HYDROGEOLOGIC INVESTIGATION

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1.0 Introduction

Enbridge Gas Distribution Inc. (Enbridge) is proposing to install a Nominal Pipe Size (NPS) 16 (16 inch diameter; 406 millimetre) steel pipeline to supply natural gas to the York Energy Centre LP (YEC). The Study Area for the planned pipeline is located in King Township, Ontario (Figure 1; Appendix A), and partially within the natural core, natural linkage and countryside designation of the Oak Ridges Moraine Conservation Plan (ORMCP).

The pipeline is proposed to originate from Enbridge's Schomberg Gate Station located at 4955 Lloydtown-Aurora Road in Pottageville, Ontario and terminate at YEC's facility, planned to be located at 18781 Dufferin Street in King Township, Ontario (Figure 2). The preferred route between the start and end points was determined through an analysis of alternative routes using published information, field reconnaissance, aerial photo interpretation and information provided by landowners, tenants, agencies and the public (JWSL, 2009) and is shown on Figure 2.

1.1 STUDY OBJECTIVES AND TASKS

The purpose of the hydrogeologic assessment is to assist in determining the groundwater conditions at each watercourse crossing and wetland feature along the preferred pipeline route. In order to meet the study objectives, the following tasks will be completed:

- Drill boreholes at each of the nine (9) watercourse crossing locations (WC1 to WC9) and the one (1) wetland crossing location (WTC1) to determine subsurface soil conditions;
- Install monitoring wells at each of the 9 watercourse crossing locations (WC1 to WC9) and one (1) wetland crossing location (WTC1) to determine the groundwater table elevation;
- Install drive-point piezometers at each watercourse/wetland to monitor vertical hydraulic gradients between the surface water and shallow groundwater system;
- Monitor water levels in the monitoring wells and the drive-point piezometers to establish the location of the groundwater table, and identify groundwater-surface water interactions;
- Perform in-situ hydraulic testing at selected monitoring wells to determine the horizontal hydraulic conductivity of the subsurface soils present in the vicinity of the watercourse/wetland crossings; and
- Identify the location of private wells in the area and assess the potential for these wells to be impacted by the proposed pipeline construction.

All the information collected from the aforementioned tasks was used to determine the potential dewatering quantities required to construct the pipeline and assess the potential impacts to the groundwater system and nearby receptors as a result of the dewatering activities.

1.2 REPORT ORGANIZATION

The following report has been organized into eight sections, including this introduction. Section 2.0 presents a general overview of background data related to the Study Area. Section 3.0 presents a description of the proposed project. Section 4.0 details the methodology applied in this study, while Section 5.0 presents the results and a discussion of the hydrogeologic assessment. Potential impacts are discussed in Section 6.0, a report summary is presented in Section 7.0, and Section 8.0 contains the report references. All figures and tables are presented in Appendices A and B, respectively. Appendix C contains the borehole logs for all completed monitoring wells, drive-points and boreholes, while Appendix D contains the results of the in situ hydraulic analysis.

2.0 Background

In a hydrologic investigation conducted by Jacques Whitford Stantec Limited (JWSL) (2009), a detailed assessment of the site conditions at the nine (9) watercourse crossings (WC1 to WC9) and one (1) wetland crossing (WTC1), and a preliminary assessment of groundwater conditions at each location was completed. The following sections provide a review of available background information.

2.1 SITE SETTING

The Study Area for the planned pipeline is located in King Township, Ontario, and partially within the natural core, natural linkage and countryside designation of the ORMCP (Figure 1). Recent aerial imagery of the area indicates the land within the Study Area is primarily used for agriculture purposes.

2.2 PHYSIOGRAPHY

The Study Area is situated within the Simcoe Lowlands physiographic region of southern Ontario (Chapman and Putnam, 1984). South of Cook Bay (Lake Simcoe) extends a broad valley nestled between uplands of high morainic hills, which was once a shallow extension of the lake and is now composed primarily of the Holland Marsh (Figure 2). South of the marsh is the glacial Lake Algonquin shoreline.

According to the Ontario Geological Survey (OGS, 2009), the Study Area is comprised specifically of tilled plains (drumlinized), sand plains and peat and muck. Surficial deposits include stone-poor, carbonate-derived silty to sandy till; fine-textured glaciolacustrine deposits: massive-well laminated; coarse-textured glaciolacustrine deposits: foreshore-basinal deposits and organic deposits.

2.3 SURFACE WATER FEATURES

The Study Area is located within the West Holland River subwatershed of the Lake Simcoe Basin. The West Holland River is the major surface drainage feature in the Study Area (Figure 2). The headwaters of the West Holland River flow north to join the eastern branch of the Holland River before draining into Lake Simcoe. South of Hwy 9, the West Holland River is also known as the Schomberg River.

In addition to the West Holland River, other notable surface water features within the Regional Study Area are the Holland Marsh Drainage System, the Holland Marsh Lowlands and five (5) provincially significant wetland areas including the Pottageville Wetland Complex. The preferred route does not traverse any of these surface water features or wetland areas. However, some of these areas receive drainage from the watercourses that intersect the preferred route, and potential interactions should be considered. Some of the more notable features in the Study Area with respect to the watercourse crossings include Pottageville Creek, the South Canal, Kettleby Creek, Keele Creek and Glenville Creek (Figure 2).

2.4 REGIONAL GEOLOGY

Geological and hydrostratigraphic conditions throughout the general Study Area have been well documented in studies completed by Gartner Lee Ltd. (2004) and Kassenaar and Wexler (2006). In summary, the subsurface is reported to consist of the following key geological / hydrostratigraphic formations, with these units being listed from youngest to oldest:

Halton / Kettleby Till:	An aquitard unit consisting of sandy silt to clayey silt till (i.e., Halton Till) and/or silty clay to clay till (i.e., Kettleby Till). Typically encountered at ground surface, these glacial tills can extend to depths of up to 30 m below ground surface (m BGS). Throughout the immediate Study Area, glaciolacustrine deposits of sand, silt and clay commonly form a veneer over these till deposits. The horizontal hydraulic conductivity of this till unit is reported to be in the range 10^{-7} m/s;
Oak Ridges Moraine (ORM):	A stratified sediment complex consisting of multiple sequences of fining upward deposits from outwash sands to glaciolacustrine silts and clays, capped by kame deposits, meltwater channel deposits and ice-contact stratified drift. Regionally, these deposits are referred to as the Oak Ridges Aquifer Complex (ORAC), and wells completed in this aquifer system are typically characterized by high yields. The ORM is also considered to be an area of high groundwater recharge potential, with most of the headwaters for various watercourses in the Study Area originating as springs along the northern slope of this feature. The bulk horizontal hydraulic conductivity of the ORM sediments is reported to be in the range 10^{-4} m/s;
Newmarket Till:	An aquitard unit consisting of stony, dense silty sand to sandy silt till that represents a regional stratigraphic marker, separating the ORM from the Lower Sediments. This till unit is considered to act as an aquitard and typically ranges from 20 m to 30 m thick, although portions of the till have been eroded by subglacial meltwater events, which have resulted in the formation of tunnel channels varying from one to two kilometers in width and tens of metres deep. These channels are generally in-filled by sandy meltwater sediments that fine upwards. A hydraulic connection between the ORAC and the aquifer systems of the underlying

Lower Sediments exist where the tunnel channels cut through the Newmarket Till. North of the ORAC boundary this unit is often identified at ground surface as a stone-poor till surficial unit. The horizontal hydraulic conductivity of the Newmarket Till is reported to range from 10^{-8} m/s to 10^{-9} m/s;

Lower Sediments: A thick sediment complex consisting predominantly of silt, sand and gravel. The Lower Sediments are divided into three distinct units that include the Thorncliffe Formation (glaciofluvial deposits of sand and silty sand), followed by the Sunnybrook Drift (clast-poor, silt and clay) and the Scarborough Formation (fluvio-deltaic deposits of sand over silts and clays). Similar to the Newmarket Till, the Sunnybrook Drift is characterized by meltwater tunnel channels in-filled with sand and gravel, which allow for hydraulic connections to exist between the Thorncliffe and Scarborough Aquifer Complexes (TAC and SAC, respectively). The horizontal hydraulic conductivity of the TAC and SAC sediments range from 10^{-3} m/s to 10^{-6} m/s, whereas the conductivity of the Sunnybrook Drift (aquitard) ranges from 10^{-7} m/s to 10^{-8} m/s; and,

Bedrock: A relatively fair water-yielding aquifer system consisting of limestone of the Simcoe Group.

Figure 3 presents the surficial geology in the Study Area as mapped by the OGS (2003). In the majority of the Study Area, the dominant surficial units are a stone poor, silty sand to sandy silt textured till that is interpreted to correspond to the Halton Till, as well as fine textured glaciolacustrine deposits which are known to commonly form a veneer over these till deposits. As shown in Figure 3, coarse-textured glaciolacustrine deposits of sand and gravel are identified at ground surface near WC1, WC2, WC5, WC9 and WTC1 and are interpreted to represent historical Lake Simcoe shoreline sediments.

2.5 REGIONAL HYDROGEOLOGY

Recent modeling by Kassenaar and Wexler (2006) considered the ORAC as a single upper aquifer system, with the overlying aquitard (Halton/Kettleby Till) acting as a surfical layer that generally restricts flow to the underlying aquifers. The ORAC represents a regional groundwater high to the south of the Study Area where it is exposed at ground surface and higher recharge occurs, with localized groundwater flow within the ORAC and corresponding upper sediments moving in a north and northwest direction towards Lake Simcoe. However, groundwater flow in the ORAC in the vicinity of the Study Area appears to be strongly influenced by local surface water features. This groundwater system of the ORAC and upper sediments generally represent the aquifers of interest throughout the Study Area, given that all the construction activities will be occurring within the upper overburden.

In general, the Lower Sediments throughout the Study Area are considered to represent a single aquifer system due to the suspected occurrence of erosion channels within the Sunnybrook Drift, with the deposits of sand and gravel in-filling these channels allowing for potential hydraulic connections between the TAC and SAC (Gartner Lee Ltd., 2004). Based on potentiometric mapping (Stantec, 2008), the interpreted direction of horizontal groundwater flow through the Lower Sediments in the central part of the Study Area is to the northwest. This localized interpretation of groundwater flow is in general agreement with regional flow patterns presented by Gartner Lee Ltd. (2004) and Kassenaar and Wexler (2006), which indicate that the horizontal movement of groundwater through the TAC and SAC is north to northwest from the groundwater divide beneath the crest of the Oak Ridges Moraine towards Lake Simcoe.

3.0 Proposed Project Description

The proposed project was previously described by JWSL (2009), and is included herein. Enbridge is proposing to install an NPS 16 (16 inch diameter; 406 millimetre) steel pipeline to supply natural gas to the YEC. The pipeline is constructed of high grade steel with appropriate wall thickness to conform to Canadian Standards Association requirements (Z662 Oil and Gas Pipeline Systems), and designed in accordance with Ontario regulations.

3.1 PROPOSED PIPELINE ROUTE

The preferred route is located within the road allowances of the Lloydtown Aurora Road, Jane Street, Davis Drive and Dufferin Street. The approximate length of the pipeline along the preferred route is 16.7 km and the location is illustrated in Figures 2 and 3.

3.2 PIPLELINE CONSTRUCTION

The conventional method of pipeline construction is the open-cut method. Although the open cut technique can be used at some watercourse crossings by isolating the crossing site from the main watercourse, this method is associated with higher risks of adverse impacts to the environment. Other pipeline construction techniques (i.e., isolated watercourse crossing techniques) that minimize the potential environmental impacts are preferred at watercourse crossings, such as dam and pump, dam and flume (pipe), as well as trenchless methods such as horizontal bores, horizontal punches and directional drills, where there is no direct physical disturbance to the bed or banks of the watercourse and reduced risk of potential environmental impacts (with appropriate setbacks).

In work areas where there are minimal constraints, Enbridge will employ the open cut construction method described in Section 3.2.1. In areas of severe constraints (work room at a minimum), and at all road, permanent and intermittent natural watercourse crossings, and wetlands, horizontal directional drilling (HDD) techniques will be employed as described in Section 3.2.2. The open cut method may also be employed in areas where the preferred route crosses waterways used primarily for drainage (i.e., are not considered natural), and ephemeral watercourses.

3.2.1 Open Cut Construction Method

Open cut construction involves three main steps: workspace preparation, welding and trenching, and backfilling.

The road Right-of-Ways (RoW) and temporary workspaces are initially prepared by surface grading and tree removal, if necessary. Topsoil along the RoW is stripped and piled for replacement post-construction. Where agriculturally productive lands are encountered, topsoil and subsoil will be stripped and stockpiled separately to avoid topsoil and subsoil mixing. The centerline of the trench is staked, and the pipe is laid out in sections along the RoW.

3.2.1.1 Welding and Trenching

Pipe sections are bent and welded together and trenches are dug by backhoes or other excavation machines. Trench depth is determined by Canadian Standards Association (Z662 Oil and Gas Pipeline Systems) and company standards. The amount of open trench is kept to a minimum to ensure safety and minimize environmental impact. Large diameter pipelines are usually buried at a minimum depth of 1.0 metre (3.3 feet) from the top of the pipe to the ground surface. Before the line is lowered into the trench, pipeline welds are examined by a third-party company using radiography (X-rays).

3.2.1.2 Backfilling

Excavated material is replaced and large stones are removed from the backfill material to prevent pipeline damage. The construction area is carefully cleaned up after the trench is backfilled. All construction materials are removed and cultivated areas are restored to their original condition.

3.2.2 Horizontal Directional Drilling

HDD is based on equipment and techniques used in horizontal oil well drilling and conventional road boring and involves four main steps: pre-site planning; drilling a pilot hole; expanding the pilot hole by reaming; and pull back of pre-fabricated pipe.

3.2.2.1 Pre-Site Planning

Subsurface conditions and characteristics that would be encountered during the drilling program are assessed to determine if HDD is a technically and geotechnically feasible construction method at that location. Once confirmed, a drill pathway is designed for the specific crossing. This includes identification of drill entry and exit locations and depth along the route.

3.2.2.2 Drilling of the Pilot Hole

The pilot hole is drilled along the predetermined path, pushed by a drill rig and drill pipes strung together. The procession is guided by sensors at the drill bit or at the surface which indicate the drill bit’s relative horizontal and vertical position to the entry point. Pressurized drilling fluid is injected ahead of the drill bit for various reasons, including stabilizing the borehole, minimizing friction between the drill and bore wall and transporting drill cuttings to the surface. Drilling fluid returns are collected at the entry and/or exit points.

The potential exists for the inadvertent release of drilling fluid returns through the bed of the watercourse (i.e. frac-out). For the HDD crossings, a hazardous substance spills response plan will be prepared along with a drilling fluids management plan.

3.2.2.3 Reaming of the Pilot Hole

After breaking ground surface at the exit location, the drill bit is removed from the drill pipe string and replaced with a back reamer, which is of a larger diameter than the drill bit. The new assembly is pulled back through the borehole to enlarge the borehole.

3.2.2.4 Pipe String Pullback

Starting again at the exit location, the pipe sections are welded together into a string slightly longer than the drill length and pulled back through the borehole.

3.2.2.5 Tie-in Pits

Once the HDD pipeline has been installed, an excavation known as a tie-in pit will be required to allow for connection of the HDD pipeline to the pipeline on either side of the watercourse crossing. Each tie-in pit will be between 1.2 m to 2.5 m deep (Enbridge, 2009), and will be located 30 m from the edge of each watercourse or water feature in accordance with the ORMCP. Sufficient measures will be taken where necessary to ensure soil stability and safe operating conditions within each tie-in pit. In the event that the tie-in pit intersects the water table, dewatering will be required. An assessment was undertaken (Section 6.14) to estimate the amount of dewatering required for each watercourse crossing.

3.2.3 Pressure Testing (Hydrostatic Testing)

The new pipeline will be hydrostatically tested. The line is sealed then filled with water at pressure higher than actual operating pressures. Hydrostatic testing checks for leaks and confirms pipeline strength. Water for the test may be obtained from the local municipality or nearest watercourse that can support the withdrawal without environmental impact. Afterward, the water is released or disposed of according to Ontario Ministry of the Environment (MOE) and local conservation authority guidelines to ensure it does not cause erosion or sedimentation of watercourses. Section 6.2.2 describes the discharge measures to be employed. Hydrostatic testing should be undertaken according to procedures set forth in Enbridge's Construction Manual (Enbridge, 2009).

3.2.4 Clean up

All construction materials are removed, and a final grading of the area is completed. Anything disturbed by the construction (such as fences and pavement) is repaired or replaced. Lastly, cultivated areas are restored to their original condition.

3.2.5 Ongoing Monitoring

Slope erosion and re-establishment of vegetation are carefully monitored following roadside construction. Enbridge will conduct the remedial work necessary to address such issues following pipeline construction.

3.2.6 Environmental Considerations

The route of the pipeline is expected to stay within existing road allowances that have been previously disturbed during initial road construction and are periodically disturbed due to road maintenance activities. No new impacts to private or previously undisturbed land are expected to occur. Potential environmental impacts that may be encountered during the construction period are presented in Section 6.0. However, appropriate mitigation measures will be used during construction to ensure that the natural environment is protected.

4.0 Study Methodology

In this Study, a total of nine watercourse crossings (WC1 to WC9) and one wetland crossing (WTC1) were evaluated. The methodology at each crossing typically involved:

- Drilling of two (2) geotechnical boreholes;
- Installation of one (1) monitoring well;
- Installation of one (1) drive-point piezometer;
- Groundwater and surface water level measurements;
- Completion of a geodetic survey; and
- Single well hydraulic response testing.

4.1 BOREHOLE DRILLING

A total of 19 boreholes were drilled as part of the geotechnical study completed by Stantec (Stantec, 2009). Two (2) boreholes were drilled at each watercourse crossing (WC1 to WC9), and only one borehole was drilled at the wetland crossing (WTC1). The purpose of the borehole drilling was to identify the local geology surrounding each watercourse/wetland crossing. The locations of all boreholes are shown on Figure 2.

4.1.1 Drilling – Monitoring Wells

All boreholes drilled and completed as monitoring wells, with the exception of BH9 (MW), were completed by Walker Drilling Ltd. between November 9 and November 13, 2009 under the observation of Stantec personnel. The drilling was completed using a Dietrich D-50 Turbo Trackmount drill-rig equipped with 8.5 in (216 mm) hollow stem augers. Split spoon samples were collected every 0.8 m (2.5 ft) to a depth of 3.7 m BGS (12 ft), after which a 0.6 m (2 ft) long split spoon sample was obtained every 1.5 m (5 ft). The exception was at BH1 (MW), where split spoon samples were collected along the entire depth of the borehole.

Borehole BH9 (MW) was completed by Ponthil Drilling on November 10, 2009 under the observation of Stantec personnel. The drilling was completed using a CME 75 drill-rig equipped with 4.5 in (114 mm) solid stem augers. Split spoon samples were collected every 0.8 m (2.5 ft) to a depth of 3.7 m BGS (12 ft), after which a 0.6 m (2 ft) long split spoon sample was obtained every 1.5 m (5 ft). The interpretive logs for all the monitoring wells (BH1, BH2, BH3, BH4A, BH5A, BH6A, BH7, BH8A, BH9 and BH10) are provided in Appendix C.

4.1.2 Drilling – Boreholes

A total of nine boreholes were completed to identify subsurface conditions, one at each watercourse crossing location (WC1 to WC9). No borehole was completed at the wetland crossing (WTC1). The total depth of the boreholes ranged from 4.4 m BGS to 9.0 m BGS. Encountered stratigraphy and borehole completion details for boreholes BH1A, BH2A, BH3A, BH4, BH5C, BH6C, BH7B, BH8 and BH9A are presented on the borehole logs in Appendix C. The borehole locations are shown on Figure 2.

Borehole drilling was completed by Ponthil Drilling on November 9 to 11 and 13 (2009) and by Atcost Drilling Inc. on November 30 (2009), under the observation of Stantec personnel. The drilling was completed using a CME 75 drill-rig equipped with 4.5 in (114 mm) solid stem augers. Split spoon samples were collected every 0.8 m (2.5 ft) to a depth of 3.7 m BGS (12 ft), after which a 0.6 m (2 ft) long split spoon sample was obtained every 1.5 m (5 ft). The exception was at BH1A, BH4 and BH6C, where split spoon samples were collected along the entire depth of the borehole. The interpretive logs (Appendix C) illustrate the depths at which samples were obtained and the details related to backfilling material for each hole. All the boreholes were backfilled to ground surface with either bentonite (BH1A, BH5C, BH6C and BH7B), auger cuttings (BH3A and BH9A), or a combination of bentonite and auger cuttings (BH2A, BH4, and BH8).

4.2 INSTALLATION OF MONITORING WELLS

A total of 10 monitoring wells were installed, with one monitoring well being located near each watercourse/wetland crossing location. The total depth of the monitoring wells ranged from 4.6 m BGS to 7.6 m BGS. Monitoring well installation details for monitoring wells BH1, BH2, BH3, BH4A, BH5A, BH6A, BH7, BH8A, BH9 and BH10 are presented on the borehole logs in Appendix C and summarized in Table 1. The monitoring well locations are shown on Figures 2, and 4 to 13.

All monitoring wells were constructed using 50 mm ID Schedule 40 Polyvinyl chloride (PVC) well casing. The wells were constructed with 3 m long, No. 10 slot (0.01 inch slot) PVC well screens. The annular space surrounding the screens was backfilled with silica sand, with the remainder of the annular space filled with bentonite to approximately 0.3 m BGS to prevent hydraulic connection within the borehole.

The monitoring wells were completed with either above ground lockable protective steel casings that were cemented into place (BH5A, BH7, BH9, and BH10) or flushmount construction cemented into place (BH1, BH2, BH3, BH4A, BH6A, and BH8A). All monitoring wells constructed as part of this investigation were completed in accordance with Ontario Regulation 903 (O.Reg.903) and marked with individual well tags.

4.3 INSTALLATION OF DRIVE-POINT PIEZOMETERS

Ten (10) drive-point piezometers (DP1-09 to DP10-09) were installed on November 19, 2009, one at each of the identified watercourse/wetland crossing locations. These drive-point piezometers were installed to provide information on groundwater and surface water interaction in the vicinity of each watercourse/wetland crossing. The locations of the drive-points are shown on Figures 2 and 4 to 13.

The drive-point piezometers consisted of a 19 mm diameter, 0.43 m long steel drive-point piezometer screen connected to 25 mm diameter, 2.1 m long galvanized steel risers, with the exception of DP7-09, which had a riser 1.2 m long. The drive-point piezometers were installed by hand driven methods to depths ranging from 0.9 m BGS to 1.7 m BGS, and the installation details are summarized in Table 1. The drive-points were equipped with a vented, threaded steel cap and were purged immediately following installation. All logs illustrating the installation details for each drive-point piezometer are presented in Appendix C.

4.4 GROUNDWATER AND SURFACE WATER MONITORING

Water level measurements were obtained at monitoring wells and drive-point piezometers on December 7, 10 and 15, 2009. These water levels were manually measured using a Solinst Water Level Meter over a period of approximately two (2) weeks to identify vertical hydraulic gradients between surface water and groundwater in the vicinity of each watercourse/wetland crossing. Tables 2 and 3 summarize the results of the groundwater and surface water level monitoring.

4.5 GEODETIC SURVEY

On December 8, 2009, a geodetic survey was completed for the installed monitoring wells, boreholes and drive-points by Stantec Geomatics. This survey included determining the top of casing (TOC) elevations for the monitoring wells and drive-points, and ground surface elevations for the boreholes and flushmount monitoring wells using a nearby benchmark. A total station survey system was employed to accurately define elevations and UTM coordinates. All elevations and coordinates obtained during the survey are provided in Table 1.

4.6 SINGLE WELL RESPONSE TESTS

To estimate the hydraulic conductivity of the screened intervals, *in-situ* hydraulic response testing at monitoring wells BH2, BH3, BH5A, and BH8A were performed. The hydraulic response testing consisted of a rising head slug test by creating a near-instantaneous change in the water level in the well by removing a slug of known volume. The volumes of water removed from each well were as follows: 30 L from BH2 (MW); 8 L at BH3 (MW); 18 L at BH5A (MW); and 15 L at BH8A (MW). The change in water level over time was monitored to determine the rate of well recovery. Well recovery was continuously monitored using Solinst Leveloggers®

temporarily installed within each well and set to record at the following intervals; 5 s for 10 min; 10 s for 10 min; 30 s for 1 hr; and 1 min for 2 days. Manual water levels were taken at the start and end of each test prior to installing and removing the Solinst Leveloggers® to allow for data correction. The results of the response testing were analyzed with the Aqtesolv™ software package using the Bouwer and Rice solution (1976) to determine the horizontal hydraulic conductivity of the formation within the immediate vicinity of the monitoring well screen. The results of the hydraulic response testing are summarized in Section 5.0 and Table 1, with the analytical solutions presented in Appendix D.

5.0 Results and Discussion

The results of the investigation are presented below for each watercourse/wetland crossing. The subsurface geology and static water levels were determined at each crossing location, with the horizontal hydraulic conductivity of the screened material being calculated at four (4) of the 10 monitoring wells. Appendix C presents the borehole logs, which provide a description of the overburden material and static water level measurements. Figures 4 to 13 present the locations of cross-sections A-A' to J-J', with the stratigraphic interpretations at each location being provided in Figures 14 to 16. Table 1 and Appendix D provide the rising head test analysis used to determine the hydraulic conductivity of the materials for the select monitoring wells.

5.1 WATERCOURSE CROSSING 1 (WC1)

5.1.1 Site Features

Watercourse Crossing 1 (WC1) is the westernmost watercourse crossing along the preferred route, located in the community of Pottageville approximately 1.1 km east of the pipeline starting point at Schomberg Gate Station. The nearest road intersection to the water crossing is 60 m east at Lloydtown Aurora Road and Concession Road 7. WC1 is a northward flowing tributary to Pottageville Creek which discharges into the South Canal. WC1 also contributes to the downstream provincially significant Pottageville Wetland Complex. The headwaters originate in sparsely populated Happy Valley Forest, approximately 3 km southeast of the stream crossing. Figure 4 illustrates the location of the water crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 14 provides a cross section at WC1. The Ministry of Natural Resources (MNR) classifies this reach, as well as the majority of upstream reaches feeding into Pottageville Creek, as intermittent.

5.1.2 Local Geology

When drilling at BH1A, asphalt and sand and gravel fill were observed from ground surface to 0.5 m BGS, changing to a silty sand fill from 0.5 m BGS to 1.4 m BGS. Below this depth, a loose, organic silty sand unit was encountered from 1.4 m BGS to 2.6 m BGS. From 2.6 m BGS to 3.5 m BGS a sand and gravel unit was identified and was determined to be compact to dense based on blow counts collected during the advancement of the borehole. Below this unit, a stiff to very stiff clayey silt unit was encountered from 3.5 m BGS to the bottom of the borehole at 6.7 m BGS.

Similar stratigraphy was encountered at the nearby monitoring well (BH1 (MW)), with asphalt and a fill material present from ground surface to 1.4 m BGS (236.3 m above mean seal level (AMSL) to 234.9 m AMSL), at which depth a loose silty sand unit was identified. The silty sand unit was found from 1.4 m BGS to 2.6 m BGS (234.9 m AMSL to 233.7 m AMSL). Below this unit, a sand and gravel material with trace silt and clay was encountered, which was described as dense based on blow counts collected during the advancement of the borehole. At the base of the borehole, from 5.5 m BGS to 8.2 m BGS (230.8 m AMSL to 228.0 m AMSL), a very stiff, clayey silt unit was identified. The monitoring well is screened across both the sand and gravel and clayey silt till units. Based on the elevation of the encountered stratigraphy, it is likely that these units correspond to the Halton/Kettleby Till.

The local geology, groundwater and surface water elevations, along with the approximate location of the pipeline, are illustrated on Cross Section A-A' (Figure 14). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the sand and gravel fill and underlying silty sand and sand and gravel units being intersected by the pipeline extending below the watercourse. The possibility exists that the clayey silt unit identified at BH1 (MW) may also be encountered during the HDD.

5.1.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH1 (MW) were 1.42 m BGS to 1.41 m BGS (234.84 m AMSL to 234.85 m AMSL) between December 7 and 15, 2009 (Table 2). WC1 is situated at an elevation of 234.8 m AMSL and consequently the water table will be encountered when drilling below WC1.

Surface water and groundwater level measurements taken at DP1-09 between December 7 and 15, 2009 found an upward vertical hydraulic gradient (Table 3), indicating that in the vicinity of WC1 groundwater is discharging to this watercourse. This supports an observation made in JWSL (2009), where a groundwater seep was noted in the vicinity of WC1.

There are a total of eight (8) private domestic water supply wells located in the vicinity of WC1 (Figure 4), ranging from 40 m to 120 m away from WC1. The wells ranged in depth from 17 m BGS to 55 m BGS, and based on MOE records, are all completed in overburden material.

5.2 WATERCOURSE CROSSING 2 (WC2)

5.2.1 Site Features

Watercourse Crossing 2 (WC2) is located in the community of Pottageville, approximately 375 m east of the road intersection of Lloydtown Aurora Road and Concession Road 7. WC2 is a northward flowing tributary to Pottageville Creek which discharges into the South Canal. WC2 also contributes to the downstream provincially significant Pottageville Wetland Complex. MNR classifies this reach as intermittent. Figure 5 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 14 provides a cross section at WC2.

5.2.2 Local Geology

When drilling at BH2 (MW), a compact, sand and gravel fill was observed from ground surface to 0.6 m BGS (237.2 m AMSL to 236.6 m AMSL), changing to a silty sand fill from 0.6 m BGS to 1.4 m BGS (236.6 m AMSL to 235.8 m AMSL). Below this depth, a loose, organic silty sand unit was encountered from 1.4 m BGS to 2.5 m BGS (235.8 m AMSL to 234.7 m AMSL). From 2.5 m BGS (234.7 m AMSL) to the bottom of the borehole at 6.7 m BGS (230.5 m AMSL) a firm to stiff clayey silt was encountered. The monitoring well is screened primarily within this lower unit.

Similar stratigraphy was encountered at the nearby borehole (BH2A), with a loose silty sand fill material present from ground surface to 1.3 m BGS (237.7 m AMSL to 236.4 m AMSL), at which depth a firm, silty clay with trace gravel was identified. The silty clay unit was found from 1.3 m BGS to 1.9 m BGS (236.4 m AMSL to 235.8 m AMSL). Below this unit, a clayey silt material with trace sand was encountered, which was described as soft to stiff based on blow counts collected during the advancement of the borehole. The clayey silt was encountered from 1.9 m BGS (235.8 m AMSL) to the bottom of the borehole at 5.2 m BGS (232.5 m AMSL).

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section B-B' (Figure 14). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in all identified units (fill, silty sand, silty clay and clayey silt) being intersected by the pipeline extending below the watercourse.

5.2.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH2 (MW) ranged from 1.68 m BGS to 1.57 m BGS (235.49 m AMSL to 235.60 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). WC2 is situated at an elevation of 235.8 m AMSL and as a result the water table will be encountered when drilling below WC2.

Surface water and groundwater level measurements taken at DP2-09 between December 7, 2009 and December 15, 2009 found an upward vertical hydraulic gradient (Table 3). This supports an observation made in JWSL (2009) which noted the area of WC2 as an apparent groundwater discharge zone. JWSL (2009) also stated the channel armouring has reduced the functions of the stream corridor by limiting the biotic and hydrologic interaction between the stream and the riparian zone.

The horizontal hydraulic conductivity of the clayey silt layer in which BH2 (MW) was screened was determined to be 1.8×10^{-7} m/s (Table 1; Appendix D) through the completion of a rising head single well response test. This is relatively consistent for materials of this type.

There are a total of seven (7) private domestic water supply wells located in the vicinity of WC2 (Figure 5), ranging from 50 m to 160 m away from WC2. The wells ranged in depth from 31 m BGS to 41 m BGS, and based on MOE records, are all completed in deep overburden material with the exception of MOE Well 6924444, which is completed in bedrock.

5.3 WATERCOURSE CROSSING 3 (WC3)

5.3.1 Site Features

Watercourse Crossing 3 (WC3) is located approximately 375 m east of the road intersection of Lloydtown Aurora Road and Cook Drive. WC3 is a northward flowing tributary to Pottageville Creek which discharges into the South Canal. WC3 also contributes to the downstream provincially significant Pottageville Wetland Complex. The MNR classifies all reaches upstream of the watercourse crossing as intermittent. Figure 6 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 14 provides a cross section at WC3.

5.3.2 Local Geology

When drilling at BH3 (MW), a compact, sand and gravel fill was observed from ground surface to 0.6 m BGS (246.6 m AMSL to 246.0 m AMSL), changing to a loose, silty sand fill from 0.6 m BGS to 3.5 m BGS (246.0 m AMSL to 243.1 m AMSL). Below this depth, a compact, silty sand unit was encountered from 3.5 m BGS (243.1 m AMSL) to the bottom of the borehole at 8.2 m BGS (238.4 m AMSL). The monitoring well is screened within the silty sand unit.

At the nearby borehole (BH3A), asphalt was found at ground surface, underlain by a loose sand and gravel fill material to 0.6 m BGS (246.3 m AMSL). The fill material then became a loose to compact silty sand with occasional topsoil inclusions from 0.6 m BGS to 3.35 m BGS (246.3 m AMSL to 243.6 m AMSL). Underlying the silt unit was a grey, compact silty sand, which was identified from 3.35 m BGS (243.6 m AMSL) to the base of the borehole at 5.2 m BGS (241.7 m AMSL).

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section C-C' (Figure 14). Based on Enbridge construction requirements (Enbridge, 2009) and site conditions, HDD will result in all identified units (fill, silty sand fill, and silty sand) being intersected by the pipeline extending below the watercourse.

5.3.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH3 (MW) were 1.04 m BGS to 0.94 m BGS (245.55 m AMSL to 245.65 m AMSL) between December 7, 2009 and

December 15, 2009 (Table 2). WC3 is situated at an elevation of 244.3 m AMSL and as a result the water table will be encountered when drilling below WC3.

Surface water and groundwater level measurements taken at DP3-09 between December 7, 2009 and December 15, 2009 found an upward vertical hydraulic gradient, indicating that in the vicinity of WC3 groundwater is discharging to the watercourse (Table 3). This supports an observation made in JWSL (2009), which noted groundwater discharge conditions in the vicinity of WC3.

The horizontal hydraulic conductivity of the silty sand layer in which BH3 (MW) was screened was determined to be 3.1×10^{-8} m/s (Table 1; Appendix D) through the completion of a rising head single well response test. This is considered a somewhat low hydraulic conductivity for silty sand, and indicates that the silt content may be higher than indicated during field observations.

There are a total of two (2) private domestic water supply wells located in the vicinity of WC3 (Figure 6), ranging from 70 m to 180 m away from WC3. The wells ranged in depth from 9 m BGS to 69 m BGS, and based on MOE records, are all completed in overburden material.

5.4 WATERCOURSE CROSSING 4 (WC4)

5.4.1 Site Features

Watercourse Crossing 4 (WC4) is located approximately 300 m north of the road intersection of Lloydtown Aurora Road and Jane Street. WC4 is an eastern flowing tributary to Kettleby Creek which discharges into the South Canal. MNR classifies this tributary as intermittent. Figure 7 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 14 provides a cross section at WC4.

5.4.2 Local Geology

When drilling at BH4A (MW), a sand and gravel fill was observed from ground surface (285.9 m AMSL) to 0.3 m BGS (285.6 m AMSL), changing to a silty sand fill from 0.3 m BGS to 2.1 m BGS (285.6 m AMSL to 283.7 m AMSL). Below this depth, a loose, grey, organic silt unit containing shell fragments was encountered from 2.1 m BGS to 3.6 m BGS (283.7 m AMSL to 282.2 m AMSL). Below this unit, grey, clayey silt was encountered from 3.6 m BGS to the bottom of the borehole at 9.7 m BGS (282.2 m AMSL to 276.1 m AMSL), and is the unit in which the monitoring well is screened. The clayey silt was determined to be very hard based on blow counts collected during the advancement of the borehole.

At the nearby borehole (BH4), a sand and gravel fill material was present from ground surface to 0.3 m BGS (286.1 m AMSL to 285.7 m AMSL), at which depth a loose silty sand fill was identified. The silty sand fill was found from 0.3 m BGS to 2.3 m BGS (285.7 m AMSL to

283.8 m AMSL). Below the fill, a very loose organic silty sand material was encountered to a depth of 4.0 m BGS (282.1 m AMSL). At the base of the borehole, from 4.0 m BGS to 5.2 m BGS (282.1 m AMSL to 280.9 m AMSL), a grey, clayey silt unit was identified and was described as firm to very stiff based on blow counts collected during the advancement of the borehole.

The local geology, groundwater and surface water elevations, along with the approximate location of the pipeline, are illustrated on Cross Section D-D' (Figure 14). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in all identified units (sand and gravel fill, silty sand fill, silty sand and clayey silt) being intersected by the pipeline extending below the watercourse.

5.4.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH4A (MW) were 2.11 m BGS to 2.08 m BGS (283.76 m AMSL to 283.79 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). Drilling at the borehole at WC4 (BH4) identified wet sediment in the overlying sand layer and therefore the water level measurements observed at the monitoring well are interpreted to be reflective of the lower clayey silt unit in which the well is screened. Therefore, the water table surface will be encountered when drilling below WC4.

Surface water and groundwater level measurements taken at DP4-09 between December 7, 2009 and December 15, 2009 found a downward vertical hydraulic gradient, indicating that in the vicinity of WC4 surface water is recharging the shallow groundwater system (Table 3).

There are a total of two (2) private domestic water supply wells located in the vicinity of WC4 (Figure 7), ranging from 150 m to 160 m away from WC4. The wells ranged in depth from 18 m BGS to 98 m BGS, and based on MOE records, are all completed in overburden material.

5.5 WATERCOURSE CROSSING 5 (WC5)

5.5.1 Site Features

Watercourse Crossing 5 (WC5) is located approximately 690 m east of the road intersection of Davis Drive and Jane Street. WC5 is known as Kettleby Creek, which flows north and discharges into the South Canal. MNR classifies this tributary as permanent. Figure 8 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 15 provides a cross section at WC5.

5.5.2 Local Geology

When drilling at BH5A (MW), topsoil was encountered from ground surface to 0.1 m BGS (225.0 m AMSL to 224.9 m AMSL) at which depth silty sand fill with organic inclusions was identified from 0.1 m BGS to 1.5 m BGS (224.9 m AMSL to 223.5 m AMSL). Below the fill material, grey, clayey silt was encountered from 1.5 m BGS to the bottom of the borehole at 8.2 m BGS (223.5 m AMSL to 216.8 m AMSL), and is the material in which the monitoring well is screened. The clayey silt was determined to be firm to hard based on blow counts collected during the advancement of the borehole.

At the nearby borehole (BH5C), asphalt and sand and gravel fill material were present from ground surface to 0.7 m BGS (228.1 m AMSL to 227.4 m AMSL), at which depth a compact silt fill with sand and gravel was identified. The silt fill was found from 0.7 m BGS to 4.6 m BGS (227.4 m AMSL to 223.5 m AMSL), and was underlain by compact sand and gravel fill from 4.6 m BGS to 7.6 m BGS (223.5 m AMSL to 220.5 m AMSL). At the base of the borehole, from 7.6 m BGS to 9.0 m BGS (220.5 m AMSL to 219.1 m AMSL), a grey, silty clay unit was identified and was described as compact.

The local geology, groundwater and surface water elevations, along with the approximate location of the pipeline, are illustrated on Cross Section E-E' (Figure 15). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the clayey silt unit and all overlying material being intersected by the pipeline extending below the watercourse.

5.5.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH5A (MW) were 1.12 m BGS to 1.08 m BGS (223.87 m AMSL to 223.91 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). WC5 is situated at an elevation of 223.3 m AMSL and as a result the water table will be encountered when drilling below WC5.

Surface water and groundwater level measurements taken at DP5-09 between December 7, 2009 and December 15, 2009 found a downward vertical hydraulic gradient, indicating that in the vicinity of WC5 surface water is recharging the shallow groundwater system (Table 3).

The horizontal hydraulic conductivity of the clayey silt layer in which BH5A (MW) was screened was determined to be 1.6 x 10^{-6} m/s (Table 1; Appendix D) through the completion of a rising head single well response test. This is relatively high for clayey silt, indicating that the material in which the well is screened is actually coarser than indicated from field observations, or has seams containing materials of a higher conductivity.

No existing wells were identified in the vicinity of WC5 (Figure 8).

5.6 WATERCOURSE CROSSING 6 (WC6)

5.6.1 Site Features

Watercourse Crossing 6 (WC6) is located approximately 325 m west of the road intersection of Davis Drive and Keele Street. The watercourse is a northern flowing tributary to Keele Creek which discharges north into the South Canal. MNR classifies this tributary as permanent. Figure 9 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 15 provides a cross section at WC6.

5.6.2 Local Geology

When drilling at BH6A (MW), asphalt and a sand and gravel fill were observed from ground surface (233.6 m AMSL) to 0.4 m BGS (233.2 m AMSL), changing to silty sand fill from 0.4 m BGs to 2.3 m BGS (233.2 m AMSL to 231.3 m AMSL). Below this depth, a loose, brown, organic silt unit was encountered from 2.3 m BGS to 3.6 m BGS (231.3 m AMSL to 229.9 m AMSL), followed by a brown to grey silty sand till to 6.1 m BGS (227.5 m AMSL). Below this unit, a grey, clayey silt was encountered from 6.1 m BGS (227.5 m AMSL) to the bottom of the borehole at 8.2 m BGS (225.4 m AMSL). The clayey silt was determined to be stiff to very stiff based on blow counts collected during the advancement of the borehole. The monitoring well is screened across both the clay silt and silty sand units identified at the base of the borehole.

At the nearby borehole (BH6C), a sand and gravel fill material was present from ground surface to approximately 1.6 m BGS (234.0 m AMSL to 232.4 m AMSL), at which depth a loose to compact clayey silt unit containing sand and gravel was identified. The clayey silt was found from 1.6 m BGS to 3.8 m BGS (232.4 m AMSL to 230.2 m AMSL). At the base of the borehole, from 3.8 m BGS to 4.4 m BGS (230.2 m AMSL to 229.6 m AMSL), a brown sand and gravel unit was identified and was described as dense based on blow counts collected during the advancement of the borehole.

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section F-F’ (Figure 15). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the clayey silt unit and all overlying material (silt and fill material) being intersected by the pipeline extending below the watercourse.

5.6.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH6A (MW) were 3.22 m BGS to 3.20 m BGS (230.37 m AMSL to 230.39 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). Wet sediment within the top 0.5 m BGS was identified at the borehole at WC6 (BH6C). The water table surface will be encountered when drilling below WC6.

Surface water and groundwater level measurements taken at DP6-09 between December 7, 2009 and December 15, 2009 found an upward vertical hydraulic gradient, indicating that in the vicinity of WC6 groundwater is discharging to the stream (Table 3).

No existing wells were identified in the vicinity of WC6 (Figure 9).

5.7 WATERCOURSE CROSSING 7 (WC7)

5.7.1 Site Features

Watercourse Crossing 7 (WC7) is located approximately 350 m east of the road intersection of Davis Drive and Keele Street. The watercourse is a northern flowing tributary to Keele Creek which discharges north into the South Canal. MNR classifies this tributary as permanent. Figure 10 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 15 provides a cross section at WC7.

5.7.2 Local Geology

When drilling at BH7 (MW), topsoil was observed from ground surface to 0.3 m BGS (248.8 m AMSL to 248.5 m AMSL). Below the topsoil, a very loose to compact, silt unit with clay and trace sand was encountered from 0.3 m BGS (248.5 m AMSL) to the bottom of the borehole at 8.2 m BGS (240.6 m AMSL). The monitoring well is screened within the silt stratigraphic unit.

At the nearby borehole (BH7B), asphalt was found at ground surface (251.8 m AMSL), underlain by a sand and gravel fill material to 0.4 m BGS (251.3 m AMSL). The fill material then became a sand fill to 1.2 m BGS (250.5 m AMSL), and a grey, silty sand fill from 1.2 m BGS to 4.9 m BGS (250.5 m AMSL to 246.9 m AMSL). Underlying the fill was black silty sand from 4.9 m BGS to 6.2 m BGS (246.9 m AMSL to 245.5 m AMSL), followed by a loose sandy silt from 6.2 m BGS (245.5 m AMSL) to the base of the borehole at 6.7 m BGS (245.1 m AMSL).

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section G-G' (Figure 15). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the silt material and overlying topsoil being intersected by the pipeline extending below the watercourse.

5.7.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH7B (MW) were 0.48 m BGS to 0.24 m BGS (248.35 m AMSL to 248.58 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). The water table surface will be encountered when drilling below WC7.

Surface water and groundwater level measurements taken at DP7-09 between December 7, 2009 and December 15, 2009 found a downward vertical hydraulic gradient, indicating that in the vicinity of WC7 surface water is recharging the local groundwater system (Table 3).

No existing wells were identified in the vicinity of WC7 (Figure 10).

5.8 WATERCOURSE CROSSING 8 (WC8)

5.8.1 Site Features

Watercourse Crossing 8 (WC8) is located approximately 1,140 m north of the road intersection of Dufferin Street and Davis Drive. The watercourse is a westward flowing tributary to Glenville Creek which discharges west into the South Canal. MNR classifies this tributary as permanent. Figure 11 illustrates the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 15 provides a cross section at WC8.

5.8.2 Local Geology

When drilling at BH8A (MW), asphalt and a sand and gravel fill were observed from ground surface (239.4 m AMSL) to 0.4 m BGS (239.0 m AMSL), changing to silty sand fill from 0.4 m BGs to 2.7 m BGS (239.0 m AMSL to 236.6 m AMSL). Below this depth, a dark brown, organic silt unit was encountered from 2.7 m BGS to 3.5 m BGS (236.6 m AMSL to 235.8 m AMSL), followed by grey, compact sand and gravel to 6.1 m BGS (233.3 m AMSL). Below this unit, grey, silty sand with trace gravel was encountered from 6.1 m BGS (233.3 m AMSL) to the bottom of the borehole at 6.7 m BGS (232.6 m AMSL). The silty sand was determined to be dense based on blow counts collected during the advancement of the borehole. The well is screened primarily in the sand and gravel unit encountered near the base of the borehole.

At the nearby borehole (BH8), a sand and gravel fill material was present from ground surface to 0.3 m BGS (239.6 m AMSL to 239.3 m AMSL), at which depth the fill changed to compact silty sand. The silty sand fill was present from 0.3 m BGS to 2.3 m BGS (239.3 m AMSL to 237.3 m AMSL), followed by a loose to compact organic silt layer from 2.3 m BGS to 3.4 m BGS (237.3 m AMSL to 236.2 m AMSL). At the base of the borehole, from 3.4 m BGS to 5.2 m BGS

(236.2 m AMSL to 234.4 m AMSL), a grey sand and gravel unit was identified and was described as compact based on blow counts collected during the advancement of the borehole.

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section H-H' (Figure 15). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the sand and gravel unit and all overlying material (silt, silty sand and sand and gravel fill) being intersected by the pipeline extending below the watercourse.

5.8.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH8A (MW) were 2.48 m BGS to 2.43 m BGS (236.88 m AMSL to 236.92 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). WC8 is situated at an elevation of 236.9 m AMSL and as a result the water table will be encountered when drilling below WC8.

Surface water and groundwater level measurements taken at DP8-09 between December 7, 2009 and December 15, 2009 found an upward vertical hydraulic gradient, indicating that in the vicinity of WC8 groundwater is recharging the stream and there is interaction between the groundwater and surface water systems (Table 3).

The horizontal hydraulic conductivity of the sand and gravel layer in which BH8A (MW) was screened was determined to be 7.3×10^{-7} m/s (Table 1; Appendix D) through the completion of a rising head single well response test. This is low for a sand and gravel material, indicating the likely presence of silt within this soil matrix.

There is one (1) private domestic water supply well located in the vicinity of WC8 (Figure 11), located approximately 110 m to the northeast. Based on MOE records, the well extends to a depth of 30 m BGS and is completed in overburden material.

5.9 WATERCOURSE CROSSING 9 (WC9)

5.9.1 Site Features

Watercourse Crossing 9 (WC9) is located approximately 3.3 km north of the road intersection of Dufferin Street and Davis Drive. The watercourse is a northward flowing unnamed tributary, and discharges to the South Canal approximately 180 m north of the crossing. MNR classifies this tributary as permanent. Figure 12 illustrate the location of the watercourse crossing in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 16 provides a cross section at WC9.

5.9.2 Local Geology

When drilling at BH9 (MW), sand and gravel fill was observed from ground surface (220.1 m AMSL) to 0.4 m BGS (219.7 m AMSL). Below this depth, a dark brown to grey organic silt unit was encountered from 0.4 m BGS to 4.6 m BGS (219.7 m AMSL to 215.5 m AMSL). Below this unit, grey, clayey silt with trace sand was encountered from 4.6 m BGS (215.5 m AMSL) to the bottom of the borehole at 6.7 m BGS (213.4 m AMSL). The clayey silt was determined to be very soft to firm based on blow counts collected during the advancement of the borehole. The well was screened across both the clayey silt and silt stratigraphic units.

At the nearby borehole (BH9A), sand and gravel fill material was present from ground surface to 0.5 m BGS (220.5 m AMSL to 220.0 m AMSL), at which depth the fill changed to brown silty sand with gravel. The silty sand fill was present from 0.5 m BGS to 2.1 m BGS (220.0 m AMSL to 218.3 m AMSL), followed by a very loose organic silt layer from 2.1 m BGS to the bottom of the borehole at 5.2 m BGS (218.3 m AMSL to 215.3 m AMSL).

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section I-I' (Figure 16). Based on site conditions and Enbridge construction requirements (Enbridge, 2009), HDD will result in the silt unit and all overlying fill material being intersected by the pipeline extending below the watercourse.

5.9.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH9 (MW) were 0.95 m BGS to 0.74 m BGS (219.17 m AMSL to 219.38 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). WC9 is situated at an elevation of 218.9 m AMSL and therefore the water table will be encountered when drilling below WC9.

Surface water and groundwater level measurements taken at DP9-09 between December 7, 2009 and December 15, 2009 found a downward vertical hydraulic gradient, indicating that in the vicinity of WC9 groundwater is discharging from the stream (Table 3).

There are a total of four (4) private domestic water supply wells located in the vicinity of WC9 (Figure 12), ranging from 80 m to 110 m away from WC9. The wells ranged in depth from 18 m BGS to 51 m BGS, and based on MOE records, are all completed in overburden material.

5.10 WETLAND CROSSING 1 (WTC1)

5.10.1 Site Features

Wetland Crossing 1 (WTC1) is located on Dufferin Street approximately 1.2 km north of the road intersection of Dufferin Street and Highway 9. The wetland is a provincially significant wetland located east of Dufferin Street. Figure 13 illustrates the location of the wetland in relation to the associated drive-point piezometer, monitoring well and borehole, and Figure 16 provides a cross section at WTC1.

5.10.2 Local Geology

When drilling at BH10 (MW), silty sand topsoil was observed from ground surface (222.5 m AMSL) to 0.7 m BGS (221.8 m AMSL). Below this surficial unit, grey, sand and silt with trace clay was encountered from 0.7 m BGS (221.8 m AMSL) to the bottom of the borehole at 5.2 m BGS (217.3 m AMSL). The sand and silt was determined to be loose to compact based on blow counts collected during the advancement of the borehole. The well was screened within this sand and silt unit. No borehole was drilled at WTC1.

The local geology, groundwater and surface water elevations, along with the approximate location of the proposed pipeline, are illustrated on Cross Section J-J' (Figure 16). Based on the observed site conditions and Enbridge construction requirements (Enbridge, 2009), it is believed that the wetland feature will be intersected by the proposed pipeline. As a result, HDD will likely be required at this location. HDD will result in the sand and silt unit and all overlying topsoil being intersected by the pipeline extending below the watercourse.

5.10.3 Local Hydrogeology

The static groundwater level measurements within monitoring well BH10 (MW) were 0.38 m BGS to 0.35 m BGS (222.16 m AMSL to 222.18 m AMSL) between December 7, 2009 and December 15, 2009 (Table 2). WTC1 is situated at an elevation of 222.40 m AMSL and therefore the water table will be encountered when drilling at WTC1.

Surface water and groundwater level measurements taken at DP10-09 between December 7, 2009 and December 15, 2009 found a downward vertical hydraulic gradient, indicating that in the vicinity of WTC1 the wetland is recharging the groundwater system (Table 3).

No existing wells were identified in the vicinity of WTC1 (Figure 13).

6.0 Potential Impacts

Based on the above hydrogeologic conditions present at each watercourse crossing location, in combination with the intended drilling practices, the following potential impacts to the environment have been identified:

6.1 FRAC-OUT

With the use of high pressure HDD, the potential exists for the release of drilling fluids into the environment due to either a spill and borehole collapse, or the rupture of mud to the surface, known as a "frac-out". The drilling fluid is typically a mixture of clean, freshwater combined with bentonite. The frac-out is caused when excessive drilling pressure results in drilling mud moving towards ground surface. There are several measures that may be taken to reduce the risks associated with frac-outs as defined by Fisheries and Oceans Canada, including maintaining a minimum distance between the drill path and the bottom depth of the watercourse, and continuously monitoring for frac-out during all phases of drilling and pipe installation. Enbridge (2009) requires a minimum of 1.5 m separation between the pipeline and the watercourse bed to minimize the risks associated with frac-outs as indicated on Figures 14 to 16.

In the event that a frac out occurs, the contractor is responsible for ensuring that appropriate measures are taken to mitigate the potential impacts. Such measures may include stopping work, containing the drilling mud, and preventing further migration into the watercourse. All materials and equipment necessary to contain drilling mud releases should be readily available onsite. If the frac-out cannot be adequately controlled, alternative measures must be considered including re-drilling at a more appropriate location, or isolating the watercourse to complete drilling at the initial location.

6.2 CONSTRUCTION DEWATERING

As described in Section 3.3.2, HDD will be used to extend the pipeline below each watercourse/wetland crossing. The tie-in pits will be constructed on each side of the water crossing approximately 30 m from the bank of the creek in accordance with the ORMCP (JWSL, 2009). The approximate location of the pipeline as well as each tie-in pit is illustrated on Figures 14 to 16. At locations where the water table is higher than the bottom of the tie-in pit, sump pumps will be used for dewatering to maintain the water table below the base of the tie-in pit during construction. The tie-in pits would be excavated in materials ranging from silty sand to clayey silt (Table 4). The tie-in pits are expected to vary from 1.2 m BGS to 2.5 m BGS (Enbridge, 2009), and are conservatively assumed to have a footprint of 3 m by 4 m. Based on December 2009 water level data, it is anticipated that dewatering will be required at all tie-in pits except at WC6 as the water table is greater than 2.5 m BGS. The height of water required to be dewatered at each watercourse crossing is summarized in Table 4.

The Dupuit-Forcheimer (1930) Flow Equation was used to estimate how much water might need to be managed to facilitate the connection of the pipe at the tie-in pit at each watercourse crossing location. The Dupuit-Forcheimer Flow Equation calculates groundwater inflow to an excavation in an unconfined aquifer and takes into account recharge from the water crossing, which will be located approximately 30 m from each tie-in pit. The amount of drawdown required was estimated based on static water levels recorded at either the water crossings and monitoring well locations (0.04 m BGS to 2.26 m BGS), whichever resulted in the greatest required drawdown (Table 4). To be conservative, the maximum estimated drawdown and the hydraulic conductivity associated with the most transmissive material encountered while drilling (silty sand) was used for the calculations. Parameters used in the calculations included conservative literature values (Freeze & Cherry, 1979) of horizontal hydraulic conductivity ranging from 1 x 10^{-5} m/s to 1 x 10^{-9} m/s based on the geologic unit that the tie-in pit would encounter (Table 4), a saturated thickness ranging from 0.08 m to 2.26 m (Table 4), and a radius of influence of 30 m assuming the creek is a constant head boundary located 30 m from each tie-in pit, with a second scenario assuming a 10 m radius of influence. In the event that the tie-in pits would need to be located greater than 30 m from the watercourse, the rate of pumping required would be reduced, and therefore using distances of 10 m and 30 m was considered a conservative approach.

Table 4 presents the individual estimated pumping rates for tie-in pits located at each watercourse/wetland crossing. Using the Dupuit-Forcheimer Flow Equation, complete drawdown at each location was predicted at pumping rates ranging from less than 50 L/day to 8,614 L/day (Table 4) and will be completed using standard sump pump techniques, as required. Given that the volumes are expected to be well below 50,000 L/day, a MOE Permit To Take Water (PTTW) will not be required at any of the watercourse/wetland crossings.

The above calculation is based on interpreted soil stratigraphy, and based on estimated hydraulic conductivity and static water level data from available borehole logs. Actual dewatering rates in the field will be dependent on material encountered, static water level and the extent of hydraulic connection with the watercourse crossing.

Construction dewatering will be completed at the tie-in pits as required, and the discharge of the water will be managed as described in Section 6.2.2.

6.2.1 Private Wells

The closest wells to any of the tie-in pits are approximately 40 m away. All identified wells in the vicinity of WC1 to WTC1 are illustrated on Figures 4 to 13, respectively. The MOE Water Well Records indicates the wells in this area are drilled and range in depth from 28 m BGS to 102 m BGS. Any construction dewatering will result in local drawdown around a given tie-in pit of less than 2.26 m. Given that the construction dewatering will typically be for a period of 1 day or less, and that the closest private wells are relatively deep, the potential drawdown interference resulting from construction dewatering is not expected to have a detrimental impact on the water quantities of these private wells.

6.2.2 Management of Discharge

As described above, groundwater dewatering will be completed within the tie-in pits on an as-required basis. The contractor is responsible for the management of discharge. The following are suggestions in managing the discharge for the construction project:

- The inlet pump head is to be surrounded with clear stone or filter fabric;
- All outlet pump discharge must be directed to a sediment bag;
- The sediment bag is to be located on top of a rock/stone pad surrounded with silt fence or approved equal, and located outside of the work area. The contractor is to ensure sediment bag run-off does not come into contact with exposed soils;
- The contractor is to place the sediment bag such that flows are dispersed/directed downgrade from the work area and within the work allowance; and
- Discharge on private property is not acceptable unless the contractor has written authorization from the property owner.

7.0 Conclusions

A summary of the key findings of the investigation are presented below:

- Groundwater recharge conditions (i.e., surface water moves downward from the watercourse/wetland into the shallow groundwater system) were observed at WC4, WC5, WC7, WC9, and WTC1, whereas groundwater discharge conditions (i.e., groundwater flows from the subsurface into the watercourse/wetland) were documented at WC1 to WC3, WC6 and WC8;
- A variety of lithologicial materials will be encountered during drilling, including sand and gravel fill material, silty sand, silt, and clayey silt;
- The water table will be encountered during the construction of the pipeline at all of the watercourse/wetland crossing locations; and,
- No Permit to Take Water (PTTW) will be required for dewatering purposes, given that dewatering rates are expected to remain less than 50,000 L/day.

All of which is respectfully submitted,

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