Peto MacCallum Ltd.
CONSULTING ENGINEERS

GEOTECHNICAL INVESTIGATION
GUELPH TRANSIT BUS WASH
GUELPH, ONTARIO

for

CITY OF GUELPH

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PML Ref.: 12KF015
Report: 1
May 28, 2012

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CONSULTING ENGINEERS

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Report: 1

Mr. Wayne Galliher, A.Sc.T.
City of Guelph
Water Services Division
29 Water Works Place
Guelph, Ontario
N1E 697

Dear Mr. Galliher

Geotechnical Investigation
Guelph Transit Bus Wash
Guelph, Ontario

Peto MacCallum Ltd. (PML) is pleased to present this report for the above referenced project. Authorization to conduct the work was given via a City of Guelph Purchase Order (No.1206614), dated April 19, 2012. Approval for additional work (the installation of one monitoring well) was given verbally by Mr. Galliher of the City of Guelph on May 4, 2012.

Project Description

The City of Guelph is currently undertaking detailed engineering design for the implementation of a water reuse and rainwater harvesting system at the City's Guelph Transit Facility. This initiative aims to reduce current potable water and wash chemical demands for washing City of Guelph buses by utilizing auxiliary water sources (rainwater), to be captured within underground storage tanks to be installed at the site. The location of the proposed tanks, with respect to the existing buildings are depicted on the appended Borehole Location Plan, Drawing 1.

The purpose of the geotechnical investigation was to explore the subsurface soil and groundwater conditions at the site. Based on the investigation findings we have prepared this report, which provides geotechnical design and construction recommendations for the planned work. Specific considerations being addressed include:

- Existing subsurface conditions (soils and groundwater);
- Foundations for underground concrete precast storage tanks, including ultimate limit state (ULS) and serviceability limit state (SLS) bearing resistance of onsite soils, recommended foundation systems, founding depth(s), settlement projections and underground tank anchoring;

- Excavating and backfilling, including construction dewatering, safe slope inclinations, suitability of native soils for reuse as backfill, pipe bedding, engineered structural fill placement and braced excavation requirements;

- Long-term groundwater control measures, including hydrostatic uplift considerations;

- Soil corrosivity and aggressiveness to concrete / recommended concrete type;

- Potential impacts of construction dewatering on nearby infrastructure; and,

- Structural pavement restoration of roads and disturbed areas.

The comments and recommendations provided in this report are based on the site conditions at the time of the investigation, and are applicable only to the proposed works as described in the report. Therefore, once the development plans (i.e., depth of tanks) are finalized, PML will require a review to asses the validity of the report.

Investigation Methodology

The field work for the geotechnical investigation was conducted on May 4, 2012 and comprised 2 boreholes drilled to depths of 5.80 m below existing grades. Due to the presence of groundwater within the depths of exploration, a monitoring well was installed in Borehole 2, to better characterize the groundwater table. The boreholes were located in the general area of the proposed tanks, as shown on the appended Borehole Location Plan, Drawing 1.

The boreholes were advanced using a CME 75 track mounted drillrig, equipped with continuous flight hollow stem augers. The drilling equipment was supplied and operated by a specialist drilling contractor.

Representative samples of the overburden were recovered throughout the depths explored. Standard penetration tests were conducted simultaneously with split spoon sampling operations to assess the strength characteristics of the subsurface strata.

The field work was supervised throughout by a member of PML's engineering staff who, directed the drilling and sampling operations, prepared the stratigraphic logs, monitored groundwater conditions and processed the samples obtained.

The borehole locations were established in the field by PML. Upon completion, the borehole locations were surveyed by PML. The ground surface elevations at the boreholes were referenced to the following temporary benchmark (TBM):

TBM: Top of office floor slab
Location shown on the appended Borehole Location Plan
Elevation: 329.60 (geodetic, metric)

The samples obtained from the investigation were brought to our laboratory for further detailed visual examination and natural moisture content determinations. Furthermore, the laboratory work included two particle size distribution analyses and one moisture density relationship. One sample of soil was submitted for chemical analyses to determine soil corrosivity parameters.

Summarized Subsurface Conditions

Reference is made to the appended Log of Borehole sheets for details of the drilling work including soil descriptions, inferred stratigraphy, standard penetration N values, groundwater observations during and upon completion of drilling, and natural moisture content determination test results.

In general, the subsurface stratigraphy encountered in the boreholes comprised pavement structure materials underlain by fill and a sand and gravel deposit that typically extended to the borehole termination depths.

Surficial pavement structure materials comprised between 120 and 130 mm of asphalt overlying between 130 and 150 mm of granular base type material.

Fill soils were encountered beneath the pavement structure in both boreholes. The fill soils extended to between 3.10 and 3.20 m below grade, corresponding to an elevation of about 325.8. The fill deposits typically comprised either sand and gravel, or silt. A 100 mm thin layer of topsoil was penetrated in Borehole 2, at a depth of 2.30 m within the fill layer. Some level of compactive effort appears to have been made during the placement of the fill soils, based on standard penetration 'N' values of between 22 and 50 blows per 0.30 m of penetration of the split spoon sampler. A Proctor moisture density relationship was completed on a sample of the fill (Figure 1, appended). The results of the proctor revealed a maximum dry density of 2290 kg/m^3 at an optimum moisture content of 6.0%. A particle size distribution chart of the same sample (Figure 2) shows the sample to be comprised of sand and gravel, trace silt. Based on the gradation test results, the sample conforms to Granular B Type I specifications.

Underlying the pavement structure and fill soils, a deposit of native sand and gravel was contacted. The sand and gravel was observed to have trace silt, and was noted to be compact to very dense based on standard penetration N values of between 30 and 86 blows. This deposit was generally observed to be saturated. A particle size distribution was completed on the native sand and gravel (Figure 3, appended). The chart shows that the native sand and gravel is similar in composition to the sand and gravel fill. The hydraulic conductivity (K) of this deposit was estimated using the following expression (Hazen, 1911):

$$K=Cd_{10}^2$$

where:

K = hydraulic conductivity (cm/s)
C = constant = 100
D_{10} = effective grain size (cm) where 10% of particles are finer and 90% are coarser.

Based on the results of the laboratory particle distribution analysis, the hydraulic conductivity (K) of the water-bearing sand and gravel stratum was estimated to be in the order of 1 x 10^{-2} cm/s.

Groundwater observations carried out in the open boreholes during and upon completion of drilling are summarized on the appended Log of Borehole Sheets. Groundwater was encountered in both boreholes during drilling, between depths of 3.35 and 3.60 m. The groundwater level was measured within the monitoring well installed in Borehole 2 on May 11, 2012. The water level readings identified water at a depth of 3.40 m, corresponding to an elevation of 325.50 (metric, geodetic). Seasonal and / or weather dependant groundwater fluctuations should be anticipated.

Discussion and Recommendations

Two underground tanks are proposed to be installed. The tanks will either be 5000 or 8000 imperial gallons (22,730 or 36,370 L). The tanks will be 6.2 m long by 2.6 m wide by either 2.0 or 2.9 m deep (for the 5000 or 8000 gallon tanks, respectively). It was indicated that it would be preferred that the invert of the tanks be located at a depth of approximately 4.0 m below existing grade.

Excavation and Groundwater Control

It is anticipated that excavations of up to 5 m will be required for the tank construction, based on the assumption that the invert of the tanks will be located at about 4.0 m below grade.

It is noted that groundwater was measured in the monitoring well installed in Borehole 2 at a depth of 3.40 m (elevation of 325.50). As discussed above, the hydraulic conductivity of the water bearing sand and gravel soils is estimated to be 1 x 10^{-2} cm/s. Based on the estimated hydraulic conductivity of this aquifer, excavations that extend below an elevation of 325.50 will likely require significant groundwater control measures. Considering the high permeability of these soils, open cut excavations below 325.50 will not likely be feasible, and groundwater control will likely require the use of a braced excavation utilizing interlocking sheet piling, in conjunction with well point dewatering as well as sump pumping within the excavation.

In accordance with the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, a Permit to Take Water (PTTW) from the Ministry of the Environment (MOE) is required if the dewatering discharge is greater than 50,000 L/day. A PTTW would likely be necessary for any excavations extending below an elevation of 325.5. However, it is noted that further geotechnical work must be completed to address the suitability of using sheet piling to control groundwater, and further hydrogeological work, including pump tests, would be required for the PTTW approval process. PML would be pleased to provide further investigatory work if deemed necessary.

Cognizant of the potential dewatering issues and increased costs associated with locating the tanks at a depth of 4.0 m, consideration should be given to locating tanks such that excavations do not extend beyond a depth of 3.4 m (elevation of about 325.5). If the excavations do not extend into the water bearing sand and gravel, no major groundwater control problems are envisaged. Conventional sump pumping from pits within the excavation should be sufficient to control any groundwater infiltration from the base of the excavation or surface water / perched water entering the excavation.

It is noted that the design of any dewatering system should be left to the Contractor's discretion and that the design and installation of dewatering and / or shoring systems should be carried out by specialists in these fields. At the time of tendering, test pits should be excavated on site to allow prospective contractors to judge the groundwater conditions and to determine the appropriate control methods required closer to the time of construction. Groundwater conditions are subject to seasonal variations. In this regard, a later summer construction schedule would be preferable.

Excavations are expected to generally extend through surficial pavement structures and fill soils, and into native sand and gravel deposits, which are classified as Type 3 Soils as defined under the Ontario Occupational Health and Safety Act (OHSA). Provided adequate groundwater control is achieved, excavations within Type 3 Soil that are to be entered by workers, may not be steeper than one horizontal to one vertical (1H:1V) from the base of the excavation. Workers should not enter an unprotected excavation if there is evidence of ongoing groundwater seepage in the banks. All work should be carried out in accordance with the OHSA and with local regulations.

Based on site limitations, it is anticipated that the use of braced excavations may be considered, to limit the overall size of the excavation. The following parameters may be used for braced excavation calculations:

PARAMETER	SAND AND GRAVEL / SAND AND GRAVEL FILL (GRANULAR B TYPE)
Angle of Internal Friction (degrees)	32
Unit Weight (kN/m^3)	22.0
Buoyant Soil Weight (kN/m^3)	12.2
Coefficient of Active Earth Pressure (K_a)	0.31
Coefficient of Passive Earth Pressure (K_p)	3.23

*Full hydrostatic pressure and buoyant soil weights should be used below 3.4 m depth.

It will be important to ensure that the excavation(s) do not undermine existing in-ground structures or other services in the proximity of the excavation. The need for underpinning or for a braced excavation can be established according to criteria illustrated in the appended Figure 4. It should be noted that a trench liner box may not be relied on for this purpose. For design of a bracing system in sand and gravel, a soil unit weight of 21 kN/m^3 may be assumed, using a rectangular stress distribution in accordance with methods outlined in the Canadian Foundation Engineering Manual, and summarized on Figure 5. Appropriate factors of safety should be applied.

Foundations

Based on the investigation findings, the insitu native sand and gravel deposits, contacted at an elevation of approximately 325.8 (metric, geodetic), would be considered suitable to support a mat footing. The insitu fill soils would not be considered suitable for the subgrade support of the tanks, due to the risk of ongoing settlement. Footings constructed on the native soils may be designed for a net bearing resistance of 150 kPa at the serviceability limit state (SLS) and a factored bearing resistance of 225 kPa at the ultimate limit state (ULS).

If the tanks are to be located above an elevation of 325.8, the footings may be supported on engineered structural fill. Structural fill should be placed in accordance with the recommendations presented in the 'backfill' section of this report along with the engineered fill construction recommendations provided in Appendix A. Prior to placement of engineered fill, the existing fill soils should be subexcavated to the competent native overburden soils contacted at depths of 3.1 to 3.2 m below existing grades (corresponding to an elevation of about 325.8). For engineering fill supporting footing loads, compaction to a minimum 98% of the materials standard Proctor moisture dry density (SPMDD) should be specified. Footings placed on structural engineered fill may also be designed using a bearing resistance of 150 kPa at the SLS and a factored resistance of 225 kPa at the ULS.

Excavations should not encroach and undercut foundations for the existing building. Based on drawings provided by Enviro-Stewards Inc., ('Wall Sections', Drawings A4.2 and A4.3, dated January 2006), the existing building foundations are anticipated to lie at a minimum depth of 1.20 m below existing grades, however prior to construction the original 'as-built' drawings should be reviewed for the existing buildings to confirm the existing founding depths.

Considering that the founding level of the proposed tanks is expected to be lower than the founding level of the adjacent building, the tanks should be located above an imaginary line drawn down from the existing building footings at an inclination of 10 horizontal to 7 vertical (10H:7V).

If site or design limitations do not allow for placing the tanks an adequate distance from the existing building foundations, the need for underpinning of the existing foundations should be evaluated. In this regard, preconstruction inspection of the existing foundation and the underlying subgrade soils is recommended to determine the general foundation condition. General recommendations regarding underpinning are provided in the 'excavation' section of this report.

All founding surfaces should be examined by PML geotechnical personnel prior to concreting, to confirm that excavations extend through any fill and to ensure that no loose zones exist and that the subgrade soils are capable of supporting the design loads. Any loose areas noted during the inspection should be subexcavated and backfilled with lean concrete.

All exterior footings should be provided with a minimum 1.20 m of earth cover or the thermal insulation equivalent to provide adequate insulation against potential frost damage.

Provided the footings are designed and constructed as outlined above, total settlements should not exceed 25 mm with differential settlements of 50% of this value, which should be within tolerable limits for the planned structure.

Hydrostatic Uplift Design and Lateral Earth Pressure

As discussed, there is potential that the invert of the underground tanks will be located below the groundwater level. Long term groundwater lowering is not considered feasible, hence the tanks must be designed to resist hydrostatic uplift pressures when empty.

The uplift resistance should be calculated considering that the groundwater pressure at the base of the structure will generally be equivalent to the groundwater level rising to an elevation of approximately 327.0 (approximately 1.5 m above the current measured groundwater level). Alternatively, long term monitoring of the groundwater level could be undertaken in the installed monitoring well to determine an anticipated maximum groundwater level for the site. The tanks must be designed to resist uplift forces when empty. The total uplift resistance will be provided by the weight of the structure, plus the weight of backfill over the outside edge of the concrete foundation (provided the mat foundation is part of, or anchored to the precast tank structure). If additional uplift resistance is required, it is recommended that the outside edge of the footings be extended laterally to provide the additional projection needed for uplift resistance. Earth anchors such as helical piers can be incorporated into the foundation system to provide additional restraint to vertical forces, including hydrostatic uplift.

Uplift loads and resistance requirements must be quantified to determine the anchorage requirements of the structure. Cognizant of the uplift capacity requirements, the quantity and configuration of the anchors can be determined. The anchorage capacity of a helical pier is dependent on the individual pier configuration and the properties of the founding soils. Helical pier systems are typically available as a proprietary product. The names of local distributors / installers can be provided if required. Design services are typically included with the product. PML can assist with a review of the helical pier design. Tensile and compressive load testing should be conducted at the installation stage as per the helical pier design consultant recommendations.

If it is not practical to design for hydrostatic pressure, then it will be necessary to provide some sort of pressure relief (flap valves) beneath the structure to equalize groundwater pressures.

The tanks must be designed to resist the lateral earth pressure and water pressure, which may be determined from the following equation:

$P = K [\gamma (h-d_w) + \gamma' d_w + q] + \gamma_w d_w$

where P = total lateral pressure at depth h (m) below ground surface (kPa)
K = lateral earth pressure coefficient of compacted backfill
= 0.5
h = depth below grade (m) at which lateral pressure is calculated
d_w = depth below ground water level at depth h below final exterior grade (m)
= h, when the design water level is at the ground surface
γ = unit weight of soil
= 22 kN/m^3 for compacted sand and gravel at 95% SPMDD
γ_w = unit weight of water
= 9.8 kN/m^3
γ' = buoyant unit weight of soil
= $\gamma - \gamma_w$
= 12.2 kN/m^3
q = vertical stress at depth h due to surcharge loads (kPa)

An appropriate factor of safety must be used in the design.

Pipe Bedding and Backfill

No bearing problems are anticipated for pipes founded in the native mineral soils encountered at the site. On stable subgrade, a minimum 150 mm thick bedding course of Granular A material, conforming to Ontario Provincial Standard Specification (OPSS) 1010, compacted to 95% SPMDD is recommended beneath the pipes. The Granular A material should extend around the pipe to at least 300 mm above the pipe obvert or as set out by OPSS, or the City of Guelph. A greater thickness of bedding material might be required at locations where unstable pipe support subgrade conditions develop.

Most of the onsite pavement structure materials, sand and gravel fills and native sand and gravel soils should be suitable for reuse as engineered fill where compaction to 95% SPMDD is specified, provided they do not become excessively wet or mixed with deleterious materials. Materials described as 'saturated' will likely be too wet to achieve proper compaction, and will have to be dried prior to use, or avoided. The silt fill and sandy silt topsoil fill contacted in Borehole 2 will not be suitable for use as fill under settlement sensitive features and may be used for landscaping purposes only.

Imported backfill material should consist of OPS Granular B Type I material or approved engineered fill. Prior to importing material to the site, the proposed source should be inspected, and the material tested, to check that the required compatibility characteristics are available. Further generic recommendations for engineered fill construction are appended. Backfill should be placed in maximum 150 mm lifts, and should be compacted to at least 95% SPMDD below sidewalks, pavements, or other settlement sensitive features, and to at least 90% SPMDD in landscaped / non-settlement sensitive areas.

Soil Aggressiveness

One representative sample (Borehole 1, split spoon 5, comprising native sand and gravel, from a depth of 3.8 m below grade) was subjected to multiple analyses to assess to potential for deterioration of concrete. Reference is given to the appended AGAT Certificate of Analysis (Appendix B) for the analytical results.

Based on the results, concrete pipes, foundations and tanks are expected to encounter sand and gravel with a maximum measured soluble sulphate concentration of 14 µg/g (<0.1%). Hence Portland cement concrete surrounded by the onsite sand and gravel will have a low degree of exposure to sulphate attack in accordance with CSA A23.1, and therefore Portland cement concrete should not require sulphate resistant cement.

It is recommended that parking lot storm water inflow into the tanks be limited due to the inherent variability of the water quality of storm runoff. The presence of salt in storm water due to de-icing applications may corrode the concrete. The interior of the tanks may be sprayed with a sealer / coating to combat the corrosion due to the presence of de-icing salts.

Pavement Reinstatement

It is anticipated that the excavations for the proposed underground tanks will be advanced through the parking lot pavements. Based on the observed condition of the parking lot pavements, it is recommended that the new pavement be reinstated to match the existing asphalt and granular thicknesses.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Sincerely
Peto MacCallum Ltd.

Ken Hanes, BASc.
Project Supervisor, Geotechnical and Geoenvironmental Services

LICENSED PROFESSIONAL ENGINEER
R. Agahzadeh
100042444
May 28/12
PROVINCE OF ONTARIO

Romin Agahzadeh, P.Eng.
Manager, Geotechnical Services

KH:fm

Enclosure(s):
Figure 1 – Moisture Density Relationship Test Report
Figures 2 to 3 – Particle Size Distribution Charts
Figure 4 – General Recommendations Regarding Underpinning
Figure 5 – Lateral Earth Pressure Distribution
List of Abbreviations
Log of Boreholes 1 and 2
Drawing 1 - Borehole Location Plan
Appendix A - Engineered Fill
Appendix B – AGAT Laboratories Certificate of Analysis

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MOISTURE DENSITY RELATIONSHIP TEST REPORT

CLIENT	City of Guelph	PML REF.:	12KF015
PROJECT	Guelph Transit Bus Wash	REPORT NO.	1
LOCATION	Guelph, Ontario	FIGURE	1

SAMPLE TYPE	Sand and Gravel, Trace Silt	SAMPLE NO.	32374
		SAMPLED BY	D. Brice
SAMPLED FROM	Borehole 1, Auger Sample 1A, Depth 0 to 0.75 m	DATE SAMPLED	May 4, 2012
		DATE RECEIVED	May 7, 2012

PROCTOR TEST RESULTS

TEST METHOD	ASTM D698-07 METHOD ☐ A ☐ B ☑ C ASTM D1557-07 METHOD ☐ A ☐ B ☐ C
MATERIAL RETAINED ON 19 mm SIEVE (%)	38.6
MATERIAL RETAINED ON 4.75 mm SIEVE (%)	70.5
MOISTURE CONTENT, AS RECEIVED (%)	3.4
MAXIMUM DRY DENSITY (kg/m^3)	2290
OPTIMUM MOISTURE CONTENT (%)	6.0
CORRECTED MAXIMUM DRY DENSITY (kg/m^3)	2324
CORRECTED OPTIMUM MOISTURE CONTENT (%)	6.0

REMARKS

For Particle Size Distribution see Figure 2.

REVIEWED BY: ______________

DATE ISSUED: ______________

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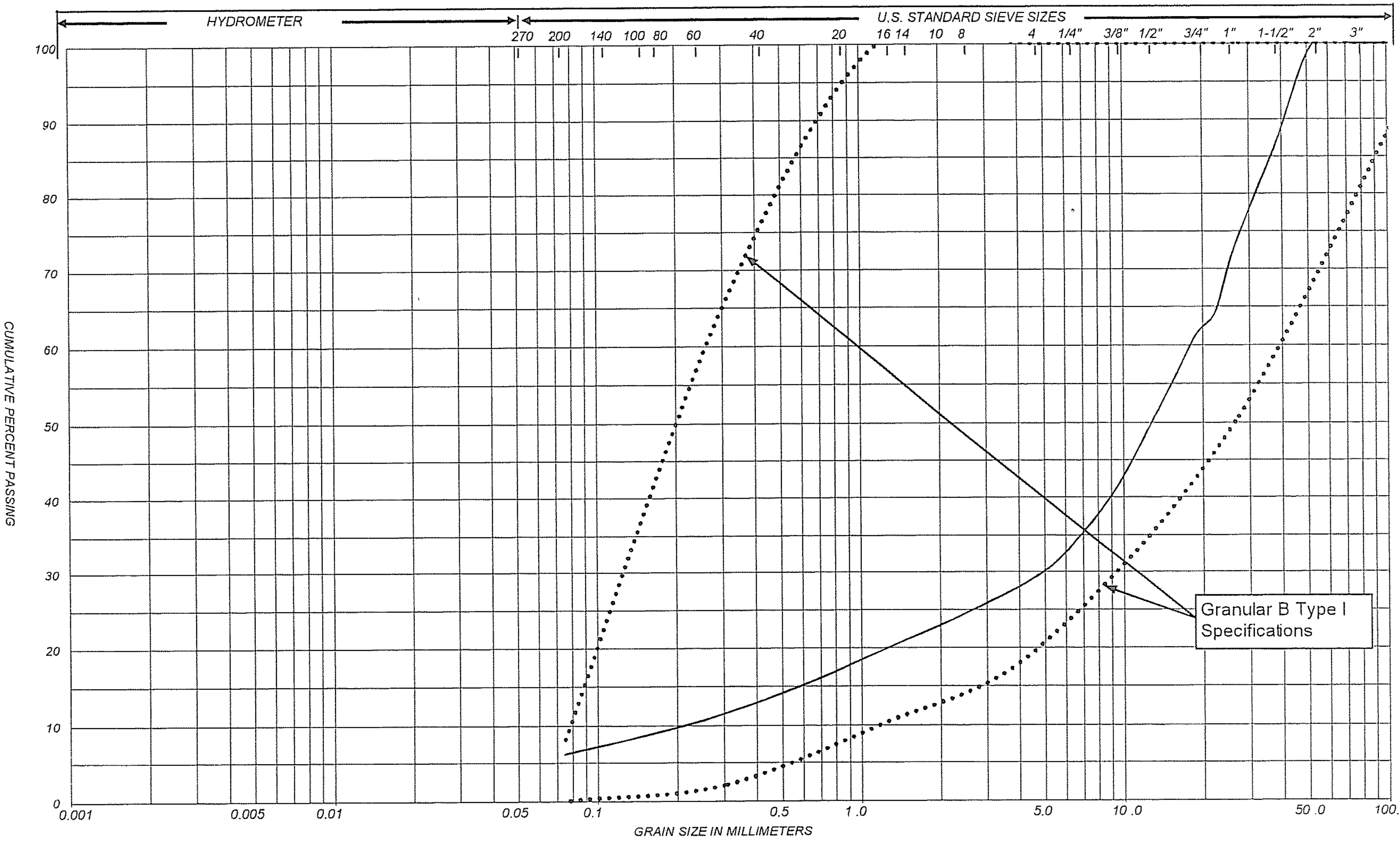
PARTICLE SIZE DISTRIBUTION CHART

PML REF. 12KF015
FIGURE NO. 2
SAMPLE NO. 32374



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<tr><td colspan="4"></td><td colspan="3">SAND</td><td></td><td></td><td></td></tr>
<tr><td>CLAY</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>GRAVEL</td><td>COBBLES</td><td>M.I.T</td></tr>
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<tr><td>CLAY</td><td>SILT</td><td>VERY FINE</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td colspan="3">GRAVEL</td><td>U.S. BUREAU</td></tr>
<tr><td></td><td></td><td colspan="4">SAND</td><td colspan="3"></td><td></td></tr>
</table>

REMARKS Borehole 1, Auger Sample 1A, Depth 0 to 0.75 m
SAND AND GRAVEL, TRACE SILT

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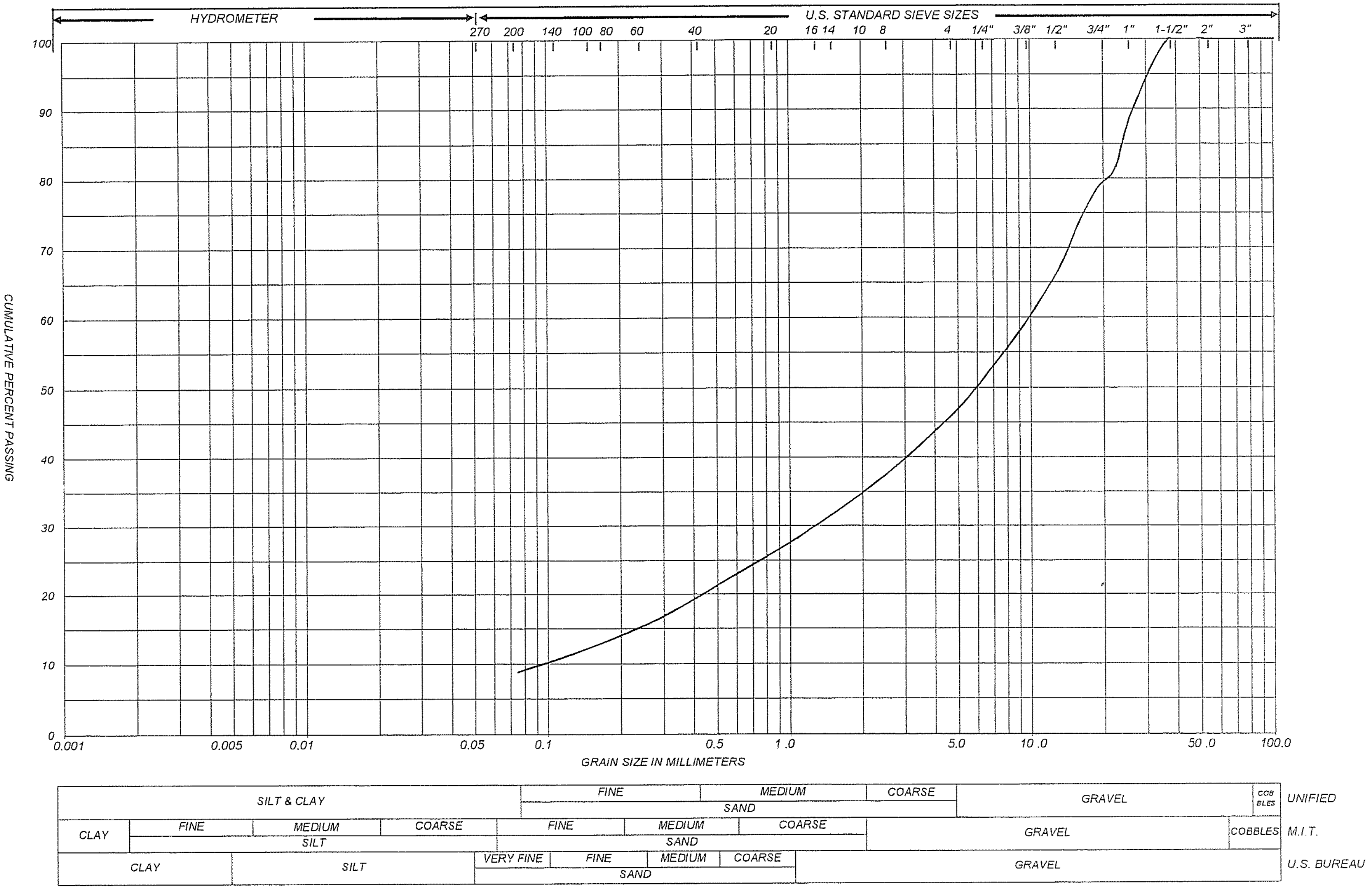
PARTICLE SIZE DISTRIBUTION CHART

PML REF. 12KF015
FIGURE NO. 3
SAMPLE NO. 32373



REMARKS Borehole 2, Sample SS5, Depth 3.80 to 4.25 m
SAND AND GRAVEL, TRACE SILT

UP – 001 1/96

NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

   ZONE A:
   Foundations of relatively heavy and/or settlement sensitive structures/ utilities located in Zone A generally require underpinning.

   ZONE B:
   Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

   ZONE C:
   Utilities and foundations located within Zone C do not normally require underpinning.

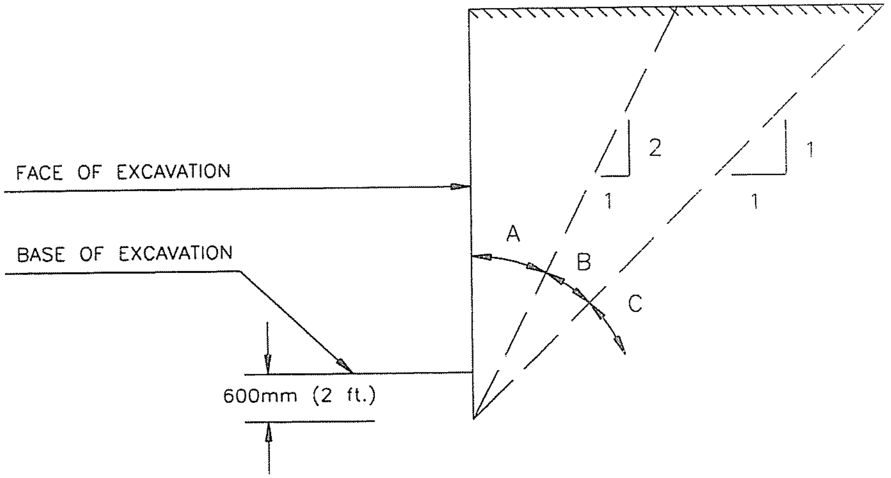
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non–underpinned foundations must be considered in the design of the support system.

3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.

4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.



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STANDARD DRAWING

GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION

DATE	JOB NUMBER	FIGURE NO.
MAY 2012	12KF015	4

BC - 005 01/96

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.

2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.

3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).

4. Structural components of bracing system should be confirmed adequate for each level of excavation.

5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.

6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.

7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.

8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and apppropriate monitoring (surface and insitu) carried out during construction.

9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, earth pressure diagram must be reviewed, and remedial works may be required.

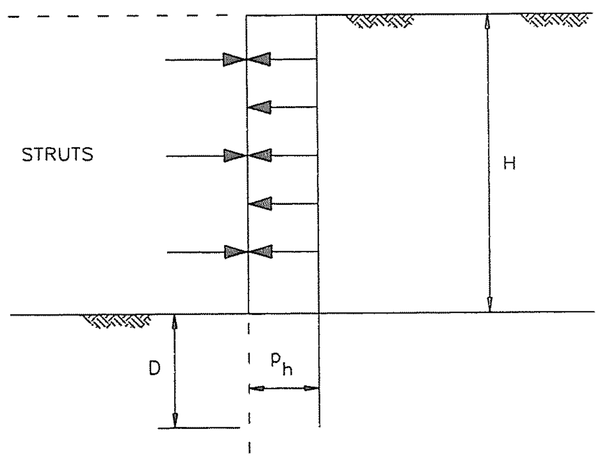
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.

11. Bracing system should be regularly examined for signs of distress.

12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



P_h = design lateral earth pressure
= 0.65 $K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

γ = 21.0 kN/m^3

K = 0.33 (movement of retained soil acceptable)
0.50 (movement of adjacent structures/facilities unacceptable)

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LATERAL EARTH PRESSURE DISTRIBUTION
MULTI-BRACED CUTS IN COHESIONLESS SOILS

DATE	JOB NUMBER	FIGURE NO
MAY 2012	12KF015	5

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. - Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow.

DESCRIPTION OF SOIL

The consistency of cohesive soils and the relative density or denseness of cohesionless soils are described in the following terms:

CONSISTENCY	N (blows/0.3 m)	c (kPa)	DENSENESS	N (blows/0.3 m)
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	> 50
Hard	> 30	> 200		

WTPL	Wetter Than Plastic Limit
APL	About Plastic Limit
DTPL	Drier Than Plastic Limit

TYPE OF SAMPLE

SS	Split Spoon	TW	Thinwall Open
WS	Washed Sample	TP	Thinwall Piston
SB	Scraper Bucket Sample	OS	Oesterberg Sample
AS	Auger Sample	FS	Foil Sample
CS	Chunk Sample	RC	Rock Core
ST	Slotted Tube Sample	USS	Undisturbed Shear Strength
PH	Sample Advanced Hydraulically	RSS	Remoulded Shear Strength
PM	Sample Advanced Manually		

SOIL TESTS

Qu	Unconfined Compression	LV	Laboratory Vane
Q	Undrained Triaxial	FV	Field Vane
Qcu	Consolidated Undrained Triaxial	C	Consolidation
Qd	Drained Triaxial		

LOG OF BOREHOLE NO. 1

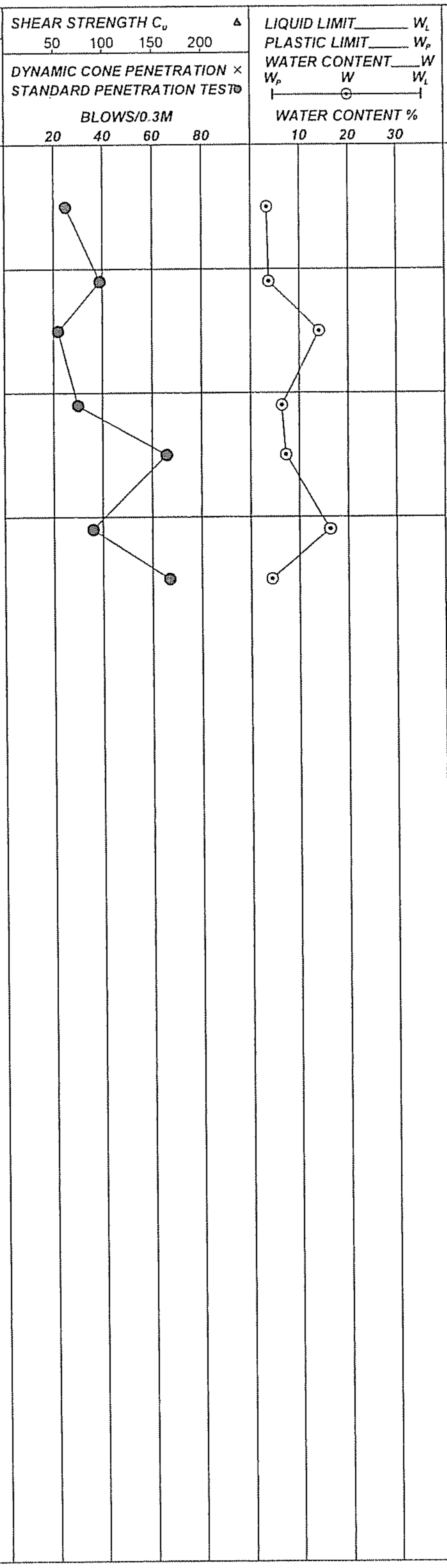
PROJECT Guelph Transit Bush Wash
LOCATION 120 Watson Road South, Guelph, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE 2012 04 05

OUR PROJECT NO. 12KF015
ENGINEER R Agahzadeh
TECHNICIAN D Brice

GROUND ELEVATION 328 95

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	GROUND WATER OBSERVATIONS AND REMARKS
0.27	PAVEMENT STRUCTURE: 120 mm of asphalt, over 150 mm of granular base, moist						
		▨		1A	AS		
	FILL: Compact to dense brown sand and gravel, trace silt, occasional cobbles, moist	▨	328	1	SS	25	
1 5		▨	327	2	SS	39	
2.30	occasional brown and grey clayey silt inclusions	▨		3	SS	22	
3 0		▨	326				
3.20	SAND AND GRAVEL: Dense to very dense brown sand and gravel, trace silt, saturated	⁘		4	SS	30	
		⁘	325	5	SS	66	Free water at 3 60 m
4 5		⁘	324	6	SS	36	
		⁘		7	SS	67	
5.80	BOREHOLE TERMINATED AT 5.80 m						Upon completion of drilling borehole caved to 2 7 m with no free water



NOTES

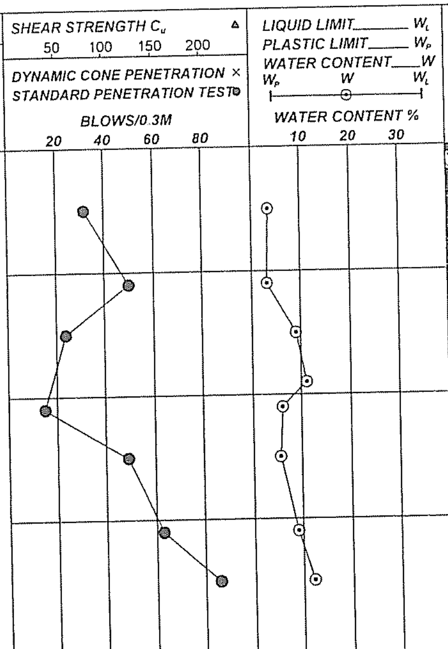
CHECKED BY RL

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

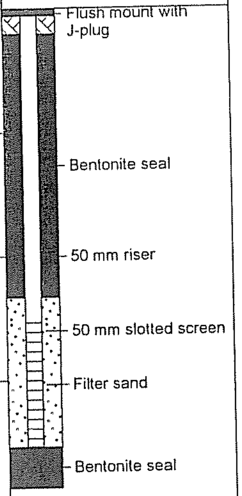
LOG OF BOREHOLE NO. 2

PROJECT Guelph Transit Bush Wash
LOCATION 120 Watson Road South, Guelph, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers
BORING DATE 2012 04 05
OUR PROJECT NO. 12KF015
ENGINEER R Agahzadeh
TECHNICIAN D Brice

SOIL PROFILE			SAMPLES		
DEPTH in METRES	DESCRIPTION GROUND ELEVATION 328 90	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES
0.31	PAVEMENT STRUCTURE: 130 mm of asphalt over 180 mm of granular base, moist				
	FILL: Dense brown sand and gravel, trace silt, occasional cobbles, moist	328	1	SS	32
1 5		327	2	SS	50
2.30			3	SS	24
2.40	becoming black sandy silt topsoil, moist				
	becoming grey silt with sand, wet	326			
3 0 3.10	SAND AND GRAVEL: Compact to very dense brown sand and gravel, trace silt, occasional cobbles, saturated		4	SS	15
		325	5	SS	49
4.5		324	6	SS	63
			7	SS	86
5.80	BOREHOLE TERMINATED AT 5.80 m				



GROUND WATER OBSERVATIONS AND REMARKS

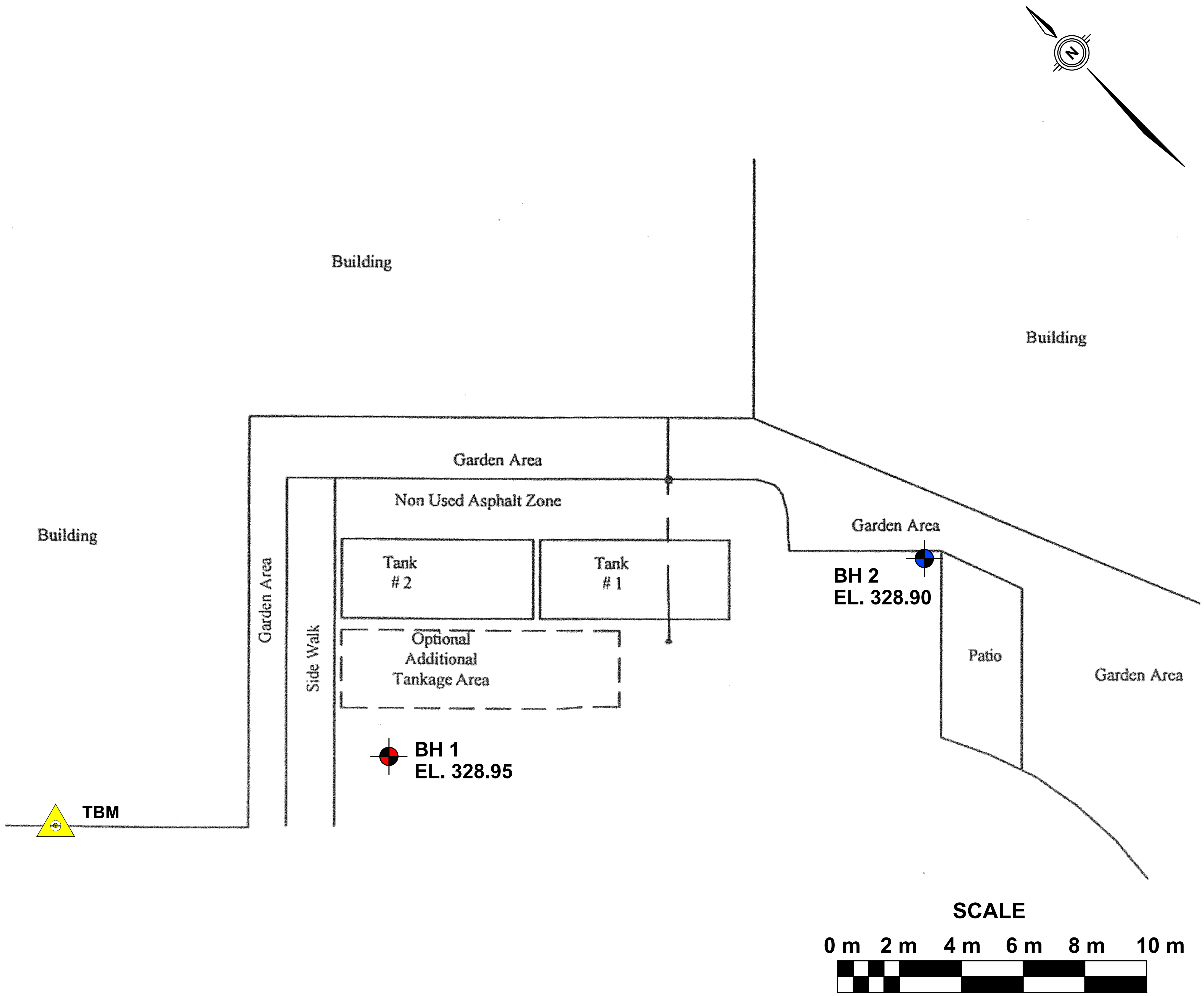


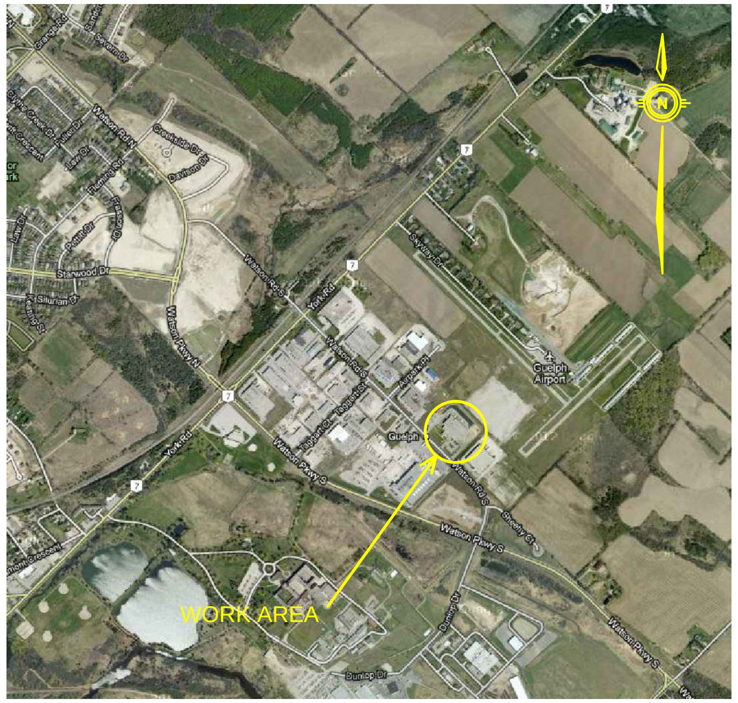
Water Level Readings:

Date	Depth (m)	Elev.
2012/04/05	3 35	325 55
2012/11/05	3 40	325 50

NOTES

CHECKED BY RL





KEY PLAN

LEGEND:

🔴 BOREHOLE

🔵 BOREHOLE WITH MONITORING WELL

▲ TEMPORARY BENCHMARK:
TOP OF OFFICE FLOOR SLAB
ELEVATION 329.60 (GEODETIC, METRIC)

REFERENCE:

BOREHOLE LOCATION PLAN REPRODUCED FROM DRAWING SUPPLIED BY CLIENT.

NOTE:

THE INFERRED STRATIGRAPHY REFERRED TO IN THE REPORT IS BASED ON THE DATA FROM THESE BOREHOLES SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY BETWEEN THE BOREHOLES MAY VARY.

CITY OF GUELPH

GUELPH TRANSIT BUS WASH

170 WATSON ROAD SOUTH
GUELPH, ONTARIO

BOREHOLE LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN	D. BRICE	DATE	SCALE	PML REF.	DWG. NO.
CHECKED	K. HANES	MAY 2012	AS SHOWN	12KF015	1
APPROVED	R. AGAHZADEH				

APPENDIX A

ENGINEERED FILL

ENGINEERED FILL

The information presented in this appendix is intended for general guidance only. Site specific conditions and prevailing weather may require modification of compaction standards, backfill type or procedures. Each site must be discussed, and procedures agreed with Peto MacCallum Ltd. prior to the start of the earthworks and must be subject to ongoing review during construction. This appendix is not intended to apply to embankments. Steeply sloping ravine residential lots require special consideration.

For fill to be classified as engineered fill suitable for supporting structural loads, a number of conditions must be satisfied, including but not necessarily limited to the following:

1. Purpose

The site specific purpose of the engineered fill must be recognized. In advance of construction, all parties should discuss the project and its requirements and agree on an appropriate set of standards and procedures.

2. Minimum Extent

The engineered fill envelope must extend beyond the footprint of the structure to be supported. The minimum extent of the envelope should be defined from a geotechnical perspective by:

- at founding level, extend a minimum 1.0 m beyond the outer edge of the foundations, greater if adequate layout has not yet been completed as noted below; and

- extend downward and outward at a slope no greater than 45° to meet the subgrade

All fill within the envelope established above must meet the requirements of engineered fill in order to support the structure safely. Other considerations such as survey control, or construction methods may require an envelope that is larger, as noted in the following sections.

Once the minimum envelope has been established, structures must not be moved or extended without consultation with Peto MacCallum Ltd. Similarly, Peto MacCallum Ltd. should be consulted prior to any excavation within the minimum envelope.

3. Survey Control

Accurate survey control is essential to the success of an engineered fill project. The boundaries of the engineered fill must be laid out by a surveyor in consultation with engineering staff from Peto MacCallum Ltd. Careful consideration of the maximum building envelope is required.

During construction it is necessary to have a qualified surveyor provide total station control on the three dimensional extent of filling.

4. Subsurface Preparation

Prior to placement of fill, the subgrade must be prepared to the satisfaction of Peto MacCallum Ltd. All deleterious material must be removed and in some cases, excavation of native mineral soils may be required.

Particular attention must be paid to wet subgrades and possible additional measures required to achieve sufficient compaction. Where fill is placed against a slope, benching may be necessary and natural drainage paths must not be blocked.

5. Suitable Fill Materials

All material to be used as fill must be approved by Peto MacCallum Ltd. Such approval will be influenced by many factors and must be site and project specific. External fill sources must be sampled, tested and approved prior to material being hauled to site.

6. Test Section

In advance of the start of construction of the engineered fill pad, the Contractor should conduct a test section. The compaction criterion will be assessed in consultation with Peto MacCallum Ltd. for the various fill material types using different lift thicknesses and number of passes for the compaction equipment proposed by the Contractor.

Additional test sections may be required throughout the course of the project to reflect changes in fill sources, natural moisture content of the material and weather conditions.

The Contractor should be particularly aware of changes in the moisture content of fill material. Site review by Peto MacCallum Ltd. is required to ensure the desired lift thickness is maintained and that each lift is systematically compacted, tested and approved before a subsequent lift is commenced.

7. Inspection and Testing

Uniform, thorough compaction is crucial to the performance of the engineered fill and the supported structure. Hence, all subgrade preparation, filling and compacting must be carried out under the full time inspection by Peto MacCallum Ltd.

All founding surfaces for all buildings and residential dwellings or any part thereof (including but not limited to footings and floor slabs) on structural fill or native soils must be inspected and approved by PML engineering personnel prior to placement of the base/subbase granular material and/or concrete. The purpose of the inspection is to ensure the subgrade soils are capable of supporting the building/house foundation and floor slab loads and to confirm the building/house envelope does not extend beyond the limits of any structural fill pads.

8. Protection of Fill

Fill is generally more susceptible to the effects of weather than natural soil. Fill placed and approved to the level at which structural support is required must be protected from excessive wetting, drying, erosion or freezing. Where adequate protection has not been provided, it may be necessary to provide deeper footings or to strip and recompact some of the fill.

9. Construction Delay Time Considerations

The integrity of the fill pad can deteriorate due to the harsh effects of our Canadian weather. Hence, particular care must be taken if the fill pad is constructed over a long time period.

It is necessary therefore, that all fill sources are tested to ensure the material compactability prior to the soil arriving at site. When there has been a lengthy delay between construction periods of the fill pad, it is necessary to conduct subgrade proof rolling, test pits or boreholes to verify the adequacy of the exposed subgrade to accept new fill material.

When the fill pad will be constructed over a lengthy period of time, a field survey should be completed at the end of each construction season to verify the areal extent and the level at which the compacted fill has been brought up to, tested and approved.

In the following spring, subexcavation may be necessary if the fill pad has been softened attributable to ponded surface water or freeze/thaw cycles.

A new survey is required at the beginning of the next construction season to verify that random dumping and/or spreading of fill has not been carried out at the site.

10. Approved Fill Pad Surveillance

It should be appreciated that once the fill pad has been brought to final grade and documented by field survey, there must be ongoing surveillance to ensure that the integrity of the fill pad is not threatened.

Grading operations adjacent to fill pads can often take place several months or years after completion of the fill pad.

It is imperative that all site management and supervision staff, the staff of Contractors and earthwork operators be fully aware of the boundaries of all approved engineered fill pads.

Excavation into an approved engineered fill pad should never be contemplated without the full knowledge, approval and documentation by the geotechnical consultant.

If the fill pad is knowingly built several years in advance of ultimate construction, the areal limits of the fill pad should be substantially overbuilt laterally to allow for changes in possible structure location and elevation and other earthwork operations and competing interests on the site. The overbuilt distance required is project and/or site specified.

Iron bars should be placed at the corner/intermediate points of the fill pad as a permanent record of the approved limits of the work for record keeping purposes.

11. Unusual Working Conditions

Construction of fill pads may at times take place at night and/or during periods of freezing weather conditions because of the requirements of the project schedule. It should be appreciated therefore, that both situations present more difficult working conditions. The Owner, Contractor, Design Consultant and Geotechnical Engineer must be willing to work together to revise site construction procedures, enhance field testing and surveillance, and incorporate design modifications as necessary to suit site conditions.

When working at night there must be sufficient artificial light to properly illuminate the fill pad and borrow areas.

Placement of material to form an engineered fill pad during winter and freezing temperatures has its own special conditions that must be addressed. It is imperative that each day prior to placement of new fill, the exposed subgrade must be inspected and any overnight snow or frozen material removed. Particular attention should be given to the borrow source inspection to ensure only nonfrozen fill is brought to the site.

The Contractor must continually assess the work program and have the necessary spreading and compacting equipment to ensure that densification of the fill material takes place in a minimum amount of time. Changes may be required to the spreading methods, lift thickness, and compaction techniques to ensure the desired compaction is achieved uniformly throughout each fill lift.

The Contractor should adequately protect the subgrade at the end of each shift to minimize frost penetration overnight. Since water cannot be added to the fill material to facilitate compaction, it is imperative that densification of the fill be achieved by additional compaction effort and an appropriate reduced lift thickness. Once the fill pad has been completed, it must be properly protected from freezing temperatures and ponding of water during the spring thaw period.

If the pad is unusually thick or if the fill thickness varies dramatically across the width or length of the fill pad, Peto MacCallum Ltd. should be consulted for additional recommendations. In this case, alternative special provisions may be recommended, such as providing a surcharge preload for a limited time or increase the degree of compaction of the fill.

APPENDIX B

AGAT LABORATORIES CERTIFICATE OF ANALYSIS

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: PETO MACCALLUM LIMITED
16 FRANKLIN STREET SOUTH
KITCHENER, ON N2C1R4
(519) 893-7500

ATTENTION TO: Dylan Brice

PROJECT NO: 12KF015

AGAT WORK ORDER: 12W597996

SOIL ANALYSIS REVIEWED BY: Anthony Dapaah, PhD (Chem), Inorganic Lab Manager

DATE REPORTED: May 18, 2012

PAGES (INCLUDING COVER): 4

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

AGAT Laboratories

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MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 12W597996
PROJECT NO: 12KF015

CLIENT NAME: PETO MACCALLUM LIMITED

ATTENTION TO: Dylan Brice

Corrosivity Package

DATE SAMPLED: May 04, 2012 **DATE RECEIVED: May 09, 2012** **DATE REPORTED: May 18, 2012** **SAMPLE TYPE: Soil**

Parameter	Unit	G / S	RDL	BH1 ;SS5 3318829
Sulphide*	%		0.01	<0.01
Chloride (2:1)	µg/g		2	49
Sulphate (2:1)	µg/g		2	14
pH (2:1)	pH Units		N/A	9.18
Electrical Conductivity (2:1)	mS/cm	0.57	0.005	0.216
Resistivity (2:1)	ohm.cm		1	4630
Redox Potential (2:1)	mV		5	202

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to T1(ALL) - Current

3318829 * Analysis was performed at AGAT's Mining Division.

EC,pH,Chloride,Redox Potential and Sulphate were determined on the extract obtained from the 2:1 extraction procedure (2 parts DI water: 1 part soil).

Certified By:

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www agatlabs com

Quality Assurance

CLIENT NAME: PETO MACCALLUM LIMITED | **AGAT WORK ORDER: 12W597996**

PROJECT NO: 12KF015 | **ATTENTION TO: Dylan Brice**

Soil Analysis

RPT Date: May 18, 2012			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Corrosivity Package															
Sulphide*	1		< 0.01	< 0.01	0.0%	< 0.01	91%	80%	120%	NA			NA		
Chloride (2:1)	1		157	152	3.2%	< 2	96%	80%	120%	95%	80%	120%	102%	70%	130%
Sulphate (2:1)	1		731	737	0.8%	< 2	103%	80%	120%	98%	80%	120%	97%	70%	130%
pH (2:1)	1	3318829	9.18	9.20	0.2%	N/A	96%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	1		0.212	0.208	1.9%	< 0.005	100%	90%	110%	NA			NA		
Redox Potential (2:1)	1	3318829	202	203	0.5%	< 5	103%	70%	130%	NA			NA		

Certified By:

Method Summary

CLIENT NAME: PETO MACCALLUM LIMITED
AGAT WORK ORDER: 12W597996
PROJECT NO: 12KF015
ATTENTION TO: Dylan Brice

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulphide*	MIN-200-12000	ASTM E1915-07a	LECO C_S
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2.1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR 1036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR 1036		CALCULATION
Redox Potential (2:1)		SM 2510 B	REDOX POTENTIAL ELECTRODE

AGAT Laboratories

TEMP①: 12.1 TEMP②: 11.5 TEMP③: 10.7

5835 Coopers Avenue
Mississauga, Ontario; L4Z 1Y2
Phone: 905-712-5100; Fax: 905-712-5122
Toll free: 800-856-6261
www.agatlabs.com
http://webearth.agatlabs.com

CHAIN OF CUSTODY RECORD

SR-12

LABORATORY USE ONLY
Arrival Condition: ☐ Good 4.5 ☐ Poor (complete "notes")
Arrival Temperature: 13.7/9.7 AGAT WO #: 12W 597996
Notes: LAB 4.1 / 3.6

Client Information

Company: PETO MACCALLUM LTD
Contact: DYLAN BRICE
Address: 16 FRANKLIN ST S, KIT, ON
N2C 1R4
Phone: 519-893-7500 Fax: 519-893-0654
Project: 12KF015 PO:
AGAT Quotation #:

Please note, if quotation number is not provided, client will be billed full price for analysis.

Invoice To Same as Above? [✓ Yes] No (circle)

Company:
Contact:
Address:
Phone: Fax:

Report Information - reports to be sent to:

1. Name: Ken Hanes
Email: khanes@petomaccallum.com
2. Name: Dean Brice
Email: dbrice@petomaccallum.com

Regulatory Requirements

☒ Regulation 153 Table
(Indicate one)
☐ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture (check one)
☐ Coarse ☐ Med/Fine

☐ Sewer Use
Region
(Indicate one)
☐ Sanitary
☐ Storm

☐ Regulation 558
☐ CCME
☐ Other (Indicate)

☐ Prov. Water Quality Objectives (PWQO)
☐ Nutrient Management Act (NMA)

Is this a drinking water sample (potable water intended for human consumption)?
☐ Yes ☐ No (If "Yes" please use the Drinking Water Chain of Custody Record)

Report Format

☒ Single Sample per page
☐ Multiple Samples per page
☐ Results by fax

Turnaround Time (TAT) Required*

Regular TAT:
☒ 5 to 7 Working Days

Rush TAT: (please provide prior notification)
Rush Surcharges Apply
☐ 3 to 5 Working Days
☐ 2 Working Days
☐ 1 Working Day

OR

DATE REQUIRED (Rush surcharges may apply):

*TAT is exclusive of weekends and statutory holidays

Sample Identification	Date Sampled	Time Sampled	Sample Matrix	# of Containers	Comments Site/Sample Information	Metals and Inorganics	Metal Scan (exclud. Hg, B, Cr6)	CCME Fractions 1 to 4	VOCs	PAHs	PCBs	TCLP Metals/Inorganics	TCLP	Storm Sewer Use	Sanitary Sewer Use	CORROSIVITY PKG					LABORATORY USE ONLY LAB SAMPLE ID
BH 1; SS 5	May 4	2:30	SOIL	2												X					

Samples Relinquished By (print name & sign)	Date/Time	Samples Received By (print name & sign)	Date/Time		
D.BRICE	May 8/	CASSANDRA S.	4:46 05/8/12	Pink Copy – Client	PAGE 1 of 1
Samples Relinquished By (print name & sign)	Date/Time	Samples Received By (print name & sign)	Date/Time	Yellow + Golden Copy - AGAT	NO: 159137
		SIMON MAY 9/12	1:17	White Copy - AGAT	

Document ID: DIV-78-1511.005

Date Issued: December 6, 2010