EVALUATION OF THE
TRI STAR
VIBROCOMPACTION PROBE

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ABSTRACT

The Franki Tri Star method for deep compaction of saturated sands has been used on a project on Annacis Island, Vancouver, B.C., Canada. The objective of soil compaction was to stabilize a deep saturated sand deposit, which was susceptible to liquefaction during strong earthquakes.

The project involved the compaction of a 750 feet long by 15 feet wide strip of sand deposits down to 10m depth. The sand was overlain by a fill and a clayey silt layer of 2 to 5m thickness. Because of the silt layer and the presence of thin silt and clay seams in the sand, the soil was judged marginal for vibrocompaction. Therefore, extensive tests were performed on site to develop the optimal compaction procedure (vibration time, frequency of compaction and grid spacing). The compaction process was monitored by vibration measurements on the ground surface. In this way, it was possible to determine the time required for densification and to establish the optimal vibration frequency.

In order to monitor the Tri Star probe the contractor carried out testing with the help of the site investigation firm, ConeTec, and in collaboration with the University of British Columbia (U.B.C.). U.B.C. monitored the long term effects, primarily with the Piezometer Cone Test (CPTU) and investigated the use of the Flat Plate Dilatometer (DMT) and Seismic Piezometer Cone Test (SCPTU). In addition, the Lateral Stress Piezometer Cone Test (LSCPTU) was used. All testing yielded before and after treatment results, with the CPTU giving progressive time and distance effects.

The contractor and consultant also carried out Standard Penetration Tests (SPT), CPTU testing and measured ground settlement before, during and after treatment.

The testing has proven that the Tri Star system is an efficient method to compact soil deposits which are susceptible to liquefaction. In particular an increase in penetration resistance of between 200 and 400% was recorded and the probe’s zone of influence was found to be of 2m radius. The DMT and CPTU provided similar results and on comparison to the LSCPTU suggested that increases in relative density and lateral stress contributed approximately equally to the soil improvement. The SCPTU provided relatively inconclusive results.

Advisor:

Dr. Richard G. Campanella
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1.0 INTRODUCTION

The Tri Star compaction method is a further improvement of the original Franki Y-probe, which was developed during the mid seventies for soil densification projects in Belgium (Holeyman, 1988) by S.A. Franki N.V. of Belgium.

A heavy vibrator is mounted on top of a probe which is vibrated into the soil. The vibrations are in the vertical direction in contrast to the vibroflot (Sparks, 1975) which vibrates horizontally. Extensive tests with different shapes of compaction probes were performed and showed that a three-bladed probe, provided with horizontal ribs, achieved optimal transfer of vibration energy to the soil. The geometry of the probe with a 120 degree angle between the steel blades avoids decompression of the soil which can occur with other probes during retraction, Wallys (1982).

The next major improvement of the system concerned taking into account the dynamic response of the soil during compaction. Theoretical analyses and model tests showed the importance of choosing optimal vibration frequency, vibration time and grid spacing, which are all site specific parameters. This can be achieved by introducing an electronic ground vibration monitoring system in combination with the use of a vibrator with variable excitation frequency.

The Tri Star system has the advantage that the vibration monitoring system provides information on ground acceleration directly on site. This offers the design
engineer an additional possibility of controlling and verifying the densification process and its effectiveness. Thus, the most efficient compaction procedure can be determined at the start of each project.

The Tri Star concept has been used successfully on several projects in Europe where it has demonstrated its technical efficiency and economy, Massarsch and Vanneste (1988). Hence, Franki decided to introduce the Tri Star system on to the North American market. Although further improvements of the Tri Star method are presently under way to widen its range of applications, it was decided to test the basic concept on a project site which by others was judged to be marginally compactable by vibratory methods. Discussions were therefore held with the geotechnical consultant to test initially the Tri Star system, having as an alternative compacted stone columns which are more costly, but are a well proven compaction solution. Thus in cooperation with the In-Situ Testing Geotechnical Research Group in Civil Engineering at the University of British Columbia, an extensive field monitoring and testing programme was worked out.

The objective of the research was to evaluate the performance of the Tri Star probe. This was achieved by using various in-house in-situ testing techniques.

The scope of the research involved investigating five aspects of how cohesionless soils are affected by the Tri Star probe. These are: (1) the time effect on strength
gain; (2) the zone of influence; (3) how the lateral stress is affected; (4) how the pore water pressure, measured by the CPTU is affected; (5) how various geotechnical parameters are affected.
2.0 REVIEW: DEEP COMPACTION OF SAND USING VIBRATORY PROBES

2.1 Introduction

Deep compaction of loose cohesionless soils is either required to eliminate excessive settlements or to minimise the risk of liquefaction caused by dynamic loading. Vibrocompaction of cohesionless soils is an efficient method to increase soil density and stiffness. A variety of dynamic methods for deep compaction are available, such as blasting, vibrocompaction and heavy tamping. The present review describes methods in which a probe is inserted in the ground without the addition of backfill material. This used to be termed vibroflotation and the probe called a vibroflot (Steuerman, 1939). However, the term vibroflotation now applies equally to stone column formation in cohesive soils and to the vibrocompaction of cohesionless soils, whether or not backfill is added to the hole. Usually the source of vibration is at the top of the probe above the ground surface, generating vertically or horizontally oscillating motion (Massarsch, 1986). Soil compaction is carried out in either a triangular or rectangular pattern and is achieved when the probe is withdrawn gradually or in steps. The densification effect depends on several factors: (1) soil type, especially soil gradation and content of fines; (2) degree of saturation and water table location; (3) initial relative density; (4) initial in-situ stresses; (5) soil structure including the
effect of aging, sedimentation etc.; (6) machine characteristics.

2.2 Densification Factors

2.2.1 Soil Type

Vibrocompaction methods are best suited for densification of clean, cohesionless soils. Experience has shown that these methods are generally ineffective when the percentage weight of fines (particles finer than 0.74mm diameter) exceeds 20%. In this case the permeability of the soil is too low to allow the rapid drainage of excess pore water pressure generated by the action of the vibratory probes. Figure 1 shows the range of particle size distribution which is suitable for densification by vibrocompaction (Mitchell and Katti, 1981).

2.2.2 Degree of Saturation and Water Table Location

Vibrocompaction methods are most efficient in saturated, cohesionless soils. In partially saturated soils, false cohesion created by capillary forces increases the effective stress (soil strength) and thus the energy required for soil compaction. The compaction effect in partially saturated soils can be improved by combining vibrocompaction with water jetting.
Figure 1 - Range of Particle Size Distribution for Suitable Densification by Vibrocompaction, Mitchell and Katti (1981)
2.2.3 Initial Relative Density

Vibrocompaction increases the density of loose cohesionless soils. The compaction effect is most pronounced in soils with a low initial density.

2.2.4 Initial In-situ Stresses

The state of in-situ stresses before compaction can have a significant influence on compaction efficiency. In saturated cohesionless soils, compaction is caused by a temporary state of liquefaction resulting from dynamic and cyclic forces. During the dissipation of the excess pore water, soil particles are re-arranged into a more dense state. The compaction effect is greater if the vertical overburden is high during re-consolidation (Massarsch, 1986). This means that vibrocompaction is more effective at greater depths. As yet the effect of vibrocompaction on the horizontal stress is not clearly understood. However, it is known that it is increased from its original $K_0$ condition.

2.2.5 Initial Soil Structure

Vibrocompaction can also have detrimental effects, e.g. on cemented soils where the soil structure can be broken down as a result of dynamic and cyclic loading.
2.2.6 Machine Characteristics

According to Sparks (1975), machine characteristics can be expected to play a role in the performance of a densification system. The important parameters are size, frequency, amplitude and eccentric force. It has been found that each of these is unique to a certain soil and therefore a field trial is required to find the optimum values to achieve maximum vibrocompaction (Massarsch, 1986). In an investigation of three machines it was found that the higher powered machines were capable of compacting soil more uniformly in the vicinity of the probe (Sparks, 1975). Whereas the lower powered led to a maximum density occurring 1m to 1.5m away from the probe. On analyzing the results of these tests it is not possible to make firm conclusions since the machines each had a different operating frequency, power and size. Morgan and Thomson (1983) also investigated three machines but failed to isolate the influence of machine characteristics.

2.3 Monitoring of Vibrocompaction

2.3.1 Introduction

Densification resulting from vibrocompaction occurs rapidly and settlement of the ground surface is essentially completed by the end of treatment. However, improvement in properties such as soil stiffness and strength may continue to increase over extended time periods. This effect is particularly pronounced in soils with a high content of
fines. Different field methods can be used to monitor the effect of compaction.

### 2.3.2 Methods of Monitoring

#### 2.3.2.1 Settlement

When no soil is added during the compaction process, settlement of the ground surface can be directly related to the increase in density of the soil layers. Surface settlements can be monitored using surface markers or settlement gauges.

#### 2.3.2.2 Penetration Tests

The most widely used methods to monitor compaction effects are penetration tests, such as the Standard Penetration Test (SPT), the static cone penetration test (CPT) or the dynamic cone penetration test. A new penetration test, the Flat Dilatometer Test (DMT) introduced by Marchetti (1980) is also used.

A direct conversion of penetration resistance to relative density may, however, be uncertain because penetration resistance can depend on many factors in addition to density. Furthermore, penetration resistance is usually measured at individual locations between compaction points, which may not represent the average density of the soil deposit. Pressuremeter Tests and Screw Plate Tests are used increasingly and give more specific
information on soil stiffness as well as on ultimate strength.

2.3.2.3 Seismic Tests

Seismic field tests, such as cross-hole and down-hole tests, have become the standard techniques for the in-situ determination of shear wave velocity. In a seismic test, the propagation of shear waves is measured with great accuracy in the horizontal or vertical direction over a distance of several metres. This test involves a larger soil volume than do penetration tests and maybe more representative of the average compaction effect. The result of seismic tests can be directly related to the change of shear modulus. Shear wave velocities are also essential for seismic design purposes.

A new test, called the seismic cone penetration test (SCPT), has been developed (Campanella et al, 1986). This test consists of a small rugged velocity seismomometer incorporated into an electronic cone penetrometer. The combination of the seismic down-hole method and the CPT logging provide an extremely rapid, reliable and economic means of determining stratigraphic, strength and modulus information in one sounding.

Finally, density meters, such as nuclear probes can also be used to check the variation of soil density in a borehole.
2.3.2.4 Real Time Monitoring

The state of densification is frequently estimated during compaction by monitoring the power input to the drive motor. According to D'Appolonia (1953) the power input reaches a maximum when the maximum soil density has been reached. However, the technique requires an experienced operator and does not account for compaction discrepancies. The above has been refuted by model studies by Metzger and Koerner (1975). They found that maximum power consumption occurred during compaction, not as generally believed from field experience, where it is found that maximum power consumption accompanies maximum compaction. Hence, D'Appolonia's method appears to be in doubt.

Another possible method of control has been reported by Morgan and Thomson (1981), in which the probe vibration is measured with transducers. Measurements are made with accelerometers mounted in the tip of the probe. It was found that the horizontal amplitude of the probe decreased with increasing sand density, as found from subsequent penetration tests. This method is potentially very reliable as it is a direct measurement of ground response.

An alternative method is to measure the vibrations caused by the probe using one and three dimensional geophones (Massarsch and Vanneste, 1988). The A.C. voltage generated is transformed into a root mean square (R.M.S.) velocity. This can be related to the densification
obtained, i.e. the higher the R.M.S. velocity, the higher the compaction effect. Hence, information on the vibration frequency, grid spacing and vibration time can be determined on site and used to optimise the compaction procedure.

2.4 History

The improvement of soils at depth using a vibrating probe was first performed in Germany over fifty years ago, and reported in the English language by Steuerman (1939). He described what was to become the most widely used deep improvement process namely, vibroflotation. The technique was initially used in only cohesionless soils, and could be performed without imported backfill. As such, it was a true vibrocompaction technique. Subsequently coarse sand or gravel backfill was added to the probe hole. Hence, the technique was adapted for use in cohesive soils, thus the vibroreplacement process was established.

2.5 Vibrocompaction Methods Using Topdrive Vibrators

2.5.1 Introduction

These methods are characterised by the insertion of a cylindrical or cross-shaped probe into the ground. At the top of the probe, a vibrator generates either vertical or torsional, oscillating motion. The probe is either pushed, vibrated or jetted into the ground and the compaction is achieved during the step-wise withdrawing of the probe to
the surface. Ground treatment depths of 15m can be routinely achieved

2.5.2 The Foster Probe

The Foster probe was developed in the U.S.A., and uses a vibro-piledriver on top of a 760mm diameter open tube. The probe is 3 to 5 m longer than the desired penetration depth and the unit operates at a frequency of 15 Hz and the vertical amplitude of motion is 10 to 25mm. About 15 probes per hour can be completed at spacings of 1 to 3m.

2.5.3 The Vibro Wing Probe

The Vibro Wing probe was developed for deep compaction of natural and dredged cohesionless soils, Massarsch and Lindberg (1986).

For this application a heavy vibrator (mass 7 metric tonnes) is attached to the top of a 15m long steel rod, which is provided with 0.8m long wings, spaced 0.5m apart. The vibratory hammer is operated from a piling rig, which is normally used for the installation of pre-fabricated concrete piles. The pull-out resistance of the probe can be monitored during the compaction process by a load cell, placed at the top of the rig. The frequency of the vibrator can be varied to fit the conditions at a particular site.

The probe is driven down to the depth to be compacted. If necessary, the driving can be helped by jetting at the
bottom of the steel rod. The soil is then vibrated vertically until the required degree of compaction has been achieved. The frequency of vibration is typically 20Hz. The duration of vibration and rate of withdrawal of the probe depends mainly on the permeability of the soil, the depth of the deposit and the spacing between compaction points.

2.5.4 The Vibro Rod Probe

The Vibro Rod probe is similar to the Foster probe but utilises a steel rod provided with short ribs, with a diameter of 0.5m and was developed in Japan, using a vibrating pile driving hammer, Saito (1977). A second probe described by Saito (1977), comprises a closed, cone tipped pipe, 300mm in diameter with, 100mm high tetrahedra spaced over the bottom 4m of the probe.

2.5.5 The Mytilus Probe

This is a large scale apparatus intended for offshore sand densification and was used for the Barrage Project in Holland, and is described by Davies et al. (1981). The vibrating probe consists of a tube with twelve radial fins, with a diameter of 2.1m. The probe is vibrated vertically from the top of the tube, at 25 Hz. Four such vibrators, spaced at 6.5m can be deployed simultaneously from a pontoon, allowing a zone up to 26m wide and 15m in depth to be densified.
2.5.6 The Tri Star Probe

The Franki company of Belgium, has developed a deep vibrocompaction method using a star shaped probe, which is inserted in the ground using a heavy vibrator. The star shaped probe was chosen to eliminate as much plugging as possible in the corners between the plates of the probe. The process is very simple and fast, (Wallys, 1982). It consists of three long steel plates, 20mm thick by 500mm wide, welded together at an angle of 120°. The probe can be up to 20m long. Small plates generally 10mm x 50mm x 300mm are welded to each side of the plates at two metre intervals which help transfer the vibration energy to the soil. The probe is attached to a heavy piling vibrator and vibrated vertically into the soil. It was first used in 1977 and has since been extensively used for under-water and onshore deep compaction of cohesionless soils.

2.5.7 The Phoenix Probe

The equipment comprises a probe which is capable of generating lateral vibration and simultaneous pumping of water. The probe is a torpedo shaped unit similar to a vibroflotation probe, but with the addition of a drainage section behind the tip. The probe is vibrated with air, at 100 psi, conveyed by custom built drill pipes to the tip which houses an air motor. This rotates an eccentric mass on a vertical axis, hence developing horizontal vibration.
Drainage is achieved when the exhaust air passes through a venturi hence causing a suction. This suction is applied across the drainage screen, thus sucking in water, which is then carried to the surface by the air.

In order to avoid clogging, the screen must be sized to suit the soil characteristics.

The probe is operated from a standard top-drive rotary drill rig and is lowered into the soil on a custom designed drill string.

2.6 **Expected Results**

Thorburn (1975) proposed guidelines to estimate the likely degree of improvement achievable. However, it is customary to perform a trial at the site of proposed soil improvement to establish the optimum configuration of improvement probes. Such design relationships will depend on soil and machine characteristics which will vary between sites.

2.7 **Conclusions**

There exists a significant body of information in the geotechnical literature describing experience with vibrocompaction. It concentrates primarily on the ability of a given technique to perform densification, i.e. soil type, probe spacing and pattern which relate primarily to performance. However, there is little information with regard to the soil mechanics aspect of the densification
process, such as the time dependency of the process, the effect on the in-situ lateral stress. Also, the pore water pressure behaviour during the densification process has yet to be adequately described.

Finally the effects of operating parameters which include, frequency, amplitude, power input and equipment dimensions are still unclear.
3.0 PROJECT DESCRIPTION

This thesis investigates the densification project executed in February 1988 on the Gray Beverage Inc. site at Annacis Island, B.C., Canada.

Development of the site involved the construction of a manufacturing storage building and a fleet maintenance building near the Annacis channel.

Because of the liquefaction potential of the sub-soil during strong earthquakes, it was judged that the proposed buildings could suffer major damage during a strong earthquake. Thus the geotechnical consultant for the project, Cook Pickering and Doyle Ltd., proposed soil compaction to increase the lateral stability of the area sloping towards the Fraser River.

As the construction site is underlain by thick deltaic and alluvial deposits, the geotechnical consultants specified the densification of a continuous strip area between the river channel and the proposed building site. This densified strip area would provide a stabilizing "dyke" and the granular fill and clayey silt layers would then be acting as a "raft", floating on top of the liquefied soil underneath. Major horizontal movements of the liquefied soil would be reduced by the "dyke", thereby preventing major damage to the buildings.

FRANKI CANADA Ltd. was awarded the densification project based on the Tri Star compaction method being able to achieve the required densification. In order to establish
the optimal compaction procedure (vibrator frequency, vibration time and grid spacing), a series of densification trials were performed before the start of the actual project. Results of these tests, which included measurements of ground vibrations, pore water pressure and settlements as well as cone and standard penetration tests before and after densification were carried out by Franki and ConeTec and evaluated by Franki. These trials were executed from February 3rd through 12th, 1988, after which the production phase of the project took place during the second half of February 1988. Further testing and evaluation was carried out by ConeTec and Franki during this stage to check the Tri Star performance. Thereafter, U.B.C. carried out its testing programme which lasted until September, 1988.
4.0 RESEARCH SITE

4.1 Introduction

Field tests were carried out to investigate the Franki Tri Star densification equipment. The site is located on Annacis Island, Vancouver, B.C. The site belongs to Gray Beverage Inc. who were building a new factory complex. The testing was carried out during the construction of this new factory and was curtailed when the test area was covered with blacktop.

4.2 Regional Geology

Annacis Island is part of the post glacial Fraser River Delta and is situated in the upper reach of the Island Arm of the Fraser River (Figure 2).

Blunden (1973) identifies the Fraser River Delta region sediments as marine deltaic deposits that have formed upon basal layers. These layers have undergone isostatic rebound for roughly 11,000 years at a rate greater than the rate of recent (i.e. post glacial) marine transgression. The total thickness of deltaic deposits is roughly 200 m. Further, the delta has been above sea level approximately 8,000 years when the sea level was about 10m below present levels (Blunden, 1973).

The surficial geology of the Annacis Island region is typical of a tide dominated delta. There is a prevalent thin deposit of clays and silts mixed with some organics
Figure 2 - Fraser River Delta Geology, Blunden, (1973)
that have been laid down in a quiescent swamp or marsh environment.

Below this top crust is a sequence of bedded sandy sites with various seams of sand, silt, clay and minor organics. Because of the non-uniformity of the zone below the top crust, the depositional history most likely represents a turbulent environment being associated with the tidal forces. The base of this succession is composed of clays and silts which were laid down in a much more quiescent marine environment.

4.3 Site Description

The site of investigation for this study is at the north side of Annacis Island, which is situated in the upper reach of the Fraser River Island Arm, shown in Figure 2. A site plan (Figure 3) shows the location of the U.B.C. test areas, tests carried out by ConeTec and other relevant details. The U.B.C. testing locations are shown in Figure 4.

The ground water table at the site varies with tidal fluctuation and is typically found at 2 to 4m below ground level. The site is essentially flat and the elevation is approximately 4.7m above mean sea level.
Figure 3 - General Site Plan, Gray Beverage Site
Figure 4 - Site Plan of Test Locations in U.B.C. Testing Area
A summary of the soil profile at the site is as follows:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>General Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 2.5</td>
<td>brown SAND FILL, dense</td>
</tr>
<tr>
<td>2.5 - 5.0</td>
<td>firm brown clayey SILT, organics</td>
</tr>
<tr>
<td>5.0 - 11.5</td>
<td>grey medium SAND, some SILT layers, medium dense</td>
</tr>
<tr>
<td>11.5 -</td>
<td>soft grey clayey SILT</td>
</tr>
</tbody>
</table>

This profile was determined from several sources, namely:

1) Cook Pickering & Doyle Ltd. site investigation using boreholes.
2) ConeTec site investigation using CPTU's
3) U.B.C. site investigation using CPTU's, DMT, and LSCTPU's

The medium dense sand from 5.0m to 11.5m was the reason for densification as it was felt that it would liquefy during a strong earthquake.

With regard to the thin silt layers in the medium dense sand from 5.0m to 11.5m these were judged to be of limited lateral extent based on the CPTU results.

In all, three testing areas were used, consisting of one treated area and two virgin areas (Figure 3). The treated area is located at the centreline of the densification at the 500 foot chainage line and is named $C_L 500 + 00$. The first virgin area is 90 feet south of $C_L 500 + 00$ and is named Virgin Area #1. The second virgin area is located
200 feet east of C_L 500 + 00. There were two virgin areas as it was thought that Virgin Area #1 may have been affected by the pre-load for the factory foundations, hence Virgin Area #2 was investigated. This showed no evidence of the suspected problem in the first area, thus Virgin Area #1 was used as the benchmark since it more closely resembled the soil profile at test area C_L 500 + 00. Virgin Area #1 was used for all the tests except the CPTU as a CPTU test had been carried out before densification on the centreline.
5.0 **SOIL INVESTIGATIONS**

5.1 **Franki Testing**

Initially a limited number of standard penetration tests (SPT) were performed using the "Donut" hammer, in different locations of the construction site.

However, only 3 SPT's were located nearby the strip area to be densified. Moreover, inspection of samples from some of the SPT's indicated the presence of thin silt layers in the sand to be densified, which are typical of the deltaic deposits of the Fraser River.

Therefore, it was judged necessary to conduct a second soil exploration phase, concentrating on the strip area to be compacted, before densification work could start. Piezocone tests were also performed as they provide detailed, continuous diagrams showing stratification and soil strength.

The CPTU's showed the existence of a number of thin, local silt layers. Negative excess pore water pressure, measured behind the tip, was observed during penetration of these silt layers, suggesting that they were dense and thus less prone to liquefaction.

As some of the CPTU's were located next to SPTs, it was possible to check the correlation between cone resistance ($q_c$) and blowcounts ($N$), Figure 5. This was done using a relationship proposed by Robertson and Campanella (1983) which takes into account the mean grain size ($D_{50}$) and is shown in Figure 6.
Figure 5 - Corresponding CPTU and SPT results Before Densification, Massarsch and Vanneste (1988)
$q_c$ : cone resistance (bar); $N$ : blow count (blows/foot)

Figure 6 - Influence of Grain Size on $q_c/N$ ratio, Robertson and Campanella (1983)
Grain size distribution curves from samples of the sand layer to be densified indicated that \( D_{50} \) is about 0.3mm, suggesting a \( q_c/N \) factor of 5 (\( q_c \) in bar). This does not agree with the actual data as it is too high. A value of 2.5 is required. This is probably due to the SPT not being calibrated for energy.

5.2 **U.B.C. Testing**

5.2.1 **Soil Profile**

The most popular method of soil type identification from a CPTU test uses the cone bearing and friction ratio (ratio of sleeve friction stress divided by cone bearing stress). Robertson et al (1986) produced the soil interpretation chart shown in Figure 7. This utilizes these parameters and is based on a combination of U.B.C. experience and earlier work by Douglas and Olsen (1981). Thus, Figure 7 was used to assist in the interpretation of soil types from the CPTU profile of 104CPT3 (\( C_L \) 500 + 00 before treatment). The resulting soil profile is indicated in Figure 8.

Referring to Figure 8 the profile indicates four distinct soil types. Below a dense sand fill which ends at 2.4m and until 5.0m the cone bearing peaks and troughs, as does the friction ratio and pore pressure, all of which indicate a silt with sand lenses. From 5.0m to 11.5m
Figure 7 - Soil Behaviour Type Classification Chart, Robertson et al (1986)
Figure 8 - CPTU Profile 104CPT3 Showing Interpreted Soil Profile
pore pressures are hydrostatic, the friction ratio remains constant and the cone bearing gradually increases. This is interpreted as naturally deposited Fraser River sand. From the initial site investigation this sand was found to be medium dense. Thereafter, the cone bearing falls off significantly to a steady state, while the friction ratio increases then become steady and the pore pressures increase to a large positive value. This is interpreted as a clayey silt.
6.0 DENSIFICATION REQUIREMENTS

The location and the size of the area to be densified are shown in Figure 3. According to the consultants specifications, densification was to be attained in the medium dense sand layer from 5.0m to 10 metres and was to be checked by SPT using the "Donut" hammer. As densification was also checked by means of CPTU's, these minimum blowcount values (N) were therefore converted into cone resistance values (q_c).
7.0 TRI STAR VIBROCOMPACTION EQUIPMENT AND METHOD

Saturated, loose soils can be densified using vibrocompaction. However, experience has shown that this densification method is only efficient if the grain size distribution of the soil satisfies the criteria as given by Mitchell and Katti (1981), see Figure 1. Based on the CPTU results it was judged that the thin silt layers in the alluvial sand were only of limited lateral extent and would thus not significantly effect the compaction efficiency.

The Tri Star probe (Figure 9) is inserted vertically into the soil using a heavy vibrator, Massarsch (1986). It consists of three long steel plates, approximately 20mm thick and 500mm wide, welded to each other at an angle to 120°. Small ribs are welded along both sides of each plate at 1m intervals to increase the contact area with the soil. The probe can have lengths of up to 20m, but for this project a 12m long probe was used.

The characteristics of the ICE (International Construction Equipment) 812 hydraulic vibrator (ICE 812) with variable frequency are given in Table 1. The vibrator frequency is proportional to the engine speed as indicated in Table 2.

The probe is moved from one location to another by a crane or a piling rig. A 60 ton crane was used to handle the probe for this project.
Figure 9 - Tri Star Probe Principal, Massarsch and Vanneste (1988)
Total mass of vibrator, including clamp  |  6670 kg
Dynamic mass, including clamp          |  4500 kg
Centrifugal force                      |  <1.13 MN
Frequency                              |  <26.6 Hz
Excentric moment                       |  46.1 kg.m
Maximum traction force                 |  0.36 MN
Maximum amplitude (without probe)      |  20 mm

Note: maximum amplitude = \( \frac{\text{excentric moment}}{\text{dynamic mass}} \times 2 \)

Table 1 Characteristics of the ICE 812 Vibrator

<table>
<thead>
<tr>
<th>Engine speed (RPM)</th>
<th>Vibrator speed (RPM)</th>
<th>Vibrator frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2300</td>
<td>1600</td>
<td>26.6</td>
</tr>
<tr>
<td>2010</td>
<td>1400</td>
<td>23.2</td>
</tr>
<tr>
<td>1730</td>
<td>1200</td>
<td>20.0</td>
</tr>
<tr>
<td>1440</td>
<td>1000</td>
<td>16.6</td>
</tr>
<tr>
<td>1150</td>
<td>800</td>
<td>13.3</td>
</tr>
<tr>
<td>850</td>
<td>600</td>
<td>10.0</td>
</tr>
<tr>
<td>575</td>
<td>400</td>
<td>6.6</td>
</tr>
</tbody>
</table>

Table 2 Conversion From Engine Speed to Vibrator Frequency, Vibrator Type ICE 812
8.0  DENSI FICATION TRIALS

8.1  Aim of the Densification Trials

The optimum densification procedure, such as optimal vibrator frequency, vibration time and grid spacing were determined in a trial area. The influence of each compaction parameter was examined using several types of measurements. These were ground vibrations, surface settlements, pore water pressure, cone and standard penetration resistance.

The contract required a test area to be densified by the compaction procedure to be used for the production phase (acceptance test). The layer that required treatment was the medium dense sand layer between 5.0m and 11.5m. Test results had to be approved by the geotechnical consultants before actual densification work could start. Therefore, it was desirable to carry out the preliminary trials well before the acceptance tests. Both the results of the preliminary trials and those of the acceptance test are described in this chapter.

8.2  Measuring Equipment

The ground vibrations generated by the Tri Star probe were measured using one- and three-directional geophones, placed on the ground surface at varying distances from the probe. With the three-directional geophone, horizontal as well as vertical vibrations could be measured simultaneously. The AC-voltage generated due to the
vibrations was transformed into a root mean square (RMS) velocity. The RMS - vibration level is measured as it can be related to the obtained densification i.e. the higher the RMS - vibration level, the higher the compaction. Hence the optimum vibration frequency, grid spacing and vibration time can be determined.

Surface settlements were measured using conventional levelling technique, carried out before and after completion of densification.

Excess pore water pressures (p.w.p) were measured by an open standpipe, installed near CPTU's 6 and 7. A plastic pipe was placed in a pre-bored, 5.8m (19 feet) deep hole. The porous tip consisted of a filter with a 2.4m (8 feet) slotted screen at the bottom end. Pore water pressure measurements were made during some trials before, during and after densification. This data is only qualitative due to the lag effect occurring between the probe hole and the standpipe.

From a practical point of view, the most important verification method was by means of penetration tests. Although CPTU's yield more detailed geotechnical information, the geotechnical consultant specified SPT's as he had extensive local experience with this method. Consequently, both types of penetration tests were performed. Extensive CPTU's made it possible to document in detail the achieved densification. Thereafter, a
limited number of SPT's were performed in order to formally check the obtained densification.

8.3 Trial Sites

Two trial sites were chosen to determine the optimal compaction procedure. The values for the various compaction parameters are shown in Table 3.

<table>
<thead>
<tr>
<th>Trial Area</th>
<th>Grid Spacing (m)</th>
<th>Steady State Frequency (Hz)</th>
<th>Vibration Time (min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>2.3</td>
<td>20</td>
<td>12</td>
</tr>
<tr>
<td>II</td>
<td>1.75</td>
<td>13</td>
<td>14</td>
</tr>
</tbody>
</table>

Table 3 - Tri Star Variables For Trial Area I and II

8.3.1 Vibration Time and Frequency

The vibration time and frequency were varied and analyzed until the maximum ground vibration was achieved.

8.3.2 Pore Water Pressure

The p.w.p. measurements for each site showed the same behaviour. At first, a small positive excess p.w.p. was noted, followed by a large negative excess p.w.p. which quickly dissipated. The positive p.w.p. was due to the soil liquefying then as it quickly drained the large negative p.w.p. occurred.
8.3.3 **Ground Surface Settlement**

It was found that the major settlement occurred immediately after densification after which only a slight increase with time was noted. Further, the maximum settlement coincided with an optimum time of vibration. Table 4 gives the settlement values. The grid configuration was triangular and the centre of grid refers to the centre of the triangle.

8.3.4 **CPTU Results**

Penetration tests executed shortly after completion of the densification at Trial Area I, indicate a small decrease of the cone resistance (Figure 10). However, a large increase of the cone resistance was observed with elapsed time at Trial Area II, as shown in Figure 11. These latter cone resistance profiles satisfied the minimum compaction criteria. Figure 12 shows that the cone bearing profile is the same for the grid point and the Tri Star probe point. Hence, the maximum compaction is being achieved across the whole width of treatment.

8.3.5 **SPT Results**

The SPT’s were only carried out in Trial Area II so no comparison can be made between the two areas. However, as can be seen from Table 5, the required compaction values were largely exceeded for Trial Area II.
<table>
<thead>
<tr>
<th>Type of point</th>
<th>Average settlement immediately after densification (m)</th>
<th>Average settlements 60 hours after densification (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Densification point</td>
<td>0.31</td>
<td>0.32</td>
</tr>
<tr>
<td>Center point of grid</td>
<td>0.22</td>
<td>0.25</td>
</tr>
<tr>
<td>Point 1.75 m away from densification point</td>
<td>0.10</td>
<td>0.12</td>
</tr>
<tr>
<td>Point 3.50 m away from densification point</td>
<td>0.05</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 4  Average Ground Surface Settlements
Figure 10 - Cone Resistance Profiles Before and After Densification: CPTU's 7 and 10; Franki Trial Area I
Figure 11 - Cone Resistance Profiles Before and After Densification: CPTU's 8, 12, and 13; Franki Trial Area II
Figure 12 - Cone Resistance Profiles After Densification at Grid Centre Point (CPTU 18) and at Densification Point (CPTU 20): Franki Trial Area
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Required minimum blowcount</th>
<th>Obtained blowcount</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DH 101</td>
</tr>
<tr>
<td>1.5</td>
<td>-</td>
<td>31</td>
</tr>
<tr>
<td>3.1</td>
<td>-</td>
<td>7</td>
</tr>
<tr>
<td>4.6</td>
<td>14</td>
<td>18</td>
</tr>
<tr>
<td>6.1</td>
<td>16</td>
<td>23</td>
</tr>
<tr>
<td>7.6</td>
<td>16</td>
<td>42</td>
</tr>
<tr>
<td>9.1</td>
<td>17</td>
<td>53</td>
</tr>
<tr>
<td>10.7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5: Standard Penetration Trial Results for Trial Area II
8.4 Production Phase

8.4.1 Execution Parameters

After an examination of all test results from the final trial area, the geotechnical consultant accepted the Tri Star compaction method. The same densification parameters as in Trial Area II were chosen to densify the complete strip area. However, it was decided to increase the compaction depth to 11m for the western half of the strip area, and to 10m depth for the eastern half. The selected densification parameters are shown in Figures 13 and 14 and are summarized in Table 6.

The complete strip area was divided into 100 feet long sections. Each section was densified in 3 passes as illustrated in Figure 13. In this manner, the elapsed time between the densification of 2 adjacent points could be increased.

8.4.2 Densification Results

8.4.2.1 Settlement Measurements

Ground surface settlements were measured immediately after densification at all densification points and at all grid "centre" points located at the centreline for the zone with 2 rows of densification points. In the zone with 3 rows of densification points, measurements were performed between two densification points, located at the centre row of densification points. The above locations can be
Figure 13 - Pattern of Densification Points During the Production Stage, Grid Spacing of 6 feet: a) 15 feet Wide Zone  b) 10 feet Wide Zone, Massarsch and Vanneste (1988)
Figure 14 - Desired Working Scheme During the Production Stage: Probe Depth and Vibrator Frequency as a Function of Time; a) West Half of Strip Area b) East Half of Strip Area, Massarsch and Vanneste (1988)
<table>
<thead>
<tr>
<th>Part of strip area</th>
<th>Western half</th>
<th>Eastern half</th>
</tr>
</thead>
<tbody>
<tr>
<td>grid: pattern spacing (m)</td>
<td>triangular 2</td>
<td>triangular 2</td>
</tr>
<tr>
<td>penetration:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>depth (m)</td>
<td>11</td>
<td>10</td>
</tr>
<tr>
<td>time (min.s)</td>
<td>2.30</td>
<td>2.30</td>
</tr>
<tr>
<td>frequency (Hz)</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>steady state:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>time (min.s)</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>frequency (Hz)</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>extraction:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>step surging levels (m)</td>
<td>9 and 6</td>
<td>8 and 6</td>
</tr>
<tr>
<td>Total vibration time (min)</td>
<td>15</td>
<td>13</td>
</tr>
</tbody>
</table>

Table 6 Compaction Procedure During the Production Phase
located using Figure 13. The results of these measurements are summarized in Table 7.

A significant increase was observed of the settlements in the compacted strip area, compared to Trial Area II, see Table 4. Although no important difference between the soil conditions in the eastern and western halves could be distinguished, it seems that the least densified zone was located between stations - 50 and 0, i.e. the western end.

8.4.2.2 SPT and CPTU Results

The achieved densification was checked in some locations by means of standard penetration tests (SPT) using the "Donut" hammer. Additional penetration tests (CPT) were also performed, using the piezocone (CPTU), and all were located at probe holes except for lateral testing. The results of the SPT's are given in Table 8.

These test results agree well with those obtained in Trial Area II and indicate that the required densification in the sand zone from 5.0m - 10.0m was exceeded. Low blowcounts or cone resistance values were measured in the silt layers for which the specified densification was not applicable.

Some other interesting comparisons are shown in Figures 15, 16, and 17. Cone resistance profiles of nearby CPTU's before and after densification are compared in Figure 15. CPTU 3 was carried out before densification and was located only 1.2m away from CPTU 24, which itself was executed 7
<table>
<thead>
<tr>
<th>Zone between stations</th>
<th>Average settlement at compaction point (m)</th>
<th>Average settlement at &quot;center&quot; point (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-50 to 0</td>
<td>0.20</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>coinciding with final densification test area</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>adjacent to final densification test area</td>
<td></td>
</tr>
<tr>
<td>0 to 100</td>
<td>0.19</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>coinciding with final densification test area</td>
<td></td>
</tr>
<tr>
<td></td>
<td>adjacent to final densification test area</td>
<td>0.45</td>
</tr>
<tr>
<td>100 to 200</td>
<td>0.40</td>
<td>0.37</td>
</tr>
<tr>
<td>200 to 300</td>
<td>0.44</td>
<td>0.42</td>
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<tr>
<td>300 to 400</td>
<td>0.44</td>
<td>0.39</td>
</tr>
<tr>
<td>400 to 500</td>
<td>0.49</td>
<td>0.45</td>
</tr>
<tr>
<td>500 to 600</td>
<td>0.48</td>
<td>0.40</td>
</tr>
<tr>
<td>600 to 700</td>
<td>0.43</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Table 7 Average Ground Surface Settlements for Production Stage Immediately After Densification
### Table 8: Standard Penetration Blowcounts for Production Stage 6 Days After Densification

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Required minimum blowcount</th>
<th>After Compaction</th>
<th>Before Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DH 104</td>
<td>DH 105</td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td>14</td>
<td>20</td>
</tr>
<tr>
<td>3.1</td>
<td></td>
<td>3*</td>
<td>5*</td>
</tr>
<tr>
<td>4.6</td>
<td>14</td>
<td>4*</td>
<td>20</td>
</tr>
<tr>
<td>6.1</td>
<td>16</td>
<td>41</td>
<td>49</td>
</tr>
<tr>
<td>7.6</td>
<td>16</td>
<td>55</td>
<td>64</td>
</tr>
<tr>
<td>9.1</td>
<td>17</td>
<td>67</td>
<td>47</td>
</tr>
<tr>
<td>10.7</td>
<td></td>
<td>22</td>
<td>23</td>
</tr>
</tbody>
</table>

* in silt layer
Figure 15 - Cone Resistance Profiles Before and After Densification: CPT's 3 and 24; Production Stage, Massarsch and Vanneste (1988)
Figure 16 - Corresponding CPT and SPT Results After Densification: Production Stage, Massarsch and Vanneste (1988)
Figure 17 - Cone Resistance Profiles 6 Days After Densification 0, 1, 2, and 3 m From the Densification Point: Production Stage, Massarsch and Vanneste (1988)
days after densification and was located near a densification point. The seven day interval was chosen by the contractor as it was felt that the most significant effect of densification would have occurred by this time. This comparison suggests that a significant densification was obtained within the sand layer from 5.0 to 10m depth. No change has occurred below the probe penetration depth of 10m. These results were also observed in Trial Area II.

The ratio of the cone resistance \((q_c)\) to the blowcount \((N)\) was checked at CPTU 26, located near SPT 107. The results of both tests are compared in Figure 15 and suggest that the \(q_c/N\) ratio was approximately 2.5 \((q_c\text{ in bar})\). This agrees with the findings at Trial Area II, for which the \(q_c/N\) ratio corresponded to 2.5.

Some CPTU's were carried out at increasing distance from the same densification point in order to investigate the influence of distance on the achieved densification. Results of these tests are given in Figure 17 and show that the obtained densification was by far the largest at the densification point itself, while a small increase occurs at 1m, but not 2 or 3m from the densification point. Note that CPTU's 21, 22, and 23 were all located outside of the area to be compacted.

For further information regarding the trials, a detailed description is given by Massarsch and Vanneste, 1988.
9.0 **U.B.C. TESTING PROGRAM**

9.1 **Introduction**

In-situ tests were performed to investigate the geotechnical conditions at the site and to monitor the performance of the Tri-Star probe. This section describes the procedures and techniques used in carrying out the various in-situ tests. The CPT and DMT tests were performed in accordance with the procedures laid down by the American Society for Testing and Materials, namely, ASTM D3441-86 for the CPT and ASTM D 18.02.10-86 for the DMT. Other in-situ tests like seismic cone and lateral stress cone have no designated standards, thus the procedure followed usual U.B.C. practice, which has evolved through wide geotechnical field testing experience.

9.2 **In-Situ Tests**

9.2.1 **Testing Vehicle**

All in-situ tests were performed from the U.B.C. Geotechnical Research Vehicle. This vehicle is designed to perform a variety of in-situ tests in a self-contained laboratory environment. These include the mechanical cone, flat dilatometer, screw plate, self boring and full displacement piezometers, together with the ability to retrieve piston samples.

The design is such that with the exception of levelling the truck and initiating a penetration, virtually all of the testing procedures can be carried out from inside.
Once it is on site and positioned for a sounding, it is elevated with two hydraulic pads to provide a level and stable working platform. A more detailed description of the vehicle is given by Campanella and Robertson (1981).

9.2.2 Tests Performed

According to Campanella and Robertson (1981), in-situ tests may be broadly divided into two categories:

1. logging methods
2. specific methods

Logging methods are generally economic and quick to perform, and are used primarily for stratigraphic profiling. They can also yield qualitative correlations to obtain geotechnical design parameters (Robertson and Campanella, 1986). Specific test methods are often slower and more costly than logging methods, and are used primarily for the measurement of soil properties at a point. The logging methods are therefore best suited to the preliminary evaluation of soil parameters.

The cone penetration test is the most rapid of the logging type in-situ tests and the addition of the pore pressure measuring element has improved the evaluation of soil parameters. Consequently, the electronic piezocone was selected as the primary tool for evaluation of the Tri Star probe.
The flat dilatometer was used selectively, to identify changes in horizontal stress and assess changes in soil modulus and also to confirm the CPT results.

Tests were also performed with the seismic piezocone to provide information on the dynamic shear modulus before and after treatment.

Finally a lateral stress electronic piezocone was used to evaluate the changes in the horizontal stress before and after treatment.

Each of the above instruments and their procedure is described below.

9.2.2.1 Piezocone Penetration Test (CPTU)

Piezocone Tests were performed using a cone with a 10cm² base area and an Apex angle of 60°, in accordance with ASTM Standard D3441, 1986. The friction sleeve, located above the conical tip, has a standard area of 150cm² and is the same diameter as the conical tip and push rods, i.e. 35.7mm.

Pore pressures are measured simultaneously behind the tip or on the face and behind the friction sleeve.

Measurements of pore water pressure, cone bearing, sleeve friction, inclination and temperature are typically recorded every 50mm or as specified, 25mm at U.B.C.

Details of the various piezocone designs are given by Campanella and Robertson (1988).
Depending on the cone design, cone measurements can be susceptible to temperature variation, therefore corrections may be necessary, Robertson and Campanella (1986).

Before going out into the field each cone is checked for operation, calibration and then carefully saturated. Porous polypropylene filter elements, 5mm wide, are saturated with glycerine under a vacuum. The instrument then has to be carefully assembled under glycerine to eliminate air from the pore pressure measurement system. Thereafter, it is kept under glycerine until required. Full details of the saturation procedure as carried out at U.B.C. is described by Robertson and Campanella (1986).

To confirm instrument calibration, baselines are recorded before and after every sounding. In the field, cone data is recorded automatically using the Hogentogler and Co. data acquisition system in the U.B.C. Geotechnical Research Vehicle. This system consists of a Radio Shack TRS 80 LED display portable computer, a standard buss interface, Z80 CPU, 12 bit A/D, a line printer, and a bubble memory storage device. Once back at the laboratory the data is downloaded from the bubble device to a personal computer for processing.

9.2.2.2 Flat Dilatometer Penetration Test (DMT)

The Flat Dilatometer test (DMT) was first introduced into North America in 1980 (Marchetti, 1980) and has become a simple in-situ test. The dilatometer is a flat blade
15mm thick, 95mm wide by 220mm in length. A flexible stainless steel membrane 60mm in diameter is located on one face of the blade. The membrane is usually inflated using high pressure nitrogen gas supplied by a tube pre-threaded through the push rods. Beneath the membrane is a sensing device which turns a buzzer off in the control box when the membrane starts to lift off and turns a buzzer on again after a deflection of 1.1mm at the centre of the membrane. As the membrane is inflated, the pressure required to just lift the membrane off the sensing device (reading A), and to cause 1.1mm deflection (reading B) are recorded. As the pressure is released and the membrane returns to its initial lift off position the reading C can be recorded. This provides a measure of the initial in-situ water pressure $U_0$ in sand soils when the material index ($I_D$) is equal or greater than approximately 2. In clays ($I_D < 0.8$) the C reading is close to the pore pressure generated during penetration. Readings are generally made every 200mm in depth. The thrust is measured at the surface with a load cell and is used to estimate the friction angle of the soil. These readings are made from a pressure gauge in the control box at the surface and entered on a standard data form.

Full details of the standard test procedures are given in suggested ASTM Method for Performing the Flat Dilatometer Test (ASTM Sub-Committee D18.02.10, 1986).
The A and B readings are corrected for membrane stiffness and noted as $P_0$ and $P_1$ respectively. The corrected membrane lift off and 1.1mm displacement pressures can be used to define three index parameters. These are described by Marchetti (1980) as the material index ($I_D$), the horizontal stress index ($K_D$) and the dilatometer modulus ($E_D$). From these indices, empirical relationships have been developed to determine geotechnical parameters. Details of the original correlations may be found in Marchetti (1980), with later developments by Schmertmann (1983).

9.2.2.3 Seismic Piezocone Penetration Test (SCPTU)

The incorporation of a small rugged velocity seismometer into a piezocone has made it possible to routinely measure the small strain shear wave velocity, during a piezocone sounding (Campanella and Robertson 1986). The cone penetrometer containing the seismometer is pushed to the first test depth. The seismometer is oriented in the horizontal direction and parallel to the signal source to detect the horizontal component of shear waves. Shear waves are generated at the ground surface by striking the truck pads horizontally with a sledge hammer, both sides of the truck being struck in turn. At any test depth, two waveforms are obtained, representing opposite polarised waves. A high quality oscilloscope is necessary to record and view the waveforms. The time required for the shear
wave to reach the seismometer in the cone is recorded. This procedure is repeated at 1m intervals. The travel time is vectorially corrected to the equivalent vertical travel path time and the interval time calculated.

The shear waves travel through the soil skeleton and are thus related to the soil shear modulus. Elastic theory relates the shear modulus, \( G \), soil density, \( \rho \) and the shear wave velocity, \( v_s \) as follows:

\[
G = \rho \times v_s^2
\]

Hence, the shear modulus can be determined using in-situ methods for the determination of the shear wave velocity. The shear modulus is largest at low strains and decreases with increasing shear strain (Seed and Idriss, 1970). The shear strain amplitude in in-situ seismic cone tests is low and of the order of \( 10^{-4} \% \). Thus, the very low strain level dynamic shear modulus, \( G_{\text{MAX}} \), is obtained.

9.2.2.4 Lateral Stress Piezocone Penetration Test (LSCPTU)

Lateral stresses are measured by means of a lateral stress module pushed into the ground behind a 15cm\(^2\) CPTU. The lateral stress is monitored 0.75m behind the tip using a second friction sleeve instrumented to measure hoop stresses in a thin-walled section of the sleeve. The lateral stress cone (LSCPTU) is capable of measuring the following parameters:
1. cone bearing
2. sleeve friction immediately behind the tip
3. pore pressure at 2 of 3 locations, i.e. on the face or behind the tip and behind the friction sleeve
4. temperature

In addition, at the location of the lateral stress sleeve the following are also measured:

1. pore pressure
2. sleeve friction
3. lateral stress

Details of the lateral stress cone are given in Campanella et. al. (1990).

The data is stored on the U.B.C. research data acquisition system and later transferred to floppy disk. In terms of lateral stress measurements the following corrections have to be applied:

1. correction for lateral stress/friction cross-talk
2. correction for temperature dependence of lateral stress baseline.

The stress measurements are performed in disturbed soil since the method of insertion is of the full displacement type. To determine the true in-situ stresses the effect of stress change due to displacement has to be evaluated. However, the difference in the measured stresses before and after ground treatment may be useful in evaluating the degree of improvement and obviate the need to evaluate actual stress conditions.
9.3 **Field Test Programme**

9.3.1 **Objective**

The objective of the field testing was primarily to investigate the time effect of strength gain in cohesionless soils after being vibrocompacted by the Tri-Star probe. Secondly, to investigate the Tri Star probe's zone of influence and also to investigate how the lateral stress, pore water pressure measured by the CPTU and various geotechnical parameters were affected.

9.3.2 **Scope of Work**

In order to attain the objectives three distinct areas were chosen for the investigation.

The location of these areas, and the virgin area penetration tests and other relevant features are shown in Figures 3 and 4. A more detailed plan of the Trial Area C_L 500 + 00 with the position of the probe holes and penetration tests is shown in Figure 18.

All areas were chosen for their ease of access and in particular, the Trial Area for comparison to the testing done by ConeTec.

The CPTU was used as the primary tool and hence the only instrument to investigate the effect of time and distance. The rest compared the effect of vibrocompaction before and after. The details of the penetration tests, i.e. type, time carried out and position, are contained in Table 9 and
Figure 18 - Detailed Plan of Trial Area C_L 500 + 00

- CPTU’s
- DMT’s
- LSCPTU’s
- Tri Star Probe Holes
--- Centreline Of Vibrocompaction
<table>
<thead>
<tr>
<th>Test Number</th>
<th>Type</th>
<th>Date (days)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT 3</td>
<td>CPTU</td>
<td>--</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; before</td>
</tr>
<tr>
<td>CPT 5</td>
<td>SCPTU</td>
<td>--</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; before</td>
</tr>
<tr>
<td>CPT 24</td>
<td>CPTU</td>
<td>+6</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-1</td>
<td>CPTU</td>
<td>+67</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-10</td>
<td>SCPTU</td>
<td>+82</td>
<td>VA1</td>
</tr>
<tr>
<td>C8-AN-11</td>
<td>SCPTU</td>
<td>+82</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-12</td>
<td>CPTU</td>
<td>+82</td>
<td>1m south of C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-13</td>
<td>CPTU</td>
<td>+82</td>
<td>2m south of C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-14</td>
<td>CPTU</td>
<td>+82</td>
<td>3m south of C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>C8-AN-18</td>
<td>CPTU</td>
<td>+203</td>
<td>VA2</td>
</tr>
<tr>
<td>C8-AN-20</td>
<td>CPT</td>
<td>+209</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>D8-AN-2</td>
<td>DMT</td>
<td>+111</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after, facing east</td>
</tr>
<tr>
<td>D8-AN-3</td>
<td>DMT</td>
<td>+111</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after, facing north</td>
</tr>
<tr>
<td>D8-AN-4</td>
<td>DMT</td>
<td>+111</td>
<td>VA1</td>
</tr>
<tr>
<td>LS-3-AN</td>
<td>LSCPTU</td>
<td>+204</td>
<td>On C&lt;sub&gt;L&lt;/sub&gt; after</td>
</tr>
<tr>
<td>LS-4-AN</td>
<td>LSCPTU</td>
<td>+209</td>
<td>VA1</td>
</tr>
</tbody>
</table>

Table 9  Details of Penetration Tests
should be read in conjunction with Figure 18. The choice of interval for testing was dictated by availability of equipment, personnel and access to the site.
10.0 **TEST RESULTS**

10.1 **Introduction**

The object of this chapter is to provide a background for Chapter 11, which discusses the interpretation of the test results. In an assessment of soil improvement it is necessary to accurately estimate the soil conditions, otherwise the degree of improvement cannot be confidently evaluated. As such, in-situ tests and the most recently developed interpretation methods have been used.

10.2 **CPTU Parameters**

The effect of vibrocompaction on the parameters of cone bearing, sleeve friction, and pore water pressure is described in the following section. The description looks at the effect of time and distance. This is done by first showing the "before" profile and then the "after" profile. Thereafter each consecutive profile in time is superimposed to show the effect of time. The same is also done with distance. With regard to any increase shown in the first 2 -3m it was not due to the Tri Star probe. This was because the water table was 2 - 3m below ground level above which the Tri Star probe is not effective. The increase was due to a surface vibrating roller which was operating across the whole site at various times during the testing period. This is confirmed by the fact that no increase is shown in the ConeTec CPTU profiles (Figure 15) carried out
7 days after compaction during which the largest increase in the medium dense sand layer occurred.

10.2.1 Cone Bearing

10.2.1.1 Time Effect

Cone bearing as a function of time is shown in Figure 19. A test was taken before and at 67, 82, and 209 days after vibrocompaction.

It can be seen from Figure 19 that the bottom of the sand fill layer has moved down 1.5m and remains at that new level for the period of testing. This is thought to be due to the mixing effect of the Tri Star probe. The process involves the sand fill dropping down into the silty sand and mixing, hence producing a medium suitable for vibrocompaction. This is interesting as it shows that a silty soil may be densified by a probe such as the Tri Star by supplying a surcharge of cohesionless material at the surface. Thus it can be seen that the silt layer has been effectively reduced from 2.5m to 1m and remains as such for the period of testing. The above was also observed at a vibrocompaction project in Saudi Arabia with a similar soil profile (Kirsch and Chambosse, 1981). This is the reason for depicting the soil profile at this location with dashed lines, so as to indicate the movement of soil boundaries.

In the medium dense sand layer (5m - 10m), the area specified for treatment, there was considerable change. The plots of Figure 19 show an increase of cone bearing with
Figure 19 - Effect of Cone Bearing With Time 67, 82 and 209 Days After
time for the 67 day and 82 day tests of 2 and 4 fold respectively. However, at 209 days the cone bearing drops back to a 3 fold increase. This is surprising as one would expect the cone bearing to increase more or to plateau. The cause for this may be due to a change in the ground water level due to a different tide level, i.e. the tide was higher during the 82 day test.

With regard to sand below the end of the probe, no effect is seen after 67 days but a jump of 50 bar occurs after 82 days and stays the same at 209 days.

Below 11.5m, the end of the sand layer no effect is seen in the clayey silt layer with time, which is as expected.

10.2.1.2 Distance Effect

The effect of cone bearing with distance is shown in Figure 20. A penetration test was taken at 1m, 2m, and 3m from the centreline of vibrocompaction, as shown in Figure 18.

The aspect of the sand fill mixing with the silt layer can be seen if one observes the series of figures. The sand fill profile gradually moves back from its peak value of 200 bar (top left graph) to almost its virgin position (bottom right graph). Also, the top of the silt layer moves back from 4m to 2.5m, its original position.

With regard to the medium dense sand it gradually moves back to its virgin state at 3m from the centreline.
Figure 20 - Effect of Cone Bearing With Distance at 0, 1, 2, 3m From The Centreline (82 days after)
Therefore, it would appear that the Tri-Star probe has a zone of influence of about 2m radius.

10.2.2 Sleeve Friction

10.2.2.1 Time Effect

The effect of sleeve friction with time is shown in Figure 21.

The sleeve friction exhibits the same behaviour in the sand fill and silt layers as did the cone bearing, i.e. the mixing effect.

The sleeve friction in the medium dense sand is different from that of the cone bearing. Instead of a gradual build up with time, there is an increase at 67 days which remains constant afterwards. However a time effect is noticeable from the CPTU's done during and after the treatment phase by the contractor (CPT 8, 12, 13, Figure 11). Hence, it could be assumed that the sleeve friction time effect is considerably less than that of cone bearing.

Again, no effect of vibrocompaction can be seen in the clayey silt layer.

10.2.2.2 Distance Effect

The effect of sleeve friction with distance is shown in Figure 22.

Again, the aspect of the sand fill mixing with the silt layer can be observed in the first 4m of the profile,
Figure 21 - Effect of Sleeve Friction With Time 67, 82, and 209 Days After
Figure 22 - Effect of Sleeve Friction With Distance at 0, 1, 2, and 3m From the Centreline (82 days After)
whereby the sand fill and silt profiles gradually return to their virgin state.

As for the medium dense sand layer it exhibits the same behaviour as found by the cone bearing, i.e. it gradually returns to its virgin state by 3m.

Thus, as for the core bearing the zone of influence with regard to sleeve friction is about 2m radius.

10.2.3 Pore Water Pressure Measured By CPTU

Unfortunately, the majority of the pore water pressure (p.w.p.) data collected was found to be unsatisfactory except for the virgin area and the 209 day test. This was due to equipment and operational problems.

With regard to the 209 day test for behind the tip, shown in Figure 23, this shows the p.w.p. to be hydrostatic in the treated sand layer from 5.0 - 10.0m. The same effect is exhibited by the behind the friction sleeve element and hence is not shown. This effect was also noted in all the ConeTec data which showed no excess p.w.p. Therefore, as expected p.w.p. is not effected in the long term after vibrocompaction in freely draining soil. This is because the initial p.w.p. increase is quickly dissipated.
Figure 23 - Effect of Behind Tip p.w.p. With Time 209 Days After
10.2.4 Friction Ratio

The friction ratio was chosen to attempt to confirm the mixing of the sand fill and silt indicated by the cone bearing and sleeve friction.

10.2.4.1 Time Effect

The effect of the friction ratio with time is shown in Figure 24.

This shows that the friction ratio effectively does not change from before and after and also with time. This is true for the sand fill and medium dense sand layers but not the silt layer. This can be clearly seen by the peak indicated at around 2.75m before treatment and disappearing after treatment. However, the silt layer from 4m to 5m is still evident but to a lesser extent. Again, this points to the mixing effect of the probe.

10.2.4.2 Distance Effect

The distance effect is not depicted in a series of figures since it is a repeat of the cone bearing and sleeve friction, and the evidence can be seen in the CPTU profiles contained in the Appendix.
Figure 24 - Effect of Friction Ratio With Time 67, 82, and 209 Days After
10.3 **DMT Parameters**

10.3.1 **Introduction**

The DMT was only carried out at one specific time after vibrocompaction, hence there are no relative time or distance effects. Thus, the DMT programme involved a test in Virgin Area #1 and two tests on the centreline between probe holes with one membrane facing East and the other facing North. This was done to investigate if the Tri Star probe densified the soil equally in the horizontal plane. The actual testing was carried out 111 days after vibrocompaction. The time was dictated by equipment availability and site access.

10.3.2 **Parameter Results**

The series of Figures 25, 26, 27, 28, and 29 show the effect of vibrocompaction on the following parameters:

1. \( P_0 \)
2. \( P_1 \)
3. Material Index, \( I_D \)
4. Horizontal Stress Index, \( K_D \)
5. Dilatometer Modulus, \( E_D \)

It can be seen that all of them exhibit the same behaviour in both the East and North directions. Thus, it may be assumed that the method of vibrocompaction by the Tri Star probe is isotropic, i.e. it does densify the soil equally in the horizontal plane.
Figure 25 - $P_0$ Versus Depth Before and After Densification
Figure 26 - $P_1$ Versus Depth Before and After Densification

NOTE: Test date 111 DAYS after.
Figure 27 - $I_D$ Versus Depth Before and After Densification

NOTE: Test date 111 DAYS after.
**Figure 28 - \( K_D \) Versus Depth Before and After Densification**

Note: Test date 111 days after.
DEPTH vs $E_D$

$E_D$ (bar)

Depth (m)

SAND FILL

SILT

MEDIUM DENSE SAND

CLAYEY SILT

---- $C_L$ 500+00 (DMT facing East)

• • • $C_L$ 500+00 (DMT facing North)

--- VIRGIN AREA #1

NOTE: Test date 111 DAYS after.

Figure 29 - $E_D$ Versus Depth Before and After Densification
By comparing the DMT profile except, $I_D$, to those of the CPTU it can be seen that the DMT confirms the soil horizons found by the CPTU. Depending on which parameter is chosen for the medium dense sand layer each has been increased from between 3 times (shown by $K_D$) and 4 times (shown by $P_0$, $P_1$, and $E_D$) which compares favourably with that found by the CPTU. The effect of vibrocompaction stops at 10m, the limit of the probe and the clayey silt is unaffected.

As mentioned, the above is true for all the parameters except, $I_D$. This is the Material Index, which is used for soil identification and hence should not change if the soil type is not changing. This is essentially true except for the sand fill and silt which underwent mixing. This is clearly shown by the before peak at 2.5m disappearing at the after stage while the peaks at 4m and 6m remain essentially the same. The peaks represent silty material and since the sand fill mixes with the silt from 2.5m to 4m the soil type is changed. Thus, the mixing idea has been confirmed.

The above also backs up the findings of the friction ratio derived from the CPTU results. Hence both correctly identify the soil type independently of possible changes to density, moduli and stress conditions.
10.4  **SCPTU Parameters**

10.4.1  **Introduction**

Like the DMT, the SCPTU was only carried out at one specific time after vibrocompaction, hence there are no relative time or distance effects. The SCPTU programme involved a test on the centreline of vibrocompaction at chainage 600 + 00 before vibrocompaction, one in Virgin Area #1 and one on the centreline at chainage 500 + 00, 82 days after vibrocompaction. The test at chainage 600 + 00 was carried out by ConeTec. The data for the first 4m is missing as it is erroneous due to the large soil variability over the small depth.

10.4.2  **Shear Modulus**

The profile of maximum shear modulus, \( G_{\text{MAX}} \), verses depth is shown in Figure 30. It was determined from the measured shear wave velocities during the seismic piezocone. The method has been described already in section 9.2.2.3.

The results show no difference in \( G_{\text{MAX}} \) for the first 3m of treated soil, i.e. 5-8m. After the 8m depth an increase of 10-15 MPa is noted until the 10m depth, which was the limit of the probe penetration.
Figure 30 - $G_{\text{MAX}}$ Versus Depth Before and After Densification

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**DEPTH vs $G_{\text{MAX}}$**

<table>
<thead>
<tr>
<th>$G_{\text{MAX}}$ (MPa)</th>
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</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>20</td>
</tr>
<tr>
<td>40</td>
</tr>
<tr>
<td>60</td>
</tr>
<tr>
<td>80</td>
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<tr>
<td>100</td>
</tr>
</tbody>
</table>

**SOIL PROFILE**

- SAND FILL
- SILT
- MEDIUM DENSE SAND
- CLAYEY SILT

- - - - - $C_L$ 500+00 (82 DAYS after)
- VIRGIN AREA #1
- - - - - $C_L$ 600+00 (before vcomp)
There is no gain shown for the 5.0 - 8.0m layer as operational problems were encountered, i.e., the 6.5m test is incorrect and the 7.5m test is missing. Therefore, because of the poor data no conclusion can be made about the effectiveness of the SCPTU to measure changes in $G_{\text{MAX}}$, brought about by the vibrocompaction process.

10.5 **LSCPTU Parameters**

10.5.1 **Introduction**

The LSCPTU was only carried out at one specific time after vibrocompaction, hence there are no relative time or distance effects. The LSCPTU programme involved a test on the centreline of vibrocompaction at chainage 500 + 00 after treatment and one in Virgin Area #1. The tests were carried out 204 days after densification, the time at which the LSCPTU became operational.

10.5.2 **Lateral Stress**

The penetration lateral stress was measured to find out how it was affected by vibrocompaction and to compare it to the vertical stress.

The profile of lateral stress versus depth is shown in Figure 31. This indicates all the relevant soil boundaries and the mixing of the sand fill with the silt. The profile is similar to those given by the other in-situ tests. The
Figure 31 - Lateral Stress Verses Depth Before and After Densification
medium dense sand layer shows an increase of between 50%-100%.

Again, the profile returns to its virgin state around the 10m depth and no change is seen in the clayey silt.

10.6 Conclusions

This chapter has shown that the medium dense sand layer was significantly compacted, i.e. an increase of 200-400%. The CPTU parameters of cone bearing and sleeve friction indicated this increase which the DMT parameters $P_0$, $P_1$, $K_D$, and $E_D$ confirmed. The CPTU has shown that the compaction effect is time dependent and it decreases with distance from the probe hole. Also, the pore water pressure measured by the CPTU does not change from before and after densification.

It has been shown that the CPTU parameter friction ratio and the material index from the DMT are good soil type indicators which are independent of changes in relative density, stiffness and stress conditions due to vibrocompaction.

The seismic piezocone results were inconclusive so no conclusions could be made as to its ability to measure the effect of vibrocompaction on the shear modulus.

The lateral stress cone measured an increase in the penetration lateral stress of 50 - 100%.

All of the penetration tests except the SCPTU indicated that part of the silt layer was improved by the sand fill above falling down and mixing with the silt.
11.0 **INTERPRETATION OF THE RESULTS AND GEOTECHNICAL PARAMETERS**

11.1 **Introduction**

The last chapter has shown that the Tri Star probe significantly increased the penetration resistance in the medium dense sand layer. The following chapter investigates the changes in soil parameters caused by vibrocompaction. To help in this process, the data has been smoothed by filtering in order to get general trends.

As the overall soil profile has only been investigated, each soil layer is scrutinized by using the CPTU parameter cone bearing. Thereafter, the various geotechnical parameters are compared.

11.2 **CPTU Results**

11.2.1 **Filtering**

Due to the presence of various lenses of different soil types within each of the soil layers, it was difficult to pick out general trends and to make comparisons. Therefore, it was decided to filter the CPTU cone bearing data in order to smooth out the profile. The program U.B.C.-FILT developed at U.B.C. by D.S. Wickremensinghe under the supervision of Dr. R.G. Campanella was used for this purpose (Wickremensinghe and Campanella, 1988).

The program filters out extremities or anomalies in the data. This is based on several options such as mean or median filtering, choice of intensity of filtering, and
selection of removal or replacement of data falling outside the acceptable region. However, it must be noted that good engineering judgement be exercised when filtering out data. This is firstly to avoid the possibility of losing genuine data and secondly, filtering is very situation dependent. A comparison of before and after filtering is shown in Figure 32. The filtering was carried out using the median with 10 data points in a group, a degree of filtering equal to 3 and using neighbouring unfiltered data points. This produces a smoother profile with less variability hence the trend is more easily recognisable. For further information a detailed description of the program U.B.C.-FILT and its use is given by Wickremensinghe and Campanella, 1988.

11.2.2 Time Dependent Behaviour

In order to analyze the time dependent soil behaviour for each layer a plot of cone bearing versus time has been produced for each of the 3 layers. These depict the progress of cone bearing with measurements taken at: before, 6, 67, 82, and 209 days after vibrocompaction.

11.2.2.1 Sand Fill Layer

The plot of cone bearing versus time is shown in Figure 33. As indicated on the figure, the section of layer investigated extends from 1.125m to 2.625m. Although the
Figure 32 - Filtered and Unfiltered Cone Bearing Versus Depth

- $C_L$ 500+00 before (unfiltered)
- $C_L$ 500+00 before (filtered)
TIME vs CONE BEARING
(SAND FILL LAYER)

NOTE: (1) The data is filtered using the median.

Figure 33 - Time Versus Cone Bearing for Sand Fill Layer
sand fill layer extends from the surface to around 2.5m, the depth of 1.125m was chosen as the start, as above this depth the sand fill had been vibrocompacted by a surface roller.

The plot indicates that the most rapid gain occurs during the first 6 days, until peaking somewhere between 6 and 82 days, thereafter reaching a plateau.

11.2.2.2 Silt Layer

The plot of cone bearing versus time is shown in Figure 34. As indicated on the figure, the section of layer investigated extends from 2.875m to 3.875m. This is actually a composite picture as it shows the mixing of the sand fill and silt.

As with the pure sand fill layer the top section exhibits the same cone bearing increase pattern. Then on moving deeper, this characteristic is gradually lessened until the silt layer is reached at 3.875m.

Hence, it can be concluded that, knowing beforehand that the silt layer extended from 2.625m to 5m it has now been significantly improved from 2.625 to 3.875m. Also, the effect of mixing gradually reduces from 2.625m to 3.875m.
TIME vs CONE BEARING
(SILT LAYER)

NOTE: (1) The data is filtered using the median.

Figure 34 - Time Versus Cone Bearing for Silt Layer
11.2.2.3 Medium Dense Sand Layer

The plot of cone bearing versus time is shown in Figure 35. As indicated on the figure, the section of layer investigated extends from 7.125m to 9.375m. Only this portion was chosen as it contained the least variable data.

Again the plot shows the rapid increase in cone bearing during the first 6 days after which the gain is reduced until 67 days. At this point the gain increases very rapidly to a peak at 82 days, after which the gain decreases.

This step wise profile does not seem quite correct as one would expect a smoother gain after the rapid one up to 6 days. As discussed in the chapter of the CPTU results with time the cause may be due to the ground water level being higher at the 82 day test than the rest due to a different tide level.

11.2.2.4 Conclusions

It may be concluded that for this particular site and the soils investigated, that the increase in cone bearing follows a curved shape with a peak occurring between 6 and 82 days and is the case for each layer. The decrease in cone bearing after 82 days is felt to be artificial due to tidal change. Hence it is suggested that the cone bearing gain would have instead plateaued.
TIME vs CONE BEARING
(MEDIUM DENSE SAND LAYER)

Figure 35 - Time Versus Cone Bearing for Medium Dense Sand Layer
11.2.3 **Distance Dependent Behaviour**

In order to analyze the distance dependent behaviour for each layer, a plot of cone bearing versus distance has been produced for each of the 3 layers. These depict the relationship with measurements taken at the centreline of vibrocompaction, 1m, 2m, and 3m away. As no data is available for the 4m distance, the centreline before data has been used to show that by 3m, the soil has returned to its original state. However, it has since been realized that the line of testing was wrongly positioned as it should have started at a probe hole. This means that the decrease in cone bearing depicted may be on the high side as the soil tested has been affected by more than one probe hole. This is evident in each of the layers as the gradient is less for the first metre from the probe hole, thereafter becoming more uniform as would be expected.

11.2.3.1 **Sand Fill Layer**

The plot of cone bearing versus distance is shown in Figure 36. As indicated on the figure, the section of the layer investigated extends from 1.375m to 2.125m. Again the lower starting depth was chosen due to the sand above being effected by a vibrating surface roller.

The plot shows the decrease of cone bearing with distance form the centreline. Good agreement is shown for the whole layer investigated. By the 3m distance no evidence of an increase in cone bearing is indicated.
DISTANCE vs CONE BEARING
(SAND FILL LAYER)

NOTE: (1) The data is filtered using the median.
(2) The test was carried out 82 DAYS after densification.
(3) The 4m distance is the $Q_c$ before densification.

Figure 36 - Distance Versus Cone Bearing for Sand Fill Layer
Therefore, for the sand fill layer, the zone of influence for the Tri Star probe is a radius of about 2m.

11.2.3.2 Silt Layer

The plot of cone bearing versus distance is shown in Figure 37. As indicated on the figure, the section of the layer investigated extends from 2.375m to 4.125m. As with the time plot this is a composite picture, showing the mixing of the sand fill with the silt.

The top section of the plot shows the same pattern as the sand fill, formerly silt, as the cone bearing gradually decreases with distance from the centreline. The cone bearing also decreases vertically with depth, indicating the mixing effect. The trend continues downwards until the silt layer is reached at which point it remains relatively constant. Thus, the Tri Star probe has improved the first 1.5m of the silt layer. No evidence of cone bearing gain is seen after 3m for the entire zone. Therefore, for the silt layer, the zone of influence for the Tri Star probe is a radius of about 2m.

11.2.3.3 Medium Dense Sand Layer

The plot of cone bearing versus distance is shown in Figure 38. As indicated on the figure, the section of the layer investigated extends from 6.625m to 9.125m, since it contained the least variable data.
DISTANCE vs CONE BEARING
(SILT LAYER)

NOTE: (1) The data is filtered using the median.
(2) The test was carried out 82 DAYS after densification.
(3) The 4m distance is the C_L before densification.

Figure 37 Distance Versus Cone Bearing for Silt Layer
NOTE: (1) The data is filtered using the median.  
(2) The test was carried out 82 DAYS after densification.  
(3) The 4m distance is the $C_L$ before densification.

Figure 38  Distance Versus Cone Bearing for Medium Dense Sand Layer
The plot shows the gradual decrease of cone bearing with distance from the centreline. This is the most important layer as it was its susceptibility to liquefaction during a strong earthquake that required it to be vibrocompacted. The plot indicates that improvement was achieved and increased the cone bearing up to a distance of 3m away from the centreline. Therefore, as with the preceding layers the zone of influence for the Tri Star probe is a radius of about 2m. This is quite significant as the same test carried out by the contractor (Massarsch and Vanneste, 1988) 6 days after vibrocompaction showed only a slight increase at 1m from the centreline for the medium dense sand layer (Figure 17). However, the percentage difference between 6 and 82 days for the 1m distance is approximately 190%. This indicates that the soil outside the area enclosed by the probe holes undergoes a slower increase of cone bearing with time, as would be expected, because of the reduced compaction effect.

11.2.3.4 Conclusions

It may be concluded that for this particular site and the soils investigated, that the cone bearing gain essentially finishes at a distance of 3m from the centreline of densification. Thus, the Tri Star probe has a zone of influence of about 2m. The soil beyond the area contained by the probe holes undergoes a slower increase in cone bearing.
11.3 Relative Density

11.3.1 Introduction

Recent research has shown that the stress-strain and strength characteristics of a cohesionless soil are too complex to be represented only by the relative density of the soil. There has been much discussion on the difficulties of uniquely determining the maximum and minimum sand densities for calculation of relative density. Recent work with cone penetrometers in large calibration chambers has attempted to develop relationships between cone bearing, effective stress and relative density. Although no unique relationship exists between these parameters, useful relationships have been derived if sand compressibility is taken into account and if cone bearing is correlated with the in-situ horizontal effective stress. One such relationship given by Baldi et al. (1982) is shown in Figure 39. This relationship applies to normally consolidated, uncemented and unaged quartz sands, of moderate compressibility. The relationship can also be applied to over consolidated sands if the in-situ horizontal stress is used instead of the vertical stress. It is considered that the sands found at the Annacis Island site are similar in nature to the Ticino test sands from which the relationships of Baldi et al. (1982) were developed, and the relationships are meaningful for the sands in question.
Figure 39  Relative Density Relationship for Uncemented and Unaged Quartz Sands (adapted from Baldi et al. 1982)
11.3.2 **Time Effect**

The effect of relative density as a function of time is shown in Figure 40.

The relative density exhibits the same behaviour in the sand fill and silt layers as did the cone bearing, i.e. the mixing effect. This is evident by the bottom of the sand fill located at 2m before vibrocompaction was then found at 4m after vibrocompaction.

In the medium dense sand, the relative density shows the same progressive build up as cone bearing and sleeve friction with the relative density being increased from 55% to 85-90%. Again, there is the unexpected peak at 82 days, the reason for which has been explained in section 10.2.1.1.

Since no account has been made of possible increases in lateral stress brought about by the densification process, the relative density so evaluated represents an upper bound value.

11.3.3 **Distance Effect**

The effect of relative density as a function of distance 1m, 2m and 3m from the centreline of vibrocompaction is shown in Figure 41.

The aspect of the sand fill mixing with silt can be seen quite clearly if one observes the series of figures. The sand fill profile gradually moves back from its peak value of 90% from 0 - 4m (top left graph) to almost its virgin
Figure 40  Effect of Relative Density With Time 67, 82, and 209 Days After
Figure 41 - Effect of Relative Density With Distance at 0, 1, 2, and 3m from Centreline (82 days after)
position of 55% from 0 - 3m (bottom right graph). Also, the top of the silt layer moves back close to its original position in stages from 4m to 3m.

The medium dense sand layer progressively returns to its virgin state at 3m from the centreline. Hence, the zone of influence from a probe hole appears to be about 2m radius.

11.3.4 Conclusions

For the medium dense sand layer, the relative density was increased from 55% to 85 - 90%. The relative density profiles quite clearly indicate the mixing of the sand fill with the silt layer hence improving the top 1.5m of the silt layer. The relative density indicates that the zone of influence appears to be about 2m radius.

11.4 Shear Resistance
11.4.1 Introduction

Theories correlating the drained resistance of sand with cone penetration resistance are either based on bearing capacity or on cavity expansion theory. The cavity expansion approach is complex in its requirement for shear strength and compressibility input data for its analysis. The two main parameters which control penetration resistance in sands are shear strength and compressibility. Since the variation of compressibility in most natural sands is small, the shear strength has more influence on
cone resistance than compressibility. For this reason, the bearing capacity theories, which cannot account for soil compressibility are able to offer reasonable predictions of friction angle. From a review of calibration chamber and triaxial test results, Robertson and Campanella (1983) compared the various theoretical relationships available, noting whether such relationships tended to under or overestimate friction angle. Following this work, they were able to propose an average relationship which has been reduced to the form shown in Figure 42. Again, the relationship is applicable to moderately compressible, normally consolidated, unaged and uncemented quartz sands. The relationship is considered meaningful for the sands in question and has been used to estimate friction angles from the cone penetration tests performed at the Annacis Island site.

11.4.2 Time Effect

The effect of the friction angle as a function of time is shown in Figure 43.

The series of graphs show the same characteristics as the relative density graphs, i.e. the sand fill mixing with the silt leading to improvement of the silt and a progressive increase of the measured parameter in the
Figure 42 - Relationship Between Cone Bearing and Friction Angle for Uncemented and Unaged Quartz Sands After (Robertson and Campanella, 1983).
Figure 43 - Effect of Friction Angle With Time 67, 82, and 209 Days After
medium dense sand layer, in this case the friction angle. This is expected as an increase in friction angle corresponds to an increase in the relative density. The friction angle was increased from $39^\circ$ to $44^\circ - 46^\circ$, an increase of about 6. Again, the peak at 82 days is in evidence.

As with relative density, no account has been taken of possible increases in lateral stress brought about by the densification, so the friction angle evaluated is also an upper bound value.

11.4.3 **Distance Effect**

The effect of the friction angle as a function of distance from 1m, 2m and 3m from the centreline of vibrocompaction is shown in Figure 44.

This shows the same behaviour as relative density with distance, i.e. mixing of the sand fill with the silt and a steady decrease of the parameter with distance in the medium dense sand layer to its original position.

11.4.4 **Conclusions**

According to the interpretation methods adopted, the Tri Star probe was capable of increasing the friction angle of the medium dense sand from 39 to 44 - 46. The friction angle profiles are similar to the relative density profiles which is as expected as they are related to each other.
Figure 44 - Effect of Friction Angle With Distance at 0, 1, 2, and 3m From the Centreline (82 Days After)
11.5 Comparison of Geotechnical Parameters

11.5.1 Comparison of the Friction Angle Evaluated from the DMT and CPTU

A percentage difference comparison of the friction angle estimated before and after by the CPTU and DMT is shown in Figure 45.

The data reduction programme supplied with the DMT uses the bearing capacity equation of Durgonoglu and Mitchell (1975) to calculate friction angles. As regards the time of testing, the 82 day CPTU test was the closest to the DMT test at 111 days. The difference in time was not felt to be important as the CPTU testing has shown that the majority of the time effect had occurred by 82 days.

The comparison reveals that the profiles are similar in shape but the CPTU gives a percentage difference of 10% more in the medium dense sand layer while it is almost 40% more in the sand fill/silt zones. This suggests that the relationship used to calculate the friction angle from the DMT may need to be modified to suit the soils in question since, unlike the CPTU, the side friction is not measured. Hence the CPTU will give a more accurate value of the friction angle. Further, the DMT has not shown the same sensitivity to the changes undergone in the sand fill/silt layer.
Figure 45  Before and After Percentage Difference Comparison of Friction Angle Evaluated from CPTU and DMT
11.5.2 Comparison of Young’s Modulus Estimated From the DMT and CPTU

The Young’s modulus (E) was evaluated from the CPTU soundings using the relationship between cone bearing and Young’s modulus for normally consolidated, uncemented sands given by Robertson and Campanella, (1983), which is shown in Figure 46. It is considered that the sands encountered at Annacis Island are applicable to the relationship and a 50% failure stress level has been used as suggested by Robertson and Campanella (1983). Further, a vertical effective stress of 1 bar was used based on the DMT data. The Young’s modulus from the DMT was evaluated using a version of the data reduction programme supplied with the DMT apparatus. It uses an empirical correlation based on Marchetti (1980).

The comparison between E from the CPTU and DMT is shown in Figure 47 for the medium dense sand only. The data between 0 - 4m is not shown as it was too variable for comparison purposes. Figure 47 shows that the Young’s Modulus evaluated separately by the CPTU and DMT is very similar for the soil in question and the applied conditions. The reasons for the test times are the same as given in section 11.5.1.

The increase in E in the medium dense sand is significant and is between 50 - 100%.

The above findings suggest that both the CPTU and DMT are equally effective in determining Young’s Modulus for the
Figure 46  Relationship Between Cone Bearing and Drained Young’s Modulus for Normally Consolidated, Uncemented Quartz Sands (After Robertson and Campanella, 1983)
Figure 47  Before and After Percentage Difference Comparison of Young’s Modulus Evaluated from CPTU and DMT.
medium dense sand at this site, provided the same conditions are applied.

11.5.3 Comparison of Shear Modulus Evaluated From the CPTU and SCPTU

The shear modulus, $G_{\text{Max}}$, was evaluated from the CPTU sounding using the relationship between cone bearing and dynamic shear modulus for normally consolidated, uncemented quartz sands given by Robertson and Campanella (1983) shown in Figure 48. Again, as for $E$, a vertical effective stress of 1 bar was used. The evaluation of $G_{\text{Max}}$ from the SCPTU has already been discussed and can be found in section 10.4.1

The comparison shown in Figure 49, shows the before and after profiles at 82 days (when the SCPTU was available) of each penetration test. This was done as $G_{\text{Max}}$ given by the SCPTU provided inconclusive results as discussed in section 10.4.2, hence a percentage difference comparison could not be carried out.

As the data from 0 - 4m is missing from the SCPTU plot only the medium dense sand will be discussed. The before profiles of each test are similar, while the CPTU gives a larger value for $G_{\text{Max}}$ for the whole layer. It has been documented that the SCPTU accurately determines $G_{\text{Max}}$ (Campanella and Robertson, 1986), therefore it is felt that the relationship which gives a $G_{\text{Max}}$ from the CPTU data would require modifying for the soils encountered.
Figure 48  Relationship Between Cone Bearing and Dynamic Shear Modulus for Normally Consolidated, Uncemented Quartz Sands (Robertson and Campanella, 1986)
Figure 49 Comparison of $G_{\text{Max}}$ Evaluated from CPTU and SCPTU
The CPTU shows an increase of 50 - 100% in $G_{\text{Max}}$ after densification. This is the same as the Young's Modulus also evaluated from the CPTU which is as expected as they are related.

11.5.4 Comparison of Lateral Stress Evaluated From the DMT and LSCPTU

The lateral stress was evaluated from the DMT sounding by combining $K_Q$ and the vertical effective stress, both provided by the DMT data reduction programme. The $K_Q$ was determined by the method developed by Marchetti (1985) for sands from the DMT data which has been shown to be a good relationship. As regards the LSCPTU, this gave a direct measurement of lateral stress which is not the true in-situ stress but a penetration lateral stress. Hence, LSCPTU tests may provide too high a measurement. The DMT and LSCPTU tests were carried out at 111 days and 204 days respectively. The difference in time was felt to be negligible as for the same reason given in section 11.5.1.

Figure 50 shows the before and after profiles from the DMT and LSCPTU. This has been done to highlight the large difference in value of lateral stress that each has measured due to densification. The DMT data falls into the same range as found by similar testing carried out at Annacis Island (Hitchman, 1989), i.e. an increase in $K_Q$ from 0.5 - 1.0, corresponding to an increase in lateral
Figure 50  Comparison of Lateral Stress Evaluated From the DMT and LSCPTU
stress from 75 kPa - 150 kPa. The LSCPTU data has been found to be too large (an increase from 400 - 600%) in comparison to testing done at other similar sites. The reason for this was that the LSCPTU was still at an experimental stage and the high values (taking into account penetration) were due to equipment and calibration problems, i.e. the lateral stress cell gave too high a value plus the pore pressure element behind the lateral stress cell did not function properly and produced erroneous data. This also meant that $K_0$ could not be evaluated.

Interestingly though, a percentage difference comparison (Figure 51) shows that the profiles are relatively similar. The DMT and LSCPTU show a 50% and 100% difference respectively for the medium dense sand layer and again for the sand fill/silt layer.

Although the LSCPTU absolute data is questionable, the LSCPTU has shown it can measure lateral stress but that it requires calibration in order to provide the actual lateral stress which is at present being addressed. Hence, any further comparisons between lateral stress will be based on the DMT data.

11.5.5 Comparison Between Lateral Stress and Relative Density

One of the objectives of this research was to investigate the mechanism of how the medium dense sand becomes more
Figure 51  Percentage Difference Comparison of Lateral Stress Evaluated From the DMT and LSCPTU
resistant to penetration and therefore to liquefaction. This is believed to be caused by the relative density, or lateral stress or a combination of both, being increased. Hence, based on the DMT, the lateral stress has been increased by 50% while based on the CPTU, the relative density has been increased by 65%. This suggests that the increase in relative density could be considered the dominant effect but it is felt the difference is not large enough to be conclusive. Therefore, it is necessary to investigate how much each parameter contributes to the cone bearing increase.

As it has been shown that the \( K_0 \) condition and hence lateral stress have been increased due to densification, it is possible to separate the influences on cone bearing of increased relative density and lateral stress. According to the relationship of Figure 39, the 75 kPa increase of horizontal effective stress would be responsible for an overestimation of relative density in the region of 15%. On this basis, the post-treatment relative densities shown in Figures 40 and 41 should be reduced by 15% over the range of treatment, whilst the friction angles of Figures 43 and 44 would be reduced by 2 degrees. Therefore, based on the fact that the absolute relative density was increased from 55% to 90%, the relative density contributes 20% while the lateral stress contributes 15% to the increase in cone bearing. Although this indicates the relative density is slightly more dominant it may be more
appropriate to suggest that both contribute equally to the increase in cone bearing.

11.6 Conclusions

The cone bearing in relation to time showed that for the three layers investigated, its gain was curved and peaked between 6 and 82 days after a sharp increase which was between 200 and 400% for the medium dense sand. After 82 days, a decrease was noted which is felt to be due to a tide change altering the ground water level. The silt layer was improved indirectly by the sand fill mixing with it from above. In terms of distance each of the layers indicated the Tri Star probe has a zone of influence of about 2m radius.

The relative density and angle of friction exhibited the same behaviour, i.e. a progressive increase with time. The relative density was increased from 55% to 85 - 90% while the angle of friction was increased from $39^\circ$ to $44^\circ - 46^\circ$. Both parameters also indicated the mixing effect of the sand fill/silt layers quite clearly.

The Young's modulus evaluated from the DMT and CPTU both indicated a similar increase of 50 - 100% from before and after vibrocompaction in the medium dense sand. Hence, both the CPTU and DMT may be used to evaluate Young's Modulus for the medium dense sand at this site providing the applied conditions are the same.
The CPTU in comparison to the SCPTU gave a larger $G_{\text{Max}}$ for the before profile. As it has been documented that the SCPTU accurately determines $G_{\text{Max}}$ (Campanella and Robertson, 1986), it is felt that the relationship used to develop $G_{\text{Max}}$ from the CPTU data would require modification for the soils encountered.

The CPTU $G_{\text{Max}}$ indicated an increase of 50 - 100% from before and after vibrocompaction which is equal to the increase in Young's modulus which is as expected as they are related.

The comparison between the DMT the LSCPTU lateral stress showed a similar profile but the LSCPTU was 50% greater in the medium dense sand and sand fill/silt layer. This suggests that the LSCPTU needs further calibration which at present is being addressed.

It is felt that the increase in cone bearing is due equally to an increase in lateral stress and relative density as both increases are relatively close.
12.0 COMPARISON WITH OTHER VIBROCOMPACTION SYSTEMS

12.1 Introduction

The compaction capacity and behaviour of the Tri Star probe have been demonstrated in the preceding sections. To properly assess the benefits of the instrument it has to be compared relative to other vibrocompaction equipment available. This has been carried out based on work done by Hitchman (1989) which uses two methods. The first compares the ability of the Tri Star and other probes to densify soil around a single compaction probe. The second method involves comparing the grid spacing/probe diameter ratio of each of the probes.

12.2 Densification Comparison

In Figure 52, the data of three other authors has been compared with the Tri Star data. On the ordinate is plotted the previously adopted quantity, the final cone bearing normalized for the initial cone bearing multiplied by the initial relative density. In this way, differences in compaction depths and initial densities may be taken into consideration. On the abscissa is plotted the distance from the probe, normalized by the diameter of the probe. This is important since the effective sizes of probes considered differ by nearly an order of magnitude. Whilst the size of the equipment is taken into consideration, there is no allowance for differences of machine power output. Comparing similar machines, the
Figure 52  Ability of the Tri Star Probe to Densify Soil Around a Single Compaction Probe Compared to Other Probes (After Hitchman, 1989)
vibroflot of D'Appolonia (1953) for which data is shown in Figure 52, developed a centrifugal force of 10 tons, compared to only 2.3 tons for the Phoenix equipment. No such data is available for the machine described by Webb and Hall (1969), but it is assumed to be of at least the capacity as that of D'Appolonia. The vibrator atop the Tri Star probe developed a centrifugal force of up to 1.13 MN (Massasch and Vanneste, 1988).

Figure 52 shows that each of the probes except the Phoenix probe are able to cause soil improvement at up to distances equal to about 2 probe diameters, whereas the Phoenix probe is capable of improving soil at distances of up to 6 or more probe diameters when drainage is used. If drainage is not used, the results are within the range of similar equipment, except in the vicinity of the probe where poorer results were obtained. However, the Tri Star and D'Appolonia probes are more effective up to 2 probe diameters.

12.3 Grid Spacing/Probe Diameter Comparison

Table 10 shows details of the major systems available for the deep compaction of soils in-situ. To compare approximately the effectiveness of the various machines, the ratio of typical probe spacing to probe diameter has been considered. Typical values of this parameter for the Tri Star and similar vertically vibrating systems are in the range of 1 to 3, while the vibroflot attains values
<table>
<thead>
<tr>
<th>Machine</th>
<th>Diameter D (m)</th>
<th>Frequency F (Hz)</th>
<th>Spacing S (m)</th>
<th>S/D ratio</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>GKN Vibroflot</td>
<td>0.45</td>
<td>30</td>
<td>2.7-3.7</td>
<td>6-8</td>
<td>Brown 1977</td>
</tr>
<tr>
<td>Foster Terraprobe</td>
<td>0.76</td>
<td>15</td>
<td>0.9-2.4</td>
<td>1-3</td>
<td>Anderson 1974</td>
</tr>
<tr>
<td>Vibrorod</td>
<td>0.5</td>
<td>-</td>
<td>1.7</td>
<td>3</td>
<td>Saito, 1977</td>
</tr>
<tr>
<td>Mytilus</td>
<td>2.1</td>
<td>25</td>
<td>6.5</td>
<td>3</td>
<td>Davis et al., 1981</td>
</tr>
<tr>
<td>Vibro-wing</td>
<td>1.6</td>
<td>20</td>
<td>2.5</td>
<td>2</td>
<td>Massarsch &amp; Broms, 1983</td>
</tr>
<tr>
<td>Franki Tristar</td>
<td>1.0</td>
<td>20</td>
<td>2.0</td>
<td>2</td>
<td>Massarsch &amp; Vanneste, 1988</td>
</tr>
<tr>
<td>Phoenix Machine</td>
<td>0.19</td>
<td>25</td>
<td>1.8</td>
<td>9</td>
<td>Present study</td>
</tr>
</tbody>
</table>

Table 10  Details of Various Vibrocompaction Systems  
(After Hitchman, 1989)
from 6 to 8. This reflects well the widely held belief that the horizontally vibrating devices are far more effective than the vertically vibrating probes. The Phoenix Machine scores the highest spacing to diameter ratio of 9. This indicates that for its size, the Phoenix Machine may be considered the most effective of the systems considered.

12.4 Summary

The first comparison indicated that the Tri Star probe is equal to similar probes in its ability to densify up to 2 probe diameters away. However, the Phoenix Probe, when drainage is used, appears to be more effective beyond 2 probe diameters. Thereafter, the second comparison showed that the Tri Star probe is equally effective as to similar systems but the Phoenix probe appears to be the most effective overall.
13.0 **CONCLUSIONS**

13.1 **Introduction**

The primary objectives of this research were the long term evaluation of the performance of the Tri Star vibrocompaction technique and to compare various penetration testing equipment. This was a continuation of the work carried out by the contractor, Franki Canada Ltd.

13.2 **Post Construction Conclusions**

It was concluded that the 750 feet long by 15 feet wide strip area was successfully densified by the Tri Star method down to a depth of 10m. Densification in the medium dense sand layer (below 5m) exceeded design criteria, as verified by extensive SPT and CPTU tests. The initial SPT blowcount varied between 8 and 16 and the required minimum had to exceed 17 at the 10m depth. The actual average blowcount after treatment varied between 40 and 65 although maximum blowcount values of 81 were achieved. No densification was expected in the silt layer, but CPTU tests showed that an improvement occurred indirectly by mixing of the silt with the sand fill above. Thus, it was concluded that the obtained densification indicated that the adopted compaction procedure (vibration time and grid spacing) was conservative.
13.3 **CPTU Results**

In terms of the long term effects, the CPTU was the primary source of information. It was used to investigate the time and distance behaviour of the soil. The results for each layer investigated, showed an increase in cone bearing with time to peak between 6 and 82 days. After 82 days, a decrease is evident which is felt to be due to ground water level changes caused by tide level differences. The gain is between 200% and 400%. The effect of distance shows that for each of the 3 layers the Tri Star probe has a zone of influence of about 2m radius.

Through the investigation it has been shown that the silt layer, a material not suitable for vibrocompaction has in fact been improved. The process was an indirect result of vibrocompaction as the sand fill above the silt mixed with the silt and caused the top 1.5m of the 2.5m silt layer to be improved.

The friction ratio proved to be a good soil type indicator.

13.4 **DMT Results**

The DMT results were only carried out at one point in time but were comparable to those of the CPTU. By carrying out DMT tests perpendicular to each other, it was found that the Tri Star probe densifies equally in the horizontal direction. Like the friction ratio, the Material Index proved to be a good soil type indicator.
13.5 **SCPTU Results**

The SCPTU did not provide good data from this site due to equipment and operational problems, hence no conclusions could be made.

13.6 **LSCPTU Results**

This instrument confirmed the way in which the soil responded to vibrocompaction but only qualitatively. This was because the data was erroneous due to the instrument still being in the experimental stage.

13.7 **Interpretation of Geotechnical Parameters**

The relative density and friction angle showed similar behaviour, i.e. a progressive increase with time. The relative density was increased from 55% to 85 - 90% while the friction angle was increased from 39° to 44° - 46°.

The comparison between Young's modulus evaluated from the DMT and CPTU was quite favourable in the medium dense sand layer, both tests indicating an increase of 50 - 100%. This suggests that the CPTU and DMT are equally effective in determining Young's Modulus for the medium dense sand at this site and the applied conditions. A similar increase (50 - 100%) was noted for $G_{\text{Max}}$ evaluated from the CPTU which was as expected as the two are related. On comparison to the before profiles of the SCPTU (as the after were erroneous) the CPTU values were greater which
suggested that the relationship would need to be modified to suit the soils encountered.

With regard to lateral stress, the LSCPTU and DMT produced a very similar percentage difference profile although the LSCPTU provided a 50% greater difference. This suggests, that the LSCPTU needs further calibration in order to provide a more accurate value of the lateral stress, which at present is being addressed. Hence, the DMT data was used to assess how lateral stress and relative density contribute to the increase in cone bearing due to densification. This indicated that the lateral stress and relative density contribute 15% and 20% respectively. The small difference is felt to suggest they contribute equally to the increase in cone bearing.

13.8 Comparison to Other Probes

The Tri Star probe compares well with other such equipment. However, it has been shown that when drainage is used as in the case of the Phoenix probe, the vibrocompaction effect is greater beyond the effectiveness of the Tri Star probe.

13.9 Recommendations For Future Research

The operation of the Tri Star probe worked very well at this particular site as it exceeded the required densification criteria. Therefore no recommendations are necessary in this regard.
Future research should focus on providing a comprehensive time study of the vibrocompacted material. It is suggested that this be done on a weekly basis. Also, instead of using only the CPTU on a regular basis, the SCPTU, DMT and LSCPTU should be used as well. This testing should be carried out within the grid centre and also laterally from an outer probe hole of the grid.
REFERENCES


Schmertmann, J.H., 1983, "Revised Procedure for Calculating $K_0$ and OCR from DMT's with $I_p > 1.2$ and which Incorporate the Penetration Force Measurement to Permit Calculating the Plane Straining Friction Angle", DMT Workshop, Gainsville, Florida.


APPENDIX - Field Test Data
CPT DATA
Site Location: ANNACIS IS VCOMP
CPT Date: 18/05/88
On Site Loc: CBB-ANN-10
Cone Used: UBC7 BTIP & BFS
Comments: VIRGIN AREA 500

UBC IN SITU TESTING

CONE BEARING
SLEEVE FRICTION
FRICTION RATIO
POROSITY
DIF. P. P.
INTERPRETED

DEPTH (meters)

Max Depth: 15.075 m

Depth Increment: 0.025 m
UBC IN SITU TESTING

Site Location: ANNACIS IS VCOMP
CPT Date: 18/05/88
Page No: 1 / 1
On Site Loc: C88-ANN-11
Cone Used: UBC7 BTIP & BFS
Comments: CL SECTION A

CONE BEARING
Dt (bar)

SLEEVE FRICTION
(bar)

FRICTION RATIO
RF (OD)

PORE PRESSURE
U (a. of water)

DIFFERENTIAL P.P.
RATIO AU/Qt

INTERPRETED
PROFILE

DEPTH
(meters)

Depth Increment: .025 m
Max Depth: 11.1 m
UBC IN SITU TESTING

Site Location: ANNACIS IS VCOMP
CPT Date: 02/05/88
On Site Loc: C88-ANN-12
Cone Used: UBC7 BTIP & BFS
Comments: IM(S) SECTION A

CONE BEARING
SLEEVE FRICTION
FRICTION RATIO
PORE PRESSURE
DIFFERENTIAL P.P.
INTERPRETED

DEPTH (meters)

Depth Increment: .025 m
Max Depth: 10.025 m
DMT DATA
Location: ANNACIS IS VIBROCOMP
Filename: D8-AN-2  Test Date: 15-6-88

U.B.C. In Situ Testing