impact energy necessary to liquefy the sand particles. Liquefaction was noted at compressive strains between the ranges of 0.05 to 1 percent. In dynamic compaction, this impact energy is usually represented as a function of its pounder weight and drop height. Generally speaking, the greater the energy input, the higher the chances of liquefaction in the sand.

2.4.2.2 Degree of Saturation

Where the degree of saturation is less than 100%, induction of liquefaction is difficult. Even small amounts of air can significantly reduce the bulk modulus of the soil-water mixture, and the compressive stress wave propagation velocity. Using the equation for the compressibility of mixtures, and the wave propagation velocity in mixtures, Richart *et al.* (1970) demonstrated that the presence of 0.01 percentage air in a saturated quartz sand will reduce the bulk modulus of the soil-water system by a factor of 16, and reduce the wave propagation of velocity by a factor of four. The presence of entrapped air act as a 'cushion' by absorbing the compressive stress during deformation thus reducing the transient, and long-term pore-water pressure increases.

2.4.2.3 Initial Relative Density

Liquefaction susceptible soils have a significant potential to decrease in volume. Earthquake-induced liquefaction typically occurs in soils with relative densities of less than 65 percentage. Loose soils have increased in relative density successfully by as much as 40 percentage through explosive loading which generally results in the soil mass being more stable. While some dense soils may become loose, and this has been attributed to the dilative behavior of dense soils, which exhibits a volume increase accompanied by a reduction in pore-water pressure. Investigations by Florin and Ivanov (1961), Seed and Idriss (1967), Finn (1972), Allen (1975), Gilbert (1976), Rischbieter (1977), Allen *et al.* (1980), and Mitchell and Katti (1981) found similar observations.

2.4.2.4 Permeability

The duration of the liquefied state depends primarily on the permeability of the soil which is a function of soil type, density, grain size, packing, concentration, and fluid properties. It determines the rate of dissipation of the residual excess pore-water pressure produced during a dynamic loading.

High permeability soils, such as coarse sands and gravels, tend to drain rapidly and reduce the residual excess pore-water pressure quickly enough to inhibit liquefaction and thus restore the effective stress. Coarse silts and medium-to-fine sands have been observed to be liquefaction susceptible, as they are not capable of relieving the residual excess pore-water pressures in a short enough period of time. Low permeability of soils, such as fine silts and low sensitivity clays, on the other hand are usually not prone to liquefaction. Other permeability-related factors that also affect the duration of the liquefied state include the compressibility, stratum thickness, and drainage path length.

2.4.2.5 Initial Effective Stress

Generally, increasing the static overburden pressure increases the effective stress, and decreases the potential for liquefaction. With increasing effective stress, the stiffness of the soil skeleton is increased, and for liquefaction to occur, the pore-water pressure must reach a higher value. Florin and Ivanov (1961), Allen (1975), Studer and Prater (1977), Allen *et al.* (1980), and Mitchell and Katti (1981) have made similar observations in their respective studies.

2.4.2.6 Gradation

It has been recognized by many researchers (D'Appolonia, 1968; True, 1969; Rischbieter, 1977; Studer and Prater, 1977; Studer and Kok, 1980) that for explosive and seismic loadings, uniformly graded soils with a narrow band of grain sizes tend to be more liquefaction susceptible than soils having a variety of particle sizes. The distribution of particle sizes in a well-graded soil is effective in filling pore spaces with soil grains, and this increases the particle-to-particle contact area and shearing resistance. Studer and Kok (1980) have noted substantial pore-water pressure increases in coarse granular soil under shock loadings in the laboratory. While Florin and Ivanov (1961) have demonstrated in

laboratory experiments that all sufficiently loose, cohesionless soils of any grain sizes can be liquefied under explosive loadings.

2.4.2.7 Particle Size and Shape

True (1969) observed that graded, angular sand shows more evidence of liquefaction than uniform, rounded sand under shock loadings in the laboratory experiments, while Studer and Prater (1977) observed liquefaction in crushed quartz sand during explosive and shock loadings. Based on these data, it appeared that it is possible to liquefy both angular and rounded cohesionless sands.

2.4.3 Laboratory Test

This section is a review of investigation of shock-induced liquefaction performed through laboratory tests.

2.4.3.1 Shock Tube Tests

Perry (1972) reported in his laboratory tests that in order to determine whether variable density inclusions would sink or rise in wet sand, one-dimensional dynamic shock loading will have to be applied. Cubes of wood, wax, brick, aluminum and steel were placed in sand in a steel container, and saturation was attempted through adding of water and applying of a vacuum environment. After carrying out 13 tests, the highest computed saturation was found to be 99.91%, while the movement of the cubes did not yield any conclusive results.

At a later research, Studer and Hunziker (1977), and Studer (1978) tried to investigate the behavior of saturated sands in shock tube tests by exposing them to an air-blast wave, which induced a direct pressure into the sands. Three uniform grade quartz sands were used in this experiment, and the saturation obtained was between 99% to 99.5%. The results of 25 tests on five specimens showed a residual pore-water pressure existing in the sand and expulsion of water to the surface. The coarser grained sands behaved basically the same as the finer grained material, with the exception of the pore-water pressures dissipating more quickly. And in successive blast tests, they noted that the results were consistent, reporting even a multiple occurrence of liquefaction at times.

Studer (1978) also carried out several tests on soils undergoing shock loadings. He detonated charges inside a tunnel with a test container being placed just outside it. In all the tests, he found that the pore-water pressures increased and fluctuated from high peak values to negative values (less than static) and then to residual levels for a finite period of time.

Quaak (1978) made an attempt to contribute to the research of shock-tube tests through an experiment where he placed the sand in a tank and promotes saturation by adding water slowly from the bottom to create a loosely packed sand sample. The sand-filled tank was then placed in a shock tube, with a bunker model being put in the sand prior to the shock loadings. He observed that the bunker floated up within a short period of time after the ground shock was applied.

Veyera (1985), Hubert (1986), and Bolton (1989) also conducted laboratory experiments where fine, uniform sands were placed in a shock tube and subjected to one-dimensional, high amplitude, short duration compression loads. They were fully saturated and being tested at varying relative densities in undrained conditions using different types of sand. Veyera (1985) tested with Monterey No. 0/30 sand, Hubert (1986) tested with Eninearly saturatedok coral sand, and Bolton (1989) tested with Poudre Valley sand. It was found that liquefaction was generally more difficult to be induced when the density and effective stress of the sand increases. When the threshold compressive strain was less than 0.01%, no significant liquefaction could be seen. The occurrence of liquefaction was generally observed only at strains greater than 0.1% (Veyera, 1985).

2.4.3.2 Impact Tests

Tanimoto (1967) presented results of impact experiments on saturated, fine-grained sand. The sand was placed loosely in a container completely submerged in water, and an impact was applied to the container with a free falling hammer. After one impact, the pore-water pressure was observed to peak at 0.36 seconds near the bottom of the sand and gradually decreased to hydrostatic levels in 25 seconds. An approximately linear triangular distribution of settlement versus time occurred until 90 percentage consolidation.

Ruygrok and Van der Kogel (1980) discussed tests to simulate wave action in asphalt-covered sand-dikes. They were concerned if impact is applied to ocean waves on these dikes, what would the result be on the liquefaction potential of the soils beneath. In their experiment, a polyurethane “slab” was placed over a sand bed with varying porosity and moisture contents, and a falling weight impact load was applied to a circular area on the slab over the sand in a test bin. In three consecutive tests, liquefaction was reported whenever the pore-water pressure increases. In the shallower regions of the test bin, as the pore-water pressure rose, the liquefaction level remained for finite lengths of times (tens of seconds). While at deeper levels, the pore-water pressure showed liquefaction for only a few tenths of a second.

2.4.3.3 Blast Tests

Ivanov (1967) reported in his laboratory experiments to investigate compaction of Volga sand with surface, underwater, and buried detonations, using charges consisting of 1.5 gram (gm) explosives. In these tests, charges were placed on the soil surface, suspended in water above the sand, or buried sufficiently deep enough to prevent crater formation. The experiments were conducted in a concrete container with a sand height of about one meter, with the saturation of the sand at a level that is less than 100 percentage. The results showed that pore-water pressures increased for over 200 seconds after some detonations. Ivanov (1967) also reported on laboratory investigations with 1.5 to 4.5 gm charges placed at depths of 30 – 50 centimeters (cm). Immediately after detonations, gases “gushed” out at the surface, camouflets were formed, and heavy objects sank in. And after 10 to 15 explosions, pore-water pressures increments stopped.

Vesic *et al.* (1967) performed small scale cratering experiments by detonating hemispherical explosives charges in nearly saturated, dense sand placed in an aluminum

box. The sand was saturated through adding water from the bottom. These tests were performed to compare with previous studies carried out with dry sand, and significant differences in crater size and shapes were observed. The apparent volumes of the craters were about ten times greater than those observed in the dry sand, and the shapes were flatter and broader. They concluded that pore-water had significant effect on the cratering mechanisms.

In a laboratory investigation, Dowding and Hryciw (1986) reported that when explosives were detonated in a 102 cm high tank of loose saturated sand, the surface settlements, and maximum water rise in piezometer standpipes ranged from 0.8 to 1.7 cm, and 152 to 178 cm, respectively.

2.4.3.4 Quasi-Static Tests

Fragaszy *et al.* (1983) reported on a series of high pressure undrained, isotropic compression tests done on nearly saturated coral sands, Ottawa sands, and Monterey No. 0 sands. In his tests, a high pressure triaxial system was used on saturated soil samples and they were brought to an initial confining and pore-water pressure. Typical tests consisted of loading and unloading to the original value. One cycle of load-unload takes approximately 1 – 2 minutes. All the initial tests on nearly saturated sand resulted in liquefaction during unloading. A sieve analysis showed that significant particle crushing and cracking occurred in the nearly saturated sand and concluded that this breakage may have been a significant factor in the development of liquefaction. Although the Ottawa sands did not liquefy, the residual pore-water pressure increased by 50 percentage. Bolton (1989) conducted similar tests on water saturated Monterey No. 0/30 sand in confined loading-unloading, and he was able to liquefy the sand.

General Soil Type	Most likely Fill Class	Most Likely AASHTO Soil Type	Degree Of Saturation	Suitability For D.C.
Pervious deposits in the grain size range of boulders to sand with 0% passing the #200 sieve Coarse portion of Zone 1*	Building Rubble	A-1-a	High	Excellent
	Boulders	A-1-b	or	
	Broken Concrete	A-3	Low	
Pervious deposits containing not more than 35% silt Fine portion of Zone 1*	Decomposed Landfills	A-1-6	High	Good
		A-2-4	Low	Excellent
		A-2-5		
Semi-Pervious soil deposits, generally silty soils containing some sand but less than 25% clay with PI 8 Zone 2*	Flyash	A-5	High	Fair
	Mine Spoil		Low	Good
Impervious soil deposits, generally clayey soils where PI > 8 Zone 3*	Clay Fill Mine Spoil	A-6	High	Not recommended Fair-minor improvements, water content should be less than plastic limit
		A-7-5	Low	
		A-7-6		
		A-2-6		
Miscellaneous fill including paper, organic deposits, metal and wood	Recent municipal landfill	None	Low	Fair-long term settlement anticipated due to decomposition Limit use to embankments
Highly organic deposits peat-organic silts		None	High	Not recommended unless sufficient granular fill added and energy applied to mix granular with organic

* These zones are identified on Figure 2.1

Table 2.1 Suitability of deposits for dynamic compaction (after Lukas, 1986)

Test No.	Pounder weight (N)	Pounder diameter (cm)	Drop height (m)	Energy/drop (N-m)
21*	220	22.9	2.0	440
22*	220	22.9	1.0	220
23	220	10.2	2.0	440
24	220	10.2	1.0	220
25*	220	15.2	1.0	220
26	220	15.2	2.0	440
27	332	15.2	0.67	220
28	332	15.2	1.99	660
29	332	15.2	1.33	440

* indicates that the test was conducted twice to verify consistency of results. All tests were carried out to 18 drops.

Table 2.2 DC Testing Program (Heh, 1991)

Sand density	Estimated Modulus of elasticity (MPa) (1)	Estimated from Results of DC Laboratory tests		Range from literature (3)	
		DSM (MPa)	E (MPa) (2)	Poisson's ratio	Elastic Modulus (MPa)
Loose $D_r \leq 35\%$	14-21	25	15-17	0.2-0.35	10-30
Medium dense $35\% \leq D_r \leq 65\%$	28-37	70	43-45	0.25-0.35	30-50
Dense $D_r \geq 65\%$	51-82	215	125-135	0.3-0.4	50-80

(1) Heh (1991) estimated the range of tangent modulus of elasticity from triaxial compression tests and results computed from constrained modulus obtained from oedometer tests.

(2) Computed based on DSM values.

(3) Based on Hunt (1986).

Table 2.3 Correlation between elastic properties from static and DC test (Poran, 1992)

E_1 tf·m	$m_p \times H_p$	A_p m^2	D_r %	N_{P3}	E_t tf·m	$m_p \cdot v_p$ t·m/s	N'_{P3}	$(m_p v_p \sqrt{N'_{P3}})/A$ $t \cdot m/s/m^2$
100	20 t x 5 m	4	78	85	8,500	198.0	96	485.0
200	10 t x 20 m	4	78	78	15,600	198.0	94	479.9
	20 t x 10 m	4	72	49	9,800	280.0	49	490.0
	40 t x 5 m	4	78	31	6,200	396.0	25	495.0
400	16 t x 25 m	4	70	30	12,000	354.2	30	485.0
	20 t x 20 m	4	74	23	9,200	396.0	24	485.0
	40 t x 10 m	4	76	15.5	6,200	560.0	12	485.0
	80 t x 5 m	4	73	7	2,800	792.0	6	485.0
800	40 t x 20 m	4	75	7	5,600	792.0	6	485.0
	80 t x 10 m	4	76	4.5	3,600	1120.0	3.5	523.8
400	20 t x 20 m	9.8	76	157	62,800	396.0	157	507.3

E_1 : Energy per one blow (= $m_p g H_p$)
 E_t : Total energy (= $E_1 N_{P3}$)
 A_p : Cross sectional area of ram
 $m_p v_p$: Ram momentum (= $m_p \sqrt{2gH_p}$)
 N_{P3} : Number of blows for 3 m penetration
 N'_{P3} : Corrected N_{P3}

Table 2.4 Test Conditions and results (after Mikasa et al., 1988)

Figure 2.1 Grouping of soils for dynamic compaction (after Lukas, 1986)

Figure 2.2 Principle of the heavy tamping method (Van Impe, 1989)

Figure 2.3 Particle motion variation for wave types: (a) P-wave. (b) S-wave (SH). (c) Rayleigh wave (Dowding 1996).

Figure 2.4 Consolidation model (Menard, 1975)

Figure 2.5 Depth of compaction influence vs square root of energy per drop (after Leonards et al., 1980)

Figure 2.6 Trend between apparent maximum depth of influence and energy per blow (after Mayne et al., 1984)

Figure 2.7 Coefficient of effective depth vs radius/depth of crater ratio (after Fang and Ellis, 1983)

Figure 2.8 Test set-up of Beine (1983)

Figure 2.9 Isobars for static and dynamic vertical stresses in the soil (Beine, 1983)

Figure 2.10 Semi-prolate-spheroid approximation for volume of density contours (Heh, 1991)

Figure 2.11 Volumetric depth ratio, b/d , vs Total specific energy (Heh, 1991)

Figure 2.12 Volumetric radius ratio, a/d , vs. Total specific energy (Heh, 1991)

Figure 2.13 $\sqrt{N} - S_p$ relations for different ram momentum (after Mikasa et al., 1988)

Figure 2.14 Relation of total energy and volume reduction of ground (after Mikasa et al., 1988)

Figure 2.15 Relation of total energy and drop height for different ram mass (after Mikasa et al., 1988)

Figure 2.16 Ground deformation (Takada & Oshima, 1994)

Figure 2.17 Vertical earth pressure at center (Takada & Oshima, 1994)

Figure 2.18 Comparison of ground deformation (Takada & Oshima, 1994)

Figure 2.19 Propagation of volumetric strain (after Takada et al., 1994)

Figure 2.20 Distribution of rate of increase of cone point resistance (after Takada et al., 1994)

Figure 2.21a Test setup of Law's (1996) experiment

Figure 2.21b Measured and computed impact deceleration (Impact weight, $W=20\text{kg}$, Drop Height, $H=1.0\text{ m}$ from Law, 1996)

Figure 2.22 Range of compression wave velocities in Ottawa sand as a function of void ratio and saturation (from Allen, et al., 1980).

Chapter 3 Design of Experiment

The experimental programme of this project comprises a series of impact tests on nearly saturated sand confined in a steel pipe. It is apparent from the review described in the previous chapter that the dynamic response of soil due to impact loading is not well understood. This aspect of the investigation will therefore be a logical first step in the study of the dynamic compaction mechanism. The choice of a one-dimensional set-up is deliberate as it eliminates the complex shearing mechanism, which inevitably accompanies the impact on an infinite half-space. It is also felt that due to the predominantly punch-through mechanism, the compressional behavior of a sand column will provide valuable insights into the parameters that governed the compaction process.

3.1 Scope of Experiment

The testing equipment consists essentially of a thick wall steel pipe with its base rigidly connected to a massive concrete block by a steel plate. Nearly saturated sand with a controlled relative density of 35% and 95% degree of saturation is placed in the steel pipe by means of a sand sieving technique, which is described in detail in section 3.3. A cylindrical steel block is then dropped from a given height to generate an impact force. The immediate and transient pore-water pressure response of nearly saturated sand at selected locations along the sand column is investigated using different weight and drop height combinations. The instrumentation and the actual test procedures are described in section 3.5.

3.2 Soil Properties

The sand used in the experimental study is nearly saturated Ottawa sand. The condition of near-saturation is defined here as having a degree of saturation higher than 90% by using a sieving technique described later in section 3.3. A series of standard laboratory tests were conducted to characterize the sand and establish its mechanical behavior. The specific gravity of the sand determined using the procedures based on ASTM 1985 (D854) is 2.67. Sieve analysis was carried out and the particle size distribution is shown in Figure 3.1. The sand used is uniform fine sand with a coefficient of uniformity (CU) of 1.87 and a mean grain size (D_{50}) of 0.41. The particle size ranges from 0.065 mm to 2 mm. In accordance to the unified soil classification system, the soil is classified as SP.

The limiting density tests were conducted using the procedure of ASTM 1985 (D4253, D4254). The minimum dry density of the sand is 1530 kg/m^3 while the maximum dry density based on ASTM using a vibrating table is 1760 kg/m^3 .

Figure 3.1 Particle size distribution of Ottawa sand

3.3 Nearly saturated sample preparation

In order to produce nearly saturated sand samples with a high degree of saturation and a low relative density to increase the liquefaction potential of the impact test, a simple sieving technique is being employed. Water is first poured into the steel column to a height of 100 mm. Sand that has already been soaked in water for 24 hours is then submerged into the water through a metal sieve with a hole diameter of 2.5 mm. The sieve is lifted up slowly as the sand particles diffuse into the water, taking care not to knock against the sides of the column and also to ensure a uniform spread of the sand. After the sand particle has settled onto the base of the column, the next lift of nearly saturated sand will be lowered again to the increased water level. In each consecutive lift, an approximately same amount of sand is added onto the column, by checking against a predetermined marking on the sieve. This process is then repeated until the full height of sample required is achieved. (Refer to Figure 3.2 for illustrations of the process).

Figure3.2 Preparation procedure of nearly saturated sand samples

3.4 Measurement of nearly saturated sand's initial properties

To verify the initial relative density of the sample and the uniformity of the sieving method described in section 3.3, a simple calibration test was done. The nearly saturated sand samples were collected by placing a metal container inside the steel column at every 100 mm depth of the sieving process. After the full height of 900 mm sand column is reached, the embedded containers filled with the nearly saturated samples were retrieved using a shovel to dig to the required depth, taking care not to over-disturb the samples. The collected samples are then oven-dried for 24 hours and weighed to obtain the nearly saturated sand's initial relative density and degree of saturation before impact. The calibration results showed an average relative density of 35% and a high 95% degree of saturation. (Refer to appendix C for detailed calibration data)

3.5 Test set-up and Instrumentation

A total of 10 tests were done according to the following test plan presented in table 3.1. The first 4 test series 1 to 4 were designed to study the pore-water pressure response of a 900 mm nearly saturated sand column under different energy inputs while the next 4 test series 5 to 8 were performed to estimate the corresponding total horizontal stress at the sides of the sand column. The last two test series 9 and 10 were included to study the impact stress on a shorter sand column of 500 mm height.

The set-up for the impact test series 1 to 4 is shown in Figure 3.3. The 900 mm sand column was contained within an instrumented hollow steel pipe 150 mm in diameter. Three horizontal pressure transducers were installed along the wall of the steel pipe. Special care were taken to ensure that the sensing diaphragms of the transducers were flushed with the inside wall of the steel pipe. The locations of these transducers were 800 mm, 600 mm and 500 mm from the surface of the sand.

The set-up for the impact test series 5 to 8 is shown in Figure 3.4. The horizontal soil stress transducers are located at 600 mm and 800 mm from the surface of the 900 mm sand column. A pore-water pressure transducer is inserted at the 600 mm position for verification purpose to ensure reliable comparison studies to be done with the test series 1 to 4.

Figure 3.5 illustrates the set-up for the impact test series 9 and 10. Three pore-water pressure transducers are inserted at 100 mm, 200 mm and 400 mm from the surface of the 500 m sand column. These positions were derived from a numerical analysis that is explained in detail in section 3.6.1.

Test No.	Description	Relative Density (%)	Pounder Weight (kg)	Height of Drop (m)	Energy/drop (kg-m)	No. of drops
1	900 mm sand column – measuring pore-water pressure response at 3 depths. Test 1 and 4 represent the least and highest energy inputs respectively	35	12	0.3	3.6	15
2		35	17	0.3	5.1	15
3		35	17	0.4	6.8	15
4		35	17	0.5	8.5	15
5	900 mm Sand Column – measuring horizontal stress at same depths in test series 1-4	35	12	0.3	3.6	15
6		35	17	0.3	5.1	15
7		35	17	0.4	6.8	15
8		35	17	0.5	8.5	15
9	500 mm Sand Column – measuring pore-water pressure response at 3 depths	35	12	0.3	3.6	15
10		35	17	0.5	8.5	15

Table 3.1 Test Plan

Figure 3.3 Setup for the Impact test series 1 to 4

Figure 3.4 Setup for the Impact test series 5 to 8

Figure 3.5 Setup for the Impact test series 9 and 10

The impact was to be provided by dropping a pounder from a pre-set height. The pounder was first lifted to the desired height by an electromagnetic mount. It was then released by cutting the power supply to the electromagnet. For safety reason, the pounder and electromagnet was enclosed within a cage attached to the steel frame. A preset trigger from the accelerometer installed in the pounder initiated the data capture. The dynamic data was then transferred to a digital oscilloscope and the digitized data was automatically stored on a floppy diskette. Each test series consist of 15 impact drops. At the end of each drop, the settlement of the sand surface will be measured. The components and instrumentation of the model test set-up are described in detail below:

- 1) *Steel pipe* (Figure 3.6). The diameter of the stainless steel pipe is 150 mm and consisted of two parts, each 525 mm in length. These two parts were connected by six bolts around the flange and made up a total height of 1050 mm. The wall thickness is 10 mm. We will however be filling up only the bottom half of the steel column with sand. The top part of the steel column is employed to accommodate overflowing of sand during sample preparation and as a safety precaution during impact tests. The entire pipe was anchored on a steel plate embedded in a concrete block, which was in turn bolted to the reinforced concrete floor. It is important to ensure the rigidity of the

soil column to avoid compaction arising from vibration of the steel pipe. The stress transducers were mounted along the steel pipe.

Figure 3.6 Stainless steel pipe and rigid concrete block

- 2) *Steel frame and pounder guide* (Figure 3.7). The guide enclosing the pounder and electromagnet was fixed on the steel frame. The main function of the guide is to serve as a safety device. It is slightly larger than the pounder diameter and will not impede the free fall of the pounder. The steel frame also serves as a mounting for the sand raining device and can be moved and locked into position.

Figure 3.7 Steel frame and pounder guide

- 3) *Pounder*. The pounders (Photo 3.1) were fabricated from cylindrical blocks of mild steel. The diameter of the pounder is 110 mm, slightly smaller than that of the steel column. The weight of the smallest pounder is 10 kg. The pounder mass can be increased by adding detachable pieces. In this manner, pounder mass varying from 10 kg to 30 kg can be obtained. The connections between the discrete pieces of pounder are achieved by means of threaded section. This ensures that the steel blocks act as a monolithic piece. The accelerometer was installed at the center of the pounder (Photo 3.2), and protected by a steel cover, which can be bolted on the pounder.

Photo 3.1 Pounders

Photo 3.2 Installation of the accelerometer within the center of a connecting pounder

- 4) *Electromagnet* (Photo 3.3). A circular electro-magnet was used to lift and release the pounder. The electromagnet enables a quick release without significant jerking. As the magnet operates on direct current, an external DC power converter was used to operate it with alternate current power source.

Photo 3.3 Electromagnet

- 5) *Horizontal pressure transducers.* The horizontal stresses at three different levels along the soil column and the vertical stress at the base were selected as possible locations of measurements. The horizontal stresses were measured using Kulite VM-750 series of pressure transducers (Photo 3.4). These transducers are designed to measure soil pressure under blast induced loading up to 2800 kPa. The sensing diaphragm has an extremely small deflection rate and high natural frequency. The unit is electron beam welded and mounted with support ring integral with the body assembly. This arrangement obviates errors due to mounting case stresses in the diaphragm area. Specially designed adapters were used to mount the transducers onto the steel column wall. Refer to 3.5.1 for a detailed description of the design process of the pressure transducers and 3.5.2 of the design process of its adapters.

Photo 3.4 Kulite VM-750 Pressure transducer

- 6) *Earth Pressure Cell.* Vertical stresses at the base of the sand column were measured using a Kulite LQ-080U stress cell (Photo 3.5) to deduce the effective vertical stress of the sand along the soil column under the effects of side friction and soil arching. A detailed description of the test procedures is given later in chapter 4. This stress cell was designed for accurate measurements of blast induced soil reactions up to 1400 kPa. The stress cell employs a pair of extremely stiff diaphragm with a diameter to thickness ratio greater than 5 and a diameter to deflection ratio greater than 2000. This design overcomes the problem of stress concentration relief due to insertion of the gauge in the soil, which disrupts the stress field. The design, with good gauge medium matching ensures accuracy and repeatability of readings.

Photo 3.5 Kulite LQ-080U stress cell

- 7) *Accelerometer.* The PCB 353 piezoelectric accelerometer (Photo 3.6a) with built in electronics was used to measure the deceleration of the pounder during impact. This accelerometer utilizes the piezoelectric effect of quartz and special polycrystalline ceramics to generate an electrical signal proportional to the vibration or shock. The accelerometer has a sensitivity of 10 mv/g with ± 500 g range and a maximum shock limit of ± 10000 g. Signal conditioner is required for

powering the sensors. A PCB 483b07 'Rack Mount' line powered signal conditioner (Photo 3.6b) was used to provide a constant current to the accelerometer and to remove DC bias from the sensor output signal.

Photo 3.6a PCB 353 piezoelectric accelerometer

Photo 3.6b PCB 483b07 'Rack Mount' line powered signal conditioner

- 8) *Data acquisition system.* The dynamic data from the stress transducers and the accelerometer were acquired and processed using a Nicolet digital oscilloscopes system 400 (Photo 3.7). This 4-channel bench top data acquisition system enables the analogue input signals to be digitized and stored in a floppy diskette for convenient storage and archiving, or instant transfer to computer in compact binary format. Each channel is capable of sampling up to 10 million samples per second.

Photo 3.7 Nicolet digital oscilloscope

All the stress transducers were connected to a Entran Multi Channel Signal Conditioning system (Photo 3.8) for power supply, signal conditioning and data amplifying. The compact conditioning and amplifier modules allow modular conditioning, monitor display, providing sensor power supply and calibration functions. It accepts a full range of inputs from 5 mv to 1 v full scale and provides gains of 2 to 2000. The amplifier features a separately buffered voltage output to drive the data acquisition system. The conditioned signal from the Entran MSC series was directed to a channel box (Photo 3.9), and then to the data acquisition system.

Photo 3.8 Entran Multi Channel Signal Conditioning system

Photo3.9 Channel Box

In dynamic data acquisition, the data of interest is limited to a short interval. The digital oscilloscope offers a reliable and accurate digital trigger. It eliminates false triggers with variable sensitivity noise rejection. Upon triggering, the system will continue to capture the data at a pre-set sampling rate for a pre-set period of time. A special software was used to convert the signal from a binary into ASCII format.

3.5.1 Design process of the adapters for the pressure transducers

To separate the pore-water pressures effectively from the horizontal pressures exerted by the sand solid particles, special adapters have to be incorporated into the steel column to accommodate a filter in doing this task. The main design considerations were:

- Water tightness is important, as any small dissipation of the water pressure would render the result to be inaccurate in a highly demanding dynamic simulation where the range of the pressure response is expected to be wide.
- Suitable filters have to be experimented to effectively remove the soil pressure from the pore-water pressure in the saturated sand. The filter must also be strong enough to withstand the high dynamic loading and protects the sensors from being damaged. At the same time, it should not impede the flow of water too much and cause a delay in the recording of the response from the sand.
- Suitable locking system has to be devised to fix the pressure transducers into position and yet not cause unnecessary overstress on the sensitive diaphragm of the pressure transducers.
- The material used to fabricate this adapter must be strong enough too to withstand the expected dynamic load and also has corrosive resistance against the water-bearing sand sample used.

After several rounds of modifications in consultation with researchers and contractors, a final design was found to be suitable for the above expectations.

3.5.2 Details of design

Below is a detailed explanation of the individual components of the newly-designed adaptor.

1) Adaptor

The final design of the adaptor (Figure 3.13) is made of stainless steel and carefully carved at the end to fit into the inner curvature of the steel column. A depression was made in front for the sitting in of a filter to permit only the pore water pressure to hit the transducer. The inside circumference and length of the adaptor are tailored precisely to the size of the pressure transducers to minimize complications of water leakage and ensure an efficient transmission of the pore-water pressure onto the sensing diaphragm of the transducers. It was fabricated with a thickness of 10.5 mm by the sides to resist any cracks from potentially high dynamic impact. A drainage compartment with two outlets was included in the middle section of the adaptor to ensure water-tight conditions. (Refer to Appendix G for detailed engineering drawing).

Figure 3.13 Adaptor

2) Filter

A special composite filter was fabricated to filter off the soil pressure from its pore-water pressure while at the same time able to withstand the high dynamic stress of the impact. Figure 3.14a shows the individual parts of the composite filter. It consists of 7 pieces of coarse steel sieve with a hole opening of 1 mm and a fine standard sieve of 200 μm sieve size, all cut into a circular shape of 19 mm diameter. 4 pieces of the coarse sieve was put in front of the fine sieve to protect it against the direct impact of the test while the fine sieve attempts to block out the sand particles from affecting the pore-water pressure readings. The remaining 3 coarse sieves are then placed behind the fine sieve to prevent the pressure transducer at the back from damaging the fragile fine sieve. This 8-piece sieve is then glued together through a strong adhesive and left to dry. The assembled product is shown in Figure 3.14b. A simple constant head test-setup is then performed to determine the coefficient of permeability of the newly designed sieve (Refer to Figure 3.14c). With a combined thickness of 6 mm, its k value was found to be 1.46×10^{-2} cm/s, which is higher than the permeability of the Ottawa sand. It was classified to have a good drainage property, adequate for the use of this experimental research.

Figure 3.14a Individual parts of composite filter

Figure 3.14b Assembly of composite filter

Figure 3.14c Constant-head test set-up for the determination of the coefficient of permeability of the filter

3) De-airing system (Figure 3.15)

A small inlet hole was fabricated on the adapter to allow water to flush out every air bubbles being trapped in the gap between the stone and the transducer. The passage of water was then channeled to flow out from another outlet hole created on the adapter. An on/off valve capable of withstanding high pressure and maintaining water tightness was installed to keep the gap saturated with water. As the gap was extremely narrow coupled with tight working area around the adaptor, high precision was needed to align the holes in the right orientation.

Figure 3.15 De-airing system for the adapter of pore-water pressure transducer

4) Open Collars

The open collar (Figure 3.16) was designed to hold the pressure transducer into place leaving an open hole through for the electrical wiring to be connected out from the adapter to the oscilloscope. O-Rings were installed round the head to ensure water tightness. The collar was threaded to a length of 20 mm locking the device into position. Careful turning torque has to be exercised to prevent overstressing the pressure transducer in front of it towards the edge support.

Figure 3.16 Open Collar

5) Blind stoppers

A blind stopper (Figure 3.17) was also fabricated to fill up the hole of the adaptor to facilitate the testing of dry sand. O-Rings were also installed on the front ring of the blind stopper to ensure water tightness during testing.

Figure 3.17 Blind Stopper

6) Attachment to cylinder

The whole adaptor was attached to the steel cylinder through welding to minimize any possible water leakages through other types of connection and also to withstand the high dynamic stress from the impact. The finished product is shown in Figure 3.18.

Figure 3.18 Attachment of adapters to steel column

3.6 Test Procedures

A checklist was established for each test. This checklist was followed meticulously in each test to eliminate any uncertainties regarding the test procedure and ensure that the various equipments are properly set to capture the dynamic data.

- 1) *Calibration.* Before the start of each test, the accelerometer and transducers were calibrated. The accelerometer was calibrated using a PCB model 394B06 hand held calibration exciter (Figure 3.19). This calibrator provides a controlled vibration level for verifying the sensitivity of the accelerometers. It is an electromagnetic exciter driven by a stabilized oscillator at a frequency of 79.6 Hz, giving a constant 1.0 g (9.81 m/s^2) rms vibration amplitude. The transducers were calibrated using a Druck Digital Pressure Indicator model DPI601 (Figure 3.20). The linearity of the stress cells was checked for the entire operating range of the transducers. (Refer to Appendix A & B for the detailed calibration curves)

Figure 3.19 PCB model 394B06 Hand-held Calibration Exciter

Figure 3.20 Druck Digital Pressure Indicator model DPI601

- 2) *Installation of soil stress transducers.* The transducers were placed in the adapters mounted at pre-determined location along the steel pipe and at the base. The VM-750 transducers were secured in place by turning in the open collars. The open collars must be tightened with sufficient torque to provide sufficient reaction for the transducers. However, the force exerted by the collars must not be too great to avoid overstressing of the transducers. After installation, the initial stress reading was taken to make sure that the transducers were not overstressed. It is of interest to note that the collar locks the transducer by applying the pressure on the support ring of the transducer. Therefore the diaphragm area is stress free. The LQ-080U transducer was placed at the center of the base plate. A recess was machined at the base plate to accommodate the flat-faced transducer.
- 3) *Sample preparation.* In sieving the sand through the water, considerable care is taken to ensure full immersion of the metal sieve to ensure high degree of saturation. Regular measurements are taken to maintain a constant water height of 100 mm above the settled sand particles.

- 4) *Drop tests.* The guide and the pounder were aligned along the steel pipe. Verticality of the guide was checked by spirit level and then the rollers on the steel frame were locked. The initial readings of the stress transducers before impact were recorded. Verification check on all the electrical connections was conducted. The electromagnet was activated to lift the pounder to the required height. The trigger level of the digital oscilloscope was set, and then the electromagnetic switch on the rectifier was turned off releasing the pounder. Upon impact, the deceleration triggered the data acquisition system. The signals were saved on a floppy diskette. The changes in the surface elevation were measured after each drop. The pounder was removed and the sand surface was leveled to prepare for the next drop.
- 5) *Step (4) was repeated for successive drops.*

Chapter 4 Impact Stress of Sand Column

4.1 Introduction

The stress produced at the surface of a nearly saturated sand column under an impact is of particular importance to the understanding of its stress-strain relationship under a dynamic loading. To simulate this impact loading, a falling pounder was used in this experiment. The deceleration data recorded by the accelerometer inside the pounder was used to find the impact stress produced at the surface of the sand (Refer to Figure 4.1 for detailed illustration). The numerical computation of the impact stress was derived using Newton's second law:

$$P = F/A \quad \text{Eqn 4.1}$$

$$F = Mg \quad \text{Eqn 4.2}$$

where,

P = Impact stress

F = Impact force

A = Impact circular area

M = Mass of steel pounder

g = Pounder deceleration

This chapter presents the impact stress records produced on top of a 900mm sand column by the falling pounder. The data have been taken from experimental test series 1 to 4 representing different energy inputs as shown in Figure 4.2. The effects of various impact parameters like impact energy, relative density of sand column and number of drops on the impact stress waves produced in nearly saturated sand column will be investigated in this research. (Refer to Appendix D for the detailed data of drops 1-5, 10 and 15 of each test series.)

Figure 4.1 Calculation of impact stress from deceleration records

Figure 4.2 Setup for experimental test series 1 to 4

4.2 Sand surface settlement

The settlement of the surface of the sand column was measured after every drop to trace the effects of the impact as the number of drops increases (Refer to Figure 4.3). There is a consistent trend of decreasing settlement as the number of drops increases across various energy inputs. We observed the main stages of the settlement to occur during the initial 3 drops with data values ranging from 10 to 15 mm at the 1st drop, 5 to 12 mm at the 2nd drop and 4 to 7.5 at the 3rd drop. The combined settlement of these initial 3 drops accounted for nearly 40% of the whole settlement process. It was followed by a relatively even settlement averaging 5mm from drop 4 to drop 9. The settlement then begins to slide down to a negligible 0.5 mm from drop 10 onwards.

Looking at the cumulative settlement graph in Figure 4.4, the total settlement ranges around 60 to 75mm which corresponds to around 7% and 8% of the full soil column height. The general trend of a parabolic curve that begins to level off after drop 10 was consistent across different energy inputs. This further reinforces the trend line of Figure 4.3 where the soil column shows no significant change in its relative density upon reaching drop 10.

Figure 4.3 Trend of settlement per drop over different impact energy

Figure 4.4 Trend of cumulative settlement over different impact energy

4.3 Dynamic measurements

The impact stress at the surface of the sand column is being compared according to the changes of the following 3 impact parameters:

- Effect of number of drops
- Effect of energy input
- Effect of sand's relative density

Each impact parameter will be discussed individually in the following sections.

4.3.1 Effect of number of drops

The general trend of the deceleration of the pounder and its computed stress upon impact under the least energy input of 3.6 kgm is presented in Figure 4.5 and Figure 4.6 respectively. The following symbols are used to present the impact parameters: W is the pounder weight, H refers to the drop height and D is the full sand column height. Only drops 1, 5, 10 and 15 are presented to highlight the progressive changes as the number of drops increases. We can see that there is a rising peak within the first 10 ms followed by a gradual decline of the stress curve from drop 1 to drop 5. Upon reaching the higher number of drops from 10 to 15, we observed a sharp triangular peak followed by a rapid tapering off of the stress wave. Thus, we see the initial drops of the test producing a longer response of the stress wave as compared to the higher drop number of 10 to 15. This phenomenon could be best explained by the gradual densification of the soil column due to the repeated impacts by the pounder. The initial drops of the impact encountered relatively loose sand with a lower soil stiffness value as compared to the higher drop numbers. We can thus see the damping effects of the loose sands coming into effect to lengthen the impact stress response. At higher drop number, the sand has become much consolidated into a dense configuration, thus reflecting a sharp triangular response and a rapid retreat to the zero line.

As we increased the energy input to the maximum of 8.5 kgm, we see a similar trend as shown in Figure 4.7 and Figure 4.8. However, we can see that due to the higher amount of energy used, the process of densification is activated at a faster rate as compared to the least energy input of 3.6 kgm. Noticeably, the sand column was observed to reach its relatively dense condition upon reaching drop 10 resulting in small difference of its stress waves in the following drops to 15.

The deceleration records by Law (1996) in dry sand of 50% initial relative density appear to have similar waveforms across the 20 drops lasting only a short time of 10 ms. However in the test that we performed on nearly saturated sands of 35% initial relative density, there is a much broader deceleration-time history especially in the first 5 drops lasting up to 30 ms.

Figure 4.5 Impact deceleration record in
the first 50 ms from the least energy input of 3.6 kgm

Figure 4.6 Calculated impact stress in the first 50ms from the least energy input of 3.6kgm

Figure 4.7 Impact deceleration record in the first 50 ms from the highest energy input of 8.5 kgm

Figure 4.8 Calculated impact stresses in
the first 50 ms from the highest energy drop of 8.5 kgm

4.3.2 Effect of energy input

To facilitate easy comparison across the effects of different energy input, we first identified the peak impact stress of every drop in each test series 1 to 4. We found a consistent trend of increasing peak impact stress following a higher number of drops as reflected in Figure 4.9 in every energy input used. A general higher stress curve was also noted as the amount of energy was increased in the impact tests.

Figure 4.9 Trend of highest impact stress over different impact energy

4.3.3 Effect of sand’s relative density

From the observed settlement of the sand column under impact, we are able to derive the changes of the sand’s relative density after every drop. By plotting the peak impact stress in every drop against the changes in the sand’s relative density, we are able to see a consistent increasing trend as represented in Figure 4.10. The results yielded a logical explanation that as the sand became more densified over the drops; the impact stress at the surface will experience a higher peak value due to the reduction in damping of the sand column.

Figure 4.10 Effect of relative density on peak impact stress

4.4 Stress-strain relationship

An especially important study is to derive the stress-strain relationship of the sand column while it is undergoing the impact process. A double integration of the deceleration curve of each impact test was first performed to derive the settlement curve over time. The axial strains are then computed by dividing the settlement by its original sand column height. The impact stress is then compared to its computed strain over the same time history for every drop and plotted a continuous fashion as represented in Figure 4.11 and 4.12 showing the results taken from the lowest energy input of 3.6 kgm and highest energy input of 8.5 kgm respectively.

We observed an increasing initial gradient of the stress-strain curve over each drop which represents an increase in the soil's modulus in both graphs. This is consistent with the discovery of the sand's increasing relative density discussed in section 4.3.3 where the sand became more densified over the drops. We can see that in Figure 4.11, the stress-strain curve overlap each other at drops number beyond 11 signifying no further changes in the sand's compaction upon reaching the maximum axial strain of around 6%. With the use of a higher energy input, we can see in Figure 4.12 that the sand column reached the axial strain of 6% at an earlier stage in drop 6 and went on to achieve a higher compression of around 8% at the final stage.

Another interesting observation that we could see in both graphs is the retractive strains that start to exist beyond drop 4 after demonstrating a damping phenomena in the first 3 drops. This could be explained by the fact that at the initial drops, the relatively loose state of the sand provided a cushion to absorb the impact loads and thus causing the axial strains to increase even at decreasing axial stress. As the sand column became more and more densified over the drops, it approaches the behavior of an elastic material and thus demonstrates higher rebounds of the immediate settlement caused by the impact. We can also observe an unstable stress-strain curve in the first 6 drops as possible liquefaction might be interfering with the compaction process. At higher drop numbers, stability is shown through a sharp triangular response.

The loading phase of Ginsburg's model (1964) was used to compare the envelope over the peaks of the stress-strain curve of each drop. Based on the uniaxial compression tests on

several types of soils, he proposed a soil model, in which the virgin loading curve is given by,

$$\sigma = M_o \frac{\varepsilon \varepsilon^*}{\varepsilon^* - \varepsilon} \quad \text{Eqn 4.3}$$

where M_o is the initial constraint modulus and ε^* is the asymptotic strain. For the purpose of comparison, the initial tangent constrained modulus of the nearly saturated sand have been estimated to be 7500 kPa with an asymptotic strain of 0.1 and 0.14 respectively for the lowest and highest energy input. As seen in both Figures 4.11 and 4.12, the stress-strain curve of drop 1 was significantly out of sync from the usual stress envelope. This discovery raised the need to re-look the suitability of Ginsburg's model to predict the response of nearly saturated sands under impact loadings due to the interference of pore-water pressure especially at the initial drop 1. A possibly different mechanism of compaction might have happened due to liquefaction in the nearly saturated sands. This phenomenon will be further discussed in later chapters.

Figure 4.11 Stress-Strain curve of 900 mm soil column under lowest energy input of 3.6 kgm over 15 drops

Figure 4.12 Stress-Strain curve of 900 mm soil column under highest energy input of 8.5 kgm over 15 drops

Chapter 5 Immediate Pore-water Pressure Response

5.1 Introduction

A typical pore-water pressure response of the sand column under impact loading consists of two important parts. They are the immediate and residual pore-water pressure as shown in Figure 5.1. The immediate pore-water pressure response of the sand column is defined here as the pore-water pressure produced during the first stage of impact. The residual pore-water pressure response produced beyond the initial impact will be presented later in Chapter 6. This chapter will present the results of the peak pore-water pressure produced during this period of time under the impact test series 1 to 4 (Refer to Figure 4.2 for experimental set-up plan and Appendix D for the detailed data of drops 1-5, 10 and 15 of each test series). The effect of various impact parameters like impact energy, relative density of sand and number of impact drops on the trend of peak pore water pressure produced will also be discussed. Three pre-determined locations along the sand column are installed with pressure transducers to study the variation of response as the drainage distance from the impact source increases. As shown in Figure 5.2, 'Top' pore-water pressure is measured at a location 500 mm below the sand surface while 'Med' pore-water pressure and 'Bot' pore-water pressure refer to 600 mm and 800 mm below the sand surface respectively.

To quantify the trend of the pore-water pressure response of the sand column in accordance with various impact parameters, a peak pore-water pressure ratio, U_h/I_h is introduced to compare the results. It is defined as the ratio of the peak pore-water pressure response, U_h at a particular position of the soil column to its corresponding peak impact stress, I_h at the surface for every drop.

Figure 5.1 The immediate and residual pore-water pressure response of a typical sample under impact loading

Figure 5.2 Location of pressure transducers along the sand column

5.2 Immediate Pore-water Pressure Time Histories

Figure 5.3, 5.4 and 5.5 shows the pore-water pressure response of the Top, Med and Bot positions in the first stage of impact under the lowest energy input of 3.6 kgm.

For the top position, we see a small rise to 25 kPa that maintains for around 30ms, gradually declining to a residual value for drop 1. Thereafter, a sinusoidal waveform appears beyond drop 5. The negative pore-water pressure produced under such a waveform indicates the interference of tensional waves which drives the potential for liquefaction to occur under an impact loading. At the final drop 15, there is only a faint peak response of 8 kPa.

At the med position, the peak value achieved at drop 1 was near the value at the top position, hovering around 21 kPa before declining to a residual flat line too. A similar sinusoidal waveform appears beyond drop 5 to the final drop which registers a slight slip of peak stress to 18 Pa.

We see a more significant difference at the Bot position where a much higher peak stress of 54 kPa was produced at drop 1 before it hits a residual plateau. A small sinusoidal waveform starts to appear at the initial drop which extends throughout the 15 drops. At the last drop, the peak stress achieved remains high at 53 kPa. It was noted that the pore-water pressure always remained positive above the zero line for the first drop at the three locations (Top, Med and Bot) throughout the various tests.

As we increased the energy input to 8.5 kgm, the same general trend was observed as shown in Figures 5.6, 5.7 and 5.8 for the three positions. The peak pore-water pressure produced at drop 1 was now increased to 48 kPa, 63 kPa and 107 kPa for the Top, Med and Bot positions respectively. The rise time to peak was faster than that produced by the above lower energy input. Interestingly at the final drop 15, we see a reduction of peak pore-water pressure to 36 kPa at the Top position, while an increase to 111 kPa for both Med and Bot Positions. This difference occurs because the top layers of sand column are denser than the bottom layers. Thus, a more compact configuration of the sand particles will result in a smaller value of the peak pore-water pressure produced. In comparison with Ruygrok, 1980's impact experiments on nearly saturated sand, it was found that typical triangular peak response was similar to his research and the rise times to be

between the same ranges of 5 to 10 ms. The existence of negative pore-water pressure fluctuation found in Ruygrok’s study was also consistent in our observation.

Figure 5.3 ‘Top’ pore-water pressure in 1st stage under lowest energy input of 3.6 kgm

Figure 5.4 ‘Med’ pore-water pressure in 1st stage under lowest energy input of 3.6 kgm

Figure 5.5 ‘Bot’ pore-water pressure in 1st stage under lowest energy input of 3.6 kgm

Figure 5.6 ‘Top’ pore-water pressure in 1st stage under highest energy input of 8.5 kgm

Figure 5.7 ‘Med’ pore-water pressure in 1st stage under highest energy input of 8.5 kgm

Figure 5.8 ‘Bot’ pore-water pressure in 1st stage under highest energy input of 8.5 kgm

5.3 Effects of Impact Parameters

The trend of the peak pore-water pressure ratio will be investigated in accordance to the effects of changes in:

- Energy input
- Number of drops
- Drainage length
- Relative density of sand column

5.3.1 Effect of Energy Input

Figures 5.9, 5.10 and 5.11 show the variation of the peak pore-water pressure ratios for the three positions over different energy inputs. We observed that there is a general increasing trend of the ratio whenever the energy input is being raised. When we reach the upper limit of the energy input, we found that there was marginal difference in the ratios suggesting that an optimal energy input range has been reached. We can see that at higher drop numbers, the effect of energy input becomes minimal as the sand reached a relatively dense condition and appears non-responsive to any further increase in impact energy.

Figure 5.9 Variation of Peak Pore-water Pressure Ratio for Top position over different energy inputs

Figure 5.10 Variation of Peak Pore-water Pressure Ratio for Med position over different energy inputs

Figure 5.11 Variation of Peak Pore-water Pressure Ratio for Bot position over different energy inputs

5.3.2 Effect of Number of Drops

Figures 5.12 and 5.13 shows the variation of the peak pore-water pressure ratio, U_h/I_h , along the soil column under the least and highest energy inputs as the drop number increases. We see a consistent declining trend due to the increase in the relative density of the sand column as the number of drops increases. The peak pore-water pressure ratio range from 0.07 to 0.15 under the least energy input of 3.6 kgm to a range of 0.15 to 0.33 at the highest energy input of 8.5 kgm for the initial drop 1. As the drop number increases, the ratio stabilized to a final value of between 0.01 to 0.09 under the least energy input to a range of 0.03 to 0.10 for the highest energy input of 8.5 kgm.

5.3.3 Effect of Drainage Length

The effect of changes in drainage path lengths are also highlighted in Figures 5.12 and 5.13 as represented in the variation of the peak pore-water pressure ratios along the soil column represented by the three positions under the least and highest energy inputs. We see a smaller response from the positions nearer to the source of impact as seen in Top and Med pressure trend line in the graph as compared to Bot Position. This was primarily due to the partially drained condition at the sand surface. As the drainage path increases along the soil column, the pore-water pressure at positions near to the base of the soil column will take a longer time to be dissipated, thus providing an ideal condition for a higher peak pore-water pressure to be produced.

Figure 5.12 Peak Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) under the lowest energy input of 3.6 kgm

Figure 5.13 Peak Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) under the highest energy input of 8.5 kgm

5.3.4 Effect of Relative Density

The settlement of the sand column following every impact drop was also translated into its corresponding change of relative density, $D_r(\%)$ to see the effect of the sand properties on its liquefaction potential as presented in Figure 5.14 (Least Energy Input of 3.6kgm). We see a decreasing peak pore-water pressure ratio at all three positions (Top, Med and Bot) as the relative density of the sand column increases. The denser packing among sand particles at higher relative density is the most likely reason for the declining trend of the peak pore-water pressure ratio. As we increase the energy input to 8.6 kgm, all three curves show the same declining trend in Figure 5.15.

Figure 5.14 Variation of Peak Pore-water Pressure ratio over Relative Density of Sand under least energy input of 3.6 kgm

Figure 5.15 Variation of Peak Pore-water Pressure ratio over Relative Density of Sand under highest energy input of 8.5 kgm

Chapter 6 Residual Pore-water Pressure Response

6.1 Introduction

The residual pore-water pressure response of the sand column is defined here as the pore-water pressure produced beyond the first stage of impact to its final decay as shown in Figure 5.1. This chapter will present the results of the residual pore-water pressure produced during this period of time under the impact test series 1 to 4 (Refer to Figure 4.2 for experimental set-up plan and Appendix D for the detailed data of drops 1-5, 10 and 15 of each test series). The effect of various impact parameters like impact energy, relative density of sand and number of impact drops on the trend of residual pore-water pressure produced will also be discussed. The three locations of interest along the sand column are as shown in Figure 5.2. 'Top' pore-water pressure is measured at a location 500 mm below the sand surface while 'Med' pore-water pressure and 'Bot' pore-water pressure refer to 600mm and 800mm below the sand surface respectively.

To better access the liquefaction potential of the sand column in accordance with various impact parameters, a residual pore-water pressure ratio is introduced to compare the results. The residual pore-water pressure ratio, U_r/V_{eff} , is defined as the ratio of the residual pore-water pressure response, U_r at a particular position of the soil column to its corresponding effective soil vertical stress, V_{eff} at that position after impact for every drop. U_r refers specifically to the excess pore-water pressure that continues to exist even in the absence of any impact stress wave. A conservative value is taken at the 200 ms mark of every test where the excess pore-water pressure is assumed to be stable enough to represent the 2nd stage of pore-water pressure response as illustrated in Figure 5.1. The calibration of V_{eff} will be discussed later in section 6.2. This ratio, U_r/V_{eff} is thus an effective indicator of the liquefaction potential as it represents the excess pore-water pressure overlying the effective soil vertical stress at each position.

By assuming that the total vertical stress at any point, after the end of impact, is equaled to the addition of the total vertical stress under the action of a static pounder in a dry condition and the hydrostatic water pressure, we can derive that a value of 1 is the condition for liquefaction of the sand column at a particular position (Refer to Figure 6.1a for an illustration). This could be done, as there is minimal spillage and tolerable surface settlement during the conduct of the experiment. The close resemblance of the residual

pore-water pressure response after 200 ms to a static condition also reaffirmed the safe use of this assumption. The total vertical stress derived under the dry condition is thus equaled to the effective vertical stress V_{eff} . The mathematical derivation for the condition of liquefaction is worked out as follows:

At calibration stage,

$$\text{Total Stress, } T_1 = V_{\text{eff}} \tag{Eqn 6.1}$$

After impact,

$$\text{Total Stress, } T_2 = V_{\text{eff (after)}} + U + U_r \tag{Eqn 6.2}$$

where,

$V_{\text{eff (after)}}$ is the effective vertical stress after impact

U_r is the excess pore-water pressure remaining

U is the hydrostatic water pressure

For liquefaction to occur, $V_{\text{eff (after)}}$ must be equaled to zero. By letting $T_2 = T_1 + U$, we have the condition:

$$U + U_r = U + V_{\text{eff}} \tag{Eqn 6.3a}$$

$$\Rightarrow U_r/V_{\text{eff}} = 1 \tag{Eqn 6.3b}$$

Figure 6.1a Illustration of the assumption required for $U_r/V_{\text{eff}} = 1$

6.2 Effect of Friction on Effective Vertical Stress

From silo theory, we understand that there is a reduction in the effective vertical stress, V_{eff} , along the sand column due to the effects of side friction and soil arching. Hence, to effectively determine the liquefaction potential along the sand column under the impact loadings, there was a necessity to find the actual effective vertical stress along the sand column through experimental calibration. Figure 6.1 is a diagrammatic illustration of the calibration process. A base pressure cell is first installed at the base of the steel column to measure the reduced vertical stress of the sand fill. The whole calibration process is divided into 9 lifts. Each lift adds an additional 100 mm layer of 50% relative density sand onto the transducer. At each stage, a pounder weight is inserted to record both the soil stress and effects of surcharge. This process is repeated 10 times to derive the distribution of the soil vertical stress along the full height of the sand column.

Figure 6.1b Calibration of reduced effective vertical stress along sand column

Figure 6.2 shows the results of the test under the surcharge loading of 17 kg. The purple line represents the effects of the surcharge loading along the sand column. The combined effects of the soil's own effective vertical stress together with the surcharge are shown in the blue line. We see a significant reduction of the soil vertical stress and surcharge loading especially in the first 200 mm of the sand column, tapering off to an asymptotic straight line beyond 300 mm. The effects of the surcharge are practically insignificant beyond the depth of 400 mm. The measured results are compared against the theoretical results (pink line) derived from Silo's theory :

$$\sigma_{vz} = \gamma z_0 (1 - e^x) \quad \text{Eqn 6.4}$$

where,

σ_{vz} = Soil's effective vertical stress along column

$x = -z/z_0$

z = depth of sand

$z_0 = A/(UK \tan \delta)$

γ = unit weight of the infill, kN/m³

U = inside perimeter of cell, m

A = inside area of cell, m²

ϕ = internal friction angle of the infill, °

δ = soil-wall friction angle $\sim \phi/2$, °

K = coefficient of lateral earth pressure, $K_0 = 1 - \sin \phi$ (at rest conditions)

The measured values were found to be smaller than the theoretical derivation while following a similar decreasing trend. At the critical locations of 500 mm, 600 mm and 800 mm where the pressure transducers are installed, we note a difference of 30%, 23% and 12% respectively. The calibrated effective vertical stresses were then used to represent the reduced effective stress of the sand at that particular depth due to the effect of side friction.

As the 2nd stage of the pore-water pressure response represented in Figure 5.1 closely resembles a static condition, the effective vertical stresses calibrated above at each depth were assumed to be adequate for the comparison of V_{eff} against its corresponding U_r after impact. The approximate reduced effective stresses, V_{eff} , of the sand column at predetermined locations are summarized in Table 6.1.

Figure 6.2 Distribution of soil's effective vertical stress along a 1 m sand column with a 17 kg surcharge load.

Distance from Surface, mm	Measured effective stress under 12 kg surcharge (V_{eff}), kPa	Theoretical effective stress under 12 kg surcharge, kPa	Measured effective stress under 17 kg surcharge (V_{eff}), kPa	Theoretical effective stress under 17 kg surcharge, kPa
100	8.12	10.11	10.80	13.70
200	5.48	8.37	5.93	10.86
400	4.61	6.33	4.59	7.53
500	4.56	5.75	4.57	6.58
600	4.55	5.34	4.55	5.92
800	4.53	4.87	4.53	5.14

Table 6.1 Summarized readings of soil reduced effective stress with surcharge

6.3 Residual Pore-water Pressure-Time Histories

The residual excess pore-water pressure-time history focused on the excess pore-water pressure remaining even after the decay of the impact stress wave. All the figures that follow are snapshots in 2nd stage of the pore-water pressure response as illustrated in Figure 5.1. The complete experimental data can be viewed from Appendix D.

Figures 6.3, 6.4 and 6.5 show the pore-water pressure response of the three positions after the immediate impact to its final decay. We observed a continuous long period of residual pore-water pressure response lasting up to 4.5 s for drop 1, 2.5 s for drop 5 and even around 2 s for higher drop numbers 10 and 15 before decaying to its zero line. The magnitude of the residual pore-water pressure also decreases as the number of drop increases. We see a reduction from 2.8kPa at drop 1 to 2.0kPa at drop 15 for the Top position, 2.4kPa to 2.2kPa for Med position and 1.7kPa to 1.4kPa for Bot position.

As we increased the energy input to 8.5 kgm, the same general trend was observed as shown in Figures 6.6, 6.7 and 6.8 for the three respective positions. Interestingly, the magnitude of the residual pore-water pressure reached a high value of 3.9 kPa, 3.3 kPa and 2.3 kPa for the Top, Med and Bot positions respectively at drop 1. This range of values was noted to approach the effective vertical stress of the sand at those particular positions. In addition, the duration of the residual pore-water pressure remains relatively unchanged as compared to the least energy input of 3.6 kgm, maintaining a significant long period of 4.5 s at drop 1. Interestingly, we note that the residual pore-water pressure response was produced despite the absence of any corresponding interference along the same time period from the impact stress wave.

The range of residual pore-water pressure ratio recorded in these tests was found to range from 0.31 to 0.85. We note that the long residual pore-water pressure response was consistent with the study conducted by Ruygrok, 1980's impact experiments on nearly saturated sand. Hence, we have reasons to believe that under the limitations of experimental imperfections due to accuracy constraints of measuring devices, the sand column have experienced a high liquefaction potential for a significant period of time.

Figure 6.3 Top Pressure in final decay under lowest energy input of 3.6 kgm

Figure 6.4 Med Pressure in final decay under lowest energy input of 3.6 kgm

Figure 6.5 Bot Pressure in final decay under lowest energy input of 3.6 kgm

Figure 6.6 Top pressure in final decay under highest energy input of 8.5 kgm

Figure 6.7 Med pressure in final decay under highest energy input of 8.5 kgm

Figure 6.8 Bot pressure in final decay under highest energy input of 8.5 kgm

6.4 Effect of impact parameters

The trend of the residual pore-water pressure ratio will be investigated in accordance to the effects of changes in:

- Energy input
- Number of drops
- Drainage length
- Relative density of sand column

6.4.1 Energy input

Figures 6.9, 6.10 and 6.11 show the variation of the residual pore-water pressure ratios for the 3 positions over different energy inputs. We observed that there is a general increasing trend of the ratio in accordance to the raise in the energy input especially in the first few drops. When we reach the upper limit of the energy input, we found that there was marginal difference in the ratios suggesting that an optimal energy input range has been reached. We can see that at higher drop numbers, the effect of energy input becomes minimal as the sand reached its optimum densification and appears non-responsive to any further increase in impact energy. At the Bot location when its distance is comparatively farther away from the impact source, the effect of energy input seems to register little changes in the residual pore-water pressure ratio.

Figure 6.9 Variation of Residual Pore-water Pressure Ratio for Top position across different energy inputs

Figure 6.10 Variation of Residual Pore-water Pressure Ratio for Med position across different energy inputs

Figure 6.11 Variation of Residual Pore-water Pressure Ratio for Bot position across different energy inputs

6.4.2 Number of drops

Figures 6.12 and 6.13 show the variation of the residual pore-water pressure ratio, U_r/V_{eff} , under the least and highest energy inputs as the drop number increases. We see a consistent declining trend as the number of drops increases. This is probably due to a denser soil condition with subsequent drops. The residual pore-water pressure ratio range from 0.49 to 0.62 under the least energy input of 3.6 kgm to a range of 0.52 to 0.85 at the highest energy input of 8.5 kgm for the initial drop 1. As the drop number increases, the ratio stabilized to a final value of between 0.31 to 0.44 under the least energy input to a range of 0.39 to 0.48 for the highest energy input of 8.5 kgm.

6.4.3 Drainage length

Figures 6.12 and 6.13 also highlight the effect of changes in drainage length by showing the variation of the residual pore-water pressure ratios along the soil column represented by the 3 positions for the least and highest energy inputs. We see a higher response from the positions nearer to the source of impact as seen in Top and Med pressure trend line in the graph. The smaller response from Bot position is probably due to the dissipation of the impact stress wave as it propagates down the sand column.

Figure 6.12 Residual Pore-water Pressure Ratio of 3 positions along soil column under lowest energy input of 3.6 kgm

Figure 6.13 Residual Pore-water Pressure Ratio of 3 positions along soil column under highest energy input of 8.5 kgm

6.4.4 Relative Density of sand column

The settlement of the sand column following every impact drop was also translated into its corresponding change of relative density, $D_r(\%)$ to see the effect of the sand properties on its liquefaction potential as presented in Figure 6.14 (Least Energy Input of 3.6kgm). We see a parabolic trend for all three positions throughout the change of relative density registering the highest R_r at around 50% to 60% relative density. As we increase the energy input to 8.6 kgm, all three curves show different trend that approaches a cubic function as shown in Figure 6.15. There was a sharp drop of the R_r in the initial range of 40% to 55% relative density. The trend then remains fairly constant until it reached the higher range of 80% to 95% relative density where we saw another sharp drop.

Figure 6.14 Variation of Residual Pore-water Pressure Ratio over Relative Density under Lowest Energy Input of 3.6 kgm

Figure 6.15 Variation of Residual Pore-water Pressure Ratio over Relative Density under Highest Energy Input of 8.5 kgm

Although the pore-water pressure ratio tends to decrease at high relative density, the pore-water pressure ratio of the sand column still remains high for such dense condition. The most probable reason is due to a non-homogeneous densification along the sand column after impact. The upper part of the sand column is relatively much denser compared to the lower part of the column after many drops. A physical inspection of the sand samples after the conduct of every experiment reaffirmed this observation. The compacted sand column was scooped out in uniform layers after every test to prepare for the next test series. While scooping out the samples using a shove, there was a higher resistance in the upper layer of the sand column as compared to the lower layers. Towards the end of the removal process, a significant looseness of the sand samples was observed when they slipped easily out of the shove. This phenomenon could be explained from the use of a 900 mm high sand column since the position of the pressure transducers is relatively far away from the sand surface. If the sand's relative density after impact approaches homogeneous condition, for example in the shorter 500 mm sand column used in test series 9 and 10; the pore water pressure ratio is seen to drop significantly at the dense condition. This observation will be discussed in more details later in chapter 8.

Chapter 7 Measurement of Total Stress

7.1 Introduction

Another possible response of nearly saturated sand during impact that are of interest to us is the total stress ratio, T_h/I_h , which is defined here as the ratio of peak total horizontal stress (T_h) to peak impact stress (I_h). It helps us to understand the amount of pressure that is distributed to the sides under a vertical impact stress. For this purpose, a different existing adaptor was used to allow the pressure transducers by the sides of the steel column to measure the total horizontal stress directly without any filter. An illustration of the detailed setup is shown in Figure 7.2b

This chapter will present the results of the total horizontal stress measurements taken from test series 5 to 8. Figure 7.1 illustrates the experimental set-up for these test series. The energy input used have been deliberately calibrated to be as the same utilized in test series 1 to 4 for easy comparison and superposition of results. As before, the 900mm sand column will undergo 15 drops of impact for each test series. The peak and residual total horizontal stress response at 2 designated locations, 600 mm (Med Position) and 800 mm (Bot Position) from the top of the sand column will be our focus of study as shown in Figure 7.2a. The detailed data of drops 1-5, 10 and 15 of each test series are presented in Appendix E. The following notations will be used:

High. T_h : Peak horizontal total stress of each drop

(U_h , U_r , U_r/T_r , U_h/T_h notations are additional pore-water pressure readings taken at the Med position as consistency checks with earlier test series 1-4)

Figure 7.1 Setup for experimental test series 5 to 8

Figure 7.2a Location of total stress measurements along the sand column

Figure 7.2b Adaptor for the measurement of total horizontal stress

7.2 Total Horizontal Stress

Figures 7.3 and 7.4 represent the total horizontal stress acting against the sides of the sand column at the Med and Bot positions using the least energy input of 3.6 kgm. We seen in drop 1 that a short triangular response was seen within the first 10ms that gradually decline to a constant value. The Med and Bot positions recorded a peak impact stress of 148 kPa and 84 kPa respectively. At higher drops, a sharp peak was produced followed by a sinusoidal decreasing curve. The final peak stress recorded at drop 15 was increased to 204 kPa and 91 kPa for the Med and Bot positions respectively.

As we increase to the highest energy input, the results plotted in Figure 7.5 and 7.6 mirror a consistent trend described above for both positions. This time, we see an immediate sharp peak and sinusoidal decreasing trend even in drop 1. The peak impact stress of Med and Bot positions were increased to 240 kPa and 210 kPa at the initial drop before declining to 232 kPa and 162 kPa at the final drop.

Looking at the trend of the total stress ratio, T_h/I_h of both positions over the different energy inputs in Figure 7.7 and 7.8, we saw a general logarithmic decrease of the total stress ratio as the number of drops increases. The Med position saw a decrease of total stress ratio ranging from 0.5 to 0.75 at drop 1 to around 0.2 to 0.3 at final drop. The Bot position has a smaller change ranging from 0.35 to 0.45 at drop 1, to around 0.1 to 0.15 at the final drop. In conclusion, the total stress ratio was found to decrease with the increase of the relative density of sand in accordance with the number of drops.

Figure 7.3 Total horizontal stresses at Med position in 1st stage
from Least Energy Input of 3.6 kgm

Figure 7.4 Total horizontal stresses at Bot position in 1st stage
from Least Energy Input of 3.6 kgm

Figure 7.5 Total horizontal stresses in 1st stage at Med position
from highest energy input of 8.5 kgm

Figure 7.6 Total horizontal stresses in 1st stage at Bot position
from highest energy input of 8.5 kgm

Figure 7.7 Variation of total stress ratio, T_h/I_h , across different energy inputs for Med position

Figure 7.8 Variation of total stress ratio, T_h/I_h , across different energy inputs for Bot position

Chapter 8 Effect of Column length

8.1 Introduction

To investigate the effect of sand column length on its pore-water pressure response under an impact loading, test Series 9 and 10 were carefully planned to measure the dynamic pore-water pressure response of a shorter 500 mm sand column under a low energy input of 3.6 kgm and a high energy input of 8.5 kgm respectively (Refer to Figure 8.1 for illustration). This chapter will present the results of its impact pressure at the surface and the peak and residual pore-water pressure response at the 3 designated locations: 100 mm (Top), 200mm (Med) and 400mm (Bot) from the top of the sand column. The detailed experimental data are presented in Appendix F. All other symbols used are consistent with the longer 900 mm sand column described in test series 1 to 4.

Figure 8.1 Setup for experimental test series 9 and 10

8.2 Impact Stress

The general trend of the deceleration of the pounder and its computed stress upon impact under the least energy input of 3.6kgm is presented in Figures 8.2 and 8.3 respectively. Only drops 1, 5, 10 and 15 are presented to highlight the progressive changes as the number of drops increases. We can see that there is a rising peak within the first 10 seconds followed by a gradual decline of the stress curve from for drop 1. Upon reaching the higher number of drops from 5 to 15, we observed a sharp triangular peak followed by a rapid tapering off of the stress wave. Thus, we see the initial drops of the test producing a longer response of the stress wave as compared to the higher drop number of 5 to 15. This phenomenon could be best explained by the gradual densification of the soil column due to the repeated impacts by the pounder. The initial drops of the impact encountered relatively loose sand with a lower soil stiffness value as compared to the higher drop numbers. We can thus see the damping effects of the loose sands coming into effect to lengthen the impact stress response. At higher drop number, the sand has become much consolidated into a dense configuration, thus reflecting a sharp triangular response and a rapid retreat to the zero line.

As we increased the energy input to the maximum of 8.5 kgm, we see a similar trend as described in Figure 8.4 and 8.5. The maximum impact pressure was increased from 700 to 1400 kPa. However, we can see that due to the higher amount of energy used, the process of densification is activated at a faster rate as compared to the least energy input of 3.6 kgm.

For both impact energies, it was found that there is a similar behavior in the impact stresses: the impact stress of drop 1 shows relatively small amplitude with longer duration, whereas for drop 5, 10 and 15, the impact stresses indicate a relatively high peak and short duration. The reason is probably related to the change of relative density of the sand column as the number of drops increases. There is no significant difference on the amplitude of the impact stresses for either 500 mm lengths or 900 mm lengths at the same drops.

Figure 8.2 Impact deceleration record in the first 50 ms
from lowest energy input of 3.6 kgm

Figure 8.3 Calculated impact pressures in the first 50 ms
from lowest energy input of 3.6 kgm

Figure 8.4 Impact deceleration record in the first 50 ms
from highest energy input of 8.5 kgm

Figure 8.5 Calculated impact pressures in the first 50 ms
from highest energy input of 8.5 kgm

8.3 Pore-water Pressure

This section will describe the differences of the pore-water pressure response in a shorter 500 mm sand column used in test series 9 and 10 as compared to the 900 mm long sand column used in previous test series. Figure 8.6 gives a pictorial illustration of the location of the 3 pressure transducer to capture the immediate and residual pore-water pressure response under impact loadings.

Figure 8.6 Location of pressure transducers along the sand column

8.3.1 Immediate Pore-water Pressure

Figures 8.7, 8.8 and 8.9 show the pore-water pressure response of the Top, Med and Bot positions in the first stage upon impact under the lowest energy input of 3.6 kgm. We see practically little response at the Top and Med positions with 5.14 kPa and 6.14 kPa. Being at a position of longer drainage path, the Bot position managed to register a small rise of 14 kPa that gradually decline to a residual value at drop 1. Thereafter, a sinusoidal waveform appears beyond drop 5 at the Bot position while the Top and Med position continue to remain silent. The production of negative pore-water pressure produced at the lower part of the sand column probably indicates the existence of tensional waves which drives the potential for liquefaction to occur under an impact loading. At the final drop 15, the peak pore-water pressure at the Top and Med positions drop to 4.5 kPa and 4.6 kPa while the Bot position increased to 30.6 kPa.

When the energy input was increased to 8.5 kgm, a corresponding increase in the peak pressure was observed as shown in Figures 8.10, 8.11 and 8.12 for the 3 positions. The top position remained slightly indifferent to the increase in energy input, recording only an increase to 6.5 kPa at drop 1 and then reducing to 6.14 kPa at the final drop 15. We see an awakening response at the Med position parallel to the Bot position now, hitting 30.6 kPa and 26.5 kPa at drop 1 and increasing to 63.0 kPa and 68.9 kPa at the final drop 15 respectively.

Summarizing the results from different energy inputs, the peak pore-water pressure ratio, U_h/I_h was plotted in Figures 8.13 and 8.14. We see a consistent declining trend as the number of drops increases. The peak pore-water pressure ratio range from 0.02 to 0.05 under the least energy input of 3.6 kgm to a range of 0.02 to 0.12 at the highest energy input of 8.5 kgm for the initial drop 1. As the drop number increases, the ratio stabilized to a final value of between 0.01 to 0.05 under the least energy input to a range of 0.0 to 0.05 for the highest energy input of 8.5 kgm. It further highlights the importance of drainage path lengths by showing the variation of the peak pore-water pressure ratios along the soil column represented by the 3 positions under the least and highest energy inputs as the drop number increases. We see a practically little response from the positions nearer to the source of impact as seen in Top and Med pressure trend line in the graph as compared to Bot Position.

Figure 8.7 ‘Top’ pore-water pressure of a 500 mm sand column in 1st stage under lowest energy input of 3.6 kgm

Figure 8.8 ‘Med’ pore-water pressure of a 500 mm sand column in 1st stage under lowest energy input of 3.6kgm

Figure 8.9 ‘Bot’ pore-water pressure of a 500 mm sand column in 1st stage under lowest energy input of 3.6 kgm

Figure 8.10 ‘Top’ pore-water pressure of a 500 mm sand column in 1st stage under highest energy input of 8.5 kgm

Figure 8.11 ‘Med’ pore-water pressure of a 500 mm sand column in 1st stage under highest energy input of 8.5 kgm

Figure 8.12 Bot’ pore-water pressure of a 500 mm sand column in 1st stage under highest energy input of 8.5 kgm

Figure 8.13 Peak Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) in a 500 mmsand column under the lowest energy input of 3.6 kgm

Figure 8.14 Peak Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) in a 500 mmsand column under the highest energy input of 8.5 kgm

8.3.2 Residual Pore-water Pressure

The residual pore-water pressure response focused on the excess pore-water pressure remaining after the decay of the impact stress wave. All the figures that follow are snapshots in 2nd stage of the pore-water pressure response as illustrated in Figure 5.1. The complete experimental data of test series 9 to 10 can be viewed from Appendix F.

Looking at Figures 8.15, 8.16 and 8.17 which represent the residual pore-water pressure response of the three positions under the small energy input of 3.6 kgm, we observed practically no response from the Top position throughout the 15 drops since it did not have enough drainage path to build up any residual pressure. We see small residual pore-water pressure only at Med and Bot position at 1.94 kPa and 1.68 kPa respectively which lasts only for about 0.5s to 1s at drop 1. Both positions experienced a much lower residual pore-water pressure of 0.57 kPa and 0.47 kPa respectively at the final drop 15, which lasts for less than 0.3 s.

As we increased the energy input to 8.5 kgm, the same general trend was observed as shown in Figures 8.18, 8.19 and 8.20 for the three respective positions. This additional increase of energy however did not translate to a significant higher potential for liquefaction to occur in the short 500 mm sand column. The Top position remained silent throughout the 15 drops while the Med and Bot position only deliver a 1.16 kPa and 1.64 kPa return respectively at drop 1. At the end drop 15, both positions register near zero residual pore-water pressure. The duration of any small response was around the same as that produced under the small energy input in test series 9.

Summarizing the results from different energy inputs, the residual pore-water pressure ratio U_r/V_{eff} was plotted in Figure 8.21 and 8.22 for the three positions. We see a consistent declining trend as the number of drops increases. The residual pore-water pressure ratio range from 0.11 to 0.36 under the low energy input of 3.6 kgm to a range of 0.03 to 0.36 at the high energy input of 8.5 kgm for the initial drop 1. As the drop number increases, the ratio stabilized to a final value of between 0.03 to 0.10 under the low energy input to a range of 0.0 to 0.01 for the high energy input. We see that the variation in energy input has little impact on the sand's residual pore-water pressure response due to its short drainage path.

Figure 8.15 Top Pressure of a 500 mm sand column in final decay under lowest energy input of 3.6 kgm

Figure 8.16 Med Pressure of a 500 mm sand column in final decay under lowest energy input of 3.6 kgm

Figure 8.17 Bot Pressure of a 500 mm sand column in final decay under lowest energy input of 3.6kgm

Figure 8.18 Top pressure of a 500 mm sand column in final decay under highest energy input of 8.5 kgm

Figure 8.19 Med pressure of a 500 mm sand column in final decay under highest energy input of 8.5 kgm

Figure 8.20 Bot pressure of a 500 mm sand column in final decay under highest energy input of 8.5 kgm

Figure 8.21 Residual Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) in a 500 mm sand column under lowest energy input of 3.6 kgm

Figure 8.22 Residual Pore-water Pressure Ratio of the 3 positions (Top, Med, Bot) in a 500 mmsand column under highest energy input of 8.5 kgm

8.4 Variation of peak pore-water pressure ratio along column

In trying to gain a rough understanding of the pore-water pressure changes with respect to the changes of the drainage path, $d(\text{mm})$ in the sand column, the results of test series 1, 4, 9 and 10 were combined assuming the same initial sand relative density of 35%. The results were compared at the least and highest impact energy used at both the 900mm and 500 mm sand column shown in Figures 8.23 and 8.24. The Top, Med and Bot results from the 500 mm sand column derived in test series 9 and 10 were assumed to represent the locations at 100, 200 and 400 mm of the 900 mm sand column in test series 1 and 4. Only the initial drop 1 and the final drop 15 were plotted to highlight the changes across the drops. The peak pore-water pressure ratio, U_h/I_h , was found to increase linearly with the increase of the drainage path as the depth of sand increases. At the final drop 15, we can see nearly straight line parallel to the x-axis which represents little changes along the sand column, probably due to the dense sand condition after 15 drops.

Figure 8.23 Variation of Peak Pore-water Pressure Ratio along Sand Column for least energy input of 3.6 kgm

Figure 8.24 Variation of Peak Pore-water Pressure Ratio along Sand Column for highest energy input of 8.5 kgm

8.5 Variation of residual pore-water pressure ratio along column

Combining the results of test series 1, 4, 9 and 10 as described in the above section 8.1, the variation of residual pore-water pressure ratio, U_r/V_{eff} , along the sand column under the least and highest energy input was plotted in Figures 8.25 and 8.26 respectively. At low energy input, the ratio was found to increase in a parabolic trend that starts to decline beyond 600 mm depth of sand column. Upon reaching the final drop 15, the 1st 150 mm depth of sand shows little response. There was a gradual decrease in the Middle section which also sees a decline beyond 600 mm. At higher energy, we see a conforming trend of drop 1 and 15 in the 1st 100 mm depth of sand, showing little changes. Both trend-lines start to increase in the Middle section and again decrease beyond 600 mm. Thus, we can conclude that the sand column probably experienced the highest liquefaction potential at around 600 mm depth which is $2/3$ of the full sand column height.

Figure 8.25 Variation of Residual Pore-water Pressure Ratio along Sand Column for least energy input of 3.6 kgm

Figure 8.26 Variation of Residual Pore-water Pressure Ratio along Sand Column for highest energy input of 8.5 kgm

Chapter 9 Summary, Conclusions and Recommendations

9.1 Summary

Pore-water pressures in sand fills have been an interesting topic of study especially in land reclamation. Of particular importance in this research study is its response under an impact loading. Bridging the gap in the unknown of soil dynamics, the liquefaction potential of wet sand under such a dynamic loading will be of interest to us. This experimental investigation represents an effort to examine and establish the possibility of compressionally-induced liquefaction under an impact loading by studying its pore-water pressure response.

A new experimental soil dynamics apparatus has been developed to investigate the immediate and transient pore-water pressure response of nearly saturated sands in regard to changes in impact energy and distance away from the dynamic source. The specimens were dynamically loaded within a 1050 mm rigid steel pipe of 150 mm diameter by dropping a steel pounder of 110 mm diameter on it at varying heights of 0.3 to 0.5 m. This simulation apparatus is capable of generating compressive shock loadings on the order of 3000 kPa with millisecond rise times to peak stress. All the test samples were prepared at a standard relative density of 35% and a 95% degree of saturation to approximate the actual field condition in a land reclamation project. Each sample was subjected to 15 one-dimensional compressive loadings under a partially drained condition. Experimental results have shown that it is possible to achieve a high liquefaction potential under these conditions. Significant peak and residual pore-water pressure were observed even at compacted conditions and low energy input.

We can see from this investigation that liquefaction becomes more difficult to induce at higher drop numbers and farther distance away from the source. The potential for liquefaction increases as we increase the energy input but reaching a limiting range beyond a certain energy input.

A study of the general trend from the experimental data has yielded simple empirical models to estimate the peak and residual pore-water pressure ratio as functions of energy input, sand depth and relative density.

9.2 Conclusion

Based on the results of this experimental investigation on wet sand, the following conclusions were drawn.

- The new experimental design to capture the immediate and residual pore-water pressure response of the sand column under impact loadings was proved to be successful and effective. In addition, the total horizontal stress during impact can be measured successfully.
- The relationship between stress and strain during the impact is non-linear, deviating from Ginsburg's model (1964) especially at the initial drop.
- The transient pore-water pressure response of nearly saturated sand to impact loading consists of two stages: the immediate pore-water pressure and residual pore-water pressure.
- A high risk of liquefaction, induced by one-dimensional, impact loadings in nearly saturated Ottawa sand under partially drained conditions at an initial relative density of 35%, has been observed. The negative excess pore-water pressure produced prior to the long period of residual pore-water pressures indicates that tension waves might have been formed. These tension waves result in a tendency for the sand particles to dilate. This could be a probable cause for the generation of residual pore-water pressure.
- The sand's drainage length, relative density of sand, number of impact drops and energy input are significant factors that determine the liquefaction potential. The magnitude of the peak pore-water pressure was observed to have an increasing trend in accordance with any increase in the above four parameters. The magnitude and duration of the residual pore-water pressure was however found to follow an inverse relationship to the increase of drainage length, relative density of sand and the number of drops. Only the increase of energy input causes an increase in the magnitude and duration of the residual pore-water pressure.

- The sand column was suggested to have the highest liquefaction potential at $\frac{2}{3}$ depth of the full height.
- There was a steep attenuation of stress waves along the sand column as seen from the decrease in pore-water pressure response as the depth of sand increases.

9.3 Recommendations

The results of this investigation and the information currently available in the literature have provided insight into the research needs to better understand the phenomenon of liquefaction as produced by impact loadings. Based on this, a number of recommendations can be made.

- Research efforts should be extended to investigate the effects of different soil types with different initial relative densities and varying degrees of saturation on liquefaction potential of soils under impact loading.
- Attention could be given to evaluate both two and three dimensional loading phenomenon.
- Develop a series of instrumentation to evaluate the effective vertical stress of the wet sand along the sand column during impact loadings.
- A more comprehensive response could be collected by using more transducers to gather reliable measurements along the sand column.

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Appendix A

Calibration Data of Accelerometer

Figure A.1 Accelerometer Calibration Curve, Peak to peak voltage = 28.25mv, Sensitivity of Accelerometer = 14.125mv/g)

Appendix B

Calibration Data of Transducers

Pressure (kPa)	Transducer reading (mV)
0	0.0
50	2.9
100	5.7
200	11.3
300	16.9
400	22.5
500	26.2
600	32.4
700	37.8
800	43.2
900	48.6
1000	54.0

Table B-1 Calibration data of Kulite pressure transducer 613

Figure B-1 Calibration graph of pressure transducer 613 reading (mV) against pressure calibrator(kPa),
Sensitivity = 0.0536 mv/kPa

Pressure (kPa)	Transducer reading (mV)
0	0
50	3
100	7
200	14
300	22
400	29
500	36
600	44
700	52
800	60
900	68
1000	77

Table B-2 Calibration data of Kulite pressure transducer 615

Figure B-2 Calibration graph of pressure transducer 615 reading (mV) against pressure calibrator(kPa),
Sensitivity = 0.0765 mv/kPa

Pressure (kPa)	Transducer reading (mV)
0	0.0
50	2.1
100	4.2
200	8.3
300	12.5
400	16.6
500	20.7
600	24.9
700	28.9
800	33.0
900	37.0
1000	41.0

Table B-3 Calibration data of Kulite pressure transducer 619

Figure B-3 Calibration graph of pressure transducer 619 reading (mV) against pressure calibrator(kPa),
Sensitivity = 0.0411 mv/kPa

Pressure (kPa)	Transducer reading (mV)
0	0
50	-20
100	-38
200	-75
300	-113
400	-151
500	-188
600	-226
700	-265
800	-302
900	-341
1000	-379

Table B-4 Calibration data of Kulite Earth pressure cell

Figure B-4 Calibration graph of LQ-080 reading (mV) against pressure calibrator(kPa), Sensitivity = - 0.3788 mV/kPa

Appendix C

Calibration Data of Sand's Initial Properties

Figure C.1 Variation of Wet Sand's Initial Degree of Saturation and Relative Density along the sand column, average Initial Relative Density = 35%, Average Degree of Saturation = 95%

Appendix D

Experimental Data of Test Series 1-4

Test Series 1

900mm Sand Column

Energy Input : 300mm Drop Height – 12.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 369.64$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.84	2.38	2.22
High.Uh/kPa	25.56	21.18	54.16
Uh/Ih	0.07	0.06	0.15
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.62	0.52	0.49

Summary of pressure readings from drop 2
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 188.74$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.89	2.39	2.13
High.Uh/kPa	22.01	24.55	34.92
Uh/Ih	0.12	0.13	0.19
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.63	0.52	0.47

Summary of pore-water pressure readings from drop 3
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 221.50$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.94	2.55	2.06
High.Uh/kPa	19.53	29.34	83.93
Uh/Ih	0.09	0.13	0.38
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.65	0.56	0.45

Summary of pore-water pressure readings from drop 4
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 418.78$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.72	3.19	2.53
High.Uh/kPa	14.60	36.40	103.18
Uh/Ih	0.03	0.09	0.25
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.81	0.70	0.56

Summary of pore-water pressure readings from drop 5
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 655.24$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.24	2.68	2.19
High.Uh/kPa	12.51	30.15	92.81
Uh/Ih	0.02	0.05	0.14
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.71	0.59	0.48

Summary of pore-water pressure readings from drop 10
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 811.22$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.65	2.37	1.91
High.Uh/kPa	9.17	26.72	78.26
Uh/Ih	0.01	0.03	0.10
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.58	0.52	0.42

Summary of pore-water pressure readings from drop 15
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 567.64 kPa
Static stress at surface, T_{static} = 12.62 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.03	1.74	1.43
High.Uh/kPa	7.94	17.54	52.56
Uh/Ih	0.01	0.03	0.09
Veff Str/kPa	4.56	4.55	4.53
Ur/Veff.	0.44	0.38	0.31

Test Series 2

900mm Sand Column

Energy Input : 300mm Drop Height – 17.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 198.67 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.10	2.54	2.34
High.Uh/kPa	31.24	34.99	73.54
Uh/Ih	0.16	0.18	0.37
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.68	0.56	0.52

Summary of pore-water pressure readings from drop 2
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 180.61 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.60	2.26	1.95
High.Uh/kPa	26.84	32.82	61.31
Uh/Ih	0.15	0.18	0.34
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.57	0.50	0.43

Summary of pore-water pressure readings from drop 3
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 329.11$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.64	2.29	1.95
High.Uh/kPa	21.78	42.50	70.73
Uh/Ih	0.07	0.13	0.21
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.58	0.50	0.43

Summary of pore-water pressure readings from drop 4
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 453.53 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.85	2.46	1.99
High. Uh/kPa	13.39	50.62	95.89
Uh/Ih	0.03	0.11	0.21
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.62	0.54	0.44

Summary of pore-water pressures from drop 5
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 534.81$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.53	3.04	2.36
High.Uh/kPa	14.47	51.83	133.59
Uh/Ih	0.03	0.10	0.25
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.77	0.67	0.52

Summary of pore-water pressures from drop 10
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 791.68$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.77	2.36	2.07
High.Uh/kPa	8.40	83.59	133.46
Uh/Ih	0.01	0.11	0.17
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.61	0.52	0.46

Summary of pore-water pressure readings from drop 15
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 899.04 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.12	1.82	1.60
High.Uh/kPa	6.62	60.10	93.17
Uh/Ih	0.01	0.07	0.10
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.47	0.40	0.35

Test Series 3

900mm Sand Column

Energy Input : 400mm Drop Height – 17.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 292.99 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. Ur/kPa	3.85	3.10	2.51
High.Uh/kPa	27.87	39.95	64.54
Uh/Ih	0.10	0.14	0.22
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.84	0.68	0.55

Summary of pore-water pressure readings from drop 2
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 143.49$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U_r /kPa	3.21	2.85	2.31
High. U_h /kPa	27.79	42.04	50.85
U_h/I_h	0.19	0.29	0.35
V_{eff} Str/kPa	4.57	4.55	4.53
U_r/V_{eff}	0.70	0.63	0.51

Summary of pore-water pressure readings from drop 3
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 430.46$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.97	2.58	2.08
High.Uh/kPa	21.92	44.15	92.71
Uh/Ih	0.05	0.10	0.22
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.65	0.57	0.46

Summary of pore-water pressure readings from drop 4
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 580.96$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.56	2.97	2.49
High.Uh/kPa	19.53	65.32	151.28
Uh/Ih	0.03	0.11	0.26
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.78	0.65	0.55

Summary of pore-water pressure readings from drop 5
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 579.96$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.35	2.74	2.37
High.Uh/kPa	22.11	85.03	143.01
Uh/Ih	0.04	0.15	0.25
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.73	0.60	0.52

Summary of pore-water pressure readings from drop 10
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 798.70$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.50	2.11	1.93
High.Uh/kPa	17.30	110.75	113.62
Uh/Ih	0.02	0.14	0.14
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.55	0.46	0.43

Summary of pore-water pressure readings from drop 15
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 1331.50$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.21	1.94	1.79
High.Uh/kPa	13.82	81.75	101.65
Uh/Ih	0.01	0.06	0.08
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.48	0.43	0.39

Test Series 4

900mm Sand Column

Energy Input : 500mm Drop Height – 17.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 323.09$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.90	3.34	2.33
High.Uh/kPa	48.29	62.61	106.48
Uh/Ih	0.15	0.19	0.33
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.85	0.73	0.52

Summary of pore-water pressure readings from drop 2
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 215.73$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.85	2.54	2.03
High. Uh/kPa	46.29	51.22	56.39
Uh/Ih	0.21	0.24	0.26
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.62	0.56	0.45

Summary of pore-water pressure readings from drop 3
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 506.71$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.87	2.59	2.39
High. Uh/kPa	33.49	69.46	110.64
Uh/Ih	0.07	0.14	0.22
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.63	0.57	0.53

Summary of pore-water pressure readings from drop 4
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 845.86$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.28	2.70	2.26
High.Uh/kPa	45.26	92.66	169.10
Uh/Ih	0.05	0.11	0.20
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.72	0.59	0.50

Summary of pore-water pressure readings from drop 5
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 564.91 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	3.37	2.83	2.27
High.Uh/kPa	49.95	99.11	118.01
Uh/Ih	0.09	0.18	0.21
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.74	0.62	0.50

Summary of pore-water pressure readings from drop 10
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 1067.61 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U /kPa	2.55	2.32	2.11
High. U_h /kPa	40.27	148.76	140.60
U_h/I_h	0.04	0.14	0.13
V_{eff} Str/kPa	4.57	4.55	4.53
U_r/V_{eff}	0.56	0.51	0.47

Summary of pore-water pressure readings from drop 15
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 1069.62$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bottom
Hgt/mm	475	575	775
Res. U/kPa	2.20	2.12	1.79
High.Uh/kPa	36.00	111.25	111.66
Uh/Ih	0.03	0.10	0.10
Veff Str/kPa	4.57	4.55	4.53
Ur/Veff.	0.48	0.46	0.39

Appendix E

Experimental Data of Test Series 5-8

Test Series 5

900mm Sand Column

Energy Input : 300mm Drop Height – 12.23kg Drop Weight

Summary of pressure readings from drop 1
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	12.90	-
Res. Ur/kPa	3.23	-
High. Th/kPa	147.86	84.08
Res. Tr/kPa	3.75	2.34
Res Ur/Tr	0.86	-
High Uh/Th	0.09	-

Highest Impact Stress, Is = 244.29 kPa

Summary of pressure readings from drop 2
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	11.29	-
Res. Ur/kPa	2.44	-
High. Th/kPa	148.82	87.06
Res. Tr/kPa	2.72	1.85
Res Ur/Tr	0.90	-
High Uh/Th	0.08	-

Highest Impact Stress = 163.10 kPa

Summary of pressure readings from drop 3
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mm Sand Colm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	9.07	-
Res. Ur/kPa	2.21	-
High. Th/kPa	186.26	104.73
Res. Tr/kPa	2.58	1.63
Res Ur/Tr	0.86	-
High Uh/Th	0.05	-

Highest Impact Stress = 353.97 kPa

Summary of pressure readings from drop 4
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	7.06	-
Res. Ur/kPa	1.99	-
High. Th/kPa	218.38	131.76
Res. Tr/kPa	2.23	1.72
Res Ur/Tr	0.89	-
High Uh/Th	0.03	-

Highest Impact Stress = 496.42 kPa

Summary of pressure readings from drop 5
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	6.55	-
Res. Ur/kPa	2.29	-
High. Th/kPa	217.82	125.28
Res. Tr/kPa	2.53	1.77
Res Ur/Tr	0.91	-
High Uh/Th	0.03	-

Highest Impact Stress = 455.82 kPa

Summary of pressure readings from drop 10
(900mm Sand Colm-35% - 300mm drop Height- 12.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	6.30	-
Res. Ur/kPa	1.51	-
High. Th/kPa	202.58	102.89
Res. Tr/kPa	1.56	1.20
Res Ur/Tr	0.97	-
High Uh/Th	0.03	-

Highest Impact Stress = 832.58 kPa

Summary of pressure readings from drop 15
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	4.53	-
Res. Ur/kPa	1.08	-
High. Th/kPa	204.27	91.06
Res. Tr/kPa	1.11	0.97
Res Ur/Tr	0.98	-
High Uh/Th	0.02	-

Highest Impact Stress = 925.88 kPa

Test Series 6

900mm Sand Column

Energy Input : 300mm Drop Height – 17.23kg Drop Weight

Summary of pressure readings from drop 1
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	17.33	-
Res. Ur/kPa	3.55	-
High. Th/kPa	126.34	80.44
Res. Tr/kPa	4.02	2.53
Res Ur/Tr	0.88	-
High Uh/Th	0.14	-

Highest Impact Stress = 207.70 kPa

Summary of pressure readings from drop 2
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	13.01	-
Res. Ur/kPa	2.64	-
High. Th/kPa	180.60	112.78
Res. Tr/kPa	2.80	2.09
Res Ur/Tr	0.94	-
High Uh/Th	0.07	-

Highest Impact Stress = 302.02 kPa

Summary of pressure readings from drop 3
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	9.68	-
Res. Ur/kPa	3.24	-
High. Th/kPa	237.35	127.79
Res. Tr/kPa	3.58	2.24
Res Ur/Tr	0.90	-
High Uh/Th	0.04	-

Highest Impact Stress = 421.43 kPa

Summary of pressure readings from drop 4
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	8.07	-
Res. Ur/kPa	3.25	-
High. Th/kPa	221.18	129.84
Res. Tr/kPa	3.64	2.23
Res Ur/Tr	0.89	-
High Uh/Th	0.04	-

Highest Impact Stress = 461.56 kPa

Summary of pressure readings from drop 5
(900mm Sand Colm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	6.55	-
Res. Ur/kPa	2.95	-
High. Th/kPa	241.92	147.01
Res. Tr/kPa	3.29	2.10
Res Ur/Tr	0.90	-
High Uh/Th	0.03	-

Highest Impact Stress = 492.67 kPa

Summary of pressure readings from drop 10
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	5.95	-
Res. Ur/kPa	2.33	-
High. Th/kPa	188.33	119.18
Res. Tr/kPa	2.47	1.72
Res Ur/Tr	0.94	-
High Uh/Th	0.03	-

Highest Impact Stress = 692.34 kPa

Summary of pressure readings from drop 15
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 300mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	4.54	-
Res. Ur/kPa	1.80	-
High. Th/kPa	205.83	103.47
Res. Tr/kPa	1.90	1.36
Res Ur/Tr	0.95	-
High Uh/Th	0.02	-

Highest Impact Stress = 1328.49 kPa

Test Series 7

900mm Sand Column

Energy Input : 400mm Drop Height – 17.23kg Drop Weight

Summary of pressure readings from drop 1
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	28.78	-
Res. Ur/kPa	3.15	-
High. Th/kPa	166.96	92.69
Res. Tr/kPa	3.63	2.17
Res Ur/Tr	0.87	-
High Uh/Th	0.17	-

Highest Impact Stress = 249.84 kPa

Summary of pressure readings from drop 2
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	35.03	-
Res. Ur/kPa	2.25	-
High. Th/kPa	132.48	79.43
Res. Tr/kPa	2.47	1.84
Res Ur/Tr	0.91	-
High Uh/Th	0.26	-

Highest Impact Stress = 255.87 kPa

Summary of pressure readings from drop 3
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	43.90	-
Res. Ur/kPa	1.98	-
High. Th/kPa	256.37	145.81
Res. Tr/kPa	2.17	1.62
Res Ur/Tr	0.91	-
High Uh/Th	0.17	-

Highest Impact Stress = 603.04 kPa

Summary of pressure readings from drop 4
(900mm Sand Colm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	46.27	-
Res. Ur/kPa	1.92	-
High. Th/kPa	257.56	143.34
Res. Tr/kPa	2.15	1.65
Res Ur/Tr	0.89	-
High Uh/Th	0.18	-

Highest Impact Stress = 687.32 kPa

Summary of pressure readings from drop 5
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	49.34	-
Res. Ur/kPa	2.31	-
High. Th/kPa	256.72	152.17
Res. Tr/kPa	2.56	1.69
Res Ur/Tr	0.90	-
High Uh/Th	0.19	-

Highest Impact Stress = 666.25 kPa

Summary of pressure readings from drop 10
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	68.20	-
Res. Ur/kPa	1.69	-
High. Th/kPa	258.65	162.34
Res. Tr/kPa	1.80	1.32
Res Ur/Tr	0.94	-
High Uh/Th	0.26	-

Highest Impact Stress = 1039.52 kPa

Summary of pressure readings from drop 15
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Pressure from drop 15
(900mmSandColm-35% - 400mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	88.06	-
Res. Ur/kPa	1.78	-
High. Th/kPa	216.40	146.52
Res. Tr/kPa	1.94	1.42
Res Ur/Tr	0.91	-
High Uh/Th	0.41	-

Highest Impact Stress = 930.15 kPa

Test Series 8

900mm Sand Column

Energy Input : 500mm Drop Height – 17.23kg Drop Weight

Summary of pressure readings from drop 1
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	26.85	-
Res. Ur/kPa	3.05	-
High. Th/kPa	240.16	209.86
Res. Tr/kPa	3.60	2.61
Res Ur/Tr	0.85	-
High Uh/Th	0.11	-

Highest Impact Stress = 626.12 kPa

Summary of pressure readings from drop 2
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	28.26	-
Res. Ur/kPa	2.30	-
High. Th/kPa	239.72	153.95
Res. Tr/kPa	2.60	1.86
Res Ur/Tr	0.88	-
High Uh/Th	0.12	-

Highest Impact Stress = 417.41 kPa

Summary of pressure readings from drop 3
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	24.94	-
Res. Ur/kPa	2.24	-
High. Th/kPa	238.53	154.77
Res. Tr/kPa	2.62	1.63
Res Ur/Tr	0.86	-
High Uh/Th	0.10	-

Highest Impact Stress = 637.15 kPa

Summary of pressure readings from drop 4
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	31.39	-
Res. Ur/kPa	2.74	-
High. Th/kPa	240.77	177.16
Res. Tr/kPa	2.90	2.04
Res Ur/Tr	0.94	-
High Uh/Th	0.13	-

Highest Impact Stress = 748.53 kPa

Summary of pressure readings from drop 5
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	33.20	-
Res. Ur/kPa	2.42	-
High. Th/kPa	240.97	192.17
Res. Tr/kPa	2.52	1.76
Res Ur/Tr	0.96	-
High Uh/Th	0.14	-

Highest Impact Stress = 831.81 kPa

Summary of pressure readings from drop 10
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	75.65	-
Res. Ur/kPa	1.52	-
High. Th/kPa	243.37	179.13
Res. Tr/kPa	1.61	1.42
Res Ur/Tr	0.94	-
High Uh/Th	0.31	-

Highest Impact Stress = 1567.30 kPa

Summary of pressure readings from drop 15
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Summary of three pressure readings for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(900mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight)

Position	Med	Bot
Hgt/mm	575	775
High. Uh/kPa	61.22	-
Res. Ur/kPa	1.65	-
High. Th/kPa	231.90	161.77
Res. Tr/kPa	1.77	1.34
Res Ur/Tr	0.93	-
High Uh/Th	0.26	-

Highest Impact Stress = 1159.92 kPa

Appendix F

Experimental Data of Test Series 9-10

Test Series 9

500mm Sand Column

Energy Input : 300mm Drop Height – 12.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 274.20$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.91	1.94	1.68
High.Uh/kPa	5.14	6.14	14.18
Uh/Ih	0.02	0.02	0.05
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.11	0.35	0.36

Summary of pore-water pressure readings from drop 2
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 353.26 kPa
Static stress at surface, T_{static} = 12.62 kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.54	1.13	1.23
High.Uh/kPa	4.84	5.34	18.23
Uh/Ih	0.01	0.02	0.05
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.07	0.21	0.27

Summary of pore-water pressure readings from drop 3
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 564.79$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.37	1.28	1.52
High.Uh/kPa	4.64	5.71	30.75
Uh/Ih	0.01	0.01	0.05
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.05	0.23	0.33

Summary of pore-water pressure readings from drop 4
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 522.06$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.38	1.53	1.60
High.Uh/kPa	5.35	5.59	17.60
Uh/Ih	0.01	0.01	0.03
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.05	0.28	0.35

Summary of pore-water pressure readings from drop 5
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 698.68 kPa
Static stress at surface, T_{static} = 12.62 kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.81	1.67	1.45
High.Uh/kPa	5.35	5.99	18.11
Uh/Ih	0.01	0.01	0.03
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.10	0.30	0.31

Summary of pore-water pressure readings from drop 10
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 555.53$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.26	0.77	0.85
High.Uh/kPa	5.24	5.62	28.79
Uh/Ih	0.01	0.01	0.05
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.03	0.14	0.18

Summary of pore-water pressure readings from drop 15
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(500mmSandColm-35% - 300mm drop Height- 12.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 580.46$ kPa
Static stress at surface, $T_{static} = 12.62$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.28	0.57	0.47
High.Uh/kPa	4.54	4.56	30.64
Uh/Ih	0.01	0.01	0.05
Veff Str/kPa	8.12	5.48	4.61
Ur/Veff.	0.03	0.10	0.10

Test Series 10

500mm Sand Column

Energy Input : 500mm Drop Height – 17.23kg Drop Weight

Summary of pore-water pressure readings from drop 1
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 1

Deceleration graph of drop 1

Impact Stress from drop 1
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 260.88$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.31	1.16	1.64
High.Uh/kPa	6.45	30.62	26.51
Uh/Ih	0.02	0.12	0.10
Veff Str/kPa	10.80	5.93	4.59
U_r/V_{eff}	0.03	0.20	0.36

Summary of pore-water pressure readings from drop 2
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 2

Deceleration graph of drop 2

Impact Stress from drop 2
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, I_h = 457.55 kPa
Static stress at surface, T_{static} = 17.78 kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.10	0.76	1.31
High.Uh/kPa	5.14	29.63	46.51
Uh/Ih	0.01	0.06	0.10
Veff Str/kPa	10.80	5.93	4.59
Ur/Veff.	0.01	0.13	0.28

Summary of pore-water pressure readings from drop 3
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 3

Deceleration graph of drop 3

Impact Stress from drop 3
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 1003.39$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.08	1.03	1.82
High.Uh/kPa	5.75	40.66	43.94
Uh/Ih	0.01	0.04	0.04
Veff Str/kPa	10.80	5.93	4.59
U_r/V_{eff}	0.01	0.17	0.40

Summary of pore-water pressure readings from drop 4
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 4

Deceleration graph of drop 4

Impact Stress from drop 4
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 810.74$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.08	0.23	0.92
High.Uh/kPa	5.24	53.62	48.86
Uh/Ih	0.01	0.07	0.06
Veff Str/kPa	10.80	5.93	4.59
Ur/Veff.	0.01	0.04	0.20

Summary of pore-water pressure readings from drop 5
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 5

Deceleration graph of drop 5

Impact Stress from drop 5
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 1176.98$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.02	0.85	1.28
High.Uh/kPa	4.94	53.33	64.25
Uh/Ih	0.00	0.05	0.05
Veff Str/kPa	10.80	5.93	4.59
Ur/Veff.	0.00	0.14	0.28

Summary of pore-water pressure readings from drop 10
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pressure locations for the first 100ms at drop 10

Deceleration graph of drop 10

Impact Stress from drop 10
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 958.24$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.15	0.55	0.75
High.Uh/kPa	5.04	66.91	90.75
Uh/Ih	0.01	0.07	0.09
Veff Str/kPa	10.80	5.93	4.59
Ur/Veff.	0.01	0.09	0.16

Summary of pore-water pressure readings from drop 15
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Summary of three pore-water pressures for the first 100ms at drop 15

Deceleration graph of drop 15

Impact Stress from drop 15
(500mmSandColm-35% - 500mm drop Height- 17.23kg drop Weight-NoCap)

Highest Impact Stress, $I_h = 1350.57$ kPa
Static stress at surface, $T_{static} = 17.78$ kPa

Position	Top	Med	Bot
Hgt/mm	75	175	375
Res. U/kPa	0.02	0.03	0.03
High.Uh/kPa	6.14	62.95	68.87
Uh/Ih	0.00	0.05	0.05
Veff Str/kPa	10.80	5.93	4.59
Ur/Veff.	0.00	0.00	0.01

Appendix G

**Engineering drawing of newly designed adaptor for pore-water
measurement**

All dimensions in mm

Figure G.1 Engineering Drawing of Adaptor for Pore-water Measurement