



July 2009

FINAL GEOTECHNICAL INVESTIGATION AND DESIGN REPORT

**Proposed Forcemain Crossing QEW,
Beach Boulevard Flood Protection
City of Hamilton, Ontario**

Submitted to:
The City of Hamilton
Capital Planning and Implementation
Division Public Works Department
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Hamilton, Ontario
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REPORT



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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation carried out at the above referenced site. The purpose of this geotechnical investigation was to determine the subsurface soil and groundwater conditions for the forcemain crossing under the QEW and provide recommendations for the temporary excavation, bedding, backfilling and pavement restoration.

The following field investigation techniques were used to obtain information regarding the existing pavement structure and subsurface conditions:

- Borehole Drilling and Sampling – to determine the types and thicknesses of the existing pavement layers, and determine soil and groundwater conditions; and
- Ground Penetrating Radar (GPR) Survey – to obtain a continuous profile of the pavement structure and identify the location of full depth asphalt repairs.

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, or if the project is not initiated within twelve months of the date of this report, Golder Associates Ltd. should be given an opportunity to confirm that the recommendations are still valid.

The contents of this report have been based on information regarding the proposed forcemain provided to us by McCormick Rankin Corporation (MRC) and have been prepared solely for use by the City of Hamilton (the City), its consultants retained to design the storm sewer/forcemain and government agencies involved in the review of/granting permission for the construction of the forcemain crossing the QEW. Golder Associates Ltd. (Golder) accepts no responsibility for any reliance upon, including any decisions made on the basis of, the contents of this report by any third party.

This report should be read in conjunction with the “Important Information and Limitations of This Report” following the text of this report. The reader’s attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 PROJECT AND SITE DESCRIPTION

The proposed forcemain crossing the QEW is part of the Beach Boulevard Flood Protection project. The crossing location is at the south end of Grafton Avenue; the forcemain will involve two 900 mm diameter pipes which discharge into the Eastport ditch on the south side of the QEW. It is understood that the forcemains will be installed within two 1200 mm diameter steel liners which will be installed under the QEW using open cut method

It is further understood that the invert of the forcemain will be at a depth of about 3.0 m below existing ground surface as shown in a drawing prepared by MRC for the Beach Boulevard interceptor.

3.0 FIELD INVESTIGATION

The field work for the geotechnical investigation was carried out between January 26 and February 25, 2009. It included 8 boreholes (designated as Boreholes BH 101 to BH 104, BH 106, BH 107, BH 201 and BH 202) drilled at the locations shown on the Borehole Location Plan, Figure A1 in Appendix A. In addition, 7 cores (CH1, CH2, CH5, CHA, CHB, CHC and CH104) were also drilled in order to confirm the thickness and type of bond layers in



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the pavement structure. The general locations of the boreholes and cores drilled on the QEW are summarized below.

| Borehole No. | Direction | Lane/Location |
|--------------|---------------|--------------------------------|
| BH 101 | Toronto bound | Outside shoulder (gravel part) |
| CH 1 | Toronto bound | Paved shoulder |
| BH 102 | Toronto bound | Paved shoulder |
| BH 103 | Toronto bound | Lane 4 |
| BH 201 | Toronto bound | Lane 2 |
| BH 104 | Niagara bound | Median |
| CH 2 | Niagara bound | Lane 1 |
| CH 5 | Niagara bound | Lane 1 |
| CH A | Niagara bound | Lane 1 |
| CH B | Niagara bound | Lane 1 |
| CH C | Niagara bound | Lane 1 |
| BH 202 | Niagara bound | Lane 2 |
| CH 104 | Niagara bound | Paved shoulder |
| BH 106 | Niagara bound | Paved shoulder |
| BH 107 | Niagara bound | Along the ditch line |

The boreholes were advanced using a truck mounted drill rig supplied and operated by a specialist drilling contractor. The boreholes were advanced using either 108 mm or 246 mm inside diameter hollow stem augers to depths ranging from about 3.5 m to 7.5 m below the existing ground surface. Samples of the overburden soil were obtained at intervals of about 0.75 m and 1.5 m depth, using 50 mm outer diameter split spoon samplers in accordance with the Standard Penetration Test (SPT) procedures described in ASTM D1586 99.

Groundwater conditions were noted in the open boreholes prior to backfilling. Three standpipe piezometers were installed in boreholes BH 101, BH 106 and BH 107 to permit further monitoring of the groundwater level at these locations.

The field work for this investigation was monitored by members of our field staff who logged the subsurface conditions encountered in the boreholes and cared for the obtained samples. All of the soil samples obtained during the investigation were brought to the Golder's Mississauga laboratory for further examination, natural water content testing and selected classification testing. The borehole locations and the ground surface elevations were surveyed by MRC and the survey data was provided to Golder.



4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface soil and groundwater conditions encountered in the boreholes, as well as the results of the field and laboratory testing, are shown in detail on the attached Record of Borehole sheets in Appendix B. Lists of Abbreviations and Symbols are also provided for use in interpreting the borehole logs. A stratigraphic section based on the borehole results, including the pavement structure, soil types, SPT 'N' values and the measured water levels, is shown on Drawing A2 in Appendix A.

It should be noted that the boundaries between the strata on the borehole records have been inferred from drilling observations and non-continuous sampling. The boundaries generally represent transitions from one soil type to another and should not be inferred to represent exact planes of geological change. Further, conditions will vary between and beyond the boreholes.

The subsurface soil conditions at the borehole locations generally consist of asphalt and/or sand and gravel fill underlain by extensive deposits of cohesionless soils varying in composition from sand to sand and gravel. Cobbles and boulders were present throughout these deposits.

4.2 Pavement Structure

The subsurface investigation indicates that the pavement structure within the project limits is of flexible type, i.e. consists of asphalt layers placed over granular base/subbase. A summary of the existing pavement structure is shown below.

| BH/CH No. | Layer Thickness (mm) | | | Comments |
|-----------|----------------------|-----------------------|-------|---|
| | Asphalt | Granular Base/Subbase | Total | |
| BH 101 | - | 760 | 760 | SB |
| CH 1 | 150 | - | - | Shoulder in the full depth patch area, NB |
| BH 102 | 150 | 1300 | 1450 | Paved shoulder, NB |
| BH 103 | 430 | 940 | 1370 | Lane 4, NB |
| BH 201 | 580 | 1400 | 1980 | Lane 2, NB |
| BH 104 | 200 | 310 | 510 | Median, SB |
| CH 2 | 425 | - | - | In the full depth patch area |
| CH 5 | 465 | - | - | Lane 1, SB |
| CH A | 465 | - | - | Lane 1, SB |
| CH B | 450 | - | - | Lane 1, SB |



| BH/CH No. | Layer Thickness (mm) | | | Comments |
|-----------|----------------------|-----------------------|-------|---|
| | Asphalt | Granular Base/Subbase | Total | |
| CH C | 465 | - | - | Lane 1, SB |
| BH 202 | 510 | 350 | 860 | Lane 2, SB |
| CH 4 | 140 | - | - | Shoulder in the full depth patch area, SB |
| BH 106 | 150 | 540 | 690 | Paved shoulder, SB |

BH 107 was drilled in the ditch, SB.

The thickness of hot-mix asphalt (HMA) encountered in Lanes 1 to 4 ranges from 425 mm to 580 mm with a mean of 475 mm. The total HMA thickness measured in three boreholes/cores in the shoulder is about 150 mm and 200 mm (one borehole) in the median.

4.3 Topsoil and Organic Material

About 130 mm of topsoil was encountered at ground surface in Borehole BH 107, located in the ditch on the Niagara bound side of the highway.

Dark brown organic silt was encountered beneath the fill materials in Boreholes BH 101 and BH 102 on the Toronto bound side of the highway. This deposit was about 120 mm and 220 mm thick and was present at a depth of about 3.5 m below the pavement surface. In borehole BH 106, about 800 mm of organic silt was encountered at about 3.7 m depth.

4.4 Fill Materials

Approximately 1.5 m to 4.0 m of granular fill, described as a silty sand to sand with some silt and gravel, was encountered beneath the pavement structure in all the boreholes. In boreholes BH 102 and BH 103, the fill was noted to contain organics and in BH 102 the fill contained some debris including wood and glass. In BH 104, from 3.1 m to 4.4 m depth, zones of black staining with hydrocarbon odour were encountered. The SPT 'N' values measured within the granular fill ranged from 2 to 86 blows per 0.3 m of penetration, typically 2 to 20 blows per 0.3 m penetration indicating that the fill typically has a very loose to compact relative density. The higher SPT values (greater than 60 blows) in the upper 1 m or so are considered to be a consequence of the granular fill being frozen.

In Borehole BH 107, a 3.5 m thick layer of fill material was encountered below the top soil; the fill consisted of silty sand, some gravel and contained coarse slag rock fragments, cobbles and boulders below 0.6 m depth. Auger refusal was encountered at a depth of 3.25 m. In BH 201, foundry sand fill was encountered at a depth of 4.0 m below the granular fill material. The one SPT 'N' value obtained within this material was 4 blows per 0.3 m of penetration, indicating a loose relative density.



4.5 Granular Deposits

Underlying the fill materials and/or the organic silt deposits, the native soils consisting of cohesionless granular soils typically varying in composition from sand to sand and gravel were encountered. In boreholes BH 102 and BH 103, the upper portion of the deposit (0.8 m and 0.3 m in thickness, respectively) consisted of sandy silt/silty sand and contained trace organics with organic staining. The underlying sand/sand and gravel deposit contains trace shell fragments.

The SPT 'N' values measured within the upper portions of the granular deposit (to depths ranging from 3.5 m to 5.5 m below ground surface) typically ranged from 3 to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density. Beneath the upper loose portion, the SPT 'N' values were typically higher, ranging from 13 to 46 blows per 0.3 m of penetration, indicating the materials below depths of 3.5 m to 5.5 m had a dense to compact relative density. Typically the SPT 'N' values increase with depth; the lower SPT 'N' values in the sandy materials may be attributed to disturbance of the sands during drilling due to the high water table, and therefore, may not be representative of the in-situ condition of the materials.

As described above, the composition of the granular deposits are variable. The results of grain size analysis performed on the obtained soil samples are shown in Figures C1 to C11 in Appendix C.

Natural water contents of selected samples of these sandy materials typically varied from 7 percent to 33 percent, with the higher water contents measured on samples containing organics.

4.6 Groundwater

Groundwater was observed in the open boreholes upon completion of drilling. During drilling, water levels were observed in all boreholes at depths between 1.4 m and 3.4 m below the ground surface. As mentioned in Section 2, standpipe piezometers were installed in Boreholes BH 101, BH 106 and BH 107. Piezometer installation details and groundwater conditions encountered during drilling are shown in the attached Record of Borehole sheets in Appendix B. The water level readings measured in the piezometers are summarized below.

Water Levels in Piezometers

| Borehole No. | January 26, 2009 | | February 16, 2009 | |
|--------------|------------------|-----------|-------------------|-----------|
| | Depth | Elevation | Depth | Elevation |
| BH 101 | 2.8 | 74.8 | 2.9 | 74.6 |
| BH 106 | 3.1 | 74.8 | 3.0* | 74.9 |
| BH 107 | 1.4 | 75.9 | 2.1 | 75.2 |

*Note: Water level in BH106 was measured on January 28, 2009

The above levels indicate that the groundwater table appears to be sloping downward to the north with the higher levels measured in piezometers located close to the Eastport ditch. It should be noted that the groundwater levels at the site are anticipated to fluctuate as a result of seasonal variations in precipitation/runoff and the water level in the Eastport ditch adjacent to the Niagara bound lanes as well as the level of Lake Ontario which is known to fluctuate by as much as 0.5 m over the course of a year. The lower levels are typically encountered in the winter (the mean is at about Elevation 74.5 m) and the higher levels in the summer months (the mean is at about Elevation 75 m). Perched groundwater conditions should be anticipated within and above fine grained materials especially during the wetter months of the year. It is generally anticipated that the groundwater levels will be higher during wet periods (i.e. spring thaw) and lower during drier, summer periods.



5.0 GROUND PENETRATING RADAR (GPR) SURVEY

Ground penetrating radar (GPR) surveys were carried out by Golder as part of the pavement geotechnical investigation for the open cut forcemain installation. The area investigated by GPR was the two traffic lanes and the median shoulder in each direction over a section approximately 160 meters long. The system used to collect the GPR data was a Sensors and Software Noggin smart-cart system which employed a 250 and 1000 MHz antenna centre frequency.

5.1 Methodology

A typical GPR system consists of two antennae (transmitter and receiver), a control console and a computer for real-time, graphic display and data recording. In reflection profiling mode, the antennae, separated a fixed distance, are moved stepwise along the survey line and readings are taken at discrete intervals. At each step, pulses of radar frequency electromagnetic energy (megahertz range) are transmitted and reflections received from subsurface horizons. Subsurface reflections occur at horizons where there is an abrupt change in the material's dielectric permittivity such as at the interface between saturated and unsaturated materials (water table), stratigraphic horizons, and interfaces between natural and man-made materials. The amplitude of received radar energy is recorded as a function of time, processed in real-time for display purposes and the raw data recorded digitally for later processing and presentation.

The resolution and penetration of a GPR system is dependent on the centre frequency of its operation. Typically, antennas having a centre frequency between 25 and 1000 MHz are used for subsurface investigations. Lower frequency antennas penetrate deeper into the subsurface, but have less vertical resolution than do higher frequency antennas.

GPR sections are presented as time-sections, with the position (in metres) of each trace recorded as the horizontal axis across the top of the section and the GPR travel time (in nanoseconds, increasing downward) as the principal vertical axis. A second vertical axis is included to provide an estimate of depth or elevation and is calculated assuming a constant GPR velocity for the subsurface.

A key aspect to interpretation of the GPR profiles is to have control at one or preferably more locations along the survey line. Although it is generally reasonable to provide preliminary interpretations of collected data, it is necessary to confirm GPR interpretations with data from intrusive investigations such as boreholes.

GPR antennas, whether shielded or unshielded, tend to pick up spurious air wave reflections from objects at surface proximal to the survey line such as columns, and the ceiling. These air wave events are, in general, distinct in their shape and frequency content as observed on reflection profiles and can usually be identified with confidence on the GPR sections during interpretation.

5.2 Field Work

The geophysical survey was conducted in December, 2008, by personnel from Golder's Mississauga office. The portion of road surveyed was the QEW near Grafton Street.

This GPR system was run at walking speed, taking readings at intervals of approximately 10 mm. The data were positioned using a grid setup, based on known location of boreholes and other fixtures on the Highway. Within each lane, data was collected along the approximate centre line of the lane.

All GPR data was recorded digitally and transferred to a computer for subsequent data processing.



5.3 Results

The collected GPR data was analyzed using proprietary software developed by Golder for GPR analysis. The data was processed using a bandpass filter centred at the frequency of the GPR system, to remove high and low frequency 'noise' from the data sets. Data was gained for display purposes using a spherical exponential gain. The data was then analyzed and interpreted for layer depths and identification of any potential subsurface anomalies.

The resulting GPR data sets were then plotted for each of the road sections for interpretation. An average GPR velocity of 0.12 m/ns was estimated based on correlation to available corehole data, and agrees well with the range of published GPR velocity values for asphalts and dry soils and granular materials.

The GPR interpreted results are presented in Figure D1 in Appendix D.

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

The geotechnical recommendations provided herein are based on our interpretation of the borehole information and on our understanding of the project requirements. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

Our professional services for this assignment address only the geotechnical (physical) aspects of the subsurface conditions at this site. The geo environmental (chemical) aspects, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside the terms of reference for this report.

The objective of a proper open cut pavement restoration is to bring the pavement to the same or better condition than it was originally. The current pavement and materials technology provides a cost-effective solution; however, the following general conditions must be met: avoid undermining the existing pavement; remove unsuitable soil; keep the excavation free of water, as much as possible; use proper bedding materials; fill excavation above bedding with proper, well compacted granular material or unshrinkable fill, if conditions warrant; and complete high quality asphalt pavement restoration. The soil and groundwater conditions encountered in the boreholes, together with the nature and extent of the proposed works, will pose some constraints and difficulties for the construction.

6.1 Pavement Removal and Temporary Excavations

The existing asphalt should be sawcut before the temporary excavation and forcemain installation. The asphalt in Lanes 1 to 4 is relatively thick, ranging from about 410 mm to almost 580 mm with a mean of 475 mm and the sawcutting could therefore be a time consuming operation. The sawcut for pavement removal should be done in one vertical cut (no step cutting). However, a step joint should be formed between the patch and the adjacent existing pavement. Large blades (about 5 feet) may be required for this operation. The asphalt should be sawcut not only along the perimeter but also into relatively small size slabs that can be removed (size depending on the equipment proposed by the contractor). It should be noted that 1 m² of 500 mm thick asphalt can weight about 1.3 tonnes.



Based on the plan drawing provided by MRC, it is understood that the proposed invert elevations for the new forcemain will be about 3.0 m below the existing pavement surface. The temporary excavations for the installation of the forcemain are expected to extend to a depth of 300 mm below the bottom of the pipe. The water levels measured in piezometers installed in boreholes BH 101, BH 106 and BH 107 were from approximately 2.1 m to 3.0 m below ground surface in February 2009. Where excavations extend below the groundwater table through the predominantly medium to coarse granular soils, groundwater may flow into the excavations. Due to relatively short time for the forcemain installation and in order to avoid potential issues with dewatering and placement and compaction of granular bedding material, it is recommended that underwater type of concrete be used for the bedding.

On the day the asphalt is sawcut, the contractor should also install the sheet piles if this is the recommended option for the support of the trench. The trench will be about 3.3 m deep including the bedding. Sheet piles will have to provide adequate support for the excavation and the traffic loading in the adjacent lanes. The sheet piles will have to be relatively stiff and may have to go relatively deep because the wall will have to be acting in cantilever unless there is some way of providing walers and rakers to support the wall. Although the soil below the trench base is generally loose (loose sands, silty sands and organic silts) with SPT 'N' values typically ranging from 1 to about 20 blows per 0.3 m of penetration, it should also be noted that some cobbles and rock fragments were encountered in some of the boreholes; this may slow down the sheet pile installation operations. After the sheet piles are installed, the existing pavement should be patched with asphalt. This temporary patch should consist of two 70 mm thick lifts of HL 8 mix incorporating PG 58-28 asphalt cement. The lower lift shall be compacted using a tamping compactor or a small roller; the second lift can be compacted using conventional rollers.

All excavation work should be carried out in accordance with the Occupational Health and Safety Act (OSHA) and with local regulations. Care should be taken to direct surface water away from open excavations. Stockpiles of excavated material should be set back from the edge of the excavation by a distance at least equal to the excavation depth. In general, the existing fill materials and the upper very loose to loose native soils are classified as Type 3 soil and the compact to dense sandy deposits are classified as Type 2 (Dense) or Type 3 (Compact) soil, according to the Occupational Health and Safety Act (Ont. Reg. 213/91) classification system. For this site, temporary open cut excavations extending through the overburden soils above the groundwater table may be developed with side slopes not steeper than 1 horizontal to 1 vertical (1H:1V). Depending upon the construction method, groundwater levels at the time of construction and weather conditions, some local flattening of the slope may be required. An adequate temporary support system should be provided to all locations where space limitations prevent construction of sufficiently shallow slopes or where required to provide protection to existing buried services, roadways or other existing facilities.

Numerous existing services are anticipated to be present within the existing road-right-of-ways. The majority of these services were likely constructed using open cut methods and the service trenches are anticipated to be backfilled with variable fill materials. In areas where the excavations intersect the existing backfill materials, further flattening of the side slopes of the excavations will likely be required. Alternatively, temporary shoring systems may be used to support the sidewalls of the excavations in these areas.

Conventional excavation equipment should be suitable for temporary excavation through the fill materials and granular deposits encountered in the boreholes. However, some difficulty may be expected in excavating cobbles and boulders encountered in the soils at this site.

6.2 Pipe Bedding and Trench Backfilling

The excavation for the forcemain pipe bedding should be carried out to a depth of 300 mm below the bottom of the pipe. The base of the excavation may be below the water table and it is therefore recommended that



20 MPa rapid setting underwater concrete be used as bedding for the forcemain; pumping of water from or dewatering of the excavation would not be necessary for this concrete bedding installation. We understand that the concrete slab should be 200 mm thick. After the concrete sets (which should not be more than 2 hours), a 100 mm thick layer of concrete sand should be placed and rolled carefully to provide uniform support for the pipes. Details on bedding construction including materials are given in SP1, Special Provision for Concrete Bedding for the Forcemain, QEW/Beach Boulevard, dated June 2009.

Before backfilling, any undermined spots observed beneath the existing pavement should be repaired. The materials excavated from trenches for the pipe installations will consist of materials including variable fill, organic silt, and sand to sand and gravel. These materials are not considered suitable for re use as trench backfill. OPSS Granular B Type II material should be used as backfill to the pipe. It should be placed in lifts not exceeding 200 mm and compacted to not less than 95 percent of Standard Proctor maximum density. The top of the granular backfill should be 1.25 m below the grade of the final pavement surface.

The pavement subbase consisting of three (3) 200 mm thick lifts of OPSS Granular B Type II should be placed and compacted to not less than 98 percent of Standard Proctor maximum density. Following the subbase compaction, a 150 mm lift of OPSS Granular A base should be placed and compacted to not less than 100 percent of Standard Proctor maximum density. Details on backfilling including construction, materials and their required moisture contents are given in SP2, Special Provision for Backfilling for the Forcemain, QEW/Beach Boulevard, dated May 2009.

6.3 Widening

Assuming that the traffic will run on the temporary widening and the median for a period of about 1 month and that the current flexible pavement on the QEW can withstand about 110 million ESAL's, the traffic anticipated in these areas will be about 450,000 ESAL's. The coring and borehole investigation indicated that there is about 200 mm of HMA in the median underlain by about 300 mm of crusher run granular base material and then sand and gravel granular fill; this should be sufficient to support the temporary traffic in the median.

For the pavement widening, the topsoil should be stripped from the side slopes and the side slope should be benched. New fill material should be placed in lifts not exceeding 200 mm and compacted to not less than 95 percent of Standard Proctor maximum density. The existing pavement on the paved part of the shoulder consists of about 150 mm of HMA underlain by about 450 mm of granular base/subbase. In the widening, the temporary pavement should consist of 150 mm of HMA placed over 150 mm of OPSS Granular A base and 300 mm of Granular B Type II subbase. The granular base should be compacted to not less than 100 percent of Standard Proctor maximum density and the subbase should be compacted to not less than 98 percent of Standard Proctor maximum density.

6.4 Asphalt Restoration

After the backfill, subbase and granular base are completed, the asphalt should be restored. The total depth of the reinstated asphalt portion of the pavement should match the existing adjacent pavement structure. As the total thickness of the existing HMA ranges from 410 mm to 580 mm with a mean of 475 mm, it is recommended that the reinstated HMA be 500 mm thick. It should consist of 450 mm of Superpave 25.0 binder course and 50 mm of Superpave 12.5 FC2 surface course. It is recommended that the binder course be placed in four equal lifts. The Superpave 25.0 mix should incorporate a PG 64-28 asphalt cement and the Superpave 12.5 FC2 mix should incorporate a PG 70-28 asphalt cement.



A step joint should be formed between the patch and the adjacent existing pavement to a depth of the top two lifts (50 mm surface course and about 110 mm top binder course). The exposed surface should be cleaned and tacked in accordance with OPSS 308, Construction Specification for Tack Coating and Joint Painting.

Details for asphalt restoration including construction and materials are given in SP3, Special Provision for Asphalt Pavement Restoration for Open Cut, QEW/Beach Boulevard, dated May 2009.

6.5 Anticipated Restored Pavement Performance

Assuming that the Average Annual Daily Traffic (AADT) on the above section of the QEW is about 120,000 and the percent of heavy trucks is about 15, the traffic loading calculated for a period of 20 years is 110 million ESAL's. The calculated design structural number to carry this traffic loading is 200 mm.

The restored pavement structure will consist of 500 mm of HMA placed over 150 mm of granular base, 600 mm of subbase and Granular B Type II backfill. The structural number of the pavement structure is at least 280 mm, well above the required 200 mm. The pavement at this location should have sufficient structural capacity to support the anticipated traffic loading. Therefore, no major pavement rehabilitation or structural improvement will be necessary.

Due to time constraints, the placement of the required HMA layers will have to be quick, it is likely that the placed HMA layer will not cool enough before the subsequent HMA layer is placed. This may cause some hair line cracking in the newly placed asphalt, thus somewhat shortening the life of the asphalt.

6.6 Decommissioning of Piezometers

Piezometers were installed at the locations of Boreholes BH101, BH102 and BH107 as part of the geotechnical investigation for this project. Effective August 1, 2003 procedures for borehole and well drilling were legislated under Ontario Regulation (O. Reg.) 903 amended by O. Reg. 128/03 of the Ontario Water Resources Act. This regulation outlines new responsibilities for the drilling/well contractor and the owners of boreholes/wells and stipulates that all boreholes/wells be properly decommissioned in accordance with the regulation. The contract documents should require the contractor to decommission the piezometers in accordance with this regulation. Full decommissioning of all piezometers should be completed prior to construction.

6.7 Further Work

Any requirements for open cut across the QEW should be confirmed by the Ministry of Transportation (MTO). Prior to tendering, the geotechnical aspects of the final design drawings and specifications and proposed construction methodology should be reviewed by this office to confirm that the intent of this report has been met.

We recommend that contractors bidding on this project supply an open cut installation method statement as part of their bid, which would include details on the proposed excavation and pavement restoration methods and machinery, and which would be reviewed on behalf of the owner by personnel with experience with trenchless installation projects prior to contract award. During construction, sufficient inspections, in situ density tests and materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications.

We trust that this report provides sufficient geotechnical engineering information for you to proceed with the preliminary design of this project. If you have any questions regarding the contents of this report, please do not hesitate to contact our office.



Report Signature Page

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LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

| | |
|-------------|---------------------------------------|
| π | 3.1416 |
| $\ln x$, | natural logarithm of x |
| \log_{10} | x or log x, logarithm of x to base 10 |
| g | acceleration due to gravity |
| t | time |
| F | factor of safety |
| V | volume |
| W | weight |

II. STRESS AND STRAIN

| | |
|--------------------------------|--|
| γ | shear strain |
| Δ | change in, e.g. in stress: $\Delta \sigma$ |
| ϵ | linear strain |
| ϵ_v | volumetric strain |
| η | coefficient of viscosity |
| ν | poisson's ratio |
| σ | total stress |
| σ' | effective stress ($\sigma' = \sigma - u$) |
| σ'_{vo} | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stress (major, intermediate, minor) |
| σ_{oct} | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| τ | shear stress |
| u | porewater pressure |
| E | modulus of deformation |
| G | shear modulus of deformation |
| K | bulk modulus of compressibility |

III. SOIL PROPERTIES

(a) Index Properties

| | |
|--------------------|--|
| $\rho(\gamma)$ | bulk density (bulk unit weight*) |
| $\rho_d(\gamma_d)$ | dry density (dry unit weight) |
| $\rho_w(\gamma_w)$ | density (unit weight) of water |
| $\rho_s(\gamma_s)$ | density (unit weight) of solid particles |
| γ' | unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$) |
| D_R | relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s) |
| e | void ratio |
| n | porosity |
| S | degree of saturation |

(a) Index Properties (continued)

| | |
|-----------|--|
| w | water content |
| w_l | liquid limit |
| w_p | plastic limit |
| I_p | plasticity index $= (w_l - w_p)$ |
| w_s | shrinkage limit |
| I_L | liquidity index $= (w - w_p) / I_p$ |
| I_C | consistency index $= (w_l - w) / I_p$ |
| e_{max} | void ratio in loosest state |
| e_{min} | void ratio in densest state |
| I_D | density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density) |

(b) Hydraulic Properties

| | |
|---|--|
| h | hydraulic head or potential |
| q | rate of flow |
| v | velocity of flow |
| i | hydraulic gradient |
| k | hydraulic conductivity (coefficient of permeability) |
| j | seepage force per unit volume |

(c) Consolidation (one-dimensional)

| | |
|-------------|---|
| C_c | compression index (normally consolidated range) |
| C_r | recompression index (over-consolidated range) |
| C_s | swelling index |
| C_a | coefficient of secondary consolidation |
| m_v | coefficient of volume change |
| c_v | coefficient of consolidation |
| T_v | time factor (vertical direction) |
| U | degree of consolidation |
| σ'_p | pre-consolidation pressure |
| OCR | over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$ |

(d) Shear Strength

| | |
|------------------|--|
| τ_p, τ_r | peak and residual shear strength |
| ϕ' | effective angle of internal friction |
| δ | angle of interface friction |
| μ | coefficient of friction $= \tan \delta$ |
| c' | effective cohesion |
| c_u, s_u | undrained shear strength ($\phi = 0$ analysis) |
| p | mean total stress $(\sigma_1 + \sigma_3)/2$ |
| p' | mean effective stress $(\sigma'_1 + \sigma'_3)/2$ |
| q | $(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$ |
| q_u | compressive strength $(\sigma_1 + \sigma_3)$ |
| S_t | sensitivity |

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

| | |
|----|---------------------|
| AS | Auger sample |
| BS | Block sample |
| CS | Chunk sample |
| SS | Split-spoon |
| DS | Denison type sample |
| FS | Foil sample |
| RC | Rock core |
| SC | Soil core |
| ST | Slotted tube |
| TO | Thin-walled, open |
| TP | Thin-walled, piston |
| WS | Wash sample |

III. SOIL DESCRIPTION

(a) Cohesionless Soils

| Density Index (Relative Density) | N Blows/300 mm or Blows/ft. |
|-------------------------------------|--------------------------------|
| Very loose | 0 to 4 |
| Loose | 4 to 10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very dense | over 50 |

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

| | c_u, s_u | kPa | psf |
|------------|------------|------------|----------------|
| Very soft | | 0 to 12 | 0 to 250 |
| Soft | | 12 to 25 | 250 to 500 |
| Firm | | 25 to 50 | 500 to 1,000 |
| Stiff | | 50 to 100 | 1,000 to 2,000 |
| Very stiff | | 100 to 200 | 2,000 to 4,000 |
| Hard | | over 200 | over 4,000 |

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

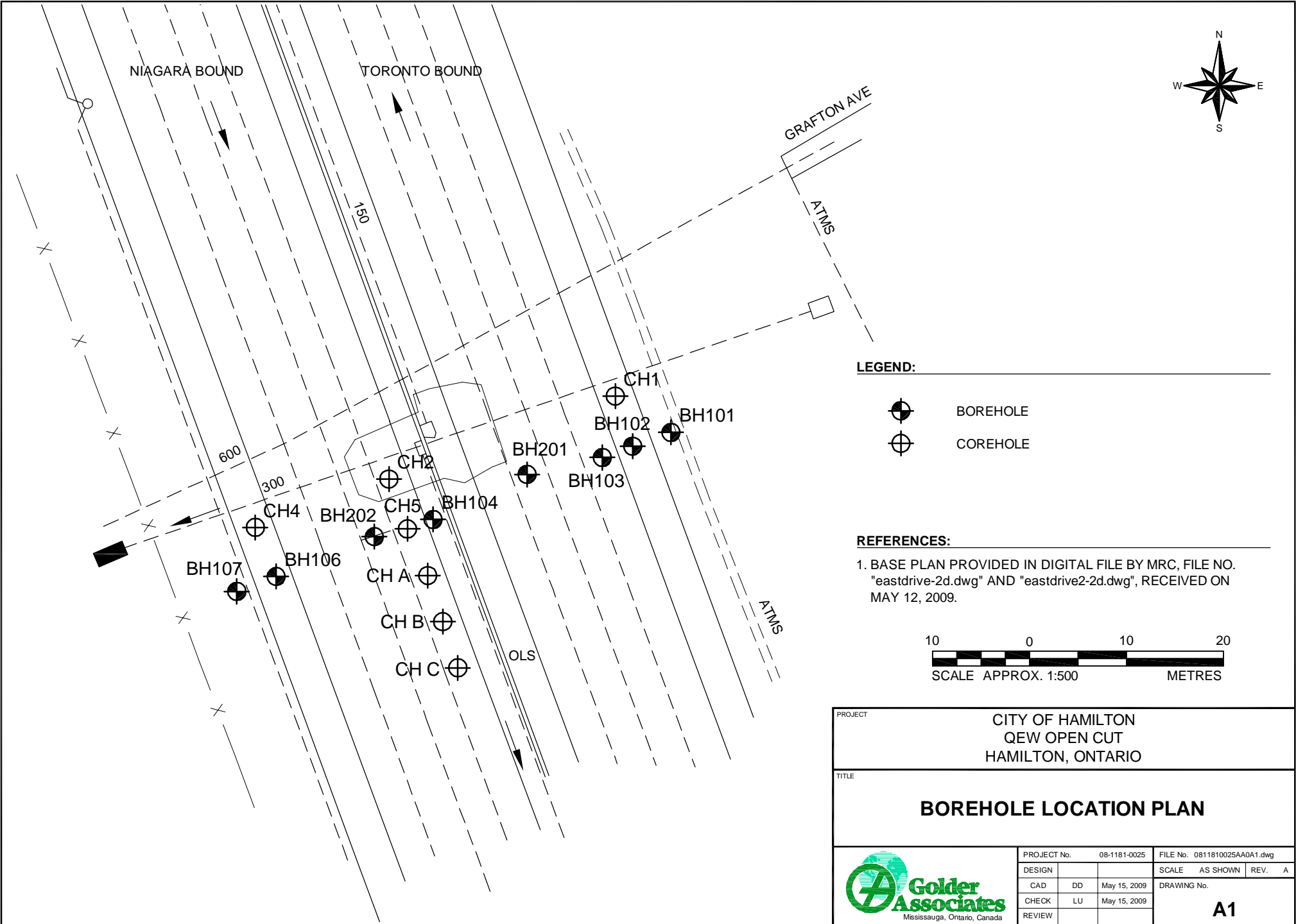
| | |
|-----------------|---|
| w | water content |
| w_p | plastic limit |
| w_l | liquid limit |
| C | consolidation (oedometer) test |
| CHEM | chemical analysis (refer to text) |
| CID | consolidated isotropically drained triaxial test ¹ |
| CIU | consolidated isotropically undrained triaxial test with porewater pressure measurement ¹ |
| D_R | relative density (specific gravity, G_s) |
| DS | direct shear test |
| M | sieve analysis for particle size |
| MH | combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | organic content test |
| SO ₄ | concentration of water-soluble sulphates |
| UC | unconfined compression test |
| UU | unconsolidated undrained triaxial test |
| V | field vane (LV-laboratory vane test) |
| γ | unit weight |

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.



APPENDIX A

Borehole location plan and road cross section



| | | | |
|---|-----------------|---|-----------------|
| PROJECT | | CITY OF HAMILTON QEW OPEN CUT HAMILTON, ONTARIO | |
| TITLE | | | |
| BOREHOLE LOCATION PLAN | | | |
|  Mississauga, Ontario, Canada | | PROJECT No. 08-1181-0025 | |
| | | FILE No. 0811810025AA0A1.dwg | |
| | | DESIGN | |
| | | CAD | DD May 15, 2009 |
| CHECK | LU May 15, 2009 | SCALE AS SHOWN | REV. A |
| REVIEW | | DRAWING No. | |
| | | A1 | |





APPENDIX B

Record of Boreholes - BH 101 to BH 202

PROJECT: 08-1181-0025

LOCATION: SEE FIGURE 2

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE BH 101

BORING DATE: January 26, 2009

SHEET 1 OF 1

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | | ELEVATION | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | INSTALLATION AND GROUNDWATER OBSERVATIONS | |
|-----------------------|---|---|-------------|-----------------------|---------|----------|------------|-----------|---|------------------|--------|--------|------------------------------------|------------------|------------------|------------------|----------------------------|--|--------------------------------------|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.3m | | 20 | 40 | 60 | 80 | 10 ⁻⁶ | 10 ⁻⁵ | 10 ⁻⁴ | 10 ⁻³ | | | |
| | | | | | | | | | SHEAR STRENGTH Cu, kPa | nat V. rem V. | + - | Q U | Wp | W | WI | | | | |
| | | | | | | | | | 25 | 50 | 75 | 100 | | 20 | 40 | 60 | 80 | | |
| 0 | POWER AUGER 200 mm Dia. Hollow Stem Augers | GROUND SURFACE | | 77.54 | | | | | | | | | | | | | | | Concrete and Protective Casing |
| | | Frost impacted brown sand and gravel, some silt (crusher run) (BASE) | | 0.00 | 1 | AS | - | | | | | | | | | | | | |
| | | | | 76.93 | | | | | | | | | | | | | | | |
| | | Frost impacted sand and gravel, some silt (crusher run) (SUBBASE) | | 0.61 | 2 | 50 DO | 18 | | | | | | | | | | | | |
| 1 | | | | 76.09 | | | | | | | | | | | | | | | |
| | | Dense brown silty sand, coarse rock fragments (FILL) | | 1.45 | 3 | 50 DO | 35 | | | | | | | | | | | | |
| 2 | | | | 75.33 | | | | | | | | | | | | | | | |
| | | Compact brown sand, trace silt, trace gravel (FILL) | | 2.21 | 4 | 50 DO | 17 | | | | | | | | | | | | |
| 3 | | | | 74.57 | | | | | | | | | | | | | | | |
| | | Loose brown SAND, trace silt, some gravel, trace shells | | 2.97 | 5A | 50 DO | 5 | | | | | | | | | | | | |
| | 200 mm Dia. Hollow Stem Augers | | | 74.03 | | | | | | | | | | | | | | Silica Sand Filter | |
| | | Loose dark brown organic SILT, some sand and sand seams | | 3.51 | 5B | 50 DO | | | | | | | | | | | | | |
| | | | | 73.81 | | | | | | | | | | | | | | | |
| | | Loose to dense brown to grey SAND, trace to some silt and gravel, trace shells | | 3.73 | | | | | | | | | | | | | | | |
| 4 | | | | | 6 | 50 DO | 9 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| 5 | | | | | 7 | 50 DO | 17 | | | | | | | | | | | | |
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| | | | | | 8 | 50 DO | 13 | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | | | | | | | |
| | 200 mm Dia. Hollow Stem Augers | | | | 9 | 50 DO | 46 | | | | | | | | | | | MH Cave | |
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| 7 | 200 mm Dia. Hollow Stem Augers | END OF BOREHOLE | | 70.83 | | | | | | | | | | | | | | Water encountered during drilling at a depth of 2.9 m, Jan. 26/09 Borehole caved to a depth of 4.6 m, Jan. 26/09 Water level in open portion of borehole at a depth of 2.8 m or elevation of 74.8 m upon completion of drilling, Jan. 26/09 Water level in piezometer at a depth of 2.90 m or elevation of 74.64 m, Feb. 16/09 | |
| | | | | 6.71 | | | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED:

LDN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE BH 102

BORING DATE: January 27, 2009

SHEET 1 OF 1

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

[illegible]

LDN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING DATE: January 27-28, 2009

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

1 : 50



CHECKED:

PROJECT: 08-1181-0025

RECORD OF BOREHOLE BH 104

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: January 22, 2009

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | | ELEVATION | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | INSTALLATION AND GROUNDWATER OBSERVATIONS |
|-----------------------|---------------|--------------|-------------|--------------|---------|------|------------|-----------|---|--|--|--|------------------------------------|--|--|--|----------------------------|--|
| | | DESCRIPTION | STRATA PLOT | ELEV. | NUMBER | TYPE | BLOWS/0.3m | | | | | | | | | | | |
| | | | | DEPTH (m) | | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED:

LDN_BHS 08-1181-0025.GPJ GLDR LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING DATE: January 26, 2009

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

LDN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

1 : 50



SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE BH 107

BORING DATE: January 25-26, 2009

SHEET 1 OF 1

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

[illegible]

LDN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE BH 201

SHEET 1 OF 1

BORING DATE: February 16, 2009

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

[illegible]

LDN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

DEPTH SCALE

1 : 50



LOGGED: RMN

CHECKED:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING DATE: February 16, 2009

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DN_BHS 08-1181-0025.GPJ GLDR_LDN.GDT 4/24/09 DATA INPUT: ph 2009/02

1 : 50





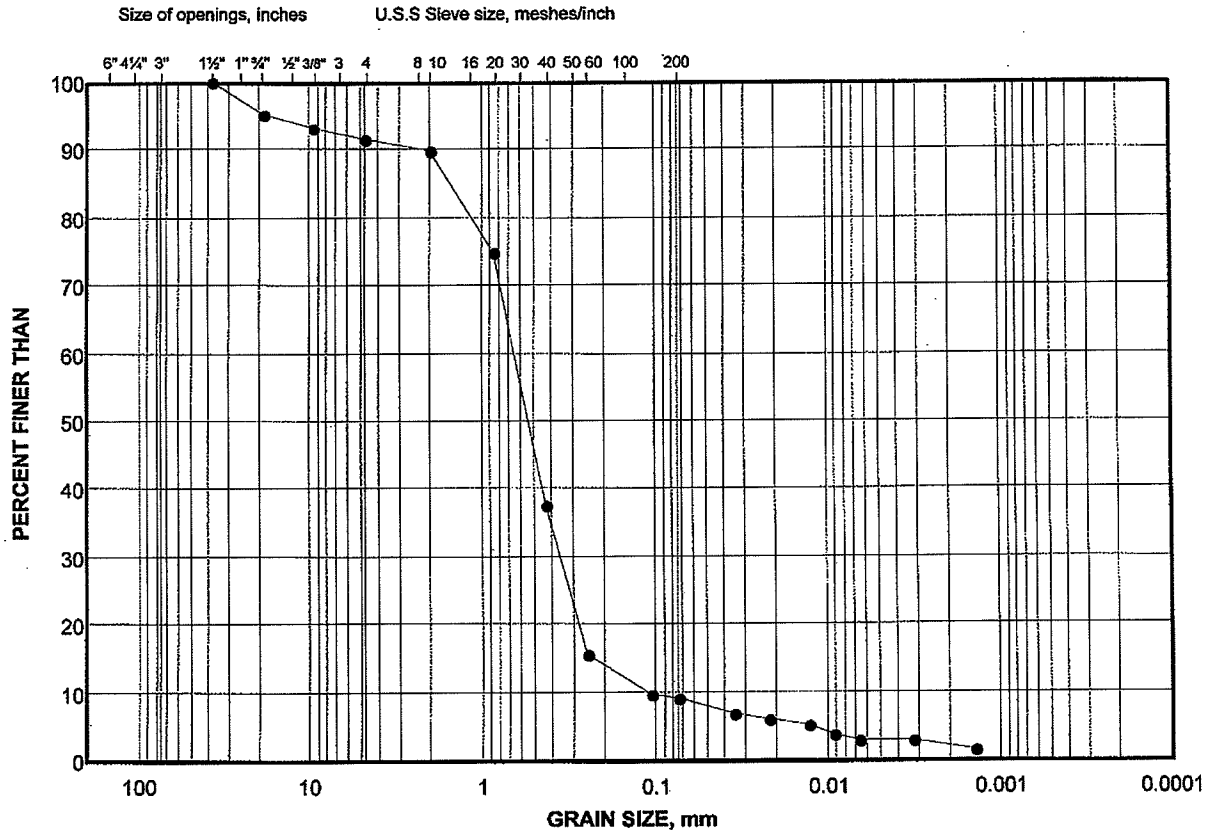
APPENDIX C

Grain size distribution - Figures C1 to C11

GRAIN SIZE DISTRIBUTION

Sand, trace silt, trace gravel

FIGURE 1



| | | | | | | |
|--------|-------------|------|-----------|--------|------|---------------------|
| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 09-101 | 8 | 5.30 - 5.90 |

Project Number: 08-1181-0025

Checked By: _____

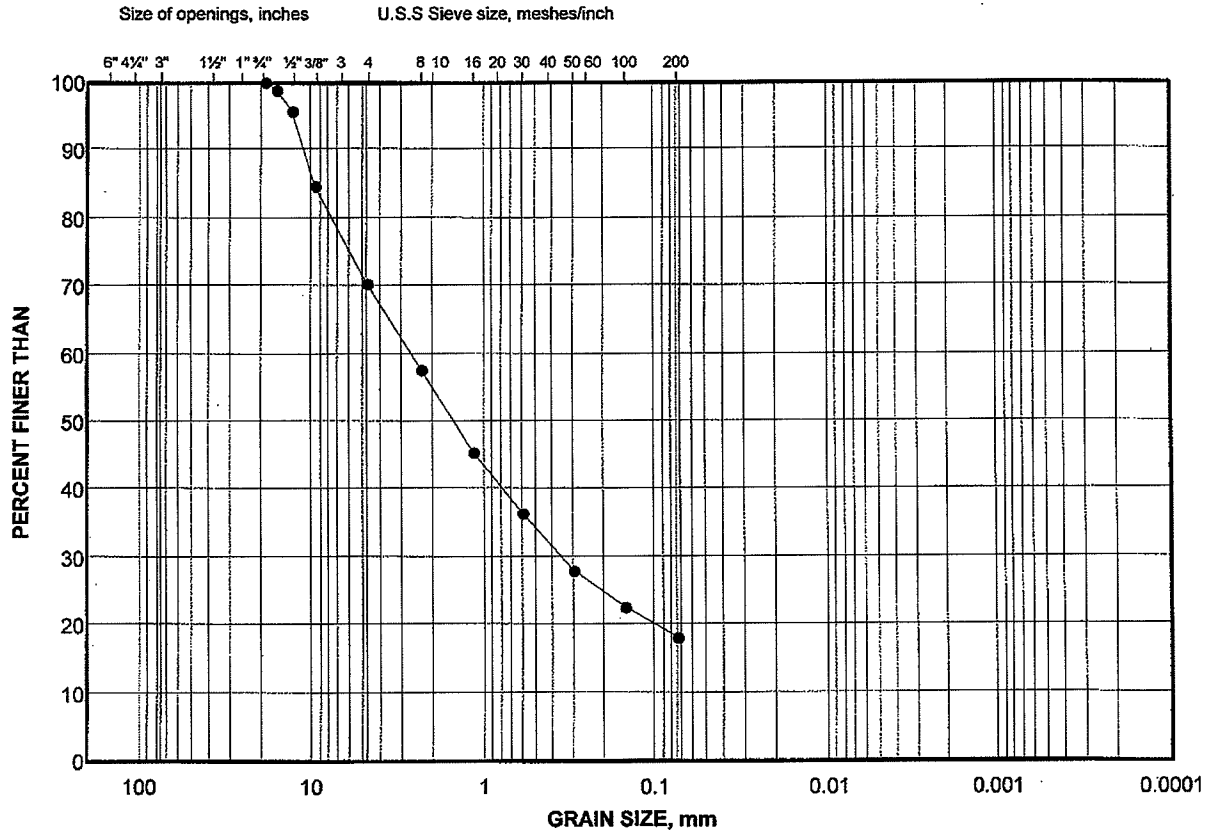
Golder Associates

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Gravelly sand, some silt

FIGURE 2



| | | | | | | |
|----------------|-------------|------|-----------|--------|------|---------------------|
| COBBLE SIZE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| | GRAVEL SIZE | | SAND SIZE | | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 09-102 | 2 | 0.80 - 1.40 |

Project Number: 08-1181-0025

Checked By: _____

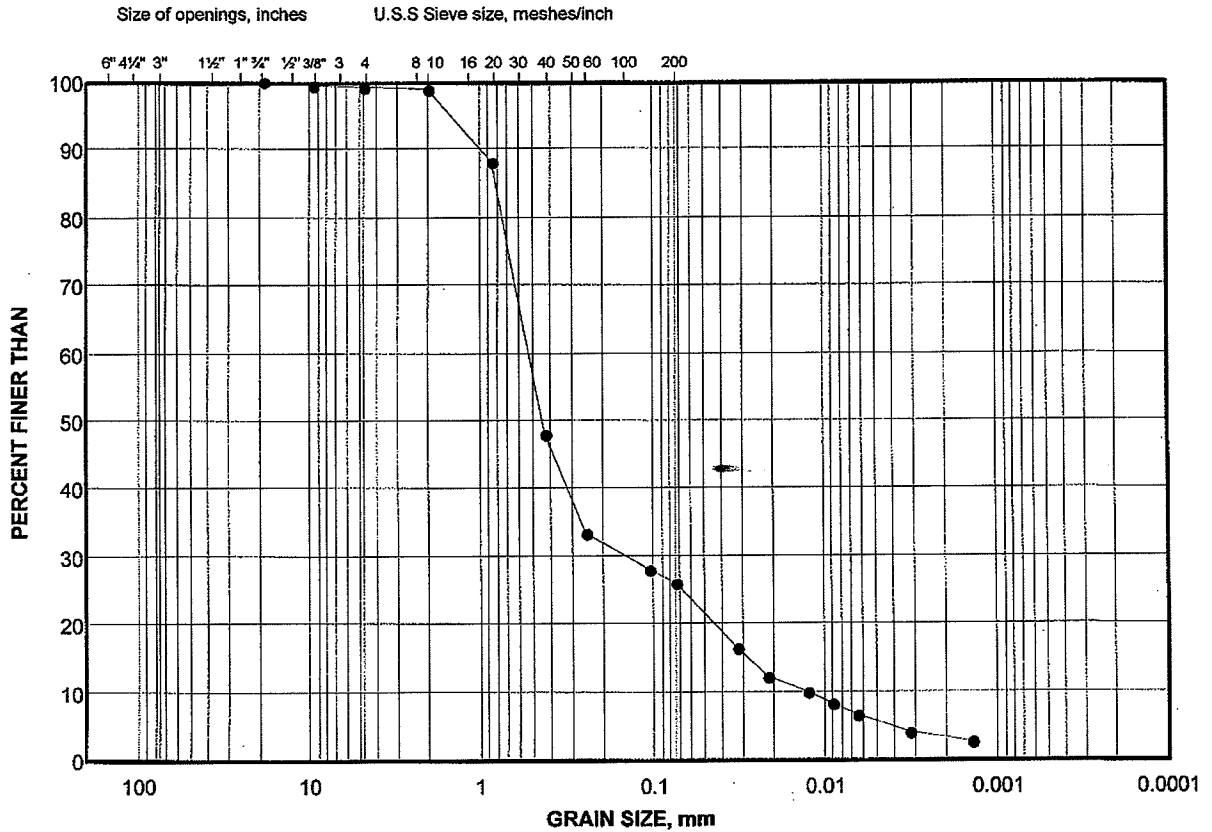
Golder Associates

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Sand with silt, trace gravel

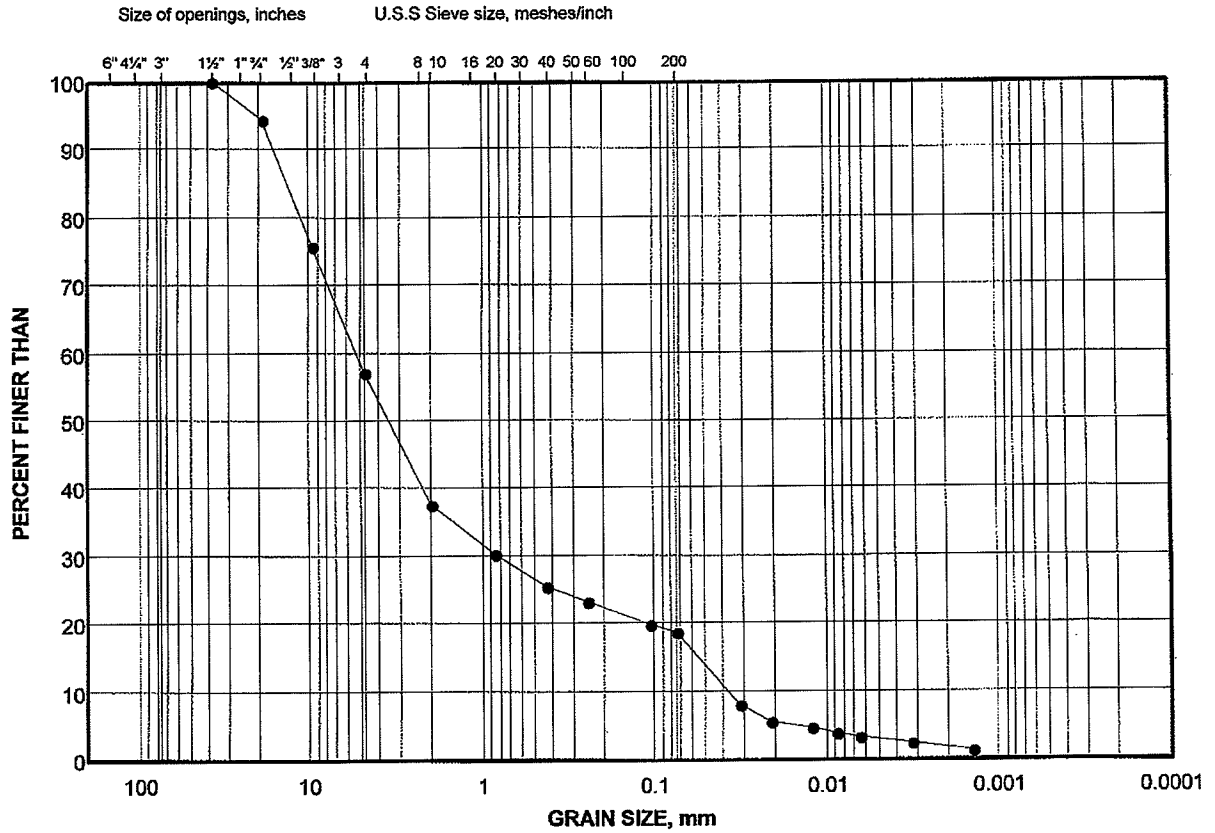
FIGURE 3



GRAIN SIZE DISTRIBUTION

Sand and Gravel, some silt

FIGURE 4



| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
|--------|-------------|------|-----------|--------|------|---------------------|
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 09-103 | 1 | 0.50 - 0.80 |

Project Number: 08-1181-0025

Checked By: _____

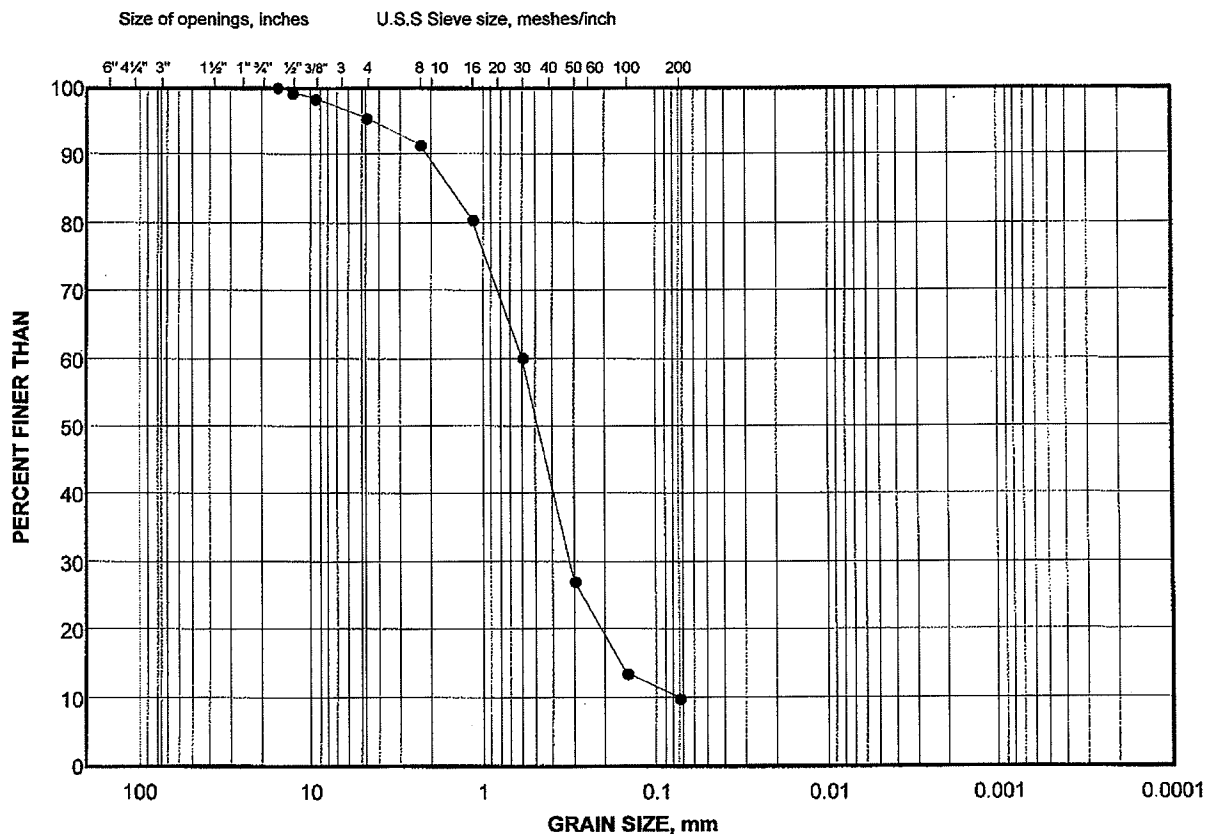
Golder Associates

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Sand, trace silt, trace gravel

FIGURE 5



Sand, trace gravel, trace silt

Size of openings, inches

U.S.S Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

| Grain Size (mm) | Sieve Size (U.S.S) | Percent Finer (%) |
|-----------------|--------------------|-------------------|
| 75 | No. 20 | 100 |
| 60 | No. 25 | 99 |
| 47.5 | No. 30 | 98 |
| 37.5 | No. 40 | 97 |
| 25 | No. 60 | 94 |
| 150 | No. 100 | 91 |
| 75 | No. 200 | 81 |
| 425 | No. 35 | 65 |
| 750 | No. 20 | 28 |
| 1500 | No. 10 | 15 |
| 2500 | No. 6 | 11 |

| | | | | | | |
|--------|-------------|------|-----------|--------|------|---------------------|
| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

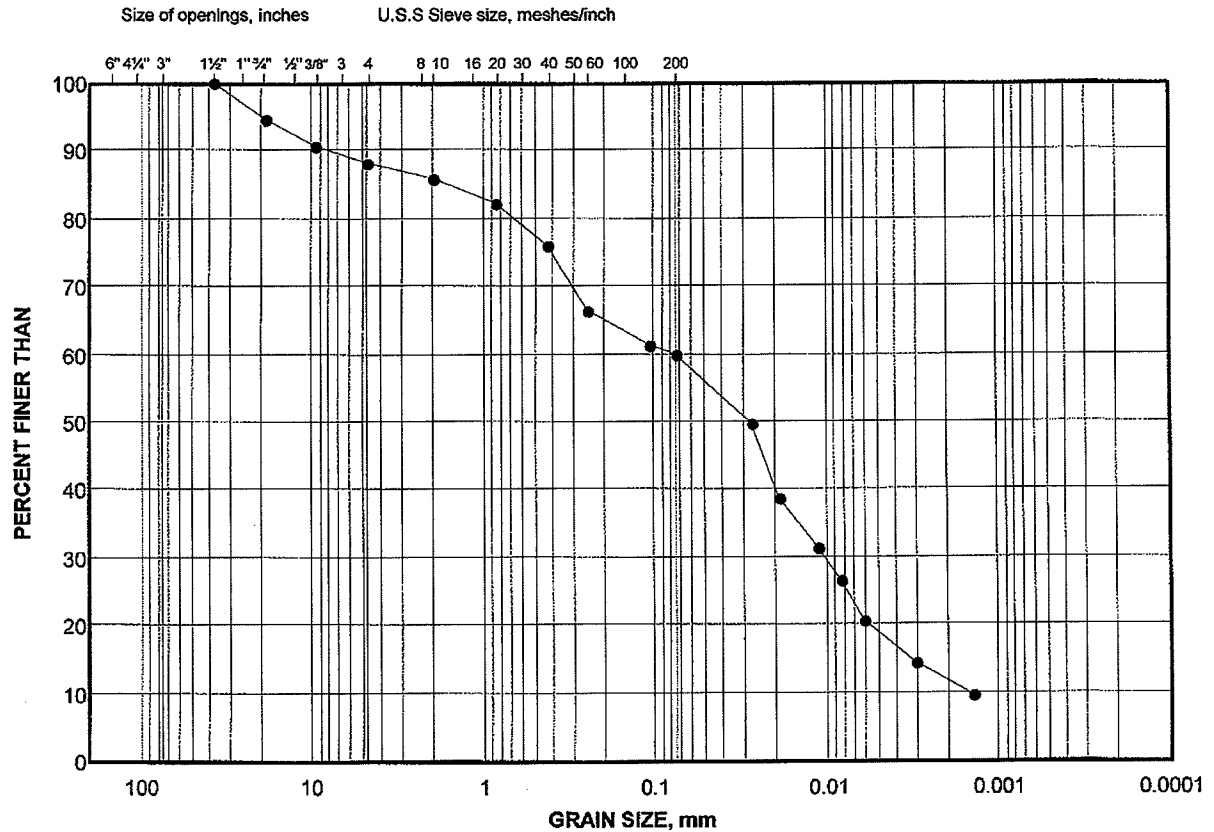
| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 09-106 | 3 | 1.50 - 2.10 |

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Sandy Silt, trace gravel

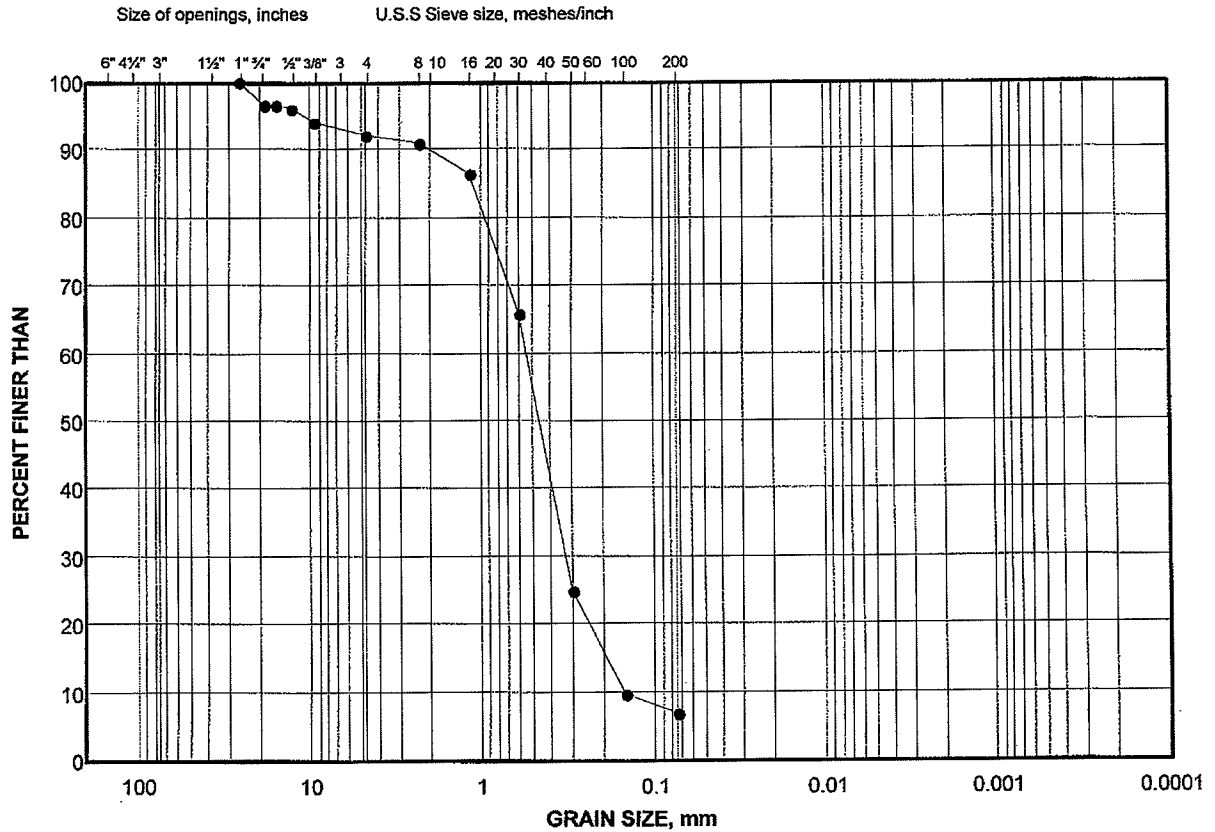
FIGURE 7



GRAIN SIZE DISTRIBUTION

Sand, trace gravel, trace silt

FIGURE 8



| | | | | | | |
|--------|-------------|------|-----------|--------|------|---------------------|
| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 09-106 | 10 | 6.90 - 7.50 |

Project Number: 08-1181-0025

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Date: 30-Apr-09

Sand with Gravel, trace silt

Size of openings, inches

U.S.S Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

| Grain Size (mm) | U.S.S Sieve Size (meshes/inch) | Percent Finer (%) |
|-----------------|--------------------------------|-------------------|
| 4.75 | 10 | 100 |
| 3.75 | 40 | 87 |
| 3.0 | 60 | 83 |
| 2.5 | 75 | 79 |
| 2.0 | 100 | 76 |
| 1.5 | 150 | 71 |
| 1.18 | 200 | 65 |
| 0.85 | 250 | 53 |
| 0.6 | 300 | 39 |
| 0.425 | 400 | 18 |
| 0.3 | 500 | 10 |
| 0.25 | 600 | 8 |

| | | | | | | |
|----------------|-------------|------|-----------|--------|------|---------------------|
| COBBLE SIZE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

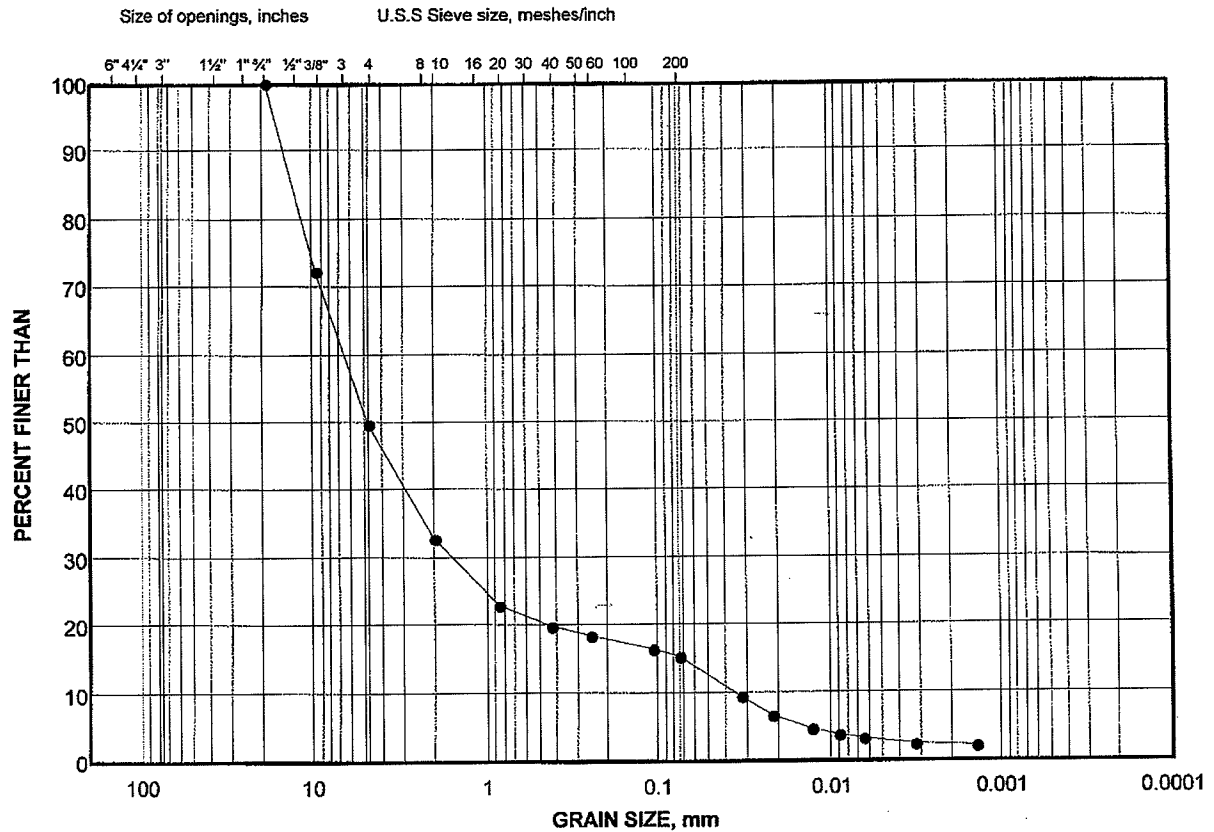
| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| ● | 201 | 3 | 2.30 - 2.70 |

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Sand and Gravel, some silt

FIGURE 10



| | | | | | | |
|--------|-------------|------|-----------|--------|------|---------------------|
| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|-------------|
| • | 202 | 1 | 0.51 - 0.86 |

Project Number: 08-1181-0025

Checked By: _____

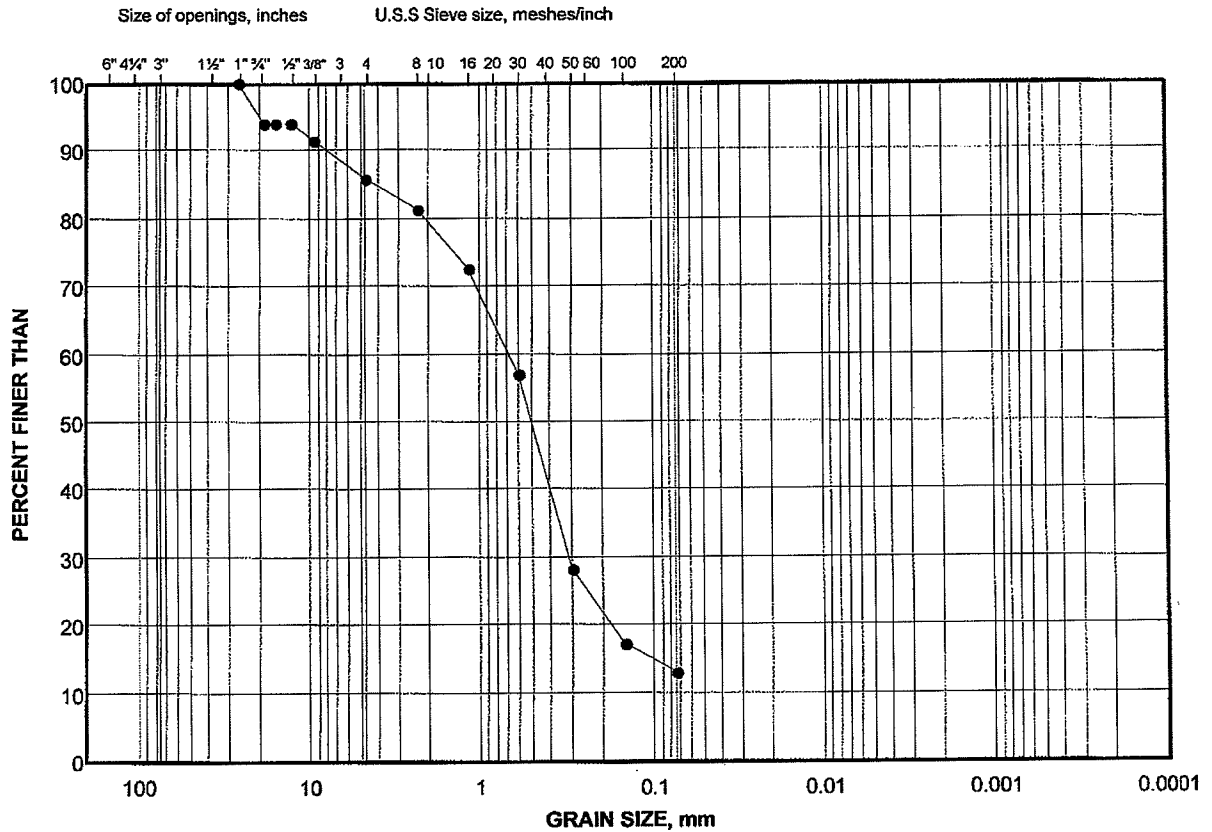
Golder Associates

Date: 30-Apr-09

GRAIN SIZE DISTRIBUTION

Sand, some gravel, some silt

FIGURE 11



| | | | | | | |
|--------|-------------|------|-----------|--------|------|---------------------|
| COBBLE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND CLAY SIZES |
| SIZE | GRAVEL SIZE | | SAND SIZE | | | FINE GRAINED |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|--------|----------|--------|------------|
| • | 202 | 4 | 3.0 - 3.50 |

Project Number: 08-1181-0025

Checked By: _____

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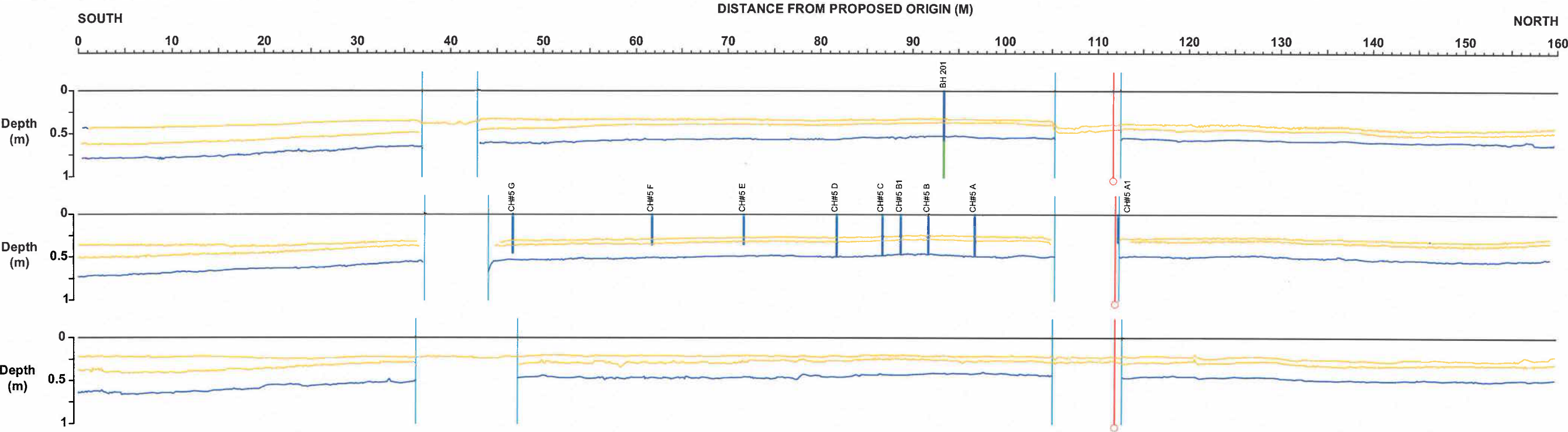
Date: 30-Apr-09



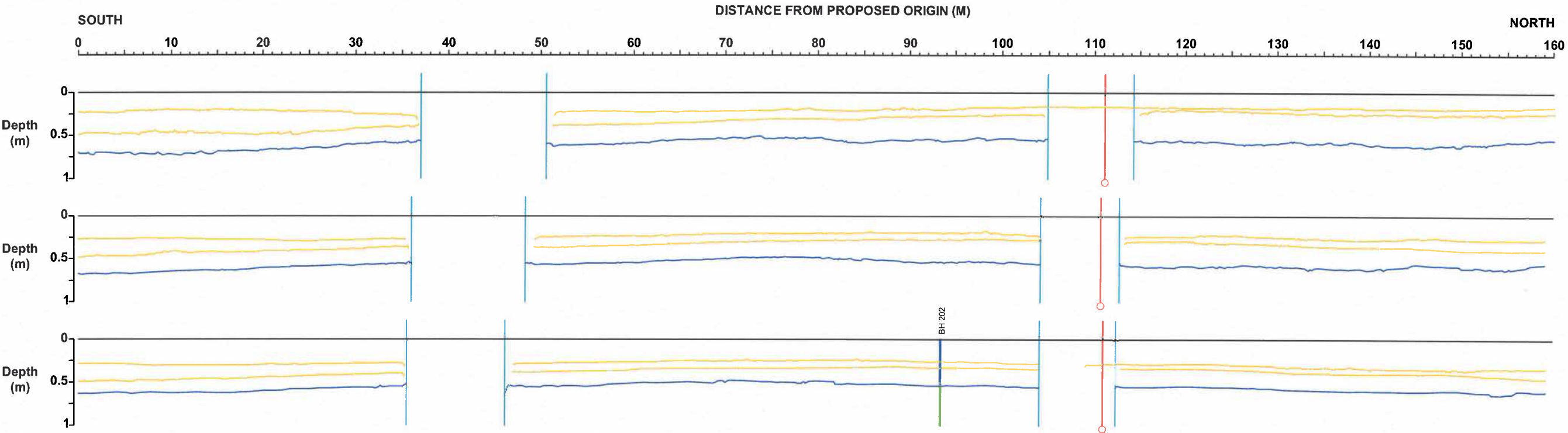
APPENDIX D

Pavement GPR profiles

NIAGARA BOUND LANES



TORONTO BOUND LANES



GPR INTERPRETED LEGEND

- BASE OF ASPHALT LAYER
- REFLECTOR WITHIN ASPHALT
- REFLECTOR WITHIN GRANULAR MATERIAL
- EXTENT OF ASPHALT PATCH AREA
- LOCATION OF BURIED PIPE

LEGEND

- ASPHALT
- ASPHALT
- ASPHALT
- GRANULAR MATERIAL

NOTES

- DEPTH BASED ON ESTIMATED GPR VELOCITY OF 0.12 M/NS
- GPR SURVEY CONDUCTED USING 1000 AND 250 MHZ ANTENNAE
- GPR DATA COLLECTED ALONG THE CENTRE LINE OF EACH LANE



FILE No.
PROJECT No. 08-1181-0025

SCALE AS SHOWN
DATE MARCH 2009
DESIGN CRP
CAD CRP
CHECK
REVIEW

TITLE
**GPR INTERPRETED RESULTS
QEW BEACH FORCE SEWER MAIN**
CITY OF HAMILTON
FIGURE **D1**



APPENDIX E

Special Provisions SP1, SP2 and SP3

**SPECIAL PROVISION FOR CONCRETE BEDDING FOR THE FORCEMAIN
QEW/BEACH BOULEVARD**

This Special Provision amends the requirements of OPSS 514, Construction Specification for Trenching, Backfilling and Compacting, April 2008.

1. EQUIPMENT

Standard construction equipment will be required.

2. CONSTRUCTION

The excavation for the pipe installation shall be carried out to a depth of 300 mm below the bottom of the pipes. As the bottom of the excavation may be below the water level, a part of the concrete bedding may have to be done under water. The bottom of the excavation shall be made as smooth as possible with the standard excavation equipment. Care should be taken not to disturb the underlying soil. Any loose material shall be removed from the bottom of the excavation.

Then, a 200 mm thick concrete bedding slab shall be poured in the bottom of the trench. The concrete shall be self leveling so finishing is not required. After the concrete sets (2 hours), a 100 mm thick layer of concrete sand shall be placed and rolled carefully. The sand is to provide a uniform support for the pipes.

3. MATERIALS

The concrete used for the bedding shall be a 20 MPa rapid setting Portland cement concrete with viscosity modifier admixtures so that the concrete can be placed and will set under water. The concrete shall set in no more than 2 hours. Calcium chloride shall not be used as an accelerator in the concrete mix. Data sheets for any accelerating admixture used shall be supplied indicating the admixture does not contain calcium chloride.

A field trial shall be carried out by the contractor at least a week before construction to confirm that the concrete meets the requirements. The trial shall be approved by the Engineer before construction commences.

The concrete sand bedding shall meet the requirements of the OPS 1020 specification for concrete fine aggregate.

SPECIAL PROVISION FOR BACKFILLING OF THE FORCEMAIN
QEW/BEACH BOULEVARD

This Special Provision amends the requirements of OPSS 514, Construction Specification for Trenching, Backfilling, and Compacting, April 2008 and OPSS 314, Construction Specification for Untreated Granular Subbase, Base, Surface Shoulder and Stockpiling, November 2004.

1. EQUIPMENT

Standard construction equipment will be required. In addition, a conveyer belt and a Bobcat or similar equipment should be used for the placement of the granular backfilling materials. A small tamping compactor or roller should be used for the compaction of the backfilling between and outside the pipes. A hand tamper/compactor shall be used for compaction in close vicinity of the pipes.

2. CONSTRUCTION

Any undermined areas along the edge of the pavement shall be repaired.

OPSS Granular B Type II material shall be used as the cover and backfill material. The backfill shall be placed to a depth of 1,250 mm below the top of the final pavement surface or 750 mm below the asphalt layers, in lifts not exceeding 200 mm and compacted to not less than 95 percent of Standard Proctor maximum dry density.

The Granular B Type II material shall then be placed on top of the backfill in three 200 mm thick lifts and compacted to not less than 98 percent of Standard Proctor maximum dry density.

Then, a 150 mm lift of OPSS Granular A base shall be placed and compacted to not less than 100 percent of Standard Proctor maximum density before the asphalt layers are paved.

3. MATERIALS

An OPSS Granular B Type II material meeting the requirements of OPSS 1010, Aggregates – Base, Subbase, Select Subgrade, and Backfill Material, April 2004 shall be used as the cover, backfill and subbase materials.

An OPSS Granular A material meeting the requirements of OPSS 1010, Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material, April 2004 shall be used in the granular base layer.

The average field moisture content of both materials shall be maintained within the range of not less than 2.0 percent lower than and not more than 1.0 percent greater than the optimum moisture content (OMC).

**SPECIAL PROVISION FOR ASPHALT PAVEMENT RESTORATION FOR
OPEN CUT, QEW/BEACH BOULEVARD**

This Special Provision amends the requirements of OPSS 507, Construction Specification for Site Restoration Following Installation of Pipelines, Utilities, and Associated Structures, November 2005.

1. EQUIPMENT

Standard asphalt paving and compaction equipment will be required. Other construction equipment required includes a small milling machine and a tack coat distributor for hand application.

2. CONSTRUCTION

A total of 450 mm of Superpave 25.0 binder course shall be placed in four (4) lifts of equal thicknesses. For construction purposes, the same thickness of asphalt shall be placed in the median and paved part of the shoulder. The binder course mix shall be placed in the direction perpendicular to the centerline of the road. A Shuttle Buggy[®] material transfer vehicle shall be used to transfer the mix from the delivery trucks to the paver operating in the trench. No tack coat will generally be required; however, if there are hairline cracks observed in the mix, a small amount of RS1 tack coat should be sprayed carefully by hand to seal the cracks. The joint with the adjacent existing asphalt shall be of a step type as shown in the latest OPSD 509.010 drawing. All vertical and horizontal surfaces in the joint shall be sawcut, cleaned carefully and tack coated.

A 50 mm thick Superpave 12.5 FC2 surface course shall then be placed. The surface course shall be placed in the direction parallel to the centerline of the road. The surface course shall not be opened to traffic until the temperature of the mat drops to 50°C; cold water can be used to cool the pavement surface.

The binder course and surface course mixes shall be placed using an asphalt paver. The binder and surface courses shall meet the requirements of OPSS 310, Construction Specification for Hot Mix Asphalt, April 2008. At least two weeks before construction, the contractor shall arrange a trial section replicating site conditions and time constraints and prove that it can produce and place all the asphalt layers to meet the specified requirements.

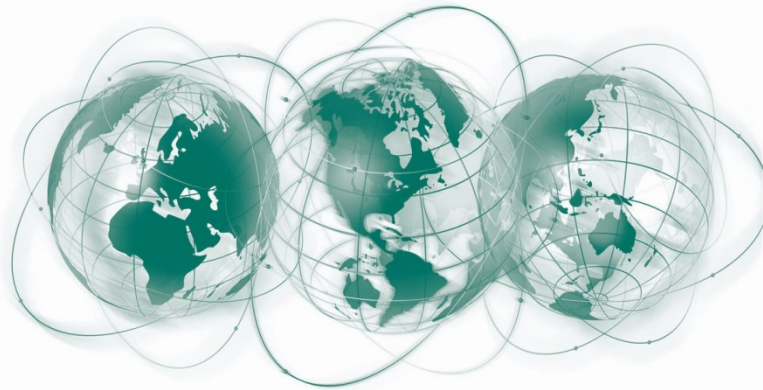
3. MATERIALS

Both, the binder course and surface course mixes shall meet the requirements of OPSS 1151, Material Specification for Superpave and Stone Mastic Asphalt, April 2007. Both mixes shall be designed for Category E traffic loading. The Superpave 25.0 mix shall incorporate a PG 64-28 asphalt cement. The Superpave 12.5 FC2 mix shall incorporate a PG 70-28 asphalt cement.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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