FOUNDATION DESIGN OF A SHOPPERS DRUG MART IN SQUAMISH, B.C.

by

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ABSTRACT

A foundation design is needed for a proposed Shoppers Drug Mart, located in the Chieftain Shopping Centre in downtown Squamish, British Columbia. The purpose of this study is to investigate the soil conditions at the proposed building site, complete settlement and liquefaction analysis, and provide recommendations for earthwork and foundation design.

Firstly, a comprehensive geotechnical site investigation will need be performed, including two Cone Penetration Test (CPT) boreholes and four auger drillholes. Thereafter, using the data obtained from these tests, as well as performing lab experiments with the soils, soil properties can be determined for the soil stratigraphy of the proposed site. Using these soil properties, a foundation type for the building can then be considered. However, one major issue to consider prior to designing for the foundation is since the proposed site is located in a seismically active region, seismic design considerations will need to be taken in the design. The following report will present the findings of the soils investigation and our recommendation for geotechnical aspects of the project.

The pre-existing building for the proposed development was needed to be demolished prior to constructing the new one. The new proposed development will include the construction of a single storey commercial building about 22 feet high with a maximum surface area of 122 feet by 158 feet.

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TABLE OF CONTENTS

APPENDICES

A. Site Map
B. Regional Geology Map
C. Aerial Photographs
D. Auger Drill Data
E. CPT Data
F. Cross Section
G. Results and Calculations
H. Site Photographs

LIST OF FIGURES AND TABLES

1.0 INTRODUCTION

1.1 Location

A Shoppers Drug Mart is proposed to be built in Squamish, British Columbia, located approximately 70 kilometres from downtown Vancouver. The proposed site has the co-ordinates of N49° 42' 6.20", W123° 9' 10.80". Located in the Squamish Valley region, the proposed development includes a maximum plan dimension of about 22 feet in height and 122 feet by 158 feet in surface area. It will be constructed at the southwest corner of the Chieftain Shopping Centre, located in downtown Squamish at the corner of Pemberton Avenue and 3rd Avenue.

Figure 1: Location map of the proposed site.

1.2 Proposed Development

In order to construct the proposed development, a pre-existing building would need to be demolished. Surrounded by asphalt-paved parking lots at the south and east corners of the mall, the existing and proposed floor grade of the building will be at Elev. 2.86m (geodetic datum). The ground surface of the pre-existing building slopes gently down from east to west, from approximately Elev. 2.5m to Elev. 1.8m respectively.

Figure 2: The pre-existing building site. (Taken March 7, 2007)

Squamish is located on the Coast Belt and is surrounded by volcanic structures, thus the region is at a high risk of seismic activity. Therefore, when designing for the building, seismic design considerations will need to be taken, such as ground motion analysis and liquefaction assessment. In addition, after a site investigation is performed, a design for the foundation type that will keep the foundation intact in the event of a large earthquake will be needed for the building.

After the building is completed, necessary testing procedures such as vibration monitoring from the construction of the foundation, pile driving analysis, and pile integrity testing will be need to be performed. This is to ensure that the piles are intact and to obtain the real bearing capacity values of the piles.

2.0 SITE INVESTIGATION

2.1 Site Physiography

2.1.1 Topography

Since the township of Squamish area is a flat valley floodplain, the elevation remains fairly constant throughout the region, as a result of large scale sediment deposition in the post-glacial period by the rivers and streams. (Squamish District Council. p.12-13) However, after the glacial period, when the glaciers retreated and created the present Squamish Valley, it deposited alluvial deposits from 300 to 400 feet above the general elevation of the valley floor. To the southeast of Squamish is a 600 metres high mountain named Stawamus Chief (otherwise known as the Squamish Chief).

2.1.2 Vegetation

The vegetation in the Squamish region is mostly occupied by a mixed forest of coniferous and deciduous trees. Since the growth rate is high, especially for deciduous trees, they occupy continuously throughout the area. However, before the area was inhabited, the river floodplain was occupied by mostly cedar and spruce trees, in which some of the large stumps still remain standing to this date. (Stathers, 1955, p.19-21) Regardless, these trees are gradually being replaced by deciduous trees, due to extensive logging in the area.

2.1.3 Land Use

Within the lower Squamish valley, land usage is divided into five major categories:

1.) area used for buildings, townsites, and industry
2.) cultivated farm land
3.) farm land abandoned or reverted to pasture
4.) waste land – sand and gravel bars, tidal flats, and natural meadow
5.) forest – virgin forest, second growth, or recently logged

(Reference – Stathers, 1955, p.60)

However, since the mountains and surrounding rivers restrict land usage only to the valley itself, the development of Squamish has followed a linear pattern.

The township of Squamish has six categories of land use: downtown commercial, mixed commercial residential, tourist recreation commercial, highway commercial, residential, and industrial. (Squamish, 1989, p.34-36) Downtown commercial is intended for the development of retail, office, personal services, institutional, entertainment, and government services. (Squamish, 1989 p.34) Mixed commercial and residential is the combination of the two, tourist recreation commercial is intended to attract recreational-tourist opportunities, and highway commercial is land for commercial uses along the side of the highway. (Squamish, 1989 p.34-36) However, with the increasing population in this area, more housing developments are being built north of downtown Squamish, into areas such as Brackendale and Southridge.

2.1.4 Climate

Due to the high surrounding mountains and the sea, the Squamish valley receives approximately 60 to 70 inches of precipitation per year. During the summer, humidity is high on average, approximately in the range of 75 to 87 percent, and daytime temperatures typically ranging from 13 to 18ºC. However, no weather records were kept for the valley, therefore all values presented will only be an estimation. In addition, the values from the Squamish valley region would have little difference to those obtained from weather stations in the Vancouver area.

Snowfall typically occurs near the end of November or the beginning of December, and ends by the end of March or beginning of April. (Stathers, 1955, p.24) During the day, wind from the Howe Sound direction blows into the valley during the day, whereas at night the wind direction is reversed. This pattern of wind allows a fairly constant temperature during the summer, and permits warm daytime temperatures during the winter. However, north winds can occur in both the summer and winter, creating high temperatures and low humidity during the summer and low temperatures and heavy snowfall during the winter. (Stathers, 1955, p.25) These north winds during the winter time can reach 50 km/h in gusts, bringing along an out-flow of cold air from interior British Columbia and spilling over the mountainous terrain of Garibaldi

Park. This will lead to very cold temperatures; however, such occurrences are not frequent, only happening once or twice each year. (Stathers, 1955, p.26)

2.1.5 Drainage

Most of the drainage in the Squamish valley area is provided by the Squamish River, as it is situated in the middle of the valley, and drained through the Squamish River delta, directly into the Howe Sound. The Squamish River receives large amounts of runoff, since the Elaho, Cheakamus, and Manquam Rivers all drain into this river, before entering into sea. (Stathers, 1955, p.30) In addition, the Stawamus River, which is situated on the eastern side of the valley, empties into Howe Sound as well.

Flooding in the Squamish valley area have been a continuous problem, since rapid changes in weather often occur in this area, especially in the month of October since the previous winter's snow has disappeared from the surrounding mountains and added high temperatures and heavy precipitation to the mountains will drain the water rapidly. (Stathers, 1955, p.42) In addition, with the combination of high tides and strong winds, overflowing of the rivers will often occur, especially in the Mamquam River. Roads and bridges will be washed out, and houses will be half-filled with water. Therefore, in downtown Squamish, basements are not allowed in residential units, as a result of poor drainage in that area. Consequently, dykes were built along the rivers surrounding the Squamish region.

2.2 Bedrock Geology

Through research papers and studies done by other geoscientists, it was found that the rock structure of the Howe Sound region, which includes Squamish, were formed mostly by volcanoes. It is part of the formation of the Coast Belt, which contains the Coast Mountains. This mountain range is divided into two segments: Southwestern Coast Mountains, and Southeastern Coast Mountains. Our area of interest is the Southwestern Coast Mountains, which dominantly consists of quartz diorite and granodiorite. (Monger, 1994, p.11) These formations were formed as a product of four distinct geological episodes: islands were first formed in the region approximately more than 140 million years ago; numerous granitic plutons intrusions on the islands between 140 and 90 million years ago; followed by uplifted rocks from 90 to 20 million years ago. Many volcanic activities forced the dykes and sills into the sedimentary rocks, constructing nearby mountain ranges such as Mount Baker and Mount Garibaldi. (Armstrong, 1990, p.42) Within this mountain range rests the District of Squamish, which contains the key attraction in this region, the Squamish Chief. The mountain contains a large mass of granodiorite, which is comprised of mostly early Cretaceous medium-grained granodiorite, in which the rock crystallized approximately 100 million years ago. However, even though these rocks were metamorphosed, they are unsheared, proven by the angular fragments contained inside the rock. These rocks are sparsely jointed, therefore proven to be resistant to erosion. (Mathews and Monger, p.162-163)

2.2.1 Formation of Basement Rocks

Approximately 167 to 91 million years ago, during the Middle Jurassic to mid-Cretaceous era, these quartz diorite, granodiorite, and minor diorite, with minor septa and fault slices of the Triassic and Jurassic strata belonging magmatic art built on the eastern parts of the Wrangellia and Harrison terranes and the overlapping Lower Cretaceous Gambier Group were formed. (Monger, 1994, p.11) The youngest plutons, located in the southwestern Coast Belt, ranges from the age of 110 to 91 million years (Ma) and they overlap in age with the oldest plutons, which are approximately 103 Ma. (Monger, 1994, p.12)

In the southwestern Coast Mountains, land is typically deformed along a discrete, contractional shear zones which are north-northwest trending, and dominantly west-southwest-vergent. Rocks between shear zones can span greater than 10 kilometres wide may be little deformed, and as young as mid-Cretaceous (96 Ma). (Monger, 1994, p.13)

2.3 Groundwater Conditions

The groundwater flow in the downtown Squamish region was found to be comparatively non-existent, because of the flat gradient of the Squamish valley and throughout the downtown core. However, surrounding the valley are many groundwater recharge zones, which includes: lower Cheakamus Valley, Cheekeye Fan, Squamish Valley floodplain, lower Mamquam Valley, and lower Stawamus Valley. There are aquifers that surround these groundwater recharge zones, but none that surrounds our site of interest. (Squamish (B.C.) District Council, p.29)

From the auger drill holes, found in Appendix D, the location of the water table was noted in auger holes A1, A2, and A3. As well, from the CPT logs, the water table was found to be where positive pore water pressure was. The water table was found to be approximately 2 metres in depth.

2.4 Geological Hazards

The Squamish region is surrounded by potentially volcanically active mountains and the risk of melting glaciers. In addition, it is located in a seismically active zone and some parts are located in a delta or alluvial fan. With these conditions and landscape, the potential for hazards is high.

2.4.1 Risk of Volcanoes and Resulting Landslides

Mount Garibaldi, one of the three major volcanic complexes of the Quaternary Cascade magmatic arc, is part of the Garibaldi volcanic belt (GVB) and the closest and largest risk to Squamish. (Monger, 1994, p.232) If a volcano were to erupt on this mountain, it would pose a serious threat to those living in the Squamish and Whistler-Pemberton area. Ash columns can rise to a few hundred metres high, and could affect the regional area's air quality and air traffic. In addition, the flow of the lava can cause slope instability, and destroy homes, roads, and river flows in its path. However this risk is low to moderate since lava tends not to travel too far from its source. Additionally, with lava melting the ice and the resulting ashfall, or tephra, it could contaminate the water supply for the regional area since the catchment area for the regional watershed is downwind from Mount Garibaldi. (GSC, 2005)

The Garibaldi volcano is situated on crystalline basement rock from the Coast Mountains. (Monger, 1994, p.239) Andesitic and dacite lava flows and pyroclastic rocks outlines the ridges of Mount Garibaldi, which was formed near the end of the ice age. The resulting debris lava flow (lahars) from the volcano flowed off the sides of the mountain, eroding the ice off the side of the peak, leaving behind a steep drop-off on the southwest-facing cliffs, and causing the resulting debris avalanche materials consisting of dacitic lavas and tuff-breccias to be accumulated and deposited into the Cheekye Fan, which consists of a upper kame terrace and a lower alluvial fan. (Monger, 1994, p.239)

As well, the resulting lava from the volcano can cause the ice and snow on the mountains in that area to melt very quickly. Most of these materials from Mount Garibaldi will likely end up in the Cheekye River, down the Cheekye valley into the Cheekye Fan, Cheakamus River, or Rubble Creek. However, these events can cause potential catastrophic floods and landslides to areas below these mountains, even affecting areas such as the downtown Squamish and other neighbouring communities such as Brackendale. As well, landslides can occur from a source named The Barrier, which is a steep rock face formed by successive failures of the margin of the Clinker Peak lava flow. (Monger, 1994, p.267) Clinker Peak is in close approximation to Mount Garibaldi, but the rock avalanche materials follow a different path, as most of the debris ended up at a large fan at the mouth of Rubble Creek sometimes down to the Cheakamus

Valley. (Monger, 1994, p.267-269) Consequently, landslides can affect Highway 99, which is the main artery that connects Whistler and Squamish with Vancouver. In addition, landslides can completely destroy communities and deposit unwanted sediments into the river. (GSC, 2005)

2.4.2 Earthquakes

In the southwestern portion of British Columbia, the chance of earthquakes is very high, as this region is a seismically active zone. There are three distinct source regions for earthquakes in this region: earthquakes within the continental crust, deeper earthquakes from subducted oceanic plates, and earthquakes on the subduction boundary between lithospheric plates. (Monger, 1994, p.221) Nonetheless, subduction earthquakes are among the world's largest earthquakes, therefore buildings and structures in this region will have to follow the National Building Code of Canada (NBCC) for seismic hazards caused by horizontal ground shaking.

Earthquakes within the continental crust in the southwestern British Columbia region typically are small earthquakes. However, these earthquakes usually happen at a considerable depth within the crust, at approximately 20km depth, therefore aftershocks are much less than typical California earthquakes, which most earthquakes occur at the top 10km of the crust. (Monger, 1994, p.222) These small crustal earthquakes are a mixture of strike-slip and thrust event with a dominant north-northwest orientation of the principal stress axes suggesting north-northwest compression (Monger, 1994, p.224)

Subcrustal earthquakes in this region refers to the subducting Juan de Fuca Plate, in which this plate is very thin and shallow in depth (maximum 10km depth), therefore aftershocks are rare. (Monger, 1994, p.225) Since the plate is very brittle, the maximum magnitude an earthquake can create is 7. There are 2 locations in which the plate can dip under: west coast of Vancouver Island, and below the Strait of Georgia and Puget Sound. (Monger, 1994, p.225) However, these earthquakes can still cause considerable damage, as demonstrated in earthquakes occurring in 1949 and 1965 at the south end of Puget Sound, with magnitudes in the range of 5 to 6. (Monger, 1994, p.225)

Subduction earthquakes are the strongest and most devastating, providing magnitudes of 8 or greater. In the event of an earthquake, the most intense area will be the subduction boundary between the Juan de Fuca Plate and the North American Plate, as shown in Figure 6. This kind of earthquake is long in duration of strong shaking, mainly associated with large rupture surfaces. In addition, the area of shaking is large compared to other forms of earthquake; therefore it can affect certain types of major structures, such as tall and large structures. The heavy weight of these buildings will increase the liquefaction potential of saturated sands. (Monger, 1994, p.228) These subducting plates are responsible for the formation of the Cascade Range as well as the Pacific Range. However, this type of earthquake is rare, with the last major earthquake of this type happening approximately in the year 1700. (Monger, 1994, p.227)

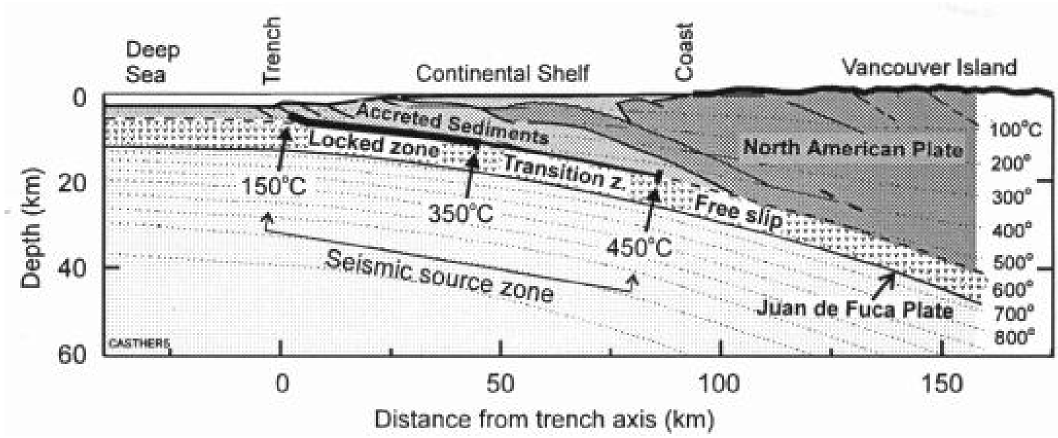

Figure 3: Illustration of the subducting plates <http://gsc.nrcan.gc.ca/geodyn/images/mega12.jpg>

3.0 METHODS OF INVESTIGATION

Prior to the construction of the building, an investigation of the soil stratigraphy of the site was needed. Several testing programs were implemented for this investigation, which included subsurface investigations using an auger drill and performing cone penetration tests. In addition, lab testing which included obtaining the in-situ moisture content of the soil was also performed. Locations of the boreholes are indicated in Appendix A.

3.1 Subsurface Investigation

Two different soils investigation programs were implemented for the site: an auger drill, and a Cone Penetration Test (CPT). The auger drill went to a depth of 20 to 40 feet, whereas the CPT was done to a depth of 100 feet. The auger and CPT logs are located in Appendix D and E respectively.

3.1.1 Auger

Through four different auger drill holes, located around the perimeter of the site of the proposed building, a conclusion of the soil stratigraphy of the site can be determined, as follows:

0 – 4 feet	Dense Fill
4 – 8 feet	Clayey, Low-Plasticity Silt
8 – 16 feet	Fine-Grained, Low-Plasticity Silt
16 – 40 feet	Clean, Fine to Medium to Coarse-Grained Sand

Table 1: Soil Stratigraphy determined by means of Auger Drilling

The dense fill was found to be mainly tan brown, typically fine to medium-grained silty sand with occasional gravel. The layer below is the clayey, low-plasticity silt which contains some organics. This layer is very soft, and will potentially liquefy upon earthquake loading, depending on its plasticity index and liquid limit. This may cause a foundation such as a footing foundation to experience a punching shear failure. The layer beneath is similar to the one above it, as the soil has low plasticity and contains some organics as well. However, this layer is grey in colour and contains some very fine-grained sand, which increases the chance of liquefaction upon an earthquake because of the presence of sand. Finally, the last layer is the sand, which is compact, clean, grey in colour, and is typically fine to medium-grained which coarsens to coarse-grained with depth.

3.1.2 Dynamic Cone Penetration Tests

Upon completion of the auger drill hole, typically a dynamic cone penetration (DCP) test followed and is part of the auger drilling. The DCP tests were performed on auger holes A1 and A4. This test allows the soil stratigraphy logger to confirm the soil properties that has been logged, as well as obtaining a rough estimation of the densities and N_{60} values of the different soil layers. In general, the shallow fill layer exhibited higher blow counts per foot, ranging from 30 to 60 blows per foot, whereas the deeper silt and sand layers lingered around the 20 blows per foot range. This confirms the fact that the silt and sand layers are not dense, therefore increases the chance of liquefaction upon an earthquake.

3.1.3 Cone Penetration Tests

Two CPT boreholes were drilled on this site, one located in the northwestern corner and the other located in the southwestern corner of the demolished building site. Using a drill bit that has a tip area of 10 cm^2 and a sleeve area of 150 cm^2, both drillhole locations exhibited similar soil characteristics, while comparing the parameters of q_c (tip-bearing resistance), f_s (sleeve friction), u_2 (pore pressure), and N_{60} (SPT N-values) to depth.

Overall, many of the layers up to 30 metres (100 feet) in depth exhibited a range in tip-bearing resistance value of 5 to 15 MPa, which would tell us that the relative density of the soil is low. However, there are two layers that exhibit particular high q_c values: the sand to silty sand layer at approximately 5 to 8 metres (16 to 26 feet) with q_c values of up to 20 MPa for CPT#1 and 50 MPa for CPT#2; and the sand layer at approximately 25 to 30 metres (82 to 100 feet) with q_c values of up to 20 MPa for both CPT#1 and CPT#2.

While comparing the sleeve friction (f_s) values, it is found that the values peak at the same places as the tip-bearing resistance values. Typically, the layers display a sleeve friction value ranging from 20 to 100 kPa. However, at the same range of depths as where high q_c values were noticed, f_s values as high as 600 kPa occurred in the sand to silty sand layer and 125 kPa in the sand layer.

During the observation of the pore pressure measurements, it is determined that the water table is located at 2 metres (6.5 feet). Pore pressure starts off as negative from the ground surface to the water table, and as depth increases below the water, pore pressure measurements gradually increased as well. However, at approximately 5 to 8 metres (16 to 26 feet), once again, exceptionally high pore pressure values of up to 35 MPa were observed for both CPT boreholes. This behaviour is caused by the thin lenses of silt contained within the sand layers, causing the building up and subsequent dissipation of pore pressure.

While performing the CPT, N_{60} values were obtained as well. The N_{60} values gives the blow count per every penetrated foot, therefore allowing the estimation of relative density of the soil layers. A high N_{60} value will indicate that the soil is dense whereas a low N_{60} value means the soil is loose. For our two CPT test boreholes, it was found that an N_{60} value of 70 was achieved at 3 metres (10 feet). In addition, at 6 metres (20 feet), the N_{60} value was 120. At these two locations, the soil has very high relative density, and proved through auger drilling that at those depths is the dense silty sand layer. Other than at these two depths, the N_{60} values generally hover around 10 to 20, with the exception at 26 metres (85 feet) for CPT#1 and 28 metres (92 feet) for CPT #2, where the N_{60} value was found to be 40, which was determined to be a silty sand layer. Overall, the soil conditions at the site to a depth of 30 metres are generally relatively loose.

Before performing analysis on the soil layers, soil classification and unit weights of the soils would need to be first obtained. Below are tables illustrating the soil stratigraphy and their respective unit weights at the given depths for the site, using data obtained from the two CPT boreholes:

Depth (m)		Layer	Unit Weight (kN/m^3)	Thickness (m)
From	To			
0.00	1.00	Sensitive Fine Grained	17.5	1.00
1.00	3.75	Clay	18.0	2.75
3.75	5.25	Sand to Silty Sand	19.0	1.50
5.25	7.00	Sand	19.5	1.75
7.00	13.00	Sand to Silty Sand	19.0	6.00
13.00	14.75	Silty Sand to Sandy Silt	18.5	1.75
14.75	19.00	Sand to Silty Sand	19.0	4.25
19.00	23.00	Sand	19.5	4.00
23.00	23.75	Sand to Silty Sand	19.0	0.75
23.75	27.75	Sand	19.5	4.00
27.75	29.25	Sand to Silty Sand	19.0	1.50
29.25	30.25	Sand	19.5	1.00
30.25	32.25	Sand to Silty Sand	19.0	2.00

Table 2: Data from CPT#1 and the unit weights for the soil layers

Depth		(kN/m^3)	Unit Weight (kN/m^3)	Thickness (m)
From	To			
0.00	2.00	Sensitive Fine Grained (Dry)	17.5	2.00
2.00	3.75	Sensitive Fine Grained (Wet)	18.5	1.75
3.75	8.75	Sand to Silty Sand	19.5	5.00
8.75	13.00	Silty Sand to Sandy Silt	18.5	4.25
13.00	13.50	Sand to Silty Sand	19.0	0.50
13.50	14.75	Silty Sand to Sandy Silt	18.5	1.25
14.75	16.00	Sand to Silty Sand	19.0	1.25
16.00	22.00	Silty Sand to Sandy Silt	18.5	6.00
22.00	24.75	Sand to Silty Sand	19.0	2.75
24.75	25.50	Sand	19.5	0.75
25.50	28.25	Sand to Silty Sand	19.0	2.75
28.25	30.25	Sand	19.0	2.00

Table 3: Data from CPT#2 and the unit weights for the soil layers

3.2 Lab Testing

After the soil samples were obtained during the drilling process, they were taken back to the laboratory for further testing. The testing program included obtaining the in-situ moisture content of the soil. The moisture content of the soil can give us a general idea of the strength of the soils as well as acquiring the parameters of the soil properties.

Through the soils investigation, it was found that the fill had relatively low moisture content values, typically ranging from 5% to 10%. However, the layer beneath that, the silt layer, had very high moisture content values. Since the silt contained some organics which were loose, its moisture content values varied from 33 % to 62%. Conversely, at deeper depths, the silt layer decreases in organics content and becomes firmer at the same time, with its moisture content decreasing to a range of 34% to 52%. Both of these silt layers are very much susceptible to liquefaction, since the density of the soil is low and the moisture content remains very high. Finally, the sand layer from the depth of 16 to 40 feet has typical moisture content values of 15% to 28%. This layer generally contains clean and compact, fine to medium-grained sand with occasional gravel. In addition, it possesses moderately high relative densities (approximately D_r of 70%), therefore this layer should be relatively unsusceptible to liquefaction.

4.0 SEISMIC DESIGN CONSIDERATIONS

The Shoppers Drug Mart located in the Squamish district is located in a seismically active area, therefore earthquake design considerations will need to be considered in the design of the structure. Several factors are taken into account in the design process:

- The intensity and magnitude of the earthquake
- The depth of the earthquake and the resulting behaviour of the subsoil
- The magnitude of the forces endured by the building in any given earthquake-induced ground motions
- Amplitude, frequency, and duration of the ground motion

As a result, extensive earthquake analyses can be done for a given site, depending on the importance and structural integrity of the structure. By assessing the hazards caused by the earthquake, it is possible to mitigate the effects of strong earthquakes, reducing the loss of life, injuries, and damages.

4.1 Seed's Simplified Analysis

Using Seed's Simplified Equation developed by Seed and Idriss (1971), the simplified procedure can be used to estimate the cyclic shear stresses due to the earthquake for level sites. For a given depth in each soil layer, typically at the midpoint, a cyclic stress ratio can be calculated for a given magnitude earthquake, depending on the vertical effective stress at that given depth. Consequently, by plotting the values of tip resistance against cyclic stress ratio, the likelihood of liquefaction can then be identified. The following formula presents the calculation of the cyclic stress ratio (CSR):

$$\frac{\tau_{av}}{\sigma'_v} = 0.65\left(\frac{A_{\max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d$$

where

a_{max} = the peak ground surface acceleration for the design earthquake
g = gravity acceleration
o_v = total vertical stress
o'_v = effective vertical stress
r_d = stress reduction factor

($r_d = 1.0 – 0.00765z$ for $z \leq 9.15$m
$r_d = 1.174 – 0.0267z$ for $9.15\text{m} \leq z \leq 23$m)
where z is the depth below ground surface, in metres

Using the chart below, comparing the corrected tip resistance measured from the CPT at each given depth with the calculated CSR, depending on where the point lies, it is then known that whether or not the soil layer will liquefy upon a Magnitude 7.5 earthquake:

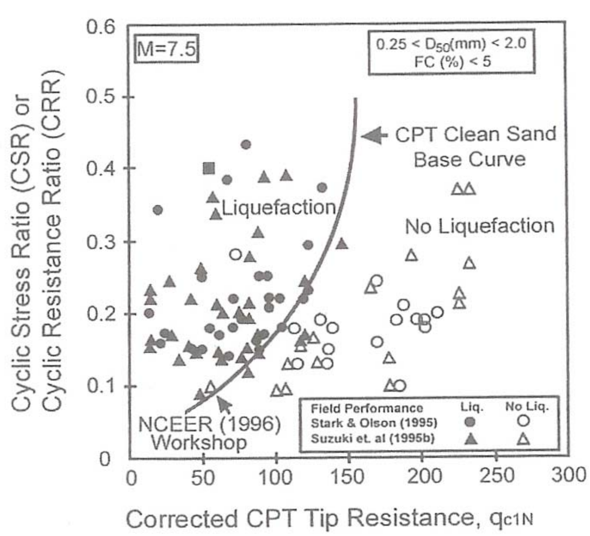

Figure 4: Seed’s simplified liquefaction assessment with Corrected CPT Tip Resistance vs. CSR (obtained from Canadian Foundation Engineering Manual, p. 107)

The table below illustrates the results of the liquefaction analysis from the CPT results:

Depth (m)		Layer Thickness (m)	$(CSR)_{eqk}$	q_{c1N} (kg/cm²)	Potential for Liquefaction
From	To				
0.00	1.00	1.00	0.162	6.61	Yes
1.00	3.75	2.75	0.109	8.47	Yes
3.75	5.25	1.50	0.180	74.23	Yes
5.25	7.00	1.75	0.193	130.65	No
7.00	13.00	6.00	0.203	48.15	Yes
13.00	14.75	1.75	0.189	24.93	Yes
14.75	19.00	4.25	0.174	56.30	Yes
19.00	23.00	4.00	0.151	72.43	Yes
23.00	23.75	0.75	0.136	54.64	Yes
23.75	27.75	4.00	0.121	137.82	No
27.75	29.25	1.50	0.103	40.97	Yes
29.25	30.25	1.00	0.095	77.89	No
30.25	32.25	2.00	0.085	33.73	Yes

Table 4: Liquefaction analysis of CPT#1 (Tip Resistance vs CSR)

As per the analysis, most soil layers are prone to liquefy upon an earthquake with a magnitude of 7.5, except at depths 6.13m, 25.75m, and 29.75m. At these depths, the soil layers are all sand, possessing high tip resistance values thus indicating high relative densities.

Depth (m)		Layer Thickness (m)	$(CSR)_{eqk}$	q_{c1N} (kg/cm²)	Potential for Liquefaction
From	To				
0.00	2.00	2.00	0.129	24.36	Yes
2.00	3.75	1.75	0.153	193.43	No
3.75	8.75	5.00	0.157	415.33	No
8.75	13.00	4.25	0.200	35.64	Yes
13.00	13.50	0.50	0.192	42.64	Yes
13.50	14.75	1.25	0.188	33.77	Yes
14.75	16.00	1.25	0.182	68.82	Yes
16.00	22.00	6.00	0.163	31.80	Yes
22.00	24.75	2.75	0.137	44.99	Yes
24.75	25.50	0.75	0.126	66.03	Yes
25.50	28.25	2.75	0.115	48.67	Yes

28.25	30.25	2.00	0.099	49.43	Yes

Table 5: Liquefaction analysis of CPT#2 (Tip Resistance vs CSR)

The soil layers from CPT#2 are generally loose and potentially liquefiable, with the exception of 2 layers: at depths 2.88m and 6.25m, which are the wet sensitive fine-grained and the sand to silty sand layers respectively. These two layers contain very high tip resistance and N_{60} values, suggesting the high relative density nature of the soil. Otherwise, the results exhibit the other soil layers have low tip resistance values, therefore suggests the low relative density nature of the soil.

In addition to a comparison between corrected CPT tip resistance and the CSR to identify the risk of liquefaction, a comparison using the corrected SPT data between $(N_1)_{60}$ values and CSR can also be done to confirm the risk of liquefaction for a M = 7.5 earthquake. This chart is similar to the one shown in Figure 4:

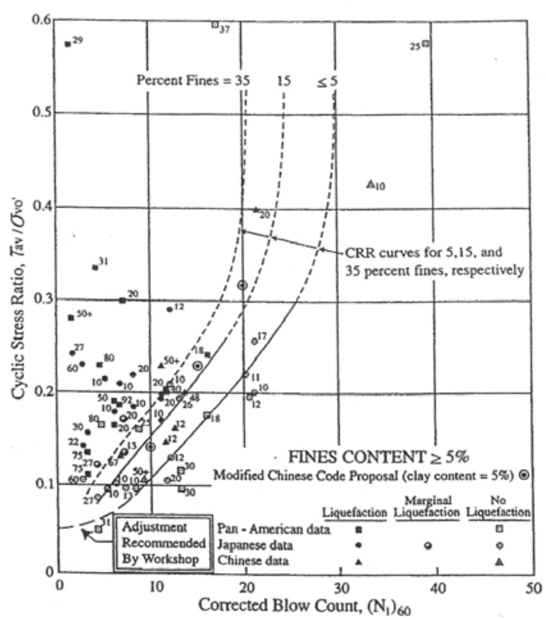

Figure 5: Seed's simplified liquefaction assessment using $(N_1)_{60}$ values vs CSR

Below are the results done for the liquefaction assessment using the SPT resistance method, for CPT#1:

Depth (m)	Layer	$(N_1)_{60}$	$(CSR)_{eqk}$	Potential for Liquefaction
1	Sensitive Fine Grained	4.04	0.13	YES
2	Clay	9.87	0.13	YES
3	Clay	14.87	0.16	YES
4	Clay + Sand to Silty Sand	28.18	0.17	NO
5	Sand to Silty Sand	14.58	0.19	YES
6	Sand to Silty Sand + Sand	25.44	0.19	NO
7	Sand	18.48	0.20	YES
8	Sand to Silty Sand	21.89	0.20	NO
9	Sand to Silty Sand	10.71	0.21	YES
10	Sand to Silty Sand	10.76	0.22	YES
11	Sand to Silty Sand	12.24	0.21	YES
12	Sand to Silty Sand	12.61	0.21	YES
13	Sand to Silty Sand	10.75	0.21	YES
14	Silty Sand to Sandy Silt	8.85	0.20	YES
15	Silty Sand to Sandy Silt + Sand to Silty Sand	10.24	0.20	YES
16	Sand to Silty Sand	14.24	0.19	YES
17	Sand to Silty Sand	15.01	0.19	YES
18	Sand to Silty Sand	13.45	0.18	NO
19	Sand to Silty Sand	14.77	0.18	NO
20	Sand	13.68	0.17	YES
21	Sand	15.52	0.16	YES
22	Sand	15.99	0.16	YES
23	Sand	15.28	0.15	NO
24	Sand to Silty Sand + Sand	14.17	0.15	NO
25	Sand	16.53	0.14	NO
26	Sand	18.30	0.13	NO
27	Sand	7.64	0.13	YES
28	Sand + Sand to Silty Sand	14.33	0.12	NO
29	Sand to Silty Sand	12.63	0.11	NO
30	Sand to Silty Sand + Sand	12.48	0.11	NO
31	Sand + Sand to Silty Sand	12.27	0.10	NO
32	Sand to Silty Sand	9.65	0.09	YES

Table 6: Liquefaction of CPT #1 ($(N_1)_{60}$ values vs CSR)

Comparing the two methods of analyzing the liquefaction potential, it can be observed that very similar results are found. Most of the layers that contain silt are the layers that are most prone to liquefaction, and as well the lower depths are also found to have lower to none liquefaction potential. The following table shows the results from CPT#2:

Depth (m)	Layer	$(N_1)_{60}$	$(CSR)_{eqk}$	Potential for Liquefaction
1.0	Sensitive Fine Grained (Dry)	1.86	0.13	YES
2.0	Sensitive Fine Grained (Dry)	8.93	0.13	NO
3.0	Sensitive Fine Grained (Wet)	26.46	0.16	NO
4.0	Sensitive F.G. (Wet) + Sand to Silty Sand	21.01	0.17	NO
5.0	Sand to Silty Sand	15.96	0.18	NO
6.0	Sand to Silty Sand	16.16	0.19	NO
7.0	Sand to Silty Sand	64.10	0.20	NO
8.0	Sand to Silty Sand	27.54	0.20	NO
9.0	Sand to Silty Sand + Silty Sand to Sandy Silt	11.39	0.20	YES
10.0	Silty Sand to Sandy Silt	7.02	0.21	YES
11.0	Silty Sand to Sandy Silt	10.71	0.21	YES
12.0	Silty Sand to Sandy Silt	7.18	0.21	YES
13.0	Silty Sand to Sandy Silt	10.08	0.21	YES
14.0	Sand to Silty Sand + Silty Sand to Sandy Silt	11.70	0.20	YES
15.0	Silty Sand to Sandy Silt + Sand to Silty Sand	11.90	0.20	YES
16.0	Sand to Silty Sand	13.75	0.19	YES
17.0	Silty Sand to Sandy Silt	9.41	0.19	YES
18.0	Silty Sand to Sandy Silt	10.06	0.18	YES
19.0	Silty Sand to Sandy Silt	10.30	0.18	YES
20.0	Silty Sand to Sandy Silt	10.55	0.17	YES
21.0	Silty Sand to Sandy Silt	10.24	0.17	YES
22.0	Silty Sand to Sandy Silt	11.86	0.16	YES
23.0	Sand to Silty Sand	14.21	0.15	NO
24.0	Sand to Silty Sand	11.35	0.15	NO
25.0	Sand to Silty Sand + Sand	12.58	0.14	YES
26.0	Sand + Sand to Silty Sand	13.77	0.13	YES
27.0	Sand to Silty Sand	12.74	0.13	YES
28.0	Sand to Silty Sand	11.16	0.12	NO
29.0	Sand to Silty Sand + Sand	17.34	0.11	NO

30.0	Sand	10.35	0.11	YES

Table 7: Liquefaction of CPT #2 ($(N_1)_{60}$ values vs CSR)

Once again, the results from the two different analysis presented similar results. Therefore, using both analysis methods, we can clearly identify the trouble zones in the subsurface, and these layers of concern will be taken into consideration when designing the foundation of the building.

5.0 FOUNDATION DESIGN OPTIONS

There are two major types of foundations that can be used for support of the building: a shallow foundation, which would consist of a shallow footing in a variety of shapes, or a deep foundation, such as using timber and steel-pipe piles. Through the analyses of the soil properties of the site, these different foundations types can be considered. As well, ground improvement techniques will be investigated, as the site situates on relatively loose soils.

5.1 Site Preparation

Due to the settlement induced by the structure in the shallow layers such as the clay layer in CPT#1 and the sensitive fine-grained layer in CPT#2, ground improvement techniques would need to be used to help limit the settlements. A few techniques will be investigated for this site, such as preloading, dynamic compaction, and vibro-compaction.

5.1.1 Preloading

Prior to the design of the foundation, settlement issues have to be considered, whether it was short-term or long-term. Therefore, to deal with these issues, preloading is one of the key options. From the two CPT boreholes, the ultimate settlement values are presented as follows:

Borehole	Ultimate Settlement (mm)
CPT#1	476.98
CPT#2	539.97

Table 8: Ultimate settlements from the two CPT boreholes.

The layers that are of main concern due to settlement would be the clay layer in CPT#1 and the sensitive fine-grained layer in CPT#2. The two

CPT boreholes vary significantly in properties, therefore the two retaining wall design structures are very different. Using block dimensions of 59” x 29.5” x 29.5”, and a surcharge consisting of sand with a unit weight of 18kN/m³. The preload time will be when the soil is 90% consolidated by the sand surcharge. The results are presented below:

Borehole	**Time (months)**	**Surcharge (kPa)**	**Height of Wall (m)**	**Number of Blocks**
CPT#1	13.2	60.17	3.34	5
CPT#2	37.2	31.97	1.78	3

Table 9: Number of retaining wall blocks required, without vertical drains

As presented above, the north side of the site would take much longer to reach 90% consolidation compared to the south side. However, this issue can be resolved if vertical drains are inserted in the impermeable layer. For instance, if drains are inserted one tenth of the thickness of the soil layer that is drained on both sides, settlements can be accelerated by up to 25 times. Thus, vertical drains is an option that can be installed in the horizontal direction during preloading, but in this scenario, due to inadequate data, the effects of installing vertical drains will not be investigated.

Consequently, for the south end of the site, since the soil will settle more and take longer to consolidate, 5 blocks will be used throughout, and along the north end of the building, 3 blocks will be used, shown in Figure 8 below. The rest of the calculations for the retaining wall design is presented in Appendix G.

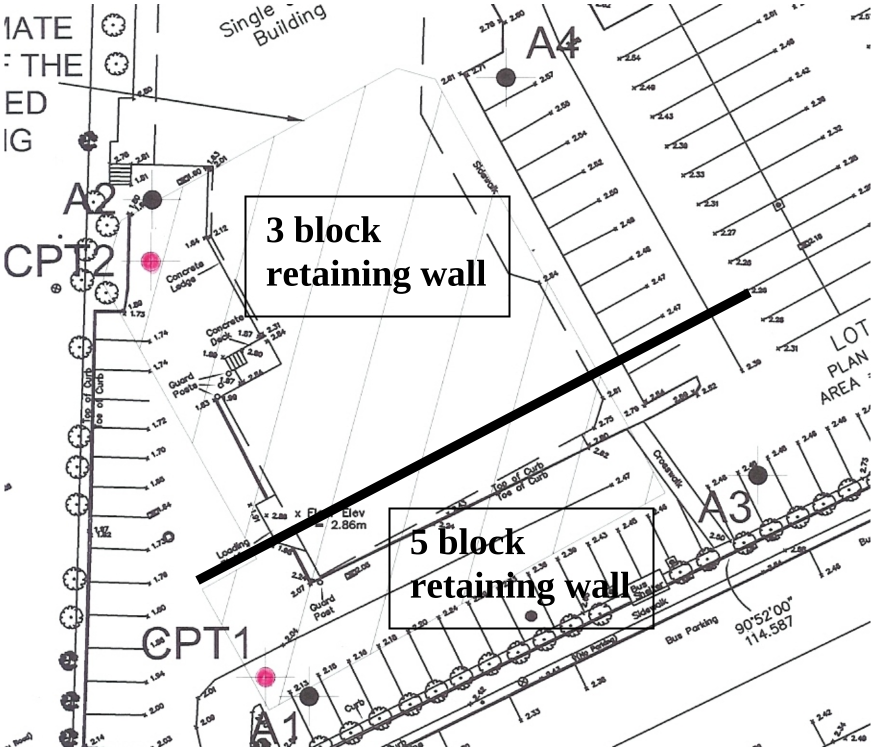

Figure 6: Illustration of the number of retaining wall blocks required for the site.

Settlement gauges and piezometers will be installed at various places on the preload. This will monitor the settlement during preloading, removal of the preload, and the construction period of the structure.

However, this method has its advantages and disadvantages, as presented in the table below:

Advantages	Disadvantages
- low cost	- not possible if there are time constraints
- preload material can be re-used as backfill material	- disposal of fill material may be difficult if need to be transported out
- quiet technique (no vibration/noise), good for the environment	- if site needs to be expanded after construction is complete, initial decision might not have been the most feasible option
- post-construction settlement is relatively small	

Table 10: Advantages and disadvantages of preloading.

5.1.2 Dynamic Compaction

By performing dynamic compaction on the soil, using a free-falling heavy weight over the ground surface, the soil can be compacted for the use of shallow foundations. In addition, this low-cost and effective method can be used to reduce the liquefaction potential of loose soils.

Using a weight of 150 kilonewtons (kN) and a drop height of 5 metres, the compaction can influence a depth of up to 4.33 metres below ground surface. These impacts will be implemented in phases, in which the early phase is a high-energy phase, which are designed to improve deeper layers, followed by the low-energy phase, which are designed to densify the surficial layers. Compaction can be done in intervals of 3 metres apart in a grid, throughout the whole area of the proposed structure site, except within 10.8 metres within other existing structures. (Calculations in Appendix G)

However, there are limitations to this method of ground improvement. Ground deformation and vibration can occur during compaction, as during the impact ground can buckle or deform due to the impact of falling weight. As well, the impact of the tamper on the ground can send waves into the ground, which will affect the nearby structures and the people living and working in them. Therefore, ground vibration of the peak particle velocities have to be monitored during this process, making sure it does not exceed 50mm/s to affect nearby residential structures. Finally, the efficiency of the process of improving clays and fine-grained material remains to be unproven, since during compaction, it creates an increase in pore water pressure.

5.1.3 Vibro-Compaction Processes

This compaction method involves using an approximately 300mm to 400mm in diameter and a 4 to 5 metres long vibrator, which can either be electrically driven or hydraulically driven vibrator with variable frequency. Then the vibrator will penetrate the soil under its own weight, with water or airjetting and the induced vibration to assist with the densification of the soil. By doing so, it can reduce the volume of the soil by up to 10%, thus, the level of the site might be altered by this process, therefore granular material can be placed around the vibrator.

The centre-to-centre spacings of this stone column method can be done in spacings of 1.5 metres, to achieve compaction of the soils of up to 90%. This method of soil improvement has four basic objectives:

- to limit total settlements
- to reduce differential settlements
- to achieve higher bearing capacity
- to increase shear strength

(*CFEM, p.250)*

5.2 Shallow Foundations

Shallow foundations can provide building loads to the earth at a shallow depth, usually consists of spread footings and mats/raft foundations. This category of foundation has a few advantages: low cost, simple to construct, uses mostly concrete (do not need to use many different types of materials), and easier labour (no need to do as many inspections as deep foundations). However, there are a few disadvantages as well, such as: settlement, foundation failures such as bearing capacity failures, punching failures, and slope failures in certain soil conditions.

5.2.1 Strip/Spread Footings

Analysis of the strip and spread footings were done for the following shaped footings: square, circular, continuous, and rectangular. Two methods were used to investigate the bearing capacity and allowable column load of the footing: the Terzaghi method, which is based on a general shear failure, and the Vesic (also known as the Meyerhof method), which includes correction factors for eccentricity, load inclination, and foundation depth. Contrary to the simpler Terzaghi method, the Vesic method includes the influence of shear strength above the base of the foundation and provides a more accurate bearing values and it applies to a much broader range of loading and geometry conditions. (Coduto, p.183) However, for our analysis, to be conservative and using a Factor of Safety value of 3.0, the Vesic values will be considered in the design since it provides a lower bearing capacity values between the two methods, since this will induce a bigger settlement values for the settlement analysis methods.

For the analysis of the shallow foundation, the data from the CPT results will be used. From these two sets of data and results, the dimensions of the footings can be determined, along with the bearing capacity and allowable column load. From the structural engineer, it was given that the column loads (dead and live loads) were 200 kips (889.64kN) and the maximum floor live loads (dead and live loads) were 400 pounds per square feet (psf), or 19.152kPa. Therefore, our design considerations were taken in accordance to these numbers.

The following dimensions and depths of the footings will provide adequate bearing capacities and column loads for the building. Below are the results from CPT#1:

	Square	**Circular**	**Continuous**	**Rectangular**
Width (m)	2	2.5	3	1
Length (m)	---	---	---	4
Depth (m)	1.5	1.5	2	1.5
q_{ult} (kPa)	1063	1056	1027	837
q_a (kPa)	354	352	342	279
Allowable Column Load (kN)	1418	1728	1027	1115

Table 11: Footing dimensions and capacities from CPT#1

For CPT#2:

	Square	**Circular**	**Continuous**	**Rectangular**
Width (m)	2	2.3	2.5	1
Length (m)	---	---	---	2.5
Depth (m)	1	1	2	2
q_{ult} (kPa)	978	988	1281	1422
q_a (kPa)	326	329	427	474

Allowable Column Load (kN)	1305	1368	1068	1185

Table 12: Footing dimensions and capacities from CPT#2

From the allowable column load values obtained from the bearing capacity calculations, resulting maximum settlement and bearing capacity values for the site conditions can then be attained. The Classical method (conventionally known as the Terzaghi method), will be used in this analysis. This theory assumes settlement is a one-dimensional process, in which all strains are vertical. (Coduto, p.218) The results from the Classical method are presented below:

From CPT #1:

	Square	Circular	Continuous	Rectangular
Width (m)	2	2.5	3	1
Length (m)	---	---	---	4
Depth (m)	1.5	1.5	2	1.5
Allowable Column Load (kN)	1418	1728	1027	1115
q (kPa)	390	388	390	314
delta (mm)	308.09	305.46	269.34	253.87

Table 13: Footing load and settlement values for CPT #1 (Classical Method)

From CPT #2:

	Square	Circular	Continuous	Rectangular
Width (m)	2	2.3	2.5	1
Length (m)	---	---	---	2.5
Depth (m)	1	1	2	2
Allowable Column Load (kN)	1305	1368	1068	1185
q (kPa)	350	353	474	521
delta (mm)	396.76	398.83	332.73	292.13

Table 14: Footing load and settlement values for CPT #2 (Classical Method)

Settlement values from these given foundations are noticeably large, greatly exceeding the values given from the guidelines for limiting settlement of framed buildings and load bearing walls, expressed in CFEM page 180. The angular distortion is a ratio between the settlement and the span between columns, and the tolerable limit of this value for a shallow foundation is between 1/150 to 1/250. That means for every 150 to 250 millimetre span between columns, 1 mm of settlement is allowed. The span between columns is determined by the structural engineers. Comparing the allowable maximum settlement values with the calculated settlement values from the four different shapes of foundations using two different computation methods, it is necessary to preload the site prior to any construction of the shallow foundations. The preloading requirements and methods are presented in Section 5.1.1.

5.2.2 Mat Foundation

Since the footprint of the building is relatively large, a mat, also known as a raft, foundation can be considered. Columns and walls of the building would be supported by this common foundation, and this type of foundation is ideal for reducing or distributing building loads in order to reduce differential settlements for surrounding areas.

However, since the building footprint is rather large, different soil conditions will be present throughout the site. Therefore when obtaining the value of coefficient modulus of subgrade reaction of the subsurface soils, the value will only be an approximation. With disturbances of the soils in events such as excavation and placement of foundations, the coefficient modulus will be presented in a range of values for a specific soil type. They are as follows:

Soil Type	**K_{v1} (MPa/m)**
Granular Soil (Moist or Dry)	
Loose	5 – 20
Compact Sand	20 – 60
Dense	60 – 160
Very Dense	160 – 300
Cohesive Soils	
Soft	< 5
Firm	5 – 10
Stiff	10 – 30
Very Stiff	30 – 80
Hard	80 – 200

Table 15: Typical subgrade reaction values (Table obtained from CFEM, p.129, Table 7.1)

Through classification of the top soil layers, it is determined that the granular soil has a K_{v1}, which is the modulus of subgrade reaction for one-foot square plate, of 20 MPa/m. In addition, since the maximum floor load is determined to be 400 psf (equivalent to 19.152 kPa), we are then able to obtain the value of settlement of footing under applied pressure, δ, through this formula:

$$k = q / \delta$$

Using the k-value of 20 MPa/m, we have to convert this modulus to the suitable actual footing dimensions, b. Therefore, we use the following formulas to obtain a foundation width and a modulus for the actual footing dimension b for granular soils:

$$k_{vb} = k_{v1} [(b+1)/2b]^2$$

$$\delta = Iqb(1-\upsilon^2) / E$$

where

I = an influence factor that is dependant on geometry of footing and thickness of compressive soil relative to footing (value is 1, according to CFEM p.161)

b = foundation width

υ = Poisson's ratio (typically 0.3 for drained conditions for most soils)

E = modulus of deformation

The following are the calculations to find the value of the foundation width, b, and the amount of settlement induced by the raft:

δ	0.174557 **174.5566**	m **mm**
b	**0.263078**	**m**

Table 16: Mats foundation results from CPT #1

δ	0.024137 **24.13674**	m **mm**
b	**0.657696**	**m**

Table 17: Mats foundation results from CPT #2

Firstly, the bearing capacities of the foundation were found from the data of the two CPT boreholes, using a Factor of Safety value of 3. Data is presented as follows:

			Terzaghi			**Vesic**		
			Bearing Capacity		Allowable Wall Load	Bearing Capacity		Allowable Wall Load
	Width (B) (m)	**Depth (D) (m)**	**q_{ult} (kPa)**	**q_a (kPa)**	**P/b (kN/m)**	**q_{ult} (kPa)**	**q_a (kPa)**	**P/b (kN/m)**
CPT #1	0.263078	0.5	243	81	21	263	88	23
CPT #2	0.657696	0.5	312	104	68	325	108	71

Table 18: Calculated bearing capacity values for mats foundation

To be conservative, the comparatively lower values will be considered for the calculations settlement analysis.

The Classical Method was used to calculate the settlement induced by the raft foundation from data of the two CPT boreholes. Treating this foundation as a continuous strip footing, with a depth of 0.5m and using allowable wall load from Terzaghi, along with the foundation width obtained from the previous table, the maximum load values can be obtained, as displayed below:

	q (maximum allowable load) (kPa)	**Total Settlement (mm)**
Classical Method		
CPT #1	92	106.83
CPT #2	115.191	192.62

Table 19: Calculated settlement values for mats foundation

As mentioned previously, the maximum live and dead floor load values of the building provided by the structural engineer is said to be 400 psf (equivalent to 19.152 kPa). Therefore, using data from both settlement methods and CPT boreholes, a Factor of Safety value of approximately 5 and 6 would be achieved for CPT borehole 1 and 2 respectively. This greatly exceeds the Factor of Safety requirement of 3, typical in design factors. However, with such large settlement values from the given widths and depths of the foundation, preloading is necessary to reduce the amount of induced settlement from the structure.

5.3 Deep Foundations

A deep foundation with piles can also be considered for this proposed building. Since the upper soil layers of this site are rather weak, pile foundations such as timber piles or steel piles are considered when designing for the foundation. These piles can provide a large load capacity because of strong mid- and lower-layered soils, and it is good for seismically active areas such as the location of this proposed site.

5.3.1 Timber Piles

Timber piles are a good and economical choice for a foundation, since they are made from trunks of straight trees, and trees are abundant in the province of British Columbia. This material is typically tapered, and usually coming from Southern pine or Douglas fir in North America. After the bark and branches are removed from these trees, they can be driven 20 to 60 feet into the ground. In addition, lengths of up to 80 feet for Southern pine and 125 feet for Douglas fir can be obtained. (Coduto, p.379-380)

Conversely, timber piles are very susceptible to damage during driving. Repeated blows by the driving hammer can induce damages which include splitting and brooming at the head and the toe. Even though preventative measures such as using a lighter weight hammer, inserting steel shoes on the toe of the pile, and predrilling prior to installation of the timber piles, it is not necessarily sufficient to prevent damage of the piles. In addition, and the most important point on this given proposed site, timber piles are not usually good for hard and dense soils, as present at the lower-layered soils at this site. (Coduto, p.381) Finally, timber piles typically have an axial design loads of 35-150 kips, but the structural engineers determined that the live and dead loads of the building is 200 kips, therefore this pile type is proven to be inadequate for this building. (Timber Pile Manual, p.14)

5.3.2 Steel Piles

Steel piles are a popular choice for foundation, given its high bearing capacity capabilities and is ideal for dense and hard soil conditions. There are two types of steel piles: H-piles, and pipe piles. Pipe piles will be considered in the design since pipe piles provide larger lateral loads, needed for this site since the great risk of seismic activity is present.

There are two types of steel piles: open-end, and closed-end. In this scenario, only closed-end piles are considered since open-ended piles are primarily for offshore construction. In addition, closed-end pipe piles have higher load capacities than open-end ones.

For our analysis of a steel pile design, a free program called Louisiana Pile design by Cone Penetration Test developed by the Louisiana Transportation Research Center will be primarily used for the analysis of the piles. This program only allows for the pile material to be concrete, so in order to obtain the pile bearing capacities for steel, a 10% reduction from the concrete pile's bearing capacity will be implemented.

This is because steel has a less frictional sliding factor than concrete, therefore the ability for the soil to hold the friction pile up would decrease the bearing capacity by 10%.

In this program, there are three types of analysis for pile capacities based on the CPT data: the LCPC method, Schmertmann method, and de Ruiter and Beringen method. The LCPC method, which is the most popular method in industry standards, relies on the tip resistance averaged over zone above and below the tip of the cone to get an Equivalent Cone Resistance, q_{ca}. Thereafter, adding the side friction and tip resistance, which uses the equivalent cone resistance to calculate, an ultimate capacity for the pile can be obtained. The Schmertmann method divides each layer into approximately equal or representative values of q_c. Then a pile group is represented as a raft, and that would be superimposed at each given depth. There, pile capacities can be calculated at each depth. (Pile Design and Construction Practice, p.185) Finally, the de Ruiter and Beringen method have different procedures for clays and sands. In clays, the friction and end bearing capacities rely on calculating the undrained shear strength, s_u, first from cone resistance, q_c. In sands, the pile end bearing, q_p, is calculated using the cone resistance in a zone 0.7 to 4 pile diameters below the pile tip. As well, pile capacities in overconsolidated sands may be partially reduced due to pile driving, therefore making it hard to obtain an exact value. (CPT in Geotechnical Practice, p.154)

An analysis using three different pile sizes, 10-inch, 12-inch, and 16-inch, is used to calculate the bearing capacity of the piles using the aforementioned three analysis methods. Below are the results from CPT 1, with capacities expressed in tons (metric):

Pile Diameter	Depth (feet)	LCPC	Schmertmann	de Ruiter and Beringen
10	20	21.9	11.9	13.5
	30	39.5	17.5	21.8
	40	53.5	22.4	29.9
	50	67.3	28.6	39.4
	60	85.1	35.0	46.3
	70	107.3	42.6	54.3
	80	129.7	53.1	63.5
	90	150.0	65.1	74.8
	100	171.6	78.9	85.0
12	20	26.3	14.9	16.2
	30	47.4	21.6	26.1
	40	64.2	27.5	35.9
	50	80.7	34.9	47.2
	60	102.1	42.6	55.6
	70	128.8	51.8	65.2
	80	155.7	64.3	76.2
	90	180.0	78.7	89.8
	100	205.9	95.3	102.1
16	20	35.1	21.4	21.5
	30	63.1	31.0	34.8
	40	85.6	38.7	47.8
	50	107.7	48.7	63.0
	60	136.1	59.0	74.1
	70	171.8	71.2	86.9
	80	207.6	87.9	101.7
	90	240.0	107.1	119.7
	100	274.5	129.1	136.1

Table 20: Pile capacity values from CPT#1 for steel piles

Below are the values calculated from CPT 2:

Pile Diameter	Depth (feet)	LCPC	Schmertmann	de Ruiter and Beringen
10	20	29.2	50.5	35.3
	30	49.6	78.4	60.8
	40	65.9	107.1	77.9
	50	86.2	134.9	103.9
	60	106.7	163.7	128.0
	70	126.7	191.5	153.5

	80	148.8	220.3	182.2
	90	170.7	248.1	210.0
12	20	35.0	60.7	42.4
	30	59.5	94.0	72.9
	40	79.1	128.5	93.5
	50	103.5	161.9	124.7
	60	128.0	196.4	153.6
	70	152.1	229.8	184.2
	80	178.6	264.3	218.6
	90	204.8	297.7	252.0
16	20	46.7	80.9	56.5
	30	79.3	125.4	97.2
	40	105.4	171.4	124.6
	50	138.0	215.9	166.2
	60	170.7	261.9	204.8
	70	202.8	306.4	245.6
	80	238.1	352.4	291.5
	90	273.1	396.9	336.0

Table 21: Pile capacity values from CPT#2 for steel piles

Since the loads (dead and alive) value provided by the structural engineers is 200 kips - equivalent to 90.718474 tons (metric), the values represented in red indicates the pile capacity values that were higher than the required loads. Since the LCPC method provides the smallest bearing capacities of the three methods, only values obtained from the LCPC method will be considered.

5.3.3 Concrete Piles

Another choice for a pile foundation could be using piles made from concrete. They usually come in two different forms: an in-situ form for bored piles and cylinders, or in precast form for driven piles in either reinforced or prestressed concrete. (Young, p.194) The in-situ form of concrete piles requires pouring concrete into a preformed hole or driven tube into the ground, with temporary or permanent steel lining tubes which will provide support for the unhardened concrete mix. It will then be left for 28 days before any testing will be done on the piles. (Young, p.196) Reinforced concrete piles are required to either have a steel or plastic reinforcement surrounding the pile. Newer technologies of concrete piles include injecting a polypropylene fibre into the concrete mix to provide the concrete with better strength and lower costs. However, this alternative will provide lower Young's modulus. (Young, p.198) Prestressed concrete piles require a certain amount of stress induced on the pile before being placed on the ground. However, these piles will need to be handled very carefully before placing into the ground. As well, during driving of these piles, adequate cushioning material must be needed between the driving head and the concrete pile; otherwise the pile is very prone to breakage. This cushioning material can be made of plywood, and it will situate between the hammer head of the pile driving rig, which will also contain a hammer cushion, and the concrete pile itself.

Using concrete for piles allows the easy adjustments of the concrete mixing material for different environments and usages of these piles. However, even though concrete piles are a cheaper and provide a more adaptive to the environment alternative to steel piles, one of its great disadvantages is its low shear strength. That means when large lateral loads are acted on the pile, it is very prone to failure. (Young, p.194)

Below are values obtained from both CPT #1 and #2. As described previously, the values will be 10% greater than the bearing capacities from steel:

Pile Diameter	Depth (feet)	LCPC	Schmertmann	de Ruiter and Beringen
10	20	24.4	13.3	15.0
	30	43.9	19.5	24.2
	40	59.5	24.9	33.2
	50	74.8	31.8	43.7
	60	94.5	38.9	51.5
	70	119.3	47.4	60.4
	80	144.2	59.0	70.6
	90	166.6	72.3	83.1
	100	190.6	87.6	94.5
12	20	29.3	16.5	18.0
	30	52.6	24.1	29.0
	40	71.4	30.5	39.8
	50	89.7	38.8	52.5
	60	113.4	47.4	61.8
	70	143.1	57.5	72.4
	80	173.0	71.4	84.7
	90	200.0	87.5	99.7
	100	228.7	105.8	113.4
16	20	39.0	23.8	23.9
	30	70.2	34.4	38.7
	40	95.1	43.0	53.1
	50	119.6	54.1	70.0
	60	151.2	65.5	82.4
	70	190.8	79.1	96.6
	80	230.7	97.6	112.9
	90	266.6	119.0	133.0
	100	305.0	143.5	151.2

Table 22: Pile capacity values from CPT#1 for concrete piles

Pile Diameter	Depth (feet)	LCPC	Schmertmann	de Ruiter and Beringen
10	20	24.4	13.3	15.0
	30	43.9	19.5	24.2
	40	59.5	24.9	33.2
	50	74.8	31.8	43.7
	60	94.5	38.9	51.5
	70	119.3	47.4	60.4
	80	144.2	59.0	70.6
	90	166.6	72.3	83.1
	100	190.6	87.6	94.5
12	20	29.3	16.5	18.0
	30	52.6	24.1	29.0
	40	71.4	30.5	39.8
	50	89.7	38.8	52.5
	60	113.4	47.4	61.8
	70	143.1	57.5	72.4
	80	173.0	71.4	84.7
	90	200.0	87.5	99.7
	100	228.7	105.8	113.4
16	20	39.0	23.8	23.9
	30	70.2	34.4	38.7
	40	95.1	43.0	53.1
	50	119.6	54.1	70.0
	60	151.2	65.5	82.4
	70	190.8	79.1	96.6
	80	230.7	97.6	112.9
	90	266.6	119.0	133.0
	100	305.0	143.5	151.2

Table 23: Pile capacity values from CPT#1 for concrete piles

Outlined in red are the values that exceed the building load of 200 kips (90.718474 metric tons). We will also consider the LCPC method as the main method of analysis for concrete piles. In addition, the same pile diameters of 10, 12, and 16-inches will be considered.

6.0 THE CONSTRUCTION PROCESS

6.1 Vibration Monitoring

During the pile driving process, and depending on the diameter of the pile being driven, ground vibration was felt throughout and outside the construction site. Therefore, around the vicinity of the construction site, vibration monitoring tests using a seismograph were performed to make sure the vibration caused by the pile driving process would not affect the structural integrity of the nearby residential and commercial buildings. These tests were done from a few feet away from the driven pile to as far as 160 feet away. According to the British Columbia Building Code 2006, the peak particle velocity (PPV) which is the maximum allowable vibration for residential complexes is 50 millimetres per second (mm/s) and for commercial complexes is 100mm/s. The table and figure below demonstrates the amount of PPV induced from the pile driving relative to the distance away from the driven pile:

Distance (ft)	Average PPV (mm/s)	Max PPV (mm/s)	Min PPV (mm/s)
20	8.345	10.4	6.1
40	8.5575	11.7	7.11
80	5.232	6.86	4.03
120	2.2	3.05	2.03
160	1.46	1.9	1.02

Table 24: Vibration monitoring data

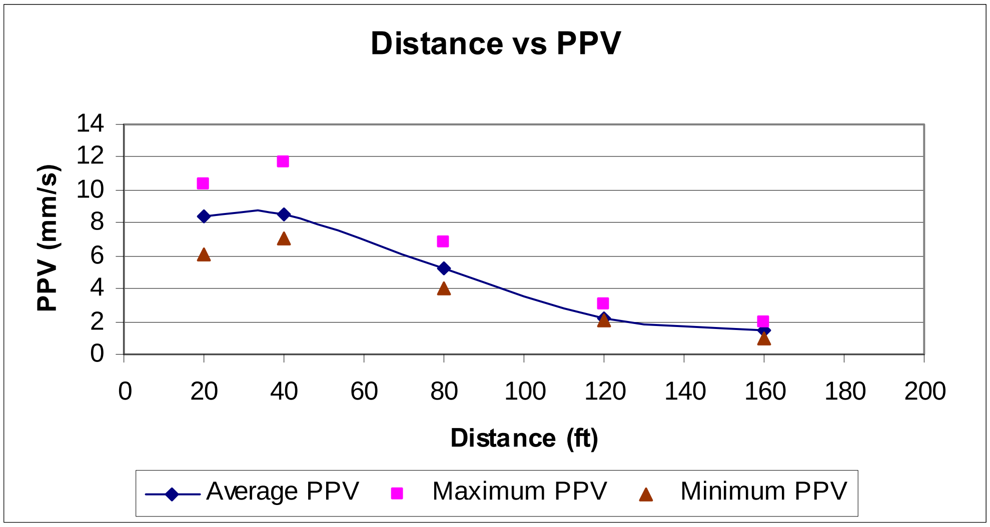

Figure 7: PPV relative to the distance away from the source

These data were collected at random times when different diameter piles were driven, therefore only upper and lower-bound values were plotted, as well as the average of the values obtained. The closest residential and commercial complexes were at least 100 feet away from the construction site, therefore from the plot above it proves that none of the surrounding structures were affected by the pile driving process.

In addition, vibration tests were done within 20 feet of the driven pile as well. This was done to ensure that the adjacent commercial complexes will not experience structural damage during pile driving. Values obtained from the monitoring are shown below:

Driving Energy (ft-lb)	Hammer Drop (ft)	Distance (ft)	PPV (mm/s)
50,000	8	3	115
		4	72
		10	28
		15	20

Table 25: Vibration monitoring close to the source

Since the typical driving energy of the hammer was 50,000 ft-lb, then using the largest hammer drop of 8 feet, it was found that only when the seismograph was placed 3 feet away from the pile that the PPV exceeded the commercial building limit of 100 mm/s. However, the closest commercial building to any pile placed was at least 10 feet away, therefore the vibration induced by the pile driving did not affect the structural integrity of the adjacent commercial buildings.

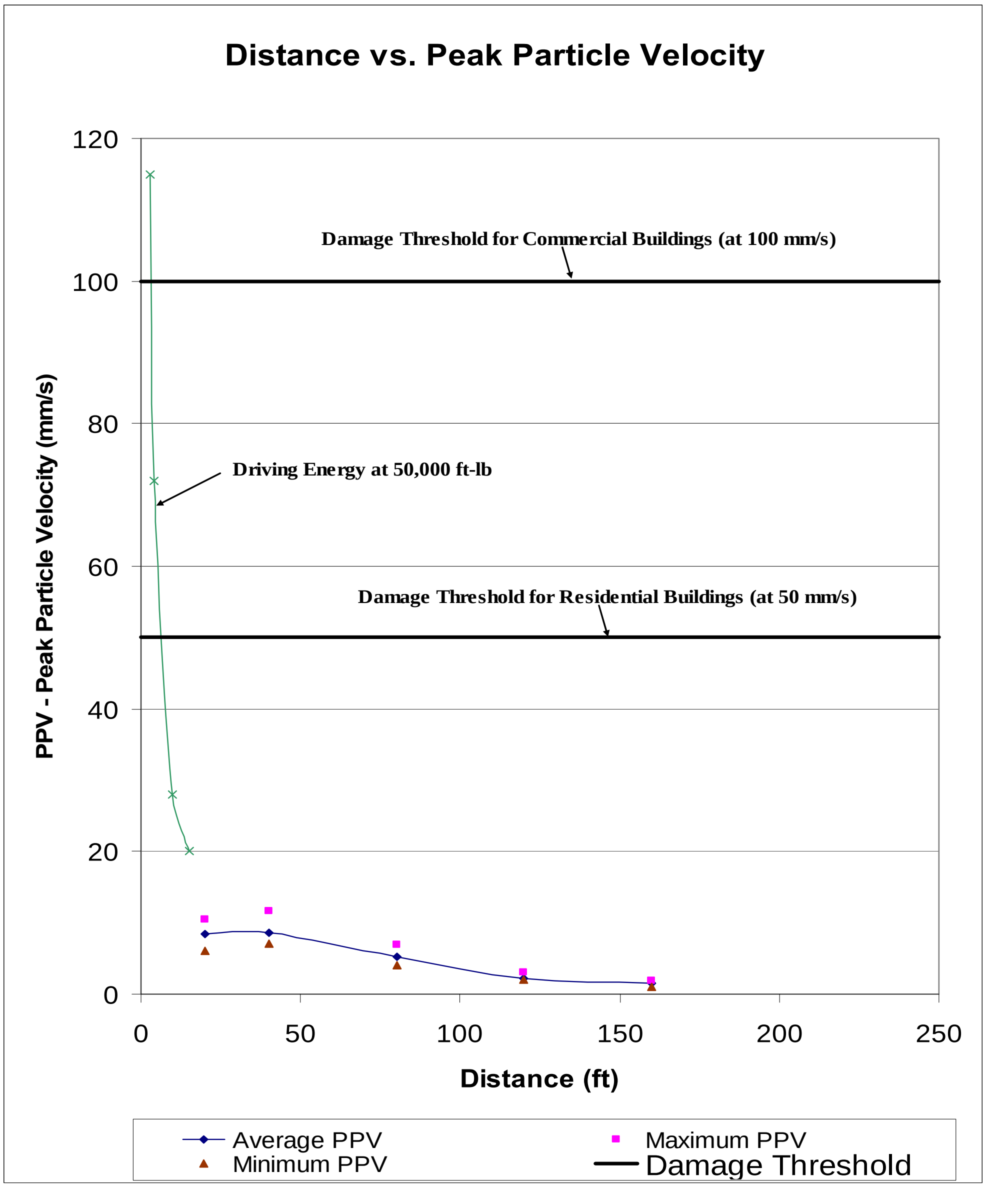

Figure 8: Distance vs Peak Particle Velocity values relative to damage threshold values

6.2 PDA Testing and Checking Integrity of Piles

To ensure the driven pile is performing as expected, pile driving analyzer tests were performed on the pile while it was being driven. The Case Pile Wave Analysis Program (CAPWAP) is a combination of the Case method, which uses wave trace data to determine the static pile capacity, and the wave equation analysis, which uses a much more precise numerical model but does a weak estimate of the actual energy delivered by the hammer. Producing the values of ultimate resistance in the soil "springs", as well as the quake and Case method damping factor, CAPWAP measures the total capacity, along with the shaft and toe resistances of the pile. In addition, with the use of the combination of an accelerometer and strain gage during a PDA test, the axial forces of the pile as well as the particle velocity of the waves travelling through the pile and the pile displacement during the hammer blow can be calculated. By obtaining these data, we can then produce a plot of time versus particle velocity to check the integrity of the pile, ensuring the pile did not break or fracture during the pile driving process, or plotting time versus force to find out whether the pile undergoes compressive or tensile forces while the pile is being driven. This analysis allows the simulation of a static load test.

PDA tests for the Shoppers Drug Mart site were done on June 19th, 2007 and July 3rd, 2007 by AATech Scientific Inc. based in Ottawa, Ontario. This analysis was done on a total of 10 randomly selected test piles, varying in pile diameter and location. The values were obtained using CAPWAP® Version 2000-1. Using a 6300 pound hammer, with height of hammer drop varying from 5 to 8 feet, values were obtained from these tests. A summary of the values obtained from the tests are displayed below:

Date	Pile Number	Pile Diameter (in)	Gridline	Total Capacity (kips)	Allowable Capacity (kips)	Factor of Safety
19-Jun-07	3067	10	1A	192.9	100	1.929
19-Jun-07	3091	16	1A	165.1	100	1.651
19-Jun-07	3091	16	1A	178.4	100	1.784
19-Jun-07	**3100**	**16**	**1**	**144.9**	**100**	**1.449**
19-Jun-07	**3113**	**16**	**2**	**180.1**	**100**	**1.801**
19-Jun-07	3207	12	4	127.2	100	1.272
19-Jun-07	3230	12	4	139.9	100	1.399
03-Jul-07	3064	10	1	185	100	1.85
03-Jul-07	3079	12	1A	187.2	100	1.872
03-Jul-07	3097	16	1	205	100	2.05
03-Jul-07	**3100**	**16**	**1**	**163.4**	**100**	**1.634**
03-Jul-07	**3113**	**16**	**2**	**204.6**	**100**	**2.046**
03-Jul-07	3120	16	2	195	100	1.95

Table 26: Final tested pile capacity values

As per the table above, as time progresses, we can see the pile increase in total capacity. When the first PDA tests were done for the piles on June 19th, 2007, the piles were only placed in approximately 3 days before the testing, therefore the piles did not have enough time to settle into the soil stratums. However, since the surrounding soil layers around the driven piles are normally consolidated, it disturbs and remolds the surrounding soil, therefore generating excess pore water pressures. But with time, through the settling of the piles and the dissipation of pore water pressure of the soil around the pile as a result of soil consolidation, the pile load carrying capacity increases over time. Therefore, several more PDA tests were done on the pile approximately 2 weeks after installation of the piles, some of which were re-tested to observe this "pile aging" affect of the piles.

7.0 RESULTS AND CONCLUSION

Preloading for the site would not be a practical choice for ground improvement for the site, because of two main reasons: firstly, it will require too much time for the preload (approximately 13.2 and 37.2 months for north and south ends of site). Secondly, and the most important reason, is after seismic design considerations, it was found that the below soil stratums contain many layers that will liquefy upon an earthquake, it is then necessary to perform soil densification measures. Therefore, the best solution for ground improvement would be to use vibro-compaction. To improve the quality of the subsurface, it is then necessary to have centre-to-centre spacings of 1.5 metres.

For the foundation of the building, it is determined that shallow foundation by itself will not be adequate, since the subsurface soils are very prone to liquefaction upon an earthquake. In addition, other than a mats foundation, settlements will be a large issue for the shallow foundation as well, mainly because of the upper weak soils. Therefore, a deep foundation is our only option for the foundation, to ensure that the foundation reaches a layer of soil that will not liquefy.

After considering the three different pile foundation types, it is found that steel piles are best suited for the given environment of the proposed site. Even though timber piles are the most cost effective choice, since the soil layers below the depth of 15 to 20 feet are relatively dense and hard soils, timber piles are prone to split or break apart while pile driving in such scenarios. In addition, the given dead and live loads exceed the typical bearing capacity of this type of pile.

Concrete piles are a cheap alternative as well, but since as stated previously, one of its greatest disadvantages is that it has a low shear strength. Since this proposed site is situated in a seismically active area, it would be very risky to place concrete piles as foundation, as in such events the piles are prone to breaking apart, causing the foundation of the building to fail. Therefore, a pile foundation made of steel would be the best choice out of all, since steel has a higher shear strength. In a case of an earthquake, the piles are less prone to breaking apart. In addition, since the soil stratum of the site contains relatively harder material, it is more suitable to use a higher strength material such as steel, allowing the piles to not break while driving through such hard soils.

Using the proper diameter of the pile and determining the amount of piles needed in a pile group at certain locations is up to the discretion of the structural engineer. A proper length of the pile to use should be 60 feet, since at such depth the steel piles would not need to be spliced and would provide an adequate amount of bearing capacity. Splicing of the pile will cause the piles to fail at the spliced joints in the event of an earthquake, therefore splicing would not be an option for this foundation.

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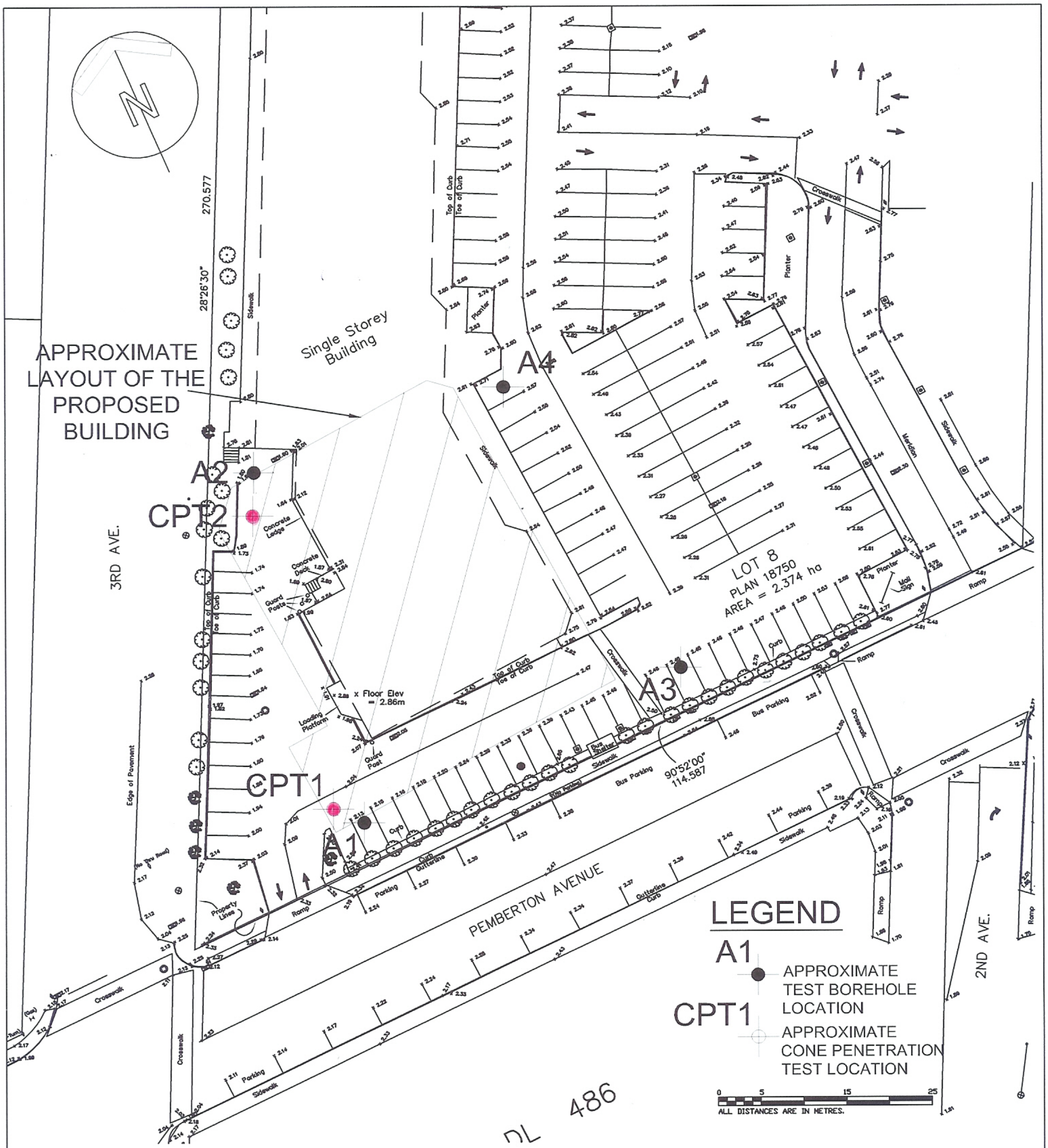
APPENDIX A: SITE PLAN



REFERENCE:
TOPOGRAPHIC SURVEY PLAN BY BUNBURY & ASSOC.
DATED JAN. 10, 2007.

PROJECT No.:	V07-105
PROJECT:	PROPOSED SHOPPERS DRUG MART
LOCATION:	CHIEFTAIN SHOPPING CENTRE, SQUAMISH, BC

CENTENNIAL GEOTECHNICAL ENGINEERS LTD.

SITE MAP WITH APPROXIMATE TEST BOREHOLE LOCATIONS

DATE:	DRAWN BY:	SCALE:	FIGURE:
FEB. 16, 07	AN	AS SHOWN	2

APPENDIX B: REGIONAL GEOLOGY MAPS

GEOLOGICAL SURVEY OF CANADA
DEPARTMENT OF MINES AND TECHNICAL SURVEYS

PRELIMINARY SERIES

LEGEND

CENOZOIC

QUATERNARY

12 Alluvial, marine, and glacial deposits

GARIBALDI GROUP

11 Basalt, andesite, dacite, and rhyodacite flows; minor pyroclastic rocks. May include some Tertiary rocks

CRETACEOUS AND TERTIARY
UPPER CRETACEOUS, MIDDLE EOCENE, AND LATER

10 10a, basalt flows or sills; dykes and minor pyroclastic rocks; 10b, sandstone, shale, conglomerate; minor tuff and coal

MESOZOIC

CRETACEOUS

9 9A. NANAIMO GROUP: shale, sandstone, conglomerate, coal
9B. HELM FORMATION: metavolcanic rocks, conglomerate, limestone
9C. EMPETRUM FORMATION: metavolcanic rocks, conglomerate, limestone
9D. CHEAKAMUS FORMATION: greywacke, conglomerate, arkose; minor argillaceous and calcareous rocks

CRETACEOUS AND/OR EARLIER
UPPER CRETACEOUS AND/OR EARLIER

8 Quartz-feldspar porphyry

JURASSIC AND CRETACEOUS (?)
GAMBIER GROUP

7 Tuff, breccia, agglomerate, andesite, argillite, chert, greywacke, quartzite, conglomerate; minor schist, granulite, lime-silicate rock, scarn

JERVIS GROUP

6 6a, basalt, andesite, associated pyroclastic rocks; minor limestone, dolomitic limestone, chert, argillite; 6b, mainly conglomerate, greywacke, sandstone, argillite; minor greenstone; 6c, metavolcanic rocks, metasedimentary rocks; metadiabase

TRIASSIC
UPPER TRIASSIC (mainly or entirely)
VANCOUVER GROUP

3 Andesite, basalt, quartzite, argillite, limestone, schist. May include some undifferentiated late Palaeozoic rocks

5 Undivided volcanic and sedimentary rocks equivalent to 3–7: minor migmatite and gneiss; bedded rocks seen from the air

PALAEOZOIC(?) AND MESOZOIC

TRIASSIC OR EARLIER
BOWEN ISLAND GROUP

2 Mainly greenstone; minor chert and greywacke

TWIN ISLAND GROUP

1 Hornblende-andesine granulite, gneiss, schist, amphibolite, conglomerate, quartzite, meta-arkose, lime-silicate rocks; migmatite

4 Undivided metamorphic rocks: 4a, gneiss, schist, hornfels, metavolcanic rocks; some quartzite, greywacke; minor slate, argillite; 4b, migmatite, agmatite; 4c, migmatite, gneiss, and metamorphic dykes in plutonic rock; observed from the air

PLUTONIC ROCK

MESOZOIC AND (?) CENOZOIC

CRETACEOUS AND EARLIER (mainly)

13 Undivided; 13a, leucocratic rocks; granodiorite, quartz monzonite, quartz diorite; minor granite and mesocratic rocks; 13b, leucocratic rocks; probably Tertiary; 13c, mesocratic and melanocratic rocks: quartz diorite, diorite, granodiorite; minor hornblende gabbro and leucocratic rocks

Geological boundary (approximate)
Limit of geological mapping
Bedding, top known (inclined, top determined by pillows)
Bedding, top unknown (inclined, vertical)
Gneissosity, schistosity, foliation (inclined, vertical, dip unknown)
Fault, shear zone (approximate)
Syncline
Glacial striae (direction of ice movement known, unknown)
Fossil locality
Mineral occurrence

MINERAL SYMBOLS

Copper	Cu	Lead	Pb
Gold	Au	Molybdenum	Mo
Iron	Fe	Silver	Ag
Zinc	Zn		

Geology by J.E. Armstrong 1946–51; W.R. Bacon 1950–51, 1959; H.H. Bostock 1956–58; A.F. Buckham 1943, 1946; C.H. Clapp 1911; W.H. Mathews 1946–47, 1958; P. Read 1959

Compilation by H.H. Bostock, 1962

Cartography by the Geological Survey of Canada, 1963

Road, all weather
Other roads
Cart track
International boundary
District boundary
Forest Reserve boundary, Indian Reserve boundary
Railway
Power transmission line
Post office
Intermittent stream
Marsh
Glacier
Sand or gravel
Contours (interval 500 feet)
Height in feet above mean sea-level

Base-map by the Surveys and Mapping Branch, 1959

Mean magnetic declination, 23°21' East, decreasing 3.4' annually. Readings vary from 24°01' E in the NW corner to 22°41' E in the SW corner of the map-area

MAP 42-1963
GEOLOGY
SQUAMISH
(VANCOUVER, WEST HALF)
BRITISH COLUMBIA

Scale: One Inch to Four Miles = $\frac{1}{253,440}$

Miles 4 0 4 8 12

5.1.2
A. Geol.

DESCRIPTIVE NOTES

Timber line in the map-area reaches about 6,000 feet above sea-level.

Units 1 to 3 comprise more or less isolated bodies of metavolcanic and metasedimentary rock. The Twin Island Group (1) comprises rocks of medium to high metamorphic grade whose contacts with the surrounding plutonic rocks are commonly either complex migmatite zones or faults. This group is overlain by the Gambier Group (7) in Mount Seymour immediately southeast of Squamish map-area. The Bowen Island Group (2) is probably a less metamorphosed equivalent of the Twin Island Group and is characterized by tight folds trending slightly north of west. Rocks of the Bowen Island Group are commonly recrystallized at their contacts with mesocratic plutonic rocks. In the north part of Bowen Island, contact zones between pendant rocks and mesocratic plutonic rocks have later been fractured and intruded by minor leucocratic plutonic rock. The Vancouver Group (3) is composed chiefly of metavolcanic rocks, but cherty and slaty sediments, correlated with the Bisher Series, are exposed in the canyon of Stawamus Creek at the extreme southwest corner of the area. Pillow structures are present locally. Rocks of the group are fractured and sheared and seem to have a general northwesterly trend and steep dips.

Gneiss, schist, migmatite, and hornfels (4) occur as isolated pendants or as discontinuous bodies along the margins of pendants. These may be remnants of Twin Island Group rocks, or more highly metamorphosed bodies of later pendant rocks. The Ashlu Creek pendant, a long narrow body that was traversed only locally, contains some volcanic and sedimentary rocks but is composed chiefly of biotite schist and gneiss. The upper part of Cloudburst Mountain is an intrusion breccia consisting of migmatite in numerous angular randomly oriented blocks from a fraction of an inch to several feet across in a matrix of unfoliated quartz diorite.

The Gambier Group (7) consists of volcanic and sedimentary rocks less altered than those of the Twin Island and Bowen Island Groups, and characterized by north to northwest trending folds. A conglomerate at the base of the Gambier Group on Gambier Island and at Alberta Bay contains detritus and boulders of plutonic rocks and rocks like those of the Bowen Island Group. The contact between the Gambier Group and the mesocratic granodiorite is therefore considered to be an unconformity. As similar plutonic rocks intrude the Bowen Island Group, the latter is believed to be older than the Gambier Group. An imprint of an ammonite in black slates at the north end of Gambier Island indicates that these rocks are Mesozoic. On the basis of lithologic and stratigraphic similarity, pendant rocks east of Britannia Beach have been correlated with the Gambier Group.

Age relations between the Jervis Group (6) and pendant rocks in the east of the area are unknown except that all are earlier than some phases of the plutonic rocks. The group is divided into three units in tentative order of increasing age. Granite cobbles are present locally in 6b.

Pendant rocks (5) northwest of Phantom Lake are tilted southwest but are otherwise largely undeformed. These rocks are almost entirely bedded argillaceous quartzite and show ripple marks, crossbedding, and concretions, but are apparently devoid of fossils. The pendant is intruded by quartz diorite sills at the head of Tzoonie Creek and a small isolated mass of dense black basaltic rock is present along the northeast margin. A belt of metamorphic rocks (4) perhaps ½ mile wide extends along the south margin of the pendant and into the plutonic rocks to the east. The lack of intense metamorphism in most of the pendant and the rather consistent moderate southwest dip suggest that it is younger than the Ashlu Creek pendant (4). Volcanic rocks, chert, and argillite (3) on Thormanby Island are tightly folded along the northwest coast, but on the south pillowed volcanic rocks and chert fragment breccias are relatively less deformed. Pendant rocks (3) north of Mamquam River are chiefly volcanic rocks but some chert and argillite is present near the east border of the map-area.

The Britannia Beach, Britannia Range, and Brunswick Point pendants (8) are intruded by dykes, small plug-like bodies, and irregular masses of green to purplish green, grey, or grey-brown, mostly fine-grained to porphyritic rocks commonly showing indistinct crystal outlines. In places these rocks resemble altered quartz diorite and patches of it may be found in some of the finer grained phases. These bodies are characteristically associated with pendants containing andesitic volcanic rocks and may be correlatives of the quartz porphyries of Jervis Inlet (8).

The Nanaimo Group (9A) is cut by part of a major northeast trending fault zone that extends along the east coast of Vancouver Island of which only the limiting northeast and southwest faults are shown. Individual faults within the group are chiefly easterly thrusts. Close breaks are exposed only in the lower formations; in the higher formations dislocation gives way to sharp overturned folds. Except where associated with dislocations, folding in the Nanaimo Group is broad and open. The Upper Cretaceous rocks north of Garibaldi Lake (9B–D) have been dated from Inoceramus sp. in the upper part of the middle member of the Cheakamus Formation. These rocks are intruded by plutonic rocks but include conglomerate beds containing fragments of plutonic rocks.

Plutonic rocks (13) are divided in part into two groups; leucocratic (13a) and mesocratic to melanocratic (13c). They are separated at approximately 15 per cent mafic minerals as estimated in the field. In general this follows the biotite-hornblende classification of Roddick and Armstrong, the darker rocks mostly but not always being characterized by higher proportions of hornblende. The more mafic plutonic rocks (13c) tend to follow the margins of pendants composed in part of volcanic rocks. Leucocratic plutonic rocks (13a) tend to occur in areas remote from such pendants except where they may have been intruded or faulted near or into these rocks. Movement of the plutonic rocks is thought to be mostly during the Upper Cretaceous or earlier and two samples with potassium-argon ages of 95 and 99 million years are reported from plutonic rocks immediately east of Horseshoe Bay. Some phases of the plutonic rocks are however known to be younger than the Upper Cretaceous north of Garibaldi Lake. The lack of marked deformation during and after intrusion of the leucocratic plutonic rocks (13b) near Squamish suggests that these also may be younger than the tilted, block faulted Upper Cretaceous sediments 15 miles to the north. Foliation is common in the older plutonic rocks near contacts with pendants and may be extensively developed elsewhere. In the Jervis Inlet and Howe Sound areas it mostly strikes northwest and dips steeply, but in lower Cheakamus Canyon more westerly strikes are common.

Sediments (10b), with an aggregate thickness near Vancouver of 4,000 feet, thicken southward. The proportion of sandstone and conglomerate to shale also decreases away from the mountain front. Plant fossils in sediments south of Burrard Inlet indicate a Middle Eocene age, but plant microfossils in similar rocks north of the Inlet suggest a probable Upper Cretaceous age. Basaltic rocks (10a) cut sediments and plutonic rocks. An unusual tuff, containing rounded pebbles up to several inches in diameter, outcrops on the south side of False Creek, and fine-grained reddish and orange-banded tuffs have been exposed in several excavations in downtown Vancouver.

The Garibaldi Group (11) is exposed mainly in the northeast corner of the map-area but small bodies and dykes of similar rock are known as far south as Kallahane Creek and as far west as the ridge west of Pokosha Creek. A particularly fresh, non-vesicular, dark purple-blue, fine-grained, volcanic rock forms a dyke-like body along the northeast contact of a small pendant on the east wall of Seymour valley below Balfour Creek. The principal phenocrysts are plagioclase. Small inclusions of altered quartz diorite are present at the north contact where the dyke is chilled against the wall-rock. This body is tentatively correlated with the Garibaldi Group but may be older.

Minerals containing Cu, Fe, Pb, Zn, Ag, Au, and Mo occur within the map-area. The most important deposit, that at Britannia mine, is in chloritic and sericitic schists adjoining the Britannia shear zone. The principal sulphides are pyrite and chalcopyrite which form stringer lodes and massive replacements. Heavy concentrations of sphalerite are present locally. The known important mineral deposits are in or associated with pendants, and the best chances of finding more appear to be by examining pendants northwest of Britannia and perhaps east of Jervis Inlet.

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MAP 42-1963
SQUAMISH
BRITISH COLUMBIA

WEST HAL

LEGEND

CENOZOIC

QUATERNARY

12	Alluvial, marine, and glacial deposits

GARIBALDI GROUP

11	Basalt, andesite, dacite, and rhyodacite flows; minor pyroclastic rocks. May include some Tertiary rocks

CRETACEOUS AND TERTIARY

UPPER CRETACEOUS, MIDDLE EOCENE, AND LATER

10	10a, basalt flows or sills; dykes and minor pyroclastic rocks; 10b, sandstone, shale, conglomerate; minor tuff and coal

MESOZOIC

CRETACEOUS

9	9A. NANAIMO GROUP: shale, sandstone, conglomerate, coal 9B. HELM FORMATION: metavolcanic rocks, conglomerate, limestone 9C. EMPETRUM FORMATION: metavolcanic rocks, conglomerate, limestone 9D. CHEAKAMUS FORMATION: greywacke, conglomerate, arkose; minor argillaceous and calcareous rocks

JURASSIC AND CRETACEOUS (?)

GAMBIER GROUP

7	Tuff, breccia, agglomerate, andesite, argillite, chert, greywacke, quartzite, conglomerate; minor schist, granulite, lime-silicate rock, scarn

TRIASSIC

UPPER TRIASSIC (mainly or entirely)

VANCOUVER GROUP

3	Andesite, basalt, quartzite, argillite, limestone, schist. May include some undifferentiated late Palaeozoic rocks

CRETACEOUS AND/OR EARLIER

UPPER CRETACEOUS AND/OR EARLIER

8	Quartz-feldspar porphyry

JERVIS GROUP

6	6a, basalt, andesite, associated pyroclastic rocks; minor limestone, dolomitic limestone, chert, argillite; 6b, mainly conglomerate, greywacke, sandstone, argillite; minor greenstone; 6c, metavolcanic rocks; metasedimentary rocks; metadiabase

5	Undivided volcanic and sedimentary rocks equivalent to 3 - 9: minor migmatite and gneiss; bedded rocks seen from the air

4	Undivided metamorphic rocks: 4a, gneiss, schist, hornfels, metavolcanic rocks; some quartzite, greywacke; minor slate, argillite; 4b, migmatite, agmatite 4c, migmatite, gneiss, and melanocratic dykes in plutonic rock; observed from the air

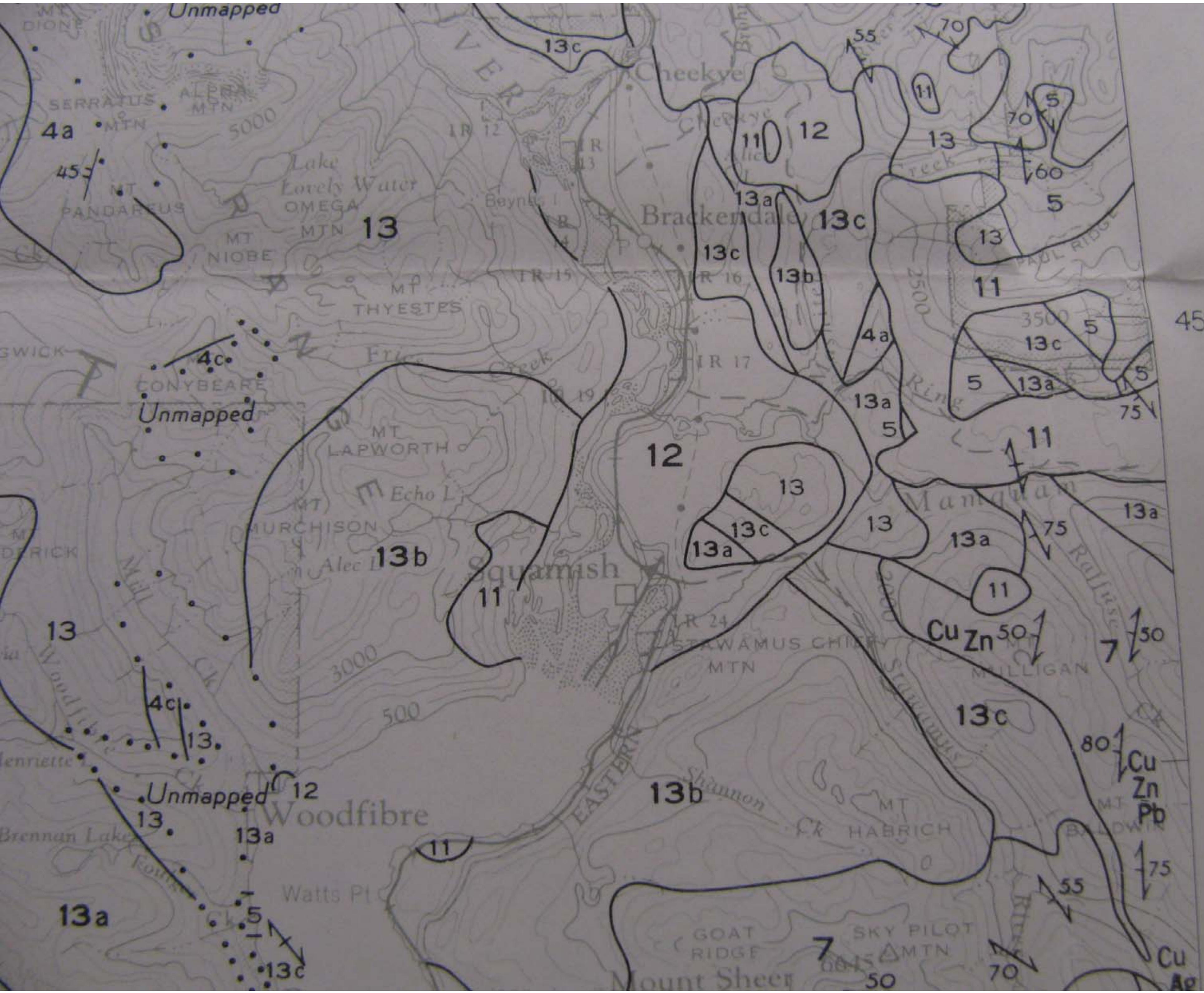
... AND MESOZOIC

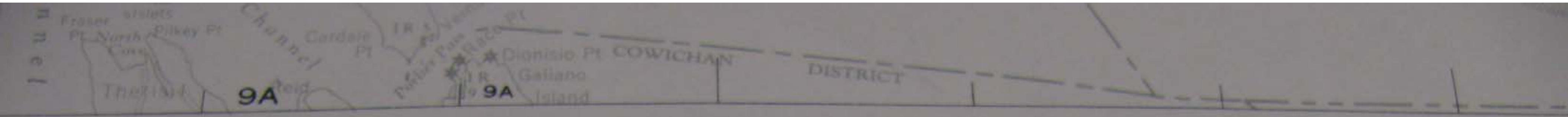
TRIASSIC OR EARLIER

BOWEN ISLAND GROUP

2	Mainly greenstone; minor chert and greywacke

PRELIMINARY SERIES


Unmapped
MT DIONE
SERRATUS MTN
ALPHA MTN
4a
45
MT PANDAREUS
Lake Lovely Water
OMEGA MTN
13
MT NIOBE
MT THYESTES
5000
IR 12
IR 13
IR 14
IR 15
IR 16
IR 17
IR 19
IR 24
Beynon
Cheekye
Brackendale
13c
11
12
13a
13b
4a
55
70
5
60
PAUL RIDGE
3500
2500
75
45′
4c
CONYBEARE
Unmapped
Fries Creek
MT LAPWORTH
Echo L
MT MURCHISON
Alec L
13b
MT DERRICK
Squamish
Mamquam
Ring Creek
STAWAMUS CHIEF MTN
Cu Zn
50
MT MILLIGAN
7
Ralfuse
3000
500
2000
Mill Ck
Woodfibre Ck
Henriette
Unmapped
Woodfibre
Brennan Lake
Watts Pt
EASTERN
Shannon Ck
MT HABRICH
80
Cu Zn Pb
MT BALDWIN
GOAT RIDGE
SKY PILOT MTN
Mount Sheer
Cu Ag



30′

15′

MAP 42-1963

GEOLOGY

SQUAMISH

(VANCOUVER, WEST HALF)

BRITISH COLUMBIA

Scale: One Inch to Four Miles = $\frac{1}{253,440}$

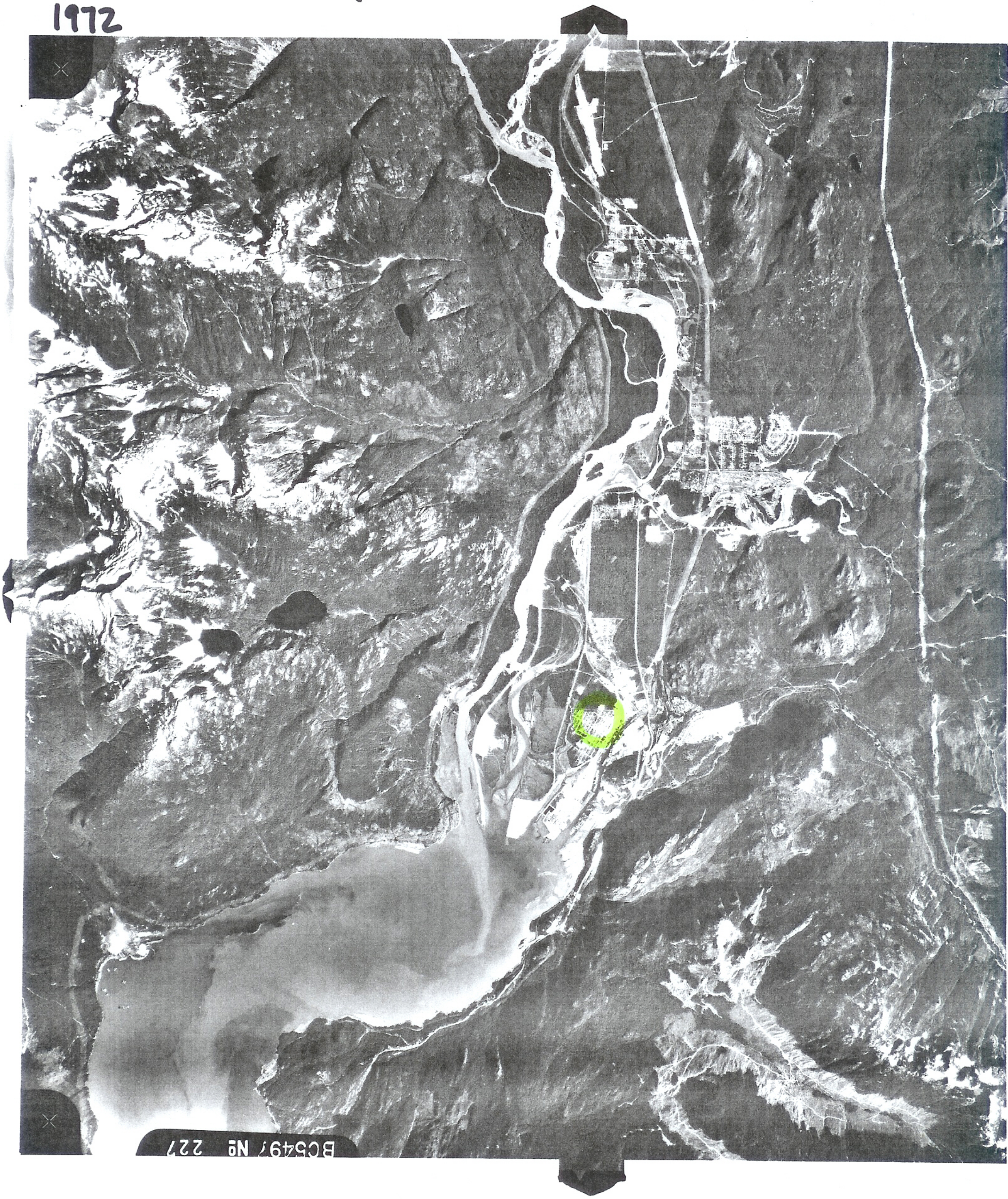
Miles

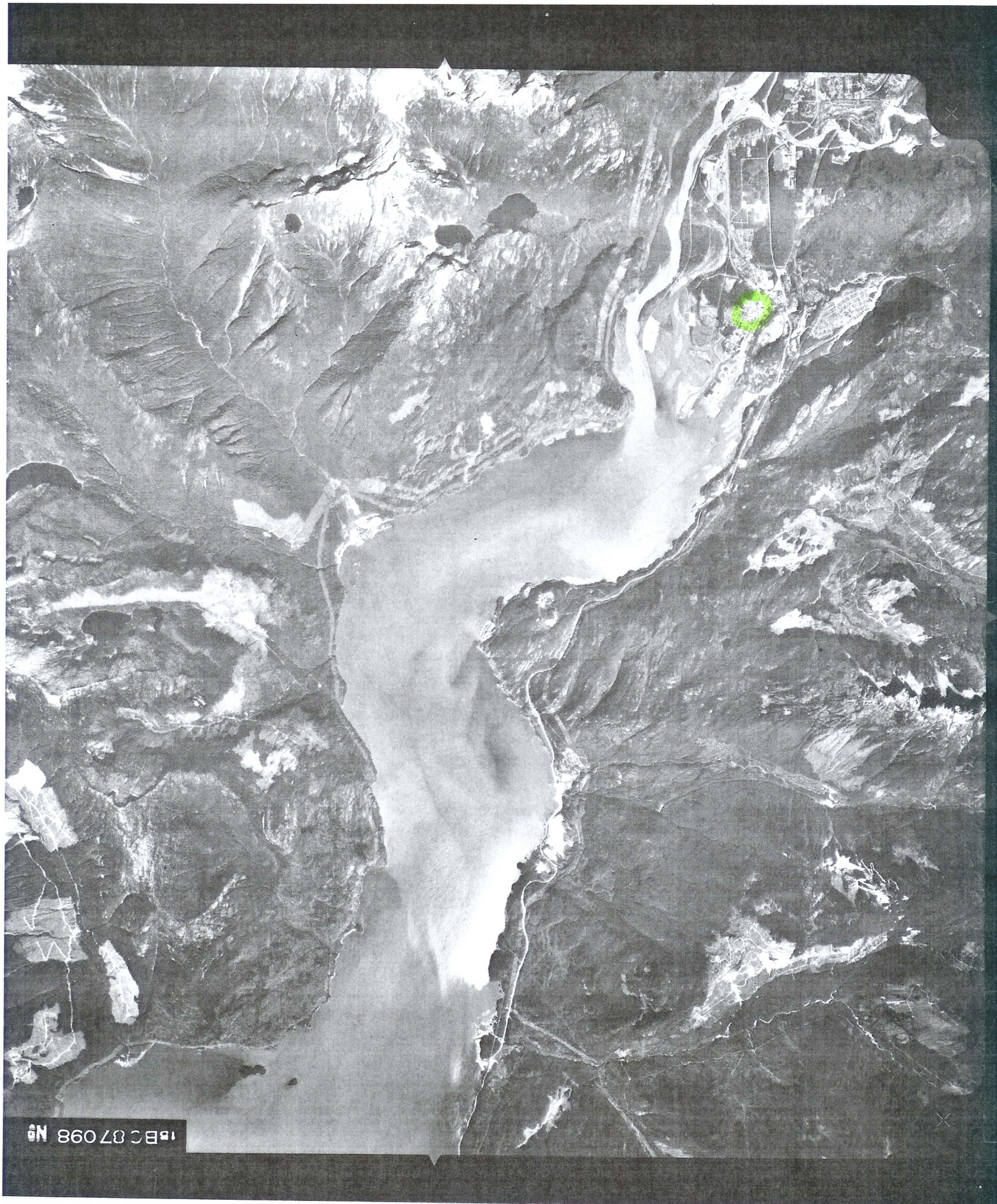
4 2 0 4 8 12

APPENDIX C: AERIAL PHOTOGRAPHS

N

1972

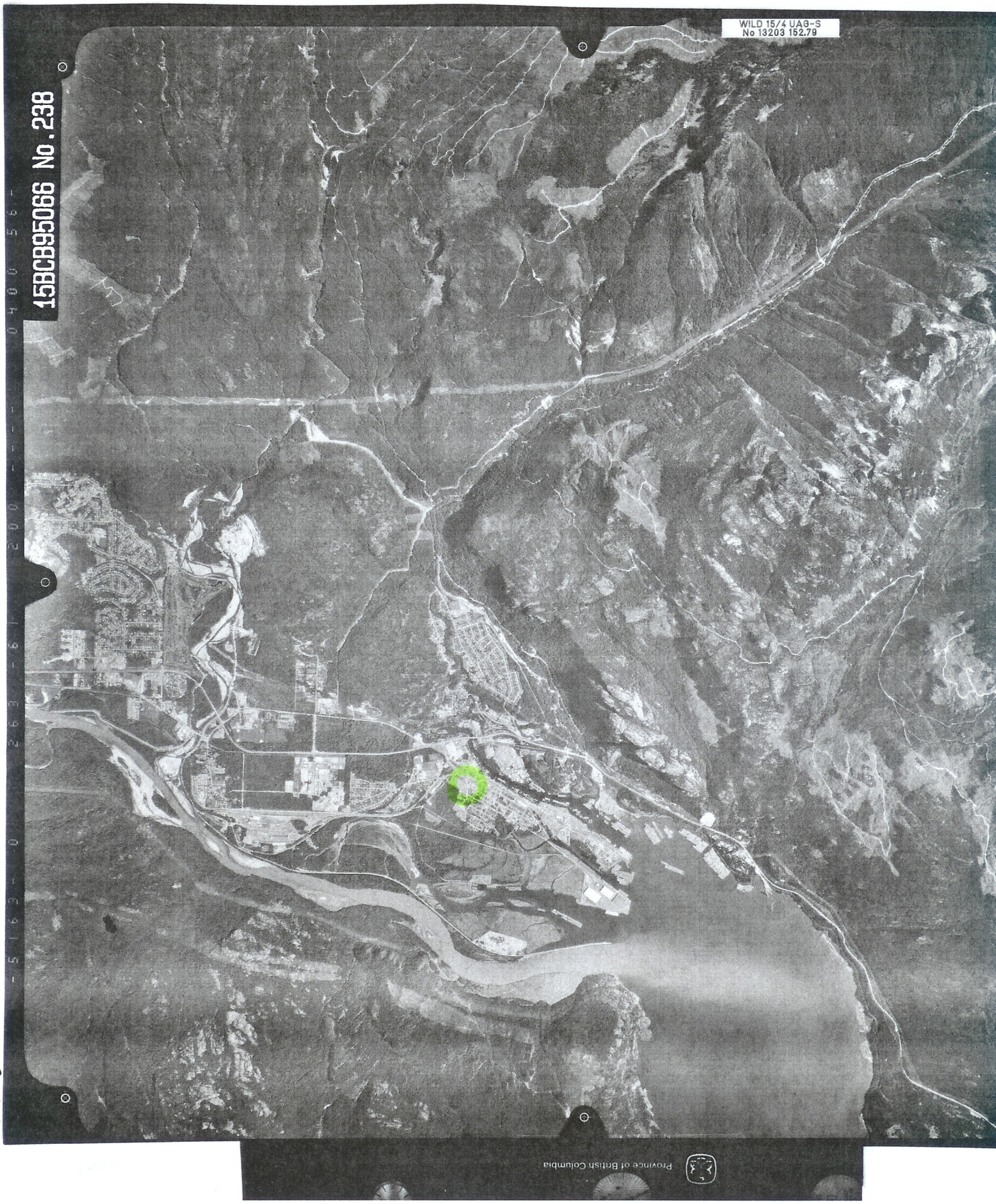




N

1987

N ←


15BCB95066 No. 238
WILD 15/4 UAG-S
No 13203 152.79
Province of British Columbia

1995

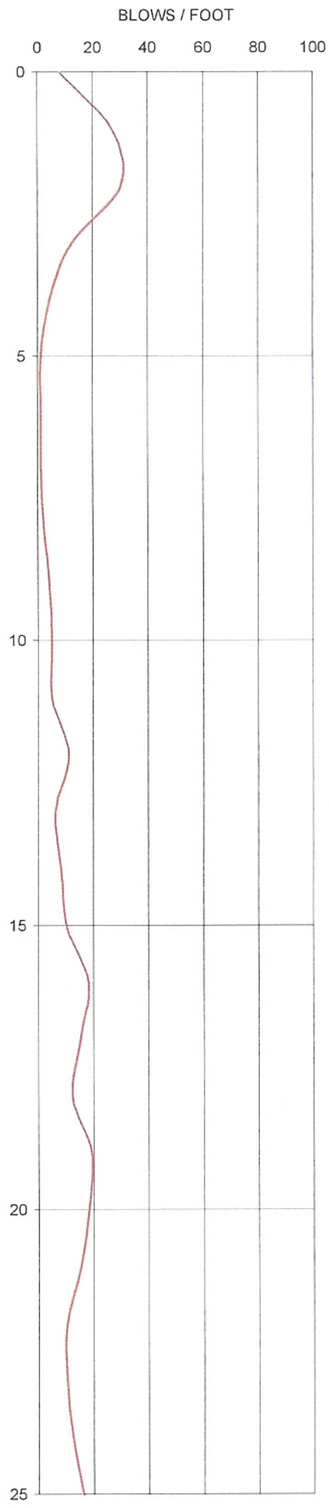
APPENDIX D: AUGER DRILL DATA

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A1
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 1.76 m	SHEET 1	OF 2

DEPTH (ft)	DESCRIPTION OF SOIL AND OBSERVATIONS		Soil Class. Symbol	SAMPLE: Sample Type	SAMPLE: Moisture Content	Soil Unit
0		2.5 inches Asphalt on 10" Sand and Gravel	SP/GP	X	5.7	
	FILL	- Tan brown, silty fine sand, 1" to 4" dia. gravel (dense)	SM/GM	X	4.5	
5	SILT	- Tan brown mottled, clayey, low-plasticity, some organics (very soft)	ML	X	54.5	
▽		- grades to grey, abundant organics, v. soft		X	62.2	
	SILT	- Grey, sandy, very fine-grained, occ. gravel, low plasticity, some organics (loose)	ML	X	43.5	
10				X	36.5	
		- grades with less organics, firm		X	34.6	
15						
	SAND	- Grey, fine to medium grained, clean (compact)	SP	X	26.0	
		- grades with occ. 1" gravel				
20						
		- grades with fine gravel		X	19.6	
25		continues on Figure 3A				

GRAB SAMPLE [X] WATER TABLE ▽

DYNAMIC CONE PENETRATION TEST

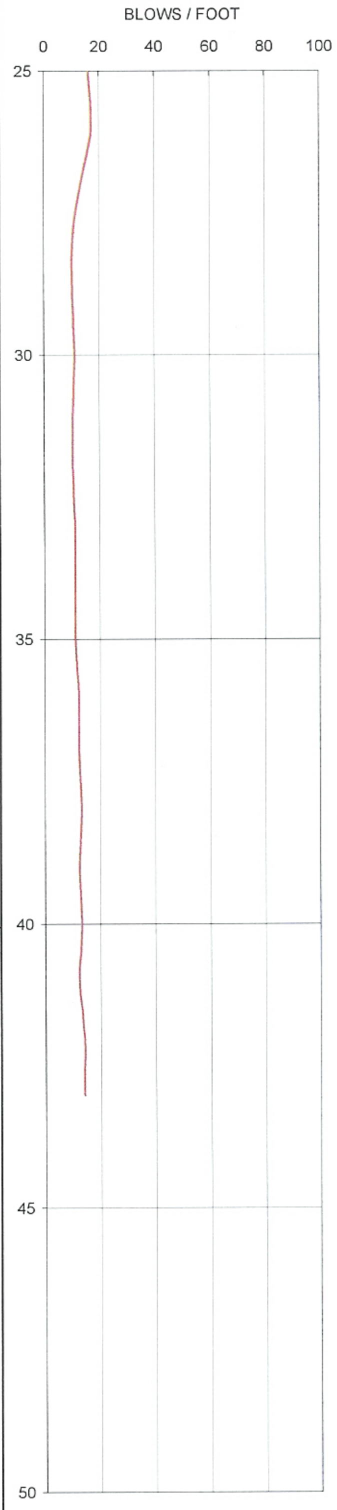


PROJECT No:	V07 - 106	Centennial Geotechnical Engineers Ltd. Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8		
PROJECT:	Proposed Shoppers Drug Mart	BOREHOLE LOG		
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC	DATE: 16-Feb-07	DRAWN BY: A.N.	FIGURE: 3

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A1
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 1.76 m	SHEET 2 OF 2	

DEPTH (ft)		DESCRIPTION OF SOIL AND OBSERVATIONS	Soil Class. Symbol	SAMPLE Sample Type	SAMPLE Moisture Content	Soil Unit
25		continues from Figure 3				
	SAND	- Grey, fine to medium grained, clean (compact)	SP			
		- grades with some 1/2" to 1" gravel		X	23.6	
30						
		- grades loose to compact		X	28.4	
35						
		- grades to medium to coarse-grained		X	23.2	
40						
		End of Test Borehole @ 40' End of DCPT @ 44'				
45						
50						

DYNAMIC CONE PENETRATION TEST

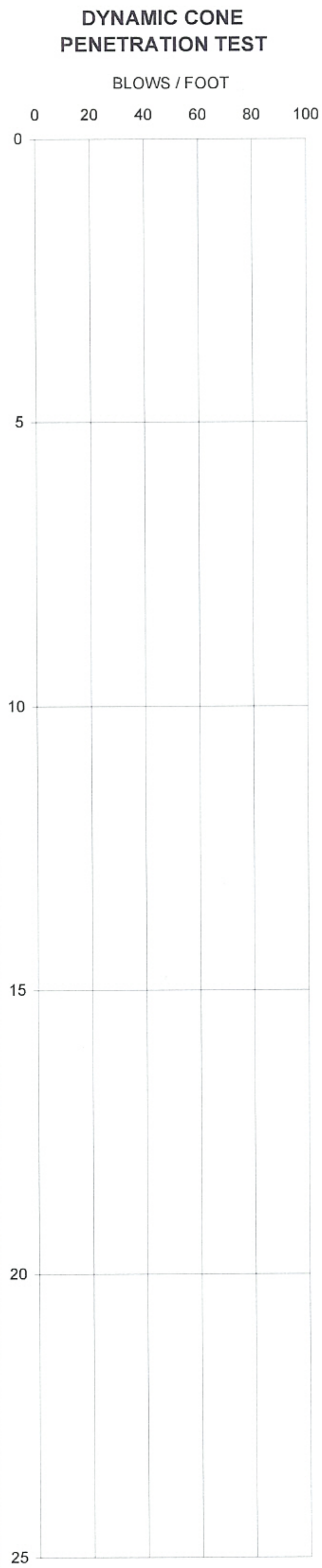


GRAB SAMPLE X   WATER TABLE

PROJECT No:	V07 - 106	Centennial Geotechnical Engineers Ltd. Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8		
PROJECT:	Proposed Shoppers Drug Mart	BOREHOLE LOG		
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC	DATE: 16-Feb-07	DRAWN BY: A.N.	FIGURE: 3A

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A2
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 1.83 m	SHEET 1 OF 2	

DEPTH (ft)		DESCRIPTION OF SOIL AND OBSERVATIONS	Soil Class. Symbol	SAMPLE Sample Type	SAMPLE Moisture Content	Soil Unit
0		2.5 inches Asphalt on 4" Sand and Gravel	SP/GP			
	FILL	- Tan brown, fine to medium-grained silty sand, 1" to 3" dia. gravel (dense)	SM/GM	X	5.2	
				X	10.5	
5	SILT	- Tannish brown mottled, clayey, low-plasticity, some organic and wood pieces (soft)	ML	X	55.0	
▽		- grades with less organics		X	49.4	
	SILT	- Grey, sandy, low plasticity, some organics (loose)	ML	X	56.9	
10						
		- grades to soft		X	51.9	
15	SAND	- Grey, fine to medium grained, clean (compact)	SP	X	21.0	
				X	18.0	
20						
		- grades coarser at 23'		X	12.9	
25		(continued on Figure 4A)				



GRAB SAMPLE [X] WATER TABLE ▽

PROJECT No:	V07 - 106
PROJECT:	Proposed Shoppers Drug Mart
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC

Centennial Geotechnical Engineers Ltd.
Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8

BOREHOLE LOG

DATE:	DRAWN BY:	FIGURE:
16-Feb-07	A.N.	4

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A2
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 1.83 m	SHEET 2	OF 2

DEPTH (ft)		DESCRIPTION OF SOIL AND OBSERVATIONS	Soil Class. Symbol	Sample Type	Moisture Content	Soil Unit
25		(continued from Figure 4)				
	SAND	- Grey, fine to medium grained, clean (compact)	SP			
		- grades finer at 28'		X	27.8	
30				X	28.5	
	SILT	- Tan grey, sandy, low plasticity (loose)	ML	X	40.7	
				X	47.8	
35						
		- grades with sand lenses		X	29.7	
40				X	38.0	
		End of borehole at 40'				
45						
50						

DYNAMIC CONE PENETRATION TEST

BLOWS / FOOT

0 20 40 60 80 100

25 30 35 40 45 50

GRAB SAMPLE [X] WATER TABLE

PROJECT No: V07 - 106

PROJECT: Proposed Shoppers Drug Mart

LOCATION: Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC

Centennial Geotechnical Engineers Ltd.
Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8

BOREHOLE LOG

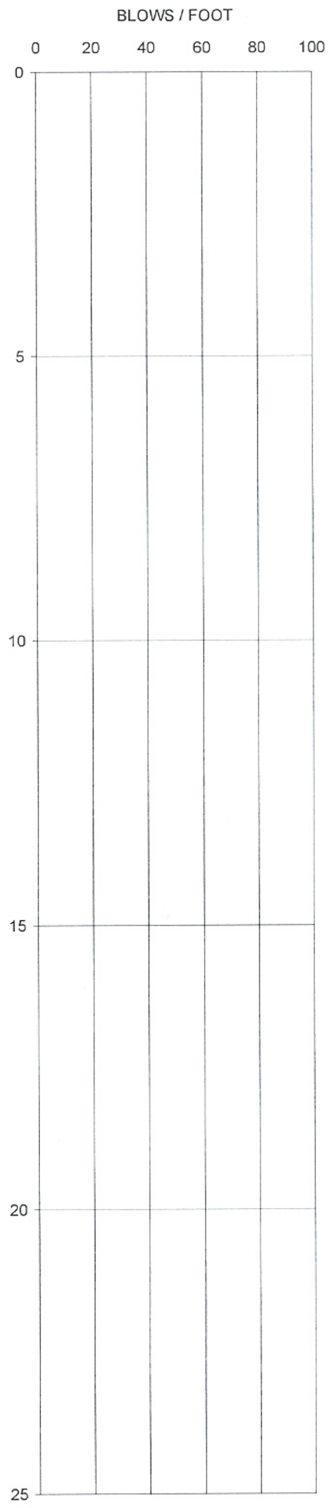
DATE: 16-Feb-07 | DRAWN BY: A.N. | FIGURE: 4A

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 2.58 m

AUGER HOLE A3
SHEET 1 OF 1

DEPTH (ft)	DESCRIPTION OF SOIL AND OBSERVATIONS		Soil Class. Symbol	SAMPLE Sample Type	SAMPLE Moisture Content	Soil Unit
0		2.5 inches Asphalt on 4" Sand and Gravel	SP/GP	X	5.0	
	FILL	- Tan brown, fine to medium-grained silty sand, 1" to 3" dia. gravel (dense)	SM/GM	X	6.8	
				X	7.4	
	SILT	- Brownish grey mottled, clayey, low-plasticity, some organics (soft)	ML	X	33.3	
5						
		- grades to moist, more organic		X	57.9	
	SILT	- Grey, sandy, low-plasticity, some organics (loose)	ML	X	43.0	
10						
		- grades with occ. gravel		X	46.0	
15						
	SAND	- Grey, fine to medium grained, clean (compact)	SP	X	26.5	
20		End of Test Borehole @ 20'				
25						

DYNAMIC CONE PENETRATION TEST



GRAB SAMPLE [X] WATER TABLE

PROJECT No:	V07 - 106
PROJECT:	Proposed Shoppers Drug Mart
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC

Centennial Geotechnical Engineers Ltd.
Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8

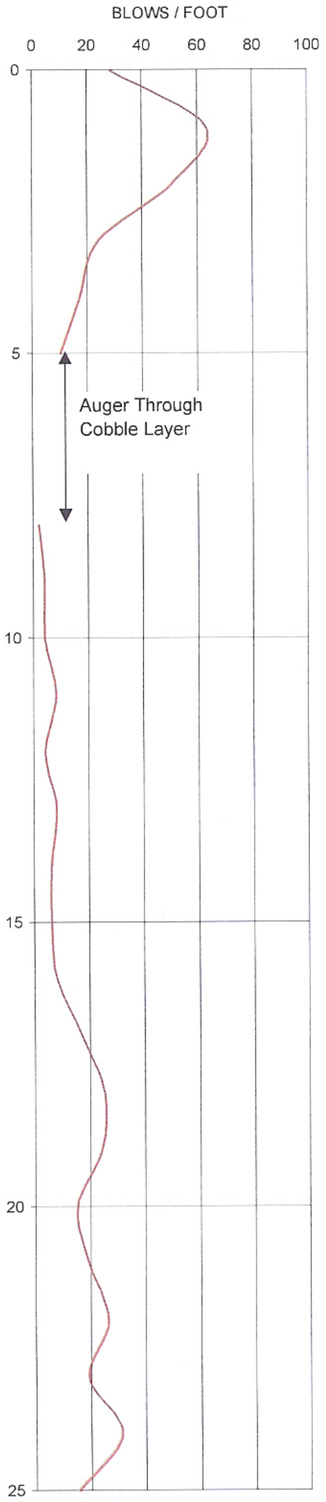
BOREHOLE LOG

DATE:	DRAWN BY:	FIGURE:
16-Feb-07	A.N.	5

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A4
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 2.37 m	SHEET 1	OF 2

DEPTH (ft)	DESCRIPTION OF SOIL AND OBSERVATIONS	Soil Class. Symbol	SAMPLE Sample Type	SAMPLE Moisture Content	Soil Unit
0	2.5" Ashpalt on 4" Sand and Gravel	SP/GP			
	FILL - Tan brown, silty fine sand, 1" to 3" dia. gravel (dense)	SM/GM	X	6.1	
5	- grades with some 5" to 8" dia. cobbles				
	SILT - Grey, clayey, low-plasticity, some organics (v. soft)	ML	X	46.7	
10					
	SILT - Grey, sandy, very fine-grained, low plasticity, occ gravel and trace organics (loose)	ML	X	34.3	
15					
	SAND - Grey, fine to medium grained, clean (compact)	SP	X	17.7	
20					
	- grades with occ. 1" to 2" gravel		X	17.8	
25	continues on Figure 6A				

DYNAMIC CONE PENETRATION TEST



GRAB SAMPLE [X] WATER TABLE ▽

PROJECT No:	V07 - 106
PROJECT:	Proposed Shoppers Drug Mart
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC

Centennial Geotechnical Engineers Ltd.
Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8

BOREHOLE LOG

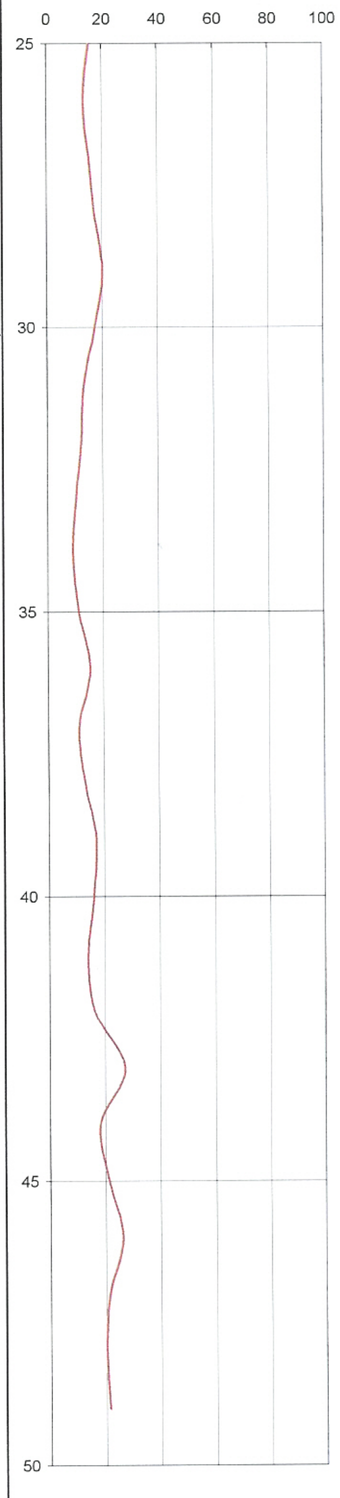
DATE:	DRAWN BY:	FIGURE:
16-Feb-07	A.N.	6

DATE DRILLED:	12-Feb-07	INSPECTOR:	A.N.	AUGER HOLE	A4
DRILL METHOD:	AUGER	SURFACE ELEVATION:	± 2.37 m	SHEET 2 OF 2	

DEPTH (ft)	DESCRIPTION OF SOIL AND OBSERVATIONS	Soil Class. Symbol	SAMPLE Sample Type	SAMPLE Moisture Content	Soil Unit
25	continues from Figure 6				
	SAND - Grey, fine to medium grained, clean (compact)	SP			
	- grades with occ. 1" to 2" gravel		X	15.3	
30					
	End of Test Borehole @ 30' End of DCPT @ 50'				
35					
40					
45					
50					

GRAB SAMPLE X WATER TABLE

DYNAMIC CONE PENETRATION TEST



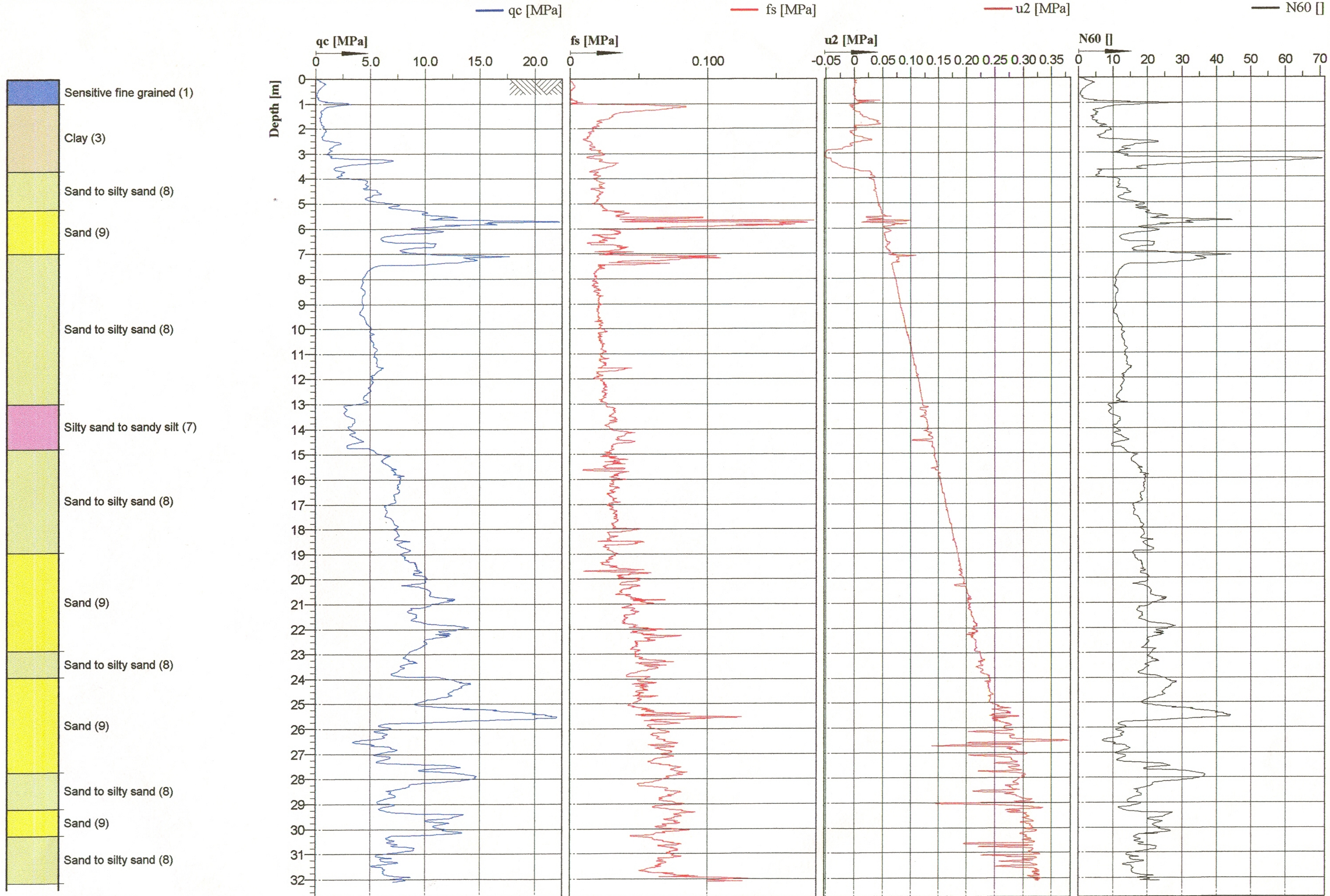
PROJECT No:	V07 - 106
PROJECT:	Proposed Shoppers Drug Mart
LOCATION:	Chieftain Shopping Centre, 1335 Pemberton Avenue, Squamish, BC

Centennial Geotechnical Engineers Ltd.
Suite 106, 2780 E. Broadway, Vancouver, B.C. V5M 1Y8

BOREHOLE LOG

DATE:	DRAWN BY:	FIGURE:
16-Feb-07	A.N.	6A

APPENDIX E: CPT DATA

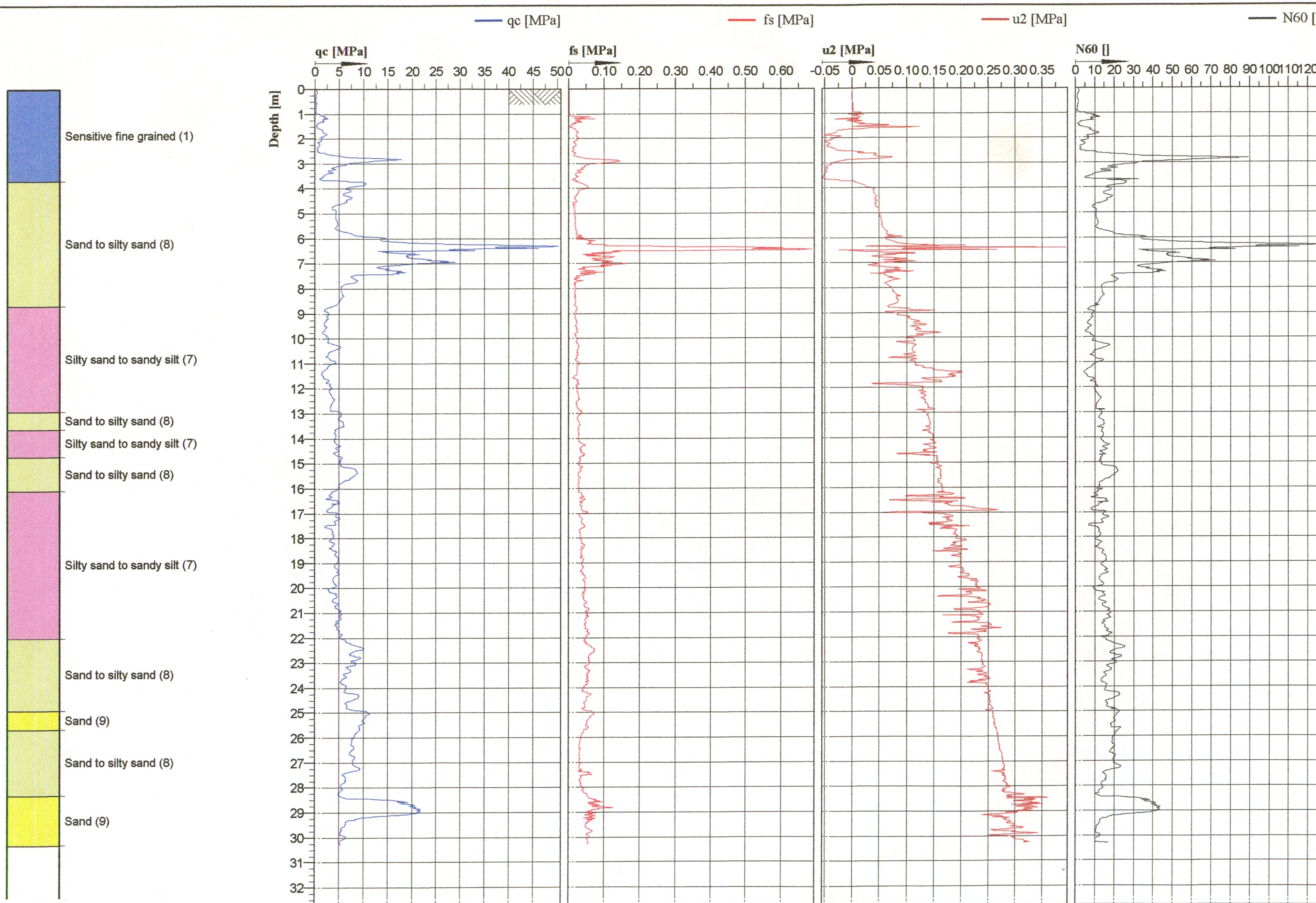


Since 1988 DOWNRITE DRILLING Ltd. 604-823-4086

U2

Cone No: 0
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Squamish	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT#1
Project ID: V07-106	Client: Centenial	Date: 3/7/2007	Scale: 1 : 210
Project: Chieften Mall		Page: 1/1	Fig:
		File: CM#1.CPT	



U2

Cone No: 0
Tip area [cm2]: 10
Sleeve area [cm2]: 150

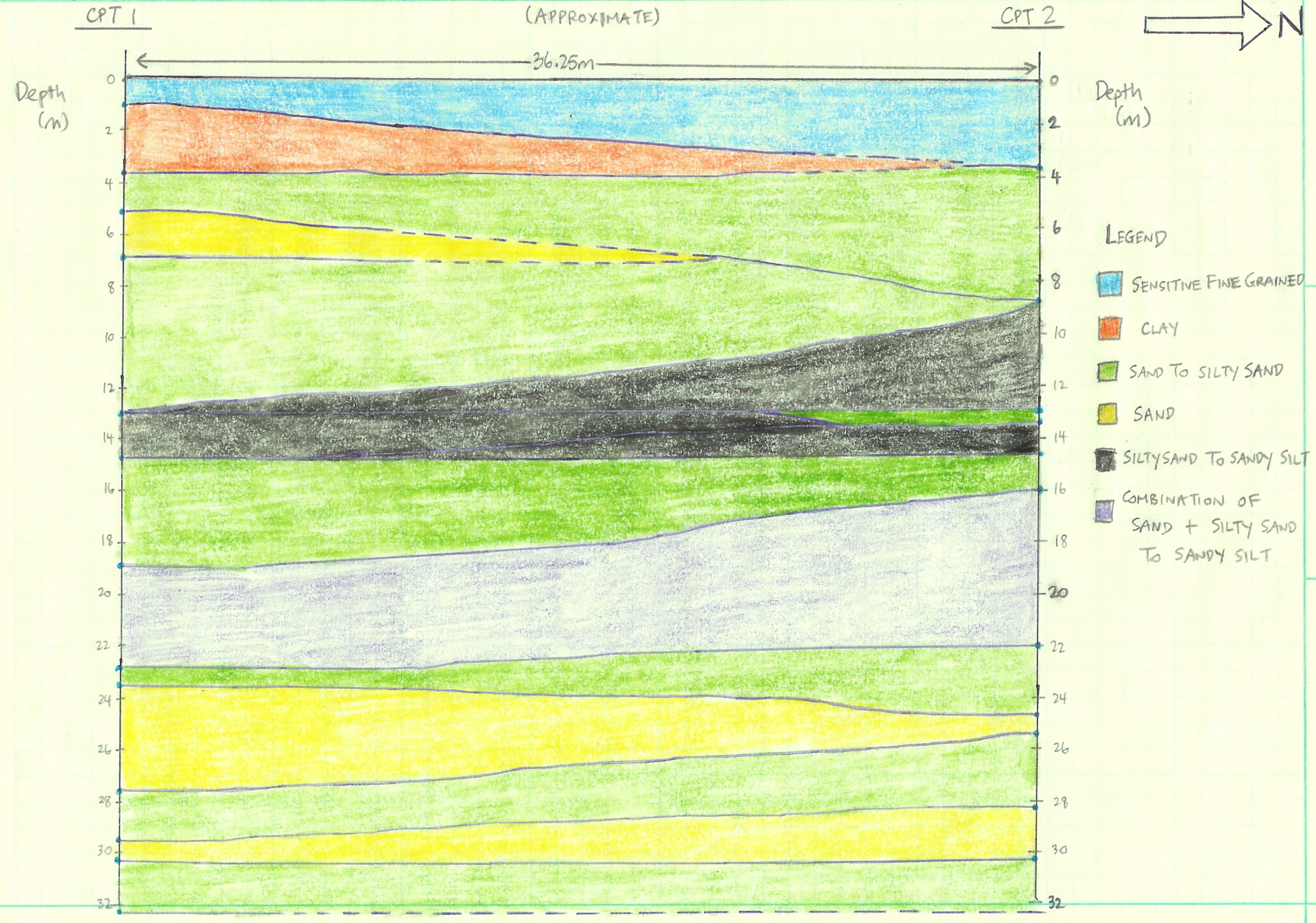
Location:	Chieften Mall Squamish	Position:	X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT#2
Project ID:	V07-106	Client:	Centenial	Date: 3/9/2007	Scale: 1 : 210
Project:				Page: 1/1	Fig:
				File: CM#2.cpd	

APPENDIX F: CROSS SECTION

SCALE 1:200

CROSS SECTION STRATIGRAPHY OF BOREHOLES CPT #1 AND CPT#2

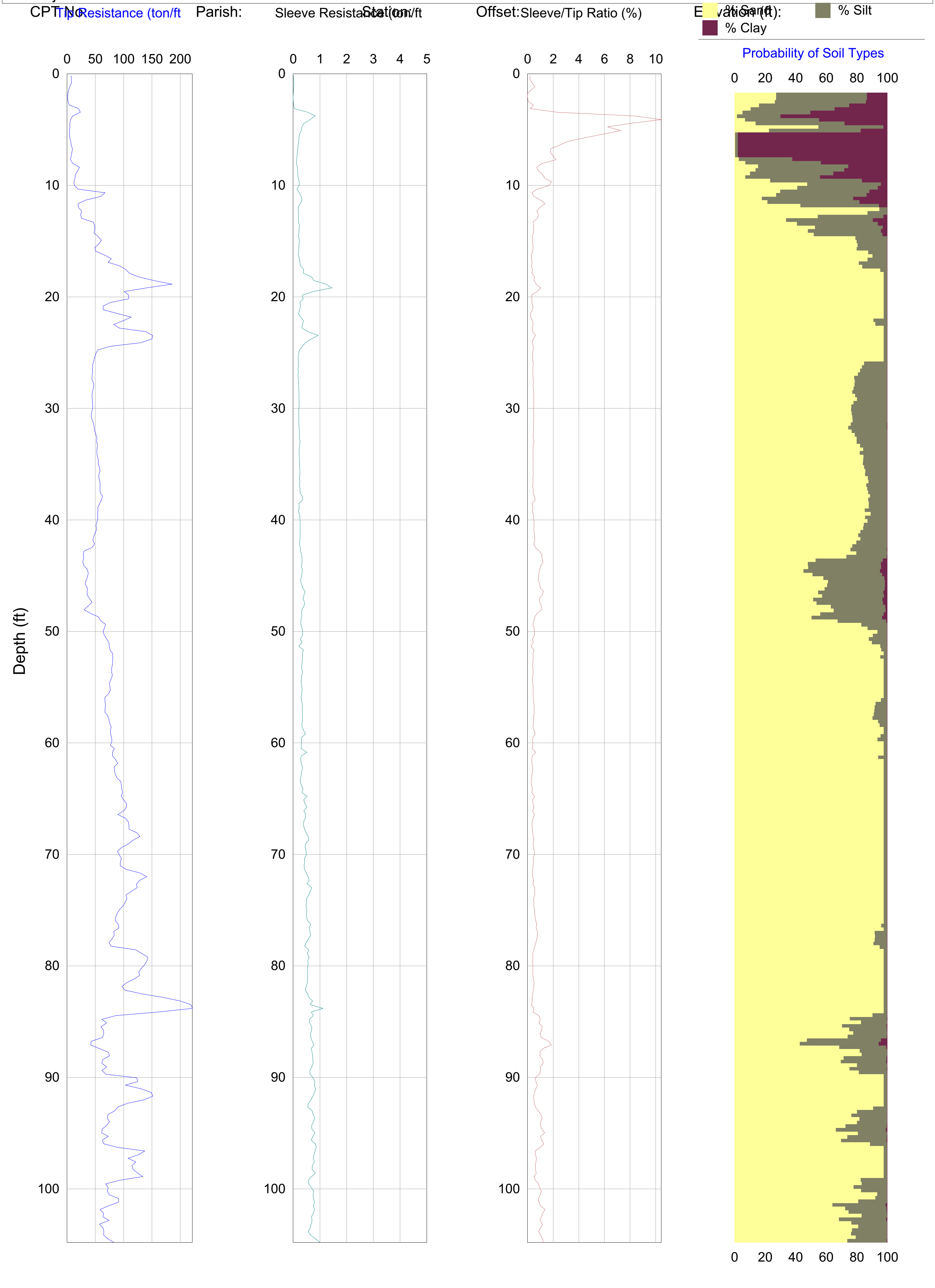
(APPROXIMATE)



APPENDIX G: RESULTS AND CALCULATIONS

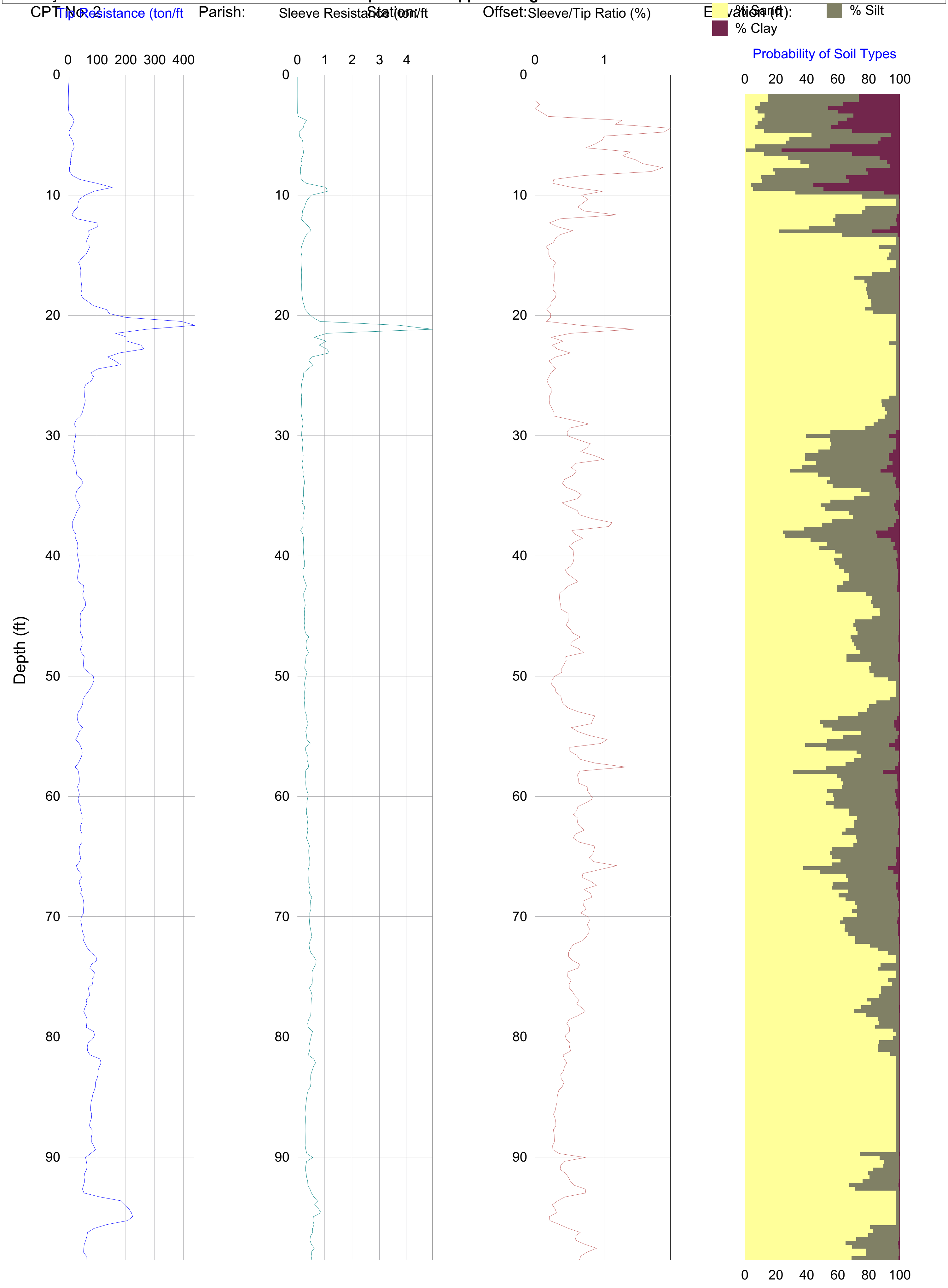
Project Number: **Title:** Date:

CPT No: Parish: Station: Offset: Elevation (ft):



Project Number: **Title: Squamish Shoppers Drug Mart** Date:

CPT No. 2 Parish: Station: Offset: Elevation (ft):



CPT #1

Depth (m)		Layer	Unit Weight (kN/m³)	Thickness (m)	σ_{z0} (kPa)	σ'_{z0} (kPa)
From	To					
0.00	1.00	Sensitive Fine Grained	17.5	1.00	8.75	7.00
1.00	3.75	Clay	18.0	2.75	42.25	49.68
3.75	5.25	Sand to Silty Sand	19.0	1.50	81.25	56.75
5.25	7.00	Sand	19.5	1.75	112.56	72.14
7.00	13.00	Sand to Silty Sand	19.0	6.00	186.63	108.23
13.00	14.75	Silty Sand to Sandy Silt	18.5	1.75	259.81	143.44
14.75	19.00	Sand to Silty Sand	19.0	4.25	316.38	170.60
19.00	23.00	Sand	19.5	4.00	395.75	209.55
23.00	23.75	Sand to Silty Sand	19.0	0.75	441.88	232.40
23.75	27.75	Sand	19.5	4.00	488.00	255.25
27.75	29.25	Sand to Silty Sand	19.0	1.50	541.25	281.55
29.25	30.25	Sand	19.5	1.00	565.25	293.30
30.25	32.25	Sand to Silty Sand	19.0	2.00	594.00	307.35

*** GWT at 1.83-2.44m* Chosen = 2.0m

Liquefaction Assessment

Layer Thickness (m)	Depth at Middle (m)	σ_{z0} (kPa)	σ'_{z0} (kPa)	r_d	$(CSR)_{eqk}$	q_c (kg/cm²)	q_{c1N} (kg/cm²)	Potential for Liquefaction
1.00	0.50	8.75	7.00	0.996	0.162	1.78	6.61	Yes
2.75	2.38	42.25	49.68	0.982	0.109	6.09	8.47	Yes
1.50	4.50	81.25	56.75	0.966	0.180	57.07	74.23	Yes
1.75	6.13	112.56	72.14	0.953	0.193	113.26	130.65	No
6.00	10.00	186.63	108.23	0.907	0.203	51.13	48.15	Yes
1.75	13.88	259.81	143.44	0.804	0.189	30.47	24.93	Yes
4.25	16.88	316.38	170.60	0.723	0.174	75.06	56.30	Yes
4.00	21.00	395.75	209.55	0.613	0.151	107.02	72.43	Yes
0.75	23.38	441.88	232.40	0.550	0.136	85.02	54.64	Yes
4.00	25.75	488.00	255.25	0.486	0.121	224.73	137.82	No
1.50	28.50	541.25	281.55	0.413	0.103	70.16	40.97	Yes
1.00	29.75	565.25	293.30	0.380	0.095	136.14	77.89	No
2.00	31.25	594.00	307.35	0.340	0.085	60.35	33.73	Yes

Friction Angle

Depth	Soil Layer	q_c (MPa)	q_c (kg/cm²)	σ'_{z0} (kPa)	Friction Angle (φ)
0.50	Sensitive Fine Grained	0.175	1.78	7.00	30
2.38	Clay	0.598	6.09	49.68	30
4.50	Sand to Silty Sand	5.597	57.07	56.75	42
6.13	Sand	11.107	113.26	72.14	44
10.00	Sand to Silty Sand	5.014	51.13	108.23	37
13.88	Silty Sand to Sandy Silt	2.988	30.47	143.44	32
16.88	Sand to Silty Sand	7.361	75.06	170.60	36
21.00	Sand	10.495	107.02	209.55	37
23.38	Sand to Silty Sand	8.338	85.02	232.40	35
25.75	Sand	22.039	224.73	255.25	40
28.50	Sand to Silty Sand	6.880	70.16	281.55	32
29.75	Sand	13.351	136.14	293.30	36
31.25	Sand to Silty Sand	5.918	60.35	307.35	30

CPT #2

Depth (m)		Layer	Unit Weight	Thickness	σ_{z0}	σ'_{z0}
From	To		(kN/m³)	(m)	(kPa)	(kPa)
0.00	2.00	Sensitive Fine Grained (Dry)	17.5	2.00	17.50	17.50
2.00	3.75	Sensitive Fine Grained (Wet)	18.5	1.75	51.19	42.60
3.75	8.75	Sand to Silty Sand	19.5	5.00	116.13	91.63
8.75	13.00	Silty Sand to Sandy Silt	18.5	4.25	204.19	117.21
13.00	13.50	Sand to Silty Sand	19.0	0.50	248.25	138.00
13.50	14.75	Silty Sand to Sandy Silt	18.5	1.25	264.56	145.74
14.75	16.00	Sand to Silty Sand	19.0	1.25	288.00	156.93
16.00	22.00	Silty Sand to Sandy Silt	18.5	6.00	355.38	188.78
22.00	24.75	Sand to Silty Sand	19.0	2.75	437.00	227.53
24.75	25.50	Sand	19.5	0.75	470.44	243.81
25.50	28.25	Sand to Silty Sand	19.0	2.75	503.88	260.10
28.25	30.25	Sand	19.0	2.00	549.00	281.95

*** GWT at 1.83-2.44m* Chosen = 2.0m

Liquefaction Assessment

Layer Thickness (m)	Depth at Middle (m)	σ_{z0} (kPa)	σ'_{z0} (kPa)	r_d	$(CSR)_{eqk}$	q_c (kg/cm²)	q_{c1N} (kg/cm²)	Potential for Liquefaction
2.00	1.00	17.50	17.50	0.992	0.129	10.40	24.36	Yes
1.75	2.88	51.19	42.60	0.978	0.153	128.86	193.43	No
5.00	6.25	116.13	91.63	0.952	0.157	405.76	415.33	No
4.25	10.88	204.19	117.21	0.884	0.200	39.39	35.64	Yes
0.50	13.25	248.25	138.00	0.820	0.192	51.13	42.64	Yes
1.25	14.13	264.56	145.74	0.797	0.188	41.61	33.77	Yes
1.25	15.38	288.00	156.93	0.763	0.182	87.99	68.82	Yes
6.00	19.00	355.38	188.78	0.667	0.163	44.59	31.80	Yes
2.75	23.38	437.00	227.53	0.550	0.137	69.26	44.99	Yes
0.75	25.13	470.44	243.81	0.503	0.126	105.23	66.03	Yes
2.75	26.88	503.88	260.10	0.456	0.115	80.11	48.67	Yes
2.00	29.25	549.00	281.95	0.393	0.099	84.72	49.43	Yes

** Assume Peak Ground Acceleration (g) = 0.2g

Friction Angle

Depth	Soil Layer	q_c (MPa)	q_c (kg/cm²)	σ'_{z0} (kPa)	Friction Angle (φ)
1.00	Sensitive Fine Grained (Dry)	1.020	10.40	17.50	32
2.88	Sensitive Fine Grained (Wet)	12.637	128.86	42.60	46
6.25	Sand to Silty Sand	39.792	405.76	91.63	48
10.88	Silty Sand to Sandy Silt	3.863	39.39	117.21	35
13.25	Sand to Silty Sand	5.014	51.13	138.00	36
14.13	Silty Sand to Sandy Silt	4.081	41.61	145.74	34
15.38	Sand to Silty Sand	8.629	87.99	156.93	38
19.00	Silty Sand to Sandy Silt	4.373	44.59	188.78	34
23.38	Sand to Silty Sand	6.793	69.26	227.53	34
25.13	Sand	10.320	105.23	243.81	36
26.88	Sand to Silty Sand	7.857	80.11	260.10	34
29.25	Sand	8.308	84.72	281.95	34

Liquefaction Assessment using SPT data

Depth (m)	Layer	σ'_{z0} (kPa)	C_N	N_{60}	$ER_r/60$	$(N_1)_{60}$	r_d	$(CSR)_{eqk}$	Potential for Liquefaction
1	Sensitive Fine Grained	17.50	1.57	2.57	1.00	4.04	0.992	0.13	YES
2	Clay	35.50	1.33	7.39	1.00	9.87	0.985	0.13	YES
3	Clay	43.69	1.27	11.76	1.00	14.87	0.977	0.16	YES
4	Clay + Sand to Silty Sand	52.13	1.21	23.37	1.00	28.18	0.969	0.17	NO
5	Sand to Silty Sand	61.32	1.15	12.66	1.00	14.58	0.962	0.19	YES
6	Sand to Silty Sand + Sand	70.89	1.10	23.06	1.00	25.44	0.954	0.19	NO
7	Sand	80.58	1.06	17.43	1.00	18.48	0.946	0.20	YES
8	Sand to Silty Sand	89.77	1.02	21.37	1.00	21.89	0.939	0.20	NO
9	Sand to Silty Sand	98.96	0.99	10.80	1.00	10.71	0.931	0.21	YES
10	Sand to Silty Sand	108.15	0.96	11.18	1.00	10.76	0.96	0.22	YES
11	Sand to Silty Sand	117.34	0.93	13.09	1.00	12.24	0.93	0.21	YES
12	Sand to Silty Sand	126.53	0.91	13.87	1.00	12.61	0.91	0.21	YES
13	Sand to Silty Sand	135.72	0.89	12.13	1.00	10.75	0.88	0.21	YES
14	Silty Sand to Sandy Silt	144.41	0.87	10.23	1.00	8.85	0.85	0.20	YES
15	Silty Sand to Sandy Silt + Sand to Silty Sand	153.22	0.85	12.11	1.00	10.24	0.83	0.20	YES
16	Sand to Silty Sand	162.41	0.83	17.24	1.00	14.24	0.80	0.19	YES
17	Sand to Silty Sand	171.60	0.81	18.58	1.00	15.01	0.77	0.19	YES
18	Sand to Silty Sand	180.79	0.79	17.02	1.00	13.45	0.75	0.18	NO
19	Sand to Silty Sand	189.98	0.77	19.09	1.00	14.77	0.72	0.18	NO
20	Sand	199.67	0.76	18.08	1.00	13.68	0.69	0.17	YES
21	Sand	209.36	0.74	20.94	1.00	15.52	0.67	0.16	YES
22	Sand	219.05	0.73	22.03	1.00	15.99	0.64	0.16	YES
23	Sand	228.74	0.71	21.48	1.00	15.28	0.61	0.15	NO
24	Sand to Silty Sand + Sand	238.06	0.70	20.30	1.00	14.17	0.59	0.15	NO
25	Sand	247.75	0.68	24.14	1.00	16.53	0.56	0.14	NO
26	Sand	257.44	0.67	27.23	1.00	18.30	0.53	0.13	NO
27	Sand	267.13	0.66	11.58	1.00	7.64	0.51	0.13	YES
28	Sand + Sand to Silty Sand	276.69	0.65	22.12	1.00	14.33	0.48	0.12	NO
29	Sand to Silty Sand	285.88	0.64	19.84	1.00	12.63	0.45	0.11	NO
30	Sand to Silty Sand + Sand	295.45	0.63	19.93	1.00	12.48	0.43	0.11	NO
31	Sand + Sand to Silty Sand	304.76	0.62	19.94	1.00	12.27	0.40	0.10	NO
32	Sand to Silty Sand	313.95	0.61	15.93	1.00	9.65	0.37	0.09	YES

Liquefaction Assessment using SPT data

Depth (m)	Layer	σ'_{z0} (kPa)	C_N	N_{60}	$ER_r/60$	$(N_1)_{60}$	r_d	$(CSR)_{eqk}$	Potential for Liquefaction
1.0	Sensitive Fine Grained (Dry)	17.50	1.57	1.18	1.00	1.86	0.992	0.13	YES
2.0	Sensitive Fine Grained (Dry)	35.00	1.34	6.67	1.00	8.93	0.985	0.13	NO
3.0	Sensitive Fine Grained (Wet	43.69	1.27	20.91	1.00	26.46	0.977	0.16	NO
4.0	Sensitive F.G. (Wet) + Sand to Silty Sand	52.63	1.20	17.47	1.00	21.01	0.969	0.17	NO
5.0	Sand to Silty Sand	62.32	1.15	13.92	1.00	15.96	0.962	0.18	NO
6.0	Sand to Silty Sand	72.01	1.10	14.72	1.00	16.16	0.954	0.19	NO
7.0	Sand to Silty Sand	81.70	1.06	60.72	1.00	64.10	0.946	0.20	NO
8.0	Sand to Silty Sand	91.39	1.02	27.05	1.00	27.54	0.939	0.20	NO
9.0	Sand to Silty Sand + Silty Sand to Sandy Silt	100.83	0.99	11.56	1.00	11.39	0.931	0.20	YES
10.0	Silty Sand to Sandy Silt	109.52	0.96	7.33	1.00	7.02	0.96	0.21	YES
11.0	Silty Sand to Sandy Silt	118.21	0.93	11.49	1.00	10.71	0.93	0.21	YES
12.0	Silty Sand to Sandy Silt	126.90	0.91	7.91	1.00	7.18	0.91	0.21	YES
13.0	Silty Sand to Sandy Silt	135.59	0.89	11.37	1.00	10.08	0.88	0.21	YES
14.0	Sand to Silty Sand + Silty Sand to Sandy Silt	144.53	0.86	13.53	1.00	11.70	0.85	0.20	YES
15.0	Silty Sand to Sandy Silt + Sand to Silty Sand	153.35	0.85	14.09	1.00	11.90	0.83	0.20	YES
16.0	Sand to Silty Sand	162.54	0.83	16.66	1.00	13.75	0.80	0.19	YES
17.0	Silty Sand to Sandy Silt	171.23	0.81	11.64	1.00	9.41	0.77	0.19	YES
18.0	Silty Sand to Sandy Silt	179.92	0.79	12.70	1.00	10.06	0.75	0.18	YES
19.0	Silty Sand to Sandy Silt	188.61	0.78	13.27	1.00	10.30	0.72	0.18	YES
20.0	Silty Sand to Sandy Silt	197.30	0.76	13.86	1.00	10.55	0.69	0.17	YES
21.0	Silty Sand to Sandy Silt	205.99	0.75	13.72	1.00	10.24	0.67	0.17	YES
22.0	Silty Sand to Sandy Silt	214.68	0.73	16.19	1.00	11.86	0.64	0.16	YES
23.0	Sand to Silty Sand	223.87	0.72	19.77	1.00	14.21	0.61	0.15	NO
24.0	Sand to Silty Sand	233.06	0.71	16.09	1.00	11.35	0.59	0.15	NO
25.0	Sand to Silty Sand + Sand	242.37	0.69	18.17	1.00	12.58	0.56	0.14	YES
26.0	Sand + Sand to Silty Sand	251.81	0.68	20.27	1.00	13.77	0.53	0.13	YES
27.0	Sand to Silty Sand	261.00	0.67	19.08	1.00	12.74	0.51	0.13	YES
28.0	Sand to Silty Sand	270.19	0.66	17.02	1.00	11.16	0.48	0.12	NO
29.0	Sand to Silty Sand + Sand	279.38	0.64	26.90	1.00	17.34	0.45	0.11	NO
30.0	Sand	288.57	0.63	16.34	1.00	10.35	0.43	0.11	YES

CPT #1 - SITE PREPARATION

SENSITIVE FINE GRAINED:

Average Water Content: $(4.5 + 54.5 + 62.2)/3 = 40.4\%$

$C_c = 0.0054\,(2.6W - 35) = 0.0054\,(2.6 \times 40.4 - 35) = 0.378216$

$e_0 = SG \times W = 2.6 \times 0.404 = 1.054$

$$\frac{C_c}{1+e_0} = \frac{0.378216}{1+1.054} = 0.184459617$$

CLAY:

Avg. Water Content $= 43.3375\%$

$C_c = 0.4194$

$e_0 = 1.13$

$$\frac{C_c}{1+e_0} = 0.1972274923$$

SAND TO SILTY SAND:

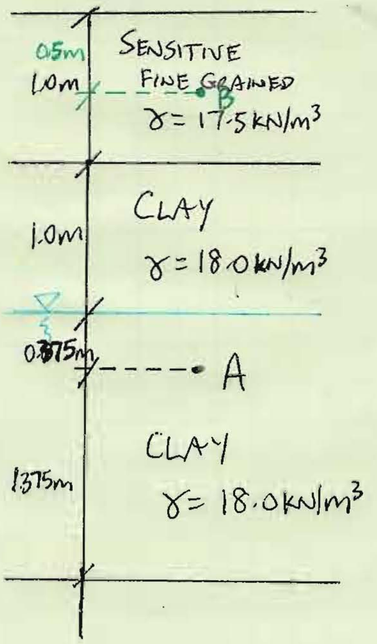
Avg. Water Content $= 21.8786\%$

$C_c = 0.118$

$e_0 = 0.5688$

$$\frac{C_c}{1+e_0} = 0.0752$$

CPT#1 SITE PREPARATION (cont'd)



At Point A:

$$\sigma'_{zo} = (17.5 kN/m^3)(1.0m) + (18 kN/m^3)(1.375m) - (9.8 kN/m^3)(0.375m)$$

$$= 38.575 kPa$$

Assume:

$$\gamma_{fill} = 18.0 kN/m^3$$

$$H_{fill} = 5.0m$$

$$\sigma'_{zf} = \sigma'_{zo} + \gamma_{fill} H_{fill}$$

$$= 38.575 kPa + (18.0 kN/m^3)(5.0m)$$

$$= 128.575 kPa$$

At Point B:

$$\sigma'_{zo} = (17.5 kN/m^3)(0.5m) = 8.75 kPa$$

$$\sigma'_{zf} = 8.75 kPa + 90 kPa = 98.75 kPa$$

$$\delta_{ult} = \delta_A + \delta_B$$

$$= \frac{C_c}{1+e_o} \log\left(\frac{\sigma'_{zo} + \Delta\sigma'}{\sigma'_{zo}}\right) H + \frac{C_c}{1+e_o} \log\left(\frac{\sigma'_{zo} + \Delta\sigma'}{\sigma'_{zo}}\right) H$$

$$= (0.184459617) \log\left(\frac{8.75+90}{8.75}\right)(1.0) + (0.1972274923) \log\left(\frac{38.575+90}{38.575}\right)(2.75)$$

$$= 0.47698471 m \cong \underline{\underline{0.477m}}$$

CPT#1 SITE PREPARATION (cont'd)

CLAY:

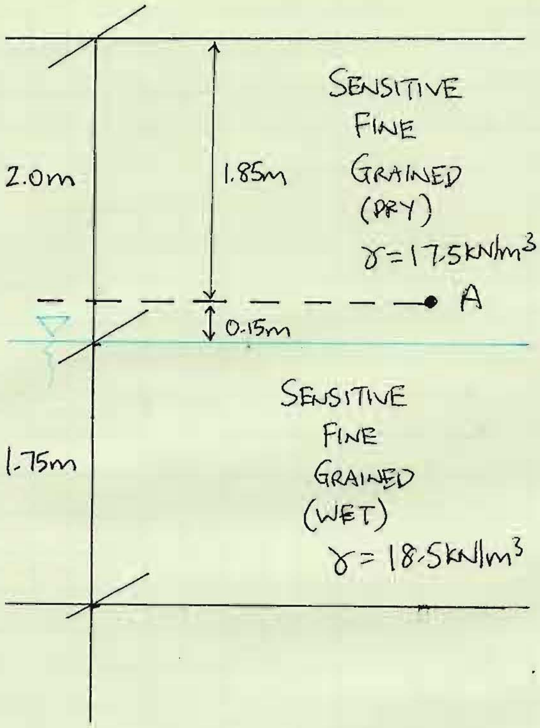
$$C_v = \left(\frac{2.30\ \sigma_z' K}{\gamma_w}\right)\left(\frac{1+e_o}{C_c}\right)$$

where $K = 1.0 \times 10^{-7}$ cm/sec
$= 1.0 \times 10^{-9}$ m/sec
$\gamma_w = 9.8$ kN/m³

$$= \left[\frac{(2.30)(38.75\ \text{kN/m}^2)(1.0\times10^{-9}\ \text{m/sec})}{9.8\ \text{kN/m}^3}\right]\left(\frac{1+1.13}{0.4194}\right)$$

$$= \underline{\underline{4.61 \times 10^{-8}\ \text{m}^2/\text{sec}}}$$

CPT#2 - SITE PREPARATION



SENSITIVE FINE GRAINED:

Avg. Water Content : $W = 53.3\%$

$C_c = 0.559332$

$e_o = 1.3858$

$\frac{C_c}{1+e_o} = 0.25120798$

SAND TO SILTY SAND:

$\frac{C_c}{1+e_o} = 0.0752$

AT Point A:

$\sigma'_{zo} = (17.5 kN/m^3)(1.85m) = 32.8125 \text{ kPa}$

$\sigma'_{zf} = \sigma'_{zo} + \Delta\sigma' = 32.8125 + 90$

$= 122.8125 \text{ kPa}$

$$\delta_{ult} = \frac{C_c}{1+e_o} \log\left(\frac{\sigma'_{zf}}{\sigma'_{zo}}\right) H$$

$$= 0.25120798 \log\left(\frac{122.8125}{32.8125}\right)(3.75)$$

$$= 0.539974615 \text{ m} \cong \underline{\underline{0.5400 \text{ m}}}$$

SENSITIVE FINE GRAINED:

$$C_v = \left(\frac{2.30\,\sigma'_z K}{\gamma_w}\right)\left(\frac{1+e_o}{C_c}\right)$$

$$= \left[\frac{(2.30)(32.8125)(1.0\times10^{-9})}{9.8}\right]\left(\frac{1}{0.25120798}\right)$$

$$= \underline{\underline{3.065544676 \times 10^{-8} \text{ m}^2/\text{sec}}}$$

DYNAMIC COMPACTION:

MAXIMUM DEPTH OF INFLUENCE (d_{max}):

$$d_{max} = \alpha \sqrt{WH}$$

where $W = 150 kN$ weight hammer
$H = 5.0 m$ hammer drop
$\alpha = 5.0 \times 10^{-3} \sqrt{m/N}$

$$= 5.0 \times 10^{-3} \sqrt{m/N} \left[(150000 N)(5.0 m)\right]^{1/2}$$

$$= \underline{\underline{4.33 m}}$$

DISTANCE FROM IMPACT (D):

$$D > \frac{\sqrt{WH}}{80}$$

$$D > \frac{\left[(1.5 \times 10^5 N)(5 m)\right]^{1/2}}{80}$$

$$D > 10.8 m$$

(Dynamic Compaction has to be at least 10.8 metres away from other neighbouring structures.)

SITE PREPARATION CALCULATIONS - CPT #1

C_v 4.61E-08 m^2/sec

***Block dimensions are 59"x29.5"x29.5"*

H_{dr} 1.375 m

$(\delta_c)_{ult}$ 0.47698 m

σ'_{zo} 38.575 kPa

Years	T_v	U	(δ_c) (m)	σ'_{zf} (kPa)	Surcharge (kPa)	Height of Wall (m)	# of Blocks
0.05	0.038	22.1%	0.1055	53.89	-36.11	---	---
0.1	0.077	31.3%	0.1492	61.89	-28.11	---	---
0.15	0.115	38.3%	0.1828	68.83	-21.17	---	---
0.2	0.154	44.3%	0.2111	75.28	-14.72	---	---
0.25	0.192	49.5%	0.2360	81.47	-8.53	---	---
0.3	0.231	54.1%	0.2581	87.39	-2.61	---	---
0.35	0.269	58.3%	0.2779	93.05	3.05	0.17	1
0.4	0.308	62.0%	0.2960	98.51	8.51	0.47	1
0.45	0.346	65.5%	0.3124	103.76	13.76	0.76	2
0.5	0.384	68.6%	0.3273	108.78	18.78	1.04	2
0.55	0.423	71.5%	0.3408	113.55	23.55	1.31	2
0.6	0.461	74.0%	0.3531	118.08	28.08	1.56	3
0.65	0.500	76.4%	0.3643	122.34	32.34	1.80	3
0.7	0.538	78.5%	0.3745	126.36	36.36	2.02	3
0.75	0.577	80.5%	0.3838	130.13	40.13	2.23	3
0.8	0.615	82.2%	0.3923	133.65	43.65	2.43	4
0.85	0.654	83.8%	0.3999	136.94	46.94	2.61	4
0.9	0.692	85.3%	0.4069	140.00	50.00	2.78	4
0.95	0.731	86.6%	0.4132	142.84	52.84	2.94	4
1	0.769	87.8%	0.4190	145.48	55.48	3.08	5
1.1	**0.846**	**89.9%**	**0.4290**	**150.17**	**60.17**	**3.34**	**5**
1.2	0.923	91.7%	0.4373	154.16	64.16	3.56	5
1.3	1.000	93.1%	0.4442	157.55	67.55	3.75	6
1.4	1.077	94.3%	0.4498	160.41	70.41	3.91	6
1.5	1.153	95.3%	0.4545	162.81	72.81	4.04	6
1.6	1.230	96.1%	0.4584	164.82	74.82	4.16	6
1.7	1.307	96.8%	0.4616	166.50	76.50	4.25	6
1.8	1.384	97.3%	0.4643	167.91	77.91	4.33	6
1.9	1.461	97.8%	0.4665	169.08	79.08	4.39	6
2	1.538	98.2%	0.4683	170.06	80.06	4.45	6
2.2	1.692	98.8%	0.4710	171.54	81.54	4.53	7
2.4	1.845	99.1%	0.4729	172.57	82.57	4.59	7
2.6	1.999	99.4%	0.4742	173.27	83.27	4.63	7
2.8	2.153	99.6%	0.4751	173.75	83.75	4.65	7
3	2.307	99.7%	0.4757	174.09	84.09	4.67	7
3.2	2.461	99.8%	0.4761	174.31	84.31	4.68	7
3.4	2.614	99.9%	0.4764	174.47	84.47	4.69	7
3.6	2.768	99.9%	0.4766	174.57	84.57	4.70	7

SITE PREPARATION CALCULATIONS - CPT #2

***Block dimensions are 59"x29.5"x29.5"*

C_v	3.07E-08	m^2/sec
H_{dr}	1.875	m
$(\delta_c)_{ult}$	0.53997	m
σ'_{zo}	32.8125	kPa

Years	T_v	U	(δ_c) (m)	σ'_{zf} (kPa)	Surcharge (kPa)	Height of Wall (m)	# of Blocks
0.001	0.000	1.9%	0.0101	33.72	-56.28	---	---
0.003	0.001	3.2%	0.0175	34.40	-55.60	---	---
0.005	0.001	4.2%	0.0226	34.88	-55.12	---	---
0.007	0.002	5.0%	0.0267	35.27	-54.73	---	---
0.01	0.003	5.9%	0.0320	35.77	-54.23	---	---
0.02	0.006	8.4%	0.0452	37.07	-52.93	---	---
0.03	0.008	10.2%	0.0553	38.10	-51.90	---	---
0.04	0.011	11.8%	0.0639	38.99	-51.01	---	---
0.05	0.014	13.2%	0.0714	39.79	-50.21	---	---
0.06	0.017	14.5%	0.0783	40.53	-49.47	---	---
0.07	0.019	15.7%	0.0845	41.22	-48.78	---	---
0.08	0.022	16.7%	0.0904	41.87	-48.13	---	---
0.09	0.025	17.8%	0.0959	42.50	-47.50	---	---
0.1	0.028	18.7%	0.1010	43.10	-46.90	---	---
0.15	0.041	22.9%	0.1238	45.82	-44.18	---	---
0.2	0.055	26.5%	0.1429	48.25	-41.75	---	---
1.2	0.330	64.1%	0.3461	83.49	-6.51	---	---
1.25	0.344	65.3%	0.3526	84.96	-5.04	---	---
1.3	0.358	66.5%	0.3588	86.40	-3.60	---	---
1.35	0.371	67.6%	0.3649	87.83	-2.17	---	---
1.4	0.385	68.7%	0.3707	89.22	-0.78	---	---
1.45	0.399	69.7%	0.3764	90.59	0.59	0.03	1
1.5	0.413	70.7%	0.3818	91.94	1.94	0.11	1
1.55	0.426	71.7%	0.3871	93.25	3.25	0.18	1
1.6	0.440	72.6%	0.3922	94.55	4.55	0.25	1
1.65	0.454	73.5%	0.3971	95.81	5.81	0.32	1
1.7	0.468	74.4%	0.4019	97.05	7.05	0.39	1
1.75	0.481	75.3%	0.4065	98.27	8.27	0.46	1
1.8	0.495	76.1%	0.4110	99.46	9.46	0.53	1
1.85	0.509	76.9%	0.4153	100.62	10.62	0.59	1
1.9	0.523	77.7%	0.4194	101.76	11.76	0.65	1
1.95	0.536	78.4%	0.4234	102.87	12.87	0.71	1
2	0.550	79.1%	0.4273	103.95	13.95	0.78	2
2.05	0.564	79.8%	0.4311	105.01	15.01	0.83	2
2.1	0.578	80.5%	0.4347	106.05	16.05	0.89	2
2.15	0.591	81.2%	0.4382	107.05	17.05	0.95	2
2.2	0.605	81.8%	0.4416	108.04	18.04	1.00	2
2.25	0.619	82.4%	0.4449	109.00	19.00	1.06	2
2.3	0.633	83.0%	0.4481	109.94	19.94	1.11	2
2.35	0.646	83.5%	0.4511	110.85	20.85	1.16	2

2.4	0.660	84.1%	0.4541	111.74	21.74	1.21	2
2.45	0.674	84.6%	0.4570	112.61	22.61	1.26	2
2.5	0.688	85.1%	0.4597	113.45	23.45	1.30	2
2.55	0.701	85.6%	0.4624	114.28	24.28	1.35	2
2.6	0.715	86.1%	0.4650	115.08	25.08	1.39	2
2.65	0.729	86.6%	0.4675	115.86	25.86	1.44	2
2.7	0.743	87.0%	0.4699	116.61	26.61	1.48	2
2.75	0.756	87.5%	0.4723	117.35	27.35	1.52	3
2.8	0.770	87.9%	0.4745	118.07	28.07	1.56	3
2.85	0.784	88.3%	0.4767	118.77	28.77	1.60	3
2.9	0.798	88.7%	0.4788	119.45	29.45	1.64	3
2.95	0.811	89.1%	0.4809	120.11	30.11	1.67	3
3	0.825	89.4%	0.4828	120.75	30.75	1.71	3
3.05	0.839	89.8%	0.4847	121.37	31.37	1.74	3
3.1	**0.853**	**90.1%**	**0.4866**	**121.97**	**31.97**	**1.78**	**3**
3.15	0.866	90.4%	0.4884	122.56	32.56	1.81	3
3.2	0.880	90.8%	0.4901	123.13	33.13	1.84	3
3.25	0.894	91.1%	0.4918	123.69	33.69	1.87	3
3.3	0.908	91.4%	0.4934	124.23	34.23	1.90	3
3.35	0.921	91.7%	0.4949	124.75	34.75	1.93	3
3.4	0.935	91.9%	0.4964	125.26	35.26	1.96	3
3.45	0.949	92.2%	0.4979	125.75	35.75	1.99	3
3.5	0.963	92.5%	0.4993	126.23	36.23	2.01	3
3.55	0.976	92.7%	0.5006	126.69	36.69	2.04	3
3.6	0.990	93.0%	0.5019	127.14	37.14	2.06	3
3.65	1.004	93.2%	0.5032	127.57	37.57	2.09	3
3.7	1.018	93.4%	0.5044	128.00	38.00	2.11	3
3.75	1.031	93.6%	0.5056	128.41	38.41	2.13	3
3.8	1.045	93.9%	0.5068	128.80	38.80	2.16	3
3.85	1.059	94.1%	0.5079	129.19	39.19	2.18	3
3.9	1.073	94.3%	0.5090	129.56	39.56	2.20	3
3.95	1.086	94.4%	0.5100	129.93	39.93	2.22	3
4	1.100	94.6%	0.5110	130.28	40.28	2.24	3
6	1.650	98.6%	0.5325	138.07	48.07	2.67	4
6.75	1.856	99.2%	0.5355	139.18	49.18	2.73	4
6.8	1.870	99.2%	0.5356	139.24	49.24	2.74	4
6.85	1.884	99.2%	0.5358	139.29	49.29	2.74	4
6.9	1.898	99.3%	0.5359	139.35	49.35	2.74	4
6.95	1.911	99.3%	0.5361	139.40	49.40	2.74	4
7	1.925	99.3%	0.5362	139.45	49.45	2.75	4
7.05	1.939	99.3%	0.5363	139.49	49.49	2.75	4
7.1	1.953	99.3%	0.5364	139.54	49.54	2.75	4

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 1, 2009
Identification	CPT 1 - Square

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	SQ	SQ, CI, CO, or RE
B =	2	m
L =		m
D =	1.5	m

Soil Information

c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	753	kPa	1,063	kPa
q a =	251	kPa	354	kPa
Allowable Column Load				
P =	1,004	kN	1,418	kN

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date April 1, 2009
Identification CPT 1 - Circular

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	CI	SQ, CI, CO, or RE
B =	2.5	m
L =		m
D =	1.5	m

Soil Information

c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	735	kPa	1,056	kPa
q a =	245	kPa	352	kPa
Allowable Column Load				
P =	1,203	kN	1,728	kN

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 1, 2009	
Identification	CPT 1 - Continuous	

Input

Units of Measurement		
	SI	SI or E
Foundation Information		
Shape	CO	SQ, CI, CO, or RE
B =	3	m
L =		m
D =	2	m
Soil Information		
c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m
Factor of Safety		
F =	3	

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	1,018	kPa	1,027	kPa
q a =	339	kPa	342	kPa
Allowable Wall Load				
P/b =	1,018	kN/m	1,027	kN/m

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date April 1, 2009
Identification CPT 1 - Rectangular

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	RE	SQ, CI, CO, or RE
B =	1	m
L =	4	m
D =	1.5	m

Soil Information

c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	n/a	kPa	837	kPa
q a =	n/a	kPa	279	kPa
Allowable Column Load				
P =	#VALUE!	kN	1,115	kN

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 1, 2009
Identification	CPT 2 - Square

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	SQ	SQ, CI, CO, or RE
B =	2	m
L =		m
D =	1	m

Soil Information

c =	0	kPa
phi =	32	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	782	kPa	978	kPa
q a =	261	kPa	326	kPa
Allowable Column Load				
P =	1,042	kN	1,305	kN

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 1, 2009
Identification	CPT 2 - Circular

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	CI	SQ, CI, CO, or RE
B =	2.3	m
L =		m
D =	1	m

Soil Information

c =	0	kPa
phi =	32	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	731	kPa	988	kPa
q a =	244	kPa	329	kPa
Allowable Column Load				
P =	1,012	kN	1,368	kN

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 1, 2009
Identification	CPT 2 - Continuous

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	CO	SQ, CI, CO, or RE
B =	2.5	m
L =		m
D =	2	m

Soil Information

c =	0	kPa
phi =	32	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	1,268	kPa	1,281	kPa
q a =	423	kPa	427	kPa
Allowable Wall Load				
P/b =	1,057	kN/m	1,068	kN/m

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date April 1, 2009
Identification CPT 2 - Rectangular

Input

Units of Measurement

	SI	SI or E

Foundation Information

Shape	RE	SQ, CI, CO, or RE
B =	1	m
L =	2.5	m
D =	2	m

Soil Information

c =	0	kPa
phi =	32	deg
gamma =	17.5	kN/m^3
Dw =	2	m

Factor of Safety

F =	3

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	n/a	kPa	1,422	kPa
q a =	n/a	kPa	474	kPa
Allowable Column Load				
P =	#VALUE!	kN	1,185	kN

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 1 - Square

Input

Units	SI	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	2	m
L =		m
D =	1.5	m
P =	1418	kN
Dw =	2	m
r =	0.85	

Results

q = 390 kPa
delta = 308.09 mm

Depth to Soil Layer													
Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	1.5				17.5							
1.5	1.6	0.197	0.066	0	18	0.05	27	27	364	391	19.39	19.393
1.6	1.7	0.197	0.066	0	18	0.15	29	29	363	392	18.94	18.944
1.7	1.8	0.197	0.066	0	18	0.25	31	31	360	390	18.48	18.482
1.8	1.9	0.197	0.066	0	18	0.35	33	33	354	386	17.99	17.987
1.9	2.0	0.197	0.066	0	18	0.45	34	34	344	378	17.45	17.449
2.0	2.1	0.197	0.066	0	18	0.55	36	36	331	367	16.96	16.957
2.1	2.2	0.197	0.066	0	18	0.65	36	36	316	353	16.50	16.504
2.2	2.3	0.197	0.066	0	18	0.75	37	37	300	337	16.01	16.005
2.3	2.4	0.197	0.066	0	18	0.85	38	38	282	320	15.47	15.472
2.4	2.5	0.197	0.066	0	18	0.95	39	39	264	303	14.92	14.915
2.5	2.6	0.197	0.066	0	18	1.05	40	40	246	286	14.34	14.344
2.6	2.7	0.197	0.066	0	18	1.15	41	41	229	269	13.77	13.766
2.7	2.8	0.197	0.066	0	18	1.25	41	41	212	254	13.19	13.189
2.8	2.9	0.197	0.066	0	18	1.35	42	42	197	239	12.62	12.617
2.9	3.0	0.197	0.066	0	18	1.45	43	43	183	226	12.06	12.056
3.0	3.1	0.197	0.066	0	18	1.55	44	44	170	213	11.51	11.507
3.1	3.2	0.197	0.066	0	18	1.65	45	45	157	202	10.98	10.975
3.2	3.3	0.197	0.066	0	18	1.75	45	45	146	192	10.46	10.460
3.3	3.4	0.197	0.066	0	18	1.85	46	46	136	182	9.96	9.964
3.4	3.5	0.197	0.066	0	18	1.95	47	47	127	174	9.49	9.487
3.5	3.6	0.197	0.066	0	18	2.05	48	48	118	166	9.03	9.029
3.6	3.7	0.197	0.066	0	18	2.15	49	49	110	159	8.59	8.592

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 1 - Circular

Input

Units	SI	E or SI
Shape	CI	SQ, CI, CO, or RE
B =	2.5	m
L =		m
D =	1.5	m
P =	1728	kN
Dw =	2	m
r =	0.85	

Results

q =	388	kPa
delta =	305.46	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	1.5				17.5							
1.5	1.6	0.197	0.066	0	18	0.05	27	27	361	388	19.35	19.350
1.6	1.7	0.197	0.066	0	18	0.15	29	29	360	389	18.90	18.900
1.7	1.8	0.197	0.066	0	18	0.25	31	31	357	388	18.44	18.435
1.8	1.9	0.197	0.066	0	18	0.35	33	33	351	383	17.94	17.935
1.9	2.0	0.197	0.066	0	18	0.45	34	34	341	375	17.39	17.389
2.0	2.1	0.197	0.066	0	18	0.55	36	36	328	364	16.89	16.886
2.1	2.2	0.197	0.066	0	18	0.65	36	36	312	349	16.42	16.421
2.2	2.3	0.197	0.066	0	18	0.75	37	37	295	333	15.91	15.910
2.3	2.4	0.197	0.066	0	18	0.85	38	38	277	315	15.36	15.365
2.4	2.5	0.197	0.066	0	18	0.95	39	39	259	298	14.80	14.796
2.5	2.6	0.197	0.066	0	18	1.05	40	40	241	281	14.21	14.214
2.6	2.7	0.197	0.066	0	18	1.15	41	41	224	264	13.63	13.627
2.7	2.8	0.197	0.066	0	18	1.25	41	41	207	249	13.04	13.042
2.8	2.9	0.197	0.066	0	18	1.35	42	42	192	234	12.46	12.463
2.9	3.0	0.197	0.066	0	18	1.45	43	43	178	221	11.90	11.896
3.0	3.1	0.197	0.066	0	18	1.55	44	44	165	209	11.34	11.343
3.1	3.2	0.197	0.066	0	18	1.65	45	45	153	197	10.81	10.808
3.2	3.3	0.197	0.066	0	18	1.75	45	45	142	187	10.29	10.290
3.3	3.4	0.197	0.066	0	18	1.85	46	46	132	178	9.79	9.792
3.4	3.5	0.197	0.066	0	18	1.95	47	47	123	170	9.31	9.314
3.5	3.6	0.197	0.066	0	18	2.05	48	48	114	162	8.86	8.857
3.6	3.7	0.197	0.066	0	18	2.15	49	49	106	155	8.42	8.420

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 1 - Continuous

Input

Units	SI	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	3	m
L =		m
D =	2	m
P =	1027	kN/m
Dw =	2	m
r =	0.85	

Results

q =	390	kPa
delta =	269.34	mm

Depth to Soil Layer												
Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	2.0				17.5							
2.0	2.1	0.197	0.066	0	18	0.05	35	35	355	390	17.45	17.446
2.1	2.2	0.197	0.066	0	18	0.15	36	36	354	391	17.29	17.292
2.2	2.3	0.197	0.066	0	18	0.25	37	37	354	391	17.13	17.135
2.3	2.4	0.197	0.066	0	18	0.35	38	38	353	391	16.97	16.970
2.4	2.5	0.197	0.066	0	18	0.45	39	39	351	390	16.79	16.795
2.5	2.6	0.197	0.066	0	18	0.55	40	40	348	388	16.61	16.607
2.6	2.7	0.197	0.066	0	18	0.65	40	40	345	385	16.41	16.406
2.7	2.8	0.197	0.066	0	18	0.75	41	41	340	381	16.19	16.191
2.8	2.9	0.197	0.066	0	18	0.85	42	42	335	377	15.96	15.964
2.9	3.0	0.197	0.066	0	18	0.95	43	43	329	372	15.73	15.725
3.0	3.1	0.197	0.066	0	18	1.05	44	44	323	366	15.48	15.477
3.1	3.2	0.197	0.066	0	18	1.15	44	44	316	360	15.22	15.221
3.2	3.3	0.197	0.066	0	18	1.25	45	45	309	354	14.96	14.959
3.3	3.4	0.197	0.066	0	18	1.35	46	46	301	347	14.69	14.693
3.4	3.5	0.197	0.066	0	18	1.45	47	47	294	341	14.42	14.424
3.5	3.6	0.197	0.066	0	18	1.55	48	48	286	334	14.15	14.153
3.6	3.7	0.197	0.066	0	18	1.65	49	49	279	327	13.88	13.883

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date	March 28, 2009
Identification	CPT 1 - Rectangular

Input

Units	SI	E or SI
Shape	RE	SQ, CI, CO, or RE
B =	1	m
L =	4	m
D =	1.5	m
P =	1115	kN
Dw =	2	m
r =	0.85	

Results

q =	314	kPa
delta =	253.87	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	1.5				17.5							
1.5	1.6	0.197	0.066	0	18	0.05	27	27	288	315	17.82	17.824
1.6	1.7	0.197	0.066	0	18	0.15	29	29	285	314	17.33	17.333
1.7	1.8	0.197	0.066	0	18	0.25	31	31	276	307	16.73	16.731
1.8	1.9	0.197	0.066	0	18	0.35	33	33	262	294	16.02	16.016
1.9	2.0	0.197	0.066	0	18	0.45	34	34	244	279	15.23	15.225
2.0	2.1	0.197	0.066	0	18	0.55	36	36	226	262	14.49	14.489
2.1	2.2	0.197	0.066	0	18	0.65	36	36	208	244	13.83	13.828
2.2	2.3	0.197	0.066	0	18	0.75	37	37	191	228	13.17	13.173
2.3	2.4	0.197	0.066	0	18	0.85	38	38	176	214	12.54	12.537
2.4	2.5	0.197	0.066	0	18	0.95	39	39	162	201	11.93	11.926
2.5	2.6	0.197	0.066	0	18	1.05	40	40	149	189	11.34	11.342
2.6	2.7	0.197	0.066	0	18	1.15	41	41	138	179	10.79	10.786
2.7	2.8	0.197	0.066	0	18	1.25	41	41	128	170	10.26	10.257
2.8	2.9	0.197	0.066	0	18	1.35	42	42	119	161	9.75	9.753
2.9	3.0	0.197	0.066	0	18	1.45	43	43	111	154	9.28	9.275
3.0	3.1	0.197	0.066	0	18	1.55	44	44	104	148	8.82	8.821
3.1	3.2	0.197	0.066	0	18	1.65	45	45	97	142	8.39	8.388
3.2	3.3	0.197	0.066	0	18	1.75	45	45	91	136	7.98	7.977
3.3	3.4	0.197	0.066	0	18	1.85	46	46	85	131	7.59	7.586
3.4	3.5	0.197	0.066	0	18	1.95	47	47	80	127	7.21	7.215
3.5	3.6	0.197	0.066	0	18	2.05	48	48	75	123	6.86	6.862
3.6	3.7	0.197	0.066	0	18	2.15	49	49	71	120	6.53	6.526

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 28, 2009
Identification: CPT 2 - Square

Input

Units	SI	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	2	m
L =		m
D =	1	m
P =	1305	kN
Dw =	2	m
r =	0.85	

Results

q =	350	kPa
delta =	396.76	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	1.0				17.5							
1.0	1.1	0.22755	0.07585	0	17.5	0.05	18	18	332	351	24.77	24.771
1.1	1.2	0.22755	0.07585	0	17.5	0.15	20	20	332	352	24.03	24.030
1.2	1.3	0.22755	0.07585	0	17.5	0.25	22	22	329	351	23.30	23.305
1.3	1.4	0.22755	0.07585	0	17.5	0.35	24	24	323	347	22.57	22.565
1.4	1.5	0.22755	0.07585	0	17.5	0.45	25	25	314	340	21.80	21.796
1.5	1.6	0.22755	0.07585	0	17.5	0.55	27	27	303	330	20.99	20.991
1.6	1.7	0.22755	0.07585	0	17.5	0.65	29	29	289	318	20.15	20.154
1.7	1.8	0.22755	0.07585	0	17.5	0.75	31	31	274	304	19.29	19.292
1.8	1.9	0.22755	0.07585	0	17.5	0.85	32	32	258	290	18.42	18.417
1.9	2.0	0.22755	0.07585	0	17.5	0.95	34	34	241	275	17.54	17.536
2.0	2.1	0.22755	0.07585	0	18.5	1.05	35	35	225	260	16.75	16.750
2.1	2.2	0.22755	0.07585	0	18.5	1.15	36	36	209	245	16.05	16.053
2.2	2.3	0.22755	0.07585	0	18.5	1.25	37	37	194	231	15.36	15.358
2.3	2.4	0.22755	0.07585	0	18.5	1.35	38	38	180	218	14.67	14.672
2.4	2.5	0.22755	0.07585	0	18.5	1.45	39	39	167	206	14.00	13.998
2.5	2.6	0.22755	0.07585	0	18.5	1.55	40	40	155	195	13.34	13.342
2.6	2.7	0.22755	0.07585	0	18.5	1.65	41	41	144	185	12.71	12.706
2.7	2.8	0.22755	0.07585	0	18.5	1.75	42	42	134	175	12.09	12.091
2.8	2.9	0.22755	0.07585	0	18.5	1.85	42	42	124	167	11.50	11.500
2.9	3.0	0.22755	0.07585	0	18.5	1.95	43	43	116	159	10.93	10.932
3.0	3.1	0.22755	0.07585	0	18.5	2.05	44	44	108	152	10.39	10.388
3.1	3.2	0.22755	0.07585	0	18.5	2.15	45	45	101	146	9.87	9.869
3.2	3.3	0.22755	0.07585	0	18.5	2.25	46	46	94	140	9.37	9.374
3.3	3.4	0.22755	0.07585	0	18.5	2.35	47	47	88	135	8.90	8.903
3.4	3.5	0.0752	0.025067	0	18.5	2.45	48	48	83	130	2.79	2.794
3.5	3.6	0.0752	0.025067	0	18.5	2.55	48	48	78	126	2.65	2.653
3.6	3.7	0.0752	0.025067	0	18.5	2.65	49	49	73	122	2.52	2.520

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 2 - Circular

Input

Units	SI	E or SI
Shape	CI	SQ, CI, CO, or RE
B =	2.3	m
L =		m
D =	1	m
P =	1368	kN
Dw =	2	m
r =	0.85	

Results

q =	353	kPa
delta =	398.83	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	1.0				17.5							
1.0	1.1	0.22755	0.07585	0	17.5	0.05	18	18	335	354	24.85	24.847
1.1	1.2	0.22755	0.07585	0	17.5	0.15	20	20	334	355	24.10	24.098
1.2	1.3	0.22755	0.07585	0	17.5	0.25	22	22	331	353	23.35	23.351
1.3	1.4	0.22755	0.07585	0	17.5	0.35	24	24	323	347	22.57	22.569
1.4	1.5	0.22755	0.07585	0	17.5	0.45	25	25	312	338	21.74	21.739
1.5	1.6	0.22755	0.07585	0	17.5	0.55	27	27	298	325	20.86	20.860
1.6	1.7	0.22755	0.07585	0	17.5	0.65	29	29	281	310	19.94	19.941
1.7	1.8	0.22755	0.07585	0	17.5	0.75	31	31	263	294	19.00	18.995
1.8	1.9	0.22755	0.07585	0	17.5	0.85	32	32	245	277	18.04	18.038
1.9	2.0	0.22755	0.07585	0	17.5	0.95	34	34	227	261	17.08	17.082
2.0	2.1	0.22755	0.07585	0	18.5	1.05	35	35	209	245	16.23	16.226
2.1	2.2	0.22755	0.07585	0	18.5	1.15	36	36	193	229	15.47	15.466
2.2	2.3	0.22755	0.07585	0	18.5	1.25	37	37	177	214	14.72	14.717
2.3	2.4	0.22755	0.07585	0	18.5	1.35	38	38	163	201	13.98	13.984
2.4	2.5	0.22755	0.07585	0	18.5	1.45	39	39	150	189	13.27	13.273
2.5	2.6	0.22755	0.07585	0	18.5	1.55	40	40	138	178	12.59	12.586
2.6	2.7	0.22755	0.07585	0	18.5	1.65	41	41	127	168	11.93	11.926
2.7	2.8	0.22755	0.07585	0	18.5	1.75	42	42	118	159	11.29	11.294
2.8	2.9	0.22755	0.07585	0	18.5	1.85	42	42	109	151	10.69	10.690
2.9	3.0	0.22755	0.07585	0	18.5	1.95	43	43	101	144	10.12	10.115
3.0	3.1	0.22755	0.07585	0	18.5	2.05	44	44	94	138	9.57	9.569
3.1	3.2	0.22755	0.07585	0	18.5	2.15	45	45	87	132	9.05	9.051
3.2	3.3	0.22755	0.07585	0	18.5	2.25	46	46	81	127	8.56	8.561
3.3	3.4	0.22755	0.07585	0	18.5	2.35	47	47	76	123	8.10	8.097
3.4	3.5	0.22755	0.07585	0	18.5	2.45	48	48	71	118	7.66	7.659
3.5	3.6	0.22755	0.07585	0	18.5	2.55	48	48	66	115	7.25	7.245
3.6	3.7	0.22755	0.07585	0	18.5	2.65	49	49	62	112	6.86	6.855

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 2 - Continuous

Input

Units	SI	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	2.5	m
L =		m
D =	2	m
P =	1068	kN/m
Dw =	2	m
r =	0.85	

Results

q =	474	kPa
delta =	332.73	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	2.0				17.5							
2.0	2.1	0.22755	0.07585	0	18.5	0.05	35	35	439	475	21.80	21.800
2.1	2.2	0.22755	0.07585	0	18.5	0.15	36	36	439	475	21.61	21.606
2.2	2.3	0.22755	0.07585	0	18.5	0.25	37	37	438	475	21.40	21.403
2.3	2.4	0.22755	0.07585	0	18.5	0.35	38	38	436	474	21.18	21.183
2.4	2.5	0.22755	0.07585	0	18.5	0.45	39	39	432	471	20.94	20.942
2.5	2.6	0.22755	0.07585	0	18.5	0.55	40	40	427	466	20.68	20.676
2.6	2.7	0.22755	0.07585	0	18.5	0.65	41	41	420	460	20.39	20.387
2.7	2.8	0.22755	0.07585	0	18.5	0.75	42	42	412	453	20.08	20.076
2.8	2.9	0.22755	0.07585	0	18.5	0.85	42	42	402	445	19.75	19.745
2.9	3.0	0.22755	0.07585	0	18.5	0.95	43	43	392	436	19.40	19.400
3.0	3.1	0.22755	0.07585	0	18.5	1.05	44	44	382	426	19.04	19.042
3.1	3.2	0.22755	0.07585	0	18.5	1.15	45	45	371	416	18.68	18.676
3.2	3.3	0.22755	0.07585	0	18.5	1.25	46	46	360	405	18.30	18.304
3.3	3.4	0.22755	0.07585	0	18.5	1.35	47	47	348	395	17.93	17.930
3.4	3.5	0.22755	0.07585	0	18.5	1.45	48	48	337	385	17.56	17.556
3.5	3.6	0.22755	0.07585	0	18.5	1.55	48	48	327	375	17.18	17.184
3.6	3.7	0.22755	0.07585	0	18.5	1.65	49	49	316	365	16.82	16.815

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date	March 28, 2009
Identification	CPT 2 - Rectangular

Input

Units	SI	E or SI
Shape	RE	SQ, CI, CO, or RE
B =	1	m
L =	2.5	m
D =	2	m
P =	1185	kN
Dw =	2	m
r =	0.85	

Results

q =	521	kPa
delta =	292.13	mm

Depth to Soil Layer												
Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	2.0				17.5							
2.0	2.1	0.22755	0.07585	0	18.5	0.05	35	35	486	521	22.59	22.587
2.1	2.2	0.22755	0.07585	0	18.5	0.15	36	36	481	517	22.32	22.317
2.2	2.3	0.22755	0.07585	0	18.5	0.25	37	37	466	503	21.88	21.882
2.3	2.4	0.22755	0.07585	0	18.5	0.35	38	38	441	479	21.28	21.276
2.4	2.5	0.22755	0.07585	0	18.5	0.45	39	39	410	449	20.54	20.542
2.5	2.6	0.22755	0.07585	0	18.5	0.55	40	40	377	417	19.73	19.731
2.6	2.7	0.22755	0.07585	0	18.5	0.65	41	41	344	385	18.88	18.885
2.7	2.8	0.22755	0.07585	0	18.5	0.75	42	42	314	355	18.03	18.028
2.8	2.9	0.22755	0.07585	0	18.5	0.85	42	42	285	328	17.18	17.179
2.9	3.0	0.22755	0.07585	0	18.5	0.95	43	43	260	303	16.35	16.346
3.0	3.1	0.22755	0.07585	0	18.5	1.05	44	44	236	281	15.54	15.537
3.1	3.2	0.22755	0.07585	0	18.5	1.15	45	45	216	261	14.75	14.754
3.2	3.3	0.22755	0.07585	0	18.5	1.25	46	46	197	243	14.00	13.999
3.3	3.4	0.22755	0.07585	0	18.5	1.35	47	47	180	227	13.28	13.276
3.4	3.5	0.22755	0.07585	0	18.5	1.45	48	48	165	213	12.58	12.582
3.5	3.6	0.22755	0.07585	0	18.5	1.55	48	48	152	200	11.92	11.921
3.6	3.7	0.22755	0.07585	0	18.5	1.65	49	49	140	189	11.29	11.290

k_{v1}	20 MPa/m	(Page 129 CFEM)
q	19.152 kPa	400psf Max Live Loads
δ	0.000958 m 0.9576 mm	
k_{vb}	0.109718 MPa/m	
q	19.152 kPa	
δ	**0.174557 m** **174.5566 mm**	
I	1	(page 161 CFEM)
q	19.152 kPa	
v	0.3	
E	4.788026 MPa	http://www.geotechnicalinfo.com/youngs_modulus.html 50tsf
b	**0.263078 m**	

RAFT FOUNDATION PROPERTIES

k_{v1}	8 MPa/m	(Page 129 CFEM)	
q	19.152 kPa	400psf Max Live Loads	
δ	0.002394 m 2.394 mm		
k_{vb} q	0.793479 MPa/m 19.152 kPa		
δ	**0.024137 m** **24.13674 mm**		
I	1	(page 161 CFEM)	
q	19.152 kPa		
v	0.3		
E	4.788026 MPa	http://www.geotechnicalinfo.com/youngs_modulus.html	50tsf
b	**0.657696 m**		

RAFT BEARING ANALYSIS

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date March 28, 2009
Identification CPT 1

Input

Units of Measurement		
	SI	SI or E
Foundation Information		
Shape	CO	SQ, CI, CO, or RE
B =	0.263078	m
L =		m
D =	0.5	m
Soil Information		
c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m
Factor of Safety		
F =	3	

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	243	kPa	263	kPa
q a =	81	kPa	88	kPa
Allowable Wall Load				
P/b =	21	kN/m	23	kN/m

RAFT BEARING ANALYSIS

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date March 28, 2009
Identification CPT 2

Input

Units of Measurement		
	SI	SI or E
Foundation Information		
Shape	CO	SQ, CI, CO, or RE
B =	0.657696	m
L =		m
D =	0.5	m
Soil Information		
c =	0	kPa
phi =	30	deg
gamma =	17.5	kN/m^3
Dw =	2	m
Factor of Safety		
F =	3	

Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	312	kPa	325	kPa
q a =	104	kPa	108	kPa
Allowable Wall Load				
P/b =	68	kN/m	71	kN/m

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 1 - Raft

Input

Units	SI	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	0.263078	m
L =		m
D =	0.5	m
P =	21	kN/m
Dw =	2	m
r =	0.85	

Results

q =	92	kPa
delta =	106.83	mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	0.5				17.5							
0.5	0.6	0.197	0.065667	0	17.5	0.05	10	10	81	91	16.33	16.326
0.6	0.7	0.197	0.065667	0	17.5	0.15	11	11	64	76	13.77	13.765
0.7	0.8	0.197	0.065667	0	17.5	0.25	13	13	47	60	11.10	11.103
0.8	0.9	0.197	0.065667	0	17.5	0.35	15	15	36	51	8.99	8.991
0.9	1.0	0.197	0.065667	0	17.5	0.45	17	17	29	46	7.38	7.375
1.0	1.1	0.197	0.065667	0	18	0.55	18	18	24	43	6.13	6.126
1.1	1.2	0.197	0.065667	0	18	0.65	20	20	21	41	5.15	5.146
1.2	1.3	0.197	0.065667	0	18	0.75	22	22	18	40	4.37	4.373
1.3	1.4	0.197	0.065667	0	18	0.85	24	24	16	40	3.75	3.753
1.4	1.5	0.197	0.065667	0	18	0.95	26	26	14	40	3.25	3.250
1.5	1.6	0.197	0.065667	0	18	1.05	27	27	13	40	2.84	2.839
1.6	1.7	0.197	0.065667	0	18	1.15	29	29	12	41	2.50	2.498
1.7	1.8	0.197	0.065667	0	18	1.25	31	31	11	42	2.21	2.212
1.8	1.9	0.197	0.065667	0	18	1.35	33	33	10	43	1.97	1.972
1.9	2.0	0.197	0.065667	0	18	1.45	35	35	10	44	1.77	1.768
2.0	2.1	0.197	0.065667	0	18	1.55	36	36	9	45	1.61	1.612
2.1	2.2	0.197	0.065667	0	18	1.65	37	37	8	45	1.49	1.494
2.2	2.3	0.197	0.065667	0	18	1.75	38	38	8	46	1.23	1.225
2.3	2.4	0.197	0.065667	0	18	1.85	39	39	7	46	1.14	1.140
2.4	2.5	0.197	0.065667	0	18	1.95	39	39	7	47	1.06	1.063
2.5	2.6	0.197	0.065667	0	18	2.05	40	40	7	47	0.99	0.994
2.6	2.7	0.197	0.065667	0	18	2.15	41	41	6	48	0.93	0.931
2.7	2.8	0.197	0.065667	0	18	2.25	42	42	6	48	0.87	0.874
2.8	2.9	0.197	0.065667	0	18	2.35	43	43	6	49	0.82	0.823
2.9	3.0	0.197	0.065667	0	18	2.45	44	44	6	50	0.78	0.776
3.0	3.1	0.197	0.065667	0	18	2.55	45	45	5	50	0.73	0.733
3.1	3.2	0.197	0.065667	0	18	2.65	46	46	5	51	0.69	0.693
3.2	3.3	0.197	0.065667	0	18	2.75	47	47	5	52	0.66	0.657
3.3	3.4	0.197	0.065667	0	18	2.85	48	48	5	53	0.62	0.623
3.4	3.5	0.197	0.065667	0	18	2.95	49	49	5	53	0.59	0.592
3.5	3.6	0.197	0.065667	0	18	3.05	50	50	5	54	0.56	0.564
3.6	3.7	0.197	0.065667	0	18	3.15	50	50	4	55	0.54	0.537

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 28, 2009
Identification CPT 2 - Raft

Input

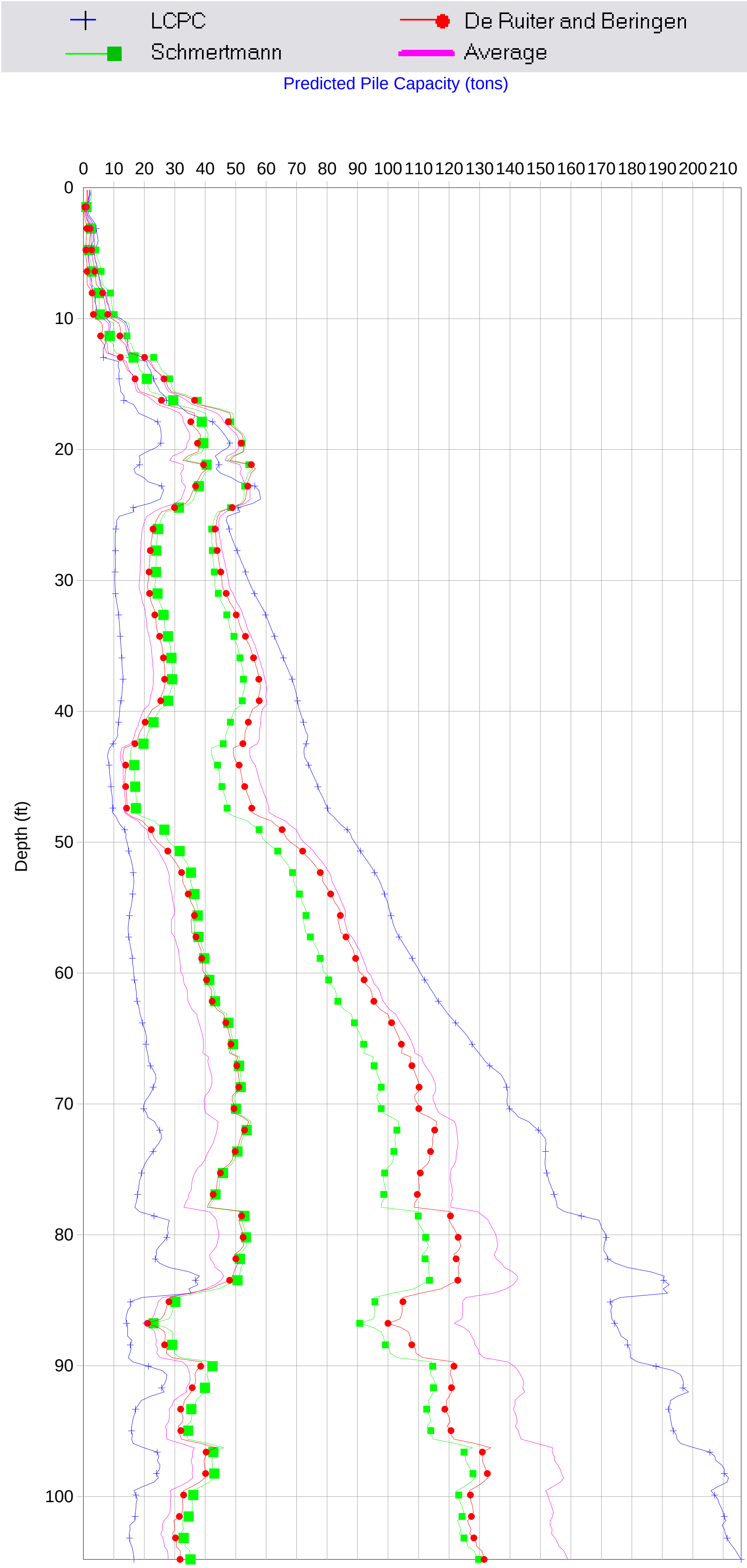
Units	SI	E or SI	
Shape	CO	SQ, CI, CO, or RE	
B =	0.657696	m	
L =		m	
D =	0.5	m	
P =	68	kN/m	
Dw =	2	m	
r =	0.85		

Results

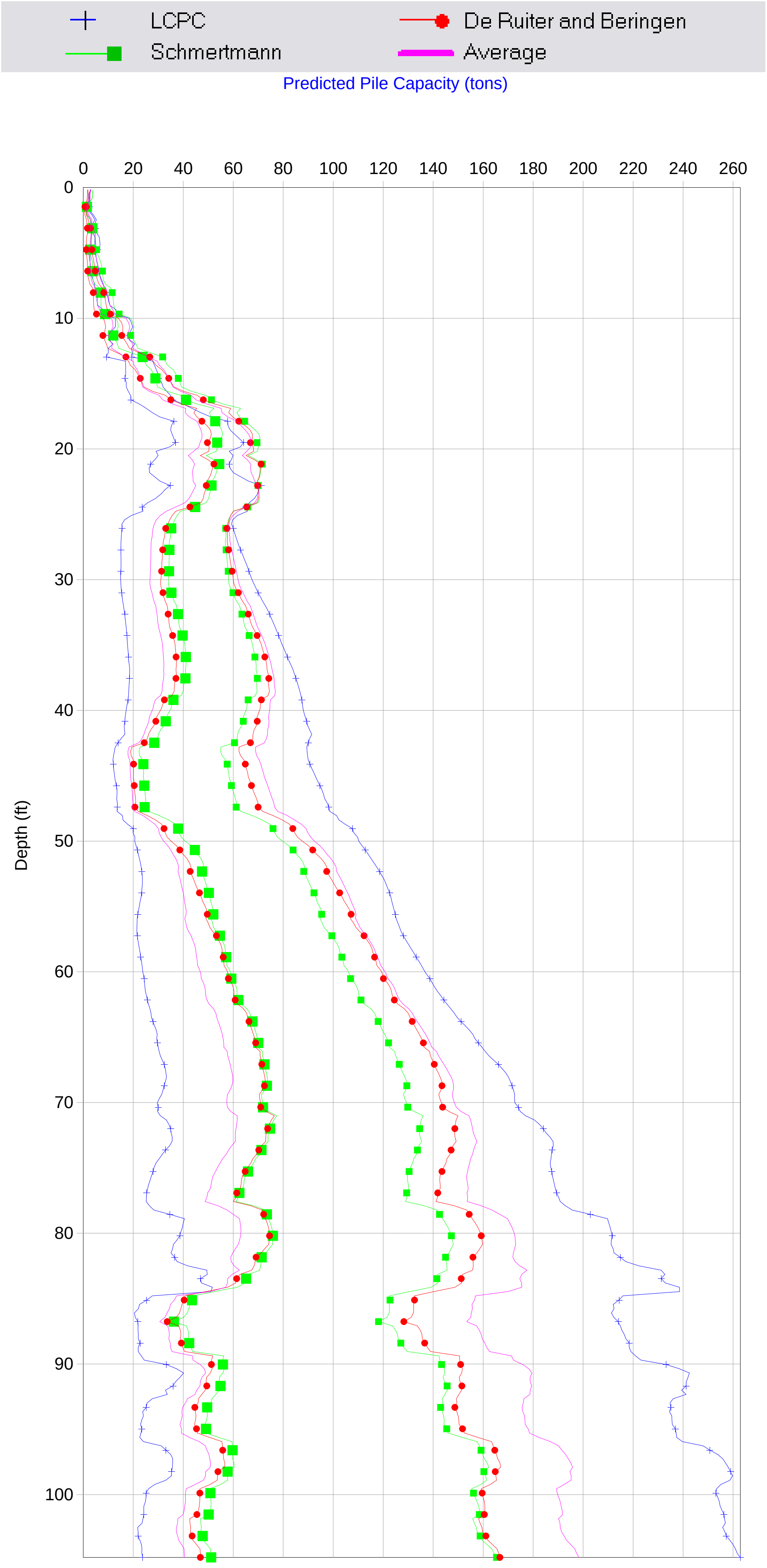
q = 115.191 kPa
delta = 192.62 mm

Depth to Soil Layer Top (m)	Bottom (m)	Cc/(1+e)	Cr/(1+e)	sigma m' (kPa)	gamma (kN/m^3)	zf (m)	sigma c' (kPa)	sigma zo' (kPa)	delta sigma (kPa)	sigma zf' (kPa)	strain (%)	delta (mm)
0.0	0.5				17.5							
0.5	0.6	0.22755	0.07585	0	17.5	0.05	10	10	106	116	20.90	20.903
0.6	0.7	0.22755	0.07585	0	17.5	0.15	11	11	103	114	19.39	19.390
0.7	0.8	0.22755	0.07585	0	17.5	0.25	13	13	95	108	17.72	17.717
0.8	0.9	0.22755	0.07585	0	17.5	0.35	15	15	85	100	15.99	15.989
0.9	1.0	0.22755	0.07585	0	17.5	0.45	17	17	75	92	14.34	14.341
1.0	1.1	0.22755	0.07585	0	17.5	0.55	18	18	66	85	12.82	12.822
1.1	1.2	0.22755	0.07585	0	17.5	0.65	20	20	59	79	11.46	11.457
1.2	1.3	0.22755	0.07585	0	17.5	0.75	22	22	53	75	10.26	10.260
1.3	1.4	0.22755	0.07585	0	17.5	0.85	24	24	48	72	9.21	9.213
1.4	1.5	0.22755	0.07585	0	17.5	0.95	26	26	44	69	8.30	8.298
1.5	1.6	0.22755	0.07585	0	17.5	1.05	28	28	40	68	7.50	7.498
1.6	1.7	0.22755	0.07585	0	17.5	1.15	30	30	37	66	6.80	6.797
1.7	1.8	0.22755	0.07585	0	17.5	1.25	31	31	34	65	6.18	6.180
1.8	1.9	0.22755	0.07585	0	17.5	1.35	33	33	32	65	5.64	5.637
1.9	2.0	0.22755	0.07585	0	17.5	1.45	35	35	30	65	5.16	5.157
2.0	2.1	0.22755	0.07585	0	18.5	1.55	36	36	28	64	4.78	4.779
2.1	2.2	0.22755	0.07585	0	18.5	1.65	37	37	26	64	4.48	4.485
2.2	2.3	0.22755	0.07585	0	18.5	1.75	38	38	25	63	4.22	4.216
2.3	2.4	0.22755	0.07585	0	18.5	1.85	39	39	24	63	3.97	3.971
2.4	2.5	0.22755	0.07585	0	18.5	1.95	40	40	22	62	1.24	1.238
2.5	2.6	0.22755	0.07585	0	18.5	2.05	41	41	21	62	1.17	1.170
2.6	2.7	0.22755	0.07585	0	18.5	2.15	42	42	20	62	1.11	1.107
2.7	2.8	0.22755	0.07585	0	18.5	2.25	42	42	20	61	0.00	0.000
2.8	2.9	0.22755	0.07585	0	18.5	2.35	41	41	19	59	0.00	0.000
2.9	3.0	0.22755	0.07585	0	18.5	2.45	40	40	18	58	0.00	0.000
3.0	3.1	0.22755	0.07585	0	18.5	2.55	39	39	17	56	0.00	0.000
3.1	3.2	0.22755	0.07585	0	18.5	2.65	38	38	17	54	0.00	0.000
3.2	3.3	0.22755	0.07585	0	18.5	2.75	37	37	16	53	0.00	0.000
3.3	3.4	0.22755	0.07585	0	18.5	2.85	36	36	16	51	0.00	0.000
3.4	3.5	0.0752	0.025067	0	18.5	2.95	35	35	15	50	0.00	0.000
3.5	3.6	0.0752	0.025067	0	18.5	3.05	34	34	15	48	0.00	0.000
3.6	3.7	0.0752	0.025067	0	18.5	3.15	33	33	14	47	0.00	0.000

Project Number:	**Title: Squamish Shoppers Drug Mart**	Date:
Parish:	Station: Offset:	Elevation (ft):
Pile Shape: Round	Pile Diameter: 10	CPT No: **1**



Project Number: **Title:** Date:
Parish: Station: Offset: Elevation (ft):
Pile Shape: Round Pile Diameter: 12 CPT No:



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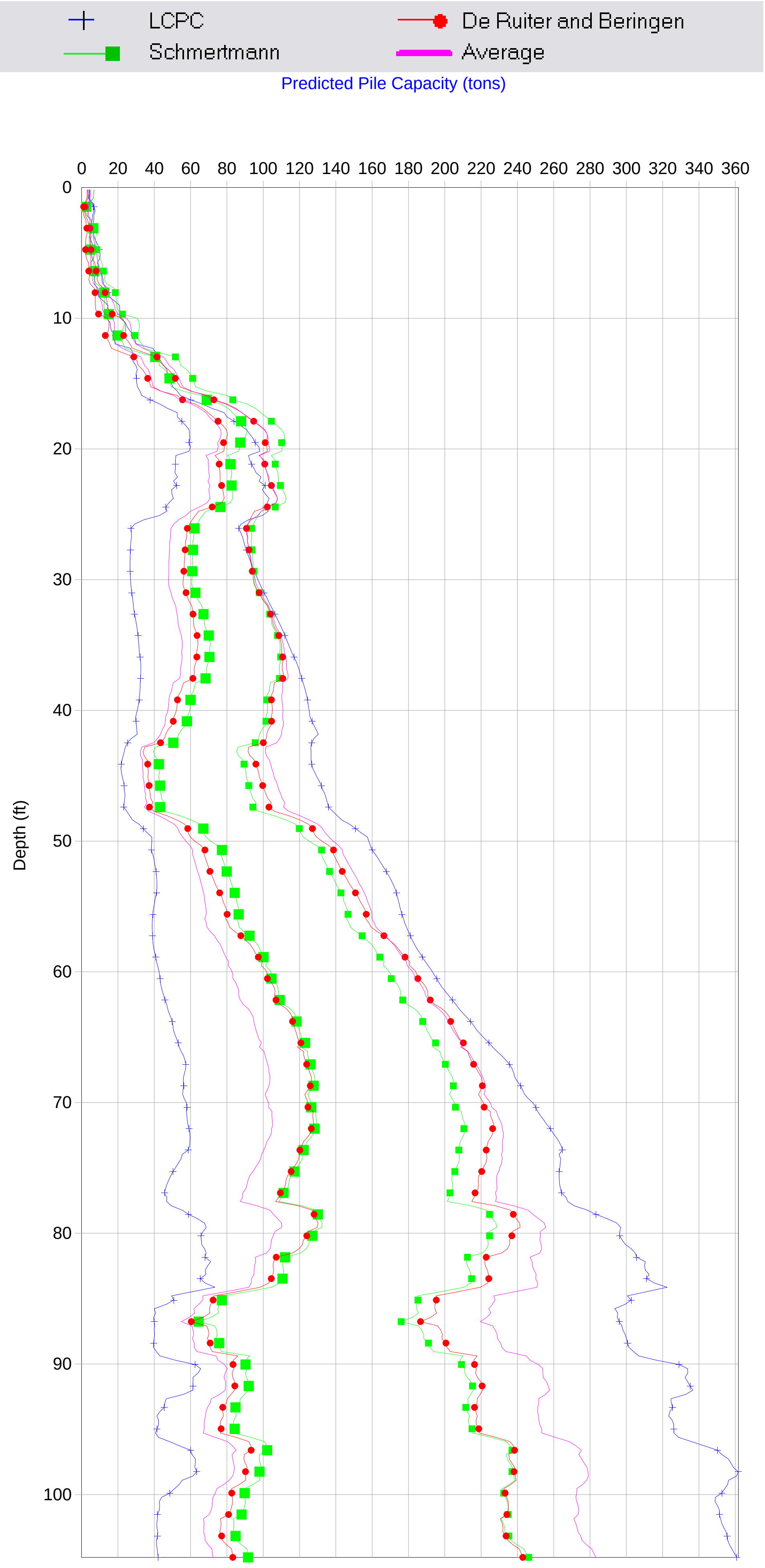
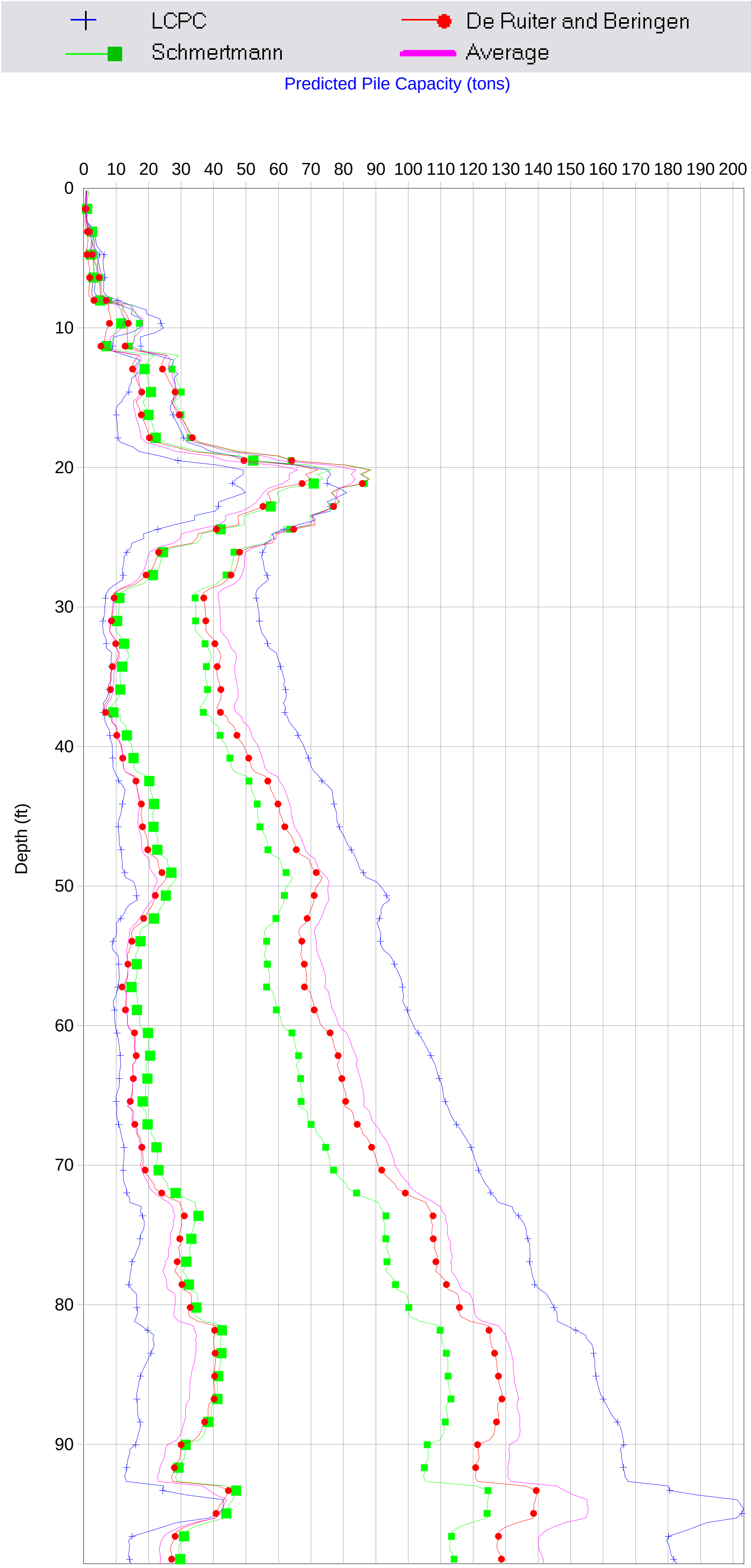
Parish: | Station: | Offset: | Elevation (ft):
Pile Shape: Round | Pile Diameter: 16 | CPT No: **1**



Project Number:	**Title: Squamish Shoppers Drug Mart**	Date:
Parish:	Station: Offset:	Elevation (ft):
Pile Shape: Round	Pile Diameter: 10	CPT No: **2**



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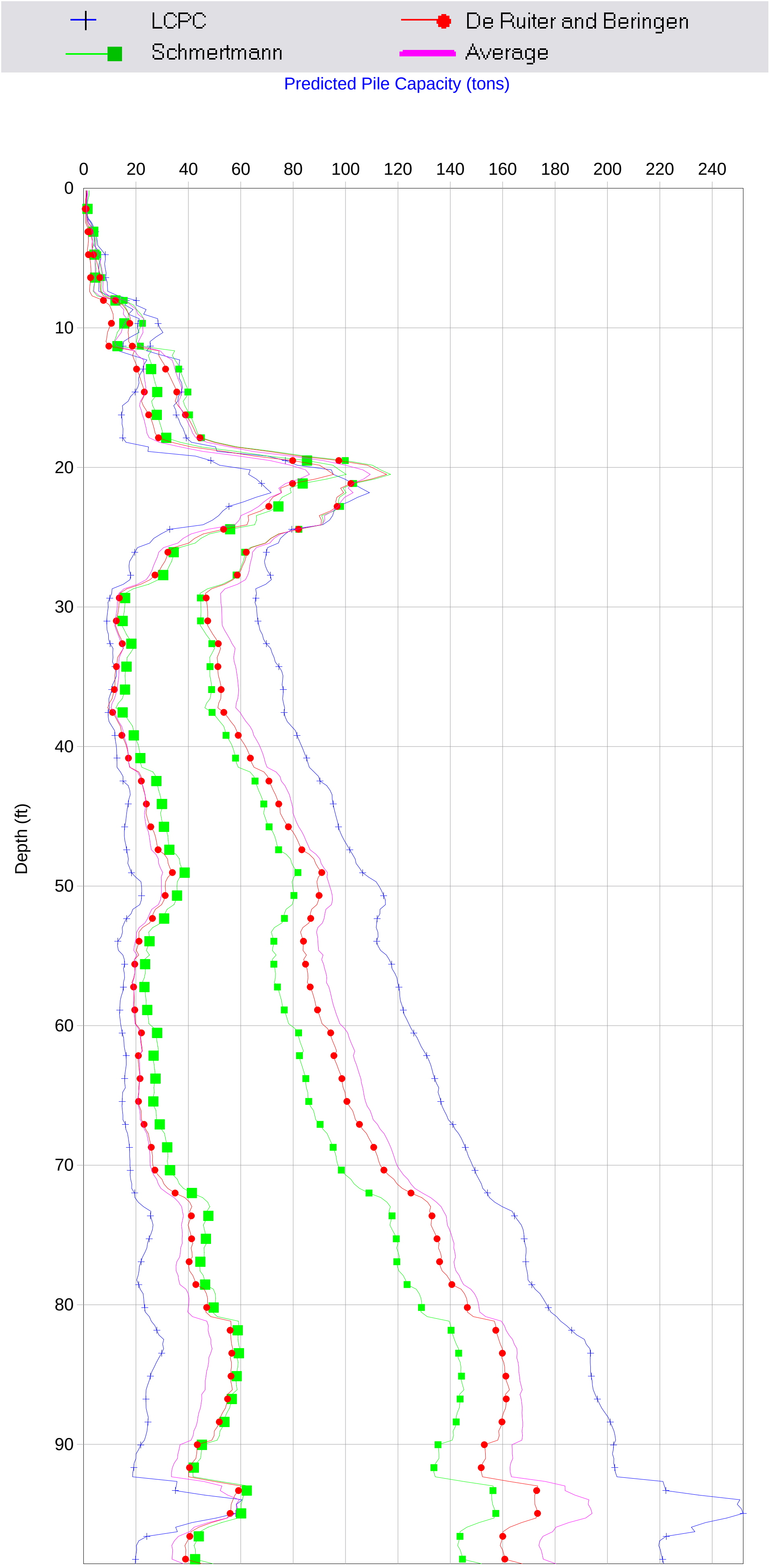
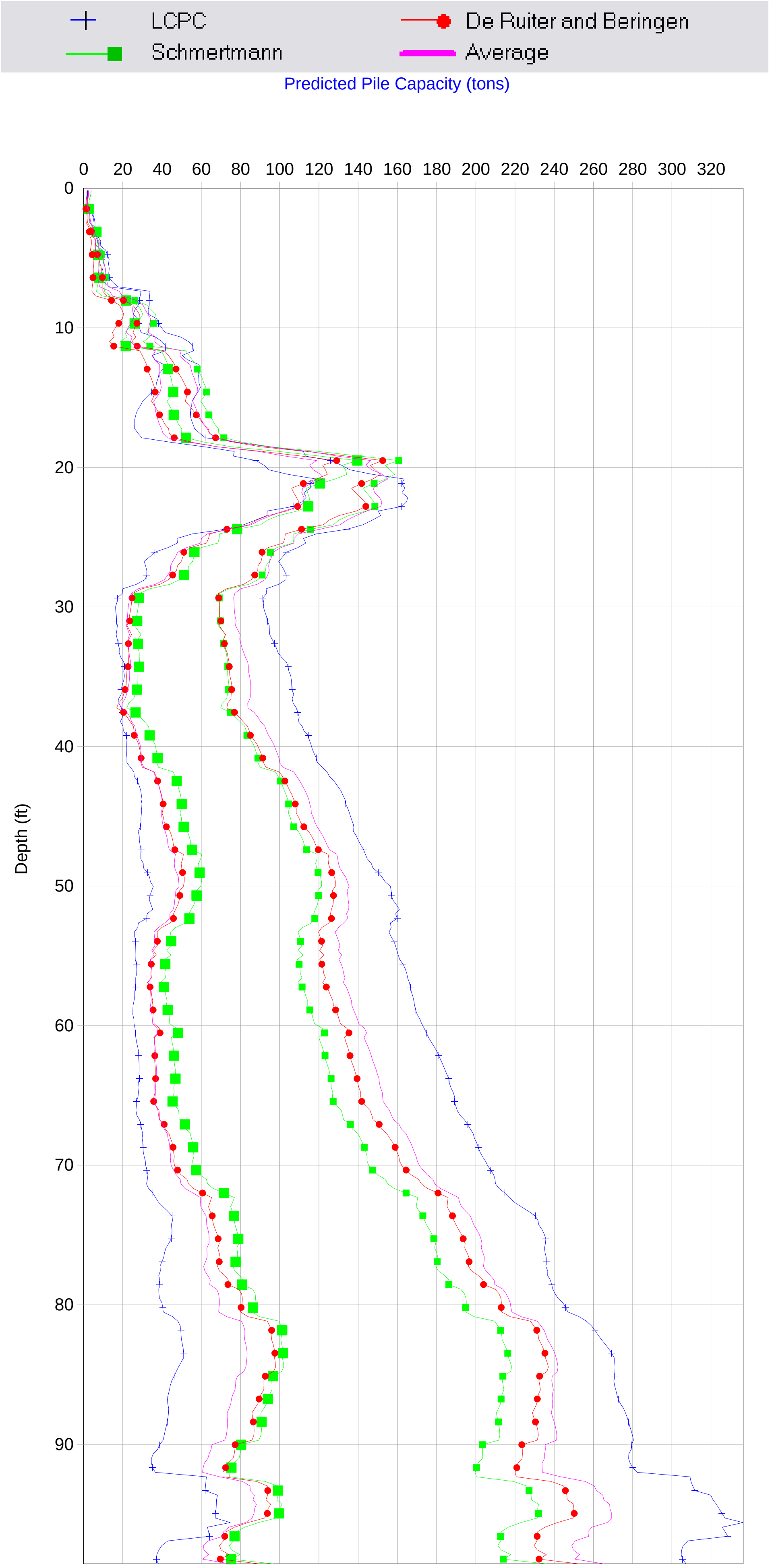
Parish: | Station: Offset: | Elevation (ft):
Pile Shape: Round | Pile Diameter: 12 | CPT No: **2**



Project Number: **Title: Squamish Shoppers Drug Mart** Date:

Parish: Station: Offset: Elevation (ft):

Pile Shape: Round Pile Diameter: 16 CPT No: **2**



APPENDIX H: SITE PHOTOGRAPHS

Figure I: Front view of the pre-existing building.

Figure II: Pile driving hammer driving pile in place.

Figure III: Pile driving rig.

Figure IV: Front view of the constructed building near completion, with compacted fill to grade in place.

Figure V: Front view of the completed building.

Figure VI: Side view of the completed building.