A CONE PENETRATION TEST (CPT) BASED ASSESSMENT OF EXPLOSIVE COMPACTION IN MINE TAILINGS

by

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ABSTRACT

Failure of tailings dams can occur due to liquefaction of saturated and loosely deposited silt and sand sized tailings under both seismic and static conditions. The consequences in terms of loss of lives and property are severe. As tailings possess soil-like structure and grain size distribution, it is possible to use ground improvement measures typically used for soils to improve the stability of these dams. It is also possible to use densification techniques to reduce the volume of the in-place tailings to provide greater storage space.

This thesis reviews the assessment of ground densification at a tailings facility in Northern Ontario at which a section of tailings dam was densified by explosive compaction (EC). The EC was complemented by surface compaction using Dynamic Compaction and Rapid Impact Compaction. Due to the nature of tailings deposition, the grain size distribution and density of materials in the dam varied considerably both laterally and with depth. This complicated the assessment of the improvement obtained.

Piezometer cone penetration test data obtained before and at various times after ground treatment were reviewed to assess the range of tailings types encountered in the dam and the level of tip resistance achieved by the ground treatment. Settlement and piezometer data were also reviewed but were of insufficient quality or quantity to be useful as indicators of the degree of improvement obtained.

Cone data were normalized for stress level and were sorted according to a unified soil behaviour type classification scheme previously used in soils and tailings. The soil behaviour type index, I_c, was found to be a useful indicator of tailings type. Despite the extreme variability of the deposits, it proved possible to identify the level of tip resistance achieved by the ground treatment in various material types at the site.

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a	Net area ratio for a given piezo cone
B_q	Pore Pressure Ratio = $\Delta u/(q_t - \sigma_{vo})$
COV	Coefficient of Variation
CPT	Cone Penetration Test
DC	Dynamic compaction
EC	Explosive Compaction
f_s	Sleeve friction stress
F	Normalized friction ratio= $f_s/(qt-\sigma_{vo})$ x 100%
I_c	Soil Behaviour Type Classification Index
N	Measured standard penetration test (SPT) value
N_{60}	N corrected for 60% of the theoretical free fall energy
PPR	Pore pressure ratio
q_c	Measured uncorrected cone tip resistance
q_t	Cone tip resistance corrected for unequal area effects
Q	Normalized tip resistance $(q_t-\sigma_{vo})/(\sigma_{vo}- u_0)$
R_f	Friction Ratio (f_s/q_t)
RIC	Rapid Impact Compaction
SBT	Soil Behaviour Type Chart
Ψ	State Parameter
σ_{vo}	Total overburden stress
u_0	In-situ equilibrium pore water pressure
u_1	Pore pressure measured on the face of the cone
u_2	Pore pressure measured behind the cone tip
u_3	Pore pressure measured behind the friction sleeve
Δu	Excess pore pressure measured

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1 INTRODUCTION

Tailings consist of ground rock and process effluents and are generated in a mine processing plant. Mechanical and chemical processes are used to extract the desired product from the run of the mine ore and produce a waste stream known as tailings. Tailings are discharged normally as slurry to a final storage area known as a Tailings Management Facility.

Dams, embankments (mostly constructed using coarser beach tailings) and other types of surface impoundments are common tailings storage methods used today. Continuous improvement in metal extraction efficiencies has led to an increase in waste generation and tailings disposal problems have become acute. Consequently tailings dams are being built higher and faster.

Tailings being by-products of crushed rock are soil-like in structure and grain size distribution. Hence the engineering behaviour of tailings is governed primarily by in situ density and stress history of the tailings deposits. Fine grained tailings, for example, exhibit all the problems of "difficult soils" such as collapsibility on account of the electro-chemical nature of particle interactions. There is also the possibility of liquefaction of saturated and loosely deposited silt sized tailings under seismic and static loading conditions. A number of case histories (Dobry & Alvarez, 1967) documenting failures of tailings dams due to liquefaction of fine grained tailings confirm that the above phenomenon is real.

Ground improvement measures used for liquefaction mitigation of natural soils can also be used for tailings deposits. Such measures are expected to improve the bearing capacity, reduce settlements, and increase resistance to liquefaction of tailings. Densification of tailings dams is also desirable to minimize the volume tailings occupy in the facility, reducing the cost of raising the dams.

Explosive compaction (EC) has been used for densification of soils in various projects world wide for the last 70 years. It involves sequential detonation of explosives in loose soils to achieve densification. It is generally applicable to loose, saturated silts, sands and gravels. Soils at larger depths (> 50m) can be treated using EC (Gohl et.al, 2000). It is also relatively

inexpensive on account of the low cost of explosives and operations (drilling and loading explosives etc.), as they are both already a part of the mining process (Gohl et al. 2000). Thus, in spite of being based on empirical designs, it presents an attractive choice for ground improvement.

EC is also expected to be effective for densification and volume reduction of non-plastic, granular, silt/sand sized tailings on tailings dams. Since most mining sites have the necessary expertise for the use of explosives, the method is easy to implement.

The in situ performance assessment of EC and ground improvement in general is carried out using penetration tests such as the cone penetration test (CPT), standard penetration test (SPT), dynamic cone penetration test (DCPT), etc. CPT testing provides a continuous record of the response of the ground to penetration to large depths, which can be interpreted for stratigraphic logging and ultimately to indicate the engineering behaviour of the ground. Interpretation techniques for this purpose for natural soils are already developed and in use. The CPT is also much faster than other field tests and hence is a preferred test for assessment of the ground improvement.

The interpretation of tailings behaviour using CPT with the interpretation techniques and correlations developed for natural soils can be misleading. Although the tailings possess structure and grain size distribution similar to soils, they may have different mechanical and hydraulic properties owing to their unique mineralogy, mode of formation and geological history since deposition, all of which affect the CPT response. Thus, CPT data interpretation in tailings is complicated by these differences between tailings and natural soils.

A review of the related literature shows that there are very few published case histories on the application and in situ performance assessment of EC in tailings (Klohn et al. 1981; Handford et al. 1988). Those dealing with the CPT based assessment (Fordham et al. 1991; Gandhi et al. 1999), do not report the use of any approach specifically developed for the interpretation of the CPT data in tailings.

While the previous work does account for the influence of factors such as time dependent gain in strength (aging), tailings type and ground stress state (mean confining stress) on the

CPT based assessment, it does not adequately consider the influence of site heterogeneity (mainly in terms of tailings grain size) on the assessment of the ground improvement. Results of ground improvement from the limited data regarding the performance of EC in tailings are encouraging. However, consideration of the above factors is critical to the CPT based assessment of densification and the feasibility of the future use of EC on tailings dams.

In this thesis, CPT data obtained from a recent case history, dealing with the use of EC on tailings dams, are reviewed to determine the ground improvement achieved at the dam site. The CPT based assessment involves the use of a combination of normalized CPT parameters to characterize the pre and the post treatment ground in the form of a soil/tailings behaviour type (SBT) classification chart, SBT index I_c (Jefferies & Davies, 1991; Been and Jefferies, 1993) and a simple statistical framework (mean, median and standard deviation) to account for the site variability in tailings type. The SBT chart is considered useful for classification of fine grained soils/tailings as it proposes to account for the pore pressures developed in these geo-materials during the penetration of a cone (Jefferies & Davies, 1991). Influence of aging and tailings type on the achieved densification is also highlighted using this framework. Additionally, piezometer and settlement marker data from the same case history are used to indicate the response of tailings to explosive compaction and assessment of densification. It is expected that the approach used in this case towards the CPT based classification of the ground and determination of degree of increase in tip resistance in tailings will lead to a better assessment of EC in tailings and will contribute towards the feasibility of future use of EC on tailings dams.

1.1 Summary of Case History

Golder Associates Innovative Applications (GAIA) undertook in 2004 a tailings densification program at the Central Tailings Facility in Copper Cliff, Sudbury, Ontario. The proposed tailings dam raise was to be founded on loosely deposited tailings that were susceptible to liquefaction.

An earlier investigation on the same site carried out by Golder Associates Limited in 2000 (Golder, 2002) concluded that foundation instability would occur for the design earthquake and ground densification was recommended as a mitigative measure. After a careful review of available options of densification, Explosive Compaction was selected for densification of deeper tailings zones (> 12m below ground level) along with surface compaction by Rapid Impact Compaction and Dynamic Compaction for the shallow zones.

A detailed site investigation program was undertaken in 2004 by GAIA, to assess the ground conditions before the densification treatment, mainly in the form of Piezo Cone Penetration Tests (CPTU). Test blast sections were designed and executed to assess the suitability of Explosive Compaction (EC) for the site and to design a blast pattern. The area to be treated was divided into panels and a number of such panels were treated with EC. The post treatment ground condition was mainly assessed using CPT. The pre and the post treatment investigation data (borehole logs and CPT) along with the blast design information and the results of the field instrumentation (piezometers and settlement markers) were provided to UBC by GAIA, to permit a study of the densification achieved at the Sudbury site for the improvement techniques used (mainly EC).

1.2 Thesis Objectives and Methodology

This thesis examines the CPT, piezometer and settlement marker data obtained during the ground improvement process in an attempt to understand the effectiveness of explosive compaction for the densification of Sudbury tailings having a range of grain sizes (clay sized to silt and sand sized tailings). A major focus of this study is on the use of CPT data for above purpose which involves an attempt to ascertain the level of improvement in CPT tip resistance achievable in these tailings following the application of treatment (EC and surface compaction).

The following methodology is proposed to achieve the above objectives:

(a) Interpretation of CPT data in tailings using the soil behaviour type classification chart (Jefferies & Davies, 1991) and soil behaviour type Index I_c.

(b) Consideration of the effect of factors such as aging and geo-material type on the available post treatment CPT data.

(c) Comparison of the pre and the post treatment CPT parameters for assessment of ground improvement with regard to the site variability.

(d) Comparison of the performance of EC (as assessed from the CPT, piezometer and settlement data) for the present case with the previous use of EC in soils and tailings.

1.3 Organization of the Thesis

Chapter 2 reviews the literature in the context of the need for ground improvement on tailings dams, applicability and CPT-based assessment of EC in tailings. The latter has been reviewed in the background of performance assessment of EC in natural soils. Shortcomings from the review have been highlighted at the end.

Chapter 3 provides a detailed background of the case history with an emphasis on the site details, site data description, data collection and data quality.

Chapter 4 involves analysis of the processed CPT data and discussion of the results of analysis for indicating the ground improvement achieved in light of factors such as aging and tailings type. The results have been related to previous results from similar assessment in tailings and soils.

Chapter 5 presents the conclusions along with the limitations of this research and guidelines for future work.

2 LITERATURE REVIEW

This review highlights the issues pertaining to the CPT-based assessment of ground improvement achieved using Explosive Compaction (EC) in tailings deposits. Factors affecting the CPT-based assessment of ground improvement such as tailings behaviour type, time dependent gain in strength (aging), ground stress state as well as site heterogeneity have been reviewed. The ground improvement achieved as evinced by the post treatment CPT results in natural soils and tailings is also reviewed.

2.1 Mine Waste Tailings and Tailings Dams

Tailings are the residual wastes of mineral processing that remain after extraction of the desired mineral of value from the ore. They are either produced or deposited in a slurry form. The mineral ore is first crushed in a crusher and then ground in a grinding mill to reduce ore from rock to tailings size. The gradation of the tailings produced is influenced by the degree of grinding and the mineral content (sand, clay etc.) of the parent ore. Tailings particles usually show a high degree of angularity. Vick (1983) gives a detailed account of the stages involved in tailings production, transport, and discharge on mineral processing plants, which forms the basis for the ensuing sections.

2.1.1 Transportation and Discharge of Tailings

Tailings are generally transported as thin slurry in pipes to suitable disposal ponds retained by tailings dams. The discharging is carried out either by a single-point discharge method (where the open end of the tailings discharge pipe is moved periodically to form a series of adjacent and overlapping deltas) or by spigotting, wherein the slurry is discharged through closely spaced spigots in the pipeline. After the discharge, coarser tailings settle out from the slurry near the point of discharge, leading to the formation of a beach area near the embankment. The finer tailings are carried to ponded water, forming a decant pond as shown in Figure 2-1.

Owing to the tendency of coarser and finer particles to settle in different areas of the containment facility (beach and ponds respectively) tailings are often described on the basis of gradation using the terms sands and slimes.

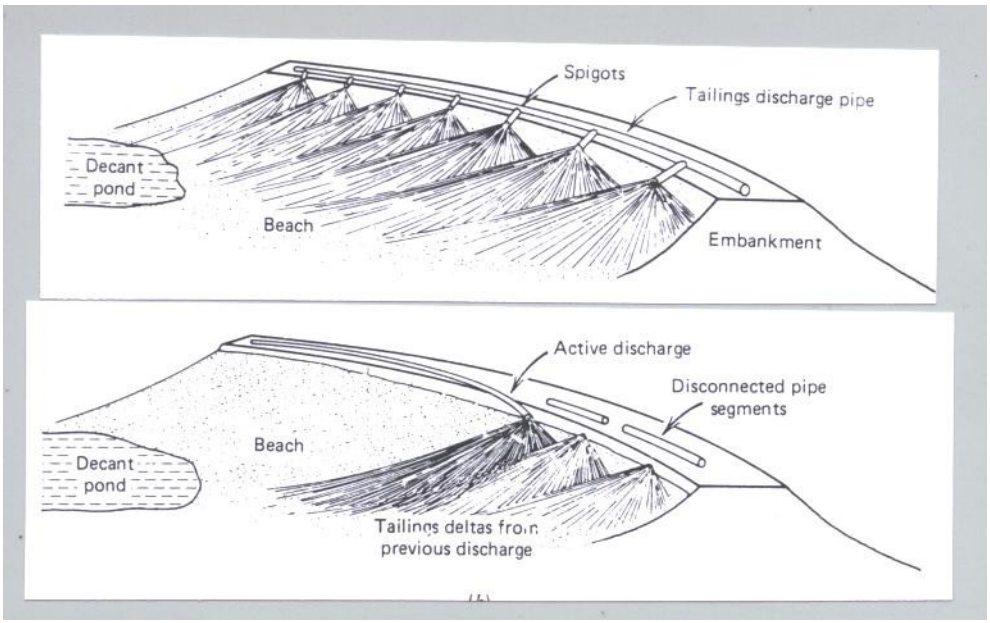


Figure 2-1: Tailings Discharge Methods (from Vick, 1983)

2.1.2 Tailings Gradation

Coarser tailings (sand sized) represent the fraction retained on the No. 200 sieve and, in theory, settle out on the beach while the finer tailings (silt or clay sized) passing through the No. 200 sieve are deposited in ponds. In reality, however, the degree of such segregation varies greatly within a deposit and from one deposit to another, and depends on factors such as gradation of the mill grind, solids content, climatic conditions, and flow rate of the slurry (Davies, 1999).

Since tailings are soil sized, geotechnical characterization of tailings is carried out by determining the index properties (similar to soils) which include gradation, specific gravity, and plasticity. A number of investigators have studied the above properties for different

types of tailings. Vick (1983) presents an exhaustive review of these studies for tailings derived from different mines located in different parts of the world. The case history cited in this thesis was implemented in Copper Cliff in Sudbury. This region is rich in copper tailings. Figure 2-2 shows the gradation curves for copper tailings from the mines in United States and Canada. Whole tailings (tailings before discharge) have been reported as non plastic while slimes are reported to have low plasticity.

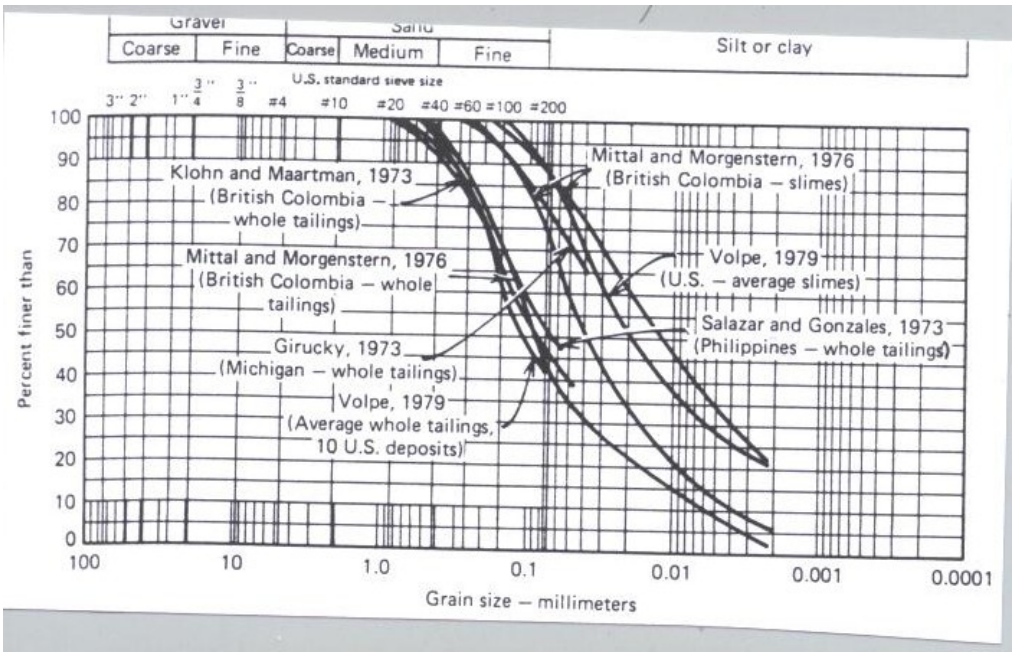


Figure 2-2: Different Gradations Reported for Copper Tailings (Vick, 1983)

Table 2-1 summarizes these index properties for copper slimes tailings. Specific gravities quoted range from 2.6 (similar to those for soils) to 3.0 (greater than that for soils) for these tailings.

Table 2-1: Plasticity of Copper Tailings (Vick, 1983)

Location	Liquid Limit (%)	Plasticity Index (%)
Western US.	40 (average)	13 (average)
British Columbia	0-30	0-11

A soil-like structure and grain size distribution means that like the soils, the engineering behaviour of tailing is governed by the density and the stress history. Also, the electro-chemical nature of particle interactions for the fine grained tailings makes them susceptible to the problems common in difficult soils, particularly the collapsibility soils (Mitchell, 1999).

The nature of the grain size distribution curves indicates the suitability of the tailings for construction of tailings dams. Choice of a particular construction method has critical implications on the seismic stability of the dam, cost of construction, fill requirements, discharge requirements and water storage capacity.

2.1.3 Construction Methods for Tailings Dams

Tailings dams usually consist of raised embankments. The construction begins with a starter dike constructed using natural soil to impound tailings from the processing plant. The dike has to be periodically raised to contain the rising level of tailings within it due to the continuing inflow. Raises are constructed using soils, pit mine wastes or the sand tailings from the same impoundment available in the vicinity. There are three types of construction methods popular in practice:

(a) Upstream construction method

(b) Downstream construction method

(c) Centreline method

The methods differ in the direction in which the crest of the embankment moves in relation to its starting position during its raising. Figure 2-3 shows design sections for the above methods. Several studies highlighting important aspects of the construction of tailings dams are reported in the literature. Mittal & Morgenstern (1975) list ‘stability’ and ‘seepage’ as the two primary performance requirements for tailings embankments. Vick (1983) has added factors such as discharge requirements, raising rate restrictions, fill requirements and relative cost to the above requirements and presented a comparative analysis shown in Table 2-2.

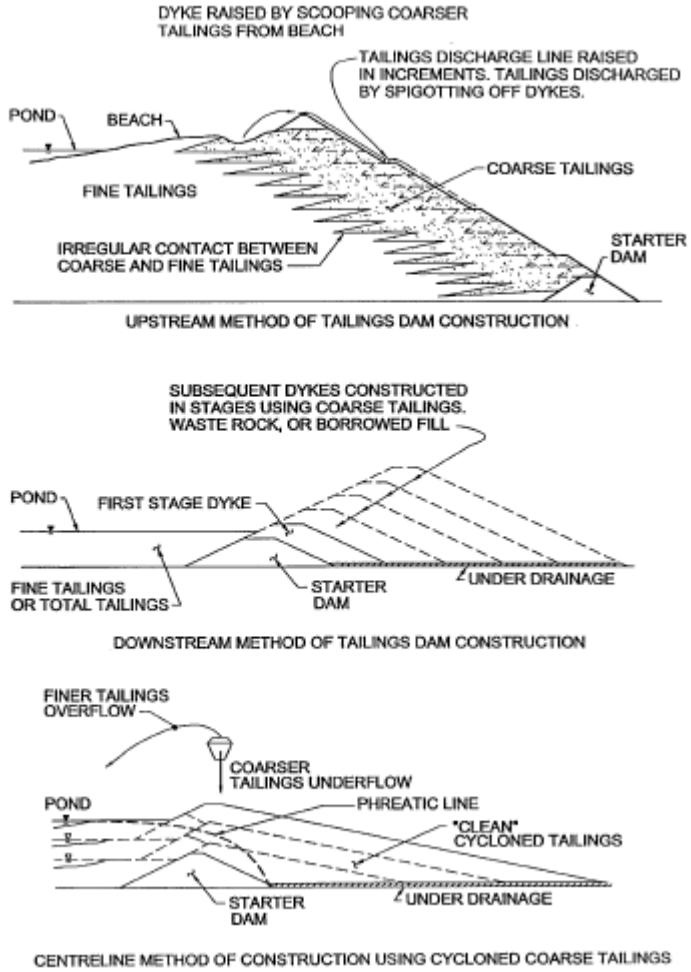


Figure 2-3: Methods of Tailings Dam Construction (Davies, 1999)

Table 2-2: Comparison of Tailings Embankment Types (Vick, 1983)

Embankment Type	Tailings Suitability	Discharge Requirements	Water Storage	Seismic Resistance	Raising rate Restrictions	Fill Requirements	Cost
Upstream	40-60% sand in whole tailings	Peripheral discharge and well controlled beach necessary	Not suitable for significant water storage	Poor in high seismic areas	Less than 15-30 ft/yr	Natural soil, sand tailings Or mine waste	Low
Downstream	Suitable for all types of Tailings	Varies according to design details	Good	Good	None	Sand tailings or mine waste or natural soil	High
Centreline	Sands or low plasticity slimes	Peripheral discharge of nominal beach necessary	Not recommended for permanent Storage	acceptable	Height restrictions for individual raises may apply	Sand tailings or mine waste or natural soil	Moderate

The upstream construction method involves raising the embankment in the direction of the pond over the previously deposited beach tailings. Raises are usually constructed using the coarser tailings from the beach. This ensures minimum fill requirement for building dams and reduces the cost of construction. It also results in an irregular contact between fine tailings loosely deposited in the pond and coarser tailings from the beach resulting in a low relative density of placement as shown in Figure 2-3. Thus, embankments constructed by the upstream methods are susceptible to liquefaction under severe ground motions. The method is however simple and economical (Vick, 1983). A number of case histories documenting the failures of tailings dams are reported in the literature. They highlight the inherent instability of tailings dams against seismic loading and the need for appropriate mitigative ground improvement measures.

2.1.4 Liquefaction on Tailings Dams and Their Stability Issues

Dobry & Alvarez (1967) report seismic failures of upstream tailings dams during the devastating earthquake of 1965 in Chile, resulting in huge losses of life and property. It is shown from the observations made at earthquake sites and accounts of eyewitnesses that the failure of the dams took place due to the liquefaction of tailings in ponds behind the dams. Similarly, Ishihara et.al (1980) report on a failure of the Barahona tailings dam in Chile following an earthquake in 1928, and a tailings dam failure that occurred on the Mochikoshi storage lagoon in Japan in 1978. Liquefaction of slimes was again identified as a trigger in these cases leading to the eventual structural failure of dams.

Slimes are generally deposited very loosely and unless adequate drainage facilities are provided, they remain under consolidated over many years. Liquefaction of such deposits and the consequent dam failures are issues of concern to geotechnical engineers (Ishihara et.al, 1980) for the consideration of the stability of the existing operational upstream tailings dams which are located in seismically active areas.

Use of ground improvement measures in tailings are then necessary for mitigation of problems similar to those encountered for natural soils; specifically liquefaction of fine

grained tailings on tailings dams. This issue assumes considerable importance considering the future safety of these dams.

2.2 Application and Assessment of Ground Improvement Methods

The case history studied in this thesis involves the use of Explosive Compaction (EC) and surface compaction in the form of Dynamic Compaction (DC) and Rapid Impact Compaction (RIC). Hence, these methods have been reviewed in detail in this section with respect to their applicability, design, and CPT-based assessment.

2.2.1 Survey of the Methods of Ground Improvement

Mitchell (1981, 1988 &1999) has presented an extensive and updated state of the art review of ground improvement techniques with respect to the site and soil conditions, choice of ground improvement methods, design, construction and performance assessment of these methods with preliminary estimation of costs involved.

The applicable grain size ranges for various soil improvement methods, including explosive compaction and deep dynamic compaction are shown in Figure 2-4. These two methods are commonly used to increase the liquefaction resistance of loose, saturated and cohesionless soils.

The use of EC involves sequential detonation of explosives in loose soils to achieve densification. It is generally applicable to loose, saturated silts, sands and gravels. The effective depth (up to which the treatment is effective) is limited by the capability of the drilling rig. It is also relatively inexpensive and simple to implement. It is based on empirical guidelines and presents an attractive choice of ground improvement. The different features of EC are reviewed in detail in the following sections with respect to the current understanding of the mechanism of densification, applicability in soils and tailings, implementation and in-situ assessment.

The case history in this thesis also features the use of surface compaction (dynamic compaction and rapid impact compaction) in addition to EC. These methods will also be reviewed in the following sections with respect to the factors listed above.

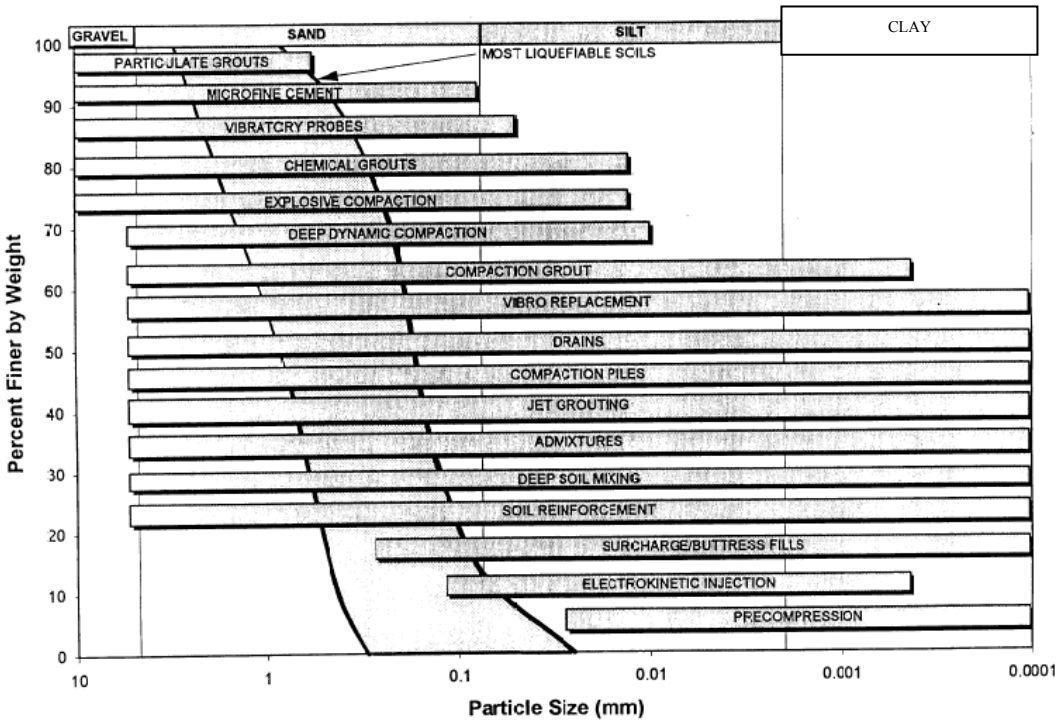


Figure 2-4: Applicable Grain Size Ranges for Soil Improvement Methods (Mitchell, 1999)

2.2.2 Ground Improvement Using Explosive Compaction

Explosive compaction has been used for the past 70 years as an important technique of densification in deep, saturated deposits of loose, cohesionless soils. The densification is achieved by sequential detonation of explosive charges placed in cased boreholes below ground level.

Lyman (1942) (cited by Narin Van Court and Mitchell (1995)) reports the first successful application of EC carried out in the 1930's for densification of foundation soils for Franklin Falls Dam in New Hampshire. Since then it has been used on a variety of projects all over

the world including densification of dam foundations (Hall, 1962; Ivanov, 1980; Solymar, 1984; Murray et al., 2005) foundations for drilling platforms (Stewart and Hodge, 1988) and harbour breakwaters (Carpentier et.al., 1985).

2.2.2.1 Explosive Compaction: Applicability and Mechanism of Densification

EC is best suited for treatment of saturated clean sands and silty sands with initial relative densities of less than 50%. The maximum achievable post treatment relative density after blasting is around 75-80% (Mitchell, 1999 & Gohl et.al. 2000). The method has no known limitation on the effective depth of compaction (Mitchell, 1999). Solymar (1984) reports successful densification of soils up to a depth of 40m.

Blasting initiates large release of energy causing soil particles to rearrange into a denser structure. Narin Van Court and Mitchell (1995) and Green (2001) give the following explanation for the mechanism behind explosive compaction: high explosive charges release large amounts of energy in two distinct forms, shock and gas energy. Shock energy results because the rate of chemical reaction in a high explosive is greater than speed of sound in the explosive material and forms a shock wave which impacts the surrounding soil grains. The pressure applied by these shock waves lasts for only a few milliseconds since it propagates through saturated soils at about the same rate as in water, i.e. about 1500 m/sec (Dowding & Hryciw, 1986).

Pressure due to shock waves propagates radially outward from the charge as a compression wave, inducing in turn the radial displacements in soil. The impact of a shock wave tends to break the existing soil structure formed over a geological time span processes (Dembicki et.al., 1992).

Gas energy resulting from the gaseous reaction products also induces radial displacements in the soil as gas expands from the initial volume of the charge to an equilibrium volume. Explosion pressure applied by expanding gases accounts for more than 85% of the useful energy released by the explosives. This is so because the explosion pressure is maintained for a relatively long time (0.05 to 0.1sec) compared to a shock wave (Dowding, 1985).

The radial displacement causes deviatoric shear strains of a cyclic nature in soils. The resulting tendency towards volume change generates excess pore water pressures. Dowding and Hryciw (1986) suggest that this generation of excess pore water pressures is an important part of the densification process. Following the dissipation of excess pore water pressures, soil surrounding the detonation zone, becomes densified due to rearrangement of soil particles after the shock/ pressure subsides.

Surface settlement takes place rapidly after blasting. Settlement of the order of 2 to 10 percent of the thickness of the treated layer is common (ASCE, 1997). Settlement is accompanied by the release of pore water in large quantities. Gohl et al. (2000) report similar observations for 9 sites on which EC was used. Within several minutes following detonation, the ground started to settle and continued settling for more than an hour. Sand boils developed on sites where the water table was near to the surface. Large quantities of water escaped for as long as 2 hours through the standpipe piezometers installed on sites. Densification induced by explosives is thus an induced consolidation over several hours. Settlements continue at slow rates depending on the soil permeability and the drainage conditions.

It is possible to achieve greater settlement due to the process of sequential detonation at the same site due to charges detonated in the adjacent areas once the excess pore pressures generated from the previous blasts have dissipated. This settlement however depends on soil density and stiffness.

2.2.2.2 Explosive Compaction: Design of Treatment

The degree of densification achieved by EC depends on design parameters such as the sequence of detonation, type of explosives, charge length, and blast hole layout (Narin Van Court & Mitchell, 1995; Gohl et.al., 2000). A typical layout for an explosive compaction program consists of a staggered rectangular grid of charge holes as shown in Figure 2-5 below. This pattern provides two or more blast passes within a uniform grid. Figure 2-5 shows that two passes can be applied for the two series shown.

Charge holes are drilled over the depth of soil layer to be treated and plastic casings are installed in the holes. The explosives are loaded in the holes at one or more levels (decks). A series of charge holes is sequentially detonated.

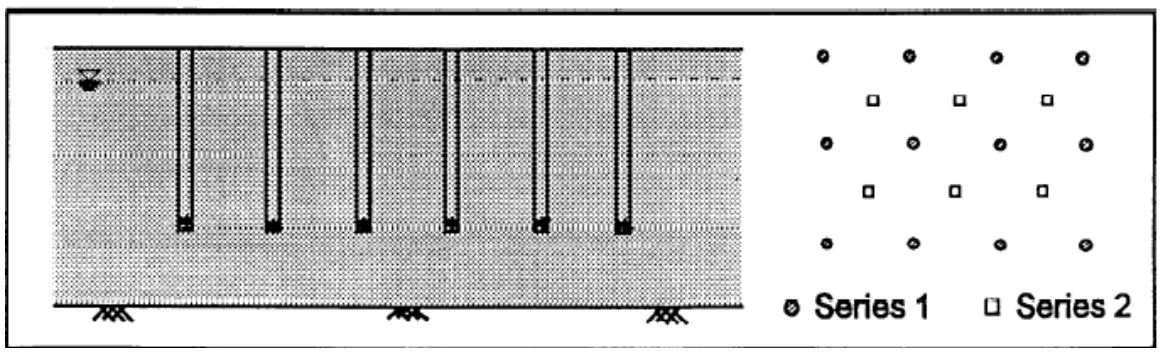


Figure 2-5: Typical Layout for Explosive Compaction (Mitchell, 1999)

Fordham et al. (1991) cites the following important factors which govern the design of an EC program:

(a) Charge density (includes hole spacing, charge depth and charge weight)

(b) The number of decks of explosives

(c) Delays between successive detonations

(d) The effect of blasting on existing structure.

Charge weights and spacing are selected based on empirical correlations between the effects of detonation (settlement of the ground), charge density and distance from the detonation of the charge (Dembicki et.al., 1992; Imiolek, 1992).

Narin Van Court and Mitchell (1995) and Green (2001) present a comprehensive review of these approaches. Their guidelines however are site specific and are not adequate to determine the general applicability of EC at another site. The use of EC has to be confirmed by conducting a trial blast at the site and this is a limitation of EC (Narin Van Court and Mitchell 1995).

Theoretical aspects of the design of an EC program have been considered by Gohl et.al. (2000) based on dimensional analysis and the cavity expansion analysis of Wu (1995, 1996). This theoretical analysis has resulted in the following practical design guidelines:

(a) Explosive charges should be distributed and timed to maximize the magnitude and number of cycles of shear strain of the soil in the zone of densification to achieve more settlement.

(b) Sufficient delays must be observed between successive passes of detonations to allow for dissipation of pore water pressures.

(c) The number of charges detonated sequentially should be restricted to minimize the duration of ground shaking. Longer duration shaking may cause damage to nearby structures and increase residual pore water pressure build up.

(d) Gohl et al (2000) recommend that the charge weights should increase with the square root of depth. The radius of the zone of disturbance caused by a charge depends on the mean confining stress and the strength and stiffness of the soil surrounding it. These increase with depth and hence charge weights should increase with depth.

The range of design parameters used in practice include charge weights of 1.5kg to 15kg, charge densities of 40-80g/$m^3$and delays between the detonation of two series are usually of the order of 1-2 days.

2.2.2.3 Explosive Compaction: Performance Assessment

The use of in-situ tests assisted by field instrumentation in the form of piezometers and settlement gauges forms the basis for assessment of densification due to EC.

In-situ tests such as cone penetration test (CPT) and standard penetration test (SPT) are the most commonly used tests for verification of ground improvement. In addition to CPT and SPT, Mitchell (1999) cites the use of Becker penetration testing, vane shear tests and shear wave velocity tests to verify ground improvement.

The CPT is faster and more reliable and avoids sample disturbance effects in loose silty soils and hence is the preferred test for such conditions.

The post treatment assessment is affected by factors such as aging, soil type, and the in-situ stress state of the soil. It is also affected by the heterogeneity in the soil type existing on the site.

The CPT is also used to assess the suitability of EC at a site by determining the "looseness" of soil in terms of its penetration resistance (Narin Van Court and Mitchell, 1995). Since considerable pre and post treatment CPT data are reviewed in this thesis, assessment of ground improvement by the CPT and the factors affecting the CPT-based assessment will be reviewed next.

2.2.2.4 CPT-Based Assessment

In a CPT, a conical shaped electronic probe along with a friction sleeve and a porous element (piezo element) are pushed into the ground and the ground response to penetration in terms of tip, friction and pore water pressure are recorded (Robertson and Campanella, 1983 a and b). The CPT is primarily used for stratigraphic logging. As the cone is advanced, forces measured on the tip and the friction sleeve vary with the material properties of the soil being penetrated. The excess pore pressure measured during penetration is also a useful indicator of soil type and helps to detect layering in soil (Davies, 1999). The secondary use of CPT data is for the estimation of design parameters through empirical correlations.

2.2.2.4.1 Raw Cone Data

ASTM D 5778 has the following specifications for the standard electric piezo cone: a conical tip with 60° apex angle is 10cm^2 in cross section at its base, a 150 cm^2 friction sleeve and pore pressures measured at one or more locations on the cone. The cone is pushed at the rate of 2cm/sec and measurements are made of resistance to penetration q_c, sleeve friction f_s and pore pressure u_2 just behind the cone tip at 2cm-5cm depth intervals. Figure 2-6 shows a schematic diagram of a standard piezocone.

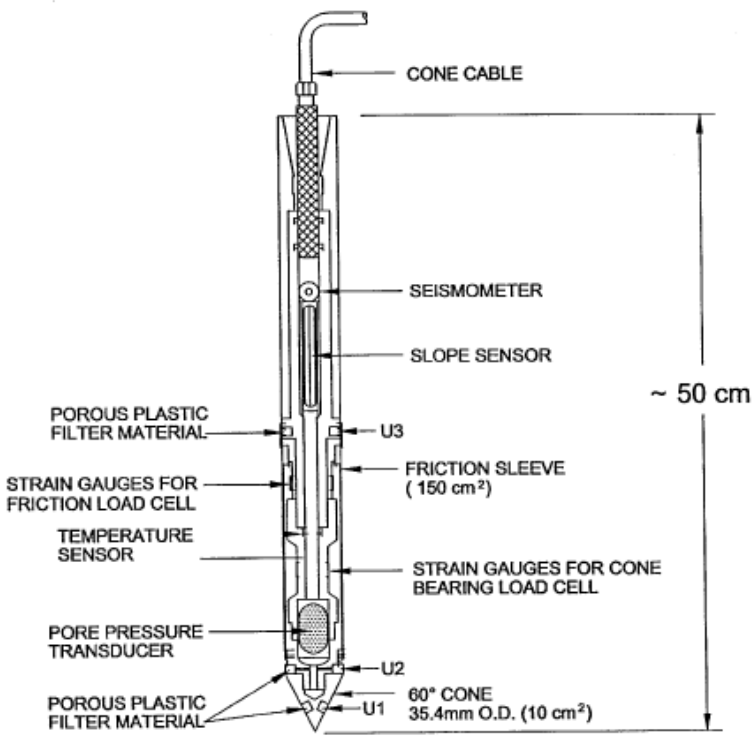


Figure 2-6: Schematic of a Modern Piezocone (Davies, 1999)

The tip and friction load cells record end bearing force divided by the 10cm^2 base area to give tip stress q_c and friction force divided by 150 cm^2 to give sleeve stress f_s respectively. The pore pressure is typically measured behind the shoulder of the cone at u_2. It can also be measured at u_1 or u_3; however u_2 is the preferred location (Campanella et.al, 1986). The temperature and inclination of the cone are also measured as the cone advances in the

ground. These measurements increase the reliability of the test as the effect of variation in the above measurements can be traced and corrected.

The raw data are acquired in electronic files in ASCII format. These files can be used to analyze data with any piezocone evaluation software or simple spreadsheet programs. The raw data collected from SCPTU are processed for a number of corrections.

2.2.2.4.2 Data Processing of CPT Results

Campanella et al. (1983) and Campanella & Howie (2005) have reviewed the factors affecting the raw CPT data. Important factors are excerpted here from the above review:

(a) Unequal Area Effects: Water pressures can act on the exposed surfaces behind the cone tip and on the ends of the friction sleeve. These water forces result in measured tip resistance values, q_c, that do not represent the true penetration resistance of the soil. This error can be overcome by correcting measured q_c for unequal pore pressure effects using the following relationship:

$$q_t = q_c + u_2 (1-a) \qquad \text{Equation 2-1}$$

Where:

q_t = corrected tip resistance

q_c = Actual measured tip resistance

a = net area ratio (typically varying from 0.38 to 0.90 for different cones)

u_2= pore pressure measured behind the shoulder of the cone

(b) Temperature Effects: The load cells within the cone are temperature dependent and are calibrated at room temperature. However, soil and groundwater are considerably cooler than the calibration temperature and a shift in zero can occur for both load cells during penetration. These changes in temperature have little significance for cone testing in sand but can be significant in very soft or loose soils. A zero shift can make friction measurements unreliable.

A temperature sensing element in the cone can help continuously monitor the temperature of the cone and zero shift calibrations are obtained and it is possible to correct all the data as a function of temperature. A temperature calibration should be performed for a new cone.

(c) Negative Friction Sleeve Measurements: Negative friction measurements are caused by (1) negative zero load offset resulting from a temperature change, (2) side loading against the friction sleeve, (3) unequal end area of the friction sleeve in soils with very high pore water pressure and (4) lack of accuracy of the load cell at very small reading. Data files containing negative friction values are edited and adjusted after the cause of such negative values (i.e. below the resolution) is identified.

(d) Cone Inclination: Most cones have built-in slope sensors to measure the non verticality of the sounding. The role of slope sensor becomes critical after about the first 15m of penetration. Excessive deflections may damage the cone. Standard CPT equipment tends to accept about 1° of deflection per meter length without noticeable damage. Test is aborted if the maximum value of 12° is reached during penetration.

(e) Cone Friction Tip Offset: The centre of the friction sleeve is approximately 10cm behind the cone tip. To calculate friction ratio, the average friction resistance and tip resistance are compared at same depth. This involves an offset of 10cm from the apex of the tip which is taken as the reference depth of the sounding.

(f) Rod Breaks: The cone is advanced in the ground with the help of standard (typically 1m long) rods. Each time a rod is added, the test is stopped. This results in reduction in tip resistance. Such tip resistance values are typically spaced at 1m interval. The tip data should be edited to remove the rod breaks before processing.

(g) Other factors affecting raw data include piezometer location, size, type and saturation. These are further explained in the references cited above.

The raw test data are processed either in a spreadsheet or in a text editor program which has provisions for the above mentioned errors and factors. A typical presentation is shown in Figure 2-7 indicating the measured parameters on the left viz. corrected cone resistance, q_t

vs. depth, sleeve friction stress f_s, pore pressure u_2 with hydrostatic pore pressure line u_0 and the calculated parameters on right including friction ratio R_f (f_s/q_t, expressed as percent) and pore pressure ratio B_q (defined in the following sections) which are used to arrive at the interpreted soil behaviour type profile. The process of normalization of the measured CPT parameters and the interpretation of the behaviour type is explained in the subsequent sections. Figure 2-7 shows different visually interpreted behaviour types for different depth intervals.

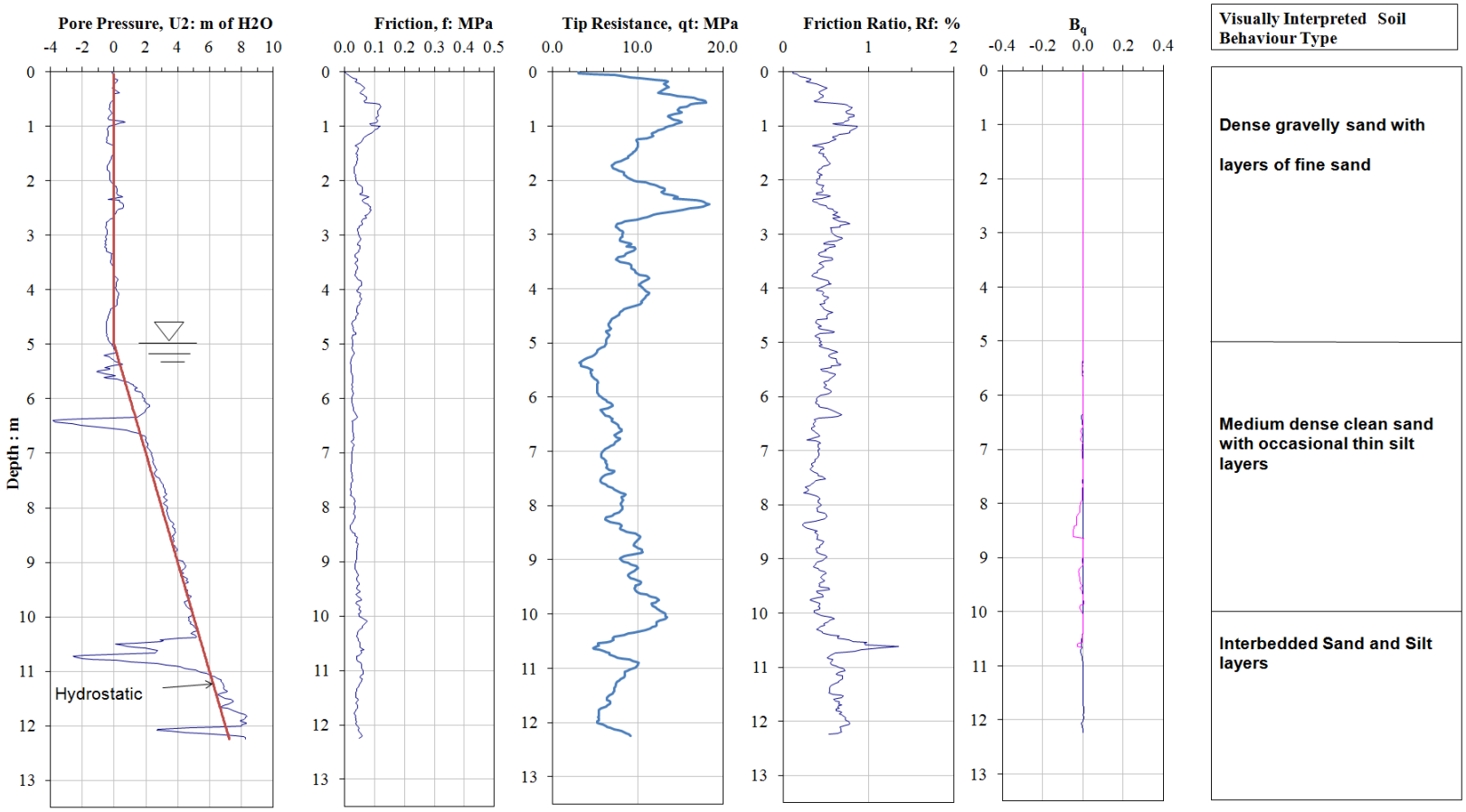


Figure 2-7: Typical CPTU Profile

2.2.2.4.3 Interpretation of Soil Behaviour Type from CPT Data

The corrected CPT data are first visually evaluated for soil behaviour type classification. Trends shown by the parameters such as tip resistance, friction and pore pressures when plotted with depth, give a good indication of layer interfaces, distinct soil behaviour types and equivalent soil types. The visual classification is typically followed by the use of automated computerized classification charts for determining the soil behaviour type.

The detailed methodology for visual classification is cited by a series of investigators: Schmertmann (1978) for determination of possible soil types from tip resistance data, Campanella et al. (1986) and Campanella & Howie (2005) for relating the rate of dissipation of excess pore water pressure to permeability of soil and hence the soil type and for relating the variation in tip resistance, friction and pore pressure to possible soil types. Similarly, the use of computerized charts for soil classification has been introduced by Robertson & Campanella (1983a, 1983b, 1984) and Robertson (1990). Modifications to the above approaches, which have applications for characterization of tailings in addition to normal soils, have been proposed by Jefferies and Davies (1991) and Plewes et.al. (1992). The detailed procedure for interpretation of soil behaviour type from visual evaluations and computerized charts is described below.

Cone penetration in gravelly or sandy soil is typically drained. Consequently, measured pore pressures reflect ambient conditions characterized by very low or negative excess pore pressures. Clays experience undrained failure during cone penetration resulting in excess pore pressures being measured at u_1, u_2 or u_3. The pore pressure data can be interpreted to indicate the stress history for clays.

Penetration in silts or sandy silts is considered as “partially drained” penetration. Silts that dilate during penetration give rise to pore pressures that are below equilibrium, while silts that contract may give positive pore pressures. Excess pore pressures may dissipate rapidly in coarse silts that are cohesionless. The tip and the friction are generally low in silty soils. Interpretation of the data in silts is difficult owing to its partially drained behaviour.

Tip resistance is influenced by the soil type (grain size) and conditions such as in-situ stress state, stress history of the deposit, compressibility and cementation and aging (Campanella & Howie, 2005). It also the reason why the CPT interpretation is considered to give the soil behaviour type rather than soil type alone which is influenced by mainly the soil gradation.

Interpretation of soil type from tip resistance is thus difficult. Schmertmann (1978) proposed various approaches for identifying the likely and possible soil types and conditions for clays and sands from the tip resistance profiles.

Friction ratio profiles are also helpful in visual classification. The friction ratio for clean sand is of the order of 0.5%, while that for finer soils such as silts and clays is greater than 1%.

An example of visual soil classification is given in Figure 2-7. Presence of gravelly sand is indicated by very high tip and friction in the top 5m zone. Increasing silt fraction is indicated by negative pore water pressures, low tip resistance, and increasing friction ratio in the lower zone. Hydrostatic pore pressures help position the water table which is approximately 5m below ground level.

With the development of personal computers, automatic classification and interpretation charts were developed. Robertson and Campanella (1983a) proposed a computerized multi zone soil behaviour type interpretation chart. In this approach, CPT data are plotted on tip resistance q_t and friction ratio space, and q_t and pore pressure ratio B_q space, where B_q is defined as

$$B_q = \Delta u/(q_t - \sigma_{vo}) \qquad \text{Equation 2-2}$$

Where,

Δu = excess pore pressure

q_t- as defined earlier

σ_{vo}= total overburden pressure

Figure 2-8 shows this soil classification chart in terms of tip resistance q_c and friction f_s. q_c is now replaced by q_t as defined earlier in this section. This chart has been modified to include

the cone data collected at the University of British Columbia's (UBC) in-situ testing program. Inclusion of B_q as an independent parameter has generally proved to be useful for the classification of saturated fine grained soils and it complements the q_t- R_f classification, where friction ratio R_f is as defined earlier, f_s/q_t %.

The measured CPT parameters used above (tip resistance q_t and friction f_s) are not normalized for the overburden stress, and hence, change with increasing depth. This will result in an error in the soil behaviour classification. The importance of stress normalizing the piezocone measurements where overburden exceeds 150 kPa to 200 kPa has been highlighted by a number of investigators viz. Houlsby (1988), Been et.al. (1988) and Robertson(1990).

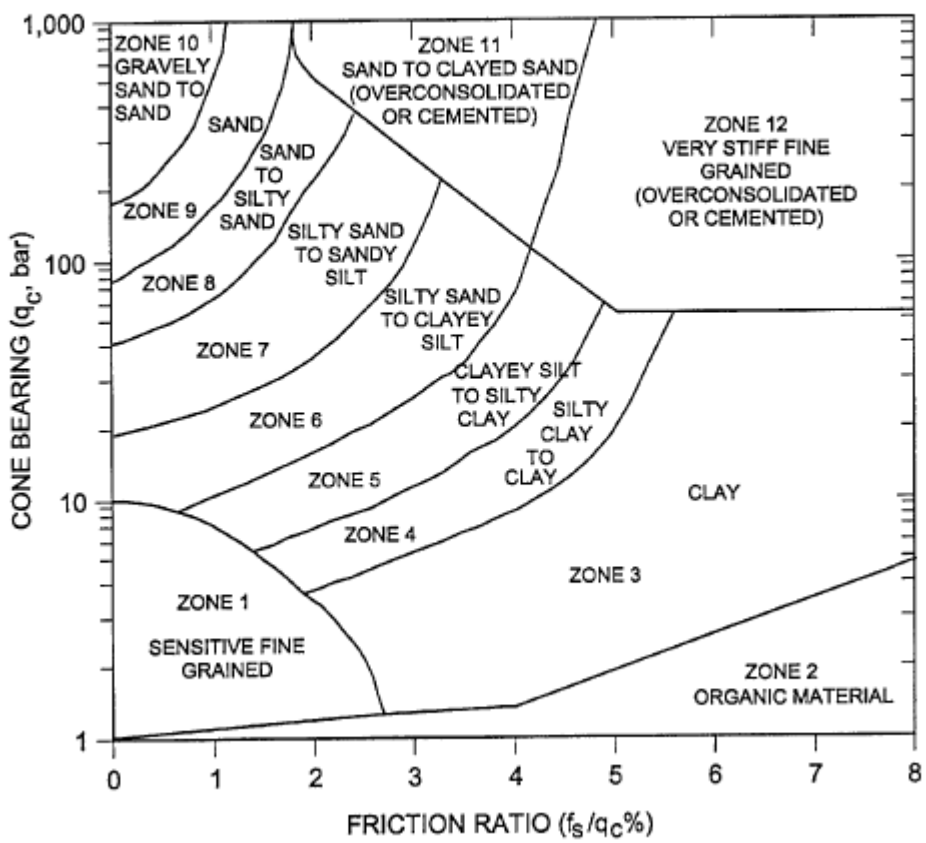


Figure 2-8: Soil Behaviour Type Interpretation Chart (Robertson & Campanella, 1984)

Robertson (1990) proposed a combined stress normalized soil behaviour type classification chart based on linearly normalized tip resistance Q, sleeve friction F, and added a companion chart with B_q. Figure 2-9 shows this approach in Q and F space. The normalized parameters are defined below:

The normalized tip resistance Q is given by

$$Q = (q_t - \sigma_{vo})/(\sigma_{vo} - u_0) \qquad \text{Equation 2-3}$$

Normalized friction ratio is given by

$$F = f_s/(qt - \sigma_{vo}) \times 100\% \qquad \text{Equation 2-4}$$

Pore pressure normalization is carried out as per Equation 2-2.

u_0- in-situ equilibrium pore water pressure

Been & Jefferies (1985) and Wroth (1988) had earlier suggested the use of mean stress or in-situ horizontal stress instead of the overburden stress for normalization. However, the piezocone is not known to provide reliable indications of lateral stress conditions and hence the use of overburden stress is convenient (Davies, 1999). Robertson & Wride (1998) proposed normalizing the tip resistance with $(\sigma_{vo} - u_0)^n$, where n is a stress exponent varying with soil type. These charts have shown their effectiveness for deposits in excess of 30m (Robertson, 1990).

Jefferies and Davies (1991) showed that above chart was deficient for fine grained, cohesionless and brittle mine tailings. Davies (1999) argues that the chart tends to misinterpret the nature of consolidation stress history. Jefferies and Davies (1991) modified the above chart for better classification of mine tailings by using the Q $(1-B_q)$ grouping which incorporates dynamic pore pressure data provided by the piezocone into the existing soil classification scheme. The concept of incorporating pore pressure data directly into the Q $(1-B_q)$ grouping, is to expand the interpretation range in finer soils while leaving the interpretation in sands unchanged, which is important for mine waste tailings (Davies, 1999). Figure 2-10 shows this chart.

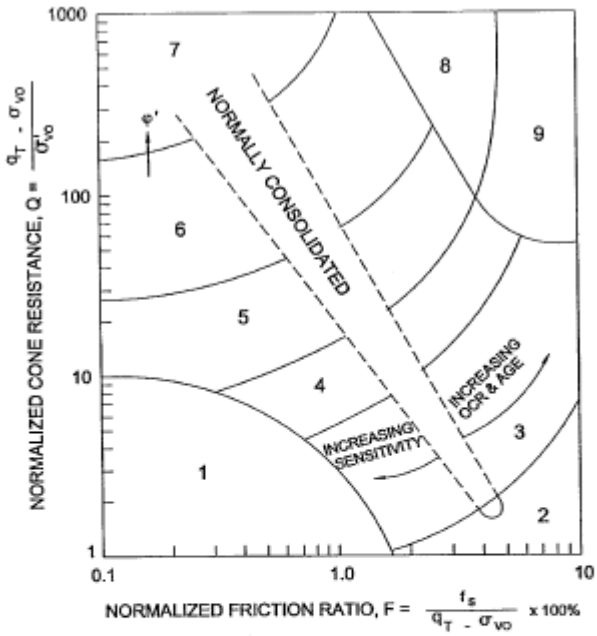
The boundaries between the soil behaviour type zones are approximated as concentric circles and can be characterized by the circle radius. The radius for any combination of Q (1-B_q) and F may be used as a soil behaviour type index. It is denoted by I_c, defined as:

$$I_c = \sqrt{\{3-\log[Q\,(1-B_q)]\}^2+[1.5+1.3\,(\log F)]^2} \qquad \text{Equation 2-5}$$

Values of I_c and the corresponding classification schemes are shown in Figure 2-10. In this thesis, I_c as proposed by Been and Jefferies (1993) shown below is used.

$$I_c = \sqrt{\{3-\log[Q\,(1-B_q)+1]\}^2+[1.5+1.3\,(\log F)]^2} \qquad \text{Equation 2-6}$$

The above version of I_c, includes a "+1" term in the log term, and hence differs slightly from that defined in Jefferies and Davies (1991). Robertson and Wride (1998) proposed another version of I_c which does not include the pore pressure parameter B_q. This version is not considered in this thesis for the reasons cited by Jefferies and Davies (1991).

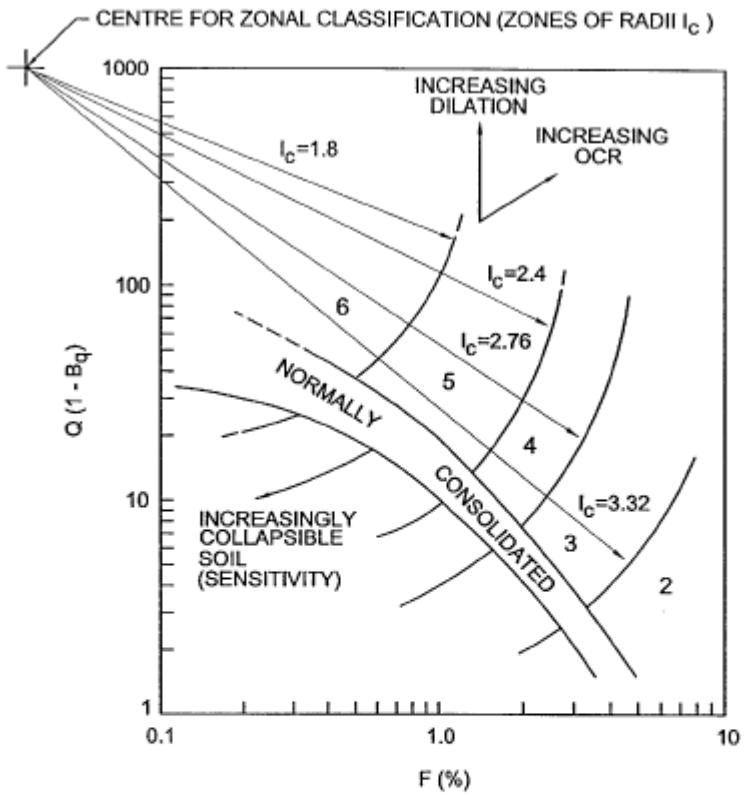


NOTES:

1. SENSITIVE FINE GRAINED
2. ORGANIC SOILS - PEATS
3. CLAYS - CLAY TO SILTY CLAY
4. SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
5. SAND MIXTURES - SILTY SAND TO SANDY SILT
6. SANDS - CLEAN SAND TO SILTY SAND
7. GRAVELY SAND TO SAND
8. VERY STIFF SAND TO CLAYEY* SAND
9. VERY STIFF FINE GRAINED*

*HEAVILY OVERCONSOLIDATED OR CEMENTED

Figure 2-9: Normalized Soil Behaviour Type Classification Chart (Robertson, 1990)



ZONE	SOIL BEHAVIOR TYPE (after Jefferies and Davies, 1991)
6	CLEAN SAND TO SILTY SAND
5	SAND MIXTURES - SILTY SAND TO SANDY SILT
4	SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
3	CLAYS - CLAY TO SILTY CLAY
2	ORGANIC SOILS - PEAT

Figure 2-10: Unified Soil Behaviour Type Normalized Classification Chart (Jefferies and Davies, 1991)

Plewes et al. (1992) proposed a similar approach for interpretation of the soil behaviour type based on the in-situ state of the soil (which is a function of state of stress, density and material compressibility) derived from the piezocone parameters. This approach links the state parameter ψ to the critical state parameters and the piezocone parameters. The relationship shown by Equation 2-7 below was used to estimate the material state from the piezocone data.

$$\Psi = \ln [Q(1-B_q)/(3.6 + 10.2/F)] / (1.33F-11.9) \qquad \text{Equation 2-7}$$

Plewes et al. (1992) proposed that this approach be used as a screening tool to select samples for testing or to identify potentially weak soils which are susceptible to liquefaction based on the initial state ψ (loose or dense) of the material. This was accomplished by plotting contours of ψ as determined by Equation 2-7 on the soil behaviour chart (Jefferies and Davies, 1991) for different values of Q $(1-B_q)$ and F as shown in Figure 2-11 which has been adapted from Davies (1999).

Plewes et al. (1992) further validated the screening tool by comparing the piezocone derived state to the liquefaction hazard predicted from the equivalent stress normalized SPT data (corrected for fines content) and by plotting the CPT data collected from several tailings sites where liquefaction slumps had occurred. CPT data collected from one such site adjacent to the failed zone is also shown in Figure 2-11. It was observed that such data plotted near or below the $\psi = 0$ line and, corresponded to an equivalent normalized SPT value of <10 suggesting, that the soil was susceptible to liquefaction. Similarly, data plotted in the $\psi < -0.1$ zone suggested that the soil behaviour was dilatant and corresponded to an equivalent normalized SPT value of > 20, suggesting that the soil was not susceptible to liquefaction. Thus, the state of soil was correctly indicated by the CPT derived state parameter and it matched with the measured and predicted (on the same sites) corrected SPT data.

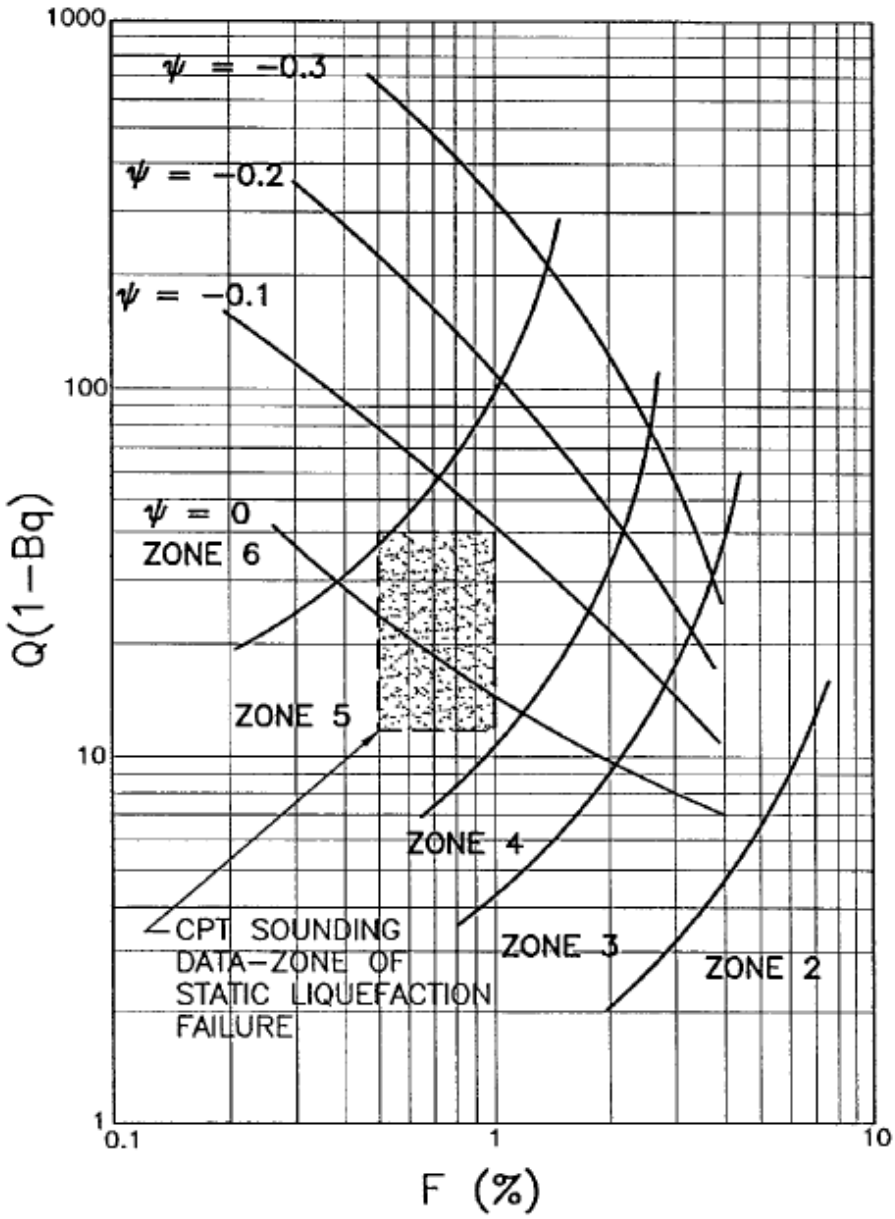


Figure 2-11: State Screening for Suncor Tailing Piezocone Data Near Liquefaction Slump (Davies, 1999)

2.2.2.4.4 Factors Affecting CPT Based Assessment of Densification due to EC

A number of investigators (Schmertmann 1975; Robertson and Campanella 1983a&b; Howie et.al. 2001; Amini et.al. 2002 & Amini, 2006; and Campanella & Howie, 2005) have cited the factors affecting the interpretation of CPT results to assess ground improvement. These factors can be broadly classified into three categories: (1) State, which includes relative density of the deposit, lateral and vertical effective stresses and mean confining stress (2) composition, which includes mineralogy, grain size, fines content and drainage characteristics, and (3) fabric, which includes aging, creep and cementation. Additionally, site variability existing on site is also a factor resulting in uncertainty in the CPT-based assessment of densification.

The CPT measures a response to penetration and hence the CPT data are influenced collectively by the above interdependent factors and CPT results are generally known to indicate the soil behaviour type rather than the soil type, although the two may coincide (Howie et.al. 2001; Amini et.al. 2002 & Amini, 2006; Davies, 1999).

Densification achieved due to the ground improvement process itself can lead to changes in some of these factors. This change is reflected in the post ground improvement CPT results. A major focus of this thesis is to review the post EC densification CPT results in light of factors such as aging (time dependent gain in strength), the geo material type, the stress state of the ground (vertical effective stress and mean confining stress), density and the natural variability at the site. These factors, their variation with EC, and their effect on the CPT results have been briefly reviewed in this section.

Aging: Mitchell & Solymar (1984) and Skempton (1986) presented the evidence of time dependent gain in strength, referred to as aging, in normally consolidated sands, suggesting that the penetration test results are affected by the time since the deposition of the soil. Tip resistance measured prior to the blasting, includes the effect of ageing. Immediately after blasting, a decrease or just a modest increase in measured tip resistance is recorded in sandy soils over its corresponding pre blasting value. Tip resistance, however, is observed to have increased in days or weeks following the blasting compared to the pre blasting values (Solymar, 1984; Mitchell & Solymar, 1984; Stewart & Hodge, 1988; Rogers et.al. 1990;

Gohl et.al 2000; Howie et.al 2001). This behaviour is not observed on all sites treated by EC, and where it does occur, the tip resistance achieved and the rate of increase in tip resistance with time are known to vary (Mitchell and Solymar 1984; Fordham et. al 1991; Charlie et.al. 1992; Narin Van Court and Mitchell, 1995).

Factors such as destruction of the soil bonds formed during geological aging, the chemical reaction at the contact of soil grains, change in soil structure due to blasting, pore pressure dissipation at the interparticle contacts, dissolution of the gases from the detonations, time dependent changes in the effective stress states, and the particle reorientation resulting in increased interlocking have all been suggested as being responsible for the observed variation in tip resistance following blasting (Mitchell & Solymar 1984; Fordham et.al. 1991; Narin Van Court and Mitchell, 1995; Gohl et.al. 2000).

Soil Type: Although the soil type (which encompasses factors such as gradation, fines content, drainage etc.) does not itself change during densification, effectiveness of the ground improvement methods depends on soil type (Refer Figure 2-4) and it affects the post treatment tip resistance. Gohl et al. (2000) report higher tip resistance achieved in clean sands as a result of densification due to EC than that achieved in silty sands. It also implies that the time dependent gain in strength in sands is higher than that in silty soils. On the other hand, Carpentier et.al. (1985) record very little or no improvement in a soil with clay lenses after blasting, indicated by low tip resistance before and after the treatment.

Stress State: This review focuses on the vertical effective stress and the mean confining stress. Both increase with increasing overburden and with OCR. This results in a change in measured CPT parameters with depth. The effect of increased effective stress on the CPT results can be accounted for by normalizing the CPT parameters. The process of linear normalization of CPT parameters is described in detail in Appendix D.

Mean confining stress increases with depth and affects the design of blast densification program. The zone of disturbance caused by the detonation of explosives depends on mean confining stress and soil properties. In order to achieve a desired level of densification throughout the depth of compaction, charge weights are typically increased with depth (Gohl et.al. 2000). The effect of accounting for the confining stress will be seen in terms of a

consistent tip resistance achieved at different depths with other factors, such as soil type already considered.

Overall, variation in CPT parameters with ground improvement is an indication of the variation in the above factors, viz. variation in ground behaviour. Few investigators report ground improvement and the change in ground behaviour due to improvement in terms of normalized CPT parameters, particularly, parameters other than the tip resistance. Amini et al. (2002) have studied ground improvement and the change in ground behaviour achieved by vibro-compaction in terms of change in normalized CPT parameters. They show that the change in ground behaviour results in an increase in normalized tip resistance, a decrease in pore pressure parameter and a decrease in soil behaviour type index I_c. Normalized friction ratio, however, does not show any clear trends.

Natural soil/site variability is another factor which complicates the assessment of ground improvement. Pre and post densification CPT data should be compared only for homogeneous soils in order to correctly assess the effects of ground improvement. However, most sites have natural variability in soil type which affects the assessment. Gohl et.al (2000) and Amini (2006) point out the difficulties in assessment of the ground improvement due to the soil variability. Gohl et.al (2000) represent the pre and the post treatment tip resistance in terms of the range (at 95% confidence) to account for the variability. They have also reported an increase in the variability of tip resistance at various sites after densification.

2.2.2.4.5 CPT Based Assessment of EC in Natural Soils: Summary

The literature review shows a number of case histories that deal with the CPT based assessment of EC. Out of these documented cases, very few assess the effectiveness of the treatment in terms of the normalized CPT parameters. Gohl et al. (2000) have presented stress normalized post EC-CPT data for six sites. The data indicate that on average normalized tip resistance $Q > 70$ has been achieved. However, $Q = 140$ is the upper limit for sands and silty sands. The soils treated by EC were mainly sands and silty sands.

The review showed that the post EC-CPT results (tip resistance) are affected by factors such as aging, soil type, ground stresses and site variability.

Few investigators have accounted for the natural variation in soil type at a site which causes variation in the post treatment CPT tip resistance, and affects the comparison of pre and post treatment CPT data.

The improvement has typically been represented by plotting the pre and the post treatment non normalized tip resistance with depth. Other measured and inferred CPT parameters such as friction ratio, pore pressures and soil behaviour type index I_c which are likely to provide clues regarding a change in ground behaviour due to densification have been rarely used.

2.2.2.5 Assessment of Ground Improvement using Field Instrumentation: Piezometers and Settlement Markers

2.2.2.5.1 Pore Pressure Response from Piezometers

Piezometers are used to measure the pore water pressures in soils. For explosive compaction projects, piezometers are used to monitor the variation of pore water pressures in the ground during and after treatment.

Multiple blast passes are typically employed on EC projects to increase the ground settlement and achieve uniform densification. This requires that the excess pore pressures generated by the initial pass dissipate sufficiently before the subsequent passes are detonated. Delays of the order of days are observed between successive passes of blasts to ensure dissipation of excess pore water pressures. Piezometer records indicate the post blast dissipation of pore pressures.

Different types of piezometers viz. open standpipe piezometer, vibrating wire type and high speed transducers have been used on EC projects (Gohl et.al, 2001a). Gohl et.al. (2001a) explain the pore pressure response captured by a piezometer following a blast: Charge detonations result in propagation of stress waves away from the blast hole generating transient hydrodynamic pore pressures, and residual pore water pressures in soils. Figure

2-12 shows a typical pattern of transient pore pressure pulses (sharp peaks) and the gradual residual pore pressure build up for the multiple hole blasts.

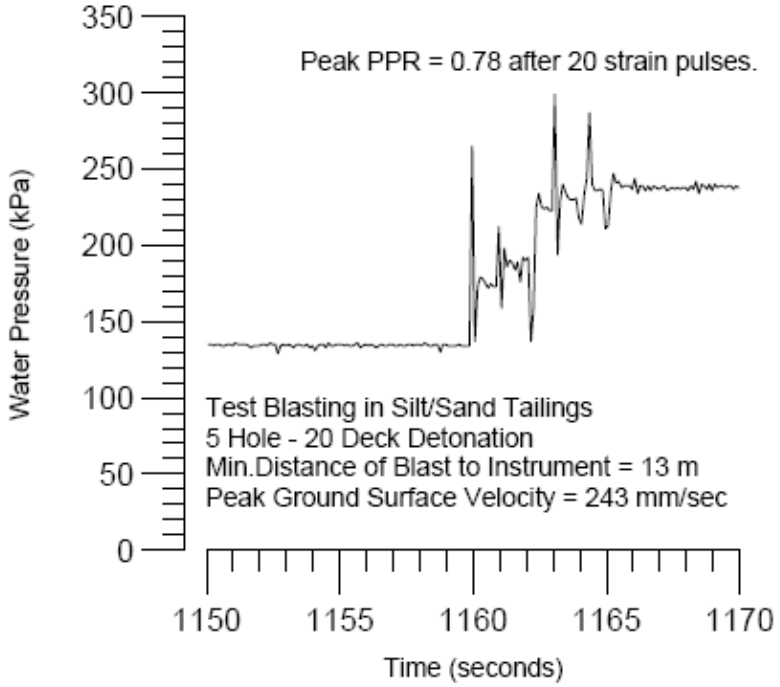


Figure 2-12: Pore Pressure vs. Time for Sequential Blasting at a Tailings Site (Gohl et.al. 2001a)

Residual pore pressure build-up following blasting depends on factors such as charge weight per delay, distance from a charge detonation, the number of charges detonated sequentially and the initial state of soil (Gohl et.al. 2001a). In the extreme, if the residual excess pore pressure equals the initial vertical effective stress (viz. the ratio of residual excess pore pressure to vertical effective stress, defined as pore pressure ratio, PPR, is unity), a condition of soil liquefaction is reached. It is important to set safe limits to PPR in a blast design in order to control the critical pore pressure levels generated due to sequential blasting and provide adequate factor of safety against liquefaction of any foundations to be treated and the structures at or adjacent to the treatment site (Gohl et.al. 2001a).

Thus, the following information is obtained from the piezometer data:

(a) PPR (higher PPR indicates greater tendency towards volume change)

(b) Indication of excess pore pressure dissipation

2.2.2.5.2 Settlement Measurement Using Settlement Markers

The settlements induced by explosives generate large residual excess pore water pressures that cause migration of pore water in the ground to the surface. Large amounts of water escape to the surface following the detonation. The ground settles concurrently with the escape of pore water.

Most of the settlement takes place immediately after the first pass (Solymar 1984; Gohl et.al. 2000). Additional settlement takes place due to multiple blast passes which generate additional cyclic strains in the soil. Settlement may continue at a slow rate depending on soil permeability and drainage conditions at the site. Settlement depends on the initial relative density of the deposit, soil gradation and the blast design used.

The effectiveness of EC is readily apparent from the settlements. Large settlements (4%-10%, over the densification zone) indicate that considerable density increase has been achieved (Gohl et.al 2000).

It is important to ensure that densification has been achieved throughout the depth of the treatment. This is achieved by the use of deep settlement gauges placed at selected depths within the treated soil layer. They are monitored at different times after the treatment: short term (within a few hours) and long term or final settlement (within days) (Walker, 2003). Figure 2-13 shows a typical settlement profile obtained as a result of use of EC.

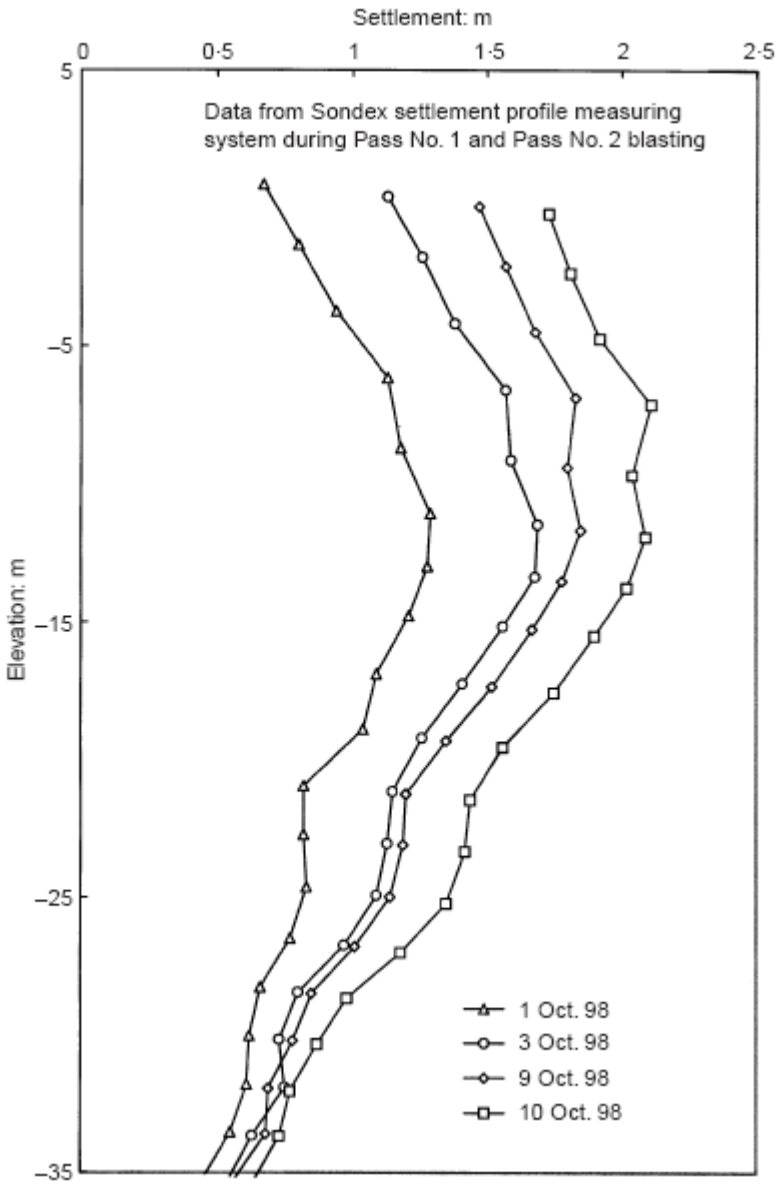


Figure 2-13: Typical Settlement Profile Obtained due to EC (Gohl et. al, 2000)

2.2.3 Ground Improvement Using Dynamic Compaction (DC) and Rapid Impact Compaction (RIC)

The use of dynamic compaction for ground improvement dates back to the times of Romans, but the approach was standardized in late 1960's in the United States by Louis Menard. DC consists of repeated dropping of heavy weights (10-30 tonnes) onto the ground surface from

heights of 15m-30m to densify the soil (Mitchell, 1999), imparting 1470 kJ- 8800 kJ energy to the soil. Rapid Impact Compaction (RIC), on the other hand, was developed in the UK in the 1990's for rapid densification of soils to repair explosion damage on military airfield runways.

2.2.3.1 Ground Improvement Using DC

Mitchell (1999) suggests that the method is applicable for saturated sands and silty sands as shown in Figure 2-4. Menard and Broise (1975) & Mitchell (1999) cite liquefaction of soils from the impact of weight, dissipation of excess pore water pressure, and consequent denser rearrangement of soil particles as the reasons behind ground improvement by dynamic compaction. Dynamic compaction results in immediate settlement, typically 5%-10% of the thickness of the material being treated. The increase in pore water pressure is instantaneous and the dissipation occurs rapidly accompanied by an apparent rise in ground water table level. Soils up to 10m-12m in thickness can be compacted effectively and economically using DC (Mitchell, 1999). The effective depth of treatment D in m, is related to the falling weight W in metric tons, and height of drop H in m by the equation

$$D = (n) * (WH)^{1/2} \qquad \text{Equation 2-8}$$

n is an empirical constant depending on factors such as efficiency of the drop mechanism of the crane, total amount of energy applied, type of soil deposit being densified etc. (Lukas, 1995).

Dynamic compaction is typically performed over a pre determined square grid pattern with controlled drops or passes of weights over the grid layout. Design of treatment (grid spacing, number of drops per impact point, applied energy, and number of passes) depend on soil conditions (such as the depth of the compressible layer, grain size and ground water table), ground response, and the dissipation of excess pore water pressure (Schaefer, 1997). Green (2001) has reviewed different design guidelines followed in practice to carry out dynamic compaction. Schaefer (1997) reports the application of the method to other geomaterials such as uncontrolled fills, mine solid waste, and coal mine spoil.

While SPT is the most commonly used test for assessment of dynamic compaction, use of CPT is increasing. Rollins and Rogers (1991) report the use of CPT for assessment of DC. Slocombe (1993) reports densification of collapsible silty sands and sandy silts using DC. A minimum tip resistance of 4 MPa was recorded as a result of DC in this case in the top 6m zone.

2.2.3.2 Ground Improvement Using RIC

RIC consists of dropping a 6 tonnes pile driving hammer (at the rate of 40-60 blows per minute) onto a circular steel plate that rests on the ground, through a controlled height of 1.2m (Watts and Charles, 1997; GAIA, 2004), imparting about 70 kJ of energy to the ground. While the two techniques (DC and RIC) are similar in operation, they differ in the amount of energy and the manner in which the energy is delivered to the ground. The rate of application of blows is higher in RIC than DC. Watts and Charles (1997) report successful densification of Ash fill deposits up to a depth of 5m as assessed using dynamic cone penetration probe.

2.3 Use and Assessment of EC on Tailings Dams

EC is expected to be useful for densification of loose, saturated, soil sized tailings (granular, low to non plastic silt/sand matrix) typically found on tailings dams. Additionally, the explosives and the expertise on use of explosives are readily available on mining projects, which enhances the feasibility of the application of EC in tailings. Besides the seismic stability, use of EC has also been proposed for tailings volume reduction in tailings ponds (Gohl et al. 2001a; Jurbin 2003). Mines increase the storage space for tailings by typically raising the height of the dam. The settlement resulting from the use of EC is expected to pack the tailings to a denser state and reduce the volume, thereby creating more space for further impoundment.

There are very few documented cases related to the application of EC in tailings dams and other industrial wastes (Klohn et al., 1981; Handford, 1988; Fordham et.al, 1991; Gandhi et.al., 1999). The following subsections briefly review these case histories with respect to

the type of tailings densified, design of EC treatment, results from treatment, and the performance assessment.

2.3.1 Application of EC in Tailings: Survey of Cases

All of the cases cited above deal with the densification of saturated loose silty/sand tailings for mitigation against static and dynamic loadings. Design of the treatment is similar to the design guidelines reviewed earlier in this chapter for natural soils. Gohl et.al. (2000, 2001a) have summarized the geotechnical characteristics of tailings densified and charge weights used on various EC projects. It is seen from this review that initial average relative density for the tailings ranges from 35% to 60%, the tailings are non plastic and the specific gravity ranges from 1.98 (for fly ash densified using explosives) to 2.65 (sand tailings). Gohl et al. (2000) report similar range of geotechnical properties for natural soils with specific gravities suggesting the similarity between the two.

In the cases cited, detonation was followed by surface settlements and ground flooding. Performance assessment has been carried out by SPT (Klohn et.al., 1981; Handford, 1988). Fordham et.al.(1991) and Gandhi et.al.(1999) report the use of CPT for performance assessment. Additionally, piezometers and settlement markers have been used to record the pore water pressure changes and settlements respectively caused by the blasting.

2.3.2 Application of EC in Tailings: CPT Based Performance Assessment

A major factor affecting the CPT based performance assessment in tailings is the interpretation of CPT data in tailings, using the techniques developed for similar interpretation in natural soils. These interpretation techniques are well established for most of the familiar natural soils. Use of these techniques to interpret the response of tailings however, requires confirmation. The material properties of tailings may differ from those of natural soils owing to the differences in formation and depositional characteristics of tailings and soils. Behaviour type classification achieved in tailings using conventional interpretation may be therefore misleading.

Howie (2004) has summarized the difficulties in characterization of "non text book" geomaterials like tailings, using conventional geotechnical testing tools, data interpretation techniques, and correlations developed for natural soils. He notes that non textbook geomaterials (tailings, municipal solid waste and residual soils) differ from the familiar text book geomaterials in terms of their mode of formation and geological history since deposition, resulting in them possessing different mechanical and hydraulic behaviour.

A number of investigators have attempted to use the existing approaches for natural soils, or develop new interpretation techniques, and correlations to interpret the CPT data in mine waste tailings (Jefferies and Davies, 1991; Plewes et al. 1992; Ulrich and Hughes, 1994; Steedman, 1997; Davies, 1999).

Factors affecting the CPT based assessment in natural soils (aging, soil type, stress state of the ground and the site heterogeneity) however, have been considered for tailings. Fordham et.al. (1991) report the use of normalized tip resistance for assessment of the pre and the post treatment ground. The tip resistance increased from a pre treatment range of 20-50 to a post treatment range of 57-80 over a period of four months. Klohn et.al.(1981) and Handford (1988) report the impact of other factors viz. site heterogeneity and tailings type on the assessment of ground improvement in tailings.

2.3.3 Application of EC in Tailings: Assessment Using Piezometers and Settlement Markers

All of the cases cited above in the review report the use of piezometers and settlement gauges. Figure 2-10 shows the stress wave propagation captured by a vibrating wire type piezometer following a blast in sand/silty tailings. Gandhi et.al. (1999) record similar observations in fly ash as a result of multiple sequential blasting as shown in Figure 2-14. The residual excess pore pressure is shown to increase after every blast.

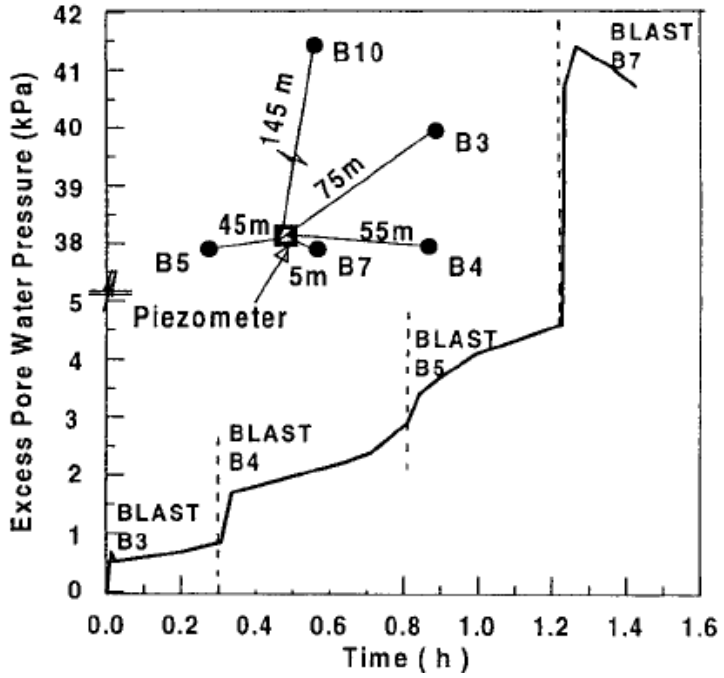


Figure 2-14: Piezometer for Group Blasts in Fly Ash (Gandhi et.al., 1999)

Figure 2-15 shows excess pore pressures measured for another group blast with 24 hours delay between two phases of the blast (Gandhi et.al., 1999). In both the figures, it is seen that the excess pore pressure rises immediately after the blast. Handford (1988) observed that the instantaneous increase in pore pressure is observed for piezometers located within the influence radius of the blast. The piezometers outside of this radius do not record an instantaneous response but do show an increase in excess pore pressure which is considered to be the effect of pore pressure migration.

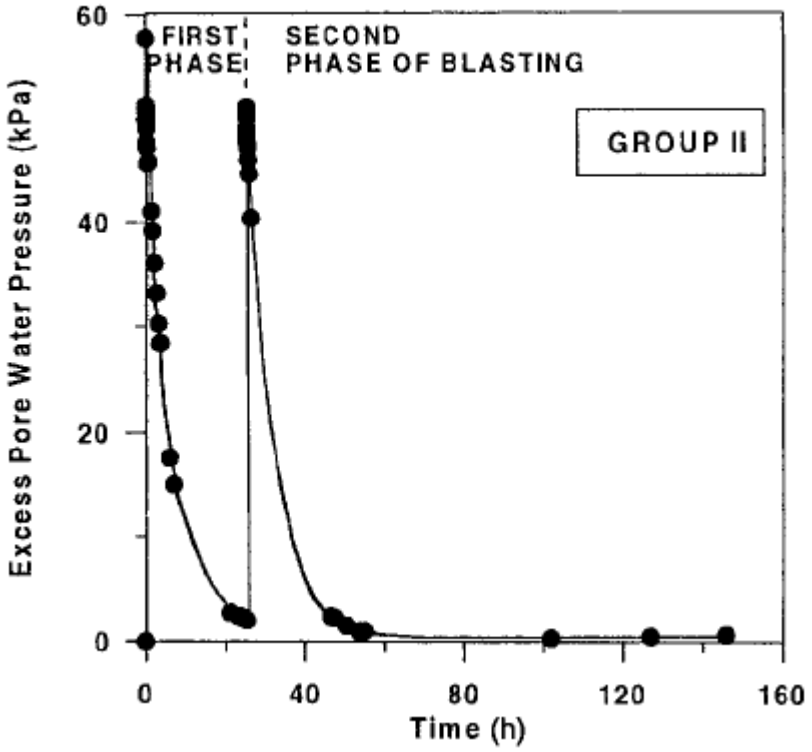


Figure 2-15: Variation of Blast Induced Pore Pressures with Time (Gandhi et.al., 1999)

Settlement response shown by all the cases is similar to that observed for natural soils. Settlement measurement shows that settlement magnitude depends on initial relative density of the deposit, soil gradation, and the blast design. Gandhi et.al. (1999) report a typical section of depression following a single blast. For group blasts, Gandhi et.al.(1999) report higher settlement, with a majority of the settlement taking place within seven days following blasting while a minor gain in settlement was observed after a month.

2.3.4 Summary

A review of four documented case histories related to the use of EC in tailings shows the following:

(a) Regardless of the differences (related to formation and deposition) between the tailings and the natural soils, response of the tailings to EC is similar to that shown

by natural soils. It is seen that the blast densification in tailings is followed by release of pore water, immediate settlement of the ground and consequent increase in relative density.

(b) Review of the CPT-based performance assessment in sand tailings shows that the normalized (for effective stress) tip resistance in the range of 70-80 has been achieved as a result of application of EC. A similar range has been reported for natural clean sands.

(c) It is also seen that the achieved tip resistance is affected by the same factors (aging, tailings type, and stress state) that affect the tip resistance achieved in soils.

(d) Piezometers and settlement markers indicate behaviour similar to natural soils during the process of EC viz. the propagation of stress wave and the settlement of the order of 4%-10%.

These observations are encouraging for the potential use of EC in tailings. However, these observations are based on a very few cases and although the performance assessment of EC for these cases has been carried out using the conventional geotechnical testing tools, (such as CPT and SPT) it does not include the use of any of the existing material behaviour classification systems for data interpretation. Also, there are no published techniques to account for the site variability in the data interpretation.

It is important to address the above gaps for better performance assessment of EC and consequent increased use of EC on tailings dams. The case history presented in this thesis involves mainly the review of pre and post treatment (EC) CPT data using,

(a) The soil behaviour classification system developed specifically for CPT data interpretation in fine grained geo-materials including tailings (Jefferies & Davies, 1991; Plewes et al., 1992) and

(b) A simple statistical framework to account for the site variability for comparison of the pre and the post treatment CPT results, to determine, the achieved ground

improvement with regard to the factors affecting the assessment such as aging, tailings type and ground stress state.

Other data for the case include: borehole logs, piezometer data and settlement marker data. These will further aid the assessment of improvement and response of tailings to the treatment. It is expected that the results from this case will address the shortcomings cited in the review and will contribute to the relatively thin data base of cases of CPT data interpretation in tailings and CPT based assessment of ground improvement resulting due to EC in tailings.

3 DATA DESCRIPTION

The case history presented in this thesis involves the use of EC and surface compaction for densification of a tailings dam foundations at the Central Tailings Facility in Copper Cliff, Sudbury. This chapter presents the background information about the case history and the site data based on the Golder Associates Limited (Golder, 2002) report and the Golder Associates Innovative Applications (GAIA, 2004) report.

3.1 Site Details

The tailings dam site is located in R tailings area of International Nickel Company`s (INCO's) Central Tailings Facility in Copper Cliff, Sudbury, Ontario. The area covers about 5,500 acres (2,225 hectares). Tailings have been continuously deposited in the impoundment since 1936 and future tailings deposition is planned for at least another 20 years. The location coordinates for the site are: 48º 28' N and 81º 04' W. The current elevation of the site is 308m above mean sea level. Prior to the tailings disposal, the site consisted of lakes and (granitic) bedrock ridges with organic material overlying the bedrock. The tailings are predominantly pyrrhotite.

From the beginning of tailings disposal in this area to the present time, perimeter dams have been constructed using the upstream construction method. Under INCO's current plans, the currently active portion of the Central Tailings Area, the "R" tailings area, will be used for storage of tailings through the year 2026.

As shown in Figure 3-1, R area forms the western portion of the Copper Cliff Central Tailings Area. R area consists of four sub areas, labelled as: R1, R2, R3 and R4.

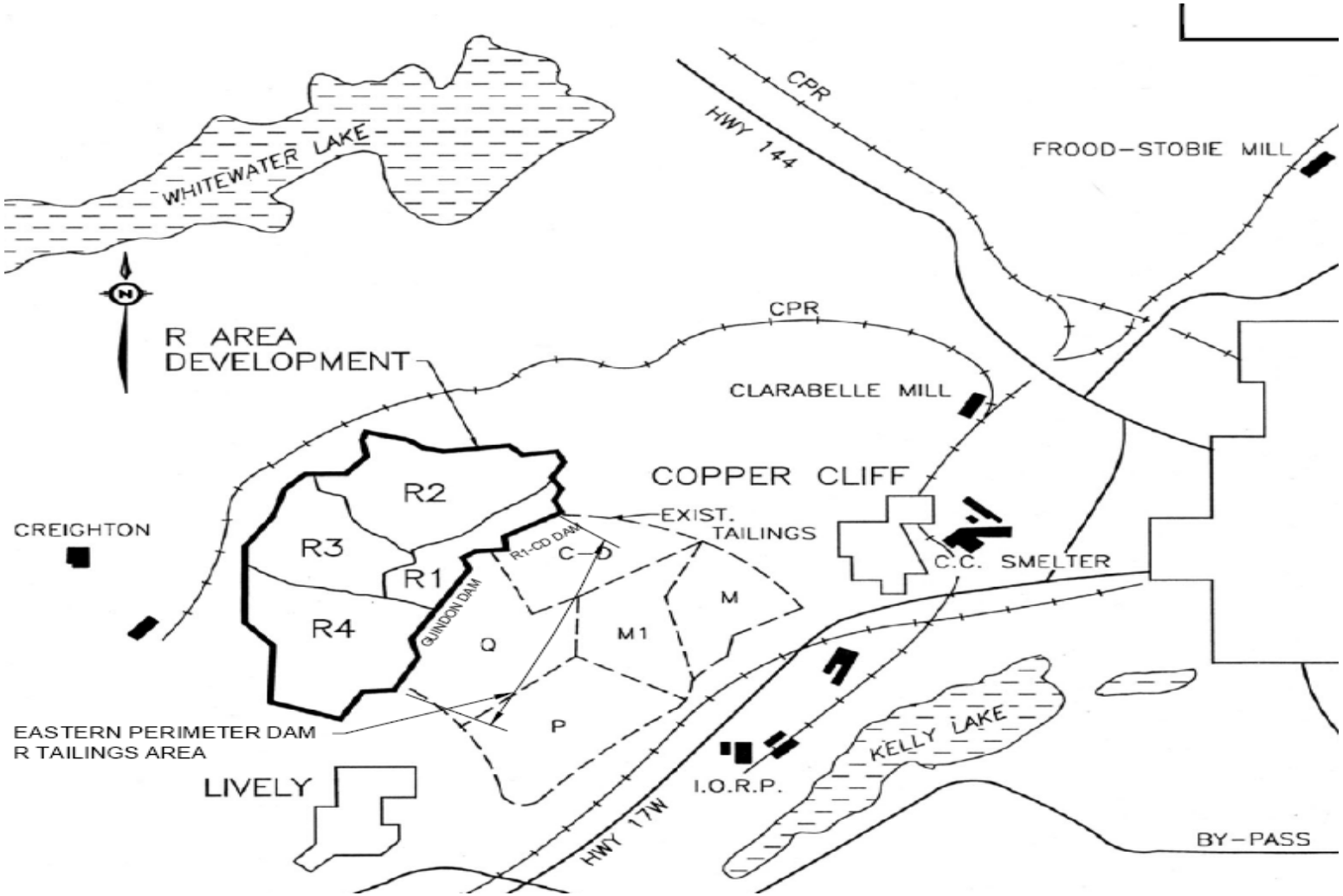


Figure 3-1: Sudbury Tailings Dam Site Location Plan (Golder, 2002)

Guindon Dam and R1-CD dam form the eastern limit of the R4 and R1 areas respectively and separate R area from the adjoining areas labelled as Q and CD areas as shown in Figure 3-1.

Figure 3-2 showing an aerial photograph of the dams, further complements the above site map. The existing foundations for these dams were considered for ground improvement.

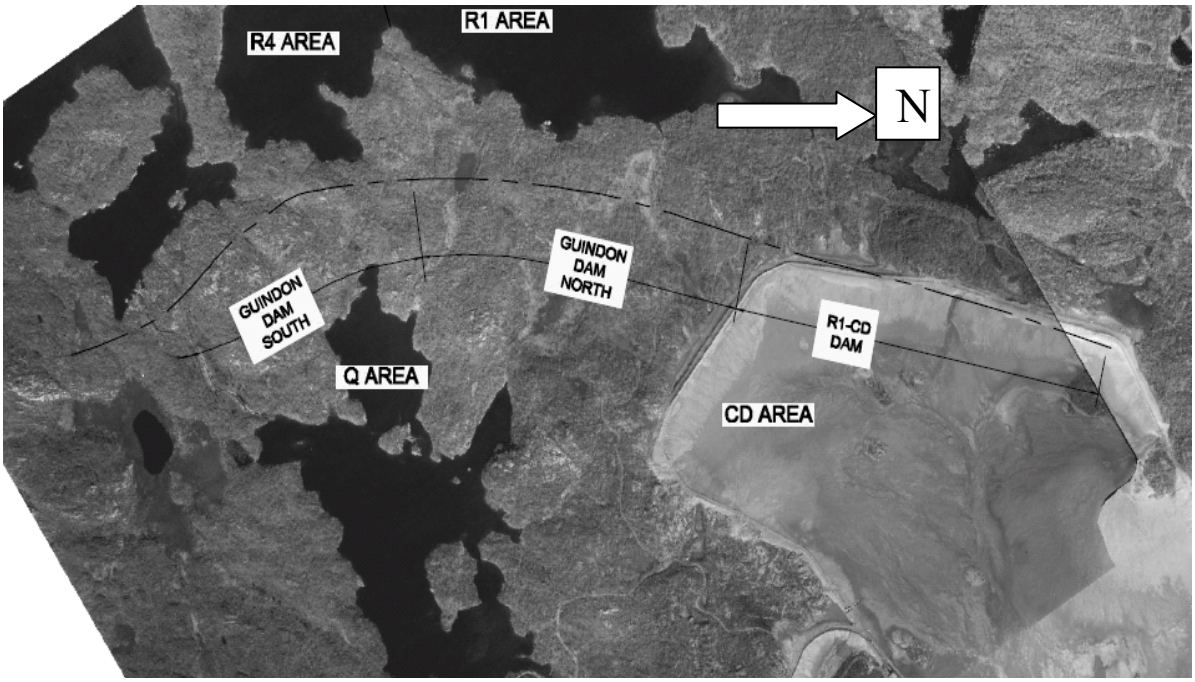


Figure 3-2: Sudbury Tailings Dam Site Aerial Photo (From Golder, 2002)

Guindon dam is approximately 1700 m long (as per 2002 records). It was constructed between 1960 and 1974 using the upstream method of construction. The previous geotechnical investigations (conducted before the year 2000), indicate that the dam was constructed using tailings. The available records on the construction of dam also show that no foundation preparation was undertaken prior to the construction of starter dams. R1-CD dam is approximately 1200 m long (as per 2002 records). It was constructed in the 1950s. This dam was built in the same manner as Guindon Dam, using tailings.

Tailings have been discharged in R area since 1987. To keep up with the filling of ponds in R areas, both the dams will be progressively raised with tailings to an ultimate crest

elevation of about 350m by 2030. These dam raises will be founded on deposited tailings that are loose, saturated, and susceptible to liquefaction as evident from the previous (pre 2000) geotechnical investigations. INCO contracted Golder in the year 2000 to provide recommendations for raising the Guindon and R1-CD dams. The results from that study are presented in the following section.

3.2 Results of Geotechnical Investigations: Golder, 2002 (2000, 2001)

Golder completed a preliminary and a detailed appraisal of options for raising the Guindon Dam in year the 2000. This required a detailed study of subsurface conditions of the Guindon and R1-CD dams and a review of previous investigations carried out on the dam site. They carried out two geotechnical investigations for this purpose in 2000 and 2001. The data collected from these investigations, process of data collection, the trends shown by the data, and their implications for raising the dam are briefly discussed in the following subsections.

3.2.1 Pre 2000 Geotechnical Investigations on Dam Site

Golder reviewed the records of pre 2000 field investigations carried out in the southern part of the Guindon Dam from 1972 through to 1998. The pre 2000 investigations concluded that the tailings within and upstream of Guindon South were in a loose to compact state of packing based on the SPT N values (N ranging from 4 to 30).

3.2.2 Details of 2000 and 2001 Geotechnical Investigations

The intent of these investigations was to characterize the dam foundation and assess the susceptibility of ground to liquefaction due to an earthquake. The year 2000 investigation was conducted within the limit of Guindon Dam, while the 2001 investigation included the R1-CD dam area. The investigations primarily involved conducting piezo-cone penetration tests (CPTU) and boreholes. The 2001 investigation also included dynamic cone penetration tests (DCPT) and geophysical investigations in the form of a seismic refraction survey and cross-hole shear wave velocity measurements which were used to estimate the depth to

bedrock. Additionally, in-situ permeability tests were performed using the falling head method.

3.2.3 Summary of Trends from Golder 2000/2001 Investigations

The key findings from the investigations are summarized below:

(a) Presence of tailings deposited up to a depth of 20 m below the ground level was indicated.

(b) The deposited tailings were heterogeneous, consisting of silt and fine sand with some silt to silty sand.

(c) The tailings were saturated and in a very loose to compact state of packing (based on the SPT N values measured during sampling). Relative density of the tailings was judged to vary from 40-60 percent.

(d) Average water content of 28% was found for the saturated tailings. Specific gravity of about 3 was measured and the void ratio ranged from about 0.6 to 1.3 (average 0.85).

(e) Lenses of low permeability silt sized tailings existed within the coarser tailings.

Based on these trends, Golder concluded that the tailings foundation would liquefy during the design earthquake event. A range of options to prevent instability within the dam were suggested. Possible remedial measures considered were:

- Adopting a very gentle slope
- Constructing a massive toe berm and
- Improving the tailings foundation to eliminate the potential for liquefaction.

After reviewing a number of available ground improvement methods, INCO, the mine owner, contracted GAIA to improve the tailing foundations. The following section deals with the ground improvement work carried out by GAIA (2004) and provide the basis for the analysis carried out in this thesis.

3.3 Ground Improvement Implementation Plan & Data Assessment

Two test blast sections were carried out to determine the suitability of the method on this site. Based on the results of these test sections, INCO decided to use EC for densification of the deeper tailings deposits, and surface compaction in the form of dynamic compaction (DC) and rapid impact compaction (RIC) for the shallower zones, to achieve the required design. INCO contracted GAIA to densify a 30 m wide and about 1800 m long zone along a proposed dam raise alignment within the Guindon Dam tailings area, and a 1200 m long zone within the R1-CD dam tailings area. Figure 3-3 shows a site map indicating the GAIA densification work zone (area bound by broken lines) for the Guindon Dam north and R1-CD dam. GAIA adopted a detailed work plan which involved a pre and a post treatment geotechnical site investigation and execution of the design treatment on the site. The data obtained from this work plan are summarized in the following section.

3.3.1 GAIA Work Plan

In order to execute the work plan, the Guindon and R1-CD dam alignments (Figure 3-3) were divided into panels for progress tracking and scheduling purposes. Areas to be treated were divided into panels of about 36.6m (120 feet) by 36.6m (120 feet).

(a) Carrying out test blasts: Test blasts were carried out to provide information for optimization of the charge pattern, spacing of charges, and delays in between the detonations for different locations on site. The final design consisted of an overlapping, two staged pattern. Charge weights increased with depth. The design pattern is discussed in detail in GAIA (2004) report.

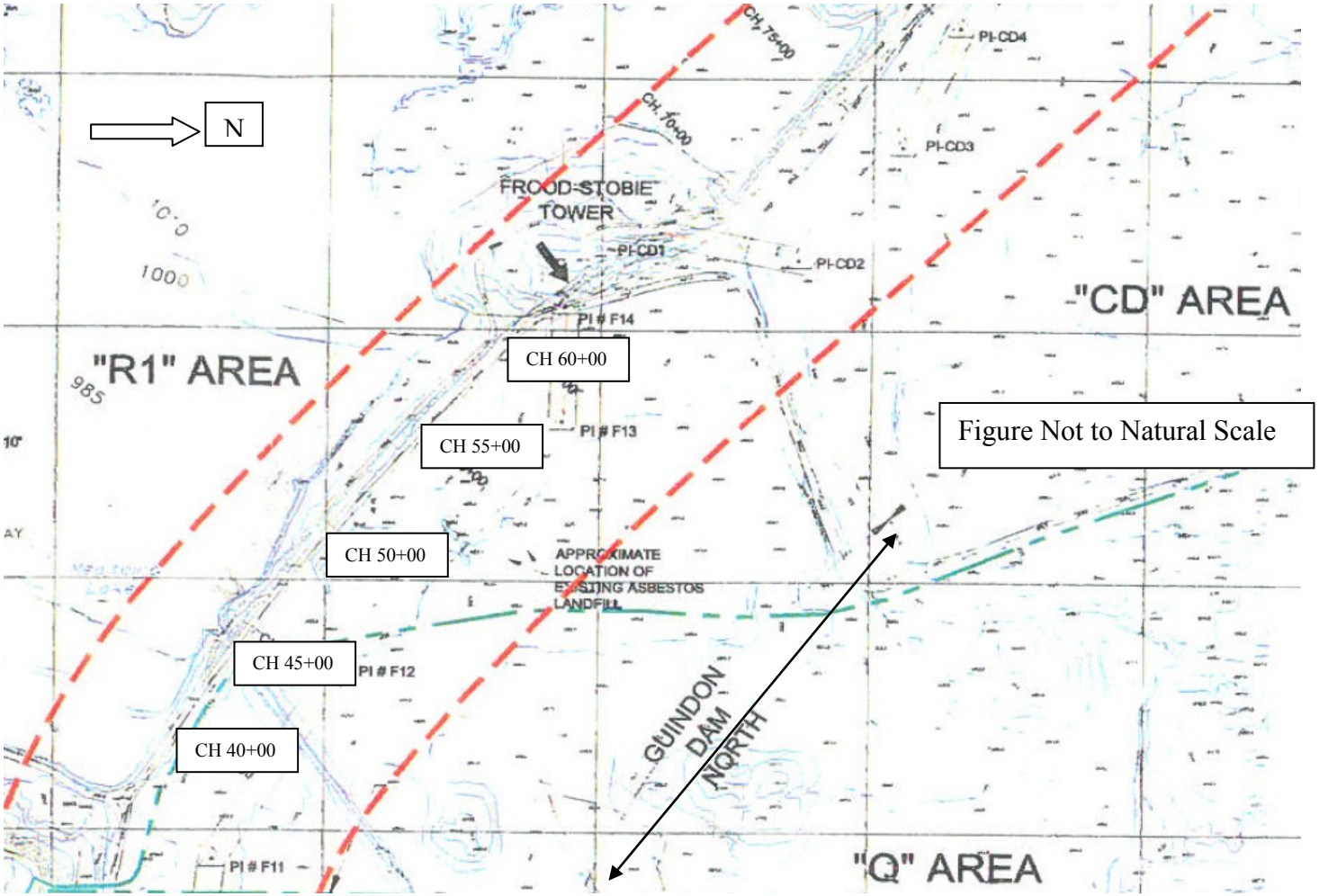


Figure 3-3: GAIA Work Zone (GAIA, 2004)

(b) A subsurface investigation before and after the design densification in the area indicated by INCO on site: The intention of this investigation was to augment the 2000/2001 Golder geotechnical investigation data made available by INCO to optimize and verify the performance of the improvement methods during the testing phase, and to provide confirmation of improvement achieved after the treatment. Subsurface investigations involved the following:

- A series of 41 CPTU conducted prior to the treatment (from 26th February 2004 to 13th May 2004) along the Guindon alignment spaced at about 45 m on alternating sides of the alignment.

- Following the densification of each panel, confirmatory CPT were carried out at random locations in each panel. 170 CPT were conducted in Guindon and 50 CPT in R1-CD. These CPT were carried out at different times after the treatment at different locations within the panels. Refer to Appendix E for the schedule of post treatment CPT data available at different times after the treatment in each panel.

- The CPTU system that was used for the project is manufactured by Geotech System of Sweden. The CPT probe is an Acoustic probe. The design details of the probe are listed in detail on www.geo.se. The net area ratio "a" required for correction for unequal end area effects is equal to 0.81 for this cone (Jefferies, 2006). This correction is explained in Chapter 2 in section 2.2.2.4.2.

- Nine boreholes were drilled in various panels after the treatment (from 26th August 2004 to 2nd September 2004). The borehole logs show SPT blow count "N" at different depths, results of grain size analysis such as fines content (% material passing # 200 ASTM Sieve) for samples obtained at different depths, and the readings from standpipe piezometers. Boring was carried out using the hollow stem power auger (4.25 inch internal diameter). SPT was conducted by raising a 140 lb ram using both rope and pulley system (safety hammer) and an automatic trip hammer system.

(c) Implementation of the prototype ground improvement treatment: This included employing EC and RIC/DC on site. The following data regarding the full scale treatment are available:

- Ground improvement was mainly carried out using EC, RIC and or DC either separately or in combination to improve the foundation tailings. These treatments were applied to 54 panels in the Guindon Dam and R1-CD dam areas.

- DC was applied to 41 panels (partially or wholly) either as the only method of compaction or in conjunction with the blast densification. DC was carried out from mid-June to early-October, 2004.

- Explosive compaction was carried out from June 18 to August 20, 2004 in 29 panels along the Guindon Dam alignment and across 4 panels along the R1-CD dam. For each panel, a schedule showing the details of the treatment is given in the following section.

- Vibrating Wire type piezometers were used to monitor the pore pressures generated as a result of the treatment. They were installed at a depth of 9m below the ground level.

- Sondex settlement markers were used to measure the settlements on site after the treatment. These are tubes used to monitor settlement with depth.

The GAIA (2004) report along with the previous geotechnical investigation data (Golder, 2002) were made available to the Geotechnical Group at the University of British Columbia in Vancouver in May 2006. The GAIA (2004) data being more current, are expected to reflect the latest operational features and activities on the tailings dams, and hence are used exclusively in this study. The following section describes the GAIA (2004) dataset given to UBC in detail.

3.3.2 Schedule of GAIA Investigation Data for Each Panel

Figure 3-4 below shows a typical layout of panels from a section of the Guindon Dam. Also seen are the panel numbers, chainage for the alignment, and other details already explained in earlier section. For a typical panel, Panel 51, from the northern portion of the Guindon Dam, the locations of the pre and the post treatment CPT data, borehole log, settlement marker, and a piezometer are shown.

The details of primary and the secondary blasts are also shown. Table 3-1 summarizes the total number of panels treated with EC in Guindon and R1-CD area, total pre and the post treatment CPT, and boreholes conducted in all of these panels as well as total number of piezometers and settlement markers employed in all the panels.

3.3.3 Data Assessment

Appendix D deals with the assessment and processing of CPT data. This section deals with assessment of settlement and piezometer data. Data from the northern part of Guindon dam have been considered. Settlement was measured using the Sondex deep settlement gauges installed in different panels close to or on the center line of the dam. Data from the settlement gauges located in the Northern part of Guindon dam are considered in this section. Total seven panels (Nos. 40, 43, 44, 45, 47, 49, and 51) had the gauges installed for measuring the post EC and RIC/DC settlements at different depths.

Deep settlement gauges typically consist of a hollow PVC pipe with concentric rings located around it at a fixed interval below ground level. Compaction results in generation of compressive strains in the ground in between the rings. These strains are calculated from the displacement of the rings. The raw data typically consist of positions of rings before and after the treatment. Table 3-2 below shows the raw settlement data for an example Panel 40.

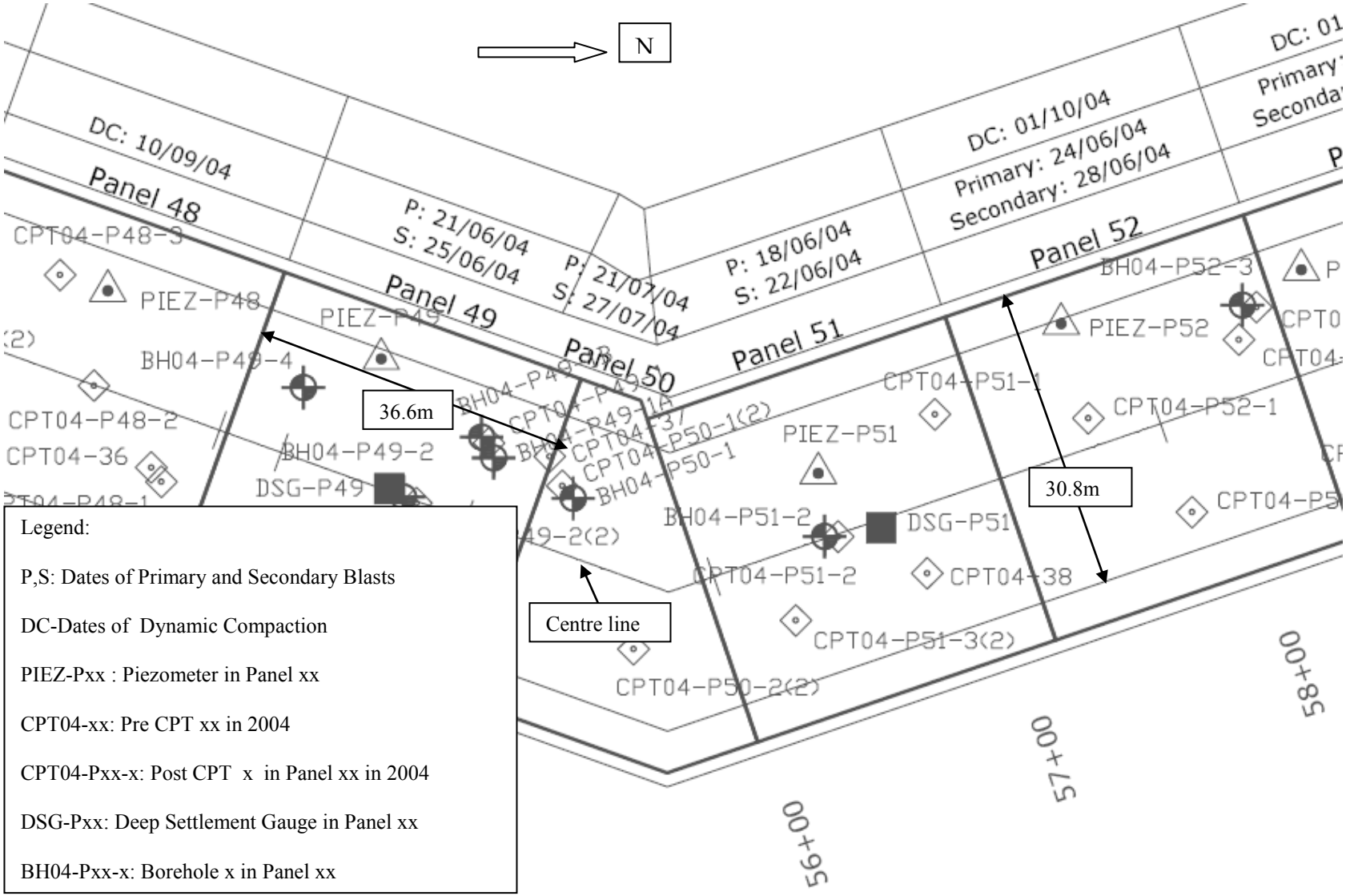


Figure 3-4: Arrangements of Panels

Table 3-1: Summary of Panel Wise Schedule of Guindon and R1-CD Dam

Total Panels Treated with EC	Total Boreholes Available	Total Pre Treatment CPT	Total Post Treatment CPT available	Total Piezometers Employed	Total settlement gauges	Total grain size curves available
33	9	22	200	27	15	100

Table 3-2: Sample Settlement Data (Panel 40)

		Depth of Rings from top of PVC Pipe (feet)						
Day	**Time**	**No.1**	**No. 2**	**No. 3**	**No. 4**	**No. 5**	**No. 6**	**Bottom**
July 9/04	13:35	15.97	21.95	27.40	32.38	37.38	42.28	43.26
July27/04	8:07	16.32	22.08	27.43	32.44	37.43	42.29	43.26

The following is observed with respect to the utility of the settlement data for analysis:

- The data showing the position of concentric rings is missing in many panels for different time periods after the treatment, and hence that dataset cannot be used.
- The hollow PVC pipe is reported to be broken due to impact of blast in number of panels and hence measurements were started from a new reference. This reference point is not reported for many panels.

Thus, out of seven panels, it was found that only two panels, 47 and 49, record settlements after both stages of EC and after RIC. Figure 3-5 shows the location of these settlement gauges on Panels 47 and 49 with chainage 52-55. Refer Figure 3-3 for the location of the panels with chainage 50-55 on the main site map. GAIA (2004) had estimated RIC to be effective up to about 7m below ground level. It is expected that settlement up to this depth would be caused by the application of combination of EC and RIC and would be greater than that expected to occur below this depth for similar soil type and relative density. Figure 3-6 shows the variation of percent compressive strains generated due to the application of EC and RIC for the two panels, 47 and 49, plotted at the mid points of the depth intervals.

The following can be observed from Figure 3-6:

(a) There is no clear effect of RIC on the settlement in 7m zone. Settlement due to EC and RIC combined, is only marginally higher than the settlement due to EC alone at this depth.

(b) The settlement profile is not uniform; settlement is increasing with depth.

(c) Panel 47 shows a lower range of settlement, 4%-10%, compared to the Panel 49, which shows settlement as high as 18% at a depth of about 11m. This value is higher than the typical value (10%).The higher range shown by Panel 49 could be due to the factors such as presence of loose material with very low relative density before the treatment, or proximity of the settlement gauge to an explosive charge, or combination of all of these factors.

(d) The settlement is short term viz. measured immediately after the treatment.

The range of ground settlement observed for Panel 47 at Guindon dam is similar to that reported in the literature for tailings (up to 10%, Gohl et.al. 2001a) and in natural soils (4%-10%, Gohl et.al. 2000). More settlement data with the gauges closely spaced to the borehole logs are required to determine the average settlement and relate the settlement achieved to the factors such as treatment design, material type and initial relative density of the layer treated by EC. Measurement of initial density is desired. This parameter is not available in the GAIA (2004) data.

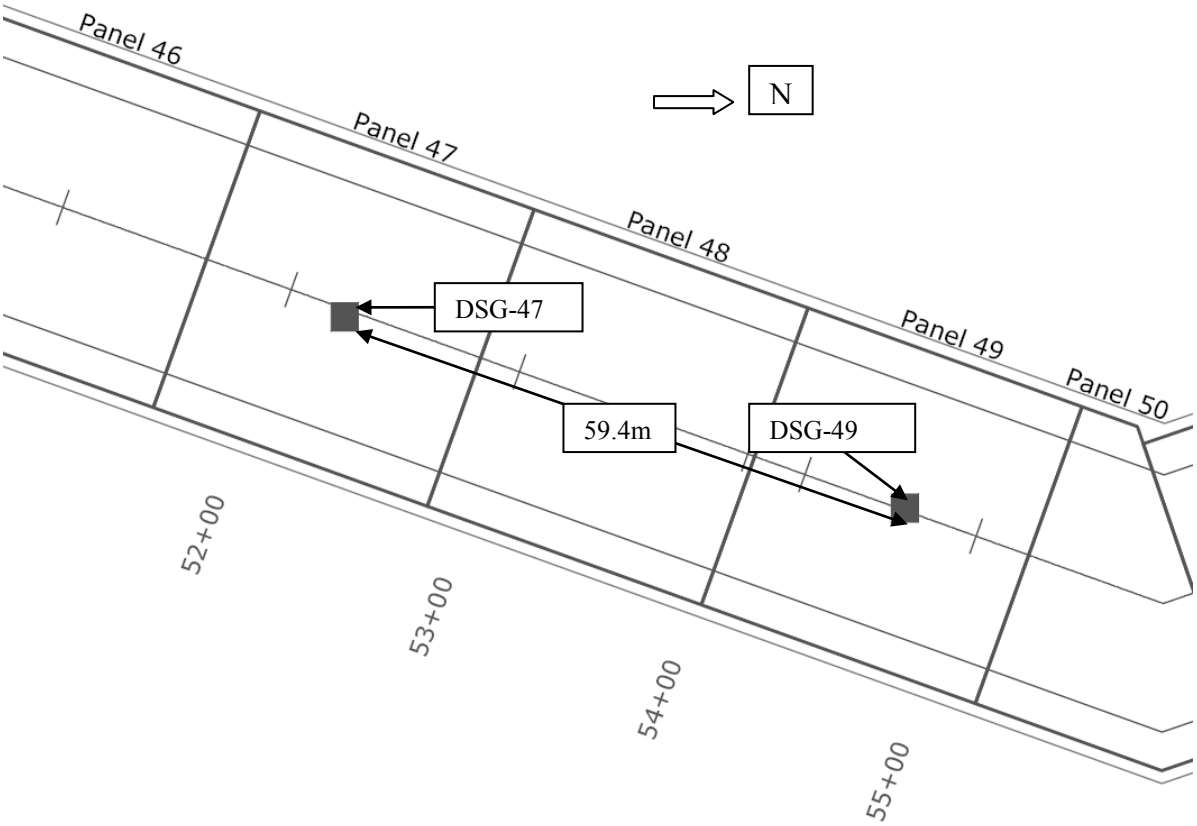


Figure 3-5: Location of Panels with Settlement Gauges on Guindon Dam

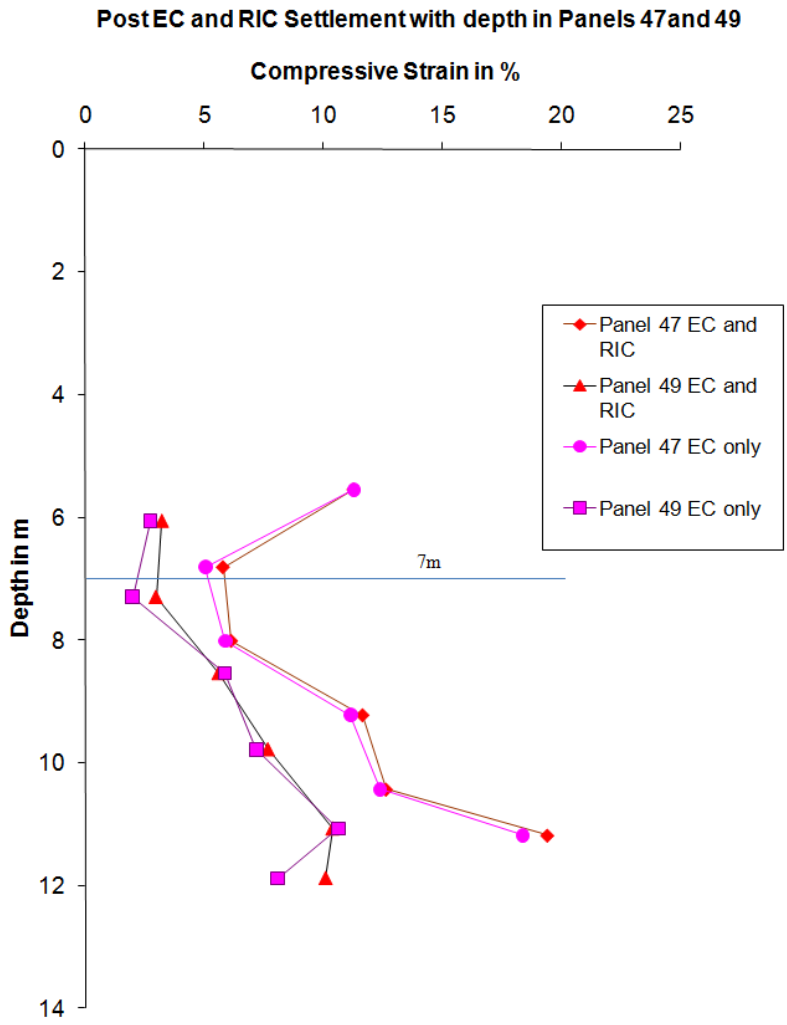


Figure 3-6: Settlement Profile for Application of EC and RIC for Panels 47, 49

3.3.4 Response of Piezometers

Piezometers are used on EC projects to measure the instantaneous rise in pore water pressures, resulting from the detonation of explosive charges, and also, the process of dissipation following the detonation. The role of piezometers on EC projects and the mechanism behind the response captured by a piezometer have been reviewed in detail in Chapter 2 in Section 2.3.3.

Vibrating wire type piezometers with RST data loggers (CR10X) were used on Guindon Dam to record pore pressure response following the blast. The data logger reads a data point (sample) at intervals ranging from of 2 to 15 seconds. Total fourteen piezometers across Panels 40 to 53 were used (Panels in the arbitrarily chosen northern section of the Guindon dam). These were installed at a depth of 9m below ground level. Each piezometer mainly records the pore pressure response after primary and secondary blast in the panel in which it is located. Some piezometers have been used to monitor the pore pressure response of the blast that occurred in adjacent panels. Figure 3-7 shows a segment of panels with locations of piezometers in these panels within chainage 43 and 48. Refer Figure 3-3 for the area within chainage 45 to 50 on the main map. Two typical plots from the data set are shown below. Figure 3-8 shows the response of piezometer in Panel 40 to the secondary blast in Panel 40 and 41. Figure 3-9 shows the response of piezometer in Panel 41 to the secondary blast in that panel. Start times given in both the plots refer to the start time of the data logger/data acquisition system as per the raw data. The timing of the blast/detonation in that panel is not known. Table 3-3 below lists initial hydrostatic stress, peak pore pressure, time required to attain peak pore pressure and pore pressure at dissipation from the two Figures. The location of the piezometers with respect to the nearest charge hole was determined from the GAIA (2004) charge hole layout. This layout shows the locations of the charge holes for the primary and the secondary phase of the treatment for all the panels treated with EC in Guindon dam. The final design consisted of square grid overlapping pattern of primary and secondary charges. Each charge hole near the piezometers considered in this chapter consisted of multiple decks of charges.

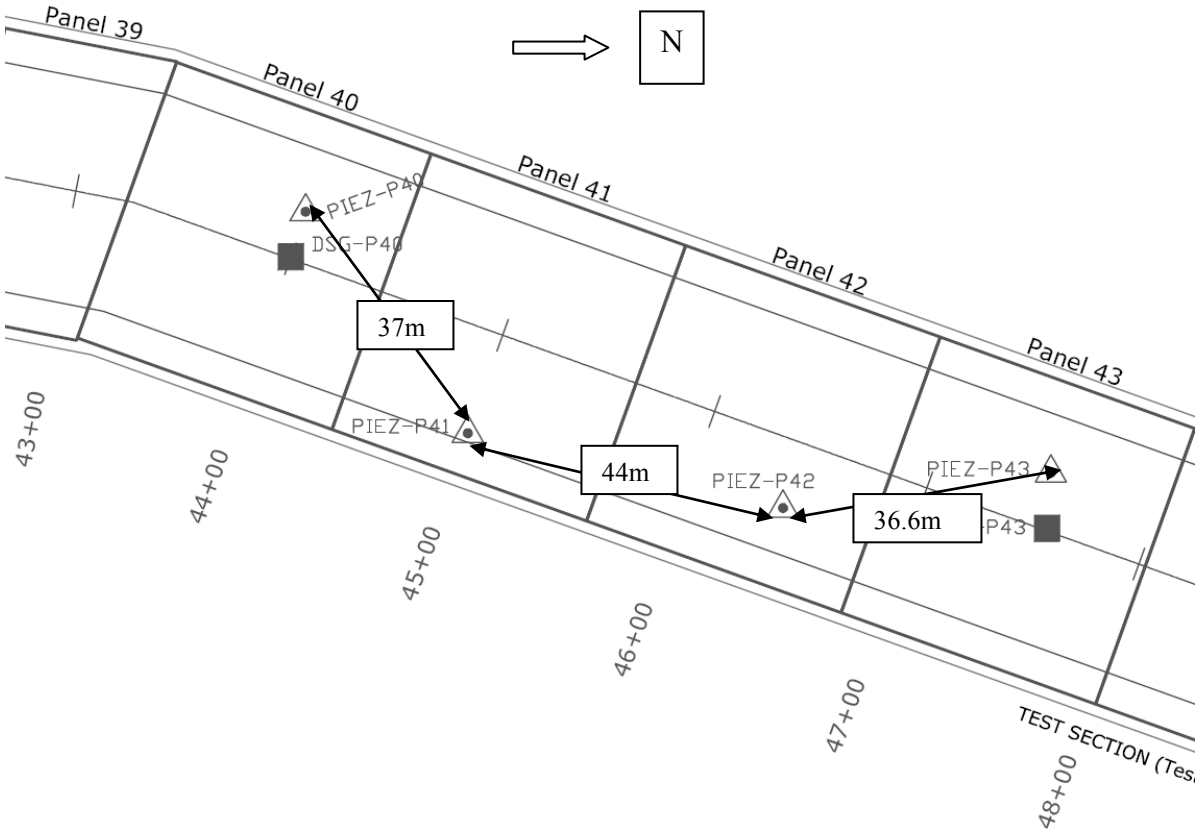


Figure 3-7: Locations of Piezometers for an Example Segment

Following observations are made from the response of piezometers for the given blast design:

(a) Slow speed piezometers were used to measure the residual pore pressure response for the Sudbury project. A number of investigators (Gandhi et al., 1999; Gohl et al., 2001a&b) have recorded a transient hydrodynamic pore pressure response along with the build up of residual pore pressures for the type of multiple blast design used in Sudbury. However, these piezometers do not show the dynamic component. Gohl et al. (2001b) show the difference between the response captured by high and low speed piezometers. The response shown by the slow speed system is similar to that observed for the present case.

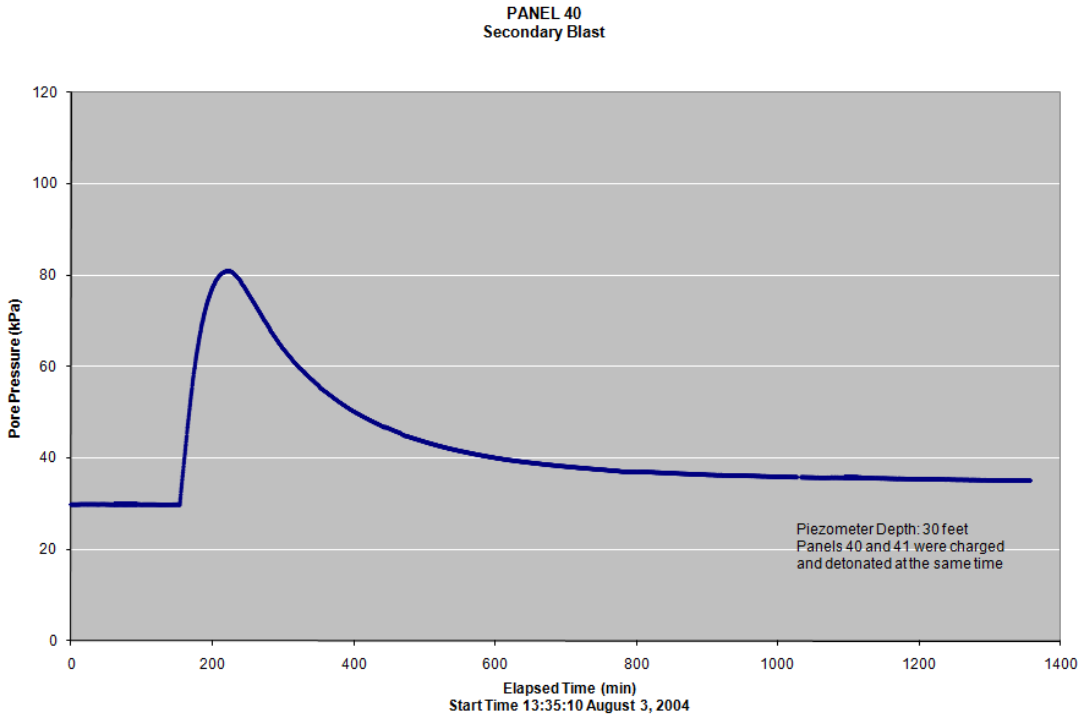


Figure 3-8: Pore Pressure vs. Elapsed Time (Secondary Blast for Panel 40)

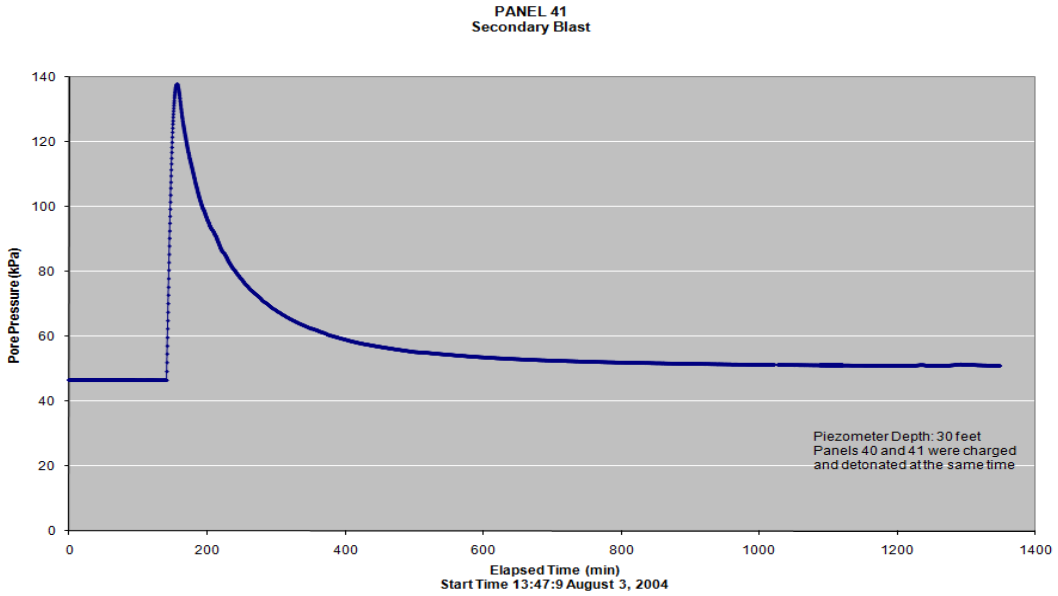


Figure 3-9: Pore Pressure vs. Elapsed Time (Secondary Blast for Panel 41)

Table 3-3: Data from Piezometer Plots

Panel No.	Hydrostatic stress (kPa)	Peak pore pressure (kPa)	Time to attain peak from the detonation (min)	Dissipation pore pressure (kPa)	Radial distance from nearest charge hole (m)
40	30	80	68	35	2.5
41	46.7	137.5	15	51	3.3

(b) Figure 3-8 shows a slower rise in pore pressure than Figure 3-9 (see Table 3-3, time required to attain peak). Handford (1988) recorded pore pressure response similar to that observed for Figure 3-8, for a piezometer outside the radius of influence of the blast, citing the effect of recording the pore pressure migrated away from the blast point. For the present case, similar observations can be made. However, given that the secondary charges in both the panels were detonated at the same time, a similar response for the two piezometers was expected.

(c) Figure 3-9 indicates a maximum pore pressure ratio (PPR) of 0.76, indicating partial liquefaction of the ground achieved due to blasting. The residual excess pore pressure for the both the piezometers is the same viz. 5kPa (difference between dissipation pore pressure and the hydrostatic stress indicated in Table 3-3). A higher peak and dissipation pore pressure is observed for piezometer in Panel 41.

(d) A number of piezometer observations were discarded due to the damage occurred to the measuring units during the blasting process.

It is observed that piezometer data are affected by the factors such as measuring system (slow speed system), variability in soil type on the site, few measuring units in a panel and the quality of the instrumentation. The data cannot be used for studying the effect of multiple charge hole design, number of decks of charges used etc. on the response of ground.

3.3.5 Discussion of Results

(a) Overall, these results point towards a typical performance of EC in Sudbury Tailings. Treatment is followed by release of pore water pressure, settlement (4%-15%) and piezometer response indicating a rise in pore water pressure as a result of blasting, dissipation and build up of residual pore pressures.

(b) However, the data suffer from a number of discrepancies associated with quality of instrumentation (speed of DAS, lack of protection of instrumentation during blasting etc.) and serve only as qualitative indicators of the ground improvement.

4 USE OF CPT DATA: ANALYSIS & DISCUSSION OF RESULTS

In this chapter, the subsurface investigation data obtained from the chosen northern section of the Guindon dam from the GAIA (2004) work plan, viz. the pre and the post treatment CPT data and borehole logs along with the standard penetration test data (SPT), have been reviewed to characterize the site before and after the treatment to determine the improvement achieved at the site in terms of CPT tip resistance for different tailings/soil types. Information from the borehole logs and the processed CPT data have been used to characterize the pre and the post treatment ground in Section 4.1. The results of the process of site characterization, their implications on the use and representation of the test data for determining the achieved tip resistance as well as consideration of factors typically affecting tip resistance, viz. aging, tailings type etc. have been dealt with and the results have been discussed in the ensuing sections of this chapter.

4.1 Data Screening: Trends in Soil Type and Soil Behaviour Type

The objective of data screening is to establish the general subsurface profile at the site including the location of the ground water table, and the thicknesses of zones of different tailings/soil types such as sand and silts that may exist on the site. Use of borehole logs and the CPT data in this regard is discussed below.

4.1.1 Screening of Borehole Log Data

The northern portion of the Guindon Dam includes fifteen Panels (numbered from 40 to 54). Six boreholes are variably spaced within these 15 panels. These boreholes were conducted after the treatment. Figure 4-1 shows the locations of boreholes from three example panels within chainage 45 to 49 in the Guindon Dam north. Refer Figure 3-3 for the zone within chainage 45 to 50 for location of these panels on the main map.

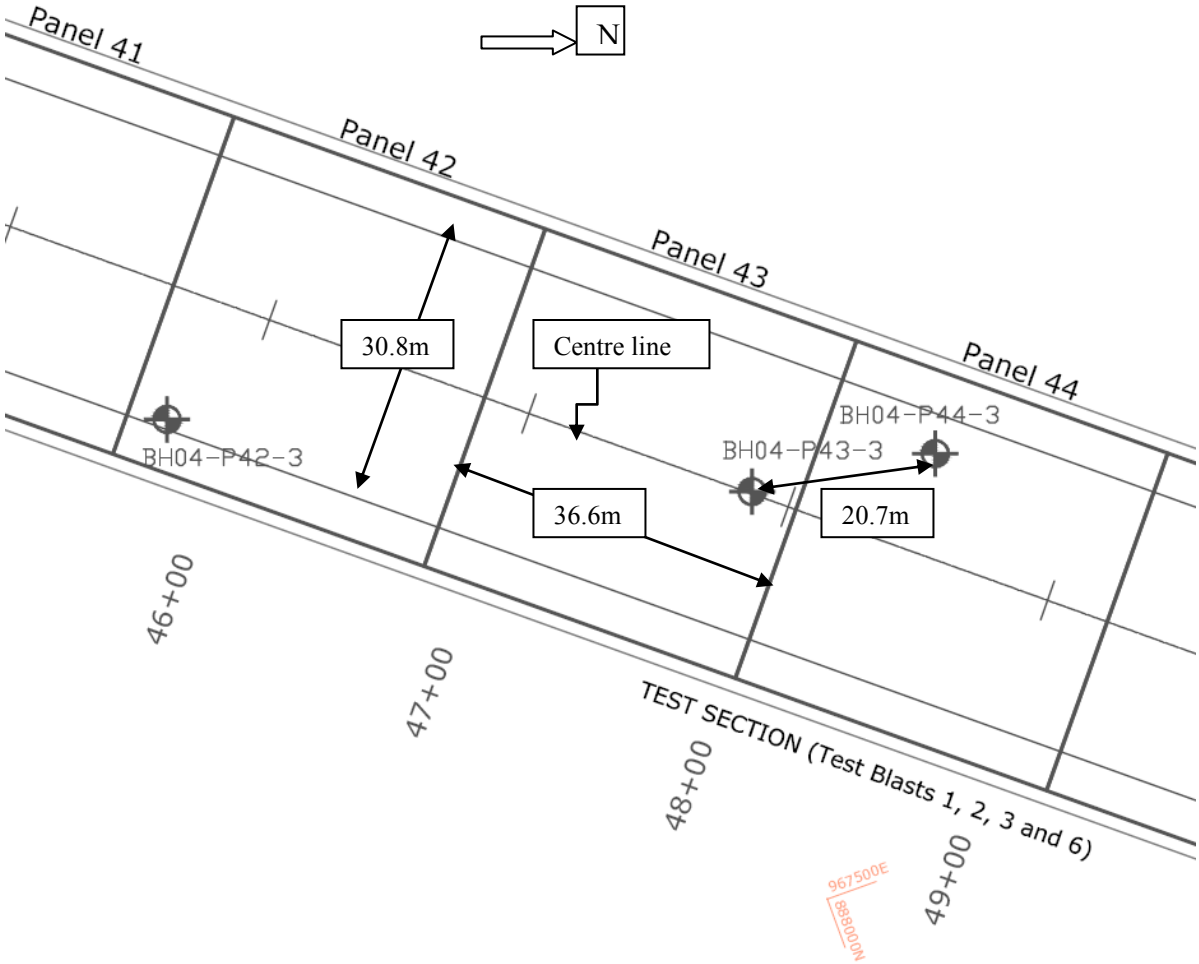


Figure 4-1: Locations of Boreholes BH-42, 43 & 44 in Guindon Dam North

The available six borehole logs are listed in Appendix A along with the ground water table level details. A comparison of the logs indicates the following:

- The boreholes in general are drilled up to a refusal depth ranging from 12m to 19m.
- The ground water table is located as indicated by the standpipe piezometer reading in the borehole log, at a depth ranging from approximately 4.3m-5.5m, below the ground level.
- Four out of the six listed logs, indicate the presence of rock fill approximately 1.5m in thickness immediately below the ground level.

- This is underlain by zones of predominantly sand and silt sized tailings. These zones are further characterized by varying presence of clay sized tailings, and traces and seams of natural soils such as silts and clays at different depths.

- The thicknesses and locations of these zones vary considerably in each borehole.

- Organic soil, namely peat, is generally found in the bottom metre or so of the borehole.

- There is considerable variability on site in terms of the extents of zones of these tailings/soil types and no trends are found in this regard.

These observations are further supported by varying standard penetration test (SPT) blow counts measured at different depths below the ground level, and fines content (percent fraction of the sample passing No. 200 ASTM Sieve Size).

Figure 4-2 shows the variation of fines content with depth for a set of three boreholes located in consecutive panels.

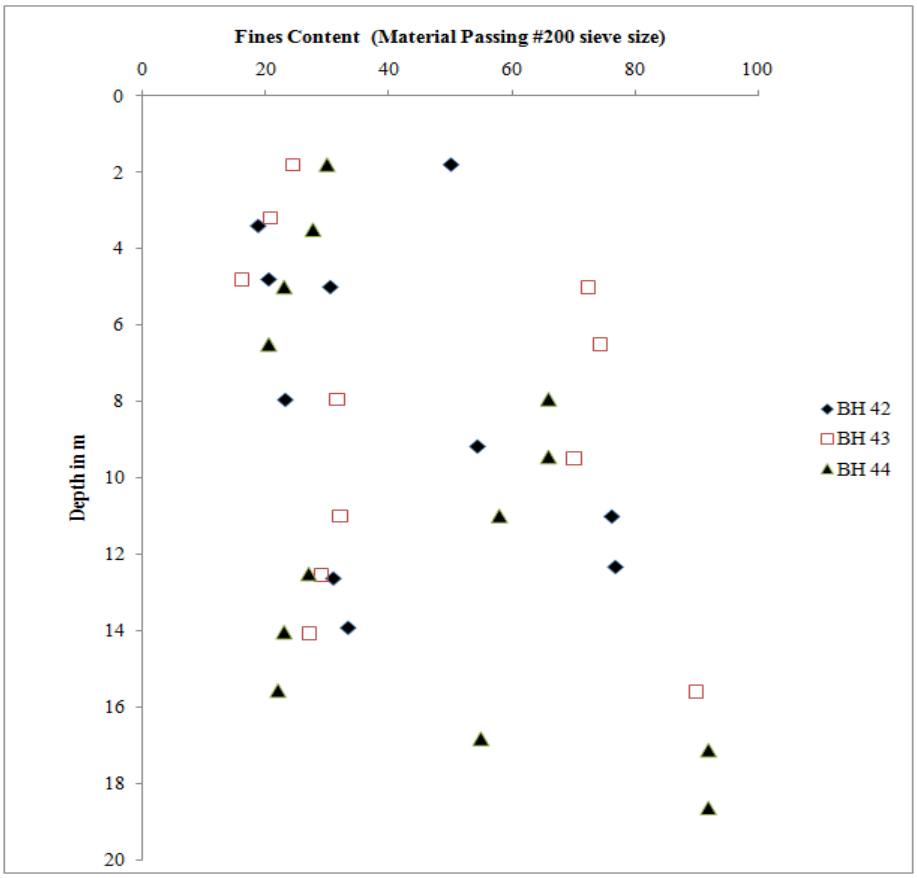


Figure 4-2: Variation of Fines with Depth for Guindon Dam North (Panels 42, 43 & 44)

The Figure shows that except for a few depth intervals (at 3m, 9m and 13m), there is a wide variation in fines content for a set of boreholes at similar depth intervals. Figure 4-3 shows similar variation of measured SPT blow counts (N), normalized to 60% energy ratio N_{60} as per Skempton (1986). Safety hammer with 2 turns of rope (55% rod energy ratio), driving a standard sampler with a rod length greater than 10m in a borehole of 107mm diameter were assumed for sets of consecutive panels in Guindon Dam North. Therefore, there is another source of variability in the N_{60} values in the form of uncertainty in the rod energy ratio.

To summarize, a comparison of borehole logs, SPT data, and fines content for the available boreholes showed no trends in terms of the location, and the thicknesses of zones of soil types on the site. Presence of sand and silt sized tailings was observed along with natural soils such as traces of clays, silts, gravels, and deposits of organic material at greater depths. However, no definite zones (having definite extents) of either material were observed.

The observations from the borehole data have limitations, as only six available boreholes from northern part of the Guindon dam were compared in this analysis. Large amounts of CPT data are however, available for this site and hence these data will be used for subsurface characterization of the ground. Since CPT indicates the soil behaviour type, which is known to change with ground improvement (Davies, 1999; Amini et.al. 2002), the pre treatment CPT data will be used first to establish the subsurface profile.

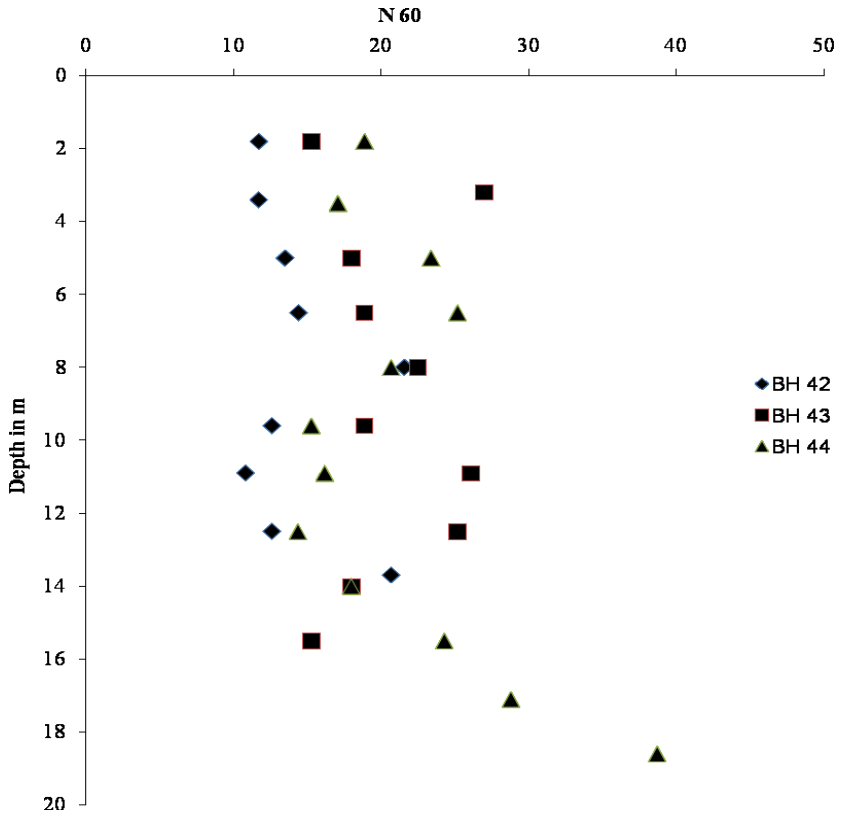


Figure 4-3: Variation of Normalized SPT Data with Depth for Guindon Dam North (Panels 42, 43 & 44)

4.1.2 Screening of the Pre-treatment CPT Data

All the Pre CPT in all of the panels have been used for determining trends of pre treatment behaviour on site. Pre CPT are located on either side of the centre line along the Guindon dam. Figure 4-4 shows the positioning of the pre CPT data with respect to the centre line for Panels 41-44. The details of the schedule of pre treatment CPT data (period during which

they were carried out on the site and their location on site), raw CPT data, and the process of correcting the raw data for various CPT specific corrections have been summarized in Appendix D.

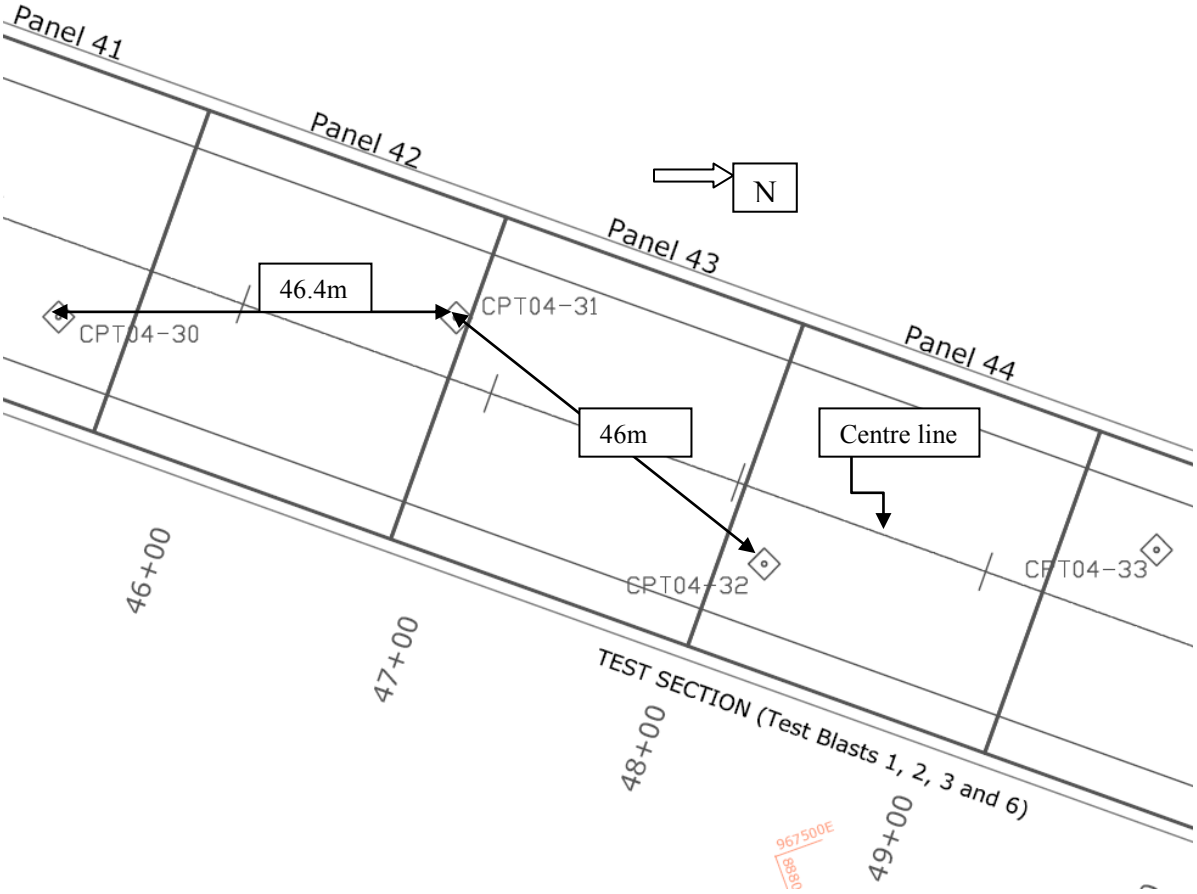


Figure 4-4: Typical Arrangement of Pre CPT Data in Guindon Dam North

The interpretation of soil behaviour type (SBT) from the measured CPT data is based on the visual inspection of the variation of CPT parameters with depth. Figure 4-5 shows the variation of CPT parameters and the interpreted soil behaviour type for a pre treatment CPT, CPT04-29, located in Panel 40. The process of interpretation of SBT is as follows:

- The water table is inferred at about 311m, as seen from the variation of pore pressure with elevation.

- The zone from ground surface to the water table shows the presence of high tip resistance material, (q_t ranging from 10-20 MPa and Q >200) indicating gravelly sand with layers of fine sand at elevations of 310.5m, 313m and 314m.

- This is underlain by about a 2m thick zone showing lower tip resistance, (5-10 MPa) and occasional negative pore pressures indicating the presence of silt layers.

- This is further underlain by increasing tip resistance, hydrostatic conditions, and lower friction ratios, likely indicating clean sand with lower silt content.

- At the bottom, the presence of sand to silty sand is indicated by lower tip resistance, and increasing friction ratio.

Similarly, the soil behaviour types have been interpreted for other pre treatment CPT, and the interpreted profiles are shown in Appendix B. The details such as ground surface elevation, depth from ground level, inferred water table location, and the panel number for each CPT are also presented in Appendix B.

A comparison of all the variations of the measured CPT parameters generally indicates a top zone of fill material underlain by dense gravelly sand to silty sand, (characterized by high tip resistance, 5-15 MPa, hydrostatic conditions, and low friction ratios) followed by a zone with increasing silt and clay content with thinner sand layers, (characterized by lower range of tip resistance, 0-5 MPa, friction ratio 6-7%, and high excess pore water pressures).

This is, however, a highly generalized description as the thicknesses and the material behaviour types for each zone vary considerably along the center line for each CPT, and it is difficult to identify an exact trend of soil behaviour type with depth, and along the center line. Some panels show the presence of silty clay to clayey material along with organic material at greater depths. Figure 4-6 shows all the pre treatment CPT parameters (measured and inferred) together, and indicates the variability in SBT at all the depth levels on the site.

It is also observed from Figure 4-6, that there is a marked reduction in normalized tip resistance below the water table approximately at 311m, possibly, indicating the loose deposition of the tailings below water table. In general, the behaviour type appears to get

finer with depth as apparent mainly from the increasing values of normalized friction ratio F, and measured pore pressure U_2. Normalized pore pressure parameter B_q also shows a maximum value of 0.4 (below the elevation of 305m), but not all the CPT indicate this trend. Observations from the pre CPT data screening are summarized below:

(a) The pre CPT do not give conclusive evidence of a pattern of soil behaviour type with depth or along the center line for the Northern Section of Guindon Dam, apart from a general observation that the SBT becomes finer with depth.

(b) Loose or finer material is apparent from the marked reduction in tip resistance below the water table. Majority of (post treatment) borehole logs show presence of compact sand and silt sized tailings, indicating presence of mainly loose material before the treatment. Some borehole logs also show presence of natural silt and clayey soils in this depth interval.

There is only one pre treatment CPT in each panel, giving an average distance between these CPT of about 45m. However, when the post treatment CPT data are included, it is possible to identify a number of closely spaced post treatment CPT. It is proposed to use the closely spaced post treatment CPT data or clusters to identify trends in SBT on site. Comparison of SBT interpreted from the clusters of closely spaced data is expected to indicate trends in subsurface profile for those localized zones.

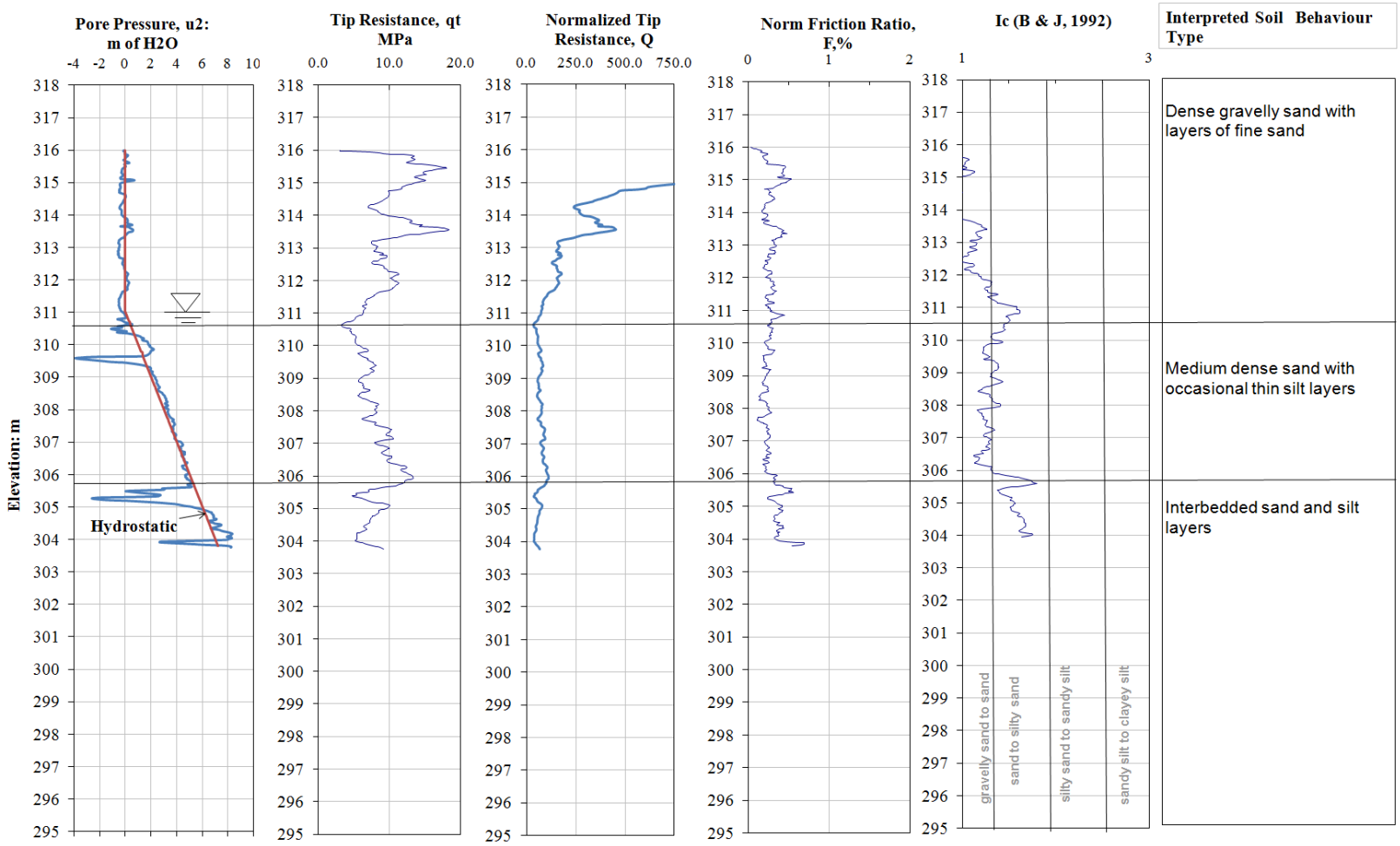


Figure 4-5: Pre CPT -29 Profile with Interpretation (combined visual and charts)

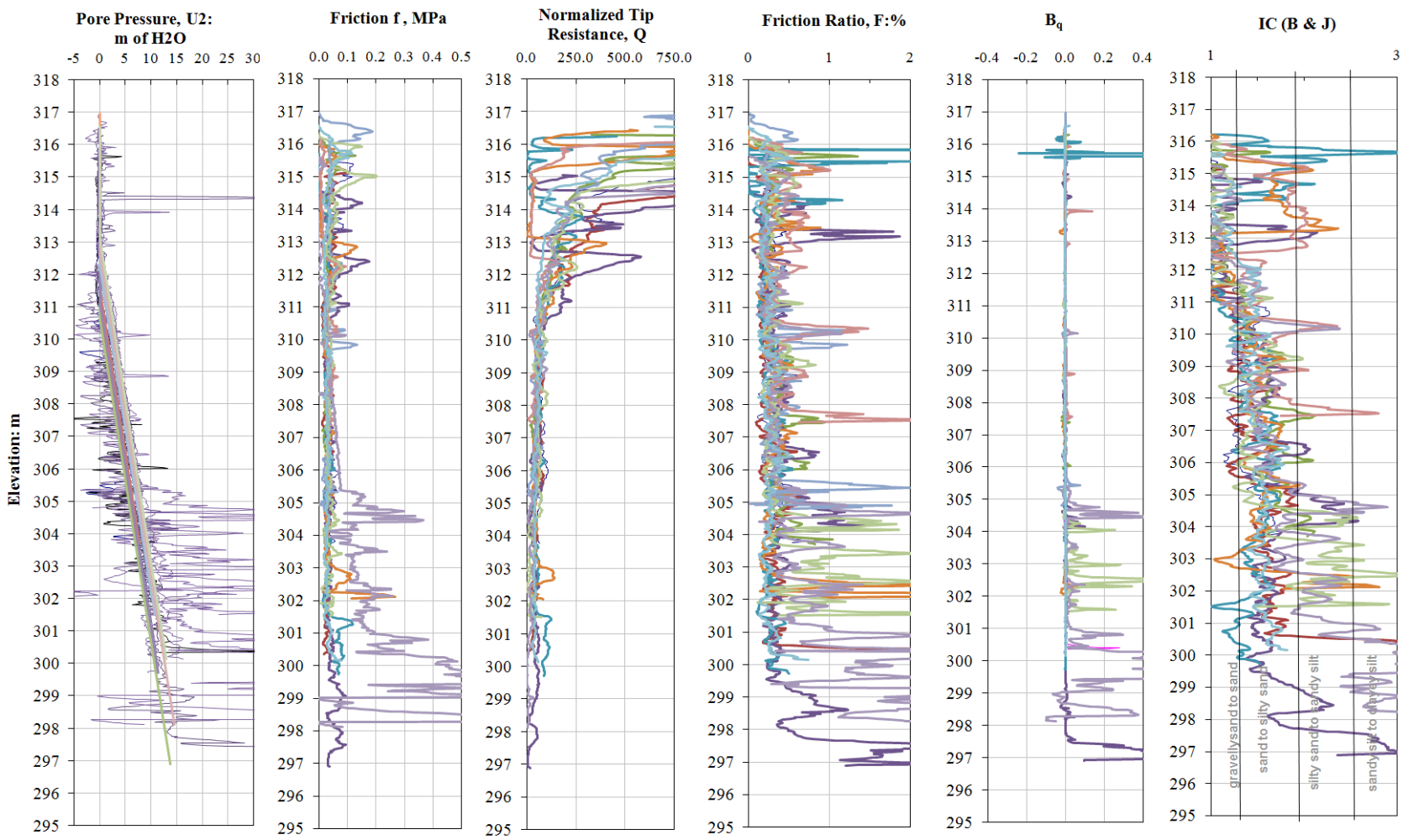


Figure 4-6: All Pre CPT Superimposed

4.1.3 Screening of Post Treatment CPT Data

Eighty post treatment CPT were conducted in various panels in Guindon Dam North area. In this section, clusters of closely spaced post treatment CPT data will be identified in different panels. The approach to the selection of a cluster (viz., choice of post treatment CPT) and comparison of variation of CPT parameters for all the chosen CPT for an example cluster, as well as the trends obtained from the analysis of other similar clusters, will be studied to establish the subsurface profile.

4.1.3.1 Selection of a Cluster

The following criteria were applied for selection of post CPT data in a cluster:

(a) All the CPT data chosen should be closely spaced (spacing between the two CPT should be at least less than the spacing between the pre treatment CPT) to account for the spatial variability.

(b) Since aging of tailings (gain in strength with time) after the treatment is expected and the post treatment CPT data are likely to reflect this, CPT conducted after a similar time period post treatment, should be chosen so that these are likely to yield similar behaviour types and are comparable. Appendix E shows the post treatment CPT data sorted with time.

(c) In each panel, a number of CPT have been conducted at exactly the same location at different times after the treatment. The earliest post CPT have been chosen for analysis because repeating the test at the same spot disturbs the soil in that location, and subsequent CPT data will reflect the disturbed ground behaviour.

4.1.3.2 Comparison of SBT for an Example Cluster

Figure 4-7 shows a cluster of post treatment CPT data from the Guindon Dam North Area that fits the criteria described above. The data are located in panels 49 and 50 between chainage 54+0 and chainage 56+00. The following post treatment CPT are included in the cluster:

(a) Post treatment CPT CPT04-P49-2 (conducted in year 2004 in panel 49 at the location shown below after the treatment)

(b) CPT04-P50-1(2) (conducted in year 2004 in panel 50 after the treatment)

(c) CPT04-P49 2(2) (conducted in year 2004, in panel 49, after the treatment)

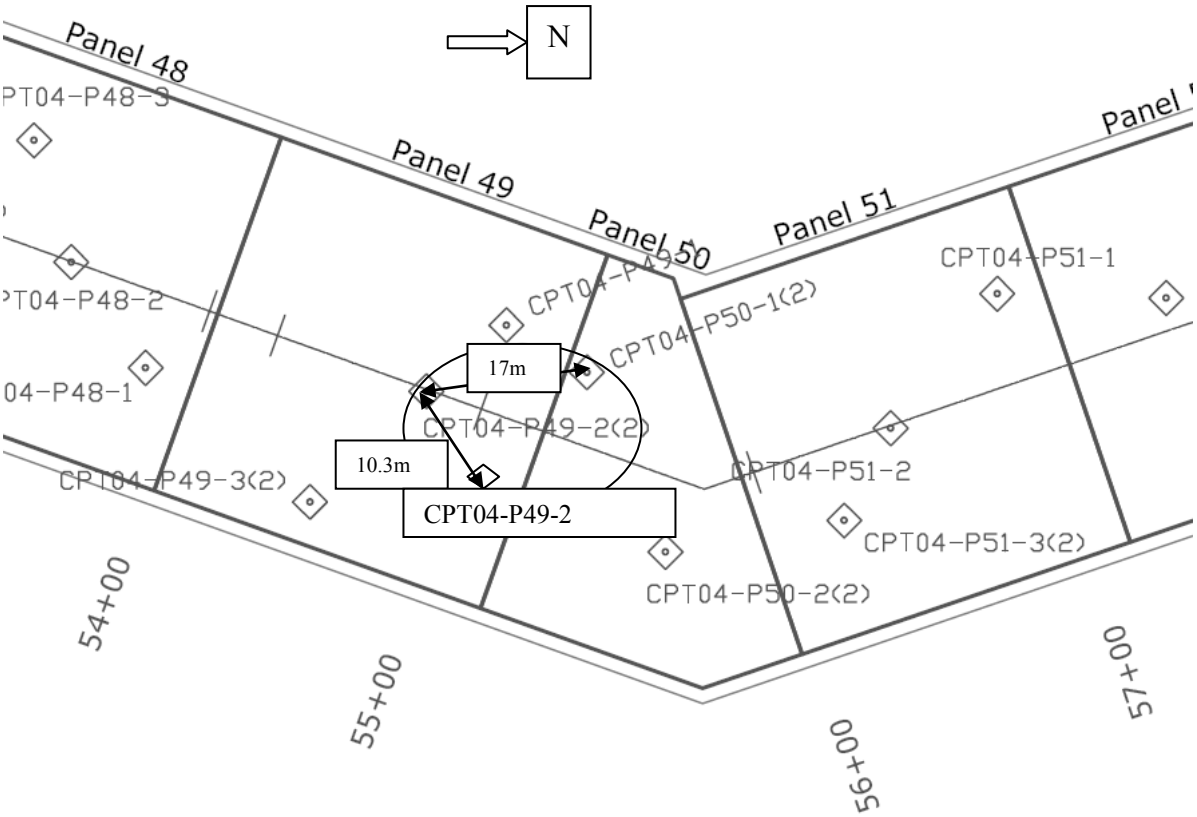


Figure 4-7: Example Cluster of Closely Spaced Post CPT Data for Panels 49 and 50

Table 4-1 shows the ground surface elevations, water table elevations, schedule of data collection, and the timing of post CPT with respect to the timing of treatment of panels considered in the cluster. The position of CPT04-49-2 is indicated by a hollow diamond on Figure 4-7. It should be noted that in spite of being conducted at different times after the treatment, the three post treatment CPT have been grouped together as they were conducted a month after the treatment as shown in appendix E and are expected to have similar aging characteristics. Figure 4-8, Figure 4-9, and Figure 4-10 show the variation of non normalized, normalized tip resistance, measured pore pressures, normalized pore pressure, and friction ratio with elevation for CPT04-P49-2, CPT04-P50-1(2), and CPT04-P49 2(2) respectively. Figure 4-11 shows all the parameters plotted together for above CPT. A comparison of the three CPT profiles indicates:

- a top zone of fill material (about 2m thick)
- underlain by gravelly sand to sand, (characterized by high tip resistance, 5-15 MPa, normalized tip resistance Q varying from 100 to 200 hydrostatic conditions and low friction ratios of 0.5)
- Followed by a zone with increasing layers of thin fine grained material (characterized by similar range of tip resistance as above, 5-15 MPa, and similar friction ratio 0.5% and dynamic pore water pressures that are below hydrostatic).

CPT04-P(50)-1(2) in Figure 4-9 at greater depths further shows the dilative response, indicating silty material, (tip resistance up to 5MPa, negative pore pressure response, and increasing friction ratio, up to 1%) and clayey silt (low tip resistance up to 2 MPa, positive excess pore pressures, higher friction ratio up to 2 and B_q value up to 0.35).

Table 4-1: Details of Post CPT Data in the Cluster

Parameters	CPT04-P49-2	CPT04-P50-1(2)	CPT4-P49-2(2)
Position: Chainage & CL	CH 54+70, 7' east of CL	CH 55+29, 30' west of CL	CH 54+80, on CL
Ground Surface Elevation (m)	314.5*	314.5*	314.5*
Water Table Elevation (m)	313.5	311.6	310.5
Date of Data Collection	12th August 2004	22nd Sept 2004	20th September 2004
Date of Last Treatment in Panels 49 &50	9th July 2004	30th July 2004	9th July 2004

* Elevations of the post CPT data determined on the basis of settlement recoded by markers after treatment. CL – centre line

Even for the closely spaced CPT data this is a generalized description, as there is only one CPT, CPT04-P(50)-1(2), which shows the presence of finer material at greater elevations (up to 298m), while CPT04-P49-2 and CPT04-P49 2(2) show refusal at 303m to 305m respectively. Further, CPT04-P49-2 shows a lower tip resistance in the upper 4m zone (312m-308m), of the order of 5 MPa. Similar variation is seen at elevations 308m-306m and at 305.5m leading to a variation in behaviour type from gravelly sand to silty sand. Based on the pore pressure response in Figure 4-11, three CPT show three different values of elevation of water table. This could be due to the presence of perched up water or undissipated excess pore pressures generated due to ground treatment making the water table appear shallower than its actual level.

It is observed from the above comparison that even for the selected closely spaced CPT data described above, considerable spatial variation in the measured CPT parameters and interpreted soil behaviour type is observed. Additionally, it was difficult to select the representative clusters for the analysis due to other limitations such as availability of few closely spaced CPT data which satisfy the selection criteria and approximate elevations for the post CPT leading to possible erroneous comparison of the CPT profiles.

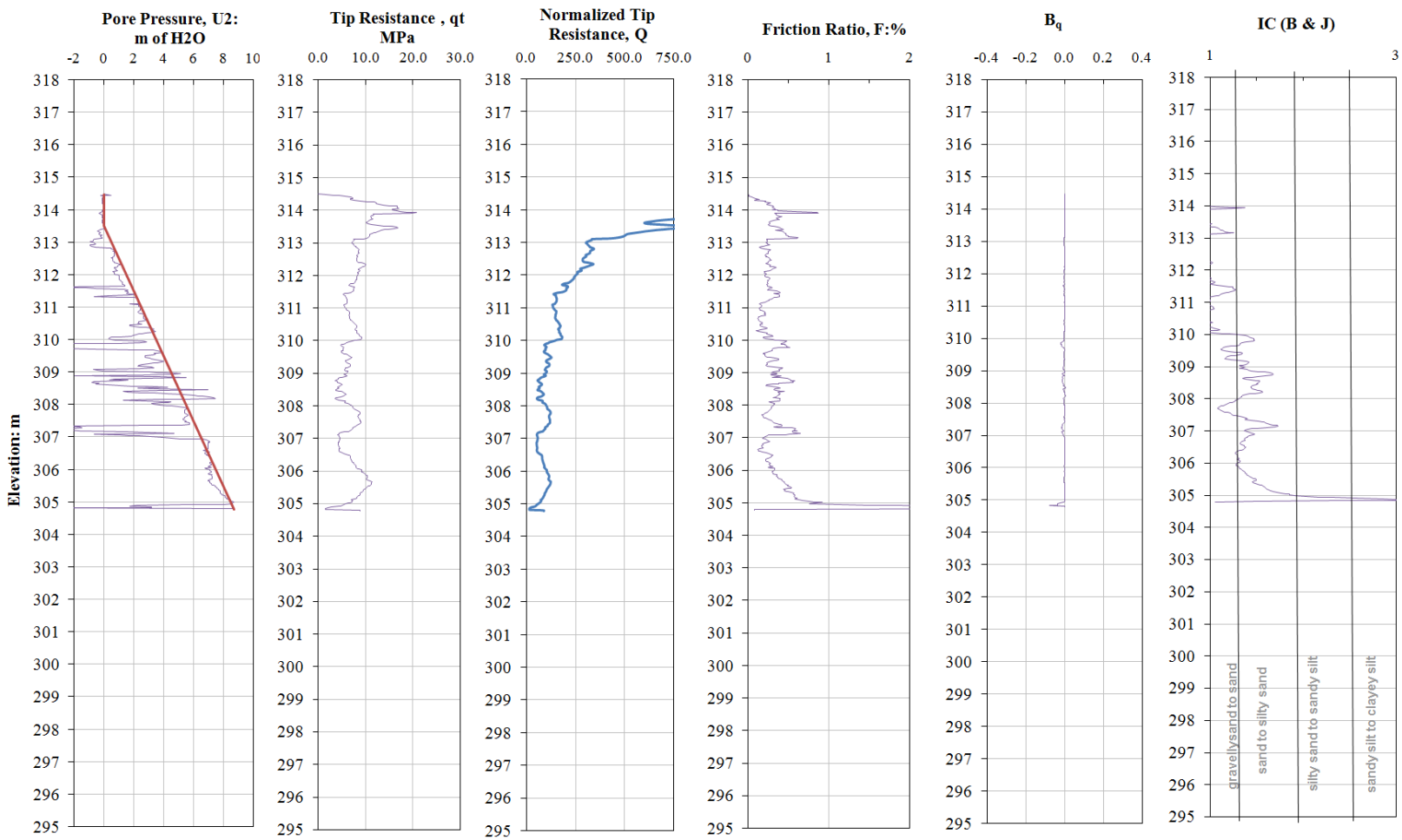


Figure 4-8: Profile CPT04-P49-2

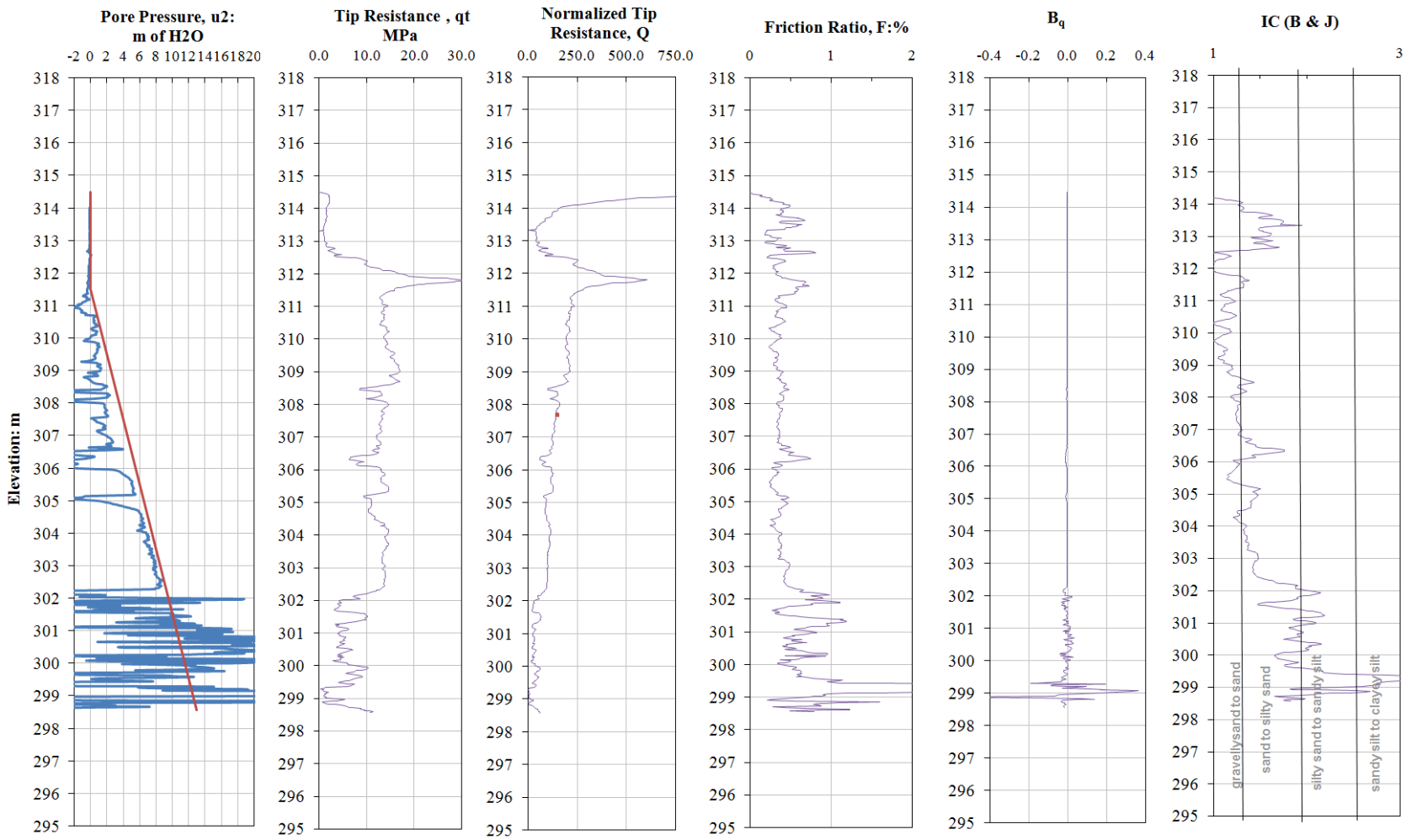


Figure 4-9: Profile CPT04-P50-1(2)

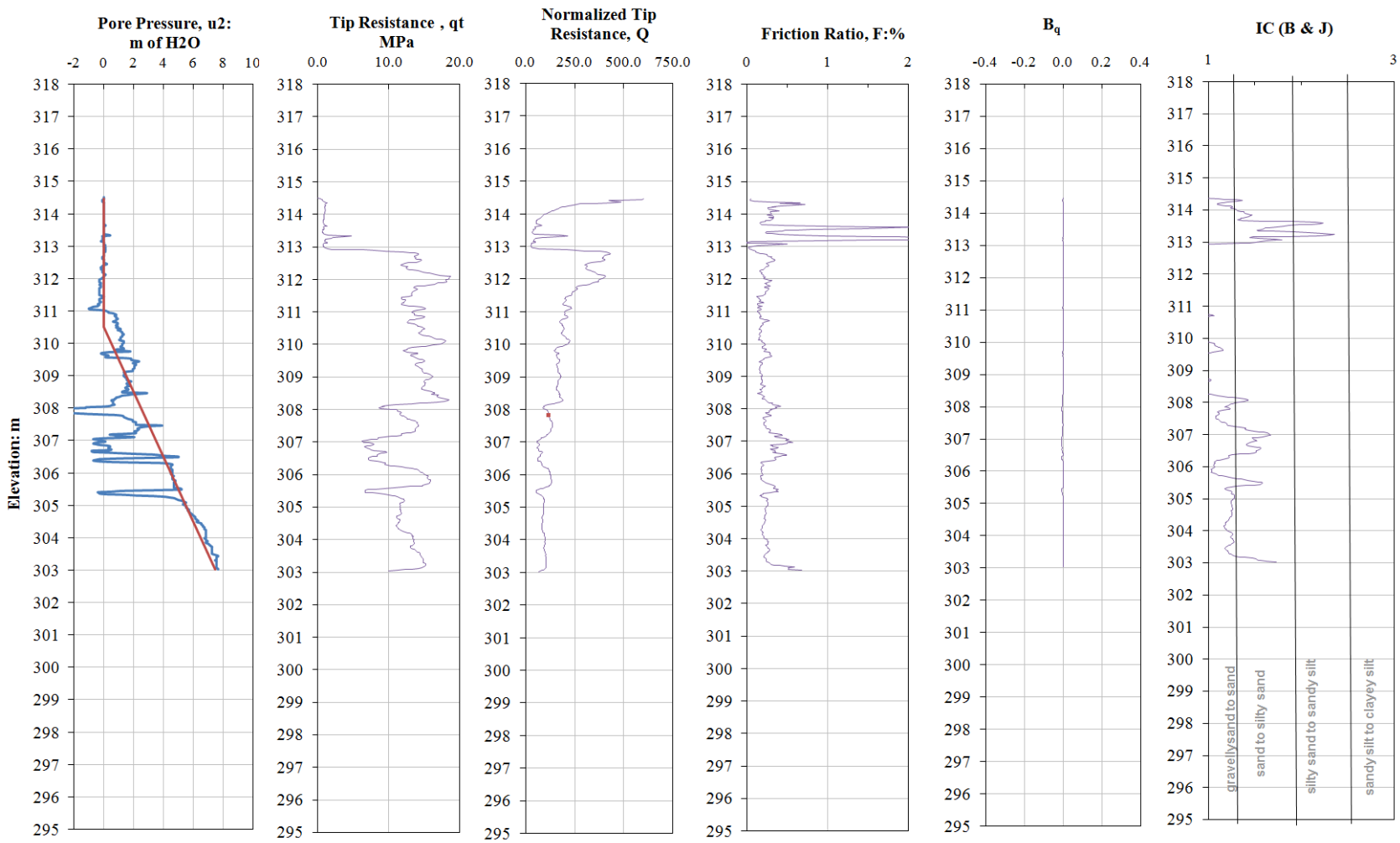


Figure 4-10: Profile CPT04-P49-2(2)

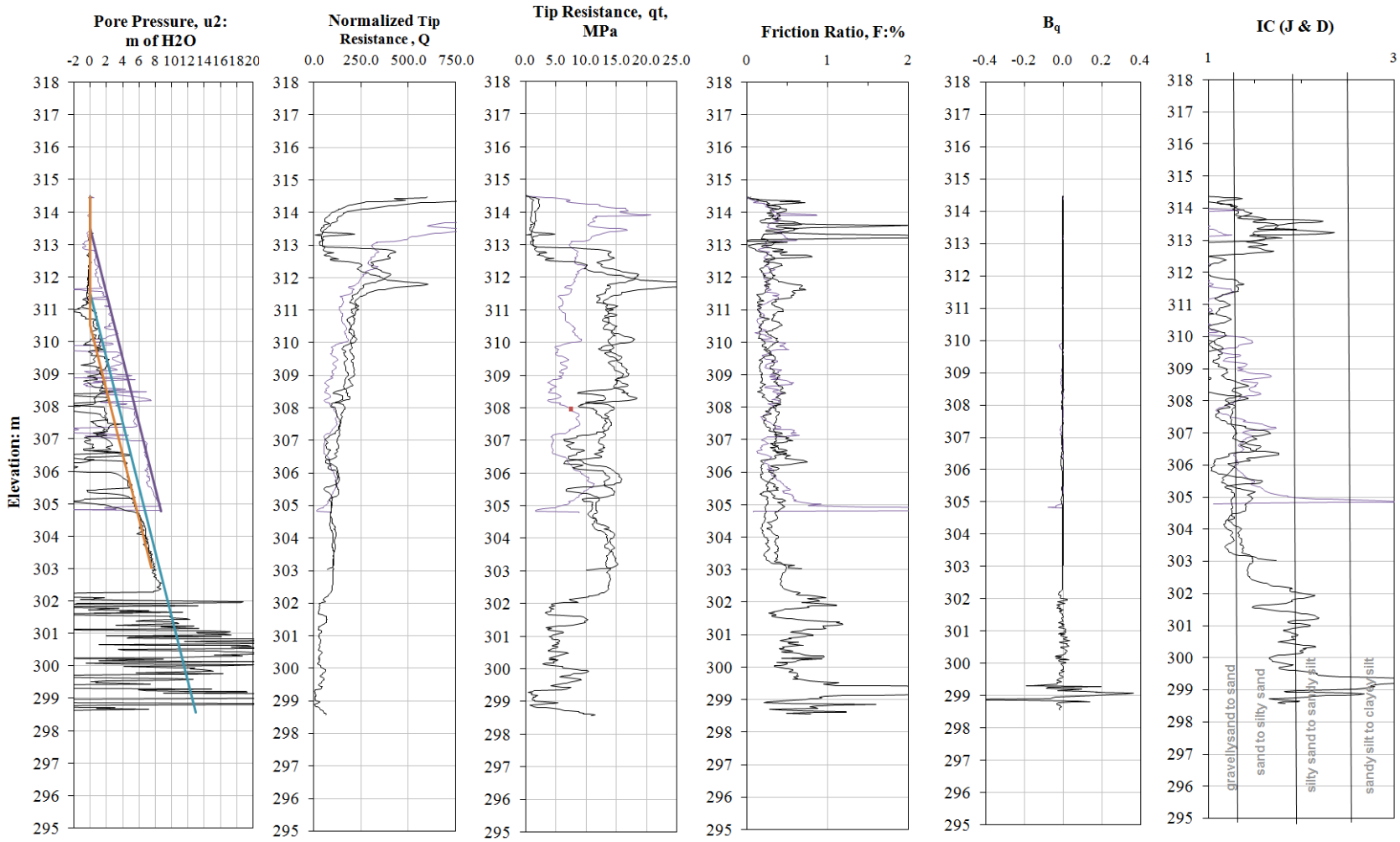


Figure 4-11: All Post CPT in a Cluster Superimposed

4.2 Implications of Data Screening: Recommendations for Addressing Site Variability

(a) Despite the attempt to recognize patterns in the clusters of CPT, it was not possible to identify zones that could be considered to be relatively homogeneous. Consequently, it was decided that the pre and the post treatment CPT data from all the panels of the chosen section of the Guindon Dam should be included in the comparison, so that the variation in behaviour type on the site can be adequately represented.

(b) It was also decided that the variability would be considered using statistics in the form of mean, median, and standard deviation.

(c) Normalized CPT parameters should be used to represent the ground improvement. Normalization attempts to account for the variation in interpreted behaviour due to variation in overburden stress, and takes away one source of variability.

Typically, ground improvement is assessed on the basis of the variation of the pre and the post treatment tip resistance with depth, within zones of consistent SBT. Since there is no clear pattern of tailings types observed at any depth level for this site, improvement achieved cannot be related to the material type reliably using the conventional representation.

4.3 Interpretation of CPT Data in Sudbury Tailings & Representation of Ground Improvement

4.3.1 Selection of Depth Intervals for Analysis

Chapter 3 described the combination of different treatments applied on site. It was shown that most of the site was treated by a combination of EC and either RIC or DC. Appendix E shows that majority of post CPT data record effects of EC and RIC and hence, this combination has been considered for the analysis. RIC is generally considered to be effective at a depth of approximately 7m below ground level. It was also noted in Section 4.1.2 that the tip resistance reduced below the water table. Consequently, in order to reduce

the effects of site variability on the data, the following depth intervals are proposed for data analysis:

(a) Top Interval (Ground Surface to Water Table):

This zone was found to be highly variable due to presence of rockfill. This zone also enables a study of the effects of ground improvement techniques above the water table. The water table considered is the pre treatment water table as indicated by the boreholes to be at about 4.5m below GL.

(b) Water Table -7m Interval:

Thus, post treatment CPT in nearly all of the panels record the effects of EC and RIC. Surface compaction is expected to be effective in treating soils up to 7m below ground level (GAIA, 2004). Hence, this zone is studied to enable the combined effect of surface compaction with EC to be differentiated from EC alone.

(c) 7m-11m Interval:

This interval is below the expected depth of compaction typically achieved by surface compaction and hence is expected to show the effects of explosive compaction. Chapter 3 showed that the charges used for explosive compaction increased in intensity (measured as charge weight in kg) with depth to account for the mean confining stress. The intensity increased after 11m. Hence, this zone is considered up to 11m.

(d) 11m-16m Interval:

This final interval below 11m, considers the effects of EC with a higher average charge weight.

4.3.2 Data Interpretation & Basis for Representation of Ground Improvement: Use of Soil Behaviour Type Chart (Jefferies and Davies, 1991)

Figure 4-12 to Figure 4-15 below show the pre treatment CPT data in the form of dimensionless Q (1-B_q) grouping and the normalized friction ratio F in %, plotted on the soil behaviour type interpretation chart proposed by Jefferies and Davies (1991).

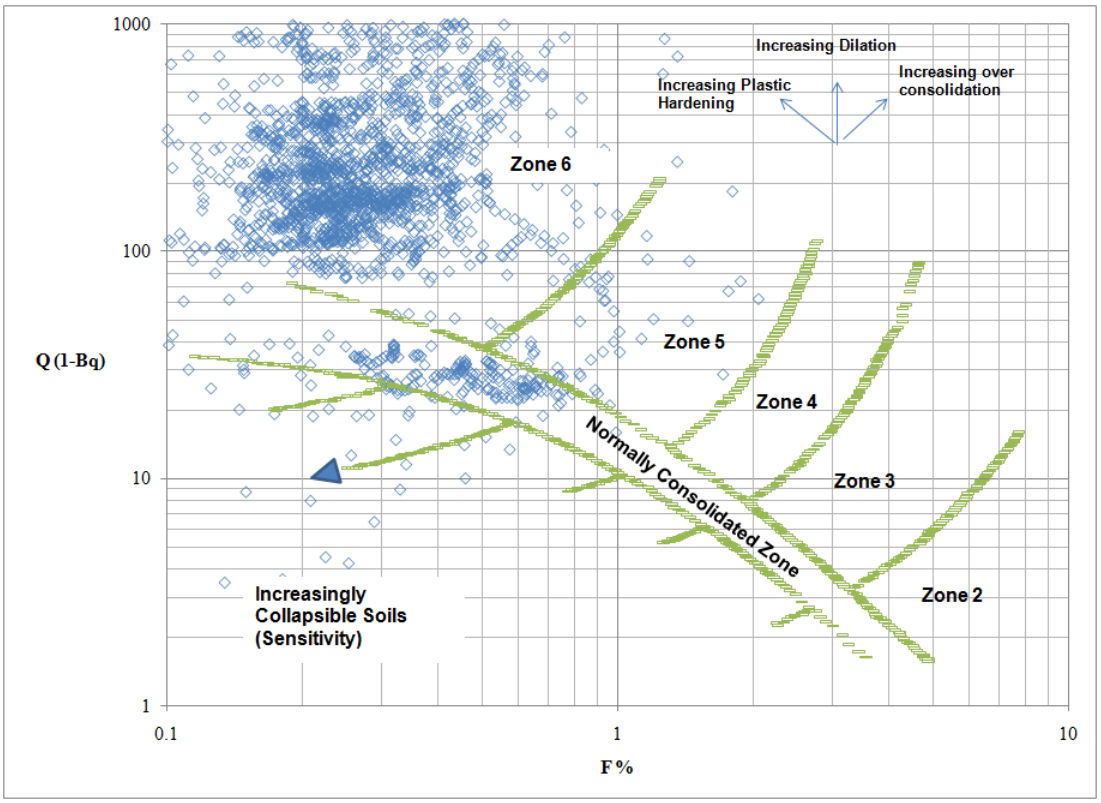


Figure 4-12: Pre –Treatment Data on SBT Chart (Jefferies & Davies, 1991) Interval I (top-wt)

The water table (as per the borehole logs) is located at a depth of about 4.5m below ground level. The Figures show that in depth intervals I, II and III, the pre data primarily lie in SBT zones 6 and 5 on the SBT chart, classifying the tailings as behaving like clean sand to silty sand and silty sand to sandy silt. Depth interval I (top-wt) shows some scatter in the data across the behaviour type zones on the plot. Very high values of Q (1-B_q) (up to 1000) are observed for tailings in this depth zone, reflecting the presence of rockfill. Low tip

resistance can be attributed to the presence of primarily loose sand and silt sized tailings along with clayey material in these zones (as seen from the pore pressure response of the pre CPT data and as described by the borehole logs in Appendix A).

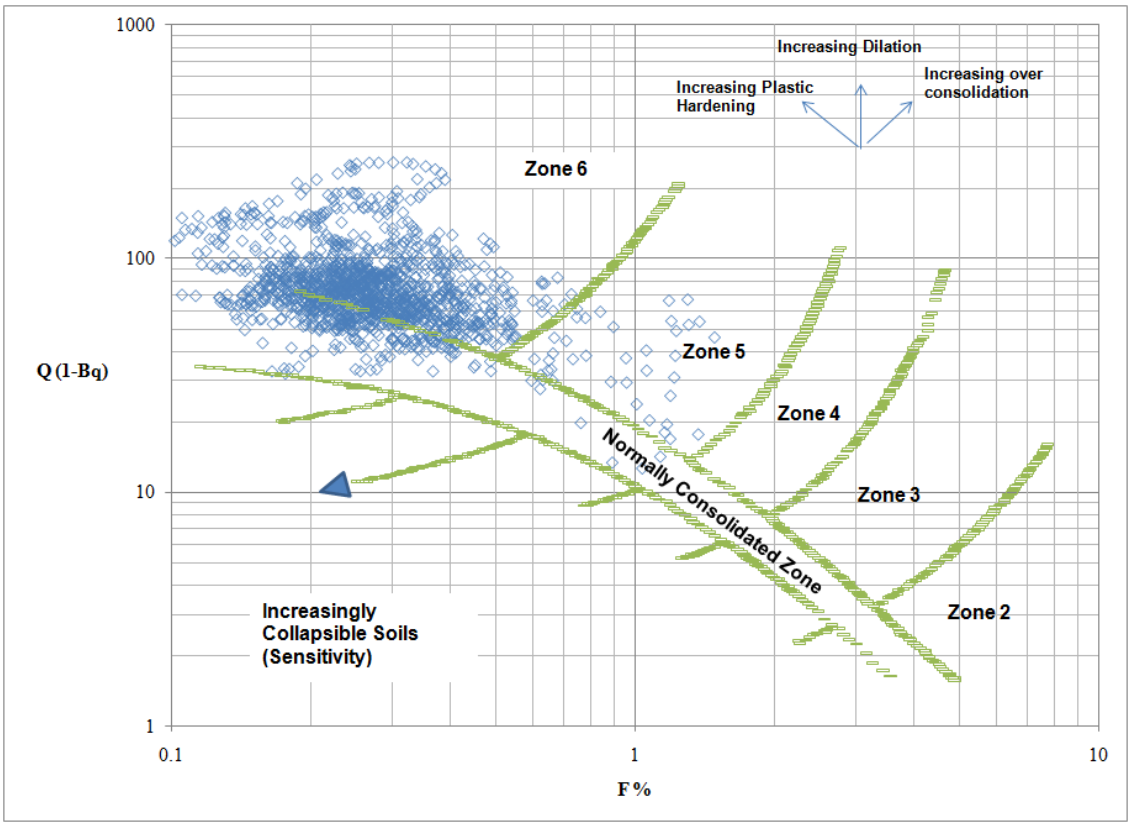


Figure 4-13: Pre-Treatment Data on SBT Chart (Jefferies & Davies, 1991) Interval II (wt-7m)

Depth intervals II (wt-7m), III (7m-11m) and IV (11m-16m) show lower range of maximum Q (1-B_q) values, up to 200. While intervals II and III show lower scatter (data plotted as a cluster on SBT zones 6 and 5 on SBT chart); interval IV shows considerable scatter with data points plotted across all the zones on the chart and the position of the points varying within a zone for a given I_c value. The presence of finer natural geo materials such as silt, clay, and sensitive, organic material is indicated for interval IV, as encircled in Figure 4-15.

The behaviour type shown by the charts is in general comparable to the soil type shown by the borehole logs. A majority of logs, as discussed earlier show presence of rock fill material in the top (2m) zone. This zone is underlain by sand and silt sized tailings along with traces

of clay tailings and natural clay, sands and silts having variable thicknesses and no clear boundaries of the soil type.

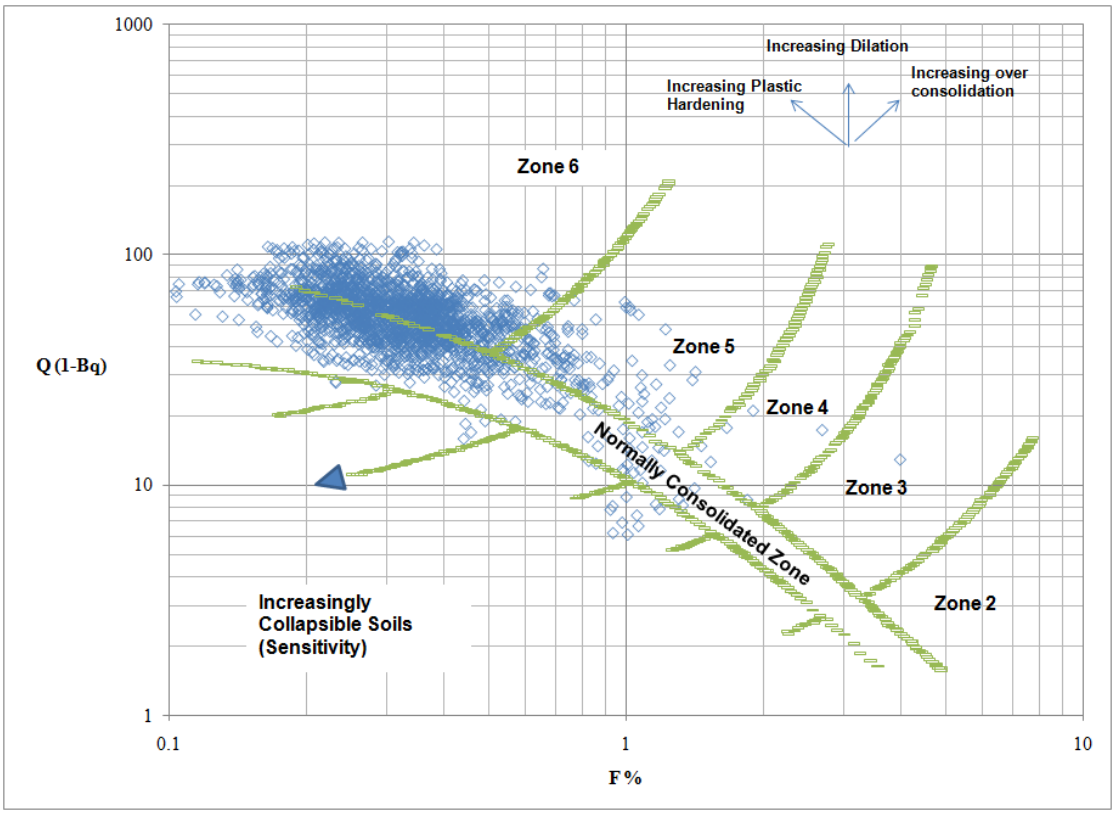


Figure 4-14: Pre-Treatment Data on SBT Chart (Jefferies & Davies, 1991) Interval III -7m-11m)

At greater depths, (last metre or so), the soil type becomes finer with presence of natural clays, clay sized tailings and organic soils (peat). Also, the overall variability in soil behaviour type and its extent is evident from the scatter on the SBT charts, particularly from Figure 4-12 and Figure 4-15.

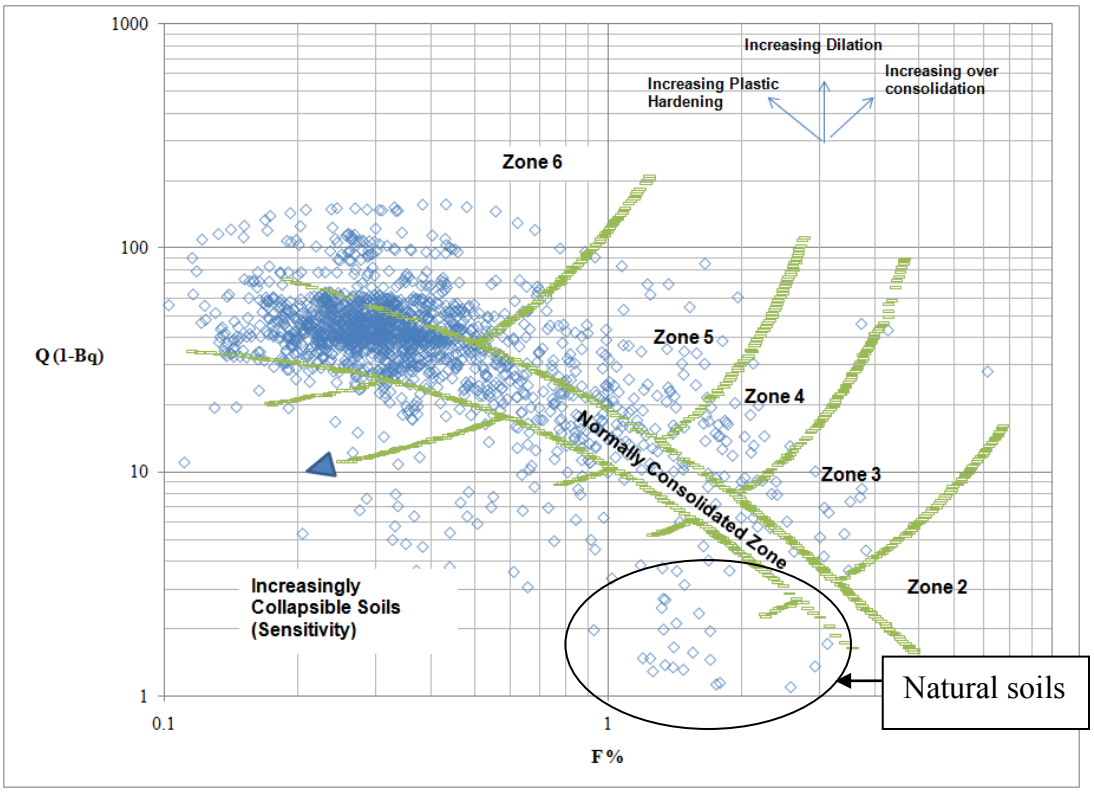


Figure 4-15: Pre-Treatment Data on SBT Chart (Jefferies & Davies, 1991) Interval IV (11m-16m)

Another aspect of interpretation of behaviour type involves determining the in-situ state with respect to the critical state ("loose" or "dense") of soils/tailings at a given stress level. The "looseness" of Sudbury tailings was reported based on the 2002 investigations (Golder, 2002). The SBT chart can also be used to indicate the in-situ state of these tailings (as a function of state of stress and density). Plewes et al. (1992) proposed a chart to allow identification of the in-situ state of the soil from the normalized CPT parameters. This approach is presented in detail in Chapter 2, section 2.2.2.4.3. Contours of state parameter, ψ as determined by Equation 2-8 (Chapter 2) were superimposed on the soil behaviour chart proposed by Jefferies & Davies (1991). It was observed that for several tailings sites where liquefaction slumps had occurred, the above data plotted near or below the $\psi = 0$ line and, in general, any value of state parameter greater than -0.1 suggested that the soil was susceptible to liquefaction, while $\psi < -0.1$ suggested that the soil was not susceptible to liquefaction. Davies (1999) further extended the use of this approach for tailings.

Figure 4-16 and Figure 4-17 show the pre treatment data for depth intervals III and IV plotted again on the SBT chart with the contours of CPT derived state parameter values. It is seen that most of the pre data plot below the ψ = -0.1 line. This suggests that Sudbury tailings in the depth intervals shown below (7m-16m) were susceptible to liquefaction and needed mitigation. The pore pressure response from the pre CPT data and the boreholes indicate presence of both loose sand and silt sized tailings and clayey material in these depth intervals.

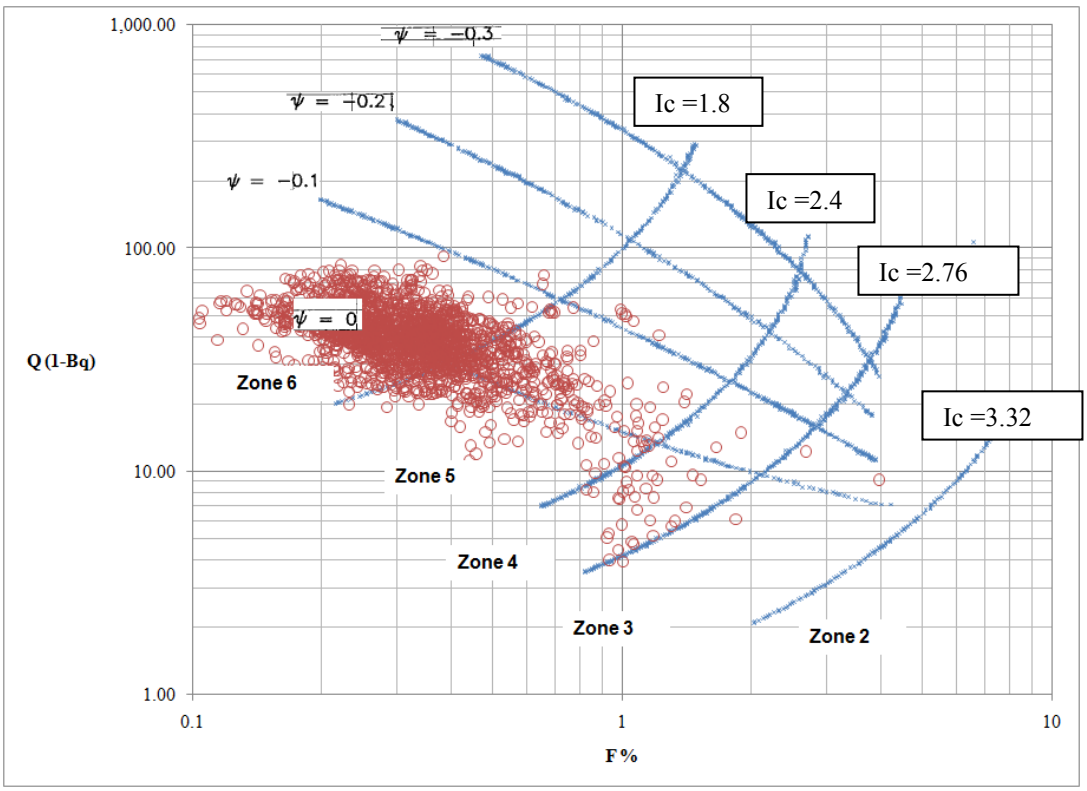


Figure 4-16: Piezocone State Screening Approach (Plewes et al., 1992) - Interval III (7m-11m)

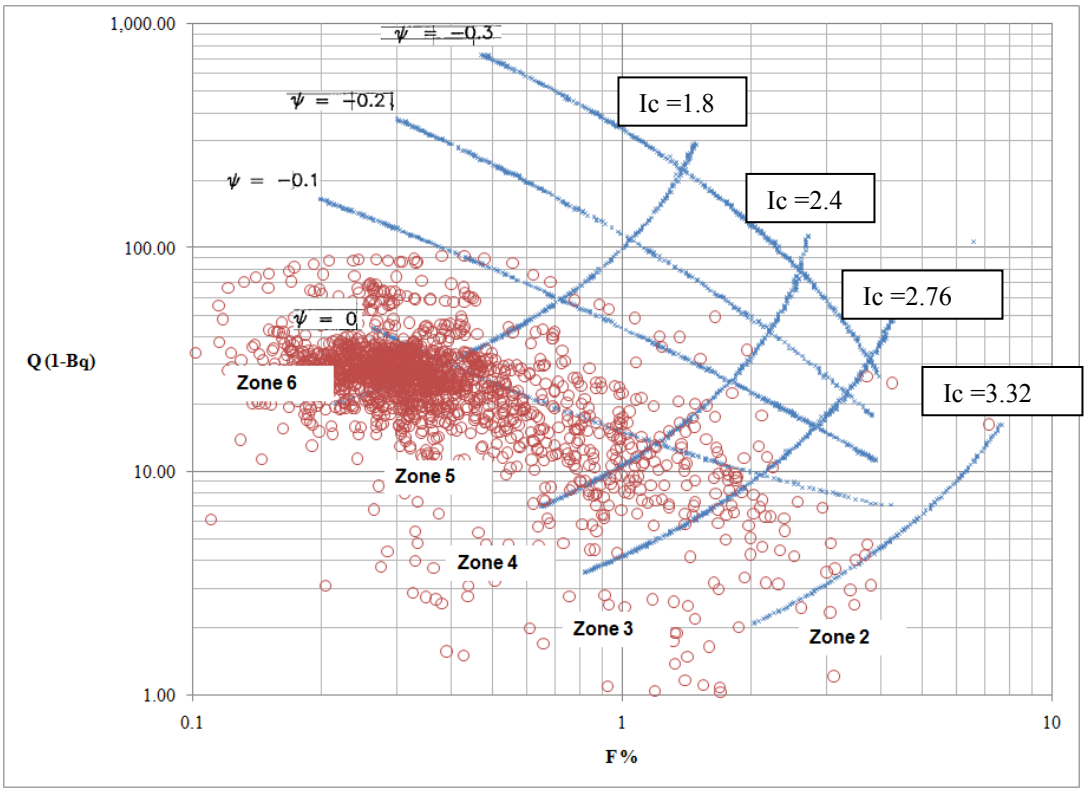


Figure 4-17: Piezocone State Screening Approach (Plewes et al., 1992) - Interval IV (11m-16m)

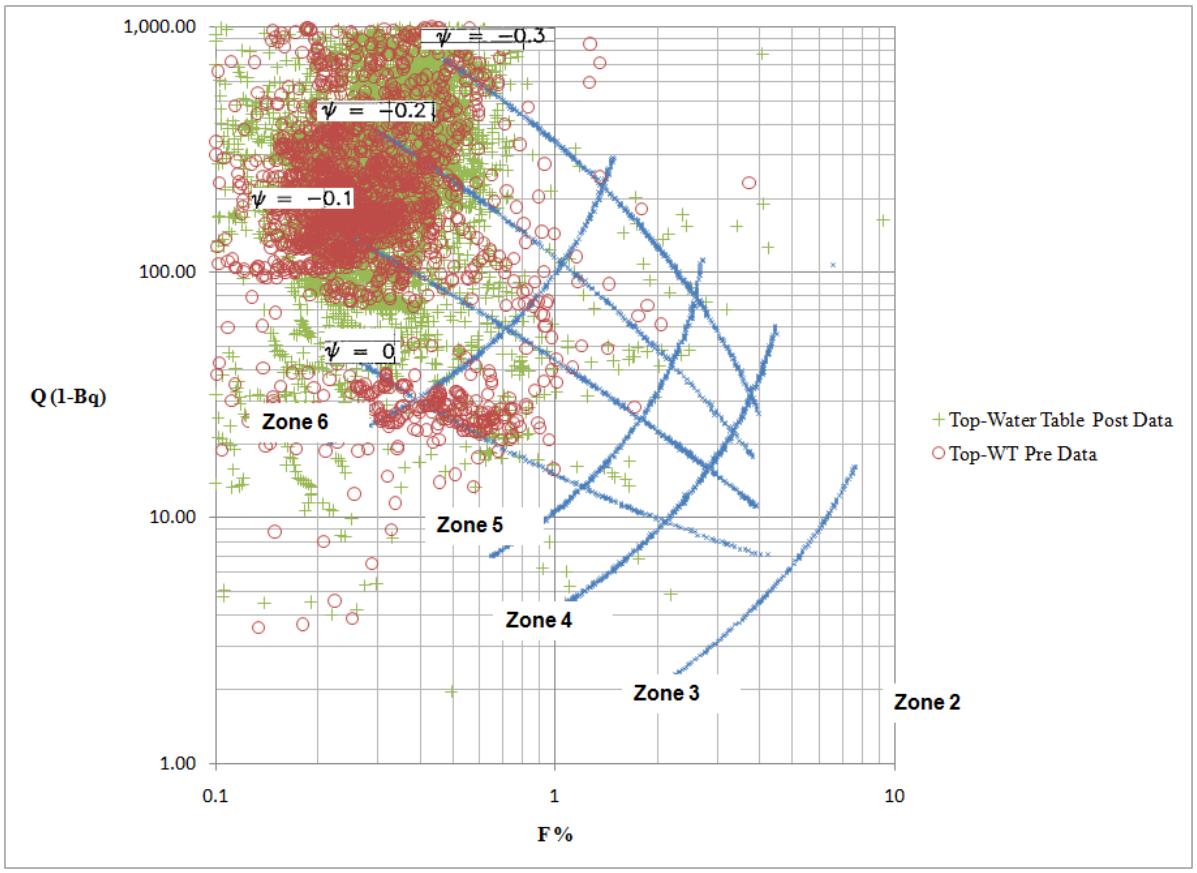


Figure 4-18: Piezocone State Screening Approach Plewes et al. (1992) – Pre & Post Treatment Data Interval I (Top-Water Table)

Similarly, Figure 4-18 to Figure 4-21 show the state as per Equation 2-8 for the post treatment ground by plotting the data from eighty sorted post treatment CPT along with the pre data on the SBT chart for the lower depth intervals. Figure 4-18 shows scatter in the post data in the top interval similar to that in the pre data in that interval and almost no noticeable improvement in the tip resistance. Figure 4-19 to Figure 4-21 show that the post data in lower depth intervals have moved towards contours of lower state parameter values (less than $\psi = 0$), indicating that the ground has become denser or more dilative. Maximum Q values of 100-300 have been achieved as a result of ground improvement with a minimum increase over pre for the lower depth interval (11m-16m). It is also observed from these Figures that most of the post treatment data (except some data points in depth interval IV) do not show presence of low tip resistance material ($Q\ (1\text{-}B_q) < 10$).

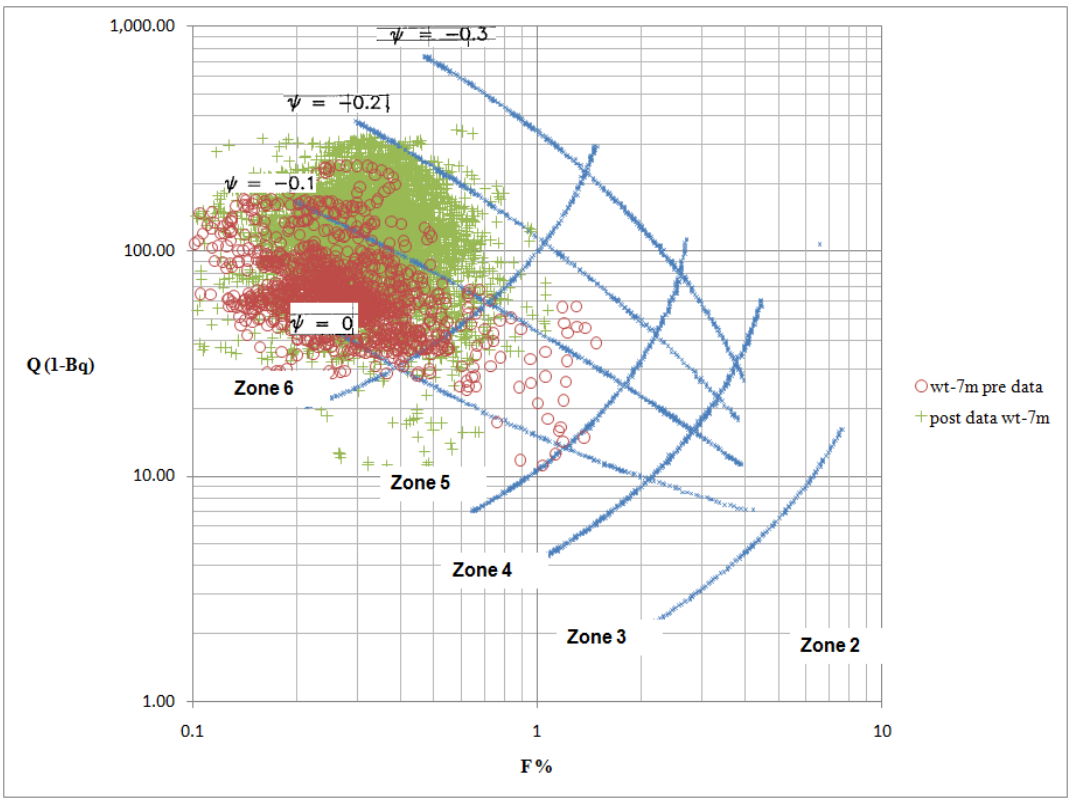


Figure 4-19: Piezocone State Screening Approach Plewes et al. (1992) – Pre & Post Treatment Data Interval II (wt-7m)

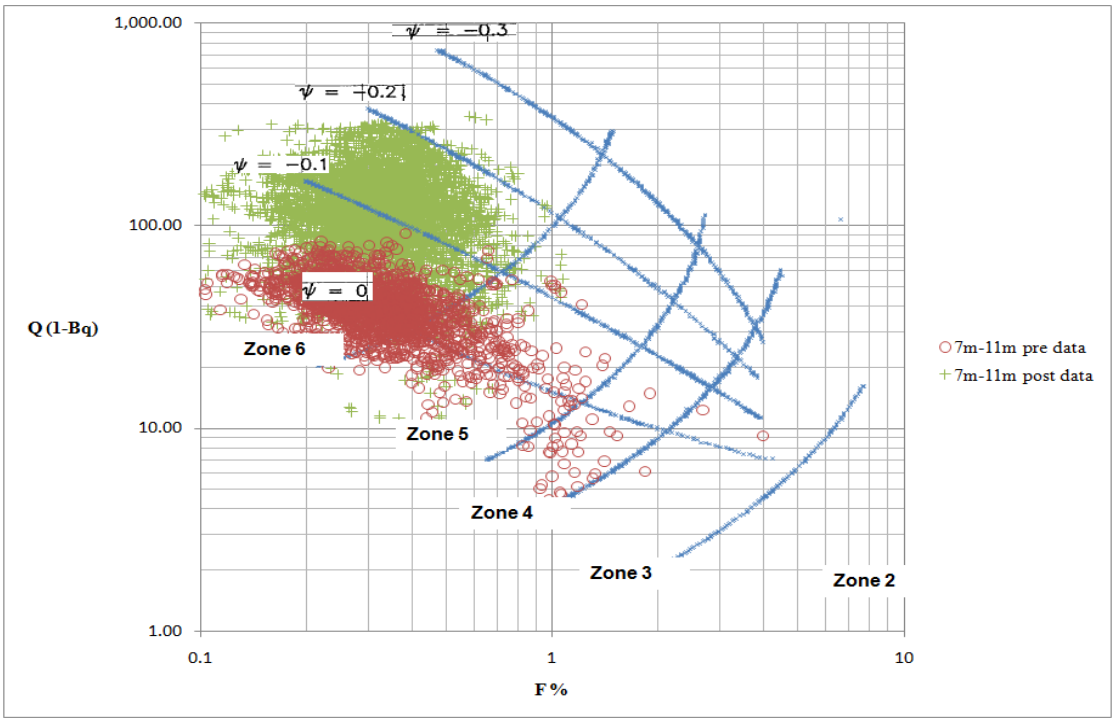


Figure 4-20: Piezocone State Screening Approach Plewes et al. (1992) – Pre & Post Treatment Data Interval III (7m-11m)

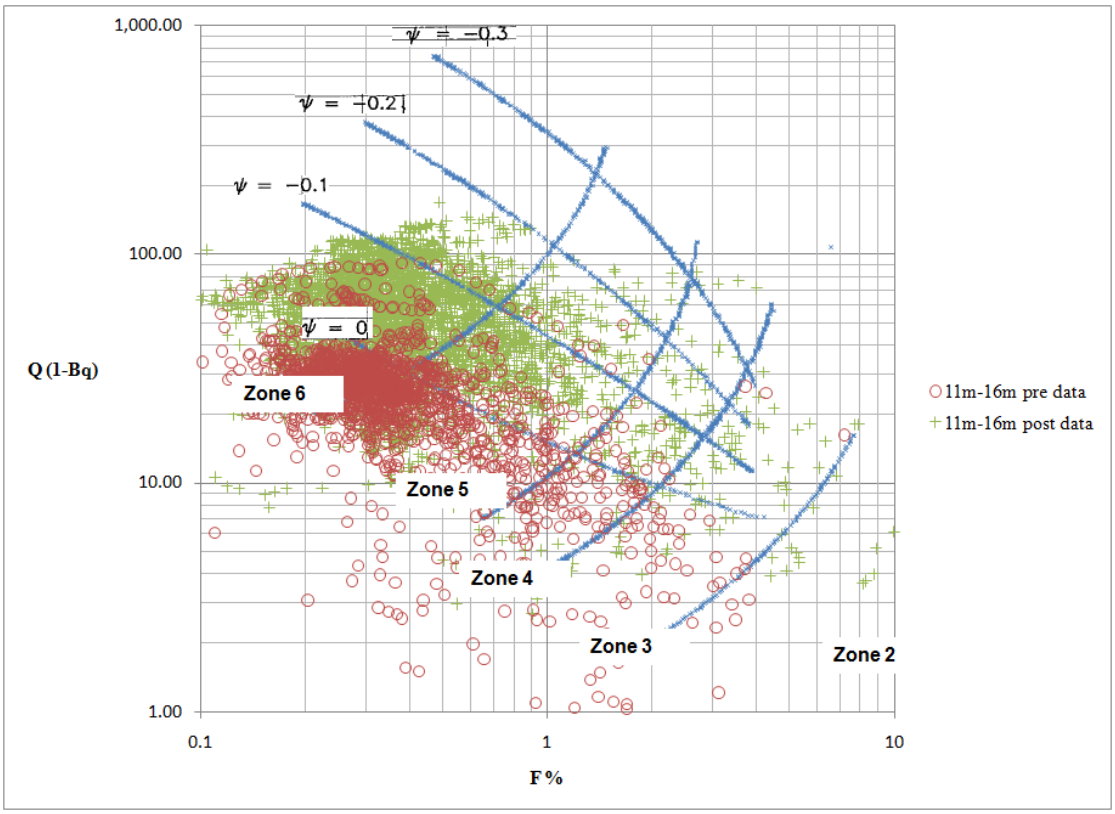


Figure 4-21: Piezocone State Screening Approach Plewes et al. (1992) – Pre & Post Treatment Data Interval IV (11m-16m)

4.3.3 Choice of Normalized CPT Parameters for Assessment of Ground Improvement

The SBT chart incorporates the following normalized CPT parameters: normalized tip resistance Q, pore pressure ratio B_q and normalized friction ratio F. Besides these parameters, the soil behaviour type index I_c which is an algebraic grouping of these parameters is also incorporated. Various zones on the SBT chart correspond to different values of I_c.

The normalized tip resistance Q has been grouped with B_q in the SBT chart to account for the pore pressures generated in fine grained tailings. Jefferies & Davies (1991) report a value of 0.45 for B_q from the offshore applications of CPT. Figure 4-6 shows that B_q is found to have a maximum value of 0.4 for Sudbury tailings observed at greater depths (greater than 12m) and for only two CPT soundings. A majority of the soundings indicate very low values of B_q especially where the normalized friction ratio F has consistently

indicated behaviour typical of finer soils (Figure 4-6, elevation 313m-305m). This means that in most cases, values of Q and Q (1-B_q) are interchangeable. Figure 4-22 compares values of Q and Q (1-B_q) for the depth interval IV for the pre treatment CPT data. Both, the bore logs and SBT charts had indicated presence of fine grained material for this depth interval. Interval IV shows that below Q of 10, Q (1-B_q) begins to differ from Q.

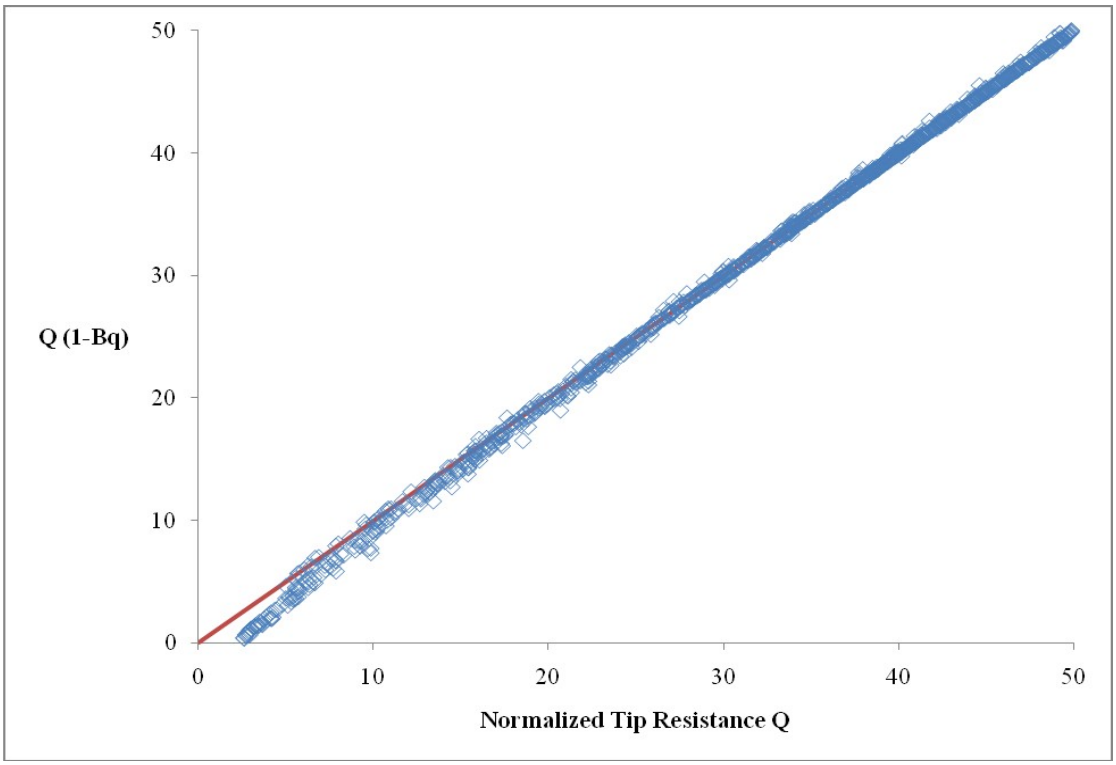


Figure 4-22: Comparison of Q and Q (1-B_q) Values for Pre CPT Data Depth Interval IV (11m-16m)

Thus, for majority of this site (intervals II, III and part of IV), Q and Q(1-B_q) are interchangeable. The other parameters representing the material behaviour type could be either F or I_c.

F is the normalized version of ratio (sleeve friction divided by net tip resistance). The change in F with treatment can be seen in Figure 4-19 to Figure 4-21 which indicate an upward and possibly rightward movement of F. The degree of variation of F with treatment

can be further investigated by considering the distribution of pre and post F values in different depth intervals. For this purpose, friction ratio data have been sorted in ascending order in equal bin size. The distribution of data points in each bin for pre and post F has been compared. Figure 4-23 and Figure 4-24 show the distribution of data points for pre and post F bins for an example depth interval III (7m-11m) in the form of a bar chart. The chart shows the percent (of the total number of data points) data for each bin number. Table 4-2 shows the F bin ranges and the median F corresponding to different bin numbers. It is observed from the chart that for the pre and the post data, the second bin is the one with maximum number of data points (median F-0.3), indicating no change in distribution of F with treatment. Similarly, Figure 4-25 and Figure 4-26 show bar chart for interval IV (11m-16m). Again, bin with median 0.3 has maximum number of data points and no shift in F value with treatment is observed.

Table 4-2: F Bin Ranges

Bin Number	Bin Range	Median F
1	0-0.2	0.1
2	0.2-0.4	0.3
3	0.4-0.6	0.5
4	0.6-0.8	0.7
5	0.8-1	0.9

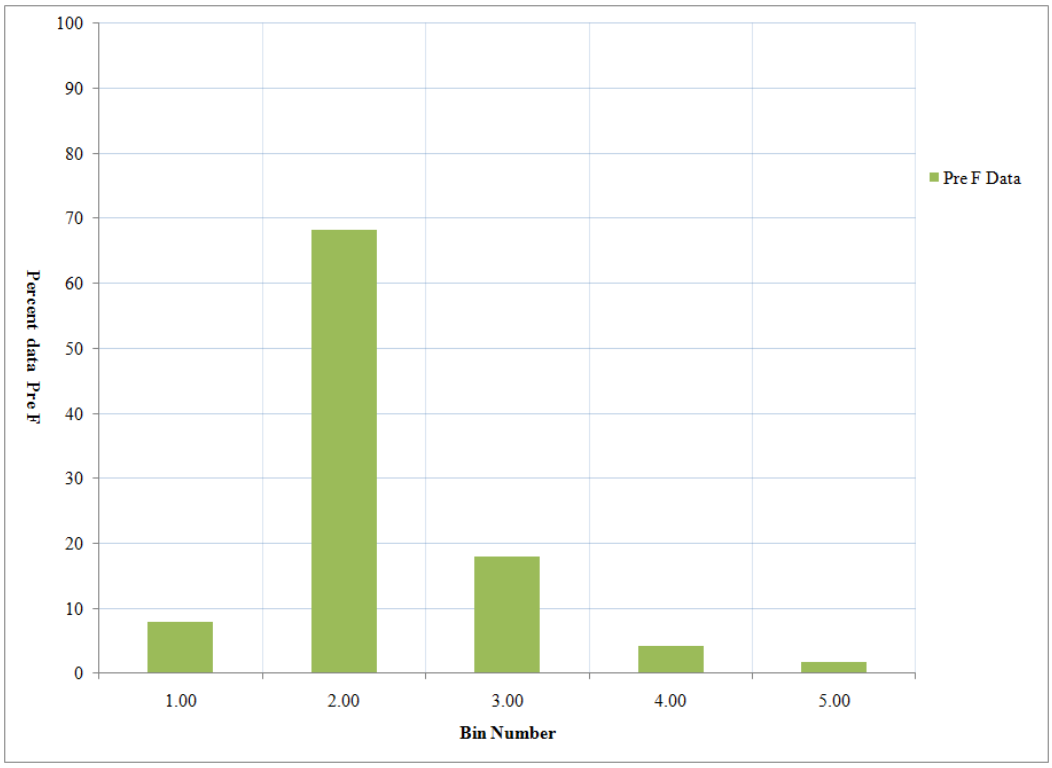


Figure 4-23 : Distribution of Data Points for Pre F Interval III (7m-11m)

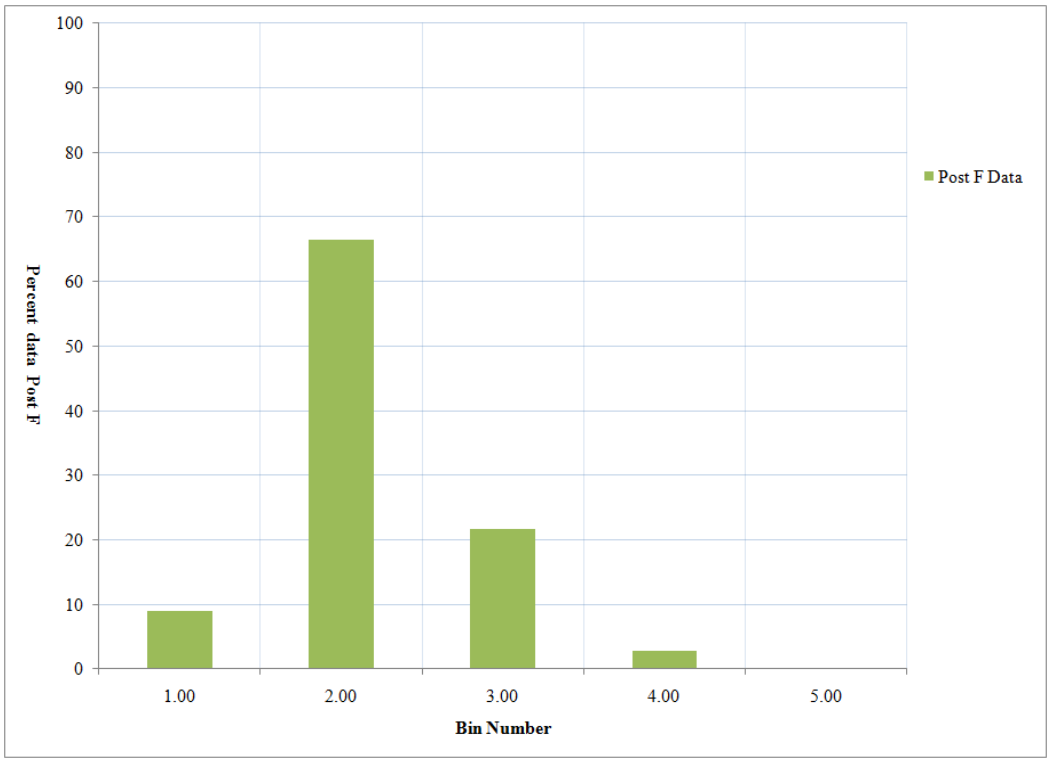


Figure 4-24 : Distribution of Data Points for Post F Interval III (7m-11m)

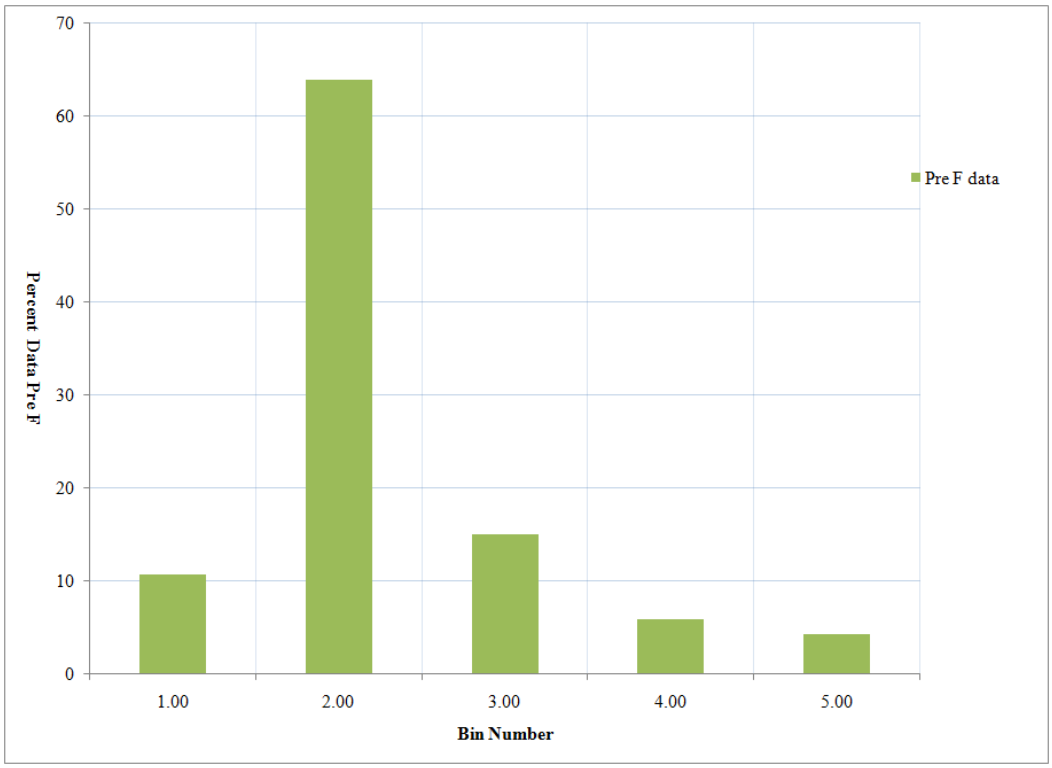


Figure 4-25 : Distribution of Data Points for Pre F Interval IV (11m-16m)

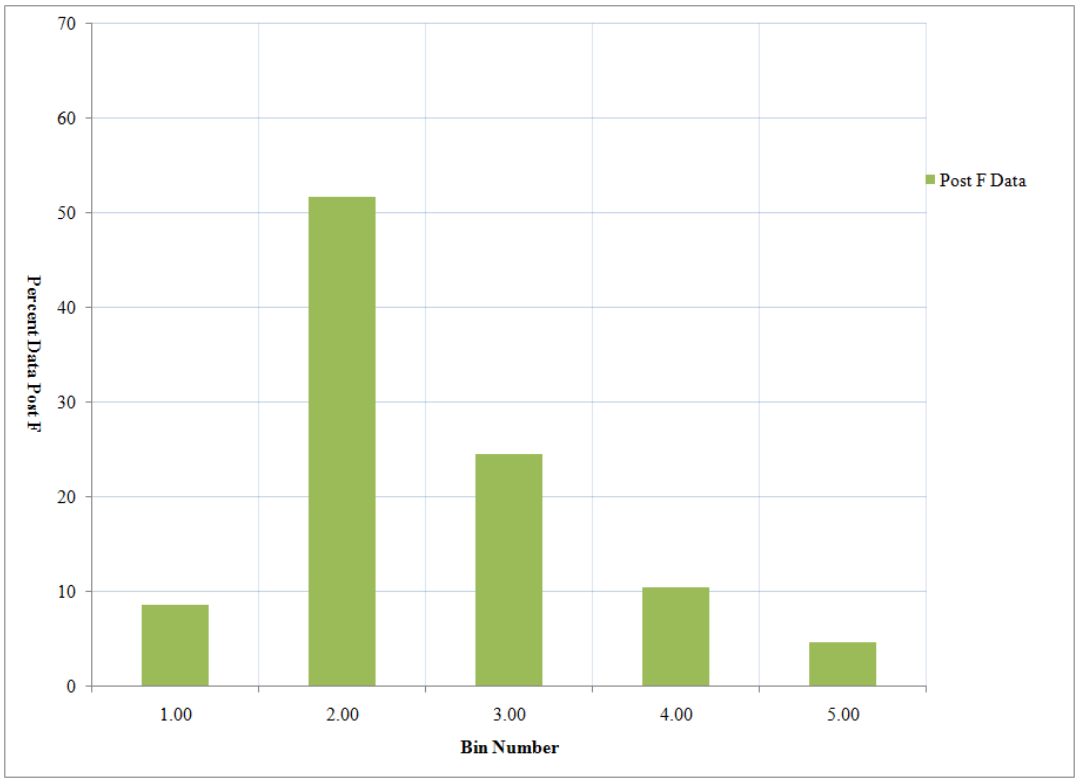


Figure 4-26 : Distribution of Data Points for Post F Interval IV (11m-16m)

Similarly, the Q-I_c variation relates the penetration resistance of the ground to the material behaviour type indicated by the soil behaviour classification index I_c (Jefferies & Davies, 1991) which incorporates Q, F and B_q. Been and Jefferies (1993) modified the above version and this modified version of I_c has been used in this thesis. Considering the variation shown by F and Q with treatment, the variation of I_c is predictable. With an increased Q and constant F, I_c is expected to decrease with treatment. Figure 4-27 and Figure 4-28 compare the distribution of data points for various pre and post treatment I_c bins for intervals III (7m-11m) and IV (11m-16m). Number of data points in an I_c bin of size 0.5 have been compared. Table 4-4 shows the I_c bin ranges with median I_c values corresponding to different bin numbers.

Table 4-3: I_c Bin Ranges

Bin Number	Bin Range	Median I_c
1	0-0.5	0.25
2	0.5-0.1	0.75
3	1-1.5	1.25
4	1.5-2	1.75
5	2-2.5	2.25
6	2.5-3	2.75

Interval III shows a leftward shift in distribution I_c. Pre data show maximum percent data for median I_c of 1.25 while, the post data show the same for median I_c of 0.75, indicating a reduction in I_c values. It should be noted that the number of pre data points in these respective bins do not differ significantly. Interval IV (Figure 4-29 and Figure 4-30) does not show any such change in I_c with treatment. Thus, it can be said that I_c does not change significantly with treatment for the bin size considered. I_c includes the effects of Q, F and B_q and provides an index of soil behaviour type, which can be related to Q. In this thesis, the Q and I_c combination will be used to relate the ground improvement to the material type. Ideally, pre I_c should be used, however, given the fact that I_c does not vary significantly with treatment, post I_c will be used to relate to post Q.

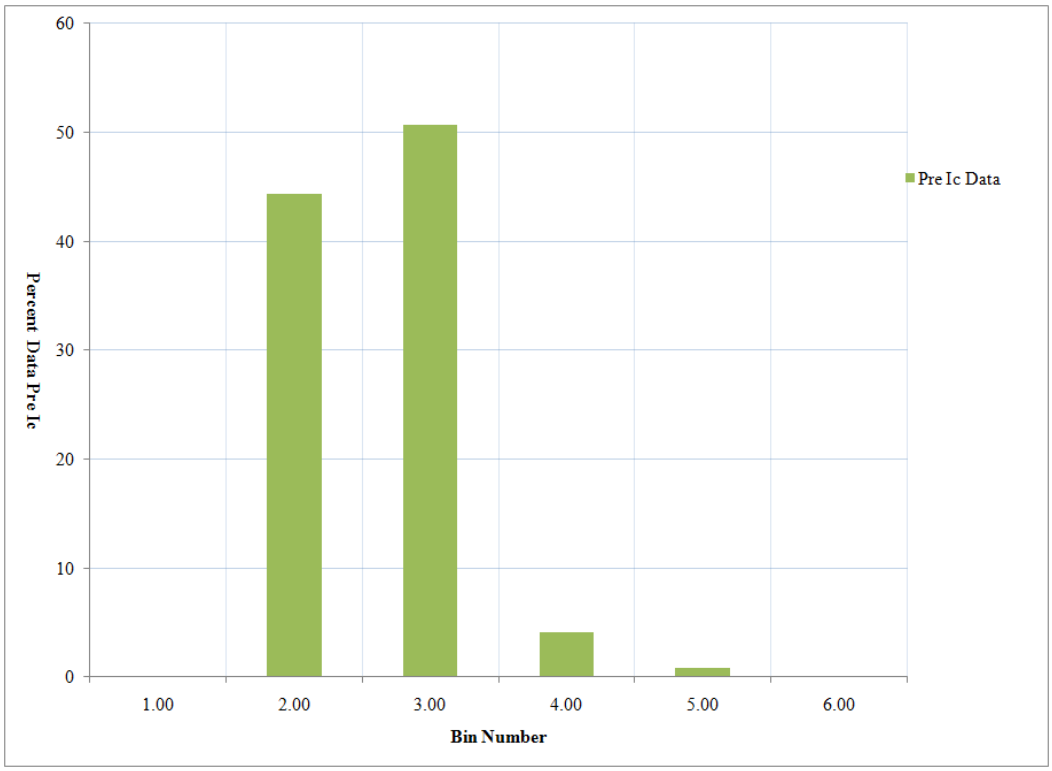


Figure 4-27 : Distribution of Data Points for Pre I_c Interval III (7m-11m)

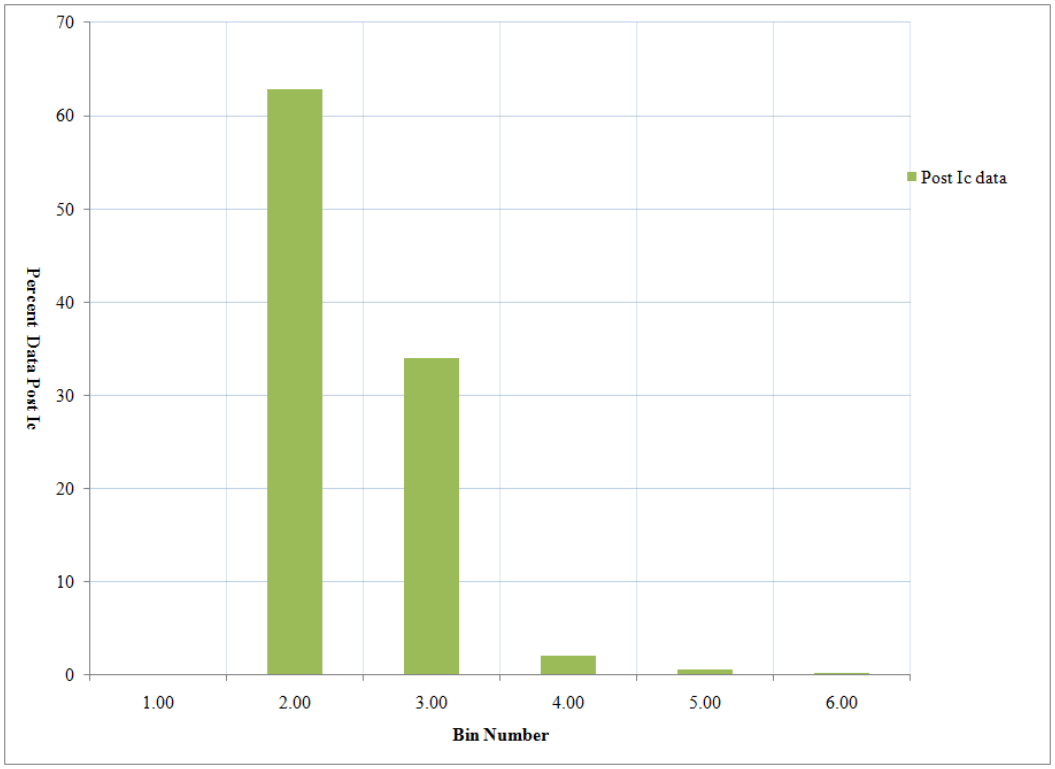


Figure 4-28 : Distribution of Data Points for Post I_c Interval III (7m-11m)

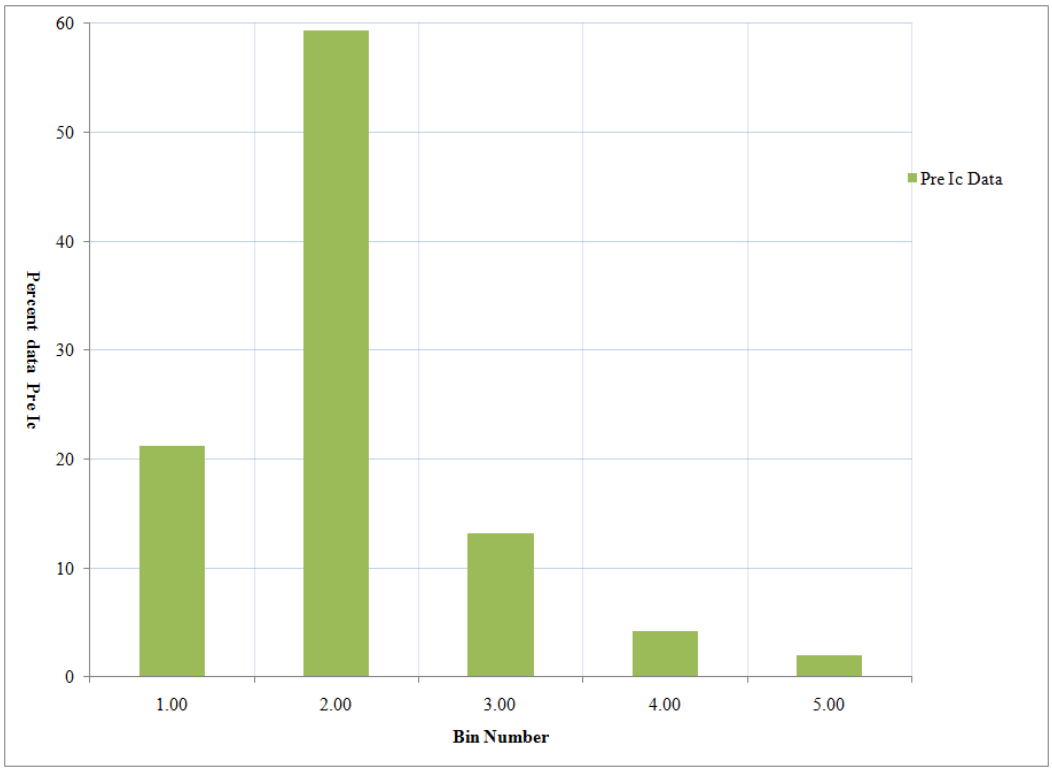


Figure 4-29 : Distribution of Data Points for Pre I_c Interval IV (11m-16m)

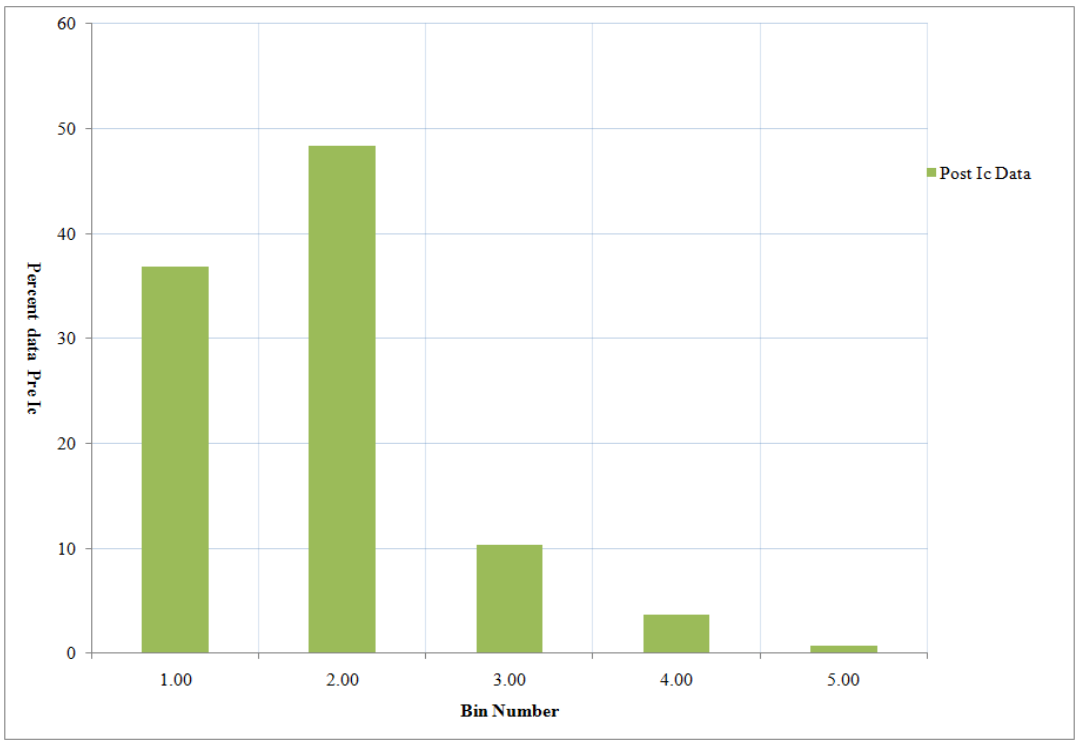


Figure 4-30 : Distribution of Data Points for Post I_c Interval IV (11m-16m)

4.3.4 Determination of Representative Pre Treatment Tip Resistance Using Q-I_c Parameters

A range of Q values for different I_c values for the given depth intervals is determined in this section. To address the variability of measured CPT response, a statistical calculation involving the number of data points, mean, median, and standard deviation for the pre and the post data is suggested. These data are plotted on Q vs I_c plots in the following manner:

(a) Using an Excel spreadsheet, the I_c data for the selected depth range are sorted in ascending order. The data are then divided in equals bins of 0.5. The bin size is so chosen that the variation in Q can be studied for a spectrum of I_c values.

(b) The mean, median, and standard deviation (plotted as the mean value plus and minus one standard deviation) for the ordinate (Q) of the CPT data set, within the selected bins are computed. Standard deviation is a measure of the spatial variability at the site. The mean gives a representative value showing the central tendency of the data set, but its value is affected by the outliers. The median is the middle value of the data set, and is unaffected by the outliers and hence, is a measure of the a typical value from the data set. Standard deviation is a measure of the spread of either the pre or the post data from the mean of the data set, and indicates the confidence in the mean. Standard deviation is plotted as mean plus and minus one standard deviation indicating how far away the sixty eight percent of the values in the data set are from the mean, assuming normal distribution for the data.

(c) The statistical parameters are then plotted against the mid point of the corresponding I_c bin. This mid-point represents the median I_c for the bin.

Ratio of standard deviation to the mean defined as coefficient of variation (COV) is another measure of variability, which relates the standard deviation to the mean and has been used in this thesis for comparing the pre and the post treatment CPT data.

Comparison of mean Q values from different data sets thus involves comparison of number of data points for these data sets, respective COV values indicating the variation, difference between the mean and the median and standard deviations. Similarly, higher COV values

indicate greater variability and hence, less reliability of the related mean. Also, an I_c bin with the highest number of data points indicates the predominant corresponding soil behaviour type on the site. Figure 4-31 to Figure 4-34 show the variation of the pre Q with pre I_c for all the depth zones considered in this analysis. Figure 4-35 shows the Q-I_c variations from different depth intervals plotted together on a semi log plot.

Table 4-4 to Table 4-7 show the trends for Q-I_c variation in each depth interval in terms of mean, standard deviation, and coefficient of variation for each median I_c, with the interpreted soil behaviour type determined using the Jefferies and Davies (1991) approach.

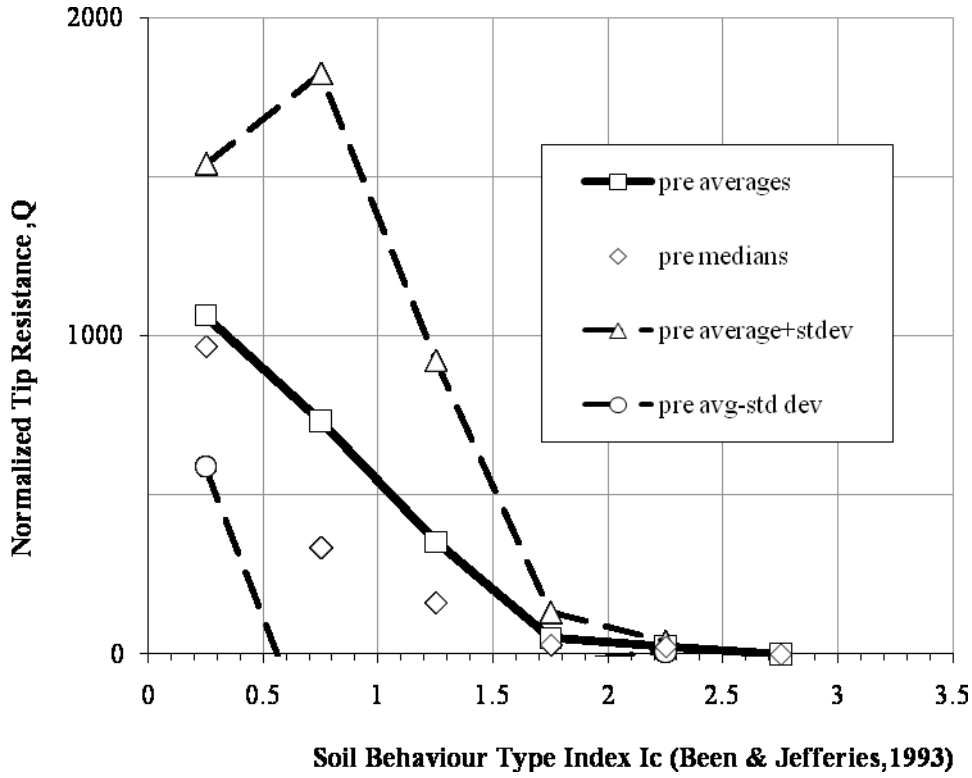


Figure 4-31: Characterization of Pre Treatment Ground Interval I (Top-WT)

The data represent all the ten pre treatment CPT. The number of data points for all the pre I_c bins for each depth zone are given in Appendix C. The data points range from 18 to 1041 for all the I_c bins and depth ranges. Generalized and the depth interval specific trends from the Q-I_c plots and tables are described below. A summary comparing the pre treatment ground

characteristics for different depth intervals is presented at the end of the section. Reference to any interval in the text refers to the related set of Figure and table for that zone. Thus, for example, description of the data in interval III would refer to the Figure 4-33 and Table 4-6.

Table 4-4: Trends from Pre Treatment Characterization of Site: Interval I (Top-WT)

Mean Pre Q	Standard Deviation	COV (%)	Median I_c For the bin	Inferred soil behaviour type (Jefferies & Davies, 1991)
1064	475	44.6	0.25	Gravelly sand to sand
733	1000	148.5	0.75	Gravelly sand to sand
352	570	161.3	1.25	Clean sand to silty sand
50	80	159.1	1.75	Clean sand to silty sand
23	15	64.5	2.25	Silty sand to sandy silt

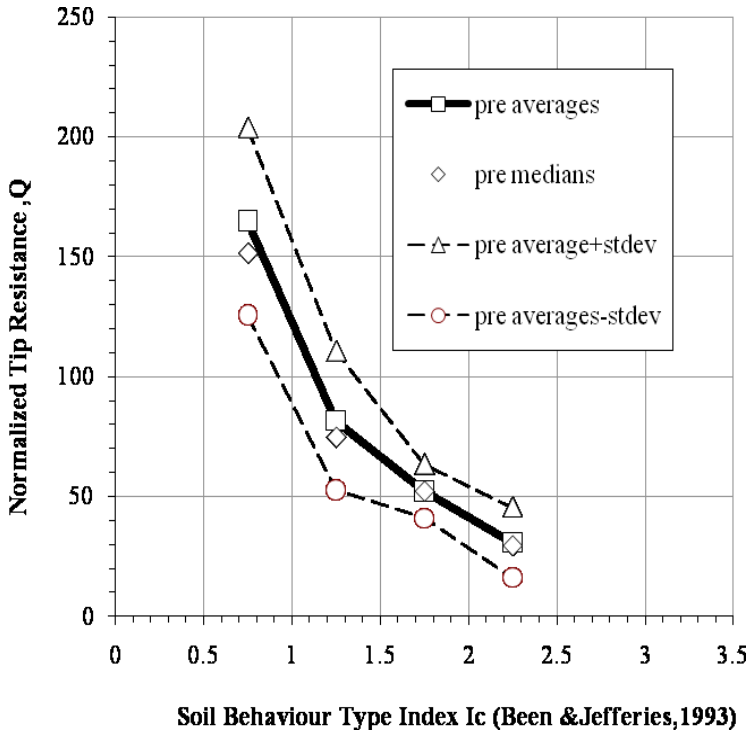


Figure 4-32: Characterization of Pre treatment Ground Interval II (WT-7m)

Table 4-5: Trends from Pre Treatment Characterization of Site: Interval II (WT-7m)

Mean Pre Q	Standard deviation	COV (%)	Median I_c	Inferred soil behaviour type (Jefferies & Davies, 1992)
165	39	23.7	0.75	Gravelly sand to sand
81	29	35.6	1.25	Clean sand to silty sand
52	11	21.5	1.75	Clean sand to silty sand
30	14	47.4	2.25	Silty sand to sandy silt

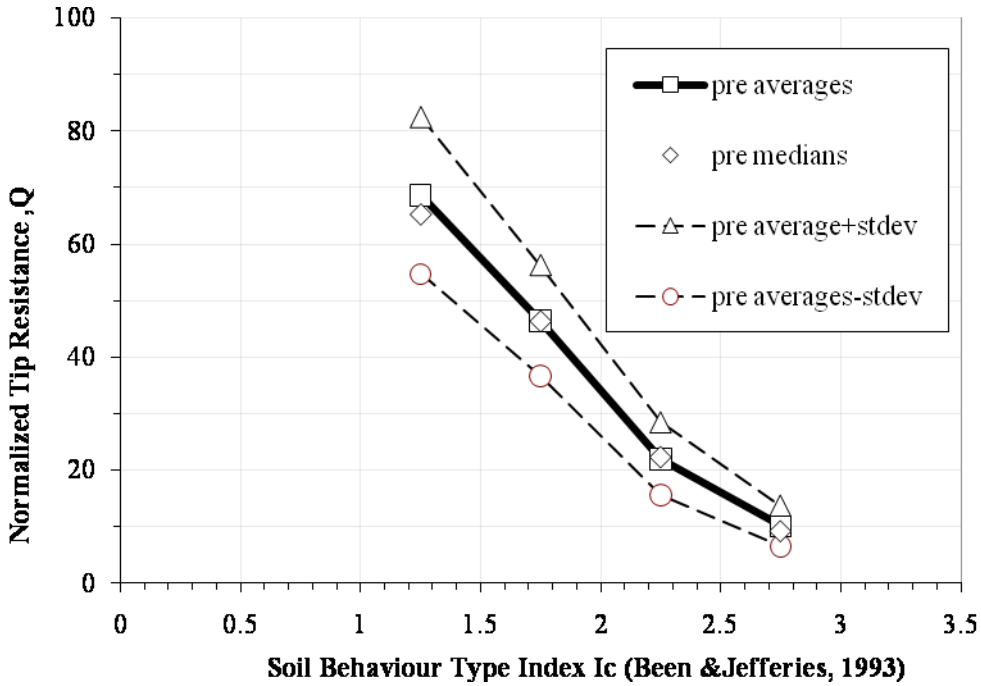


Figure 4-33: Characterization of Pre treatment Ground Interval III (7m-11m)

Table 4-6: Trends from Pre Treatment Characterization of Site: Interval III (7m-11m)

Mean Pre Q	Standard deviation	COV (%)	Median I_c	Inferred soil behaviour type (Jefferies & Davies, 1992)
68	14	20.2	1.25	Clean sand to silty sand
46	9	21	1.75	Clean sand to silty sand
22	6	29	2.25	Silty sand to sandy silt
10	3	35.3	2.75	Clayey silt to silt

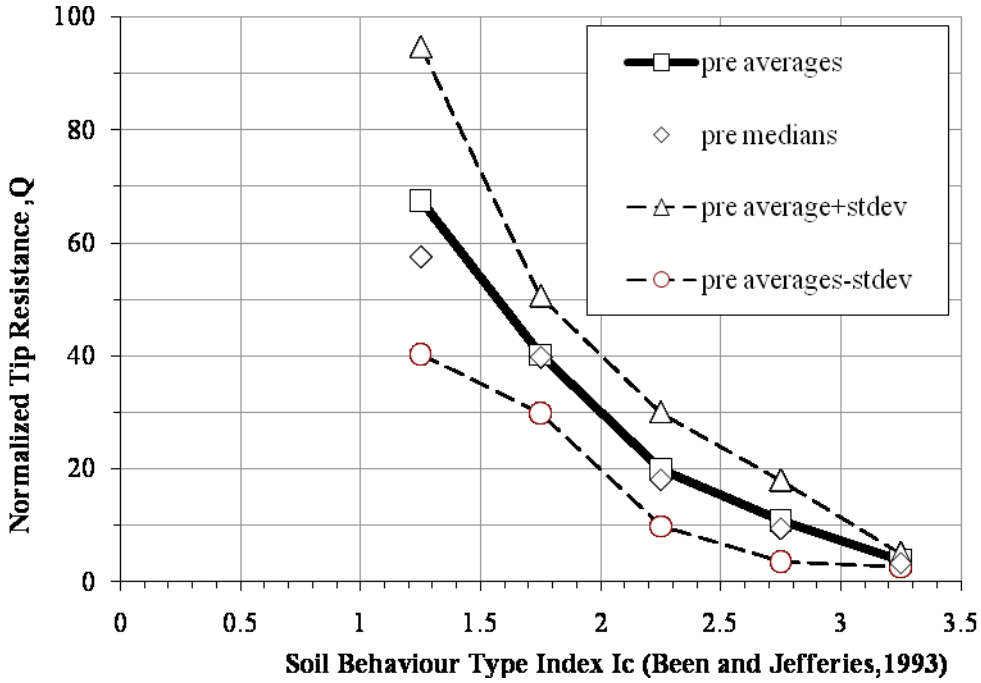


Figure 4-34: Characterization of Pre treatment Ground Interval IV (11m-16m)

Table 4-7: Trends from Pre Treatment Characterization of Site: Interval IV (11m-16m)

Mean Pre Q	Standard deviation	COV(%)	Median I_c	Inferred soil behaviour type (Jefferies & Davies, 1992)
67	27	40.1	1.25	Clean sand to silty sand
40	10	25.8	1.75	Clean sand to silty sand
20	10	50.4	2.25	Silty sand to sandy silt
11	7	66.3	2.75	Clayey silt to silt
4	1	32.1	3.25	Clay

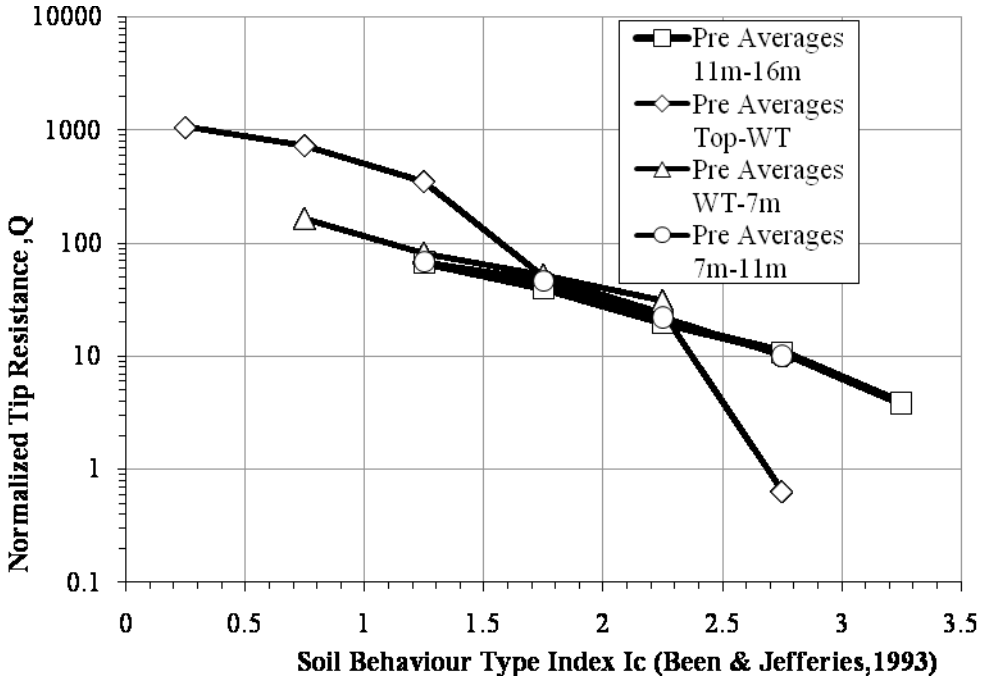


Figure 4-35: Pre Treatment Data for All Depth Intervals

The following generalized and depth interval specific trends are observed from the above plots and tables:

(a) Based on number of data points available for different I_c bins, clean sand to silty sand (I_c-1.25 to 1.75) and silty sand to sandy silt (I_c-2.25) are the two predominant behaviour types observed for all the depth intervals. Top interval (I) also indicates presence of gravel (I_c 0.25-0.75) while the lower ones (III and IV) indicate presence of finer material such silty clay and clay (I_c greater than 2.25).

(b) The top interval in general is characterized by very high Q values, (up to 1000) and scatter (COV up to 160%) compared to other lower intervals. The standard deviations, particularly for I_c 0.75, 1.25, and 1.75 are greater than the respective means making most of the average minus one standard deviation points negative. Negative values are physically unrealistic, and are not shown on these plots.

(c) Lower depth intervals indicate reduced Q for all I_c values (maximum 160) and reduced scatter (COV up to 60%) indicating less variability.

(d) For these intervals, for comparable number of data points and scatter, I_c of 1.25 indicates Q ranging from 67 to 80 across different depth intervals with scatter (COV) ranging from 20 to 35 % respectively. For a higher I_c of 1.75 (similar material behaviour), Q varies from 40 to 52 with COV ranging from 21-25%.

(e) Similarly, I_c of 2.25 (silt sand to sandy silt) indicates Q in the range of 20-30 for all depth intervals, with scatter expressed by COV as 50%-60% respectively.

(f) The material with median I_c greater than 2.25 (up to 3.25) is found in interval III and IV with lack of comparable number of data points for these intervals and hence the means are not reliable.

4.3.5 Determination of Post Treatment Tip Resistance Using Q-I_c Parameters

4.3.5.1 Effect of Aging on Post CPT Data

In this section, the effect of time on the post treatment tip resistance achieved using EC and RIC is shown (aging effects). Available post treatment CPT data in various panels have been sorted for time of conducting a CPT in a panel with respect to the time when the panel was treated, for the following time periods: within a week (3-6 days), a week (8-12 days), three weeks (21-23 days) and a month (43-67 days) after the treatment. CPT data two weeks after the treatment have not been considered owing to few available CPT in that data set. These have been shown in Appendix E. Other factors such as repetition of CPT at the same location have also been considered for CPT selection. Tip resistance values for these have been compared with those from the pre data to study the effect of time on the CPT parameters. The Q (pre and post treatment)-respective I_c framework along with statistical calculations as shown in the previous section has been used to represent the improvement with time for different material behaviour types and three depth intervals (II, III and IV). Figure 4-18 shows considerable scatter for the pre and the post CPT data in the interval I (top-WT), making the comparison difficult. Therefore, this interval has not been considered in this analysis.

Figure 4-36 to Figure 4-38 show the variation of mean pre and the post treatment Q measured at different time periods after the treatment with the respective median I_c (Been and Jefferies, 1993) for the three depth intervals. Table 4-8 to Table 4-10 compare the coefficient of variation (COV) values of the Q data points for CPT conducted at different time periods after the treatment corresponding to the median I_c values of the data points. COV values along with the number of data points for each bin (as given in Appendix C) indicate the reliability of the mean for that bin. A higher COV value is indicative of greater variation in the data set. Similarly, it is important to compare means for data sets with comparable number of data points. A mean of a dataset with too few data points may not be reliable. Effects of aging on the pre and the post treatment CPT parameters were discussed in Chapter 2 (Section 2.2.2.4.4). Initial post treatment (blasting) CPT are expected to show a

decrease or a modest increase in tip resistance over its pre blasting value. The tip resistance increases with time over the pre treatment value (Gohl et al., 2000; Howie et al., 2001).

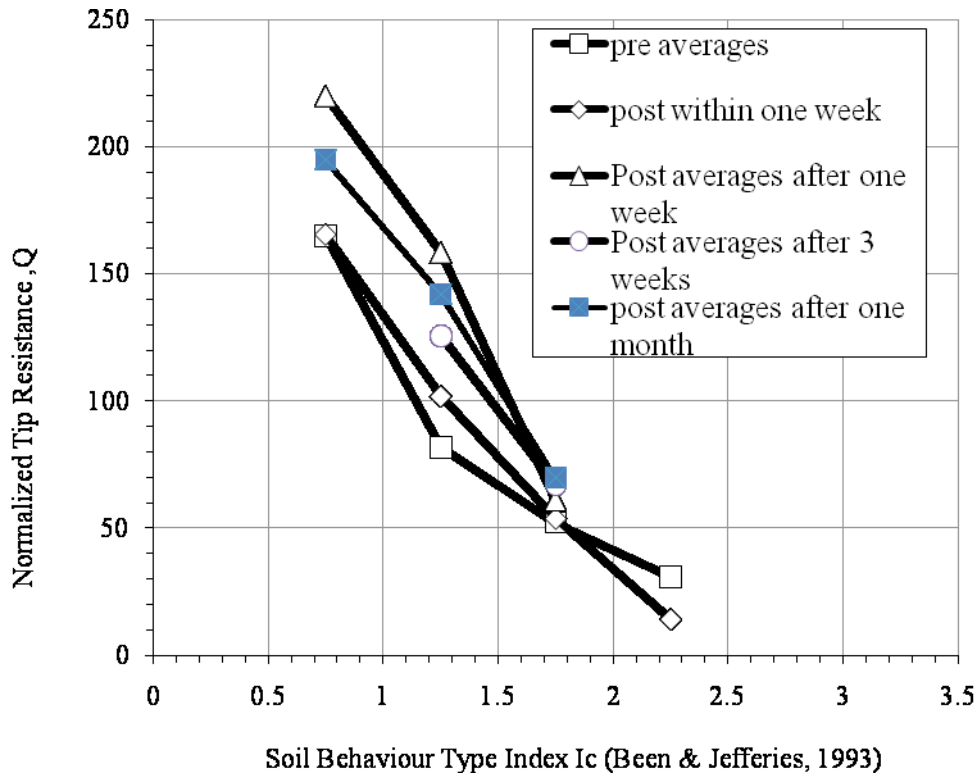


Figure 4-36: Aging Effect Depth Interval II (Water Table-7m)

Table 4-8: Statistical Analysis for Aging Interval II (WT-7m)

Median I_c	Cov (%) Pre Data	Cov (%) Post within one week	Cov (%) Post one week	Cov (%) Post three weeks	Cov (%) Post one month
0.75	23.7	18.1	31.1		16.3
1.25	35.6	29.2	44.8	26.4	28.7
1.75	21.5	30.09	49.9	49.8	23.4
2.25	47.5	17.6			

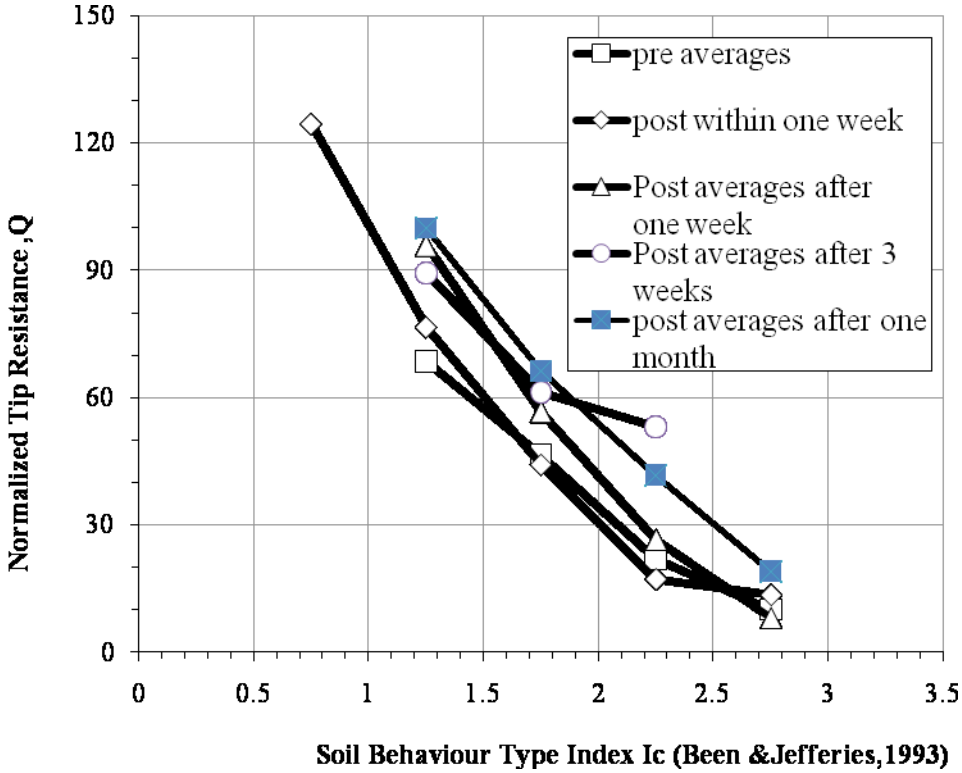


Figure 4-37: Aging Effect Depth Interval III (7m-11m)

Table 4-9: Statistical Analysis for Aging Interval III (7m-11m)

Median I_c	Cov (%) Pre Data	Cov (%) Post within one week	Cov (%) Post one week	Cov (%) Post three weeks	Cov (%) Post one month
1.25	20	27.3	28.5	20.2	23
1.75	21	31.5	35.7	33.4	29
2.25	29	47.3	44.7	22.2	40
2.75	35	44.7	62.7		41

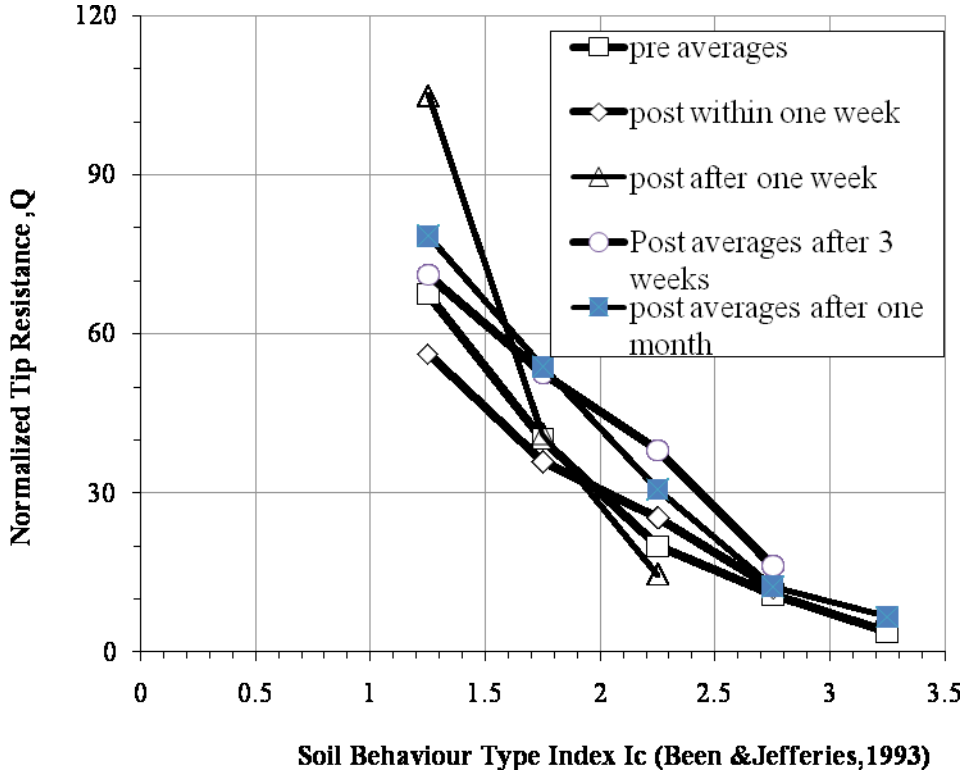


Figure 4-38: Aging Effect Depth Interval IV (11m-16m)

Table 4-10: Statistical Analysis for Aging Interval IV (11m-16m)

Median I_c	Cov (%) Pre Data	Cov (%) Post within one week	Cov (%) Post one week	Cov (%) Post three weeks	Cov (%) Post one month
1.25	40	15.2	19.03	18.8	22
1.75	26	30	87.8	26	31.2
2.25	50.4	41.7	23.3	49	37
2.75	66.3	47.09		26.1	100
3.25	32.1	44.2			89

Following observations are made from the variation of mean Q with time, corresponding COV values and the number of data points in Q bins:

(a) For median I_c values of 2.25 or higher, there are fewer data points (pre and post) with higher COV values than those observed for lower median I_c values. Thus, observations for mean Q values corresponding to these higher I_c value are affected by variability.

(b) Increase in mean Q with time can be seen for all the three depth intervals for different median I_c values. For example, I_c of 1.25 for depth interval II (WT-7m) and IV (11m-16m) and I_c of 1.75 for interval III (7m-11m). The mean Q values corresponding to these median I_c values for most of the time periods are characterized by comparable COV values and number of data points in respective bins.

(c) Means showing inconsistent values (post one week means greater than those for the post one month) are observed for all the depth intervals. These mean Q values are characterized by very high COV and/or fewer data points, indicating the variability in the data set. For example, mean Q post one week for median I_c value of 1.25 for interval II shows a COV of 44%, greater than COV for other times.

(d) Such an inconsistency could be due to either variable soil type or possibly due to effects of concurrent blasting viz. post treatment CPT conducted after a month in a panel adjacent to a recently treated panel may show reduced tip resistance due to destruction of aging effects by the treatment.

(e) For interval II, mean Q values from initial CPT (conducted within a week of the treatment) appear to respond as expected-either lower than the pre means or slightly greater than the pre values. These means are affected by variability in other depth zones.

(f) Means Q values corresponding to higher median I_c values show reduced gain compared to those for lower I_c values, but are characterized by greater variability due to fewer data points and greater COV.

While the data do indicate that there is an increase in Q with time after treatment, the variability of soil conditions at the site affects the data and makes it difficult to derive firm conclusions on the magnitude of the aging effect. Consequently, all further analysis of post treatment CPT data presented in this thesis will ignore the time effect.

4.3.5.2 Assessment of Ground Improvement

A total of forty-three post treatment CPT were sorted to arrive at mean Q values. Figure 4-39 to Figure 4-41 show the variation of the mean pre and the post Q values for all the CPT, for respective median I_c values, for the three depth intervals (II-IV). The pre and post CPT data in the top depth interval showed high variability as evident from the earlier analyses and so this zone has not been considered in the analysis in this section. Table 4-11 to Table 4-13 show the trends from the plots in terms of range of pre and post means, COV, and the percent increase over the pre mean for corresponding median I_c values. The following trends with regard to the variation of post mean values with post I_c are observed:

(a) For median I_c value of 0.75, post Q achieved is 200 in the interval II (WT-7m) and 124 in interval III (7m-11m). In interval III, the lower mean value is characterized by lower scatter in the data indicated by lower COV (4%), but also fewer data points than the interval II value, which although characterized by higher COV (24%), has more data points.

(b) For median I_c value of 1.25, Q shows a maximum value of 134 in interval II and a lower range-76-92 in lower intervals III and IV. The low values in the lower intervals are characterized by lower COV (25%-26%) and comparable number of data points. The high value in the top interval is characterized by higher COV (37%), for similar number of data points. Refer to Appendix C for the number of data points for the post data in different depth intervals.

(c) For median I_c value of 1.75, Q shows a narrow range of 63-47 for intervals II-IV with COV ranging from 40% in interval II to 38% in interval IV. The number of data points being comparable for intervals III and IV.

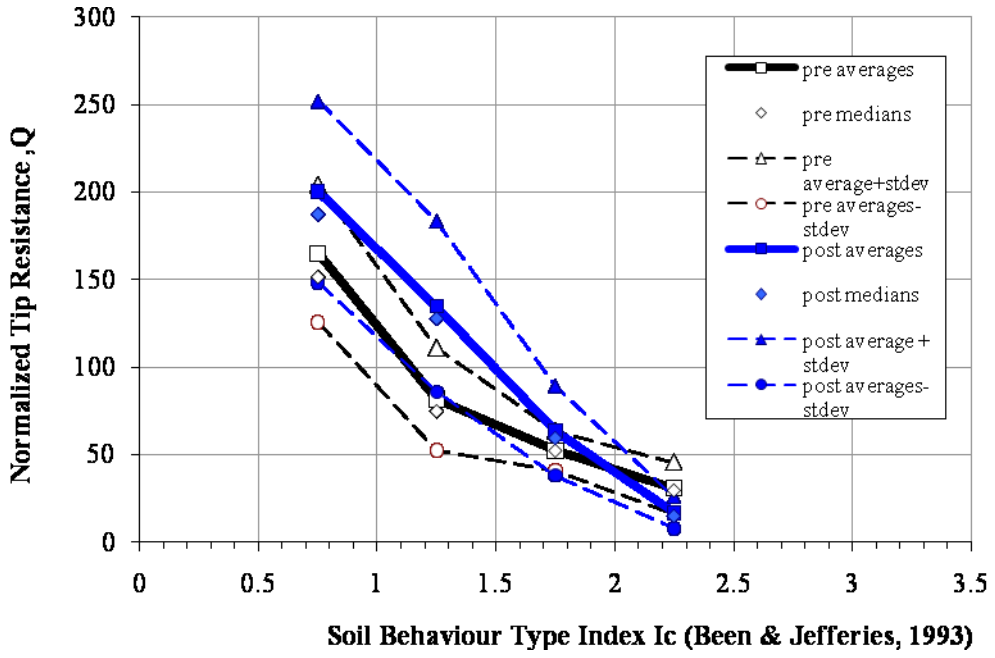


Figure 4-39 : Ground Improvement Trends Intervals II (WT-7m)

Table 4-11: Ground Improvement Trends Interval II (WT-7m zone)

Median I_c	SBT (J&D 1992)	Mean Pre Q	COV %	Mean Post Q	COV%	% increase over pre mean Q
0.75	Gravelly sand to sand	165	24	200	26	21
1.25	Clean sand to silty sand	82	36	134	36	63.4
1.75	Clean sand to silty sand	52	21	63.6	40	22.3
2.25	Silty sand to sandy silt	31	47	16.7	53	---

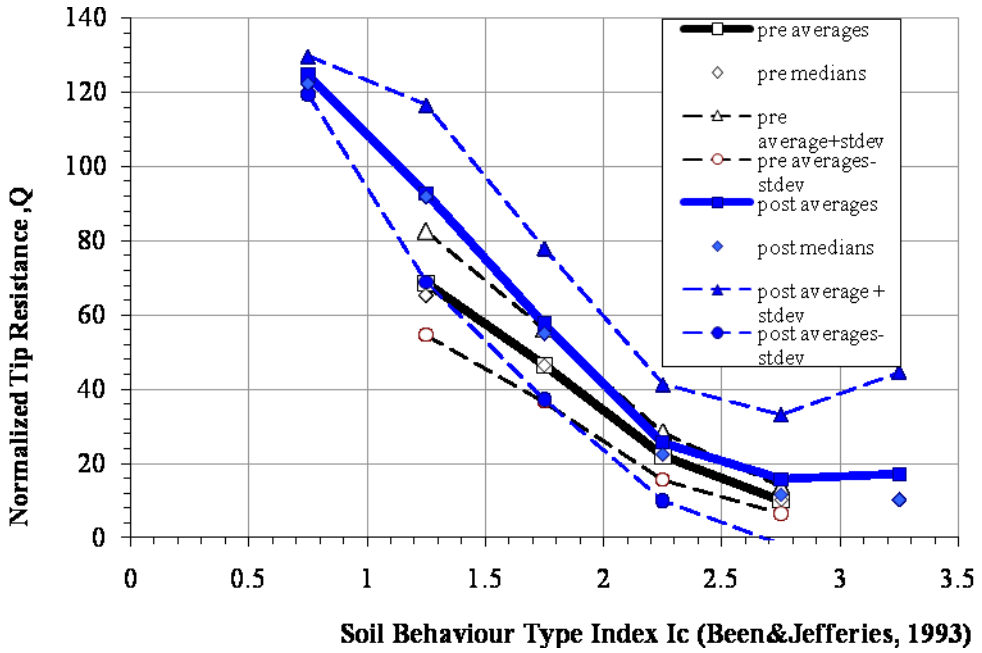


Figure 4-40: Ground Improvement Trends Interval III (7m-11m)

Table 4-12: Ground Improvement Trends Interval III (7m-11m)

Median I_c	SBT (J &D 1992)	Mean Pre Q	COV %	Mean Post Q	COV%	% increase over pre mean Q
0.75	Gravelly sand to sand	---	---	124.6	4	
1.25	Clean sand to silty sand	69	20	92.7	25.7	34.3
1.75	Clean sand to silty sand	46	21	57.6	35.3	25.2
2.25	Silty sand to sandy silt	22	29	25.6	61	16.3
2.75	Clayey silt to silty clay	10	35	15.7	110.8	57
3.25	Organic soils			17.1	160	

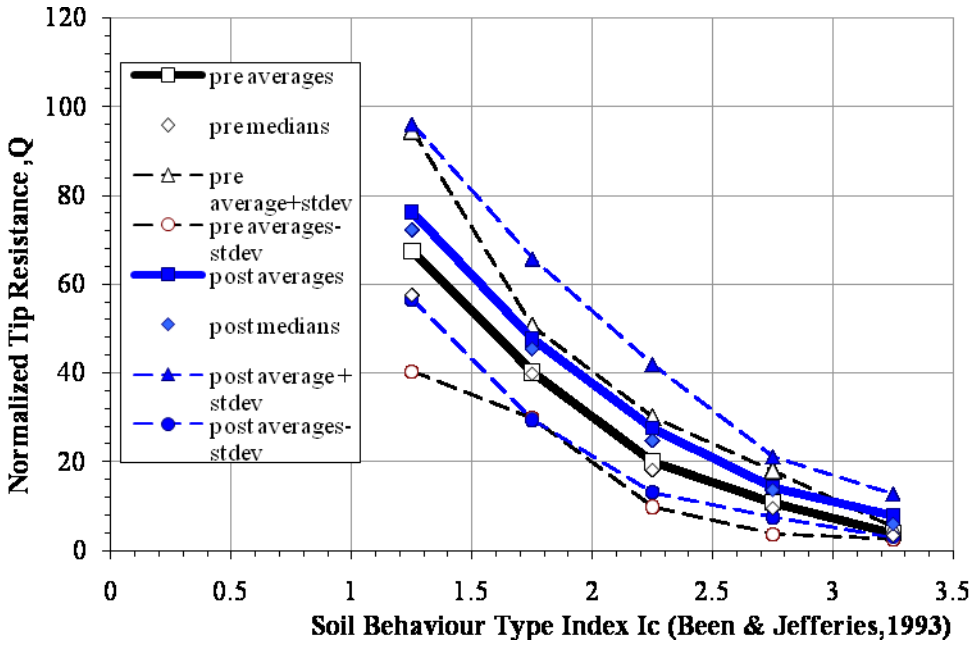


Figure 4-41: Ground Improvement Trends Interval IV (11m-16m)

Table 4-13: Ground Improvement Trends Interval IV (11m-16m)

Median I_c	SBT (J &D 1992)	Mean Pre Q	COV %	Mean Post Q	COV%	% increase over pre mean Q
1.25	Clean sand to silty sand	68	40	76.3	26	12
1.75	Clean sand to silty sand	40	26	47.6	38	19
2.25	Silty sand to sandy silt	20	50	27.5	52	37.5
2.75	Clayey silt to silty clay	11	66	14.3	47	30
3.25	Organic soils	4	32	8	51	100

(d) For median I_c value of 2.25, Q shows a narrow range of 16-27 for intervals II-IV. The values are characterized by COV in the range of 53%-60%.

(e) Material with higher median I_c values 2.75-3.25, classifying as clay and silt mixtures and clays, are found in the lower two intervals viz. III and IV. Q ranges from 8-15 with COV ranging from 51% to 110%. These are characterized by fewer data points.

Table 4-14 presents the pre and the post mean Q values for all the depth intervals and median I_c values with respective with COV values.

Table 4-14: Comparison of Pre and Post Treatment Mean Q Values

Median I_c	Depth Interval	Mean Pre Q	COV %	Mean Post Q	COV %
0.75	II	165	24	200	21
	III			125	4
	IV				
1.25	II	82	36	134	36
	III	69	20	93	26
	IV	68	40	76.3	26
1.75	II	52	21	63.6	40
	III	46	21	58	35.3
	IV	40	26	47.6	38
2.25	II	31	47	16	53
	III	22	29	25.6	61
	IV	20	50	27.5	52
2.75	II				
	III	10	35	15.7	110.8
	IV	11	66	14.3	47
3.25	II				
	III			17.1	160
	IV	4	32	8	51

Figure 4-42 to Figure 4-44 show the trendlines drawn through the mean pre and the post data points on Q-I_c plot for each depth interval. The lines and their equations, R^2 values etc. have been drawn using the functions in Excel spreadsheet.

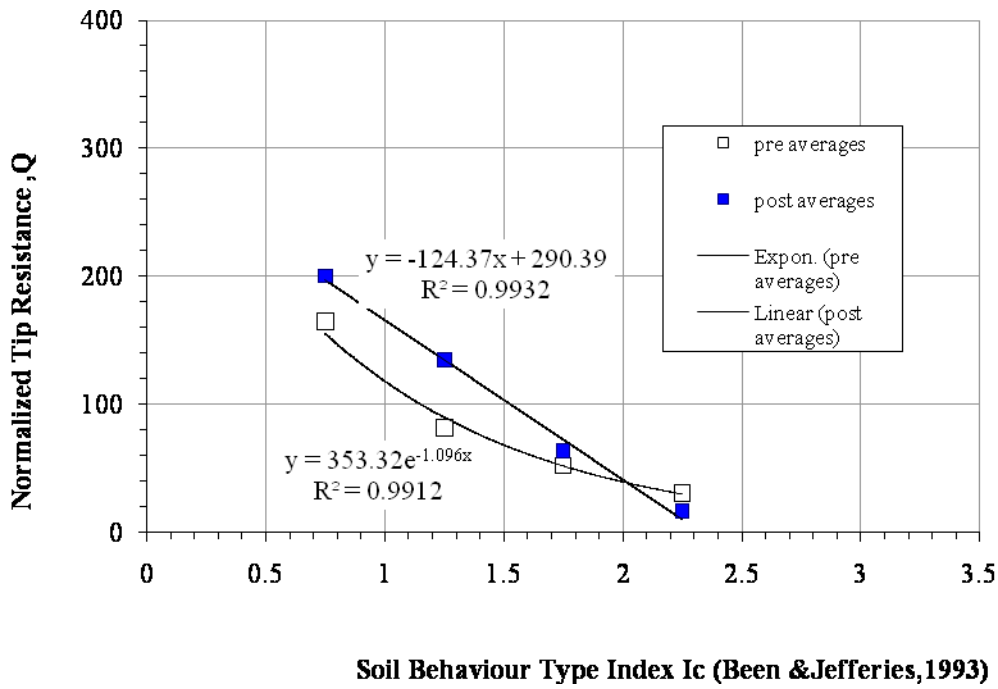


Figure 4-42: Mean Pre and Post Q for Interval II (WT-7m)

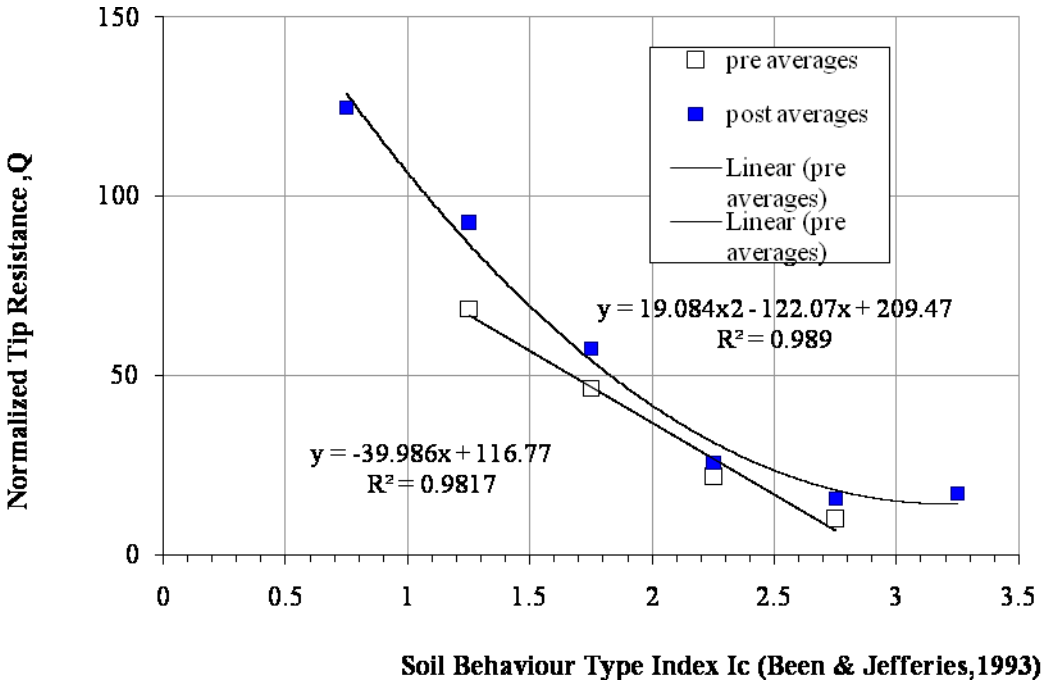


Figure 4-43: Mean Pre and Post Q for Interval III (7m-11m)

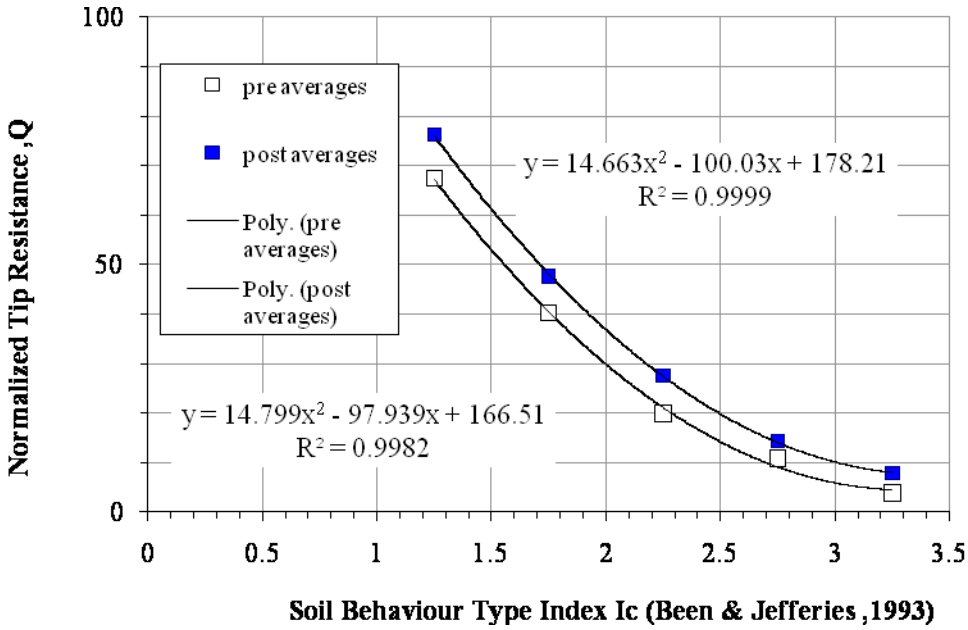


Figure 4-44: Mean Pre and Post Q for Interval IV (11m-16m)

It is observed from figures that the polynomial and exponential fit give a good R^2 value for both the pre and the post data.

4.4 Summary

4.4.1 Overview

In this chapter, the site investigation data (provided by GAIA, 2004) including borehole logs, SPT values, fines content and the pre and post treatment CPT data were used for characterization of the site at Sudbury. Both, borehole and the CPT data showed the presence of rock fill material in the top 1.5m-2m zone (CPT classifies this material as gravelly sand to silty sand) underlain by mainly sand and silt sized tailings along with natural clay, clay sized tailings, and organic soils at greater depths. Considerable site

variability was indicated with a general observation that the material type became finer in deeper zones. Observations from the Soil Behaviour Type (SBT) chart (Jefferies & Davies, 1991) used for CPT based classification of tailings were in agreement with those from the boreholes confirming the validity of the chart for classification of Sudbury tailings.

To account for the site variability, all the pre and the post treatment CPT carried out on the site were considered in the analysis. Normalized CPT parameters were used in the statistical calculations. The parameters were chosen so that the ground improvement could be related to the material type for different depth intervals. Depth intervals were chosen to reflect the effect of combination of treatments carried out on the site towards ground improvement..

Normalized CPT parameters were chosen based on the SBT chart to relate ground improvement (difference between pre and post treatment normalized tip resistance Q) to the material behaviour type (indicated by soil behaviour type Index $I_{c)}$. The effect of time after ground improvement on CPT response was also shown. Table 4-14 compares the mean pre and post Q values for different depth intervals and for different median I_c values.

4.4.2 Discussion of Results: Use of Soil Behaviour Type Chart

Interpretation of the CPT data was carried out by using the soil behaviour type chart (SBT) proposed by Jefferies and Davies (1991). The same SBT chart has also been used for screening of the data in the form of soil behaviour type index I_c, where all the CPT parameters (measured and inferred) including I_c were plotted with depth. However, plotting the CPT data directly on the chart for a given depth interval allows for a more comprehensive interpretation of the behaviour type.

Since the identified behaviour type from the charts is in agreement with the soil type from the borehole logs, the equivalent soil type for this site is established. SBT chart was also the basis for choice of normalized CPT parameters used for representing the ground improvement. Q-I_c combination was used as I_c is an algebraic combination of all the normalized CPT parameters and that itself presents an advantage as it includes the effect of all the measured CPT parameters.

4.4.3 Discussion of Results: Post Treatment Tip Resistance Achieved Due to Treatment

Table 4-14 compares the mean pre and the post treatment normalized tip resistance (Q) for respective I_c values and for different depth intervals. A number of factors affecting the mean post Q achieved in these tailings such as the use of combined treatments, effective depth of a treatment, tailings type and aging are discussed in this section. Effects of some of these factors (treatment type, effective depth and tailings type etc.) on the tip resistance can be better studied by plotting the mean pre and post treatment Q data with depth. Figure 4-45 shows the mean pre and the post treatment Q values plotted with depth and statistical calculations and predominant behaviour types (as determined by data plotted on SBT charts) in each depth interval. Table 4-15 compares the range of the pre and the post Q values along with the corresponding ranges of COV values for each depth interval. Appendix C shows the number of data points corresponding to the above table.

Table 4-15 : Comparison of Mean Pre & Post Treatment Q at Different Depths

Depth Zones	Pre Q Range	Pre COV %	Post Q Range	Post COV%
WT-7m	61-94	23-45	98-149	39-40
7m-11m	44-56	30-40	63-78	32-42
11m-16m	40-42	54-66	50-52	46-50

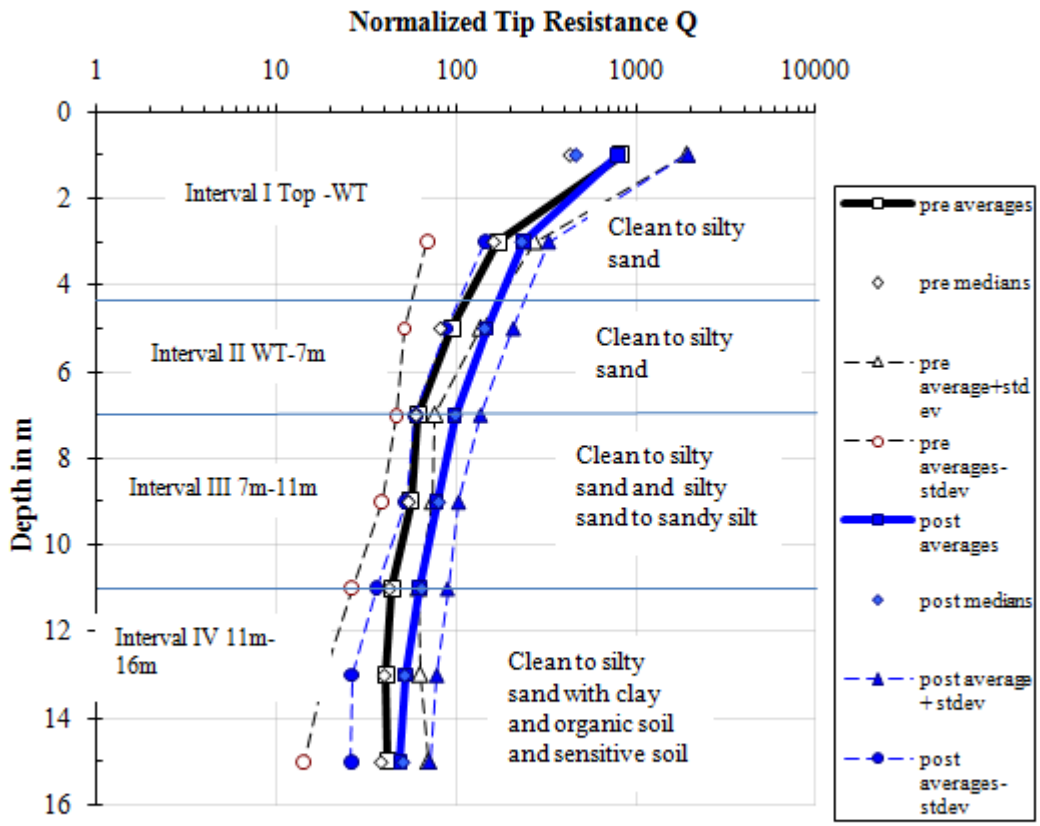


Figure 4-45: Variation of Mean Pre and Post Q with Depth

In order to further highlight the effect of soil type on the achieved tip resistance, variation of mean post Q has been plotted with depth for different I_c values. Figure 4-46 shows the variation of the mean plus and minus one standard deviation of post Q values for median I_c values of 0.75, 1.25, 1.75 and 2.25 with depth.

4.4.3.1 Effect of Material Type

The Q-I_c relation, for both pre and post data shows the effect of material type represented by I_c on the tip resistance Q. The Q achieved by the treatment reduces with increasing I_c. Figure

4-45 shows a reduction in post treatment Q with depth for the lower depth intervals with presence of higher I_c material. Figure 4-46 confirms above observation.

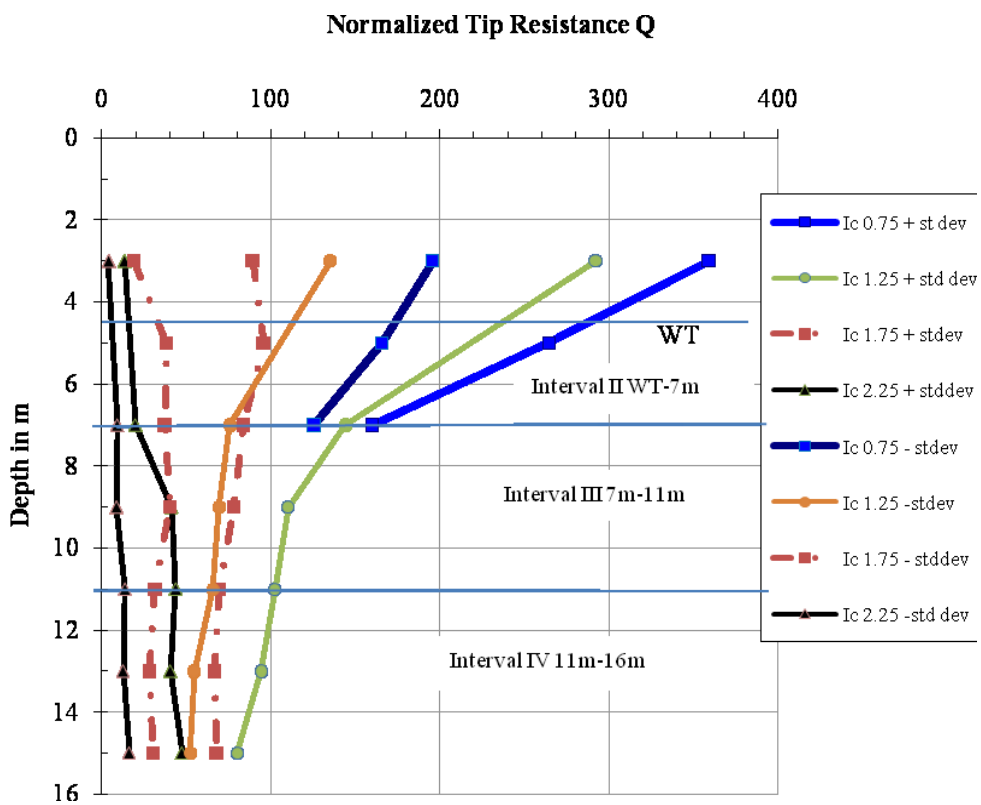


Figure 4-46: Variation of Mean Post Treatment Q with Depth for Different I_c Values

4.4.3.2 Effect of Treatment Type

All of the post treatment CPT considered in the analysis record the effects of application of EC and RIC. RIC was employed to densify the shallow deposits (up to 7m) and particularly the ground above the water table. Considerable variability (very high mean pre Q values, of the order of 1000 with COV of 160%) has been observed in the depth interval above the water table in the pre and the post treatment tip resistance values. Thus, a comparison of the

pre and the post CPT data was not possible and the effect of treatment above water table could not be verified.

Figure 4-46 shows the range of mean post treatment tip resistance recorded in different depth intervals for different material behaviour types. There is no data available for average I_c of 0.75 for the intervals below 7m. For I_c of 1.25, a higher value of mean Q for the interval II (WT-7m) is observed compared to other lower intervals. Mean Q for I_c of 1.75 shows similar increase for interval II while I_c of 2.25 shows an increase in mean Q for lower depth intervals. The increase shown by I_c of 0.75-1.25 in the interval II is marked by higher standard deviation, as indicated by wider range of mean Q values. This increase shown by I_c of 2.25 in the lower depth intervals is also characterized by high standard deviation values indicating greater variability. It was also shown in the previous analyses (aging and entire post CPT data) that fewer data points are available for this I_c value. Variations in the mean post treatment Q values across different intervals (such as II and III) can be attributed to the effect of treatment type (RIC and EC for interval II against EC only for interval III) provided the range of pre values for the respective intervals and similar behaviour types is not varying. Table 4-14 shows that mean pre Q values across different intervals for similar behaviour types (indicated by I_c) are comparable (for example I_c of 1.75 in interval II and III for similar COV ranges). Thus, a combination of EC and RIC has resulted in higher tip resistance in interval II. However, this observation is characterized by greater site variability in interval II than that observed in zone III as shown by the wider range of mean post Q data in the upper zone.

4.4.3.3 Aging Effects

Variation (increase) in mean post Q values with time was observed for all the depth intervals. The observations for aging effects were affected by site variability. The results of this analysis pertain to a limited number of post treatment CPT available for each time period and from a particular zone on the site. Therefore, the ground improvement was worked out using all the valid post treatment CPT irrespective of time from the entire northern portion of Guindon dam.

4.4.3.4 Performance of Different Ground Improvement Treatments Used on Site

Table 4-16 is a revised version of Table 4-14 and presents the mean pre and the post Q values for different median I_c values and depth intervals as per the earlier Q-I_c analysis.

Table 4-16: Normalized Tip Resistance Achieved for Different I_c Values

Median I_c	Inferred Soil Behaviour Type (Jefferies & Davies, 1991 & 1993)	Mean Pre Q Range	COV%	Mean Post Q Range	COV %
0.75	Gravelly sand to sand	165	24	125-200	4-21
1.25	Clean sand to silty sand	68-82	40-36	76-134	26-36
1.75	Clean sand to silty sand	40-52	26-21	48-64	38-40
2.25	Silty sand to sandy silt	20-31	50-47	27-16	52-53
2.75	Clayey silt to silt	10-11	35-66	16-14	110-47

The ground improvement achieved in terms of normalized tip resistance for the present case due to application of EC (mean post Q values in lower depth intervals- 7m to 16m) is compared with that achieved previously in soils and tailings due to EC. Table 4-17 compares the normalized tip resistance achieved in various documented cases of EC treated soils and tailings to the average results from the Guindon Dam (based on Table 4-14).

Table 4-17: Summary Normalized Tip Resistance Achieved due to Blasting in Soils and Tailings

Sr. No.	Case History/Collective description	Soil Type/Behaviour Type	Q Achieved	Mean Pre Q
1	Fordham et.al. (1991)	Sand Tailings	57-80***	
2	Various Cases Gohl et.al. (2000)	Sand tailings	70-80	
3	Various cases Gohl et.al. (2000)	Natural Sands	Q >70 , up to 145	
4	Sudbury Tailings**	Clean sand to silty sand Silty sand to sandy silt Clayey silt to silt	47-93 25-27* 14-16*	40-68 20-22 10-11*

*Mean values marked by high COV values

** Data from lower depth intervals (7m-16m), not carrying effects of RIC

*** Tip resistance normalized with respect to a pressure of 1atmosphere.

Tailings in (1) and (2) have been reported to be non plastic and are reported to have an average relative density in the range of 35% to 60%. The specific gravities range from 1.95 (for the fly ash case) to 2.65 for sand tailings. Natural soils reported by Gohl et al. (2000) have relative densities in the similar range with specific gravities ranging from 2.5 to 2.6. Sudbury tailings (Chapter 3) are reported to have relative densities ranging from 40% to 60% with a higher specific gravity range (about 3). Based on these properties, Sudbury tailings appear to be heavier and denser compared to the material densified in above cases. The above table shows that the achieved tip resistance is comparable to the results in documented cases. The predominant behaviour type on Guindon dam was clean sand to silty sand with I_c (1.25-1.75). This material has shown good performance compared to the material documented in the literature, which is primarily clean sand.

5 SUMMARY CONCLUSIONS AND RECOMMENDATIONS

This chapter summarizes the objectives of the thesis, background of the case, site data obtained, and the approach adopted for the analysis of the data. Conclusions are presented based on the results of the analysis and the evaluation of these results. At the end, limitations of the research carried out in this thesis have been highlighted, and recommendations have been made with regard to the future use of EC, and its assessment for tailings dams.

5.1 Summary

INCO contracted Golder Associates Ltd (Golder) in year 2000 to provide recommendations for raising the Guindon and R1-CD dams in their Central Tailings Facility in Copper Cliff, Sudbury. Golder carried out two geotechnical investigations in 2000 and 2001 comprising borehole logs, piezo cone penetration tests, dynamic cone penetration testing and geophysical investigation. Based on the results of these investigations, Golder concluded that the dam raise was going to be founded on loose saturated tailings which were susceptible to liquefaction in the event of a design earthquake. After reviewing a number of available options for preventing the instability during the raise, INCO decided to improve the tailing foundations. They contracted Golder Associates Innovative Applications (GAIA) in 2004 to execute the densification work. Based on the performance of two test blasts carried out on site, INCO decided to use EC for densification of the deeper tailings deposits (>12m), and surface compaction in the form of dynamic compaction (DC) and rapid impact compaction (RIC) for the shallower zones (up to 7m), to achieve the required ground improvement.

GAIA adopted a work plan which included pre and the post treatment geotechnical site investigation and execution of the design treatment on the site. The results of the site investigation program (GAIA, 2004) comprising raw pre and the post treatment CPT, borehole logs, results from the piezometers and settlement markers, and details of the treatment design along with results of previous site investigations (Golder, 2002) were made

available to UBC with a broad objective of assessing the effectiveness of ground improvement. This thesis examined all of the data. The emphasis however was on the pre and the post treatment CPT data (GAIA, 2004) for determining the level of tip resistance achievable in tailings with different grain sizes following application of EC.

The analysis began with processing of the raw CPT data. Data associated with the boreholes such as log descriptions, SPT data, and fines content were screened for trends in terms of tailings /soil type (as indicated by the boreholes) and tailings behaviour type as indicated by the pre treatment CPT. No clear trends in terms of location and extent of soil/tailings type were obtained and considerable site variability was observed.

In view of the site variability, for a reliable assessment of ground improvement, all the pre and the post CPT were considered in the analysis along with statistical calculations to identify global trends for data. The ground was divided into four depth intervals to assess the effect of different treatment types (EC and RIC) and their intensities on the tip resistance achieved. Tailings classification was carried out for different depth intervals using the Soil Behaviour type (SBT) chart (proposed by Jefferies & Davies, 1991) and it was found that the behaviour type interpreted using the SBT chart showed good agreement with the soil type shown by the borehole logs. Based on the usefulness of the SBT chart for classification of tailings, normalized CPT parameters viz. normalized tip resistance Q and the soil behaviour type Index I_c (Jefferies and Davies, 1991; Been and Jefferies, 1993) were used along with the statistical calculations involving the mean, median and standard deviation of the Q, for a median I_c value for ground characterization. The Q-I_c framework was used for the identified depth intervals to compare the pre and the post CPT data, and determine the achievable tip resistance due to EC and other treatments for different material types and in light of the effect of factors such as ageing.

The quality of data obtained from settlement markers and piezometers was affected by the factors such as quality of instrumentation and has been used as a qualitative indicator of the volume change and hence ground improvement (settlement of the order of 4%-15% and piezometer indicating generation and dissipation of excess pore pressure).

5.2 Conclusions

The following conclusions are presented:

(a) For the type and combination of improvement methods employed (EC and RIC) on site, an increase in mean normalized tip resistance Q over the pre mean value has been achieved for the range of grain sizes. Chapter 4 shows the level of the post treatment tip resistance achieved in different grain sizes characterized by I_c for different combinations of treatments employed on site viz. EC and RIC.

(b) Effectiveness of EC and RIC could not be verified for the top depth interval (Surface-water table) due to site variability.

(c) For depth interval II (WT-7m), the treatment has resulted in a higher mean normalized tip resistance than that achieved in lower zones for different behaviour types (I_c of 0.75-1.75). This can be attributed to the combined use of EC and RIC. This increase, however, is hard to define due to greater site variability in the top depth interval than that observed for the lower intervals.

(d) The effectiveness of this combination has decreased for the deeper intervals (7m-16m) as evident from the reduced mean post normalized tip resistance observed in these intervals compared to other upper intervals. A lower percent increase over pre treatment resistance in these zones is due to the intensity of EC and the increasing finer behaviour type.

(e) The EC and RIC combination is not found to be effective for behaviour types greater than I_c of 1.25. The mean normalized tip resistance achieved for I_c of 1.75 in interval II is not significantly greater than that achieved in lower intervals suggesting the applicability of RIC for geo-material with I_c of 1.75. The other behaviour types (I_c-0.75 to 1.25) show significant increase in interval II compared to other lower intervals.

(f) It is observed that in spite of higher presence of finer material (I_c of 1.75-2.75) on site, use of EC has resulted in a mean Q value comparable to that observed for clear coarser material.

(g) CPT data interpretation for the Sudbury tailings was carried out using a soil behaviour type (SBT) classification chart (Jefferies and Davies, 1991). The chart was found useful for the classification of tailings behaviour based on the comparison of the interpreted behaviour type and the material type indicated by borehole logs. However, the CPT data for Sudbury tailings show a very low value of B_q, making Q $(1-B_q)$ interchangeable with Q for a majority of the site.

(h) A comparison of the magnitude of achieved mean Q in these tailings with that achieved for natural soils using similar ground improvement methods (EC along with a surface compaction technique) indicates soil-like behaviour of these tailings.

(i) Effect of time dependent gain in strength (ageing) on the post treatment CPT data was shown. Aging affected the post CPT data on this site.

(j) Degree of site variability, as determined from the mean, median, standard deviation and coefficient of variation for the CPT parameters, has stayed the same or has marginally increased after the treatment.

5.3 Recommendations for Future Work

The following recommendations are made with regard to the assessment of EC in tailings in respect of future use of EC in tailings:

(a) Identification of zones of fine grained material on the site can be achieved by conducting CPT dissipation tests at different depths. The time required for 50% or a certain percent of full dissipation, gives an indication of the soil permeability and hence, the soil type. This may also help optimize the charge design for that zone.

(b) The post treatment CPT program must account for the sequence of application of treatment on site. This means that the timing of a CPT should not coincide with the timing of treatment in nearby area on site, so that these CPT do not record the de structuring of the soil due to concurrent treatment in nearby areas.

(c) Field instrumentation (piezometers and settlement markers) must be properly secured or maintained during the treatment, so that the quality of the data from these instruments is consistent.

(d) The piezometers should not only record the pore pressures generated in an area where the treatment was carried out, but also record the pore pressures generated by the treatment elsewhere on site. This helps in establishing the stress wave that propagates as a result of blasting and is an indicator of the successful application of the multiple blasting.

In order to improve the CPT based assessment in tailings, it is important to continue the process of validation of the existing approaches (mainly developed for soils) for tailings or develop new CPT based interpretation approaches in tailings by widening the existing database for tailings.

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APPENDIX A: LIST OF BOREHOLE LOGS IN GUINDON DAM NORTH

Six boreholes were drilled after the ground treatment. The logs show soil description results of grain size distribution (percent fines), SPT blow counts and standpipe piezometer readings.

The boreholes listed are variably spaced on the site. Table A- 1 gives the panel number, date of borehole taken, and the ground water table as given in each bore log. All the borehole logs were taken after the treatment. The actual borehole logs follow the table.

Table A- 1: Summary of Borehole Logs in Guindon Dam North

Panel No.	Borehole No.	Date of Boring	Water Table (m) (As read from piezometers)
42	BH04-P42-3	2nd September 2004	4.5
43	BH04-P43-3	26th August 2004	5.5
44	BH04-P44-3	14th September 2004	4.4
49	BH04-P49-2	2nd September 2004	4.5
50	BH04-P50-1	2nd September 2004	4.8
51	BH04-P51-2	14th September 2004	4.5

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P42-3

SHEET 1 OF 2

LOCATION:

BORING DATE: Sept. 2, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp W Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Ground Surface		0.00							
1											
2					1	51 DO	13	50.1			
3					2	51 DO	13	18.8			
4		Compact, moist to wet, black to dark grey, fine to medium SAND TAILINGS, some to trace silt.									
5					3	51 DO	15	20.5, 30.5			
6					4	51 DO	16				
7											
8					5	51 DO	24	23.2			
				8.38							
9		Compact, wet, grey, sandy SILT TAILINGS, occasional sand seams, trace to some clay.			6	51 DO	14	54.5			
10		*CONTINUED NEXT PAGE*									

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

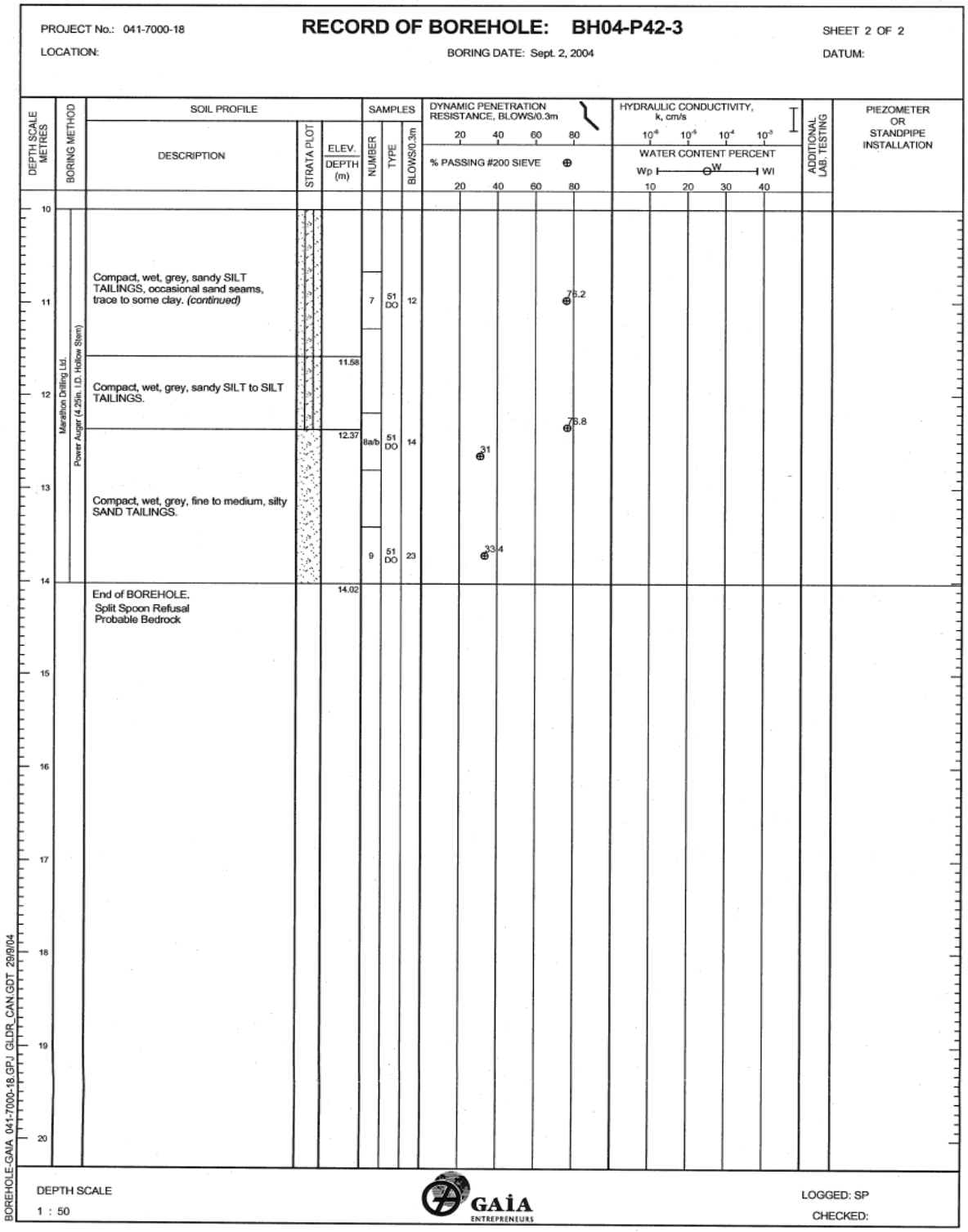

Figure A- 1 : Borehole BH04- P42-3

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P43-3

SHEET 1 OF 2

LOCATION:

BORING DATE: Aug. 26, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / % PASSING #200 SIEVE	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		0.00							
0–5	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Compact to dense, moist, grey-black, fine to medium SAND TAILINGS, trace to some silt.			1	51 DO	17	24.4			
					2	51 DO	30	20.7			
					3	51 DO	20	16.1			
				5.03				72.4			
5–10		Compact, wet, grey SILT TAILINGS, trace to some sand layers/pockets of sand, silty sand.			4	51 DO	21	74.3			
					5	51 DO	25	31.6			
					6	51 DO	21	70.1			
		CONTINUED NEXT PAGE									

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

PROJECT No.: 041-7000-18

LOCATION:

RECORD OF BOREHOLE: BH04-P43-3

BORING DATE: Aug. 26, 2004

SHEET 2 OF 2

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp W Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Compact, wet, grey SILT TAILINGS, trace to some sand layers/pockets of sand, silty sand. *(continued)*		10.36							
11					7	51 DO	29	32.1			
12		Compact, wet, dark grey black, fine to medium SAND to silty SAND TAILINGS, trace to some silt.			8	51 DO	28	29			
13											
14					9	51 DO	20	27.1			
15		Compact, wet, grey SILT TAILINGS, some peat at base.		14.94	10	51 DO	17		80		
16		End of BOREHOLE. Refusal - Probable Bedrock.		15.70							
17											
18											
19											
20											

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

Figure A- 2: Borehole BH04- P43-3

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P44-3

SHEET 1 OF 2

LOCATION:

BORING DATE: September 15, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	% PASSING #200 SIEVE (20 40 60 80)	WATER CONTENT PERCENT (Wp W Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Marathon Drilling, Power Auger - 107mm I.D. Hollow Stem	Ground Surface		0.00							
1		ROCKFILL.		1.52							
2		Compact, brown, fine, silty SAND to sandy SILT TAILINGS.		2.74	1	51 DO	21	30			
3		Compact, dark grey, fine to medium, silty SAND to SAND TAILINGS, some to trace silt, occasional silt seam.			2	51 DO	19	27.7			
4											Sep.14/04 ▽
5					3	51 DO	26	22.8			
6					4	51 DO	28	20.5			
7				7.32							
8		Compact to very stiff, grey, interlayered, clayey SILT, SILT and sandy SILT TAILINGS, occasional sand seams.			5	51 DO	23		65.4		
9					6	51 DO	17		66.4		
10		*CONTINUED NEXT PAGE*									

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80)

HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3})

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: J.T.

CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

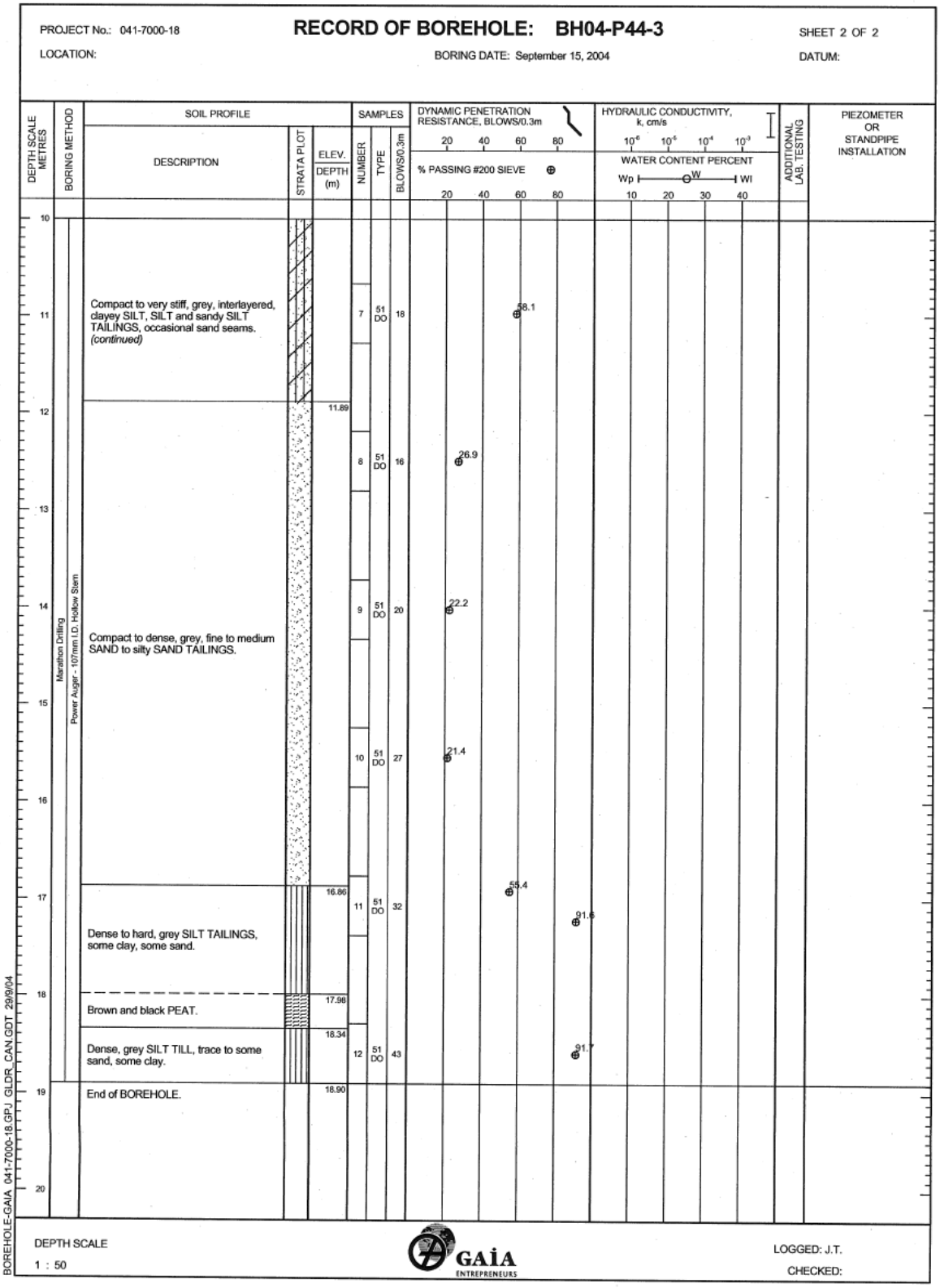

Figure A- 3: Borehole BH04- P44-3

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P49-2

SHEET 1 OF 2

LOCATION:

BORING DATE: Sept. 2, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp — W — Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		0.00							
0–1	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	ROCKFILL.		1.52							
1–2		Compact, moist, brown, sandy SILT TAILINGS.		2.59	1	51 DO	21	24.3			
2–3		Compact, moist to wet, black, fine to medium, silty SAND to SAND TAILINGS, some silt.			2	51 DO	11	24; 22.5			
3–4											
4–5					3	51 DO	13	21.4			∇
5–6											
6–7					4	51 DO	20	35.6			
7–8				7.77	5	51 DO	10	11.9; 75.4			
8–9		Compact, wet, grey SILT TAILINGS, some clay, trace to some sand.		8.69							
9–10		Compact, wet, grey, fine to medium, silty SAND TAILINGS, trace clay, occasional silt pockets.			6	51 DO	11	39.9			
10		CONTINUED NEXT PAGE									

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

PROJECT No.: 041-7000-18

LOCATION:

RECORD OF BOREHOLE: BH04-P49-2

BORING DATE: Sept. 2, 2004

SHEET 2 OF 2

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp — W — Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Compact, wet, grey, fine to medium, silty SAND TAILINGS, trace clay, occasional silt pockets. *(continued)*									
11		Loose, wet, black to dark grey, fine to medium SAND TAILINGS, trace silt.		10.67	7	51 DO	9	22.9			
12		End of BOREHOLE. Split Spoon Refusal Probable Bedrock		12.19							
13											
14											
15											
16											
17											
18											
19											
20											

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

DEPTH SCALE 1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

Figure A- 4 : Borehole BH04-P49-2

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P50-1

SHEET 1 OF 2

LOCATION:

BORING DATE: Sept. 2, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	% PASSING #200 SIEVE	WATER CONTENT PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Ground Surface		0.00							
1		ROCKFILL.		1.52							
2					1	51 DO	24	34.5			
3					2	51 DO	15	30.8			
4		Compact, moist to wet, black to dark grey, fine to medium SAND TAILINGS, some silt.			3	51 DO	14	18.6			▽
5											
6					4	51 DO	16	62.4			
7											
8				7.98	5	51 DO	14	24.3	86.9		
		Stiff, wet, grey, clayey SILT to silty CLAY TAILINGS, trace sand.		8.84							
9		Compact, wet, grey, sandy SILT TAILINGS, trace to some clay and stiff, clayey silt pockets.			6	51 DO	10	55.1			
10		*CONTINUED NEXT PAGE*									

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

% PASSING #200 SIEVE: 20, 40, 60, 80

WATER CONTENT PERCENT: 10, 20, 30, 40

DEPTH SCALE

1 : 50

GAIA ENTREPRENEURS

LOGGED: SP

CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P50-1

SHEET 2 OF 2

LOCATION:

BORING DATE: Sept. 2, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp — W — Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Marathon Drilling Ltd. Power Auger (4.25in. I.D. Hollow Stem)	Compact, wet, grey, sandy SILT TAILINGS, trace to some clay and stiff, clayey silt pockets. *(continued)*		10.36							
11		Compact, wet, dark grey, fine to medium SAND TAILINGS, some silt.		11.28	7	51 DO	18	16.3			
12		Very stiff to stiff, wet, clayey SILT to SILT TAILINGS, some sand.			8	51 DO	16	68.3			
13											
14					9	51 DO	9	82.7			
15											
		Stiff, wet, black PEAT and brown, clayey SILT, trace sand and gravel.		15.54	10	51 DO	11	63.1			
16		End of BOREHOLE.		15.85							
17											
18											
19											
20											

DEPTH SCALE
1 : 50

GAIA ENTREPRENEURS

LOGGED: SP
CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

Figure A- 5 : Borehole BH04-P50-1

PROJECT No.: 041-7000-18

RECORD OF BOREHOLE: BH04-P51-2

SHEET 1 OF 2

LOCATION:

BORING DATE: September 15, 2004

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / % PASSING #200 SIEVE (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp W Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		0.00							
0–1.37	Power Auger - 107mm I.D. Hollow Stem (Marathon Drilling)	ROCKFILL.									
1.37–2.44		Compact, brown, fine, silty SAND to sandy SILT TAILINGS.			1	51 DO	21	28			
2.44–4.94		Compact, dark grey, fine to medium, silty SAND TAILINGS, occasional silt seams, trace clay.			2	51 DO	13	32.8			Sep.14/04 ∇
					3	51 DO	17	23.3	64.3		
4.94–5.79		Compact, dark grey SILT TAILINGS, some sand, some clay.									
5.79–10		Compact, grey, fine to medium SAND to silty SAND TAILINGS, trace clay, occasional silt layers.			4	51 DO	17	35.3			
					5	51 DO	16	38.2			
					6	51 DO	17	20.8			
10		*CONTINUED NEXT PAGE*									

DEPTH SCALE
1 : 50

GAIA ENTREPRENEURS

LOGGED: J.T.
CHECKED:

BOREHOLE-GAIA 041-7000-18.GPJ GLDR_CAN.GDT 29/9/04

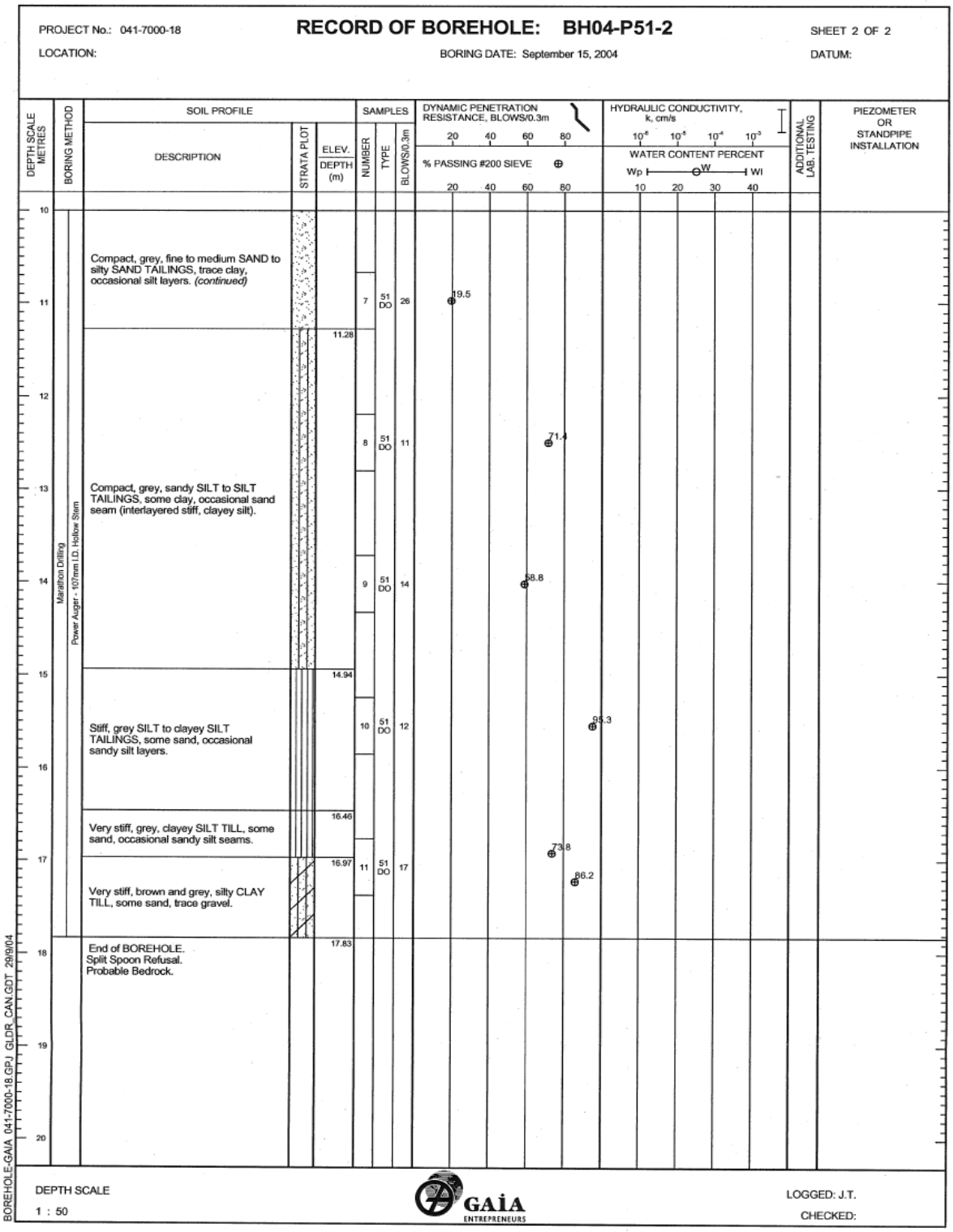

Figure A- 6: Borehole BH04-P51-2

APPENDIX B: PRE CPT DATA WITH INTERPRETED SOIL BEHAVIOUR TYPE

This appendix gives the details of the ten pre treatment CPT from the Guindon Dam North analyzed in the thesis. The details include: the elevations, CPT numbers, panel numbers, date of CPT, ground water table inferred from the CPT, and the interpreted soil behaviour type for each CPT. The soil behaviour types obtained for all the CPT at different depths were compared in Chapter 4 to establish the trends in terms of soil/tailings behaviour type on site. Details as described above for one example pre CPT were also given in Chapter 4. Table B-1 lists the panel number, date of test, elevation, and the water table inferred for all the pre CPT considered in the analysis.

Table B- 1: Schedule of Pre Treatment CPT in Guindon Dam North

CPT No.	Panel No.	Date of CPT	Ground Elevation (m)	Inferred Water Table Elevation (m)	Inferred Water Table (m) Depth
40	CPT04-29	13th May 2004	316	311	5
41	CPT04-30	13th May 2004	315.5	311.5	4
42	CPT04-31	26th February 2004	316.34	311.74	4.8
44	CPT04-32	26th February 2004	315.11	310.61	4.5
45	CPT04-33	26th February 2004	316.3	311.1	5.2
46	CPT04-34	26th February 2004	316.5	312.2	4.3
47	CPT04-35	26th February 2004	317	311.5	4.5
49	CPT04-37	5th May 2004	316.3	312.7	4
51	CPT04-38	5th May 2004	315.7	312.7	3
52	CPT04-39	13th May 2004	316.6	313.1	3.5

Figures B-1 to B-10 show the variation of non-normalized tip resistance, pore pressure u_2, normalized friction ratio F, and the I_c (Jefferies and Davies, 1991) for all the pre treatment CPT considered in the analysis. The plots also include the interpreted soil behaviour type for different depths. The interpreted behaviour type is based on the visual inspection of the variation of CPT parameters and the soil behaviour type classification chart (Jefferies and Davies, 1991).

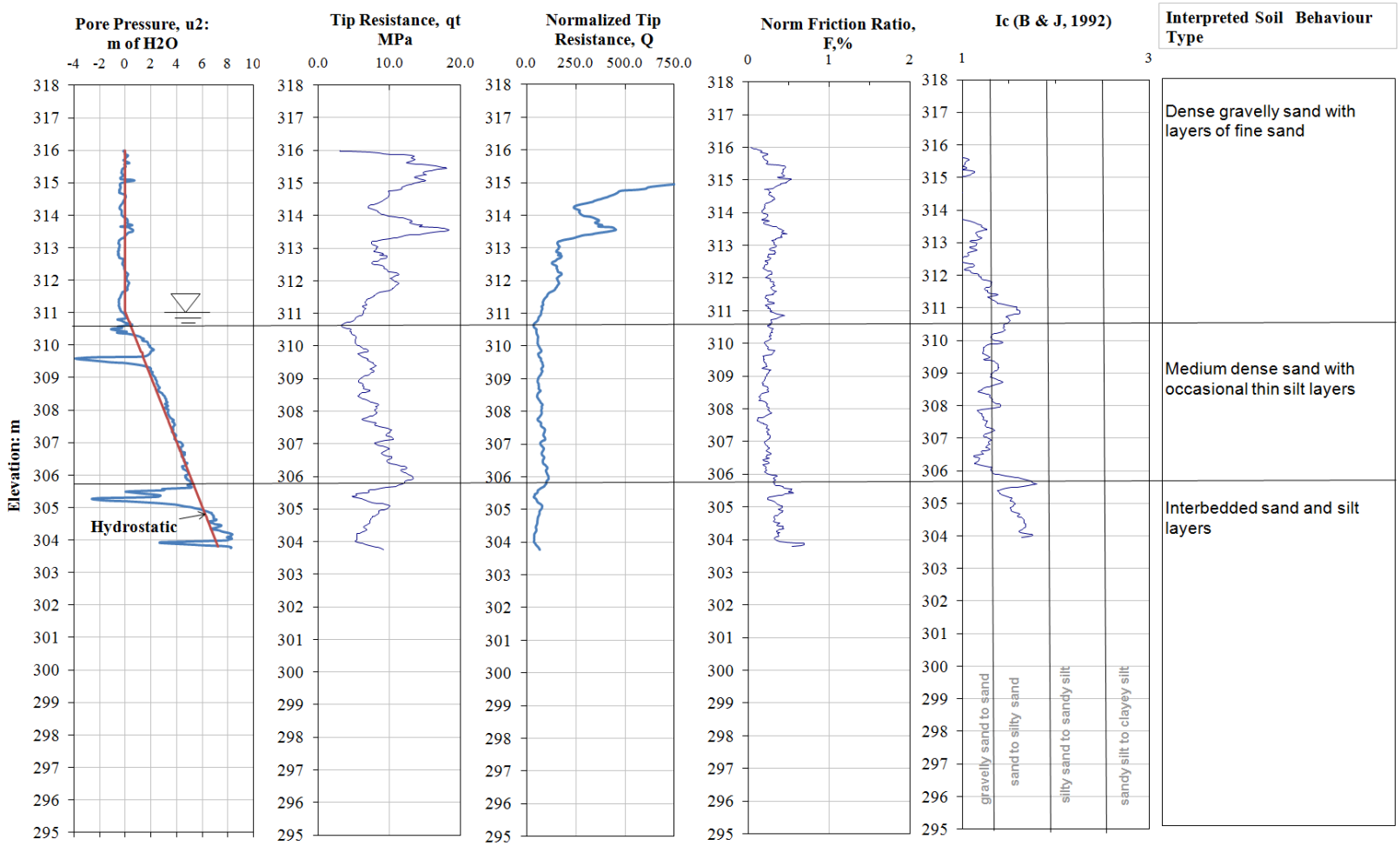

Figure B- 1 : Pre CPT -29 Profile

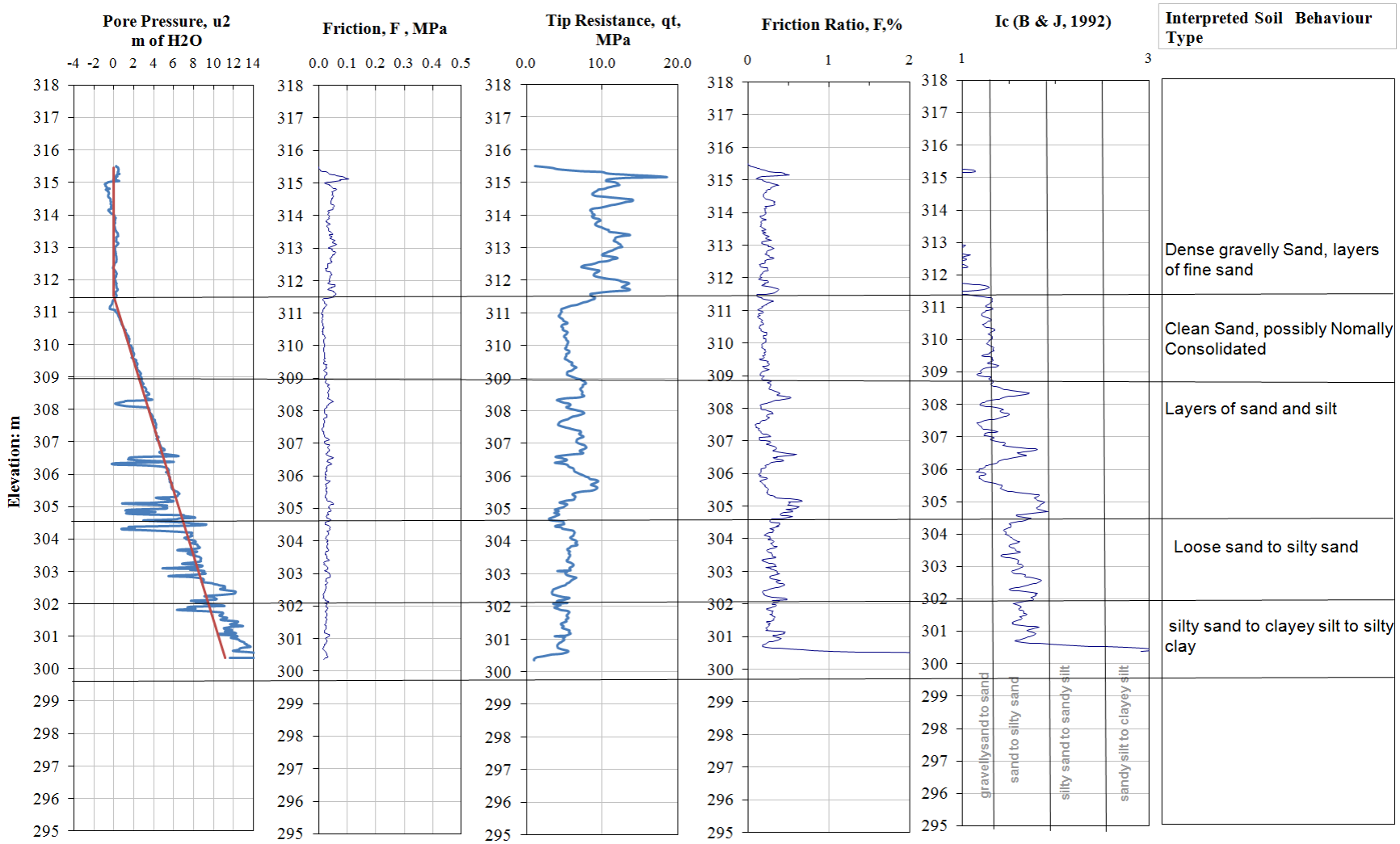

Figure B- 2 : Pre CPT -30 Profile

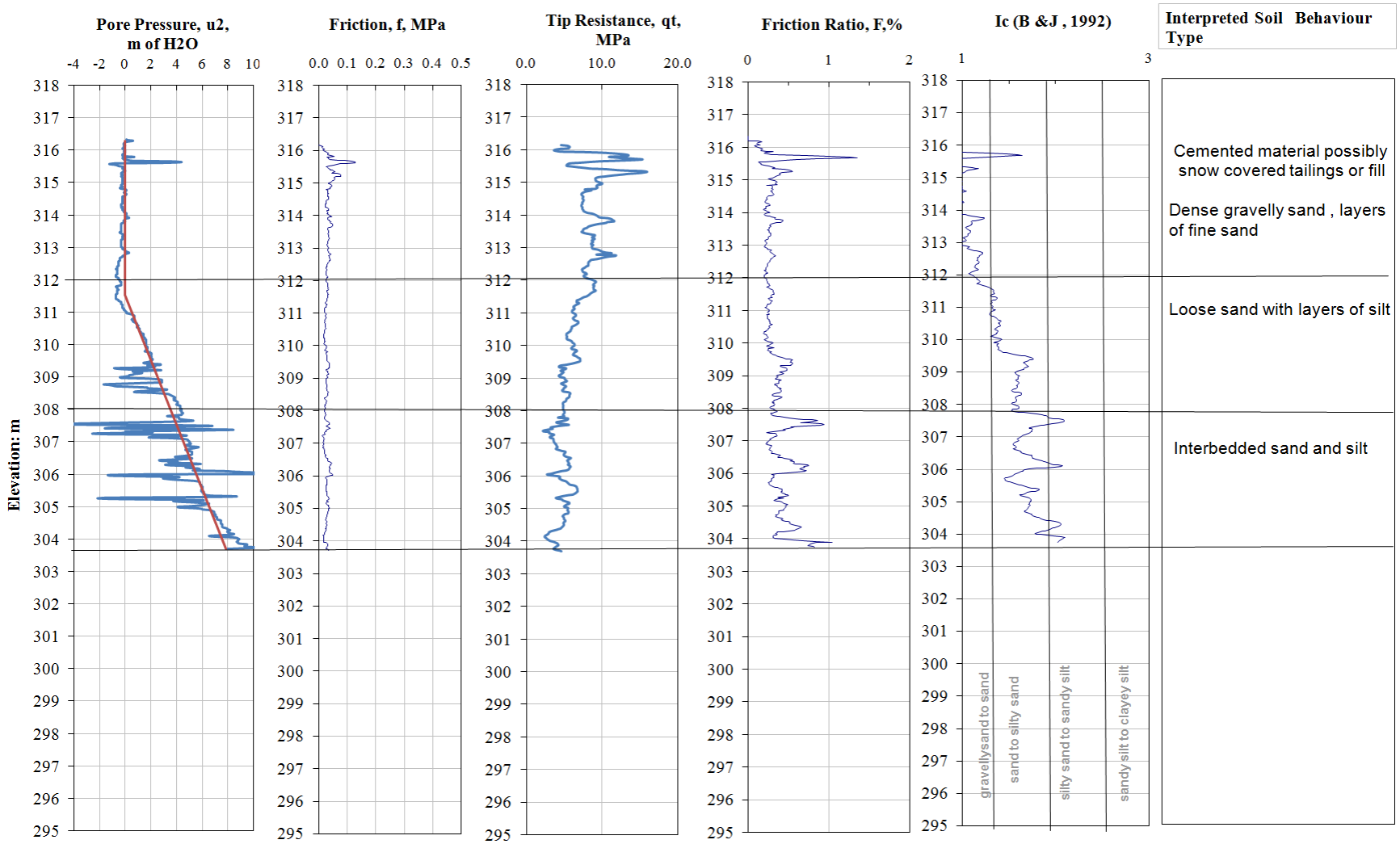

Figure B- 3 : Pre CPT -31 Profile

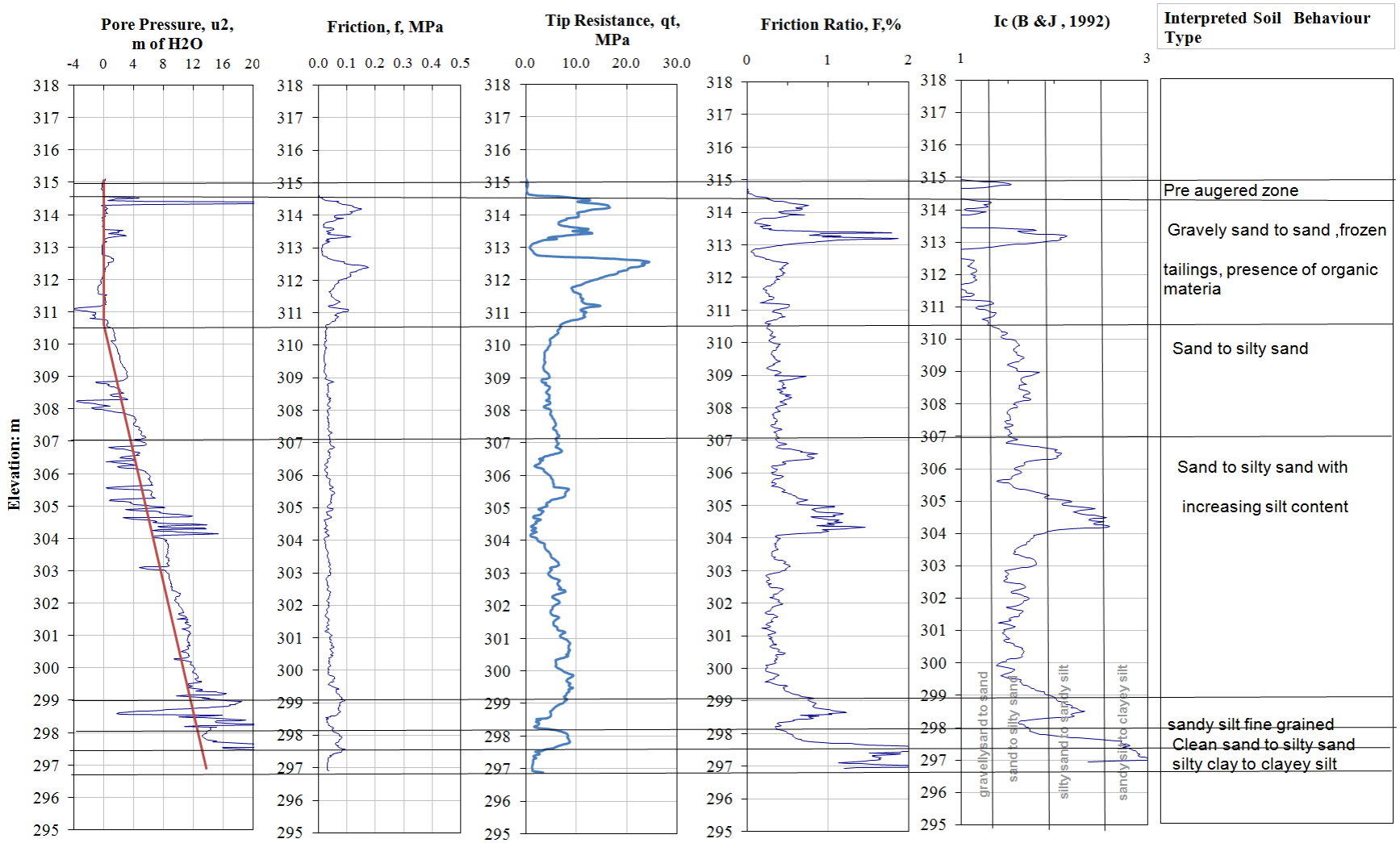

Figure B- 4: Pre CPT -32 Profile

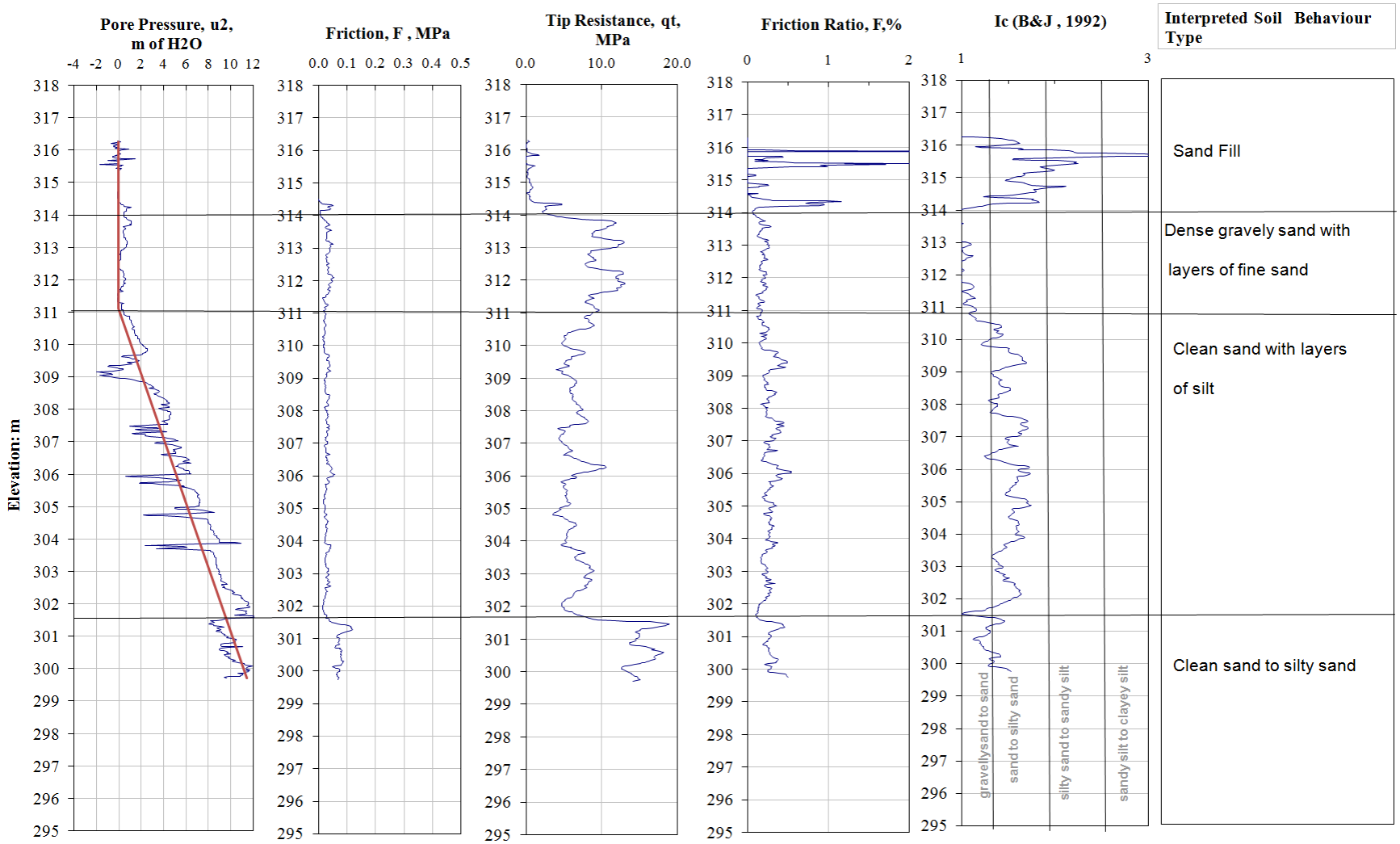

Figure B- 5 : Pre CPT -33 Profile

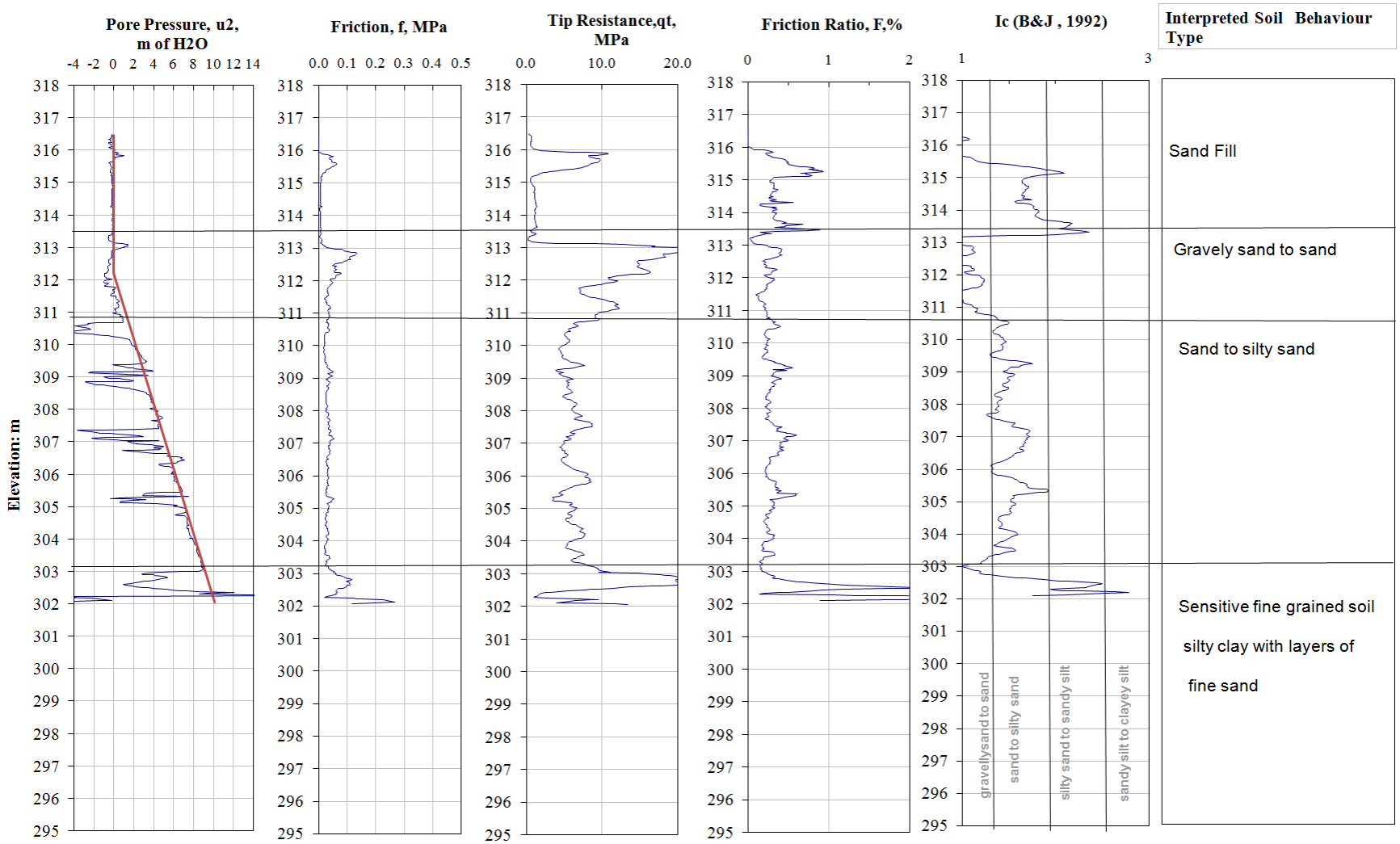

Figure B- 6 : Pre CPT -34 Profile

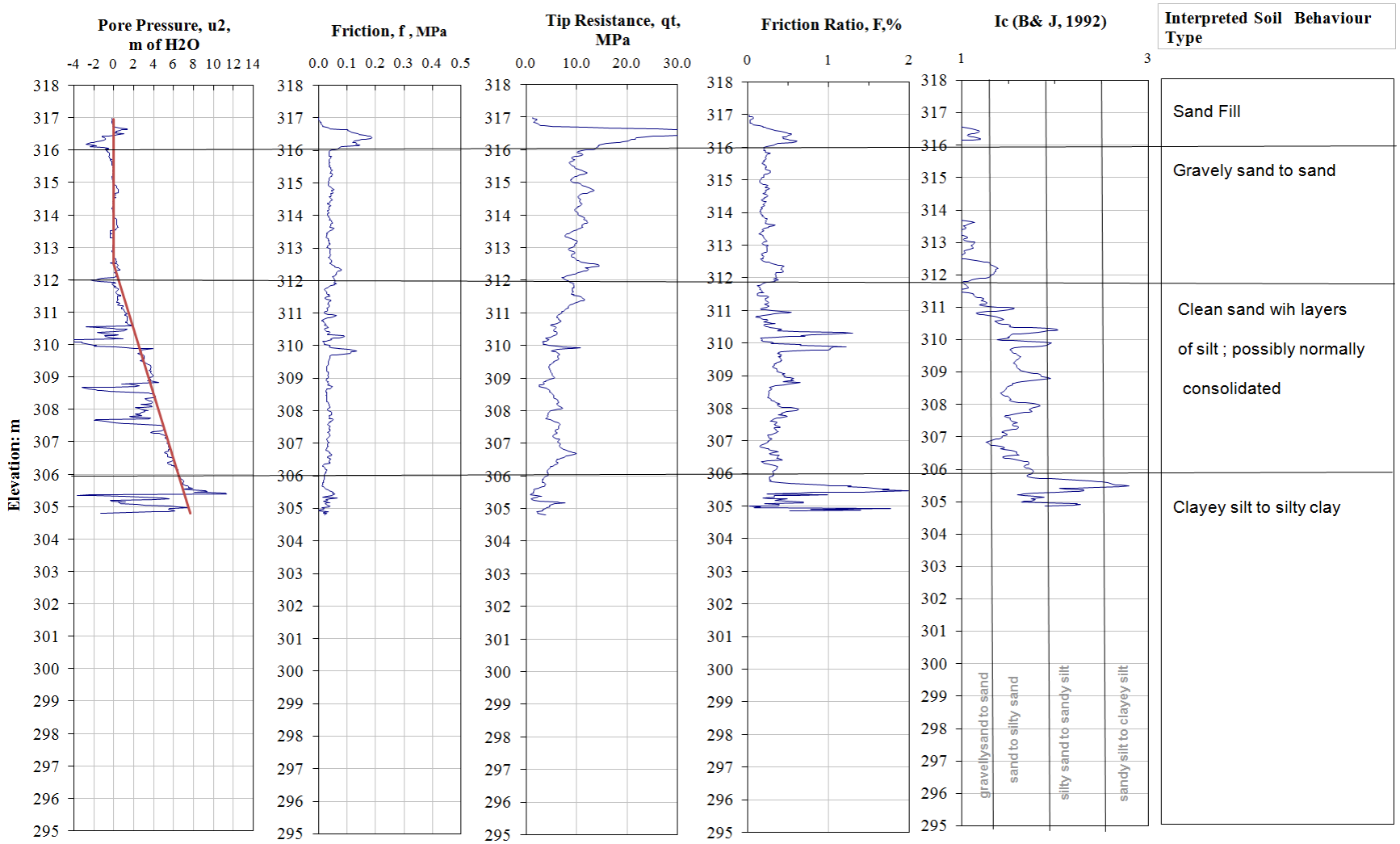

Figure B- 7: Pre CPT -35 Profile

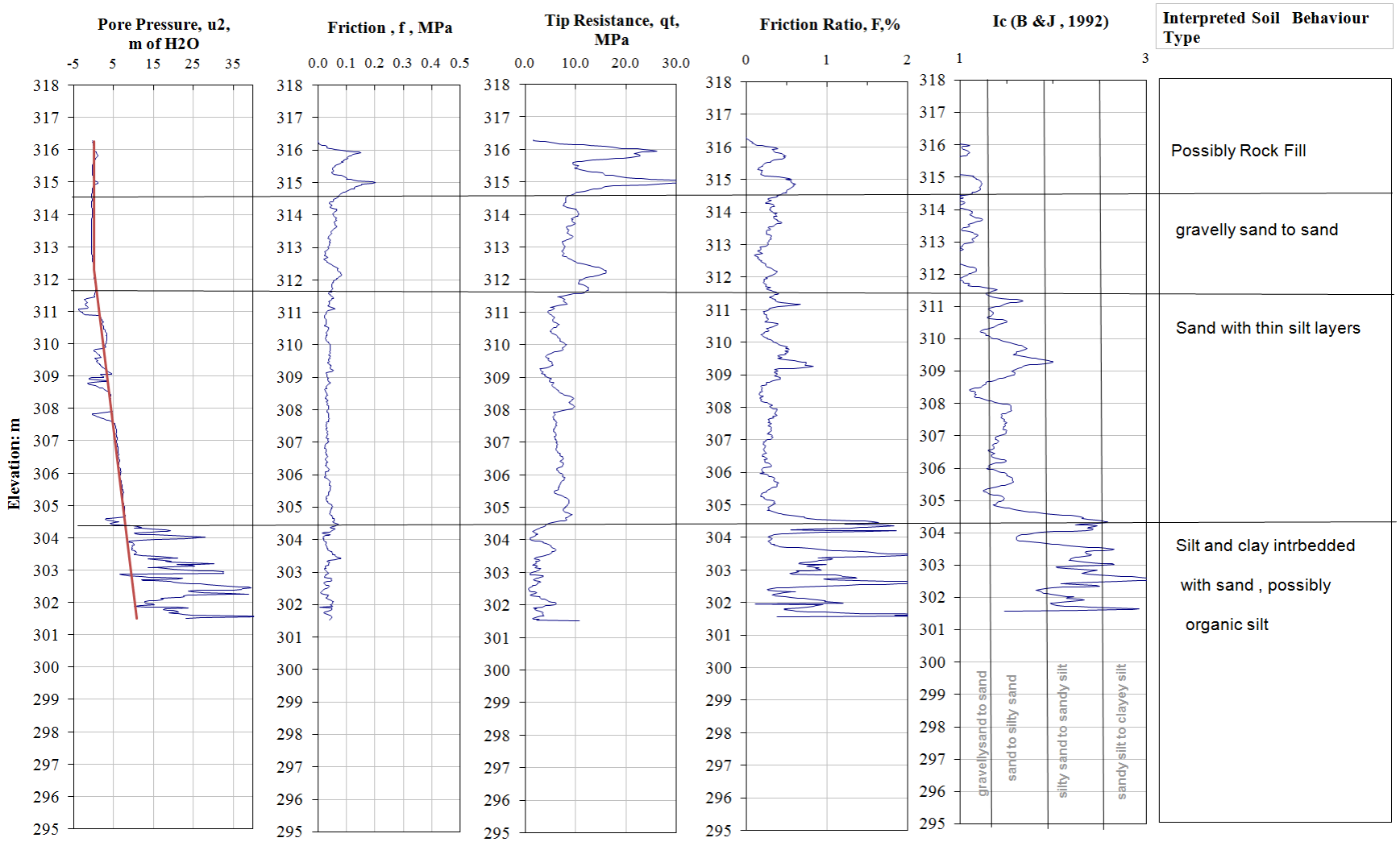

Figure B- 8: Pre CPT -37 Profile

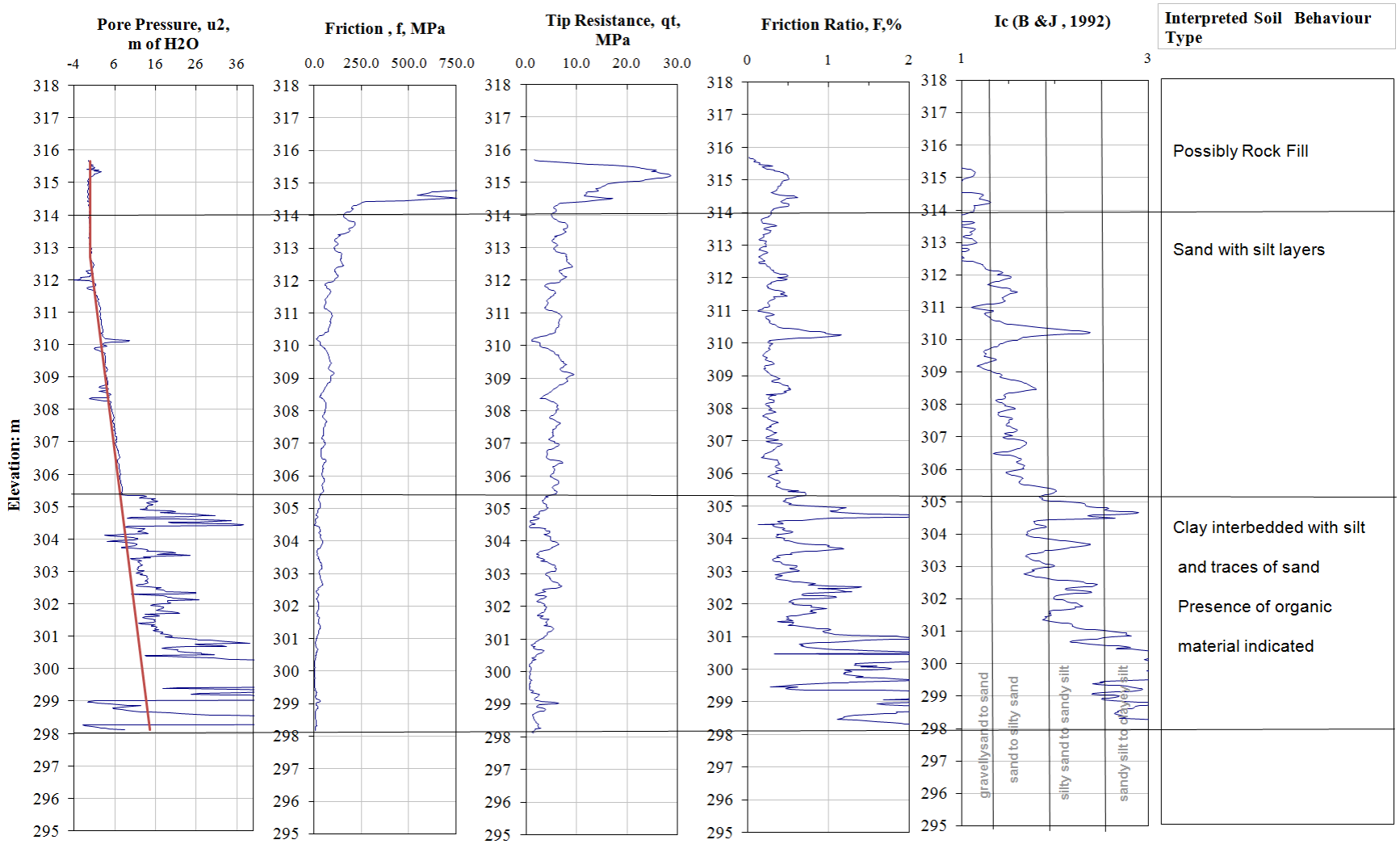

Figure B- 9: Pre CPT -38 Profile

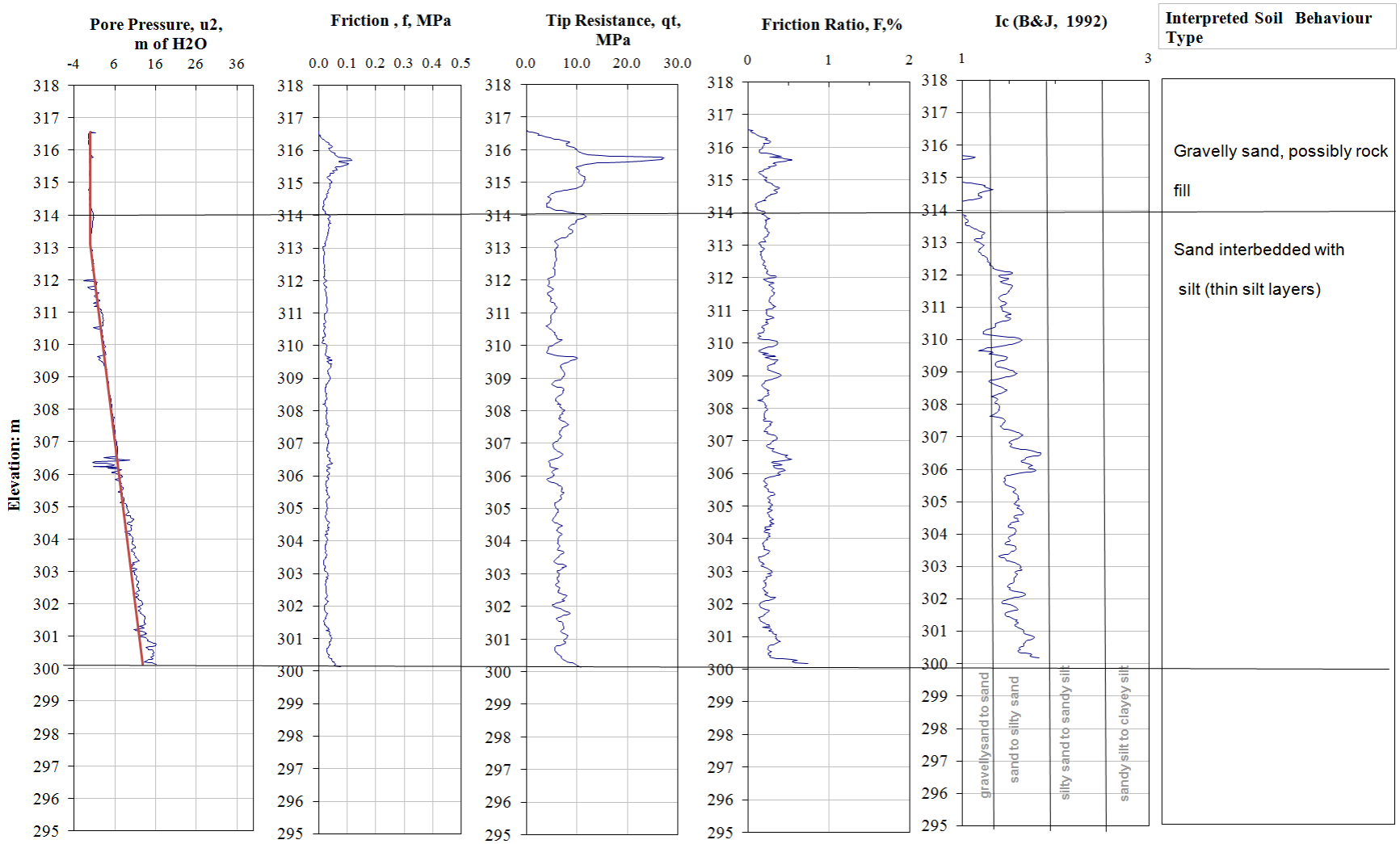

Figure B- 10: Pre CPT -39 Profile

APPENDIX C: NUMBER OF CPT DATA POINTS FOR STATISTICAL CALCULATIONS

This appendix lists the number of data points within an I_c bin for the pre treatment CPT data, all post CPT data, and the post CPT data collected at different times after the treatment. Each bin has a size of 0.5 as measured in the units of I_c and is represented by the median value of the bin. These data points are listed for different depth ranges as identified in the analysis. Number of data points is also listed for the variation of the pre and the post treatment normalized tip resistance with depth. For such a representation, the number corresponds to the number of data points within a fixed depth interval. In general, they are used for statistical calculations, for comparison of the pre or and the post CPT data for assessment of the ground improvement. In this thesis, number of data points enables comparison of the two CPT data sets.

Pre Treatment Data

Table C- 1: Number of Data Points for Pre CPT Data in Interval I (Top-WT)

Range and Median of I_c bin	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points
(0-0.5) 0.25	Gravelly sand to sand	84
(0.5-1) 0.75	Gravelly sand to sand	844
(1-1.5) 1.25	Clean sand to silty sand	1012
(1.5-2) 1.75	Clean sand to silty sand	288
(2-2.5) 2.25	Silty sand to sandy silt	83

Table C- 2 : Number of Data Points for Pre CPT Data in Interval II (WT-7m)

Range and Median of I_c bin	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points
(0.5-1) 0.75	Gravelly sand to sand	59
(1-1.5) 1.25	Clean sand to silty sand	1031
(1.5-2) 1.75	Clean sand to silty sand	309
(2-2.5) 2.25	Silty sand to sandy silt	23

Table C- 3: Number of Data Points for Pre CPT Data in Interval III (7m-11m)

Range and Median of I_c bin	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points
(1-1.5) 1.25	Clean sand to silty sand	910
(1.5-2) 1.75	Clean sand to silty sand	1041
(2-2.5) 2.25	Silty sand to sandy silt	85
(2.5-3) 2.75	Clayey silt to silt	18

Table C- 4: Number of Data Points for Pre CPT Data in Interval IV (11m-16m)

Range and Median of I_c bin	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points
(1-1.5) 1.25	Clean sand to silty sand	370
(1.5-2) 1.75	Clean sand to silty sand	1034
(2-2.5) 2.25	Silty sand to sandy silt	229
(2.5-3) 2.75	Clayey silt to silt	73
(3-3.5) 3.25	Clay	35

Ageing Effects

Table C- 5 : Number of Pre and Post CPT Data Points in Interval II (WT-7m)

Median Pre I_c	Pre Data	Post within one week	Post one week	Post three Weeks	Post one month
0.75	59	86	182		187
1.25	1031	921	541	1186	1697
1.75	309	239	151	140	190
2.25	23	24			

Table C- 6 : Number of Pre and Post CPT Data Points in Interval III (7m-11m)

Median Pre I_c	Pre Data	Post within one week	Post one week	Post three Weeks	Post one month
1.25	910	944	670	1313	2078
1.75	1041	687	503	642	524
2.25	85	108	28	9	31
2.75	18	13	30		17

Table C- 7 : Number of Pre and Post CPT Data Points in Interval IV (11m-16m)

Median Pre I_c	Pre Data	Post within one week	Post one week	Post three Weeks	Post one month
1.25	370	159	88	252	583
1.75	1034	503	100	490	359
2.25	229	131	65	55	62
2.75	73	39		16	15
3.25	35	16			9

Post Treatment Data

Table C- 8 : Number of Data Points for Entire Post CPT Data Interval II (WT-7m)

Median Pre I_c	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points Post	Number of Data points Pre
0.75	Gravelly sand to sand	438	59
1.25	Clean sand to silty sand	5094	1031
1.75	Clean sand to silty sand	791	309
2.25	Silty sand to sandy silt	26	23

Table C- 9 : Number of Data Points for Entire Post CPT Data Interval III (7m-11m)

Median Pre I_c	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Post Data Points	Number of data points Pre
0.75	Gravelly sand to sand	15	----
1.25	Clean sand to silty sand	5662	910
1.75	Clean sand to silty sand	3059	1041
2.25	Silty sand to sandy silt	185	85
2.75	Clayey silt to silt	59	18
3.25	Clay	23	---

Table C- 10: Number of Data Points for Entire Post CPT Data Interval IV (11m-16m)

Median Pre I_c	Inferred soil behaviour type (Jefferies & Davies, 1991)	Number of Data Points Post	Number of data points Pre
1.25	Clean sand to silty sand	1416	370
1.75	Clean sand to silty sand	1859	1034
2.25	Silty sand to sandy silt	396	229
2.75	Clayey silt to silt	142	73
3.25	Clay	29	35

Depth wise Representation

Table C- 11: Number of Data Points for Pre and Post CPT Data Plotted with Depth

Depth Zones	Pre data points	Post data points
Zone II (WT-7m)	1618	6970
Zone III (7m-11m)	2048	8822
Zone IV (11m-16m)	1740	3868

APPENDIX D: CPT DATA PROCESSING

This appendix deals with the processing (for various CPT specific corrections listed in Chapter 2, Section 2.2.2.4.2) of the raw pre and the post treatment CPT data, provided by GAIA (2004), using an MS Excel based Visual Basics Applications (VBA) routine (Jefferies, 2006). Examples of the raw and the processed data are given at the end to illustrate the data processing. Normalization of the processed data using a linear normalization technique to account for the variation of the measured data with the increasing overburden pressure is also carried out in this chapter. Typical representation of the normalized and the non normalized CPT data is shown.

D.1 Corrections for CPT Data

CPT data (made available to UBC) were unprocessed. The data were made available in two types of electronic files with: files with .CPG and .CPT extensions. The file with .CPT extension is a text file and shows the raw data (depth, tip resistance, friction, and pore water pressure, u_2) presented from left to right in columns. The electronic (acoustic) cone used for this project had the tilt derivative alarm (to indicate excessive inclination of the cone during penetration), point resistance alarm, (to indicate zero tip resistance) and other indicators indicating rod breaks, end of the test and the start of a dissipation test, facilitating the correction of raw data for various correction factors listed in Chapter 2. File with .CPT extension shows the signals (if any) from these indicators in the data. It is thus possible to correct the data manually in the text editor at depths where these signals appear.

The file with .CPG extension, apart from having the unedited cone data, (from .CPT file) also has the details such as the test date, test location, inferred water table, and other design input data which can be used to generate CPT plots using a compatible commercial software. The test date from the .CPG file was used to relate that CPT to the timing of treatment in a panel (pre or post treatment).

Raw CPT data are summarized below:

- The CPT were carried out at a depth ranging from 8m-20m with an average depth of about 13m.
- The data were manually processed for rod breaks using the signals from the cone.
- No dissipation test records are available.

The raw data obtained from a cone penetration test, as described in detail in Chapter 2, (Section 2.2.2.4.2) include: tip resistance q_c (units of stress), sleeve friction f_s (units of stress), and the pore water pressure u_2 measured behind the cone tip (units of stress) for the depth of penetration. The data are processed for factors such unequal end area effects, temperature effects, negative friction sleeve measurements, cone inclination, friction tip offset and the rod breaks. All of the above have been reviewed and explained in detail in Chapter 2 in Section (2.2.2.4.2 a-g). The CPT data supplied to UBC were already corrected for the temperature of the ground and inclination of the cone tip during penetration. The process of data correction adopted for the remaining factors for this research is given below.

(a) Unequal End Area Effects: The measured tip resistance q_c is corrected to q_t by using the equation given in Equation 2-1 in Chapter 2. u_2 is the pore pressure generated immediately behind the cone tip and "a" is the net area ratio, taken equal to 0.81 for the cone used in this study (Refer Chapter 3 , section 3.3.1 b).

(b) Rod Breaks: The reduced tip resistance values at the rod break (generated due to stoppage of the CPT, when an additional rod is added) were deleted manually from the data for the entire depth of measurement for all the pre and the post treatment CPT.

(c) Friction-tip Resistance Offset: This factor comes into the consideration for data processing due to the depth offset of 0.1m between friction sleeve and the cone tip. The excel based VBA routine used in this analysis to process and plot the measured CPT data, aligned, the measured friction sleeve values with the corresponding cone tip readings.

(d) Negative Friction Sleeve Measurements: Negative friction values are typically caused by the change in the temperature of the ground. As these values are physically unrealistic, they need to be removed from the data. In the VBA routine used in this project, all the measured friction values are compared to a minimum positive value. Using an in built "max" function which returns the maximum of two numbers, a positive friction value is ensured for all the depth intervals in the data.

D.2 Use of Normalized Dimensionless CPT Parameters

The advantages and the process of normalization of measured CPT parameters have been reviewed in detail in Chapter 2. The measured data, tip resistance q_t, pore pressure u_2, and sleeve friction f_s would increase with increasing overburden pressure for the same geo-material type and the density, hence data are normalized with respect to the effective overburden stress, creating a set of dimensionless CPT parameters and removing one source of variability.

According to the linear normalization approach used by Robertson (1990), normalized tip stress, Q, is given by

$$Q = (q_t - \sigma_{vo})/(\sigma_{vo} - u) \qquad \text{Equation D-1}$$

Normalized friction ratio is F given by

$$F = f/(q_t - \sigma_{vo}) \times 100\% \qquad \text{Equation D-2}$$

and the pore pressure data normalization is shown below

$$B_q = \Delta u/(q_t - \sigma_{vo}) \qquad \text{Equation D-3}$$

Where

Δu = excess pore pressure

σ_{vo}= total overburden pressure

All other parameters are already defined in Chapter 2 and were calculated in the VBA routine itself. Along with above normalized CPT parameters, soil behaviour type Index, I_c, which is an algebraic combination of the normalized CPT parameters, has also been used in

this research. The concept of soil behaviour type Index I_c and the potential usefulness of the soil behaviour chart (Jefferies & Davies, 1991),which gives rise to I_c for characterization and classification of mine waste tailings have been reviewed in detail in chapter 2 (section 2.2.2.4.3). Jefferies and Davies (1991, 1993) grouped the above normalized CPT parameters into a soil classification index I_c. Soil behaviour type can be estimated from the I_c value. It is argued that the incorporation of the pore pressure parameter B_q along with Q and F makes I_c a useful tool for classification of fine grained tailings (Jefferies and Davies, 1991) and hence will be used in this thesis. A slightly modified and the latest version of I_c as proposed by Been and Jefferies (1993) has been used in this thesis and is given below:

$$I_c = \sqrt{\{3-\log[Q\,(1-B_q)+1]\}^2+[1.5+1.3\,(\log F)]^2} \qquad \text{Equation D-4}$$

D.3 CPT Data Representation

The plots include the following measured and interpreted parameters with depth:

(a) Tip resistance q_t (MPa) vs. depth (m)

(b) Measured sleeve friction stress, f_s (MPa) vs. depth (m)

(c) Measured pore pressure u_2 (MPa) vs. depth (m), also shown is the inferred hydrostatic pore pressure line u_0 (MPa)

(d) Normalized friction ratio F (%) vs. depth in m

(e) Normalized pore pressure parameter B_q vs. depth (m)

(f) Soil Classification Index I_c vs. depth (m)

Figure D-0-1 shows the CPT data representation plotted using the VBA routine for the unprocessed data, while Figure D-0-2 shows similar representation for the processed data. The latter has been used in this thesis to present all the CPT data. The projecting lines in the tip resistance profile in Figure D-0-1 were manually removed. These are the result of the rod breaks as they are observed every meter apart along the depth; equal to the length of the rod.

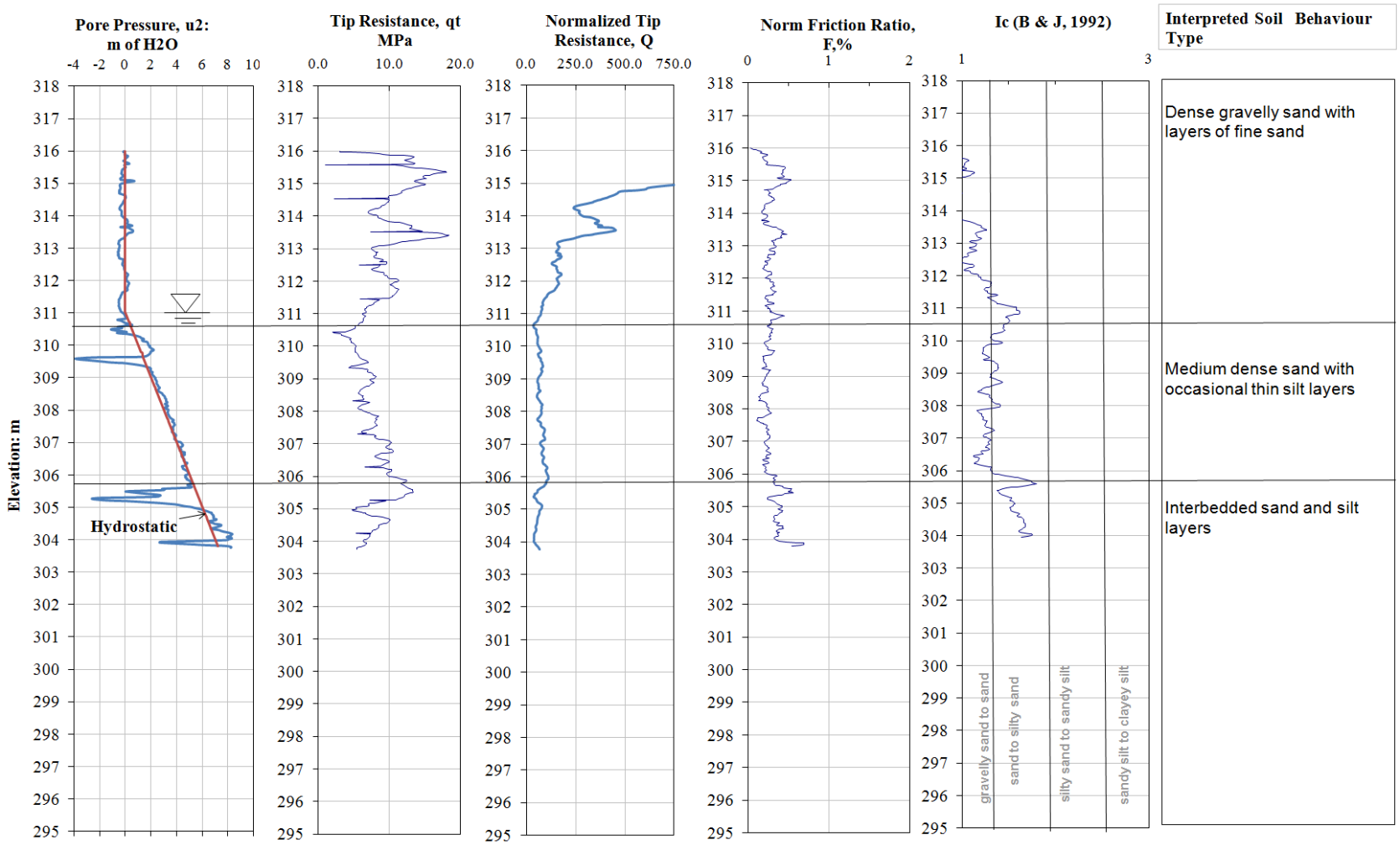

Figure D-0-1: Typical CPT Representation Plot for Raw Data

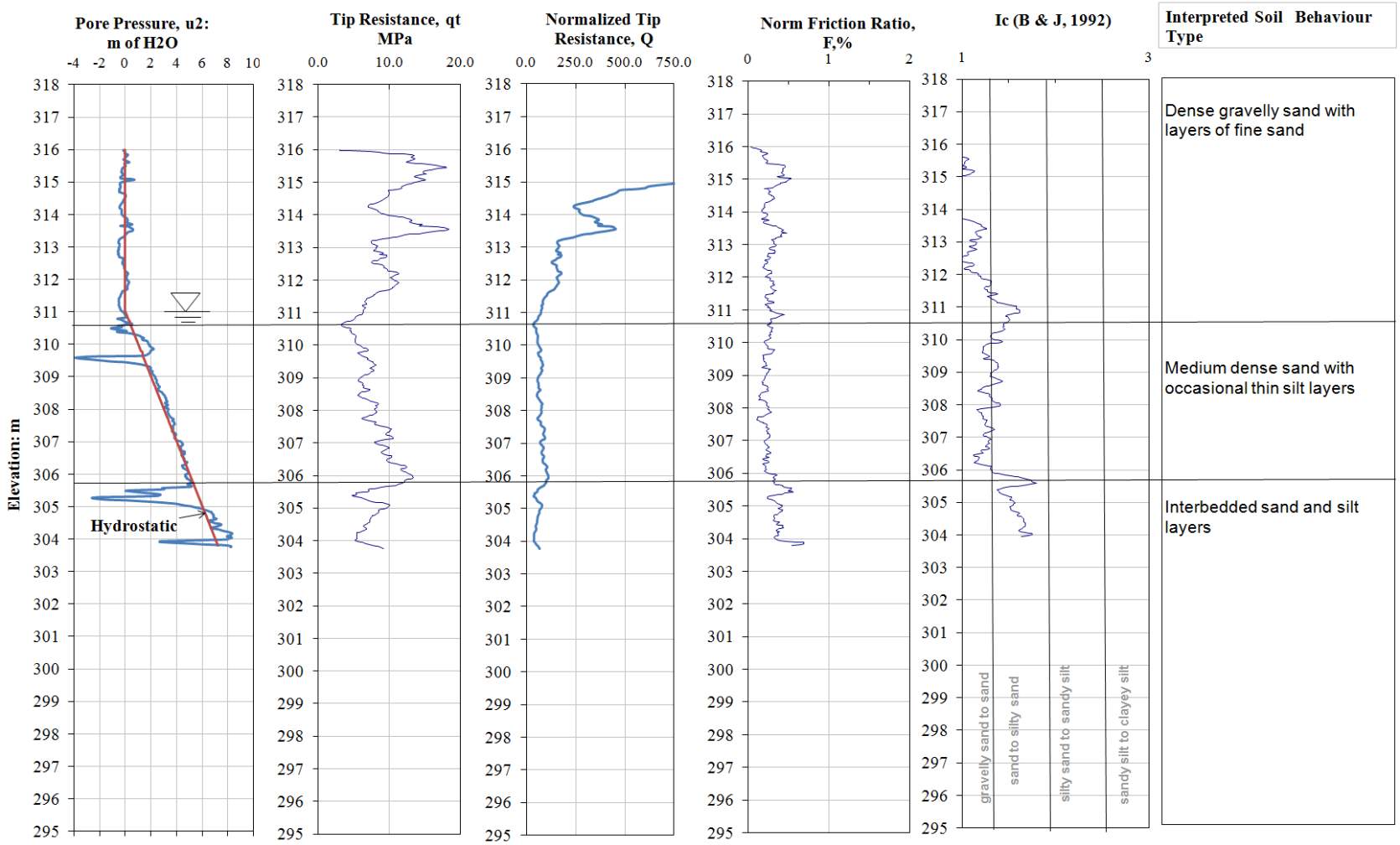

Figure D-0-2: Typical CPT Representation Plot for Processed Data

APPENDIX E: SORTING OF POST CPT DATA

This appendix lists the post CPT data used in the analysis for characterization of the post treatment ground, and assessment of ground improvement. Post treatment CPT data were sorted for different conditions such as the repetition of a CPT on site, sequence of application of treatment, and aging effects. In this chapter, the post treatment CPT sorted for sequence of treatment on site, and aging effects are listed.

Most of the post treatment CPT have been conducted after the application of RIC. These CPT have been selected for the post treatment analysis. Table E- 1 below lists the pre and the post treatment CPT for the panels in Guindon Dam north. The post CPT in each panel have been sorted with regard to their timing between the timing of application EC and RIC, shortly after RIC, and those conducted after a month or later after RIC. This division allows further sorting of post CPT with time as shown later in this appendix. Dynamic compaction (DC) was another surface compaction method used on site. Very few post DC CPT data are available as shown in Table E- 1 and hence the effect of DC on the CPT data have not been considered in this analysis.

To determine the effect of time on the improvement achieved, post CPT data sorted in Table E- 1 were sorted for time. Table E- 2 shows the post treatment CPT conducted at different times after the treatment viz. within a week (3-6 days), one week (8-12 days), two weeks (17-18 days), three weeks (21-23 days), and a month (43-67 days) after the treatment in the respective panels. These CPT have been compared with the pre CPT data to determine the time dependent gain in strength (aging) at site.

Table E- 1: Post Treatment CPT Sorted with Sequence of Treatment Type

Panel No.	Pre CPTs Before First Blast	CPTs after Last blast and Before First surface Compaction (RIC/DC)	CPTs after EC and first surface compaction (RIC/DC) but before second surface compaction (if employed)		CPTs after 2nd Surface compaction (if employed)
			Conducted shortly post EC and (RIC/DC)	Conducted later (month or so) post EC and (RIC/DC) pre 2nd Surface compaction	
40	CPT 29	None	None	P40-1 P40-2 P40-3	Not used
41	CPT 30	None	None	P41-1 P41-2 P41-3	Not used
42	CPT 31	None	P113 P114 P115	P42-2 P42-3	Not used
43	None	None	None	P43-1 P43-2 P43-3	DC used but no valid post CPTs *
44	CPT 32	P44-1 P44-2	P44-1(2) P44-3	None	None

Panel No.	Pre CPTs Before First Blast	CPTs after Last blast and Before First surface Compaction (RIC/DC)	CPTs after EC and first surface compaction (RIC/DC) but before second surface compaction (if employed)		CPTs after 2nd Surface compaction (if employed)
			Conducted shortly post EC and (RIC/DC)	Conducted later (month or so) post EC and (RIC/DC) pre 2nd Surface compaction	
45	CPT 33	P122			P45-1 P45-2 P45-3 P45-1(2) P45-2(2)
46	CPT 34		P-118 P-119	P46-2(2) P46-3	
47	CPT 35		P-120	P47-2 P47-3	DC used but no valid post CPTs *
48***	CPT 36	Not applicable	P48-1 P48-2 P48-3		
49	CPT 37	None	P-110(2) P-111A, P116 P-1110A	P49-2 P49-3,P49-4 P49-2(2)	DC date not Known

Panel No.	Pre CPTs Before First Blast	CPTs after Last blast and Before First surface Compaction (RIC/DC)	CPTs after EC and first surface compaction (RIC/DC) but before second surface compaction (if employed)		CPTs after 2nd Surface compaction (if employed)
			Conducted shortly post EC and (RIC/DC)	Conducted later (month or so) post EC and (RIC/DC) pre 2nd Surface compaction	
50	NA	None	P50-1 P50-2	P-50-1(2) P-50-2(2)	None
51	CPT 38	None	105 A 106 A P51-1	*	None
52	CPT 39	None	P-121, P123 P123(2), P127(2), P127	P52-1, P52-3	
53	None	None	None	P53-1, P53-1(2) P53-2, P53-3	None

* Subsequent Post CPT conducted at exactly the same location and hence not useful. Only Initial post CPT considered.
*** Only DC panel

Table E- 2: Post Treatment CPT Sorted with Time

CPT within One week post treatment	CPT one week post treatment	CPT two weeks post treatment	CPT three weeks post treatment	CPT a month or more post treatment
P111A (3 days)	P44-1(2) (8 days)	P50-1 (17 days)	P40-1 (22 days)	P42-2 (33 days)
P116 (3 days)	P44-3 (10 days)	P50-2 (17 days)	P40-2 (21 days)	P42-3 (33 days)
P1110A (3days)	P118 (12 days)	P127 (17 days)	P40-3 (22 days)	P46-2(2) (43 days)
P105A (3 days)	P119 (12 days)	P43-1 (18 days)	P41-1 (22 days)	P47-2 (30 days)
P106A (3 days)	P120 (8 days)	P43-2 (18 days)	P41-2 (22 days)	P47-3 (30 days)
P51-1 (4 days)	P48-1 (12 days)	P43-3 (18 days)	P41-3 (22 days)	P49-2 (33 days)
P121 (6 days)	P48-2 (10 days)		P46-3 (23 days)	P49-2(2) (71 days)
P123 (6 days)	P48-3 (12 days)		P123(2) (21 days)	P49-3 (33 days)
P113 (3 days)	P-110(2) (10 days)		P127(2) (21 days)	P49-4 (33 days)
P115 (3 days)			P53-1 (23 days)	P50-1(2) (52 days)
				P50-2(2) (51 days)
				P52-1 (67 days)
				P52-3 (67 days)
				P53-1(2) (67 days)
				P53-2 (67 days)
				P53-3 (67 days)