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SUPPLEMENTARY GEOTECHNICAL INVESTIGATION

Proposed Colchester Civic Centre

Truro, Nova Scotia

Report Prepared for:
Municipality of the County of Colchester
1 Church Street
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File: Project No. 1041919.01

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1.0 Introduction

Stantec Consulting Ltd. (Stantec), acting at the request of the Municipality of the County of Colchester (MCC), has conducted a supplementary geotechnical investigation of the proposed site for the new Colchester Civic Center (CCC), located along Abenacki Road in Truro, Nova Scotia. The purpose of the supplementary investigation was to assess the subsurface conditions throughout the revised footprint of the proposed building, verify that the subsurface conditions are similar to those encountered during the preliminary geotechnical investigation and provide additional geotechnical recommendations as required, to support continued site preparation and foundation design.

The supplementary geotechnical investigation was carried out in general accordance with a letter entitled Truro Civic Center/Geotechnical, dated October 15, 2009 and submitted to our office by Campbell Comequ Engineering Ltd. (CCE). In general, the scope of work included the drilling of six boreholes at locations specified by CCE and the preparation of this geotechnical report.

This report has been prepared specifically and solely for the project described herein and presents all of our geotechnical findings and recommendations.

1.1 BACKGROUND

The preliminary geotechnical investigation of the site was completed by our office in September 2008. In general, the scope of work for the preliminary investigation included the excavation of twelve test pits throughout a specified study area, geotechnical laboratory testing and the preparation of a preliminary geotechnical report. The preliminary geotechnical report included recommendations for preliminary planning and conceptual design of site grading, building foundations and other associated structures for the development. The preliminary report was entitled Preliminary Geotechnical Investigation Proposed Colchester Civic Center and was issued on November 28, 2008.

Following the preliminary investigation, backfill material was imported from the nearby Colchester East Hants Health Authority Regional Hospital (CEHHARH) construction site and used to begin construction of two structural pads. The location of the two pads was illustrated in Drawing No. 3 of our 2008 report. The footprint of each pad was stripped of vegetative cover/soft soils and Stantec personnel were onsite between July and October 2008 to provide quality control services during fill placement. Results of this work were detailed in the preliminary report.

Our office was supplied with a topographic survey of the site, prepared by E.C. Keen Land Surveying Ltd. (Plan No. 7003) and dated December 7, 2008. Three cross-sections taken from Plan No. 7003 show the existing grades of the site at the time of the survey and final grades for the proposed development. A review of the as-built was carried out as part of this evaluation and our comments are included herein.

2.0 Site Description

The proposed project site is located adjacent to Exit 13 on Nova Scotia Highway 102 and is bounded by Abenaki Road to the south and McClure Mills Drive to the north. A carpool parking lot remains at the northeast corner of the property next to large piles of asphalt millings, understood to have been placed there by Nova Scotia Department of Transportation and Infrastructure Renewal (NSDTIR). The original grades of the site sloped from south to north, terminating at the toe of the embankment for McClure Mills Drive. Surface drainage of the site is controlled by a series of swales and culverts at the edge of the property that drain under McClure Mills Drive. The pre-construction site topography was shown on Drawing No. 3 of our 2008 report.

The two structural pads, constructed in 2008 lie adjacent Abenaki Road within the west half and southeast quarter of the site. For the purpose of this report the pads will be referred to as the west and east pads. The west and east pads are separated by a large swale that remains close to the original grade of the site, sloping from south to north. The site topography at the time of the above-mentioned as-built survey is shown on the appended Drawing No. 4.

Various stockpiles of loosely placed fill and or grubbing material were noted throughout the property at the time of our supplementary investigation. The loosely placed stockpiles are summarized as follows:

- A windrow of end dumped grubbing material was present along the toe of the west pad's northern slope, between the pad and the embankment for McClure Mills Drive;
- A windrow of end dumped material, similar to that previously imported from the CEHHARH site, was observed along the toe of the east pad's northern slope, between the pad and a portion of the above-mentioned NSDTIR asphalt piles.
- A stockpile had been placed and formed into a ramp, sloping upward from Abenaki Road along the southwest edge of the west pad; and
- A large volume of additional material was observed to have been placed atop the east pad, above that monitored by our office in 2008. This material now covers the entire surface of the pad.

3.0 Investigative Procedure

3.1 REVIEW OF AS-BUILT SURVEY (PLAN NO. 7003)

Following a review of Plan No. 7003, it is our opinion that fill thicknesses within the limits of the west pad are in accordance with those inspected by our office in 2008. However, fill thicknesses within the limits of the east pad are beyond those inspected by our office. It appears that fill material continued to be hauled to site after our involvement in 2008. We are not aware when the additional material was stockpiled atop the east pad, but it appears as though the finished surface of the pad, as monitored by our office, was not picked up during the December topographic survey.

3.2 SUPLEMENTARY GEOTECHNICAL INVESTIGATION

3.2.1 General

The supplementary geotechnical investigation was carried out between November 24th and 25th, 2009. An experienced geotechnical technician from our Dartmouth office was onsite to supervise the drilling of six boreholes and log the subsurface conditions encountered. The borehole locations are illustrated on the appended Drawing No. 4.

Detailed descriptions of the subsurface conditions encountered and the sampling conducted are provided on the appended Borehole Records.

3.2.2 Boreholes

The six boreholes were advanced using a track mounted CME850 drill owned and operated by Logan Geotech Inc. The total depths of the boreholes ranged from about 3.6 m to 9.3 m beneath the existing grades of the site.

The boreholes were advanced through the overburden using standard 100 mm diameter flight augers. Soil samples were taken using conventional 50 mm split-spoon samplers while performing Standard Penetration Tests. The Standard Penetration Test (N-value) is the number of blows required to advance a 50 mm OD split-spoon sampler 300 mm into the soil using a standard fall height and weight. N-values can be used as an indication of relative density and can also be used to estimate other soil parameters.

3.2.3 Surveying

Borehole locations were positioned as specified by CCE, based on local landmarks and distances taken from Plan No. 7003. The exception was borehole BH4, which had to be moved slightly due to soft ground conditions. The actual location of borehole BH4 was picked up by our representative using a handheld GPS unit and the actual location is reflected in the appended Drawing No. 4.

The borehole elevations were surveyed by our representative and tied into the known elevation of a nearby fire hydrant, as included on Plan No. 7003. Borehole elevations are provided on the appended Borehole Records.

3.2.4 Laboratory Testing

Extensive laboratory testing was carried out on soil samples collected during our preliminary investigation. The stratigraphy encountered during the supplementary geotechnical investigation was similar to those encountered during the preliminary investigation and therefore, additional laboratory testing was not carried out to classify the soils.

4.0 Subsurface Conditions

Details of the soil strata encountered at the site during the geotechnical investigation are described in the appended Borehole Records. The Symbols and Terms used on Borehole and Test Pit Records provide a brief explanation of the terminology and graphics used in this report and are also appended.

Soil classification was based on procedures described in ASTM D2487 *Standard Practice for Classification of Soils for Engineering Purposes* (*Unified Soil Classification System*) and ASTM D 2488 (*Standard Practice for Description and Identification of Soils, Visual-Manual Procedure*).

The information gathered showed that the vegetative cover was removed from all six borehole locations during the initial site preparation in 2008. Large amounts of fill material are present throughout portions of the site, including the above-mentioned structural fill placed in 2008 and stockpiles of fill/grubbing materials. The remaining native soils present across the site remain relatively consistent with our preliminary findings and comprise a sand layer overlying glacial till. To summarize, the principal strata encountered at the site are as follows:

TABLE 4.1 Borehole Summary Table

BH No.	Ground Surface Elev. (m)	Total Depth (m)	Thickness (m)			Depth to Groundwater (m)
			Fill	Sand	Clayey Sand TILL	
BH1	38.8	9.3	5.3	0.8	> 3.2	1.3
BH2	34.1	7.3	3.3	1.3	> 2.7	2.1
BH3	33.7	3.6	-	0.5	> 3.2	0.6
BH4	31.2	3.7	-	0.3	> 3.3	0.3
BH5	37.7	7.0	3.6	0.6	> 2.8	3.2
BH6	39.1	8.5	5.4	0.5	> 2.6	> 6.1

4.1 FILL

4.1.1 Previously Constructed Fill Pads

Borehole BH 1 was located within the fill limits of the east pad. Boreholes BH5 and BH6 were located within the fill limits of the west pad. A surficial layer of reddish brown clayey sand to sandy clay fill material, similar to that imported from the CEHHARH site, was encountered in the areas of boreholes BH1, BH5 and BH6.

Based on N-values obtained during the Standard Penetration testing, the compactness of the fill within boreholes BH5 and BH 6 (west pad) is described as very loose to loose within the upper 300 mm of the layer, becoming compact below this depth. The thickness of the fill layer within boreholes BH 5 and BH 6 was about 3.6 m and 5.4 m respectively.

The compactness of the fill within borehole BH 1 (east pad) is described as very loose within the upper 3.6 m of the layer, becoming loose below this depth. This represents fill placed without inspection and control. The thickness of the fill layer within borehole BH 1 was about 5.3 m.

4.1.2 NSDTIR Asphalt Pile

Borehole BH 2 was located within a leveled off portion of the above-mentioned NSDTIR asphalt piles. The surface of the stockpile was approximately level with the grade of the nearby carpool parking lot. A surficial layer of dark grey to brown silty sand with gravel fill material was encountered in the area of borehole BH 2. Pieces of asphalt were present throughout the fill.

Based on N-values obtained during the Standard Penetration testing, the compactness of this layer is described as compact within the upper 1.2 m of the layer, becoming loose to very loose below this depth. The thickness of the fill layer was about 3.3 m.

4.2 SILTY SAND TO SANDY SILT

A layer of reddish brown to brown silty sand to sand with silt, was present in all six boreholes. The sand layer was present at the surface of boreholes BH3 and BH4, and just beneath the surficial fill layer of the remaining four boreholes. The thickness of the layer ranged from about 0.3 m to 1.3 m in boreholes BH4 and BH2 respectively. Trace organics were noted within the upper portions of the sand layer in boreholes BH2, BH5 and BH6.

Based on N-values obtained during the Standard Penetration testing, the compactness of this layer is described as very loose to loose in the areas of boreholes BH3 and BH4, and loose to compact in the areas of the remaining boreholes, where the layer was encountered at depth.

4.3 TILL LAYER

A layer of glacial till comprised of reddish brown to brown sandy clay with gravel was present directly beneath the sand layer in all six of the boreholes. All six boreholes were terminated within the till layer at depths ranging from 3.6 m to 9.3 m in boreholes BH3 and BH1 respectively. Occasional cobbles were present throughout the layer in borehole BH1.

Based on N-values obtained during the Standard Penetration testing, the consistency of this layer is described as stiff to hard.

4.4 GROUNDWATER

Standpipes were installed prior to the backfilling of all six boreholes. Groundwater readings were taken within the standpipes on December 21, 2009. The readings indicated that the groundwater level was about 0.3 to 0.6 m below the existing surface in the areas of boreholes BH 3 and BH 4 and about 1.3 to greater than 6.1 m below the existing surface of the other four boreholes. It should be noted that groundwater levels will fluctuate with seasonal weather trends, during particular precipitation events, or site use and construction activities.

5.0 Discussion and Recommendations

5.1 GENERAL

Preliminary geotechnical recommendations for the site were provided in our 2008 report. The preliminary recommendations were based on the findings of 12 test pits, excavated at discrete locations throughout a limited study area. Use of the preliminary recommendations for other areas of the site, located outside the initial study area, was dependant on further assessment to verify that the subsurface conditions within the extended area were similar. Based on the findings of the six boreholes discussed herein, the native subsurface conditions at the borehole locations were similar to those encountered within the initial 12 test pits.

However, supplementary geotechnical recommendations are included below and although they are not substantially different than those supplied previously, the recommendations contained herein should be considered to supersede the previous recommendations wherever differences are noted.

Further recommendations pertaining to site preparation, shallow foundation design, slab on grade design, retaining wall design and asphalt pavement design are included below.

5.2 SITE PREPARATION

5.2.1 Excavation - Previously Unprepared Areas

The footprints of the two structural pads, previously constructed in 2008, were stripped of vegetative cover and soft soils at that time and inspected by qualified Stantec personnel prior to fill placement. The remainder of the site was not prepared in this way or inspected at that time. Therefore, it is recommended that any areas outside the footprints of the structural pads and located within the zone of influence for building foundations or pavement structures, be stripped of the existing cover material (surficial vegetation, organics, NSDTIR asphalt piles, loose fill material, etc.) to expose the native sand or glacial till layer beneath.

The exposed native sand or glacial till subgrade should then be proof rolled under the supervision of qualified geotechnical personnel. The proof roll should be carried out using appropriate equipment, such as a loaded tandem dump truck or 10-ton vibratory roller. Any soft spots identified should be recompacted or over excavated and replaced with structural fill, in accordance with Section 5.2.4 below.

The zone of influence for the proposed CCC building and associated pavement structures shall be taken as the footprint of the building/paved area plus a horizontal distance beyond the outside edges of the footprints to include a splay of 1 horizontal to 1 vertical (1 H: 1 V). This expanded footprint is referred to as the stress zone of influence below.

5.2.2 Excavation – Previously Constructed Structural Pads

West Pad: The upper 300 mm of the west pad has been softened by either freeze/thaw cycles or over trafficking following the initial construction. Therefore, the upper 300 mm of the pad will need to be excavated and the exposed surface proof rolled, prior to additional fill placement or foundation construction atop this pad. If this material is in dry condition suitable for compaction at the time of construction it may be practical to simply compact it in place. The side slopes of the pad will likely require similar treatment and should be cut into during additional fill placement adjacent the pad, to remove any soft soils from the surface and expose the compact structural fill beneath.

In addition, the haul access area used during the initial infilling of the west pad was identified as being soft due to over trafficking within our 2008 report (Section 5.3.2, Jacques 2008). It was recommended that the area be removed, wasted and replaced with structural fill under the supervision of geotechnical personnel. This work has not been completed.

East Pad: The upper approximately 3.6 m of the east pad was not monitored by Stantec personnel during placement and appears to comprise of either very loose stockpiled material or poorly placed backfill material. The lower 1 to 2 m of the east pad was monitored throughout placement. Therefore, it is recommended that the upper 3.6 m of the east pad be removed to expose the approximate thickness of structural fill placed under our supervision. The exposed surface should then be dealt with in the same fashion as described above for the west pad (proof roll and soft soils removed).

In addition, an area of the infill along Abenaki Road was identified as having been placed over organics within our 2008 report (Section 5.3.2, Jacques 2008). The area in question was located between TP 11 and TP 12 and was identified on Drawing No. 3 of that report. It was recommended that the fill be removed to allow proper grubbing in this area. This work has not been completed.

5.2.3 Water Control

It is anticipated that portions of the required excavations will extend below the water table. Therefore, it will be necessary to provide temporary dewatering measures, such as ditches and swales along the bottom edges of the excavations leading to submersible pumps. Surface water flow should be directed away from excavations using ditches/swales.

5.2.4 Structural Fill

Structural fill should consist of a well-graded inorganic material similar to the imported CEHHARH fill material or a well graded sand and gravel. The maximum particle size of fill should be less than 300 mm with moisture contents within 2 percent of optimum moisture content.

The reuse of any existing fill material, identified for removal within Section 5.2.2 above, will be subject to the above requirements. It is anticipated that the moisture contents of the stockpiled fill materials currently present onsite may be too high for immediate reuse as structural fill. Any of the existing fill deemed unsuitable for use as structural fill, could be considered for landscaping purposes or disposed of offsite.

The compacted lift thickness for structural fill placement should not exceed 300 mm assuming that each lift receives 5 passes from a 10-tonne vibratory roller to achieve compaction; one pass is considered to be a full back and forth motion of the roller over the area. The use of larger or smaller compaction equipment may require that the lift thickness be adjusted at the discretion of qualified geotechnical personnel to assure the desired compaction is achieved throughout the entire thickness of the lift.

It is recommended that inspection by experienced geotechnical personnel be carried out prior to the placement of any structural fill to verify that all unsuitable materials are removed from the zone of influence. Further monitoring should be carried out during the placement of any structural fill to assure that the required compaction is achieved.

Structural fill below footings or slabs should be compacted to 100 percent of the standard Proctor maximum dry density (SPMDD) determined for the material. Structural fill below pavement structures should be compacted in accordance with Section 5.4 below.

Fill slopes should be constructed no steeper than 2.5H:1V for temporary stability. It is expected that the self weight settlement of the structural fill material will be less than approximately 2 percent of the fill height and would be complete within 4 months of placement.

5.3 FOUNDATION DESIGN

Once the site has been prepared as outlined above, spread/strip footings and slabs on grade would be suitable for the support of the proposed CCC facility. It should be noted that the existing roadway embankments along the perimeter of the site, appear to fall within the area being proposed for the CCC development. Care should be taken to assure that the stress zone of influence for any new developments does not overlap with such embankment fills, as they are likely poorly compacted and additional loading could result in excessive settlement. Therefore, a suitable set back should be maintained from all existing roadways unless existing fills are replaced.

Shallow Foundations

For preliminary design of spread/strip footings, the following geotechnical bearing resistances are recommended:

	Approved Structural Fill/Native Sand/Native Till
Geotechnical Resistances at Serviceability Limit States (SLS)	125 kPa
Factored Geotechnical Resistance at Ultimate Limit States (ULS)	225 kPa

These values are based on foundations with a minimum width of 0.6 m and bearing at a minimum depth of 1.2 m below finished grades when founded on structural fill or native sand/glacial till. These values also reflect the observed variability in the soils present throughout the site and unknowns regarding the bearing surface for the footings. Therefore, SLS and ULS values provided above, can be reviewed as planning and design advance, if necessary.

The serviceability limit states geotechnical resistances are based on a maximum settlement of 25 mm. Unfactored loads were used for assessment with the SLS bearing resistances.

If water seepage softens the native sand/glacial till or structural fill at the bearing surface of foundations, it should be removed and replaced with compacted 200 mm minus approved granular material, NSDTIR Type 2 Gravel or similar approved material.

Exterior footings and footings in unheated areas, founded on soil, should have a minimum soil cover of 1.2 m, or an equivalent combination of soil and insulation, for frost protection.

Slabs on Grade

The slabs on grade should be bedded on a layer of free draining gravel, such as 25 mm clear stone. The specific required material and thickness of bedding will be dependent on underslab components, however, a minimum thickness of 150 mm is recommended for general purposes. For design of slabs, a preliminary subgrade modulus (k) of 70 MPa/m (based on a 300 mm diameter plate) may be used for the site sand/till or structural fill. Final design subgrade modulus will depend on actual materials at the constructed subgrade levels.

Perimeter footing drains, with a positive discharge should be provided in all areas where the floor slab is located below final exterior grade.

All bearing surfaces should be inspected by qualified geotechnical personnel prior to placement of concrete.

Slabs on grade placed at the base of the ice surface area and any swimming pools can be designed based on the above recommendations. However, additional consideration should be given to the potential for frost heave beneath the ice surface and water drainage from beneath any swimming pool areas.

In general, the use of either insulation or insulation with an in ground heating system would be appropriate to minimize the potential for frost heave below an ice surface used seasonally or year round respectively. The CEHHARH fills imported to the site are considered to be frost susceptible.

The minimum thickness of the free draining gravel layer beneath slabs placed at the base of any swimming pools should be at least 200 mm to allow for additional drainage. Considering that the base of the swimming pools are likely to be below the building foundations separate under slab and perimeter drains will be required to prevent positive hydrostatic pressure when the pool is empty.

Specific design recommendations pertaining to slabs on grade placed at the base of the ice surface area and any swimming pools would require additional information pertaining to final grades and intended usage.

5.4 ASPHALT PAVEMENT DESIGN

It is anticipated that the majority of the site will support light vehicle traffic (cars and light trucks) and only select areas will support heavier truck traffic (travel way for deliveries). Based on the anticipated subgrade, the recommended asphalt pavement structures are provided in the Tables 5.1 and 5.2 below:

TABLE 5.1 Light Vehicle Traffic Areas – Asphalt Pavement Design

Layer	Thickness
Asphalt – Mix Type C	75 mm
Gravel Type 1	300 mm

TABLE 5.2 Heavy Vehicle Traffic Areas – Asphalt Pavement Design

Layer	Thickness
Asphalt – Mix Type C	40 mm
Asphalt – Mix Type B	50 mm
Gravel Type 1	200 mm
Gravel Type 2	400 mm

The material types noted in the above tables are in accordance with the Nova Scotia Department of Transportation and Public Works Standard Specifications.

Prior to the placement of Gravel Types 1 and 2 the subgrade should be compacted to 98% of the SPMDD. Following compaction, the subgrade should be proof rolled with a loaded tandem truck and areas exhibiting bowl type deflections greater than 20 mm should be over excavated by 450 mm and replaced with 100 mm to 150 mm blast rock or an approved fill material.

It may be advantageous to establish a designated route for haul truck traffic throughout construction. If established, it is recommended that the designated route be constructed using Gravel Types 1 and 2 in accordance with Table 5.2 above. The installation of filter fabric between the subgrade and the Type 2 material would help to minimize migration of fines from the subgrade into the Type 2.

The Gravel Types 1 and 2 shall be placed and compacted in accordance with the Nova Scotia Department of Transportation and Public Works Standard Specifications except that the compaction shall be at least 100 percent of the SPMDD.

The asphalt base and/or surface shall be placed in accordance with the standard specification of the Nova Scotia Department of Transportation and Infrastructure Renewal except that the required compaction shall be at least 92 percent of the maximum theoretical density

5.5 RETAINING WALL DESIGN

Footings for retaining walls may be designed based on the criteria provided in Section 5.3 above. Embedment depth should be at least 300 mm below the finished grade or frost depth where applicable. A detailed slope stability analysis should be performed to ensure stability of the wall.

The retaining wall should be backfilled within a 45 degree wedge from the heel of the wall with a free draining material such as clear stone or Type 2 gravel. Backfill should be placed in lifts of less than 300 mm and compacted to 95% of standard Proctor maximum dry density except within the top 300 mm of any pavement subgrade which should be compacted to 98% of standard Proctor maximum dry density. A drainage system with a positive outlet should be provided at the base of the gravel wedge. Compaction immediately adjacent to the wall should be accomplished with light compaction equipment to prevent over-stressing of the wall. The design parameters provided in Table 5.3 are recommended for design:

TABLE 5.3 Soil Parameters for Design of Retaining Walls

Parameter	Imported Granular Fill	Native Till or Imported CEHHARH Fill
Total Unit Weight, γ	21 kN/m^3	20 kN/m^3
Effective Angle of Internal Friction, ϕ'	35°	30°
Concrete/Granular fill Friction Angle, δ	20°	N/A
Active Earth Pressure Coefficient, k_a	0.27 (assuming level ground)	0.33 (assuming level ground)
At Rest Earth Pressure Coefficient, k_o	0.43 (assuming level ground)	0.50 (assuming level ground)
Passive Earth Pressure Coefficient, k_p	3.69 (assuming level ground)	3.00 (assuming level ground)

Note: The earth pressure coefficients indicated assume horizontal ground; the values should be adjusted for sloping backfill or surcharge loads behind the wall and downward slopes in front of walls.

Sliding resistance at the base of retaining walls can be calculated using friction factors provided in Table 5.4. A geotechnical resistance factor of 0.8 should be used in the sliding analysis.

TABLE 5.4 Friction Factors for Different Materials Placed Against Precast Concrete

Material	Friction Factor
Native Till or Imported CEHHARH Fill	0.40
Granular Fill	0.45
Rock Fill	0.50

6.0 Closure

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of the Municipality of the County of Colchester who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these be not satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions

This report was prepared by Jason A. Smith, EIT and reviewed by Dan R. McQuinn, P.Eng. Should you have any questions, please do not hesitate to call us at 902-468-7777.

Yours very truly,

STANTEC CONSULTING LTD.

Jason A. Smith, EIT

Dan R. McQuinn, P.Eng.

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APPENDIX A

Statement of General Conditions
Symbols and Terms Used on Borehole and Test Pit Records
Borehole Records 1 to 6
Drawing No. 4, Borehole Location Plan

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd's present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited. extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
Very Soft	<0.25	<12.5
Soft	0.25 - 0.5	12.5 - 25
Firm	0.5 - 1.0	25 - 50
Stiff	1.0 - 2.0	50 – 100
Very Stiff	2.0 - 4.0	100 - 200
Hard	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor, Crushed, Very Severely Fractured*
25-50	*Poor, Shattered and Very Seamy or Blocky, Severely Fractured*
50-75	*Fair, Blocky and Seamy, Fractured*
75-90	*Good, Massive, Moderately Jointed or Sound*
90-100	*Excellent, Intact, Very Sound*

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

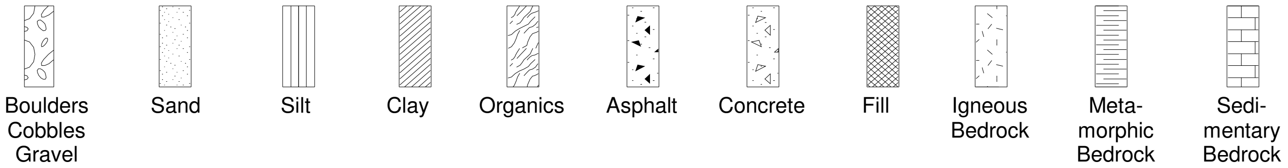
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

Term	Description
Fresh	No visible signs of rock weathering. Slight discolouration along major discontinuities
Slightly Weathered	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
Moderately Weathered	Less than half the rock is decomposed and/or disintegrated into soil.
Highly Weathered	More than half the rock is decomposed and/or disintegrated into soil.
Completely Weathered	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

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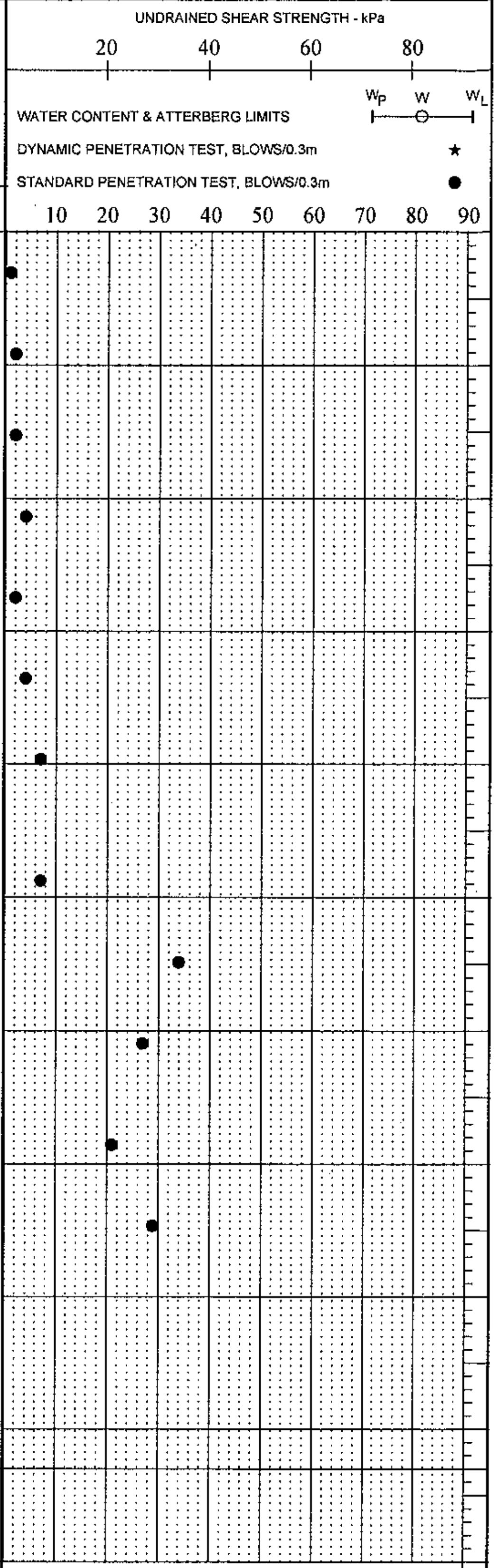
BOREHOLE RECORD

BH1

CLIENT **MUNICIPALITY OF THE COUNTY OF COLCHESTER**
PROJECT No. **121610239**
LOCATION **CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS**
BH SIZE **FLIGHT**
DATES: BORING **2009/11/24**
WATER LEVEL
DATUM **GEODETIC**

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR-RQD %	OTHER TESTS
0	38.75	FILL: Reddish-brown silty sand trace clay to clayey sand		SS	1	225	1	
1				SS	2	250	2	
			▼	SS	3	200	2	
2				SS	4	225	4	
				SS	5	425	2	
3				SS	6	175	4	
4				SS	7	300	7	
5				SS	8	175	7	
	33.44	Compact brown silty SAND to SAND with silt		SS	9	550	34	
6	32.65	Stiff to hard brown sandy clay with gravel: TILL		SS	10	450	27	
7				SS	11	250	21	
				SS	12	500	29	
8				SS	13	300	86/200mm	
9		- 100-mm cobble encountered		NQ	14	400	-	
	29.45	End of Borehole - standpipe installed						
10								



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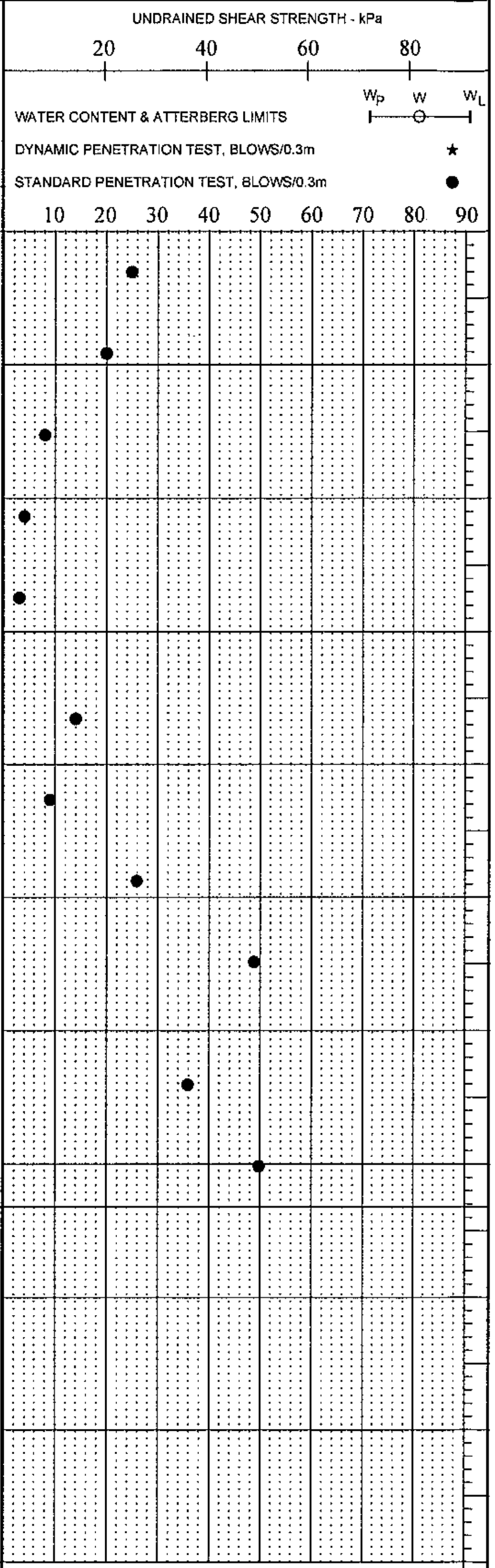
BOREHOLE RECORD

BH2

CLIENT MUNICIPALITY OF THE COUNTY OF COLCHESTER
PROJECT No. 121610239
LOCATION CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS
BH SIZE FLIGHT
DATES: BORING 2009/11/24
WATER LEVEL
DATUM GEODETIC

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR-RQD %	OTHER TESTS
0	34.10	FILL: Dark gray to brown silty sand - gravel size asphalt pieces throughout - trace amounts of clay (decreasing with depth)	▨		SS	1	425	25	
1					SS	2	350	20	
					SS	3	350	8	
2				▼	SS	4	375	4	
					SS	5	50	3	
3	30.75	- wood encountered							
		Compact to loose reddish brown silty SAND - trace gravel - trace organics in top 100 mm			SS	6	250	14	
4	29.53				SS	7	225	9	
5		Stiff to hard reddish brown sandy clay with gravel: TILL			SS	8	225	26	
					SS	9	250	49	
6									
					SS	10	100	36	
7	26.78				SS	11	200	50	
8		End of Borehole - standpipe installed							
9									
10									



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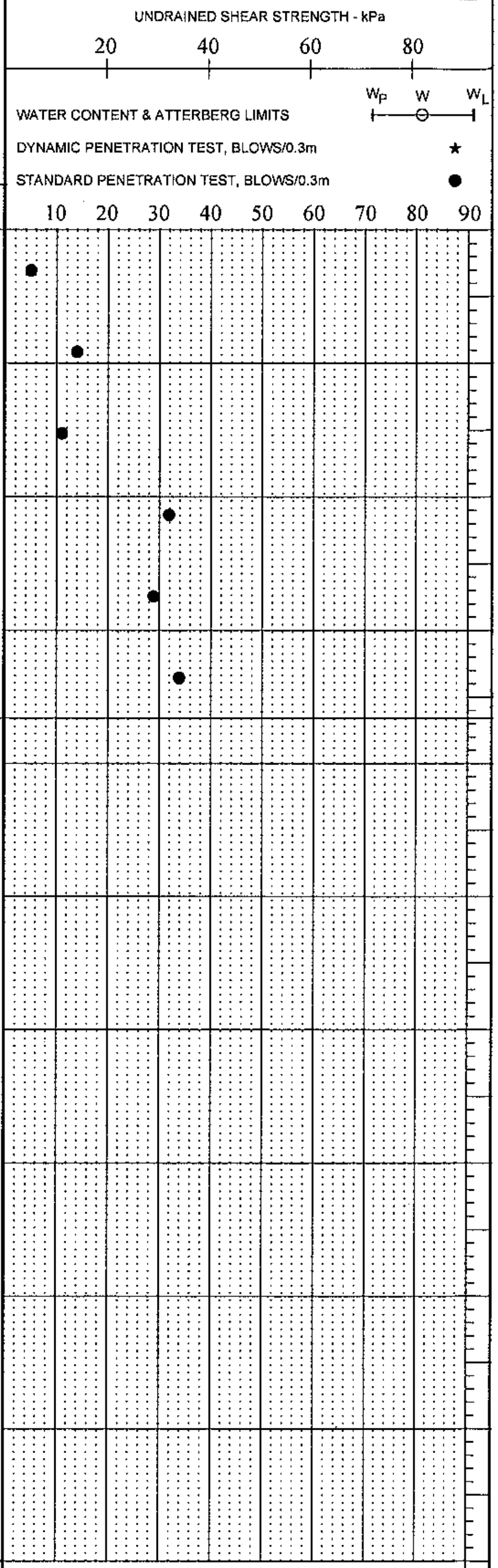
BOREHOLE RECORD

BH3

CLIENT MUNICIPALITY OF THE COUNTY OF COLCHESTER
PROJECT No. 121610239
LOCATION CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS
BH SIZE FLIGHT
DATES: BORING 2009/11/24
WATER LEVEL 2009/11/25
DATUM GEODETIC

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR-RQD %	OTHER TESTS
0	33.65	Loose reddish brown silty SAND			SS	1	400	5	
	33.12	Firm to very stiff reddish brown sandy clay with gravel: TILL		▼	SS	2	550	14	
					SS	3	250	11	
					SS	4	525	32	
					SS	5	475	29	
					SS	6	425	34	
	29.99	End of Borehole - standpipe installed							



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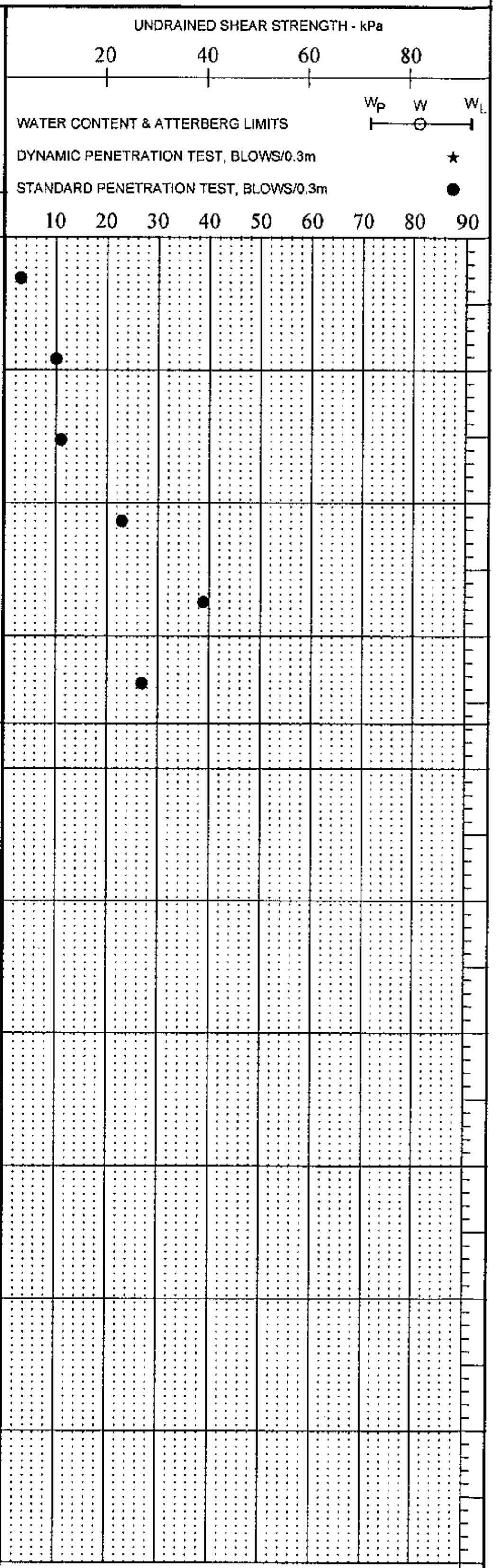
BOREHOLE RECORD

BH4

CLIENT **MUNICIPALITY OF THE COUNTY OF COLCHESTER**
PROJECT No. **121610239**
LOCATION **CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS**
BH SIZE **FLIGHT**
DATES: BORING **2009/11/24**
WATER LEVEL **2009/11/25**
DATUM **GEODETIC**

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR-RQD %	OTHER TESTS
	31.20						mm		
0	30.92	Very loose brown SAND Firm to very stiff reddish brown sandy clay with gravel:TILL		▼	SS	1	100	3	
1					SS	2	425	10	
					SS	3	150	11	
2					SS	4	175	23	
3					SS	5	300	39	
	27.54				SS	6	150	27	
4		End of Borehole - standpipe installed							



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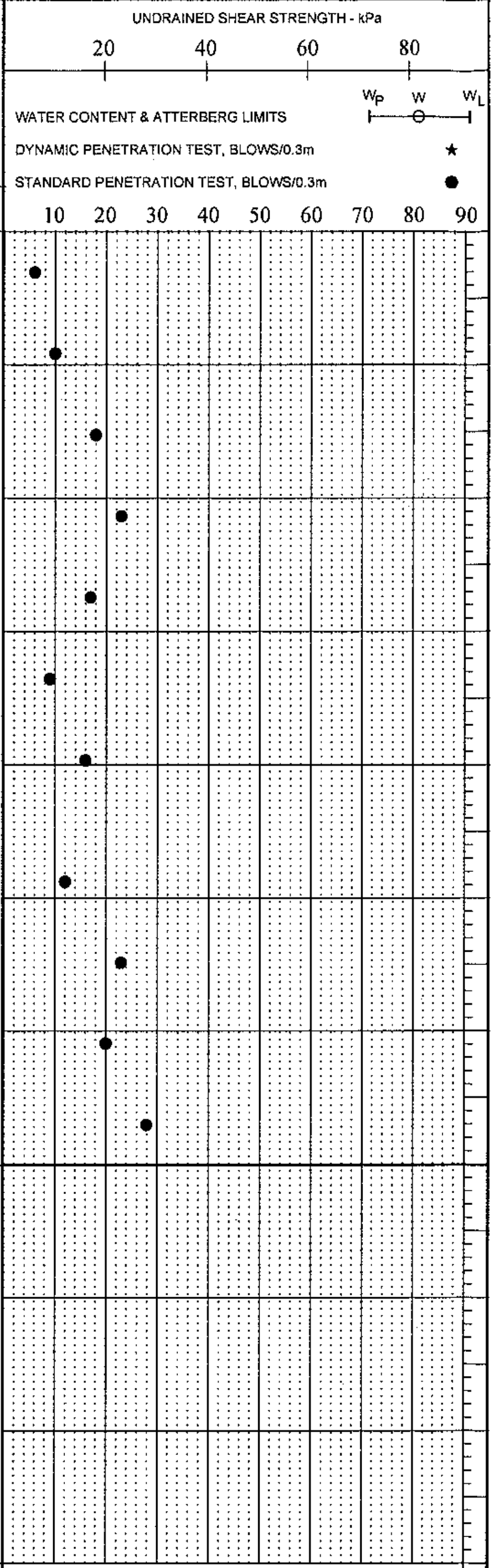
BOREHOLE RECORD

BH5

CLIENT MUNICIPALITY OF THE COUNTY OF COLCHESTER
PROJECT No. 121610239
LOCATION CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS
BH SIZE FLIGHT
DATES: BORING 2009/11/24 to 2009/11/25
WATER LEVEL
DATUM GEODETIC

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR-RQD %	OTHER TESTS
0	37.71	FILL: Reddish brown silty sand to clayey sand - trace gravel			SS	1	450	6	
					SS	2	350	10	
					SS	3	575	18	
					SS	4	500	23	
					SS	5	575	17	
				▼	SS	6	550	9	
	34.13	Compact dark grey to brown silty SAND - trace organics and rootlets			SS	7	400	16	
	33.47	Firm to very stiff reddish brown sandy clay with gravel: TILL							
					SS	8	525	12	
					SS	9	200	23	
					SS	10	375	20	
					SS	11	200	28	
7	30.70	End of Borehole - standpipe installed							



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BOREHOLE RECORD

BH6

CLIENT MUNICIPALITY OF THE COUNTY OF COLCHESTER

LOCATION CENTRAL NOVA SCOTIA REGIONAL CIVIC CENTRE, TRURO, NS

DATES: BORING 2009/11/25

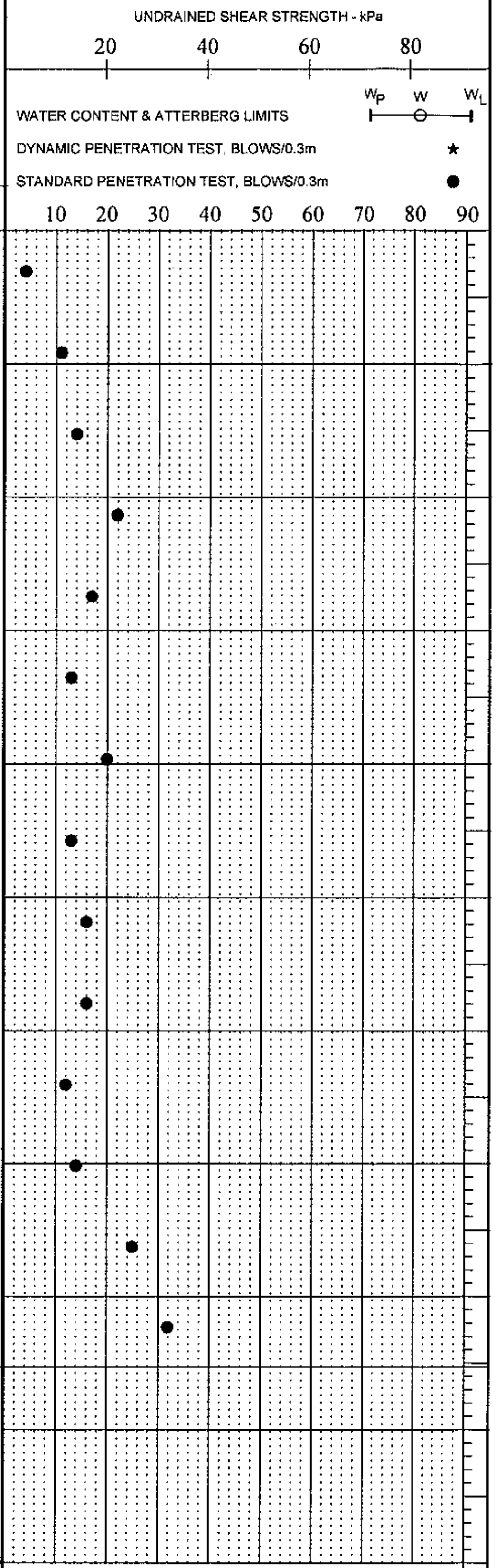
WATER LEVEL

PROJECT No. 121610239

BH SIZE FLIGHT

DATUM GEODETIC

ELEVATION(m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR-RQD %	OTHER TESTS
39.08	FILL: Reddish brown clayey sand with gravel		SS	1	400	4	
			SS	2	325	11	
			SS	3	450	14	
			SS	4	400	22	
			SS	5	200	17	
			SS	6	500	13	
			SS	7	550	20	
			SS	8	475	13	
			SS	9	600	16	
33.70	Compact dark grey to brown silty SAND - organics throughout		SS	10	300	16	
33.21	Firm to very stiff reddish brown sandy clay with gravel: TILL		SS	11	550	12	
			SS	12	575	14	
			SS	13	425	25	
			SS	14	425	32	
30.55	End of Borehole - standpipe installed						



DEPTH(m): 0 1 2 3 4 5 6 7 8 9 10

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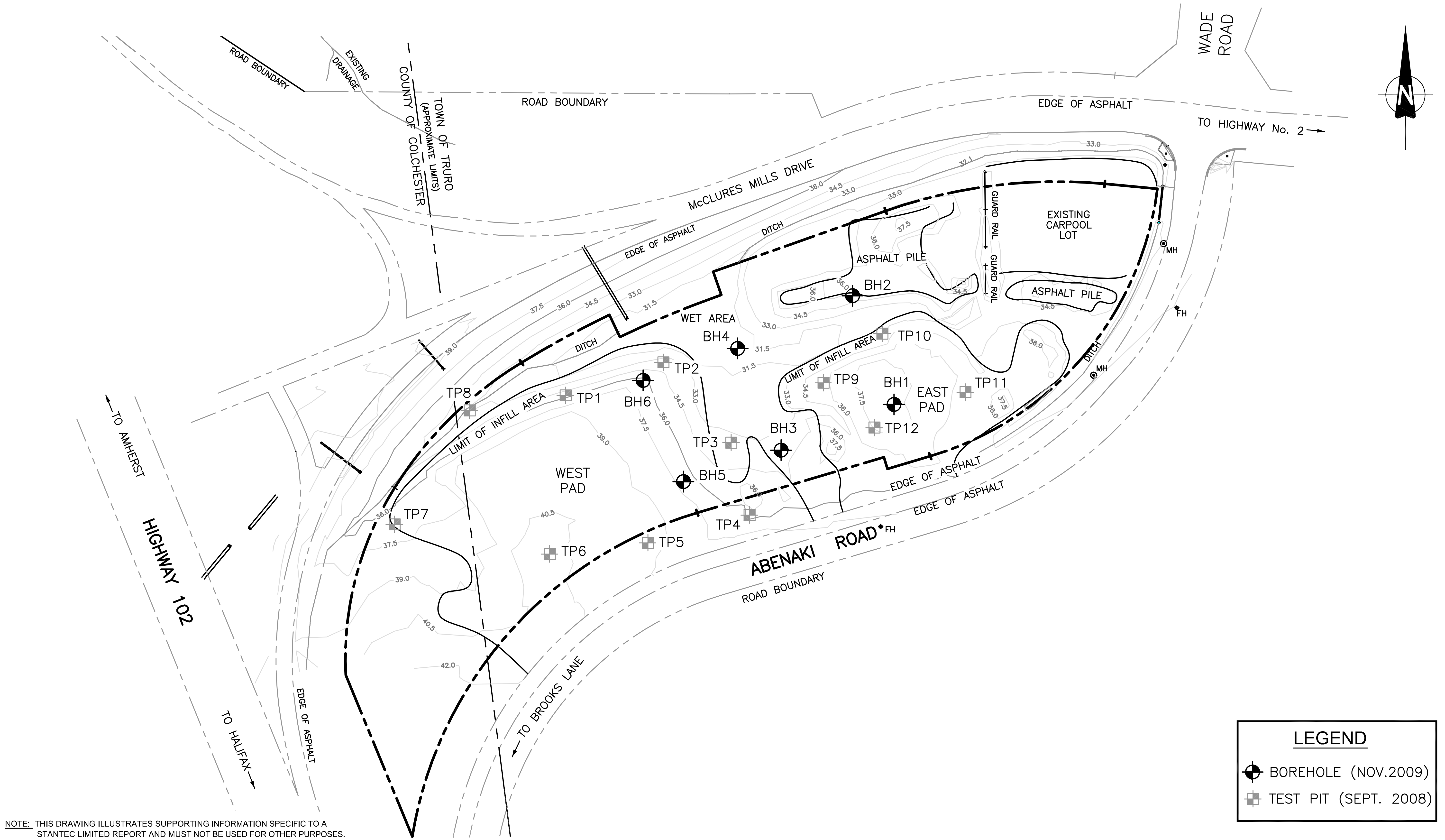

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Reference:

CAD FILE "CIVIC CENTER DRAWING" RECEIVED FROM CLIENT 8 AUG/08.

Job No.: 1041919
Scale: 1:1500
Date: 2009/12/23
Dwn. By: SJT
App'd By:

Client:
MUNICIPALITY OF COLCHESTER

Site Address
ABENAKI ROAD,
TRURO, NOVA SCOTIA

Project:
PROPOSED TRURO
CIVIC CENTRE

Drawing Title:
BOREHOLE LOCATION PLAN

Dwg. No.:
4

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