Trow

Geotechnical Investigation

Proposed Distribution Warehouse Centre
Boundary Road,
City of Cornwall, Ontario

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Table of Contents

Summary I

1.0 Introduction 1

2.0 Procedure 3

3.0 Site Description 4

4.0 Subsurface Soil and Groundwater Conditions 5

5.0 Grade Raise Restrictions 7

6.0 Foundation Considerations 8

6.1 Proposed Warehouse 8

6.2 Proposed Reclaim Building (Building B) 8

6.3 Additional Comments Applicable to Both Buildings 8

7.0 Floor Slab and Drainage Requirement 11

8.0 Lab Earth Pressure 13

9.0 Excavations 13

10.0 Pipe Bedding Requirements 14

11.0 Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes 15

12.0 Earthquake Considerations 16

13.0 Pavement Structure for Proposed Surface Parking Facility 17

14.0 Subsurface Concrete Requirements 19

15.0 Tree Planting 20

16.0 Future Addition (Phase II Construction) 21

16.1 Foundation Considerations 21

16.1.1 Pile Foundation 21

16.1.2 Caisson Foundations 23

Draft

17.0 Floor Slab Construction 26

18.0 General Comments 28

Tables

Table I: .... Recommended Pavement Structure Thicknesses
Table II: .... Chemical Test Results
Table III: .... Recommended SLS and ULS Loads on Steel Pipe and H Piles Driven to Refusal in Till or Bedrock
Table IV: .... Uplift Capacity of Piles
Table V: .... Uplift Load Capacity of Caissons

Figures

Figure No. 1: .... Borehole Location Plan
Figure Nos. 2 to 62: .... Borehole and Test Pit Logs
Figure No. 63, 64, 67 to 70: .... Grain Size Analyses
Figure No. 65, 66: .... One-Dimensional Consolidation Test Results

Appendix

Appendix A: .... Shear-wave Velocity Sounding

Geotechnical Investigation
Proposed Distribution Warehouse Centre
1651 Tenth Line Road, City of Cornwall, Ontario

Summary

A geotechnical investigation was undertaken at the site of the proposed Distribution Centre to be located on the property registered by the street address of 1651 Tenth Line Road in the City of Cornwall, Ontario. This work was authorized by [redacted]

It was initially proposed to locate Phase I of the Distribution Centre in the central and east portion of the site. However, the investigation revealed that normally to lightly overconsolidated silty clay is present in the east part of the site. This silty clay is highly compressible. Consequently the proposed structure and the floor slab would have to be founded on piles. At Trow's recommendation, the building was relocated to the west and central portion of the site. However, the relocated building was shifted approximately 90 m north of the previous location. As a result, only a few of the boreholes drilled at the site were within the area of the new building location. Therefore, additional boreholes (Boreholes 37 to 47) were drilled at the new building location. This geotechnical investigation report is based on the information obtained from all the boreholes drilled at the site.

Forty seven boreholes drilled at the site have revealed that the predominant soil in the west half of the building area is silty sand till although at certain locations, it is overlain by a surficial deposit of silty clay up to 3 m thick. The till is compact to dense. The predominant soil in the east half of the building area is silty clay which extends to a depth of up to 11.7 m. The silty clay is firm to stiff with an undrained shear strength of 14 to 86 kPa. It is normally to lightly overconsolidated and as a result is very compressible. The bedrock at the site is expected to be present at a depth of 12.8 m to 14 m below the existing ground surface. Available information indicates that the bedrock is likely to be shale of Rockcliffe Formation. The groundwater table at the site was measured at a depth ranging between 0.6 m to 2.8 m below the existing ground surface i.e. Elevation 51.2 m to 54.3 m.

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the Phase I of the relocated building on spread and strip footing foundations. The Serviceability Limit State (SLS) bearing pressure of the till is 190 kPa. The factored geotechnical resistance at Ultimate Limit State (ULS) may be taken as 285 kPa. Footings designed to bear on engineered fill pad compacted as per the recommendations stated in the report may be designed for SLS and ULS bearing values of 150 and 225 KPa respectively. A minimum of 1.5 m of earth cover should be provided to the exterior footings of a heated structure. Settlements of the structure designed for this SLS bearing pressure and properly

constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

The lowest level floor slab of the proposed Phase I building may be constructed as a slab-on-grade provided it is set on a well compacted bed of 19 mm clear stone at least 300 mm thick placed on natural undisturbed soil or engineered fill. Perimeter and underfloor drainage system should be provided for the proposed basementless structure, since the groundwater table at the site is high.

Excavations at the site are expected to extend a maximum depth of 3 m to 4 m below the existing ground surface. These excavations will extend through the surficial silty clay into the underlying silty sand till. They will be up to 3 m below the groundwater table. However, a 'base heave' type of failure of the excavations is not anticipated because of the dense nature of the till. Excavation sides are expected to be stable for the construction period when cut back at a slope of 1H:1V above the groundwater table and 2H:1V below the groundwater table. Excavations for the service trenches may be undertaken within the confines of a trench box which meets the requirement of Occupational Health and Safety Act, 1981.

The backfill against the subsurface walls, in footing trenches and in service trenches inside the building should be free draining granular material preferably conforming to Ontario Provincial Standard Specifications for Granular 'B' Type II. It should be compacted to 98 percent of standard Proctor maximum dry density. The backfill in service trenches outside the building areas and the fill to raise grades on access road and parking areas should be compacted to 95 percent Standard Proctor Maximum Dry Density. The soil to be excavated from the site will comprise of silty clay and silty sand till. Depending on the time of year and prevailing weather conditions, it may be possible to use the upper 1 m of the till from above the groundwater table as backfill in service trenches and footing trenches outside of the building areas. The silty clay and till may be used for grading purposes in the landscaped areas. It is therefore anticipated that majority of the material required for backfilling will have to be imported.

The site of the proposed Phase I construction has been classified as Class 'C' in accordance with the requirements of the Ontario Building Code, 2006. The site classification for Phase II addition is Class 'D'. This site classification may improve subsequent to consolidation of the silty clay.

The pavement structure thickness for the access roads and parking areas located in silty clay subgrade to be used by heavy traffic (trucks) should comprise of 130 mm of asphaltic concrete underlain by 150 mm of Granular 'A' base and 900 mm of Granular 'B', Type II. For parking areas with silty clay subgrade to be used by light automobile traffic, the pavement structure may consist of 65 mm of asphaltic concrete underlain by 150 mm of Granular 'A' and 450 mm of Granular 'B'. Heavy duty access and parking areas with silty sand till subgrade should be provided with 130 mm of asphaltic concrete underlain by 150 mm of Granular 'A' base and 600 mm of Granular' B' sub-base. Parking areas on silty sand till subgrade to be used by light automobile traffic may consist of 65 mm of asphaltic concrete underlain by 150 mm of Granular

'A' base and 300 mm of Granular 'B' sub-base. The grade of the asphalt should be as per the recommendation of the report.

General Use (GU) Portland cement may be used in the subsurface concrete at this site.

It is recommended that the area of the future addition (Phase II) should be surcharged to consolidate the silty clay present in this area to facilitate construction of the floor of the addition as slab-on-grade. The quantity of fill required to consolidate the silty clay such that spread and strip footing foundations may be feasible would be extremely large (to a height of 11 m approximately) and is expected to be uneconomical. It is therefore recommended that the proposed Phase II building should be founded on piles. Steel H and closed end pipe piles are feasible and may be designed according to the recommendations contained in the report. Similarly, cast-in-place concrete piles (caissons) are feasible although their installation may experience difficulties due to cobbles and boulders in the silty sand till and high groundwater table. Therefore, driven piles are to be preferred.

In order to facilitate a slab-on-grade floor construction, the area of the proposed addition should be surcharged with 2.2 m of fill (in addition to the fill required to raise the grade) to consolidate the silty clay. For this purpose, all the topsoil and any surficially softened soil should be removed. Engineered fill, preferably conforming to OPSS Granular 'B', Type II should be placed in 300 mm lift and compacted to 98 percent of standard Proctor maximum dry density to the subgrade level. Surcharge fill may then be placed in 500 mm lift thicknesses and nominally compacted. Consolidation settlements of the silty clay due to the surcharge load were estimated to vary from 75 mm to 750 mm depending on the thickness of the silty clay. The estimated time required to achieve 90 percent consolidation of the silty clay is expected to vary from 24 months to 30 months. It is recommended that additional samples of the silty clay should be collected and tested for consolidation in order to collect additional data on the engineering characteristics of the clay prior to surcharging the site.

The consolidation of the silty clay should be monitored by installation of settlement plates and pneumatic piezometers. Once the monitoring indicates that 90 percent consolidation of the silty clay is complete the surcharge load may be removed and construction proceed.

The above and other related considerations have been discussed in greater detail in the main body of this report.

1.0 Introduction

A geotechnical investigation was undertaken at the site of the proposed Distribution Warehouse Centre to be located on 1651 Tenth Line Road in the City of Cornwall, Ontario. This investigation was authorized by Mr. [redacted]

It is proposed to construct a Distribution Centre on the subject property which would comprise of an approximately 1.2 million square foot warehouse, Reclamation Building, container storage yard, storm water management pond, parking lots, associated roads and underground services. The warehouse is to be constructed in two phases. Phase I would comprise of 658,880 sq. ft. building. The Phase II addition would have a footprint of 540,000 sq. ft. The finished floor slab of the Distribution Centre will be set at Elevation 56.75 m. The exterior grade around the structure will be at Elevation 55.55 m approximately to facilitate construction of loading docks to be located along the north and south exterior walls.

It was originally proposed to locate Phase I of the Distribution Centre in the centre and east portion of the site and future extension (Phase II) in the west part of the site. However, the investigation revealed that the east of the site contains normally to lightly over-consolidated silty clay and as a result with the proposed grades, it was established that spread and strip footing foundations and a slab-on-grade floor will not be feasible. It was therefore recommended that Phase I of the Distribution Centre should be relocated in the west and central part of the site where the predominant soil is silty sand till although it is overlain by a shallow deposit of silty clay in some areas. In this portion of the site, spread and strip footings set on the silty sand till underlying the surficial silty clay will be feasible. Based on Trow recommendations, the Phase I of the Distribution Centre has been relocated to the west and central part of the site. However, it has also been shifted in the northerly direction by 90 m approximately.

The investigation was undertaken to:

(a) Establish the geotechnical and groundwater conditions at the locations of boreholes;

(b) Establish grade raise restrictions on the site;

(c) Discuss liquefaction potential of the on-site soils and classification of the site for seismic design in accordance with requirements of 2006 Ontario Building Code (OBC);

(d) Make recommendations on the most suitable type of foundations, founding depth and Serviceability and Ultimate Limit States bearing pressures of founding soil for the proposed addition;

(e) Provide comments on anticipated total and differential settlements;

(f) Discuss floor slab construction and subsurface drainage requirements;

(g) Provide recommendations for pipe bedding for the sewers;

(h) Recommend pavement structure thicknesses for the proposed access roads and surface parking areas;

(i) Provide recommendations for subsurface concrete requirements;

(j) Discuss design and construction of storm water management pond;

(k) Discuss excavation conditions for installation of the sewers and construction of the proposed structures including possible effects of groundwater during construction; and,

(l) Comment on backfilling requirements and suitability of the on-site soils for backfilling purposes.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2.0 Procedure

The fieldwork for the geotechnical investigation comprised the drilling of 47 boreholes between July 22, 2010 and August 27, 2010 using a CME-55 track mounted drill rig equipped with continuous flight hollow stem auger equipment to depths ranging between 2.3 and 14.5 m. In addition, fourteen test pits were excavated to a depth of 1.5 m each.

Prior to the commencement of the fieldwork, the borehole locations were established in the field and their locations cleared of any underground services by USL-1 Cable Locator. The locations and geodetic elevations of the boreholes were established by Speight, Van Nostrand and Gibson Limited, Ontario Land Surveyors. The locations of the boreholes are shown on Site Plan, Figure 1. Elevations of the boreholes refer to the geodetic datum.

Standard penetration tests were performed in all the boreholes at 0.75 to 1.5 m depth intervals and soil samples retrieved by split barrel sampler. In-situ field vane tests were performed in the silty clay to establish its undrained shear strength. Undisturbed shelby tube samples of the silty clay deposit were also obtained from selected depths from Borehole Nos. 18, 19 and 21.

All the soil samples collected were visually examined in the field, logged, stored in plastic bags and identified.

Water levels were measured in the open boreholes on completion of drilling. In addition, 19 mm diameter slotted standpipes were also installed in Borehole Nos. 2, 3, 4, 6, 7, 9, 13, 14, 16, 17, 21, 22, 23, 26, 32, 35, 38, 42 and 44 whereas 50 mm monitoring wells were installed in Borehole Nos. 10 and 11 for long term monitoring of the groundwater at the site. The installation configuration is documented on the respective borehole logs. All the boreholes were backfilled upon completion of the fieldwork.

All the soil samples were transported to the Trow laboratory in the City of Ottawa where they were examined by a geotechnical engineer and borehole logs prepared. Laboratory testing comprised of performing moisture content on all the soil samples and grain size analyses, unit weight, pH, sulphate content and electrical resistivity tests on selected soil samples. In addition, two one dimensional oedometer tests were performed on the silty clay samples obtained from the site.

3.0 Site Description

The site is located on the north side of Canadian National Railway tracks and Tenth Line East and West of the City of Cornwall Storm Water Management Pond in the City of Cornwall, Ontario. Industrial Park Drive extends approximately 475 m into the site from the east property boundary. The site is approximately 140 acres in area. Donihee drain crosses the site from the north mid property boundary in a southerly direction for a distance of 450 m approximately and thereafter runs in the easterly direction to drain into Grays Creek. Water level elevations in the Donihee drain vary from Elevation 55.3 m approximately at the north property boundary of the site to Elevation 52.7 m approximately close to the east property boundary. A drainage ditches runs in northerly direction from the south property boundary for a distance of 200 m approximately and thereafter traverses in an easterly direction to the Donihee Drain. Several ponds are located on the site. A fill stockpile measuring approximately 250 m by 125 m is located in the north east part of the site. The north central portion of the site is a wooded area. A number of ponds and former lagoons (which have since been backfilled) are located at the site. These areas may contain organic or surficially softened soils. The ground surface at the site slopes down from the west to east and from north to south. Ground surface elevations at the site vary from Elevation 61.8 m to Elevation 53.4 m approximately based on the spot elevations shown on the topographical survey plan of the site.

4.0 Subsurface Soil and Groundwater Conditions

A detailed description of the geotechnical conditions encountered in the forty seven (47) boreholes and fourteen (14) test pits is given on the borehole/test pit logs, Figure Nos. 2 to 62 inclusive. The borehole and test pit logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Notes on Sample Descriptions" preceding the borehole logs form an integral part of this report and should be read in conjunction with this report.

A review of Figures Nos. 2 to 62 inclusive indicates the following subsurface conditions.

The site contains 50 mm to 410 mm of topsoil.

Beneath the topsoil, silty clay was encountered in all the boreholes except Boreholes 6, 8, 12, 13, 22, 24, 27, 28, 29, 33 to 35, 37 to 43 and 45. The silty clay extends to the entire depth investigated in Boreholes 10, 11, 23, 25, 26, 30 and 31 i.e. 3.0 m to 7.6 m depth (Elevation 46.1 m to 51.6 m) and to a depth of 0.5 m to 14.7 m depth in the other boreholes (elevation 42.1 m to 56.4 m). The silty clay is generally firm to stiff as indicated by its undrained shear strength of 14 kPa to 86 kPa although a couple of higher shear strength values were also recorded in surficial silty clay. The natural moisture content and unit weight of silty clay are 20 to 78 percent and 16.1 to 19.2 kN/m^3.

The results of two grain size analyses performed on the silty clay samples are given on Figures 63 and 64 inclusive. A review of these figures indicates that the silty clay comprises of 58 to 60 percent clay, 38 to 41 percent silt, and 1 to 2 percent sand.

The results of the two, one dimensional oedometer tests performed on the silty clay samples from Boreholes 19 to 21 are given on Figures 65 and 66 . These figures indicate that the silty clay at the site is reconsolidated to 62 kPa to 66 kPa. The overconsolidation pressure of the silty clay was established as 19 to 35 kPa. The compression and re-compression indices of the silty clay vary from 1.0 to 1.06 and 0.059 to 0.065 respectively.

Beneath the silty clay in Boreholes 1 to 5, 7, 9 and 14 to 21, 32, 36, 44, 46 and 47 and the topsoil in Boreholes 6, 8, 12, 13, 22, 24, 27, 28 and 29, 32 to 35, 37 to 43 and 45 silty sand to silty sand till extends to the entire depth investigated in all the boreholes i.e. 2.1 m to 14.5 m, Elevation 39.8 m to 54.9 m. The silty sand till contains frequent cobbles and boulders. It is generally compact to very dense although some localized loose layers are also present. The natural moisture content and unit weight of the till varies from 6 to 20 percent and 22.6 to 23.8 kN/m^3 respectively.

The results of four grain size analyses are given on Figures 67 to 70. A review of these figures indicates that the till comprises of 6 to 18 percent clay, 30 to 39 percent silt, 31 to 40 percent sand and 12 to 30 percent gravel.

Refusal to augering was met at 2.1 m to 14.5 m depth (Elevation 39.8 m to 53.9 m) in Boreholes 2 to 3, 14, 16, 17, 21, 22, 34, 38, 39, 41, 42 and 44 to 46. The refusal at shallow depth i.e. 2.1 m to 5.9 m is likely to have been met on cobbles and boulders in the till. The refusal at a greater depth (i.e. 12.1 m to 14.5 m depth) was likely met on the bedrock surface. However, additional drilling would be required to establish the exact depth of the bedrock at the site. Available information indicates that the bedrock at the site is likely grey green shale of Rockcliffe Formation which is interbedded with sandstone.

Water level observations made in the boreholes during the fieldwork and in standpipes installed in some of the boreholes subsequent to completion of the fieldwork indicate that the groundwater table was measured at a depth of 0.6 m to 2.8 m (Elevation 51.3 m to 56.7 m).

Water levels were made in the exploratory boreholes at the times and under the conditions stated in the scope of services. These data were reviewed and Trow's interpretation of them discussed in the text of the report. Note that fluctuations in the level of the groundwater may occur due to seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

5.0 Grade Raise Restrictions

57.20m

It is proposed to set the floor slab of the Distribution Centre at Elevation 56.75 m. The existing ground surface elevations in the vicinity of the proposed new location vary from Elevation 55 m to 57.2 m approximately. Therefore, the proposed construction will result up to 1.75 m of cut and fill operation.

The geotechnical investigation has revealed that the predominant natural soil in the area of Phase I construction is expected to be silty sand till although a limited depth of surficial silty clay (1.0 m to 1.5 m approximately) was encountered in some of the boreholes. It is therefore considered that the proposed grade raise in the area of Phase I construction is feasible.

6.0 Foundation Considerations

6.1 Proposed Warehouse

The geotechnical investigation has revealed that it is feasible to construct the proposed Phase I Warehouse on spread and strip footings set on the silty sand till.

Footings set in the silty sand till at a depth of 1.5 m approximately below the existing ground surface or on the silty sand till underlying the silty clay may be designed for Serviceability Limit State (SLS) bearing pressure of 190 kPa. The factored geotechnical resistance at Ultimate Limit State (ULS) may be assumed as 285 kPa. It is noted that because of the presence of surficial silty clay in some areas, some of the footings may have to be founded at a greater depth than 1.5 m.

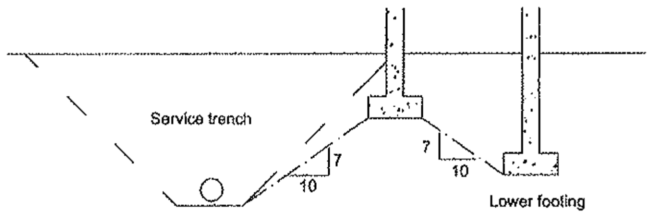
Also, a number of drainage ditches, ponds and former lagoons which have since been backfilled are located on the property. These areas may contain organic soils and surficially softened soils. All the organic and surficially softened soils must be excavated from the building areas, access roads and parking lots and backfilled with engineered fill.

6.2 Proposed Reclaim Building (Building B)

The proposed new location of the Reclaim Building B is located in the vicinity of Boreholes 45 to 47. Available information indicates that this area is likely to contain a surficial deposit of silty clay approximately 0 m to 2.5 m thick. The silty clay is underlain by compact to very dense silty sand till. It is therefore considered that this structure may also be founded on spread and strip footings set in the silty sand till and designed for SLS bearing pressure of 190 kPa. The factored geotechnical resistance at Ultimate Limit State may be assumed as 285 kPa. It is noted that the footings in the vicinity of Borehole 47 would have to be founded at 3 m depth approximately below the existing ground surface. Alternatively, engineered fill may be used to raise the grade form the top of the silty sand till to the design underside of the footings as described in section 6.4 below. .

6.3 Additional Comments Applicable to Both Buildings

Footings which are to be placed at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing, as indicated on the following sketch:



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

The lower footings should be constructed before the upper footings to prevent the latter from being undermined during subsequent construction.

A minimum of 1.5 m of earth cover should be provided to the exterior footings of all heated structures to protect them from damage due to frost penetration. The earth cover to the footings should be increased to 2.1 m if the structures are not heated and snow will not be removed from the vicinity of the structures. The earth cover should be increased to 2.4 m if snow will be removed from the vicinity of the structures.

6.4 Engineered Fill

In areas where it is required to found at a greater depth than 1.5 m due to the presence of silty clay, engineered fill may be used to raise the grading from the top of the silty sand till to the proposed underside of the footings. The engineered fill should comprise of material conforming to OPSS 1010 Granular B Type II and should be placed in 300 mm lifts and each lift compacted to 100% of the Standard Proctor Dry Density. Footings designed to bear on engineered fill pad compacted as per the recommendations stated above may be designed for SLS and ULS bearing values of 150 and 225 KPa respectively .

It should be noted that due to the high groundwater table at the site, continuous pumping of the excavation (i.e. 24 hrs a day, 7 days a week) may be required during placement and compaction of the engineered fill.

6.5 General Comments

All the footing beds should be inspected by a geotechnical engineer to ensure that the founding surfaces are capable of supporting the design bearing pressure and that the footing beds have been properly prepared.

The recommended bearing capacities have been calculated by Trow Associates Inc. from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when foundation construction is underway. The interpretation between boreholes, and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

Settlements of footings designed for the SLS bearing pressure recommended and properly constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

7.0 Floor Slab and Drainage Requirement

The floor slabs of the proposed Warehouse and Reclaim Building B may be constructed as slab-on-grade prepared as described below.

As part of the preparation of the floor slab construction, all the topsoil and any fill or disturbed soils should be removed and replaced with well compacted engineered fill. The exposed native subgrade should be examined by a geotechnical engineer. Following approval, OPSS Granular 'B' Type II fill to raise the grade should be placed in 300 mm lift thickness and each lift compacted to 98 percent of the Standard Proctor Maximum Dry Density. The placement and compaction of the engineered fill can in this way be undertaken to the underside of the subgrade level. The floor slabs may be set on beds of 19 mm clear crushed stone at least 300 mm thick placed on the engineered fill. The clear stone will prevent the capillary rise of moisture from the subsoil to the floor slab. Alternatively, the clear stone may be replaced with Granular 'A' fill compacted to 98 percent of standard Proctor maximum dry density and polyethylene sheeting.

The engineered fill should be placed under the full time supervision of a geotechnician working under the direction of a geotechnical engineer. In-place density tests should be undertaken on each lift of the engineered fill to ensure that it is properly compacted prior to placement of subsequent lift.

The finished floor slab of the Distribution Centre will be set at Elevation 56.75 m and the investigation has revealed the groundwater table to be depth of 0.6 m to 2.8 m (Elevation 51.3 m to 56.7 m). It is therefore recommended that perimeter and under floor drains should be provided for the proposed basementless buildings due to the prevailing high groundwater table level at the site in the area of the Phase I construction.

The finished floor of the proposed buildings should be at least 150 mm higher than the finished exterior grade and the exterior grades should be sloped away from the building at an inclination of 1 to 2 percent to prevent surface ponding close to the exterior walls.

8.0 Lateral Earth Pressure

Since the exterior grade will be 1.2 m lower than the finished grade, the subsurface walls of the proposed structure will be subjected to lateral static earth pressure as well as lateral dynamic earth pressure during a seismic event. The lateral static earth pressure that the subsurface walls would be subjected to may be computed from the following equation:

$$P_A = \tfrac{1}{2} K_a \gamma H^2$$

where
- P_A = lateral active force kN
- K_a = active earth pressure coefficient = 0.4
- γ = unit weight of backfill = 22 kN/m^3
- H = height of wall, (m)

The lateral force due to seismic loading may be computed from the equation given below:

$$\Delta P_E = 0.1 \gamma H^2$$

where
- ΔP_E = resultant force due to seismic activity acts at 0.6 H from the footing base
- γ = unit weight of backfill = 22 kN/m^3
- H = height of wall, (m)

9.0 Excavations

Excavations for the proposed buildings are expected to extend to a maximum depth of 1 m to 3 m whereas excavations for the installation of underground services are expected to extend somewhat deeper i.e. to approximately 4.0 m below the existing ground surface. These excavations will be undertaken through the upper silty clay to the underlying silty sand till and are expected to be 1 m to 3 m below the groundwater table. However, a 'base heave' type of failure of the excavations is not anticipated due to the dense nature of the till.

All excavations should meet the requirements of the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects, Ontario Regulations 213/91. Open cut excavations may be i.e. cut back at a slope of 1H to 1V above the ground water level. Below the groundwater table, the excavation sides are expected to slough and may eventually stabilize at a slope of approximately 2H:1V. It is noted that a number of drainage ditches, ponds and lagoons (which have been backfilled) are located on the property. These areas may contain topsoil, organic soils and surficial softened soils to a greater depth than observed elsewhere on the site. These organic and surficial softened soils would have to be sub-excavated to the underlying natural silty clay or till.

If space restrictions prevent open cut excavation, the services may be installed within the confines of a prefabricated support system which is designed and installed in accordance with the Ontario Occupational Health and Safety Act and Regulations for Construction Projects, Ontario Regulations 213/91 i.e. a trench box.

Seepage of the surface and subsurface water into the excavations should be anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps. Continuous pumping of the groundwater will be required during the placement and compaction of the engineered fill in areas where removal of the clay or soft/unsuitable soils will be required.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

Although this investigation has estimated the groundwater levels, at the time of the field work, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems. It is noted that a permit to take water may be required from Ontario Ministry of the Environment if the quantity of water to be pumped from the site exceeds 50,000 litres per day. This process may take 3 to 4 months.

10.0 Pipe Bedding Requirements

It is anticipated that the services will be founded in the silty sand till.

It is recommended that the bedding for the storm and sanitary sewers including material specifications, thickness of cover material and compaction requirements conform to the Ontario Provincial Standard Specification November 2005, Drawing Nos. 802.030 to 802.032.

In areas where paved surfaces are considered over services trenches, it is recommended that the trench backfill material within 1.8 m of the finished grade, should match the on-site soils to minimize differential frost heaving of the subgrade. The trench backfill should be placed in 300 mm lift thickness and each lift should be compacted to 95 percent of the SPMDD. If granular fill is used in the trenches, frost taper in accordance with OPSD 205.010 and 205.040 should be provided to minimize sharp distortions of the pavement.

11.0 Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The backfill against the subsurface walls and in footing trenches and in service trenches inside the building should be free draining granular material conforming to the Ontario Provincial Standard Specifications (OPSS) for Granular 'B' Type II. The backfill in service trenches exterior to the building and the fill to be placed in parking areas should be compactible i.e. free of organics and debris and with a moisture content which is within 2 percent of the optimum value. Interior backfill should be compacted to 98 percent of the Standard Proctor Maximum Dry Density whereas the exterior backfill may be compacted to 95 percent of Standard Proctor Maximum Dry Density.

The material to be excavated from the site will comprise of silty clay and silty sand till. With the exception of the upper 1 to 2 m of the silty sand till, the silty sand till below the groundwater table and the silty clay are expected to be wet for adequate compaction and may be used for general landscaping purposes. The silty sand till from above the groundwater table is expected to be compactible and may be used to backfill service trenches outside the building and in parking lots. The silty sand till from below the groundwater table may be compactable if it can be spread in the sun to dry and provided that its moisture content be maintained within +/- 2 percent of its optimum value.

The fill should be placed in 300 mm lifts and each lift should be compacted to the specified density.

The silty sand and silty clay soils are considered susceptible to moisture absorption due to inclement weather condition, such as precipitation. Therefore these soils should be protected from the elements if they are stockpiled on-site.

Any material required for backfilling purposes which has to be imported should preferably conform to the OPSS 1010 requirements for Granular B Type II.

12.0 Earthquake Considerations

The subsoil and groundwater information at this site has been examined in relation to Section 4.1.8.4 of the OBC 2006. The subsoil in the area of the proposed building generally consisted of, silty clay underlain by silty sand till or silty sand till. Shear wave velocity of the upper 30 m of the overburden/bedrock was measured by Geophysics GPR International Inc. using the Multi-channel Analysis and Surface Waves and the Micro-tremors Array Method. Three geophysical soundings were undertaken approximately 350 m apart located in the west, central and east part of the site. The proposed new location of the building is in the vicinity of west and central soundings. A review of the geophysical survey report indicates that the average shear wave velocity in the upper 30 m of the soil/bedrock varies from 767.7 m/s in the west part of the site to 430.2 m/s in the central part of the site. Therefore, the site has been classified as Class 'C' for seismic site response in accordance with Table 4.1.8.4A of the Ontario Building Code, 2006.

The site of the future addition (Phase II) is located in an area with current site classification of Class 'D'. It is noted that this classification may improve subsequent to consolidation of the silty clay. However, this would have to be verified by performing additional shear wave velocity measurements subsequent to completion of consolidation of the silty clay. The results are included in Appendix A.

The investigation has also revealed that the on-site soils are not subject to liquefaction during a seismic event.

13.0 Pavement Structure for Proposed Surface Parking Facility

Pavement structure thicknesses for the proposed light and heavy duty traffic were computed for the silty clay and silty sand till subgrades and are shown on Table No. I. It is noted that the pavement structure thicknesses required in areas where silty sand will form the subgrade may be somewhat more than recommended for the silty sand till. This would depend on the gradation of the fill, its degree of compaction etc. and would have to be determined during construction. The recommended pavement structure is based upon an estimate of the subgrade soil properties determined from visual examination, textural classification of the soil samples and functional design life of eight to ten years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table No. I: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Light Duty Parking		Access Roads and Heavy Duty Parking	
		Clay Subgrade	Till Subgrade	Clay Subgrade	Till Subgrade
Asphaltic Concrete	92 to 97 % MRD	65 (SC) (PG 58-34)	65 (SC) (PG 58-34)	50 mm SC 80 mm BC (PG 70-34)	50 mm SC 80 mm BC (PG 70-34)
OPSS Granular 'A' Base (crushed limestone)	100% SPMDD*	150	150	150	150
OPSS Granular 'B' Sub-base, Type II	100% SPMDD*	450	300	900	600

1. MRD Denotes Maximum Relative Density , ASTM D2041
2. SPMDD denotes standard Proctor maximum dry density, ASTM, D-698
3. Any subgrade fill must be compacted to 95% SPMDD for at least the upper 300 mm
4. SC Denotes Surface course asphalt and should comprise of OPSS 1151 12.5 mm Mix
5. BC Denotes Base Course asphalt and should comprise of OPSS 1151 19 mm Mix
6. Category C is recommended for Light Duty traffic and Category D for Heavy Duty Truck Traffic

The foregoing design assumes that construction is carried out during dry periods and that the subgrade is stable under the load of construction equipment. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of granular material and/or geotextile may be required.

Additional comments on the construction of parking areas are as follows:

1. As part of the subgrade preparation, the proposed parking areas should be sub-excavated to subgrade level. The subgrade should be properly shaped, crowned, then proofrolled with a heavy vibratory roller (sheepfoot) in the full-time presence of a

representative of this office. Any soft or spongy subgrade areas detected should be subexcavated and properly replaced with suitable approved backfill compacted to 95% SPMDD (ASTM D698).

2. The long term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. The need for adequate drainage cannot be over-emphasized. Sub-drains must be installed in the proposed parking area at low points and should be continuous between catch basins to intercept excess surface and subsurface moisture and to prevent subgrade softening. This will ensure no water collects in the granular course, which could result in pavement failure during the spring thaw. The location and extent of subdrainage required within the paved areas should be reviewed by this office in conjunction with the proposed site grading.

3. To minimize the problems of differential movement between the pavement and catchbasins/manhole due to frost action, the backfill around the structures should consist of free-draining granular preferably conforming to OPSS Granular "B", Type II material. In addition, the catch basins should be perforated just above the invert level of the sewer. The perforations should be surrounded with filter cloth and clear stone.

4. The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavorable weather.

5. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

6. Relatively weaker subgrade may develop over service trenches at subgrade level. These areas may require the use of trenches/courser sub-base material and the use of a geo-textile at the subgrade level.

7. The granular materials used for pavement construction should conform to Ontario Provincial Standard Specifications (OPSS) for Granular 'A' and Granular 'B', Type II and should be compacted to 100 percent of the standard Proctor maximum dry density. The asphaltic concrete used and its placement should meet OPSS requirements. It should be compacted to 97 percent of the Marshall Density.

It is recommended that Trow be retained to review the final pavement structure design and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

14.0 Subsurface Concrete Requirements

Chemical tests limited to pH and sulphate content were undertaken on seven (7) selected soil samples and the results are shown on Table No. II.

Table No. II : Chemical Test Results					
Borehole No	Depth (m)	pH	Chloride %	Sulphate %	XX Ohm-cm
1	1.5 – 2.1	7.9	≤0.002	0.04	3130
5	1.5 – 2.1	8.3	≤0.002	0.08	7690
13	0.8 – 1.4	8.1	≤0.002	0.04	4000
16	1.5 – 2.1	8.4	≤0.002	0.03	2940
17	5.8 – 6.4	8.3	≤0.002	0.09	1890
21	6.2 – 6.7	8.6	≤0.002	0.05	1960
26	1.5 -2.1	8.1	≤0.002	0.03	3450

Electrical resistivity measurements indicate that the soils on site are mildly to moderately corrosive to buried steel.

The test results indicate slightly alkaline soils with sulphate content of less than 0.1 percent. This concentration of sulphates in the soil would have a negligible potential of sulphate attack on subsurface concrete. In such cases, National Standards of Canada, CAN/CSA - A23.1-04 permits the use of General Use (GU) Portland cement in the concrete. The concrete should, however, be dense, well compacted and cured.

15.0 Tree Planting

The clay in the Ottawa area is prone to shrinkage on drying. This process is largely not reversible. Therefore settlement and cracking of the structures can result if trees are planted too close to the residences. During dry seasons, the tree roots suck moisture from the clay thereby resulting in the clay drying and shrinking.

Published literature indicates that a good working rule is to preferably plant a tree no nearer a building on shrinkable clay than the eventual height to which the tree may be expected to grow. Obviously, evergreens are better as they have a lower water demand than deciduous trees.

In order to assist you in landscaping on the property, following is a list of more common trees in order of decreasing water demand.

- Poplar
- Aspen
- Elm
- Birch
- Beech
- Larch
- Fir
- Alder
- Maple
- Ash
- Oak
- Spruce
- Pine

It is recommended that for more information, an arborist should be consulted.

16.0 Future Addition (Phase II Construction)

16.1 Foundation Considerations

The area of the proposed future warehouse expansion is underlain by normally to lightly overconsolidated silty clay up to 11.5 m thick. The silty clay is therefore susceptible to large settlements if loaded. The proposed construction would result in placement of up to 3 m of fill under the floor slab of the building in addition to the anticipated floor loading of 25 kPa. Therefore, the silty clay would be subjected to a load of 90 kPa approximately excluding the footing load. If the structure is to be founded on spread and strip footing designed for SLS bearing pressure of 100 kPa, the silty clay would be subjected to a total load of 190 kPa. Consolidation of the silty clay to enable construction of the proposed addition on spread and strip footings designed for SLS bearing pressure of 100 kPa will require surcharging the site with 8.0 m of fill. Also, the fill will have to stage loaded otherwise failure of the silty clay may result. This process is expected to be expensive and time consuming. It is therefore recommended that the proposed addition should be founded on piles and that the site should be surcharged to consolidate the silty clay so that a slab-on-grade floor would be feasible.

16.1.1 Pile Foundation

It is recommended that the proposed Phase II addition to the warehouse should be founded on pile foundations driven to practical refusal in the till or bedrock. Closed end pipe or steel H piles driven to practical refusal in the till or the bedrock may be designed for the following loads in compression (Table II) and in tension (Table III).

Table III - Recommended SLS and ULS Loads on Steel Pipe and H Piles Driven to Refusal in Till or Bedrock

Type of Pile	Size	Piles to refusal in till		Piles to refusal on bedrock
		SLS Load kN	ULS load kN	SLS/ULS Load (kN)
Pipe	245 mm OD by 12 mm wall	700	875	1070
	324 mm OD by 12 mm wall	940	1175	1380
Steel H	HP 310 x 79	800	900	900
	HP 310 x 110	1130	1270	1270
	HP 310 x 125	1270	1430	1430

Table IV – Uplift Capacity of Piles			
Type of Pile	Size	Uplift Capacity (kN)	
		SLS	ULS
Pipe	245 mm x 12mm	40	50
	324 mm x 12 mm	55	65
Steel H	HP 310 x 79	65	80
	HP 310 x 110	65	80
	HP 310 x 125	65	80

The piles are expected to meet refusal on bedrock at 13 m to 15.0 m below the existing ground surface. It is possible that some of the piles may meet refusal at a higher level on cobbles and boulders present in the till. The Serviceability Limit State load carrying capacity of these piles may be lower and must be confirmed by testing these piles with pile driving analyser. It is therefore possible that some additional piles may have to be driven at the site to supplement piles that 'hang up' in the till (a 10 percent allowance for additional piles should be included in the contract as a provisional item.)

It has been assumed that the silty clay will be preconsolidated prior to commencement of construction and that the piles will not be subjected to negative skin friction. Placement of fill on the site to raise the grade would result in consolidation settlements of the clay and down-drag forces on the piles. Appropriate allowance for negative skin friction would have to be made to the Serviceability Limit State loads indicated above if the silty clay is not fully consolidated prior to commencement of construction. This office should be contacted to compute the negative skin friction if necessary.

It is anticipated that some piles may be driven in a group. Since the piles would be driven to practical refusal in the bedrock, the total bearing value of a group of piles may be taken as equal to the product of the bearing value of an individual pile multiplied by the number of piles in the group. The uplift capacity of the piles, driven in a group is lesser of the following two values:

(1) Sum of uplift resistance of individual piles in the group; and,

(2) The sum of shearing resistance mobilized on the surface perimeter of the group plus the effective weight of soil and piles enclosed in this perimeter.

The proper evaluation of the lateral performance of piles installed in a group requires an approach that accounts for the soil non-linearity, especially near the ground surface. Computer programs are available to facilitate the analysis and design of laterally loaded pile groups e.g. GROUP (Reese and Wong, 1996), PYLATG (El Naggar and Mustafa, 2001). Computation of lateral capacity of piles in a group is beyond the scope of this investigation.

In order to achieve the capacity given previously, the pile driving hammer must seat the pile into bedrock without overstressing the pile material. For guidance purposes, it is estimated that a hammer with rated energy of 54 kJ to 70 kJ (40,000 to 52,000 ft lbs) per blow would be required to drive the piles to practical refusal in the bedrock. Practical refusal is considered to have been achieved at a set of 5 blows for 6 mm or less of pile penetration. However, the driving criteria for a particular hammer-pile system must be established at the beginning of the project. This may be undertaken with the Pile Driving Analyzer.

The site contains sand and gravel and till with cobbles and boulders. It is therefore recommended that the piles should be equipped with a driving shoe, such as Titus or equivalent to protect them from damage during driving.

A number of test piles should be monitored with the Pile Driving Analyzer during the initial driving and restriking at the beginning of the project. This monitoring will allow for the evaluation of transferred energy into the pile from the hammer, determination of driving criteria and an evaluation of the ultimate bearing capacity of the piles. Depending on the results of the pile driving analysis, the pile capacity may have to be proven by at least one pile load test for each pile type before production piling begins. If necessary, the pile load test should be performed in accordance with ASTM D 1143.

Closed end pipe piles tend to displace a relatively large volume of soil. When driven in a cluster or group, they may tend to jack up the adjacent piles in the group. Consequently, the elevation of the top of each pile in a group should be monitored immediately after driving and after all the piles in the group have been driven. This is to ensure that the piles are not heaving. Any piles found to heave more than 3 mm should be retapped.

Piles driven at the site in the shale bedrock may be subject to relaxation i.e. loss of set with time. It is therefore recommended that all the piles should be retapped a minimum of 24 hours after initially driving and at 24 hour intervals thereafter until it can be proven that relaxation is no longer a problem.

The installation of the piles at the site should be monitored on a full time basis by a geotechnician working under the direction and supervision of a qualified geotechnical engineer to verify that the piles are driven in accordance with the project specifications.

Settlements induced by the above recommended pile loads are expected to be less than normally tolerated limits of 25 mm total and 19 mm differential movements.

16.1.2 Caisson Foundations

An alternative to driven pile foundations is to found the proposed structures on caissons socketed in the sound shale bedrock. However their installation may experience difficulties due to presence of boulders in the till and high groundwater table at the site. The following recommendations are provided for the design and installations of caisson foundations;

The founding level of the caissons in the vicinity of the proposed addition is expected to vary from Elevation 39 m to 41 m. Caissons socketed at least 1.0 m depth into the sound bedrock below any fractured/weathered zone may be designed for a Ultimate Limit State (ULS) bearing pressure of 1.0 MPa (10 tsf) to 3.0 MPa (30 tsf) depending on the amount of inspection and testing undertaken during construction. If SLS pressure of 1.0 MPa (10 tsf) is used, only visual inspection of the caisson bases would be required to ensure that the bases have been satisfactorily prepared. The minimum diameter of the caissons would have to be 0.9 m to allow for access by cleaning and inspection personnel.

A higher ULS bearing value of 2.4 MPa (24 tsf) is available for caissons founded on bedrock provided additional inspection and testing is performed. The additional work would require extending 5 to 6 randomly selected caissons across the site to a depth equal to twice the diameter of the caisson below the anticipated general founding level. The bedrock would be examined to establish the sound rock level and to ascertain that it is free from any weakened features such as clay seams. In addition, all the caissons would have to be star drilled to a depth below the founding level equal to the diameter of the caisson. These holes would be probed to ensure that there are no soft seams or voids in the bedrock immediately below the founding level. Visual inspection of all the caissons would be required to ensure that the founding level is in sound bedrock and that all loose and disturbed material has been removed from the caissons.

ULS bearing pressure values up to 3.0 MPa (30 tsf) are possibly available at the site but would require confirmation with a load test.

The uplift capacity of 0.9 m diameter and 2.0 m diameter caissons was computed. The following assumptions were made:

(1) The caissons will be socketed at least 1.0 m into sound shale bedrock;

(2) The resistance to uplift of the caissons will be provided by bond between the concrete and the shale bedrock and friction between the soil and concrete.

The computed values have been listed on Table V.

Table V – Uplift Load Capacity of Caissons			
Caisson Diameter (m)	Socket Depth Into Sound Rock (m)	Uplift Capacity (kN)	
		SLS	ULS
0.9	1.0	1400	1850
2.0	1.0	3200	4265

It is noted that if the above assumption are not correct, this office should be contacted for additional recommendations.

Installation of the caissons will require the use of at least one liner in order to minimize soil losses when penetrating the water bearing fill and the underlying till. The liner should be driven to the shale bedrock. It may be necessary to loosen the overburden material by augering through to the bedrock. The liner may then be advanced through the soil slurry to the bedrock. It is noted that the caissons would require extensive and continuous dewatering since the groundwater table at the site is high. If the caissons cannot be dewatered, concrete may have to be placed by 'tremie' technique.

All caissons must be inspected by a geotechnician working under the supervision of a geotechnical engineer in order to confirm the ULS/SLS bearing pressure of the founding rock and to ensure that the caisson bases have been prepared satisfactorily.

The recommended bearing capacities have been calculated by Trow Associates Inc. from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when foundation construction is underway. The interpretation between boreholes, and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage. An additional investigation should be performed at the site in order to collect additional data on the depth to sound bedrock throughout the site to enable the contractors to provide accurate bids for installation of piles.

17.0 Floor Slab Construction

It is proposed to construct a 540,000 sq. ft. future expansion to be located on the east side of the Phase I Building. The geotechnical investigation has revealed that this area contains normally consolidated to lightly overconsolidated silty clay which extends up to 11.5 m depth. It was therefore recommended that this area should be surcharged in order to consolidate the silty clay such that a slab-on-grade floor will be feasible for the future addition.

With the proposed finished floor elevation of Elevation 56.75 m for the warehouse, up to 3 m of fill will have to be placed under the floor slab in the area of the proposed addition. In addition, the anticipated floor loading will be in the order of 25 kPa. Therefore, the silty clay will be subjected to a load of 85 kPa excluding the footing load. In order to consolidate the silty clay, a surcharge load of 45 kPa would be required i.e. 2.2 m of surcharge fill. The total height of fill to be placed at the site would therefore be 5.2 m.

The area to be surcharged should extend at least 3 m beyond the perimeter of the structure and should the fill be sloped at 45 degrees thereafter. It is noted that placement of fill adjacent to Phase I Building construction to consolidate the silty clay may result in some differential settlements of the Phase I Building and the floor slab. In order to prevent this, it is recommended that the toe of the surcharge fill should be kept at least 5 m away from the Phase I Building. Consequently, it will not be possible to surcharge approximately 11 m wide area adjacent to Phase I construction. In order to facilitate slab-on-grade floor construction in this interim area, it is recommended that all the clay in this area should be sub-excavated and replaced with silty sand fill.

The following construction procedure is recommended:

The area to be surcharged should be stripped of all topsoil and proof rolled in the presence of geotechnical personnel. Any soft areas identified should be sub-excavated and replaced with well compacted fill. The fill to be placed under the floor slab should preferably conform to OPSS Granular 'B' Type II. It should be placed in 300 mm lift thicknesses and each lift compacted to 98 percent of standard Proctor maximum dry density. The placement and compaction of the fill should be undertaken under the full time supervision of geotechnical personnel to ensure that the placement and compaction of the fill is undertaken in a systematic manner. In-place density test should be performed on each lift to ensure that the specified degree of compaction is being achieved. The placement and compaction of the engineered fill for slab-on-grade floor construction can in this way be undertaken to the subgrade level. 2.5 m surcharge fill should then be placed in 500 mm lift thicknesses and each lift compacted to 90 percent standard Proctor maximum dry density.

It is recommended that consolidation of the silty clay should be monitored to determine when settlements will be complete. For this purpose, settlement plates and pneumatic piezometers should be installed in the surcharge area on approximately 50 m to 60 m grid. Settlement of the fill and the pore pressure in the silty clay should be monitored on a regular basis. The surcharge may be removed when monitoring indicates that 95 percent consolidation of the silty clay is complete.

It has been estimated that the consolidation settlements of the silty clay would vary from 110 mm to 700 mm depending on the thickness of the silty clay and thickness of the underfloor fill to be placed. In addition, the engineered fill is expected to settle by an additional 5 mm to 10 mm. Therefore, total settlements of the fill due to consolidation are expected to be in the order of 115 mm to 710 mm. It is difficult to accurately estimate the time required to achieve 100 percent consolidation of the silty clay since it is a function of the thickness of the silty clay, its permeability, presence or absence of sand or silt seams etc. Preliminary computations based on the coefficient of consolidation values obtained from one dimensional oedometer tests indicate that it will take 24 to 36 months in order to achieve 95 percent consolidation of the silty clay.

17.0 General Comments

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

We trust that this report meets your requirements, should you have any further questions feel free to contact this office.

Yours truly,

Trow Associates Inc.

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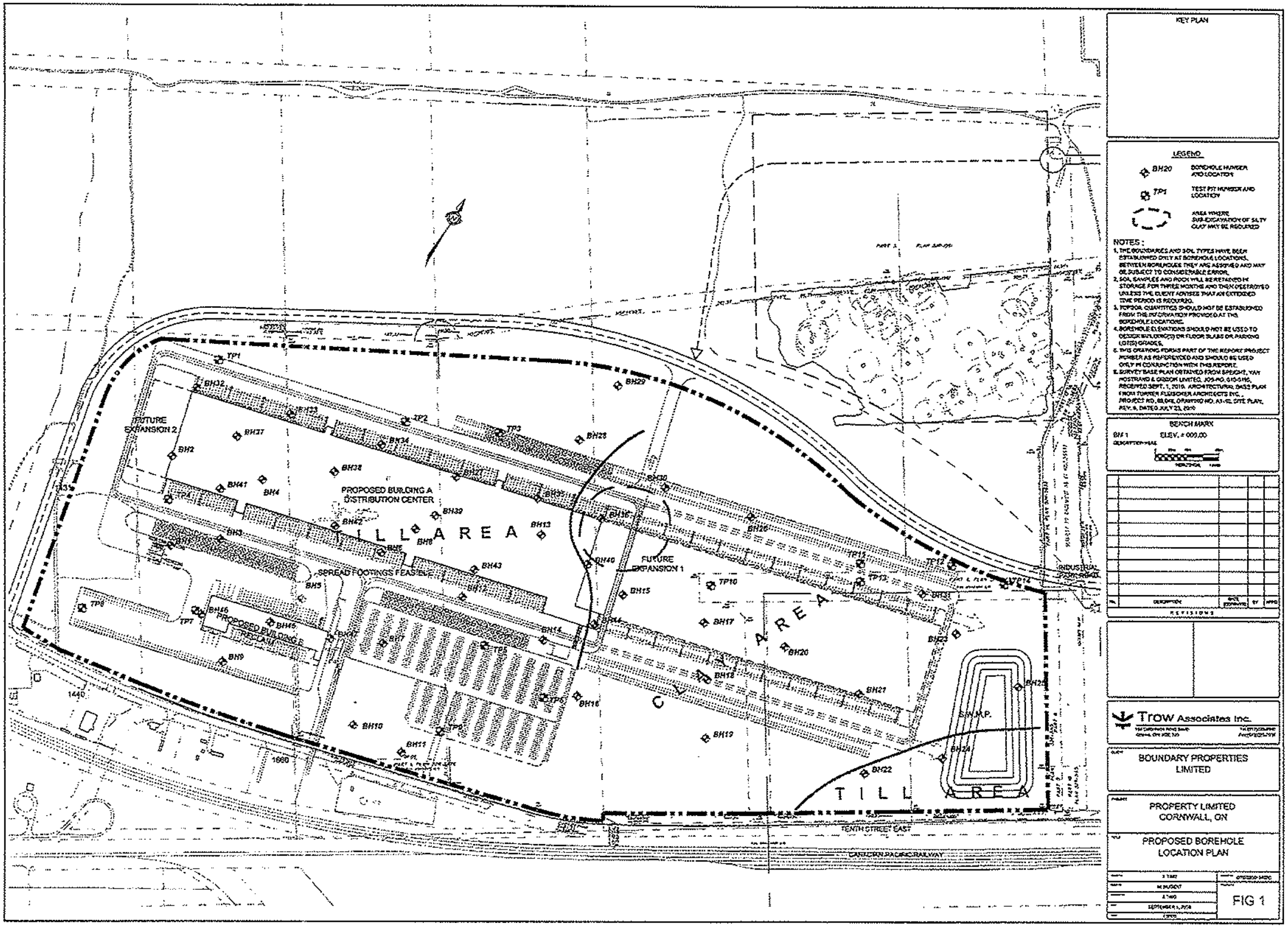
Earth and Environment

Figures



KEY PLAN
LEGEND
BH20 BOREHOLE NUMBER AND LOCATION
TP1 TEST PIT NUMBER AND LOCATION
AREA WHERE SUB-EXCAVATION OF SILTY CLAY MAY BE REQUIRED
NOTES :
1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. SURVEY BASE PLAN OBTAINED FROM SPEIGHT, VAN NOSTRAND & GIBSON LIMITED, JOB NO. 010-016, RECEIVED SEPT. 1, 2010. ARCHITECTURAL BASE PLAN FROM TORRIER FLEISCHER ARCHITECTS INC., PROJECT NO. 09,049, DRAWING NO. A1-02, SITE PLAN, REV. 4, DATED JULY 23, 2010
BENCH MARK
BM 1 ELEV. = 000.00
REVISIONS
Trow Associates Inc.
BOUNDARY PROPERTIES LIMITED
PROPERTY LIMITED
CORNWALL, ON
PROPOSED BOREHOLE LOCATION PLAN
FIG 1
PROPOSED BUILDING A
DISTRIBUTION CENTER
PROPOSED BUILDING B
TILL AREA
CLAY AREA
TILL AREA
SPREAD FOOTINGS FEASIBLE
FUTURE EXPANSION 1
FUTURE EXPANSION 2
S.W.M.P.
TENTH STREET EAST
CANADIAN PACIFIC RAILWAY
INDUSTRIAL PARK ROAD
1440
1660
TP1
TP2
TP3
TP4
TP5
TP6
TP7
TP8
TP9
TP10
TP11
TP12
TP13
BH2
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BH45
BH46

Figures

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by Trow Associates Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO SILT (NONPLASTIC)	FINE	MEDIUM	CRS.	FINE	COARSE
	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Appendix A