

re: Geotechnical Review and Recommendations
Proposed Building Addition – Existing Orleans KIA Motors Dealership
2045 Mer Bleue Road, Ottawa, Ontario

to: Q9 Planning + Design – Christine McCuaig – christine@q9planning.com

date: March 13, 2026

file: PG7933-MEMO.01

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide design and construction recommendations in support of the proposed building addition from a geotechnical perspective. The following memorandum should be read in conjunction with and provides recommendations that supersede Paterson Group Report PG2866-1 dated December 12, 2012, which is also appended to this memorandum for further reference.

Geotechnical Review and Recommendations

The following sections of this memorandum are provided as recommendations and information that supersedes the associated portions of the attached and aforementioned Geotechnical Report. These recommendations are provided in support of the currently proposed addition. Subsurface conditions encountered during the previous investigation surrounding the proposed structure are expected to be consistent with those that will be encountered throughout the proposed additions and associated structures. Portions of the report that are not identified in this memorandum remain applicable and pertinent to the subject site and proposed addition.

Section 2.0 – Proposed Development

Paterson understands the currently proposed development will consist of two separate one-storey additions along the east and south portions of the existing structure and an extension of the existing mezzanine structure.

Section 5.3 – Foundation Design

The bearing resistance values and associated footing size limitations for conventional shallow spread foundation placed upon an undisturbed, stiff silty clay bearing surface provided in the previous Geotechnical Report (i.e., **150 kPa** at serviceability limit states and **225 kPa** at ultimate limit states) are considered applicable and suitable for the proposed additions and other footings considered for this project. All bearing surfaces are recommended to be reviewed at the time of construction by Paterson field personnel. The perimeter strip footings provided for the addition are recommended to be founded at the same founding elevation as the connecting strip footings for the existing structure.



Section 5.4 – Design for Earthquakes

The previous structure is anticipated to have been designed using the 2006 Ontario Building Code and designed using a seismic site classification **Site Class C**. The proposed additions are recommended to be designed using an equivalent seismic site classification and associated seismic design loading as would have been considered for the existing structure. The current seismic site classification for the subject site under the 2024 Ontario Building Code may be considered as a **Site Designation X_c**. The soils underlying the proposed addition and subject site are not considered susceptible to liquefaction or cyclic softening.

Section 5.6 – Pavement Structure

Re-Use of Existing Crushed Stone Fill

Existing crushed stone fill that is appropriately separated from the building and other sub-excavations may be considered for re-use for building up subgrade and as a portion of the sub-base layers of newly paved areas provided the fill does not contain high amounts of fine soil particles (i.e., clays and silts) and generally consists of well-graded crushed stone. The potential re-use material should be reviewed and tested by Paterson field personnel at the time of exposing the material to assess the suitability for re-use and overall quality of the material, since it was not encountered at the time of this investigation.

Pavement Joint Tie-in

Where the proposed pavement structure meets an existing pavement structure, the following recommendations should be followed:

- ☐ A 300 mm wide section of the existing asphalt should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
- ☐ It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
- ☐ The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- ☐ Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.

Light Post Construction

It is expected that light posts will be constructed within the proposed development. Light post pole bases are considered unheated structures and therefore their subgrade would require a minimum of 2.1 m of soil cover or equivalent to be protected from frost action.



It is recommended that the post bases be founded below the depth of frost migration for unheated structures. Furthermore, it is anticipated that the proposed pole bases are to be installed using open excavation and will be founded upon an undisturbed stiff silty clay. The site excavated material is considered frost susceptible, and not suitable for backfill directly against the bases.

To mitigate potential frost action from the backfill material surrounding the bases, it is recommended that the buried portion of the bases be surrounded by a layer of engineered fill such as OPSS Granular A or Granular B Type II with a minimum thickness of 600 mm surrounding the pole base. This thickness may be reduced to 300 mm if the proposed bases are surrounded by a suitable non-fixed casing material such as a sono-tube shell or other approved separation layer product capable of remaining intact below the ground surface.

All backfill used to reinstate the excavation should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the materials SPMDD to provide a suitable subgrade for the surrounding pavement structure.

The pole bases may be founded on a silty clay bearing surface once approved by Paterson at the time of construction. Based on these recommendations, an end-bearing bearing resistance value of 100 kPa at serviceability limit states and 150 kPa at ultimate limit states may be considered for these structures, if required. It should be noted that the pole bases founded on the silty clay deposit will be subject to post-construction settlements of up to 25 mm.

6.7 Landscaping Considerations

While the subject site is underlain by silty clay, the immediate area surrounding the proposed structure will be surfaced with asphalt and medians being located more than 10 m away from the proposed structure. While tree planting setbacks are not expected to be applicable to the subject site, setbacks in accordance with the City of Ottawa's current 2017 Guidelines for tree planting setbacks (i.e., tree to foundation setback equal to the mature height of the tree) would be considered applicable to any landscaping considered throughout the subject site.

7.0 Recommendations

All excess soils generated by the proposed works are to be managed in accordance with the current version of *Ontario Regulation (O.Reg.) 406/19: On-Site and Excess Soil Management*. All of the above-noted items and remaining information noted in the previous Geotechnical Report are recommended to be verified by Paterson field personnel at the time of construction.



8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once it is available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Ride Automotive Group or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

We trust that the current submission meets your immediate requirements.

Best Regards,

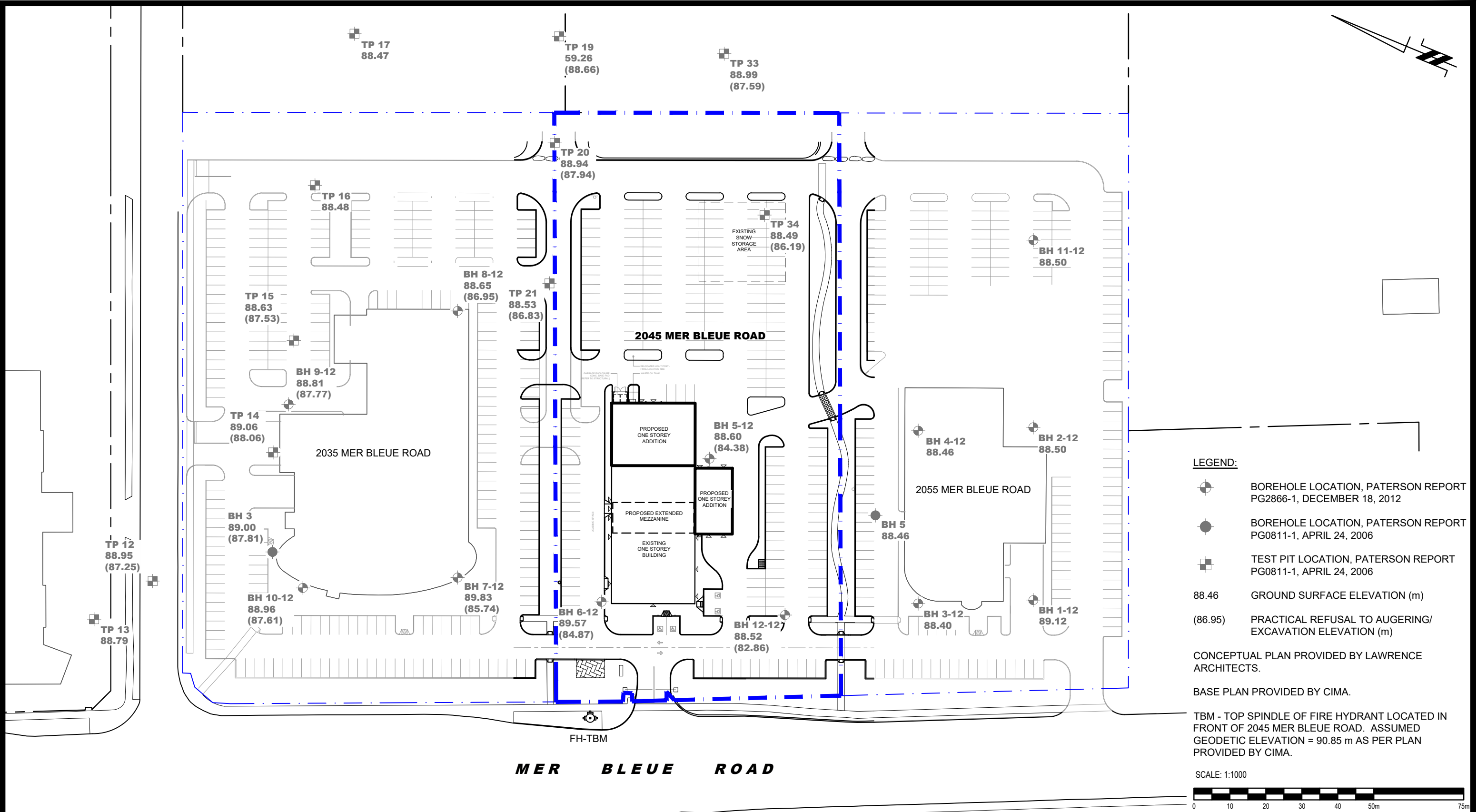
Paterson Group Inc.

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Attachments:

- ☐ Drawing PG7933-1 – Test Hole Location Plan
- ☐ Previous Geotechnical Report





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GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING ADDITIONS
2045 MER BLEUE ROAD

OTTAWA,
Title:

ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:1000	Date:	03/2026
Drawn by:	ZS	Report No.:	PG7933-1
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PREVIOUS AND EXISTING PATERSON GROUP GEOTECHNICAL REPORT

PG2866-1 DATED DECEMBER 18, 2012

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Studies

Geotechnical Investigation

Proposed Commercial Development
2025, 2035 and 2107 Mer Bleue Road
Ottawa, Ontario

Prepared For

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Report: PG2866-1

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APPENDICES

Appendix 1 Soil Profile and Test Data Sheets
 Symbols and Terms

Appendix 2 Figure 1 - Key Plan
 Drawing PG2866-1 - Test Hole Location Plan

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by CIMA to conduct a geotechnical investigation for the proposed commercial development to be located at 2025, 2035 and 2107 Mer Bleue Road, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes and from existing soils information.
- ☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 PROPOSED DEVELOPMENT

It is our understanding that proposed development consists of three (3) commercial buildings of slab on grade construction along with associated car parking areas, access lanes and landscaped areas. It is further understood that the subject site will be serviced by municipal water and sewer.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the present geotechnical investigation was carried out on December 12 and 13, 2012. At that time, a total of twelve (12) boreholes were placed across the subject site. Boreholes and test pits completed as part of a previous investigation for the subject site are detailed in the present report. The locations of the test holes are shown on Drawing PG2866-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or auger flights. All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Flexible standpipes were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground services and site features. The location and ground surface elevation at the test hole locations were surveyed by Paterson personnel. The ground surface elevations at the test hole locations were reference to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located to the west of the subject site. A geodetic elevation of 90.85 m was provided for the TBM. The TBM location, test holes and the ground surface elevations at the test hole locations are presented on Drawing PG2866-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site consists of agricultural land and two (2) residential lots. Based on our observations, the existing foundations of the former residential buildings are still intact. The ground surface across the subject site is relatively flat and slight lower than Mer Bleue Road. Bedrock outcrops were observed along the east property boundary.

The site is bordered to the north, east and south by vacant agricultural land and to the west by Mer Bleue Road.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of topsoil underlain by a very stiff to stiff weathered silty clay crust overlying a stiff to firm grey silty clay and/or bedrock. Practical auger refusal was encountered at BH 5 to BH 10 and BH 12 at depths ranging between 1 to 4.7 m.

Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for details of the soil profiles encountered at the test hole locations.

Based on available geological mapping, the bedrock in this area consists of interbedded silty dolostone, limestone and shale of the Gull River formation and limestone and shale of the Lindsay Formation with an overburden drift thickness of 5 to 10 m depth.

4.3 Groundwater

Based on the moisture levels, colouring and consistency of the recovered soil samples, the long-term groundwater is expected between 2 to 3 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed commercial development. It is expected that the proposed buildings can be founded by conventional style shallow foundations. However, due to the presence of a silty clay deposit within the south portion of the subject site, a permissible grade raise restriction will be required where footings are founded over a silty clay bearing surface.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the buildings areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Bedrock Removal

If bedrock removal is required, consideration should be given to line-drilling in conjunction with hoe-ramming or controlled blasting. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system (if required).

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, stiff silty clay bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A permissible grade raise restriction of 2 m is recommended where footings are founded over the silty clay deposit.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on a clean surface sounded bedrock surface at the proposed founding elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa** incorporating a geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

Based on the subsoil profile encountered across the subject site, foundation design at the subject site can be designed using a seismic site classification **Class C** according to Table 4.1.8.4.A of the Ontario Building Code 2006. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the native soil surface, free of deleterious and organic materials, will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of an OPSS Granular A crushed stone.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car parking areas and access lanes.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - HL 3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 2 - Recommended Pavement Structure - Truck Parking and Access Lanes	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

Landscaping Around Hard Surfaces

Where trees and other vegetation are proposed to be in close proximity to hard surfaces, such as sidewalks or pavement structure, it is recommended that the vegetation be planted in CU-Structural Soil or equivalent product. Under areas of hard surfaces, the CU-Structural Soil should be compacted to 100% of the material's SPMDD using suitable compaction equipment.

The CU-structural soil should be extended to at least 1 m below grade and extend to a 3 m radius around the trees or vegetation.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

Frost Susceptibility of Bedrock

When bedrock is encountered above the proposed founding depth and soil frost cover is less than 1.5 m, the frost susceptibility of the bedrock should be determined. This can be accomplished as follows:

- ☐ Drill probeholes within the bedrock and assess its frost susceptibility.
- ☐ Examine service trench profiles extending in bedrock in the vicinity of the foundation to determine if weathering is extensive.

If the bedrock is considered to be **non-frost susceptible**, the footings can be poured directly on the bedrock without any further frost protective measures.

If the bedrock is considered to be **frost susceptible**, the following measures should be implemented for frost protection:

- ☐ Option A - Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.
- ☐ Option B - Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to potential undulating bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weathered bedrock.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by "cut and cover" methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use the organic free, moist (not wet) overburden material above the cover material. Wet materials will be difficult to re-use as their high water contents make compacting these materials impractical without an extensive drying period. All stones greater than 300 mm in their longest dimension and other deleterious materials should be removed prior to reusing these materials.

Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 300 mm or larger in their longest dimension are removed. Where blast rock is used a blinding layer (OPSS Granular A crushed stone) or a geotextile may be required above the blast rock to reduce the loss of fine particles within the voids of the rockfill.

Based on the soil profile encountered, the subgrade for the services will be placed in both bedrock and in overburden soils. It is recommended that the subgrade medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment should be provided where the bedrock slopes at more than 3H:1V. At these locations, the bedrock should be excavated and extra bedding be placed to provide a 3H:1V (or flatter) transition from the bedrock subgrade towards the soil subgrade. This treatment reduces the propensity for bending stress to occur in the service pipes.

Trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site where services are completed within the silty clay deposit, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOE.

6.6 Winter Construction

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur. Precautions should be taken if winter construction is considered for this project.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

7.0 RECOMMENDATIONS

It is recommended that the following be carried out once the master plan and site development are determined:

- ☐ Review detailed grading plan(s) from a geotechnical perspective.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to placing backfilling materials.
- ☐ Field density tests to ensure that the specified level of compaction has been achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than CIMA or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Richard Groniger, C. Tech.



David J. Gilbert, P.Eng.

Report Distribution:

- ☐ CIMA (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
2025, 2035, 2107 Mer Bleue Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

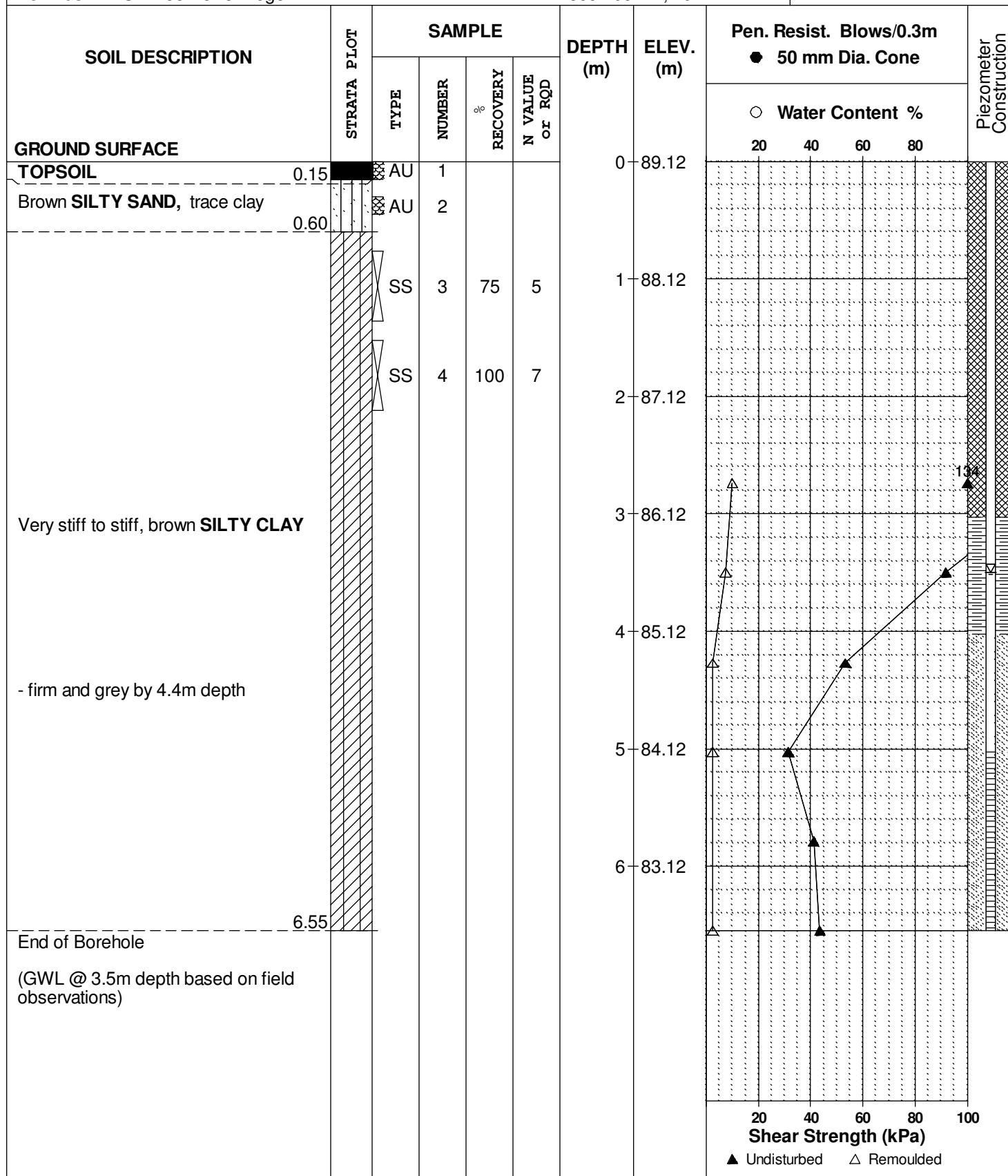
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 12, 2012

FILE NO. PG2866

HOLE NO. BH 1-12



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
2025, 2035, 2107 Mer Bleue Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

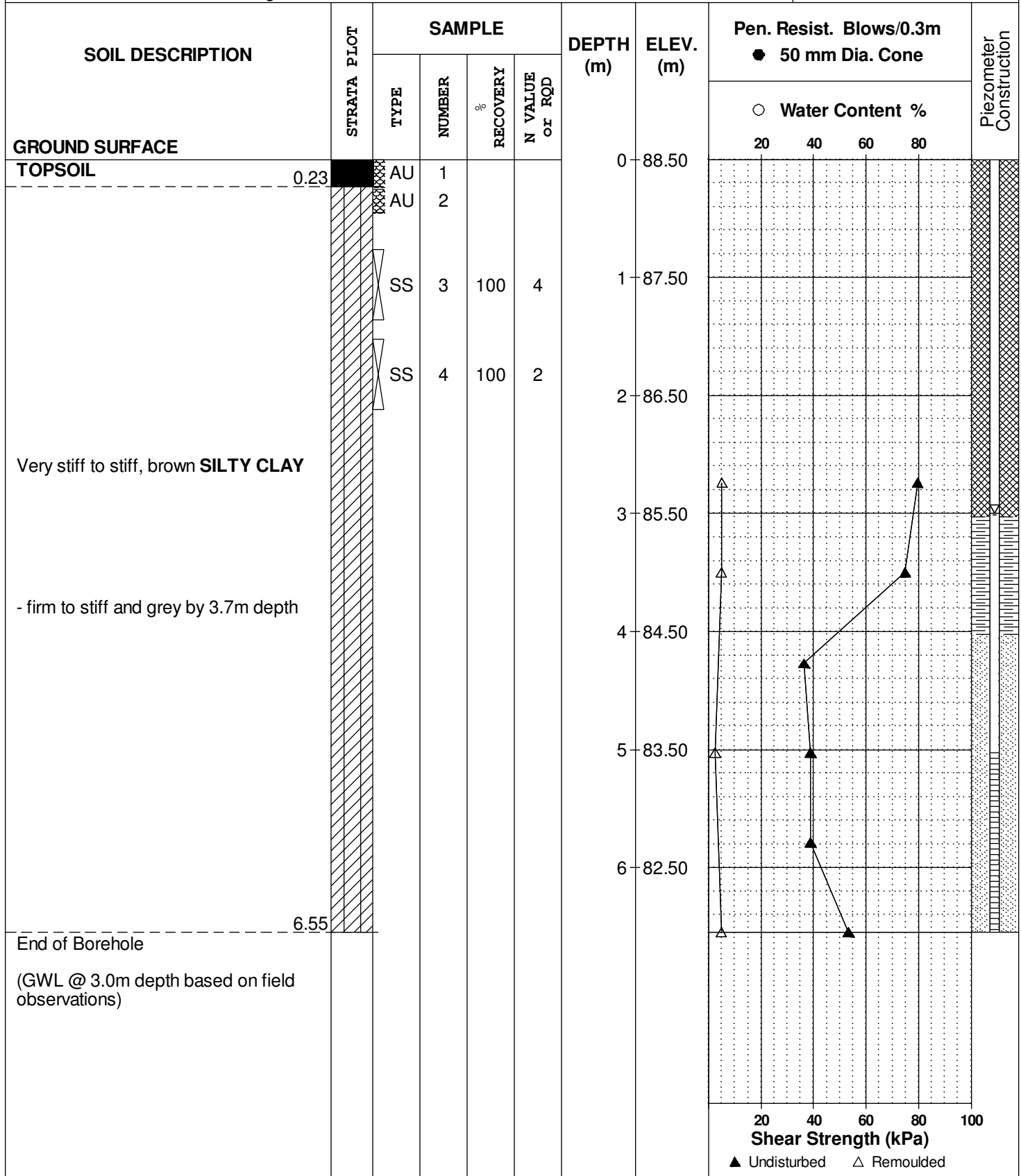
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 12, 2012

FILE NO. PG2866

HOLE NO. BH 2-12



DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

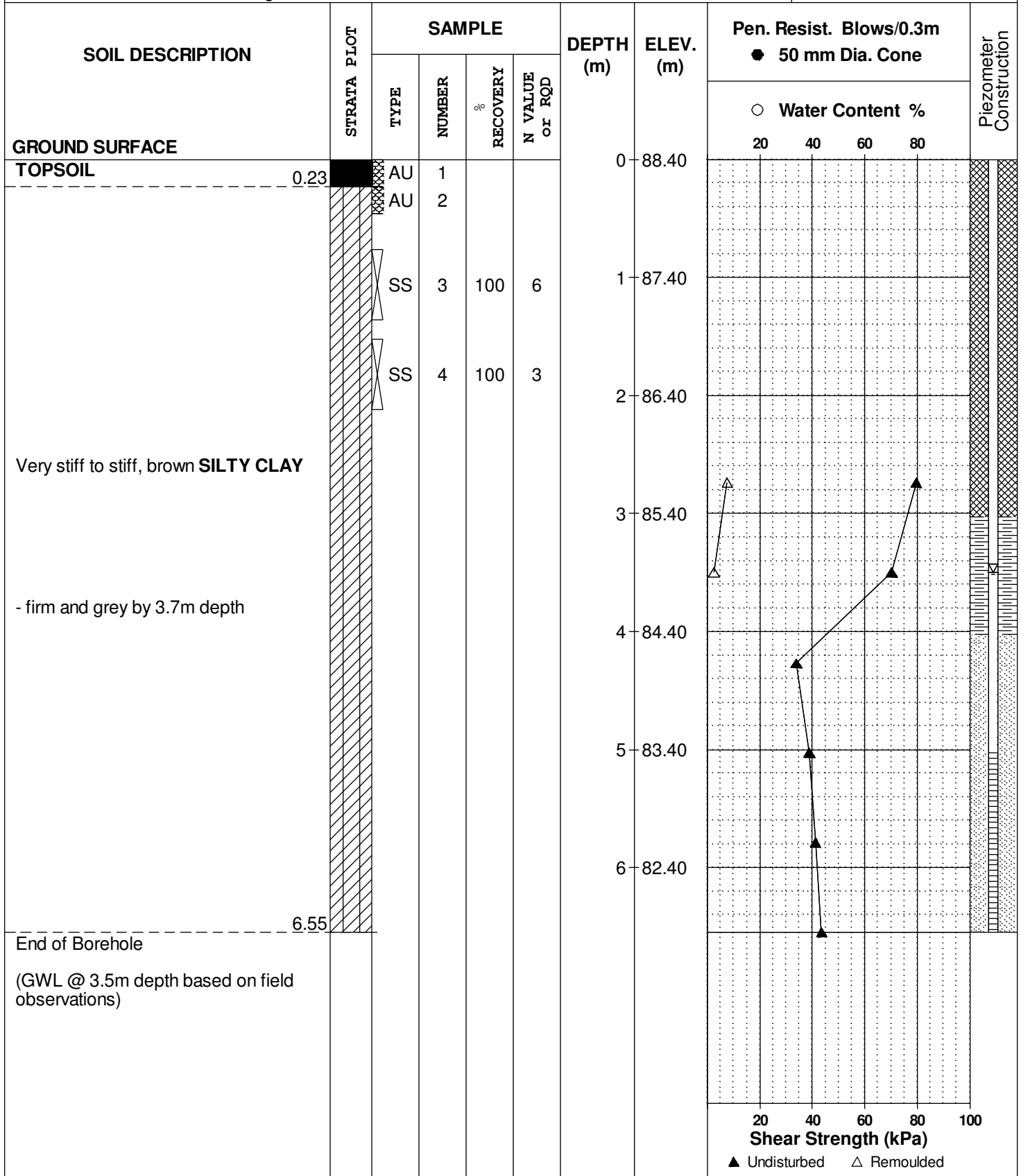
REMARKS

FILE NO.
PG2866

HOLE NO.
BH 3-12

BORINGS BY CME 55 Power Auger

DATE December 12, 2012



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
2025, 2035, 2107 Mer Bleue Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

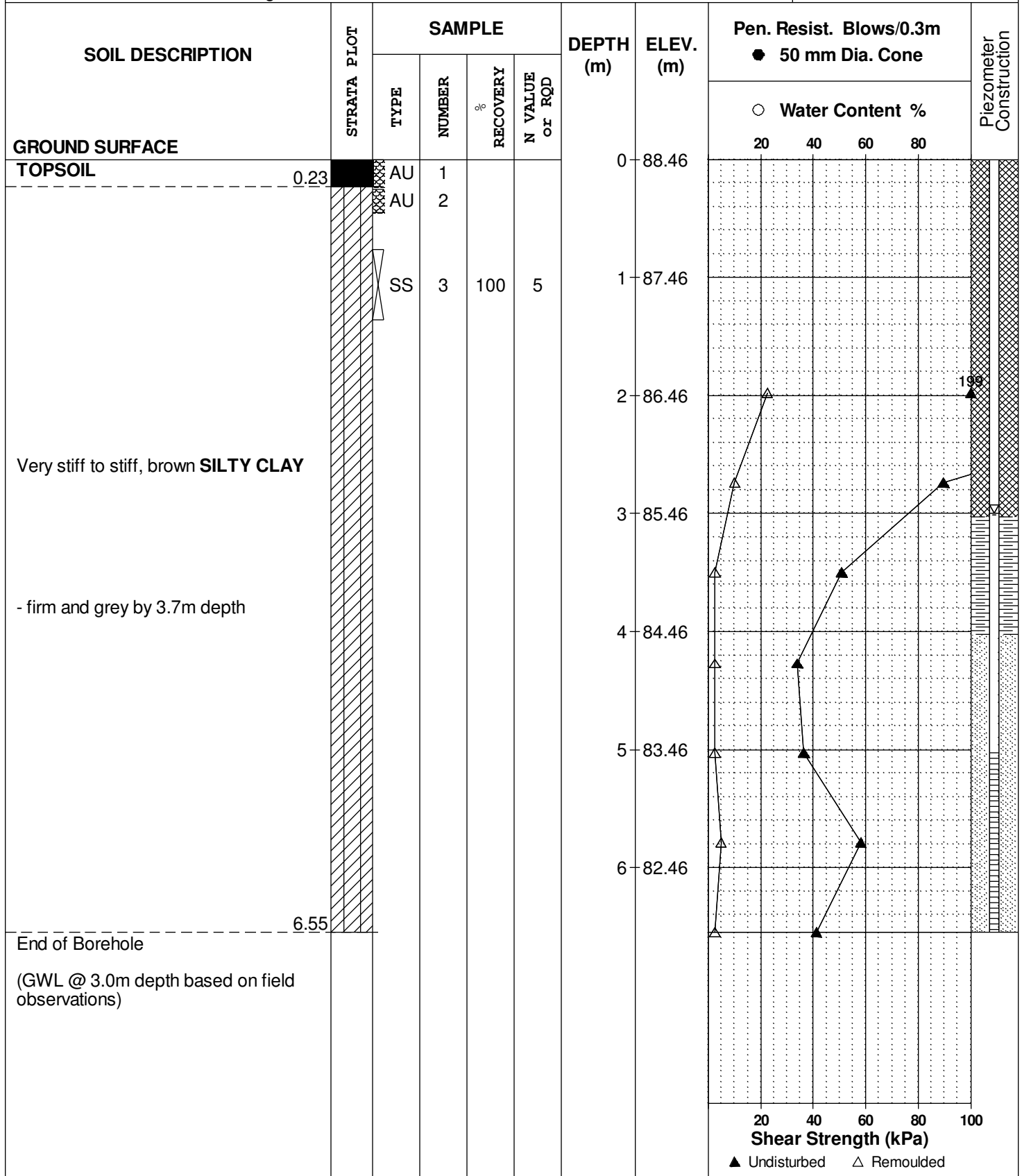
REMARKS

FILE NO.
PG2866

HOLE NO.
BH 4-12

BORINGS BY CME 55 Power Auger

DATE December 12, 2012



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
2025, 2035, 2107 Mer Bleue Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

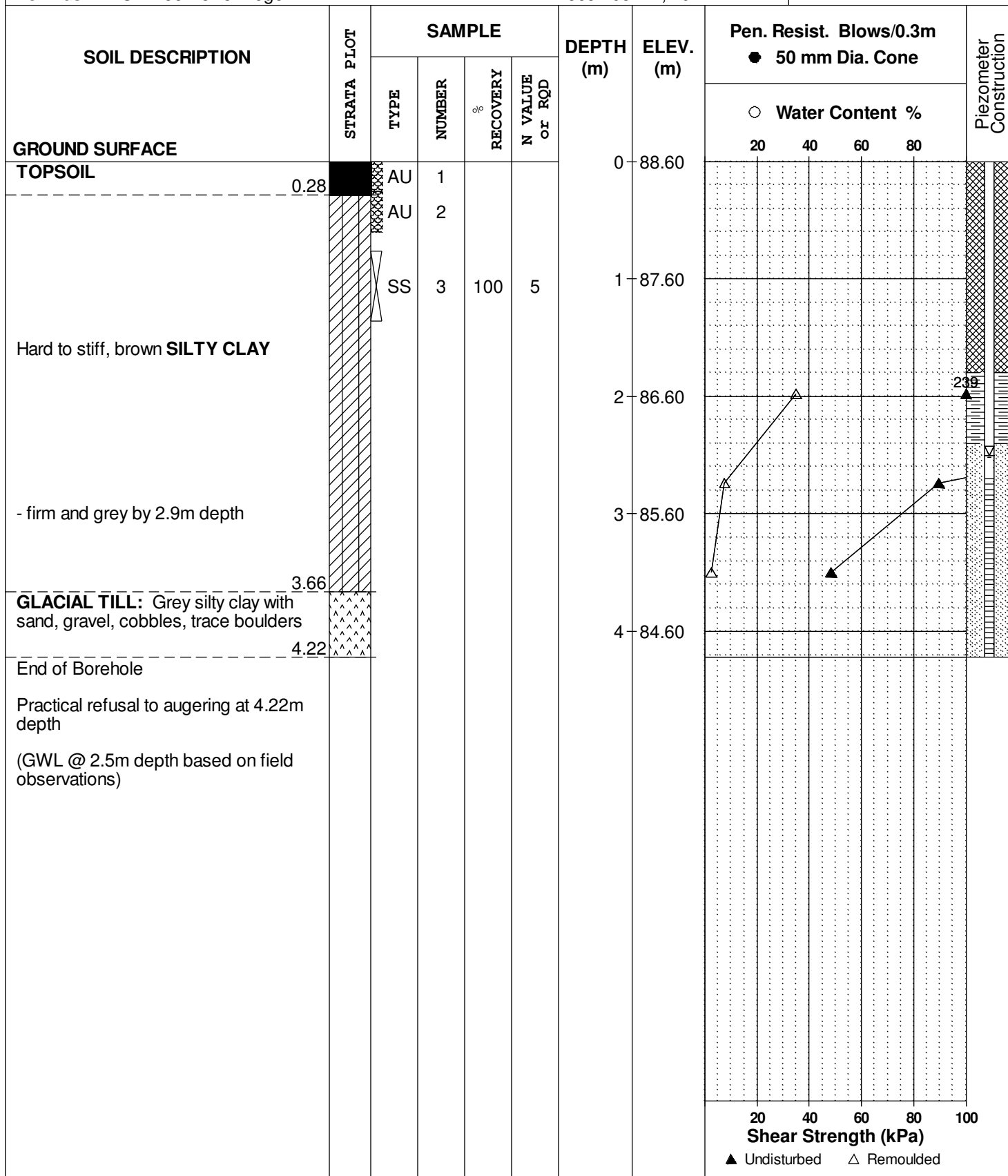
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 12, 2012

FILE NO. PG2866

HOLE NO. BH 5-12



DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

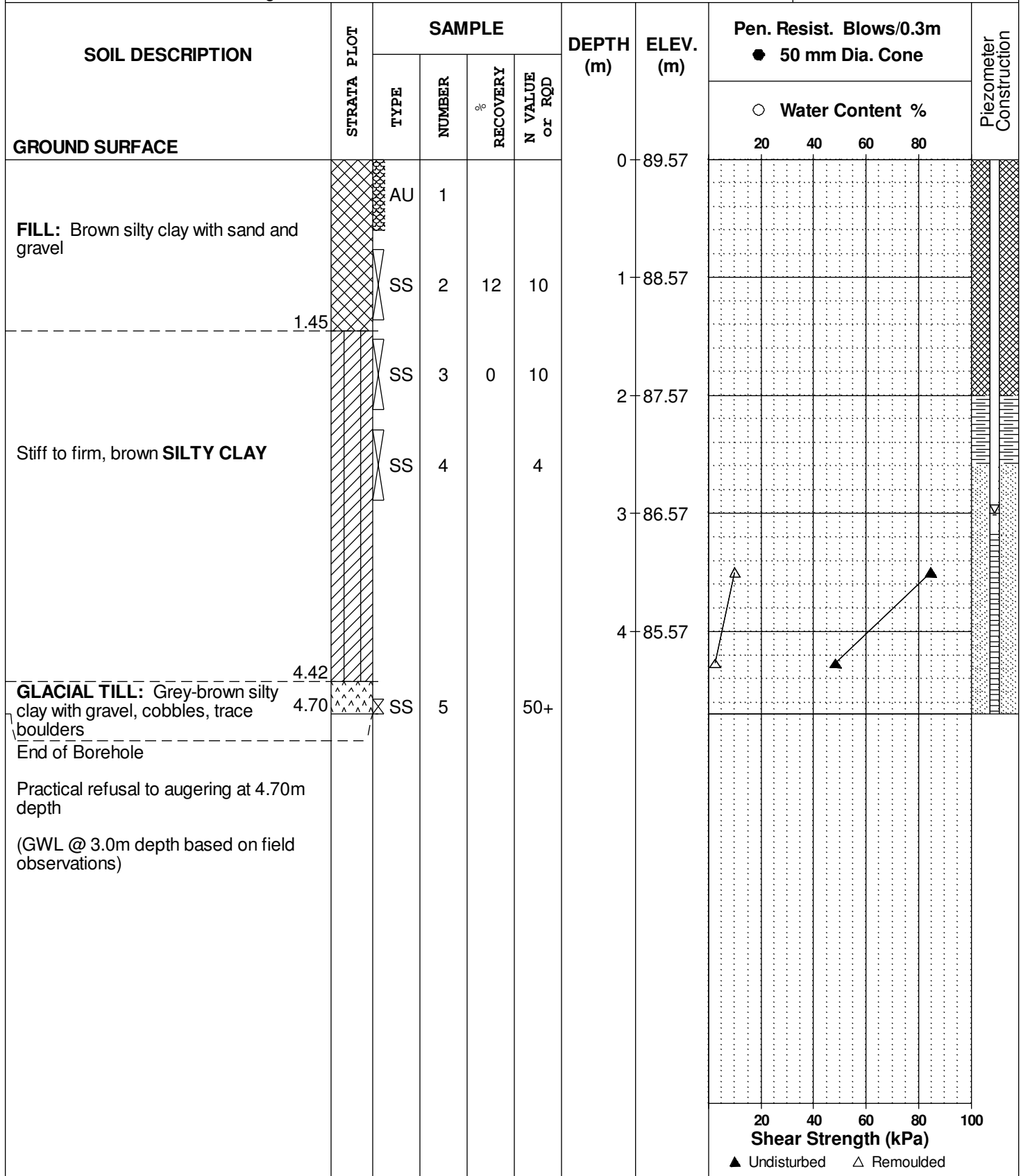
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 12, 2012

FILE NO. PG2866

HOLE NO. BH 6-12



SOIL PROFILE AND TEST DATA

FILE NO. PG2866

HOLE NO. **BH 7-12**

REMARKS

DATE December 12, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
FILL: Brown silty sand with gravel ----- 0.60	[Pattern]	AU	1			0	89.83					
FILL: Grey-brown silty clay, trace sand ----- 1.45	[Pattern]	SS	2	62	6	1	88.83					
Very stiff, brown SILTY CLAY ----- 2.90	[Pattern]	SS	3	100	10	2	87.83					
GLACIAL TILL: Grey-brown silty clay with sand, gravel, cobbles, trace boulders ----- 4.09	[Pattern]	SS	4	8	19	3	86.83					
End of Borehole Practical refusal to augering at 4.09m depth (GWL @ 2.5m depth based on field observations)		SS	5		50+	4	85.83					

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

HOLE NO. **BH 8-12**

DATE December 12, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
Very stiff to stiff, brown SILTY CLAY		AU	1			0	88.65					
		SS	2	100	8	1	87.65					
		SS	3		50+							
End of Borehole	1.70											
Practical refusal to augering at 1.70m depth (BH dry upon completion)												

SOIL PROFILE AND TEST DATA

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

HOLE NO. **BH 9-12**

DATE December 12, 2012

[illegible]

SOIL PROFILE AND TEST DATA

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

HOLE NO. **BH10-12**

DATE December 12, 2012

[illegible]

DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

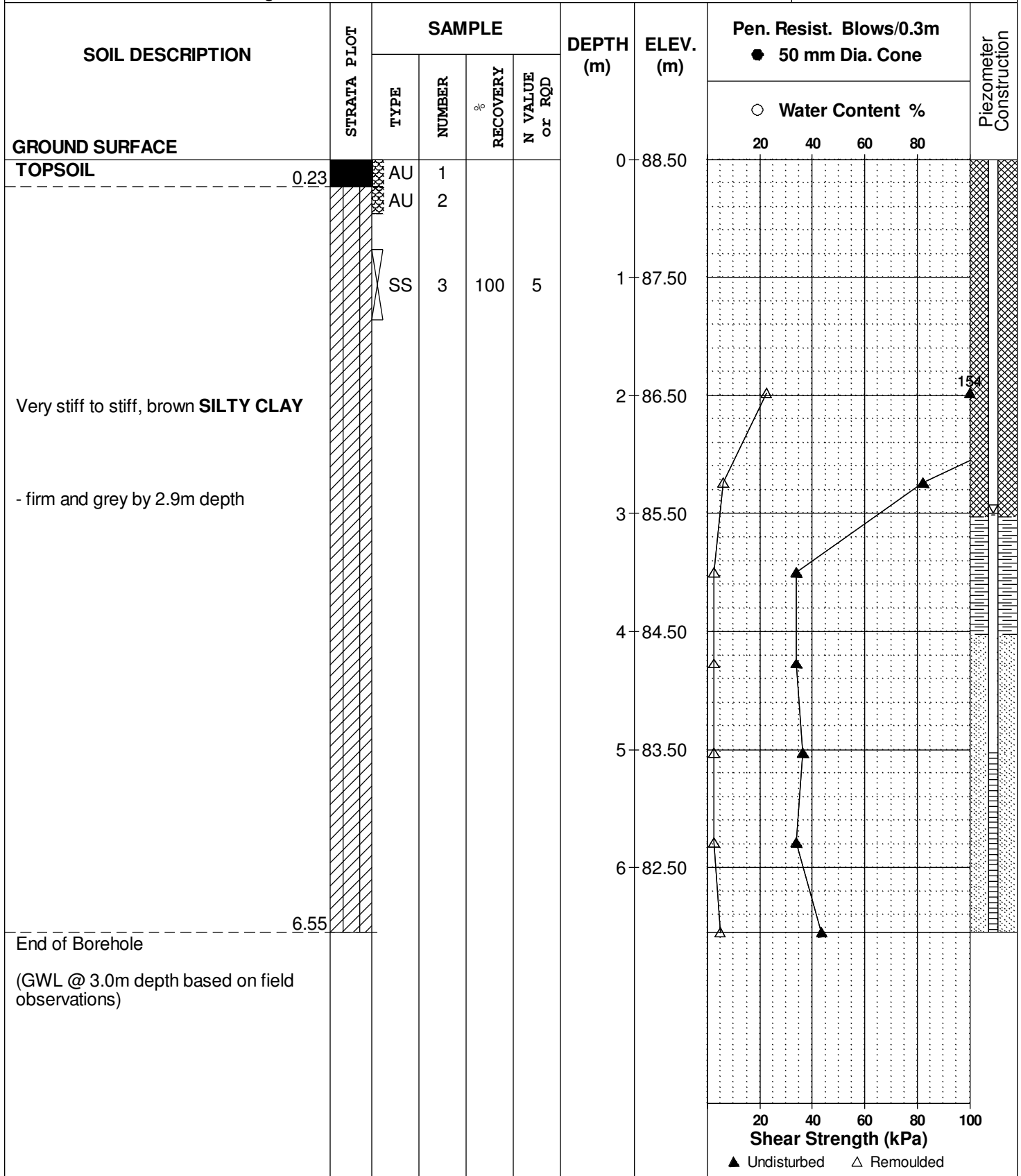
REMARKS

FILE NO.
PG2866

HOLE NO.
BH11-12

BORINGS BY CME 55 Power Auger

DATE December 12, 2012



DATUM TBM - Top spindle of fire hydrant located on Mer Bleue Road, in front of subject site.
Assumed geodetic elevation = 90.85m, as per plan provided by CIMA.

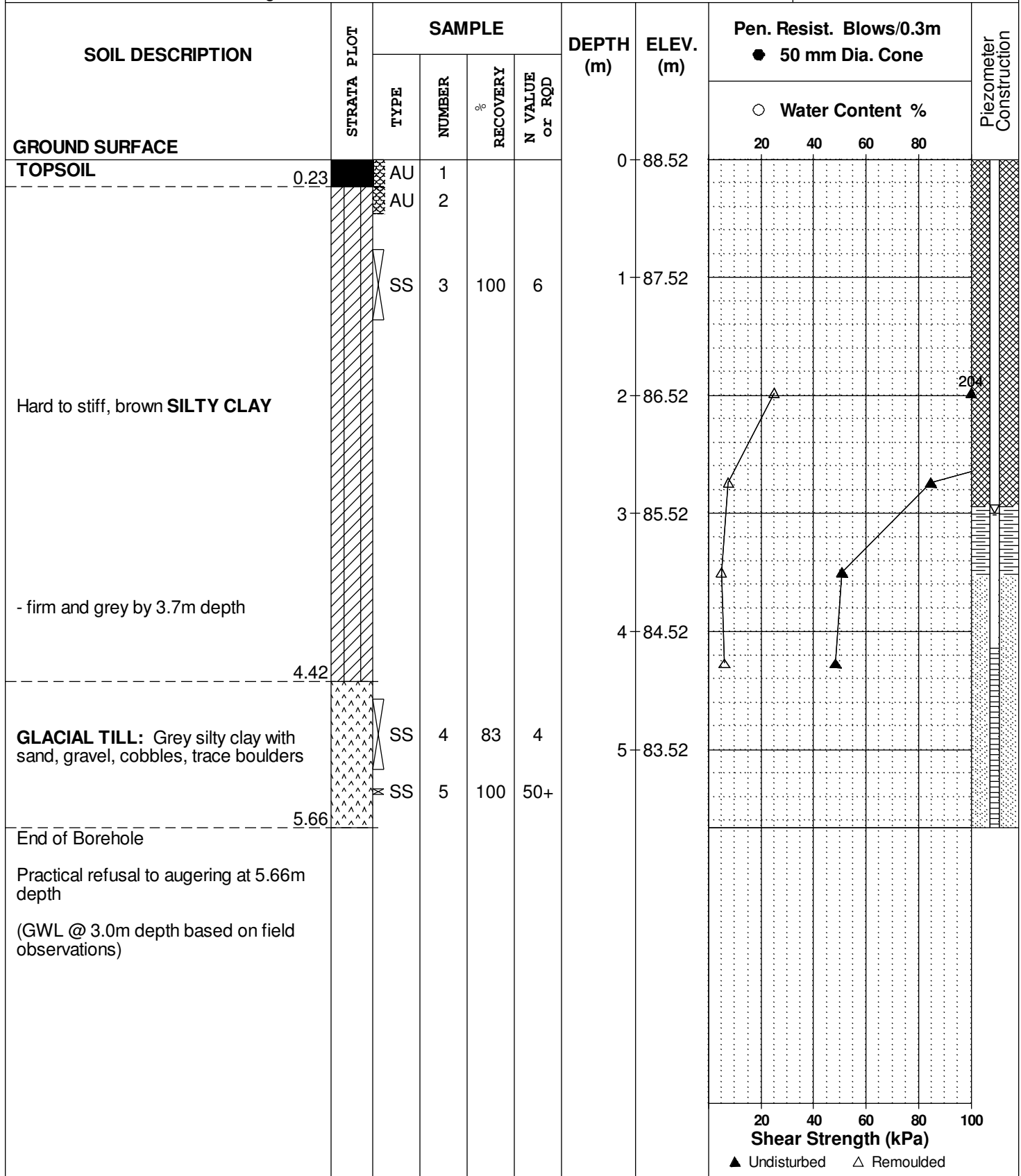
REMARKS

BORINGS BY CME 55 Power Auger

DATE December 12, 2012

FILE NO. PG2866

HOLE NO. BH12-12



SOIL PROFILE & TEST DATA

DATUM Geodetic, as provided by Stantec Consulting Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
TOPSOIL ----- 0.23	[Pattern]					0	89.00					
Very stiff, brown SILTY CLAY ----- 0.76	[Pattern]											
GLACIAL TILL: Dense, brown silty sand with clay, gravel, cobbles and boulders ----- 1.19	[Pattern]	SS	1	67	50 +	1	88.00					
End of Borehole												
Practical refusal to augering @ 1.19m depth												
(GWL @ 0.45m-Apr. 12/06)												

Shear Strength (kPa)
 ▲ Undisturbed △ Remoulded

DATUM Geodetic, as provided by Stantec Consulting Ltd.

FILE NO.

PG0811

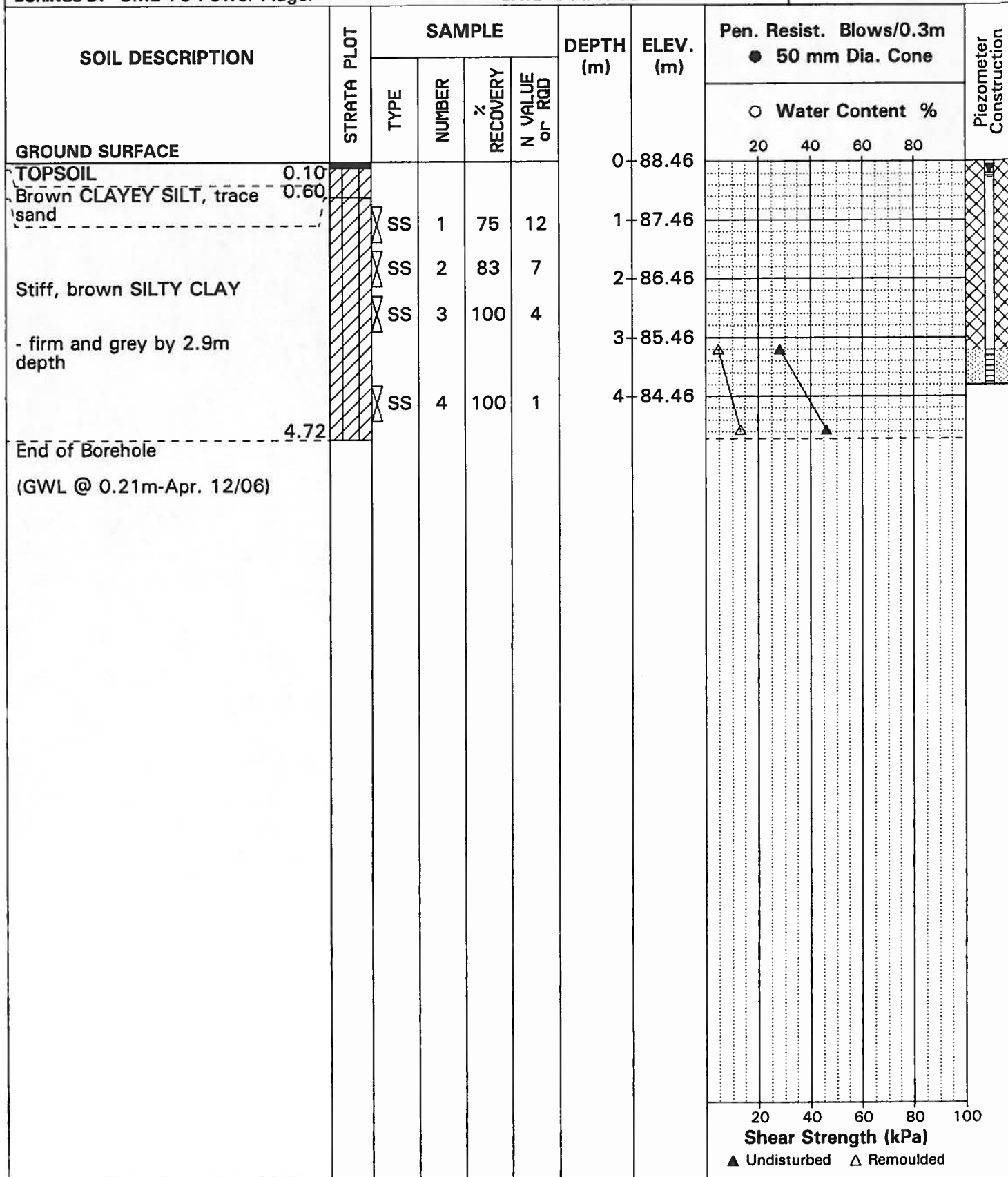
REMARKS

HOLE NO.

BH 5

BORINGS BY CME 75 Power Auger

DATE 5 APR 06



SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleu Road
Ottawa, Ontario**

FILE NO. PG0811

HOLE NO. TP12

DATE 12 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	88.95					
TOPSOIL	0.20											
GLACIAL TILL: Brown silty clay with sand, gravel and boulders						1	87.95					
End of Test Pit	1.70											
Bedrock @ 1.7m depth, east of test pit and 0.3m depth at west of test pit (Open hole GWL @ 1.6m depth)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleeu Road
Ottawa, Ontario**

FILE NO.

PG0811

HOLE NO.

TP13

DATE 12 APR 06

[illegible]

DATUM Geodetic, as provided by Stantec Consulting Ltd.

FILE NO.

PG0811

REMARKS

HOLE NO.

TP15

BORINGS BY Backhoe

DATE 12 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE						0	88.63						
TOPSOIL	0.15												
Very stiff, grey-brown SILTY CLAY		G	1										
End of Test Pit	1.10					1	87.63						
TP terminated on bedrock surface @ 1.10m depth (Open hole GWL @ 1.0m depth)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleeu Road
Ottawa, Ontario**

FILE NO.

PG0811

HOLE NO.

TP16

DATE 12 APR 06

[illegible]

FILE NO.

REMARKS

HOLE NO.

BORINGS BY Backhoe

DATE 12 APR 06

[illegible]

SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleu Road
Ottawa, Ontario**

FILE NO. PG0811

HOLE NO. TP20

DATE 12 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	88.94					
TOPSOIL												
- - - - - 0.30												
Stiff, brown SILTY CLAY												
- - - - - 1.00												
End of Test Pit						1	87.94					
TP terminated on bedrock surface @ 1.00m depth (TP dry upon completion)												

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleue Road
Ottawa, Ontario**

FILE NO.

PG0811

HOLE NO.

TP21

DATE 12 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	88.53					
TOPSOIL												
----- 0.30												
Stiff, grey-brown SILTY CLAY												
						1	87.53					
----- 1.70												
End of Test Pit												
TP terminated on bedrock surface @ 1.70m depth (TP dry upon completion)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE & TEST DATA

**Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleu Road
Ottawa, Ontario**

FILE NO. PG0811

HOLE NO. TP33

DATE 12 APR 06

[illegible]

FILE NO. PG0811

HOLE NO. TP34

DATE 12 APR 06

[illegible]

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation 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 layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

Cc and Cu are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$

Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

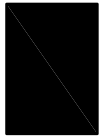
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.
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SYMBOLS AND TERMS (continued)

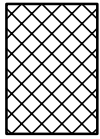
STRATA PLOT



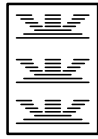
Topsoil



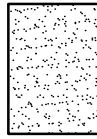
Asphalt



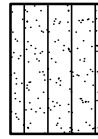
Fill



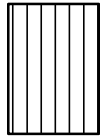
Peat



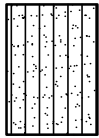
Sand



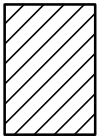
Silty Sand



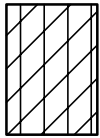
Silt



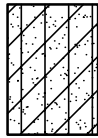
Sandy Silt



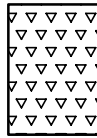
Clay



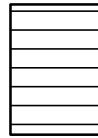
Silty Clay



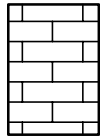
Clayey Silty Sand



Glacial Till



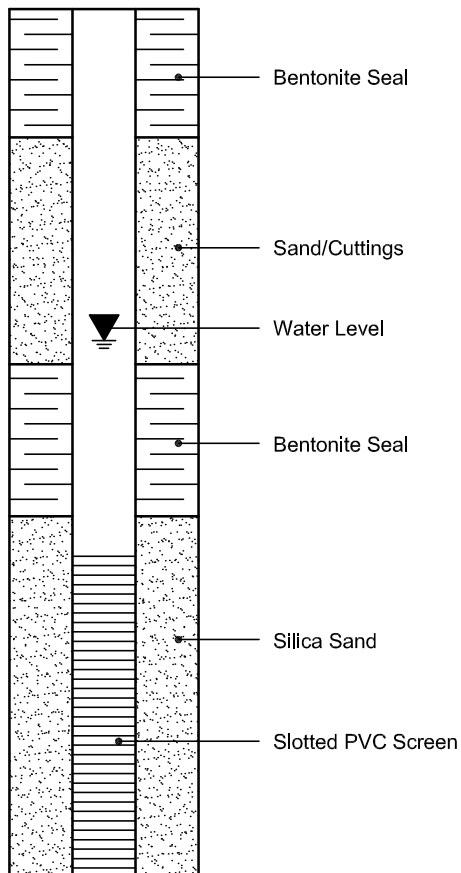
Shale



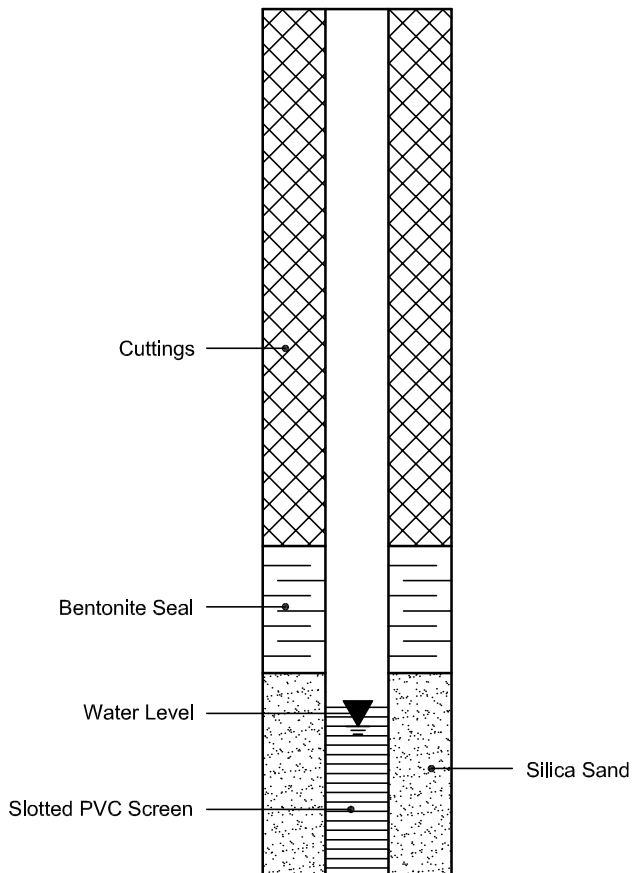
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2866-1 - TEST HOLE LOCATION PLAN

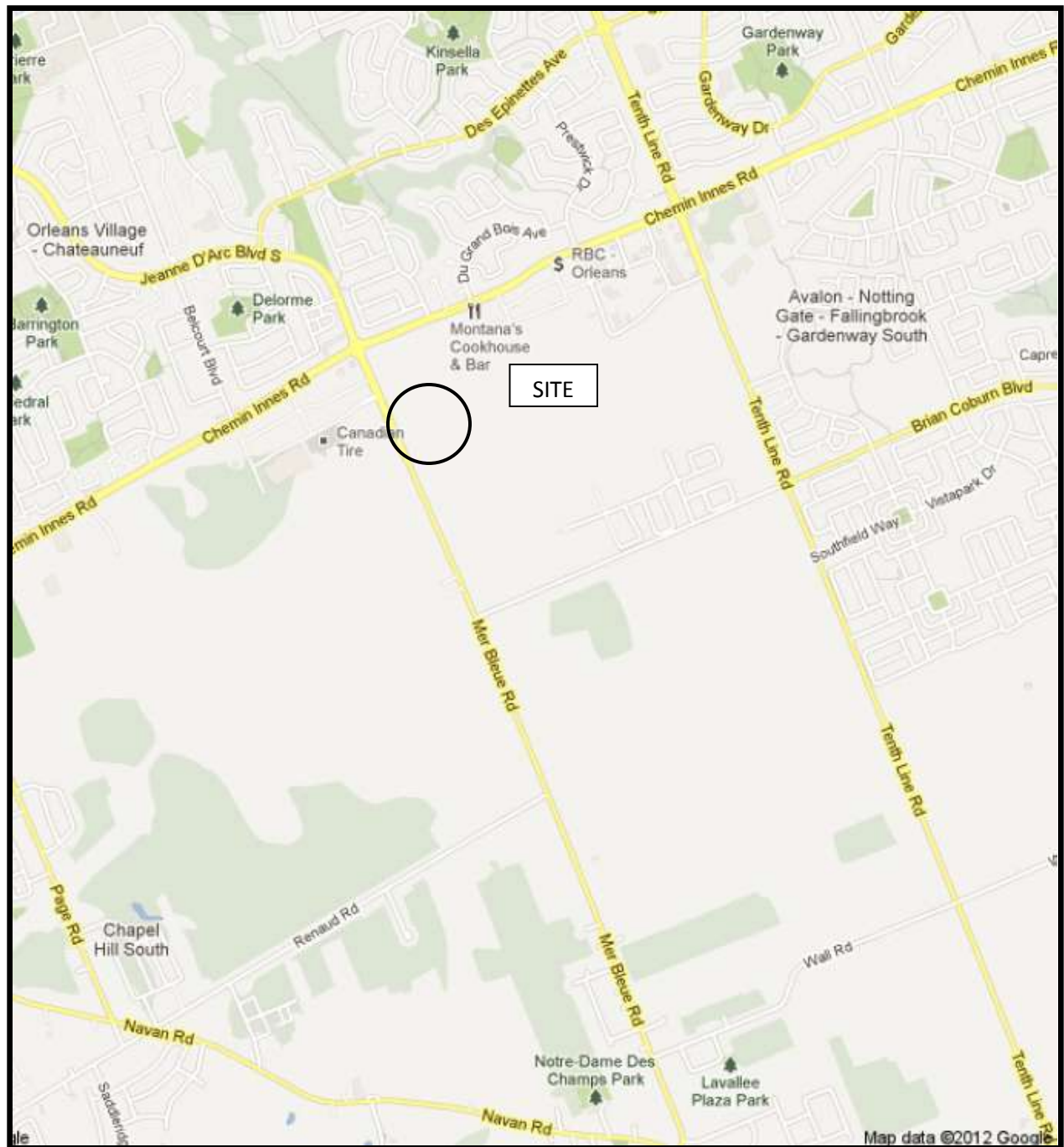
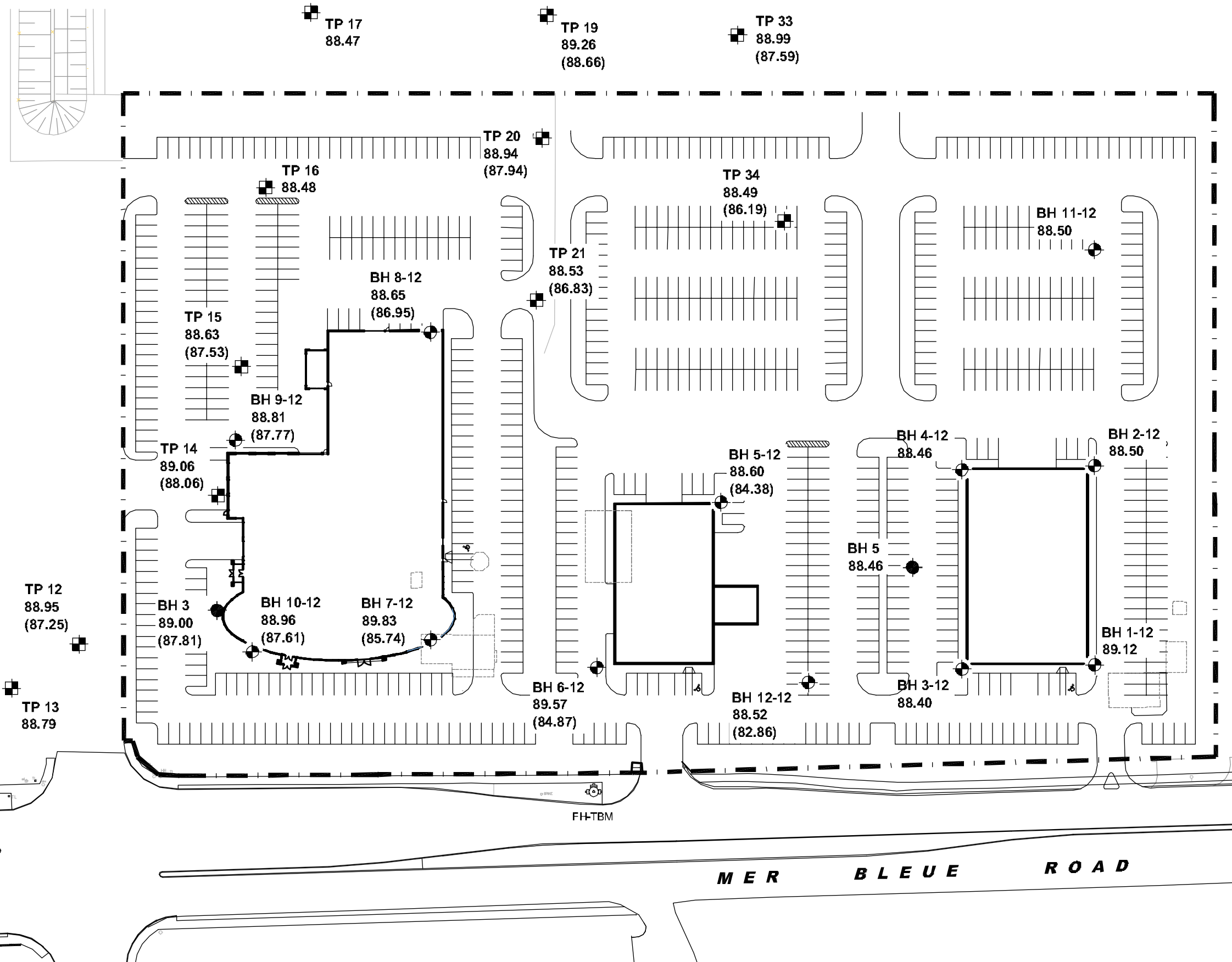


FIGURE 1
KEY PLAN



LEGEND:

- BOREHOLE LOCATION
- BOREHOLE LOCATION, PATERSON GROUP REPORT PG0811-1, APRIL 24, 2006
- TEST PIT LOCATION, PATERSON GROUP REPORT PG0811-1, APRIL 24, 2006
- 88.52 GROUND SURFACE ELEVATION (m)
- (82.86) PRACTICAL REFUSAL TO AUGERING/ EXCAVATION ELEVATION (m)
- EXISTING CONCRETE FOUNDATION

TBM - TOP SPINDLE OF FIRE HYDRANT.
ASSUMED GEODETIC ELEVATION = 90.85m, AS
PER PLAN PROVIDED BY CIMA.

BASE PLAN PROVIDED BY CIMA.



paterson group

consulting engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale: 1:1000
Des.: RG
Dwn: MPG
Chkd: DG

CIMA
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
2025, 2035 & 2107 MER BLEUE ROAD
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.
PG2866-1
Report No.: PG2866-1
Date: 12/2012