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Preliminary Hydrogeological Investigation Report

**North Halton Coordinated Municipal Class Environmental Assessment Study
Halton Regional Road 4 (James Snow Parkway)
From Britannia Road to Highway 401
Town of Milton, Regional Municipality of Halton, Ontario**

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1. INTRODUCTION

This report presents the results of a preliminary hydrogeological investigation completed for the proposed improvements and potential widening of Halton Regional Road 4 (James Snow Parkway) between Halton Regional Road 6 (Britannia Road) and Highway 401 in the Town of Milton as part of the North Halton Coordinated Municipal Class Environmental Assessment (MCEA) Study, initiated by the Regional Municipality of Halton (Halton Region).

James Snow Parkway is a major north-south arterial road in Halton Region. In order to support future travel demand and a transportation system that is safe, continuous, connected, and coordinated for all users and abilities, the recommended solution for James Snow Parkway includes improving active transportation facilities, improving traffic operations at intersections through physical and operational modifications, and widening James Snow Parkway to six lanes to provide additional travel lanes and opportunities for transit supportive infrastructure.

The purpose of the hydrogeological investigation is to establish baseline hydrogeological conditions within the site, assess groundwater table conditions, evaluate construction dewatering requirements, assess the potential impacts that the proposed construction works may have on the local groundwater quantity and quality, and determine preliminary water taking permit requirements.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to HDR Inc. (HDR) who is conducting the MCEA Study for Halton Region.

A preliminary geotechnical investigation report was completed concurrently for this project under separate cover by Thurber and should be read in conjunction with this report.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. BACKGROUND REVIEW

2.1 Site and Project Description

James Snow Parkway is a major north-south arterial road. Between Britannia Road and Derry Road, the roadway presently comprises two-lane with a gravel shoulder and ditching on the northbound side and curb-and-gutter on the southbound side. North of Derry Road, the roadway has a four-lane urban cross section with a raised grass and concrete median and a separated multi-use path along the west side. The project area is approximately 5.8 km in length located

along James Snow Parkway between Regional Road 6 (Britannia Road) and Highway 401 in the Town of Milton, Ontario (the Site). The study area for the hydrogeological investigation was defined as 500 m around the Site (Study Area). The locations of the Site and Study Area are shown on Figure 1.

Based on a review of the Town of Milton Official Plan (Schedule B, urban area land use plan, March 2023), the land uses adjacent to the Site and within the Study Area consist of the Halton Urban Structure Plan (HUSP) urban area, residential area, business park area, industrial area, Sustainable Halton Plan (SHP) growth area, parkway belt west plan area, and natural heritage system. In general, the land use consists of residential subdivisions mainly to the west of James Snow Parkway, agricultural/farmland, woodlot, parkland, and vacant land mostly to the east of James Snow Parkway, and commercial/industrial properties to the north of the Site. Milton Fire Station is located approximately 500 m north of Derry Road West, on the east side of James Snow Parkway. Major crossings of the Site include tributaries of Sixteen Mile Creek and a CPKC rail overpass between Derry Road and Main Street East.

2.2 Topography and Drainage

The Site is located within the Sixteen Mile Creek watershed and falls under the jurisdiction of the Conservation Halton (CH). The watershed includes parts of the Town of Halton Hills, Town of Milton, Town of Oakville, and the City of Mississauga. The Sixteen Mile Creek includes three (3) main branches, East, Middle, and West branches, with a total length of approximately 150 kilometers. The watershed drains from its headwaters near Highway 7 in a southeasterly direction to its outlet south of Lakeshore Road into Lake Ontario (Gore & Storrie Ltd. and Ecoplans Ltd., 1996).

The regional topography slopes southeasterly. The topography at the Site varies from approximate elevations of 210 m near Highway 401 to 190 m near Britannia Road (Figure 2). Overland flow is expected to be influenced by existing topography and local surface water features. It is anticipated that overland flow follows the existing topography in the southeast direction.

2.3 Physiography

A review of the Physiographic Regions of Southern Ontario indicated that the Site is located within the Physiographic Region of the Peel Plain. The Peel Plain is a level-to-undulating tract of clay soils, covering an area of about 780 km² across the central portions of the Regional Municipalities of York, Peel, and Halton. It is a relatively flat area with soils that are predominantly clay with

localized clay loam and loam (Chapman and Putnam, 1984). A physiographic region map of the Site and surrounding area is shown on Figure 3.

2.4 Regional Geology and Hydrogeology Settings

Geological and hydrogeological conditions were based on publicly available information obtained from the Ontario Geological Survey (OGS), York, Peel, Durham, Toronto and the Conservation Authorities Moraine Coalition (YPDT-CAMC) Oak Ridges Moraine Groundwater Program (ORMGP).

The surficial geology of the Site primarily consists of fine-textured glaciolacustrine deposits of silt and clay, minor sand and gravel, interbedded silt and clay and gritty, pebbly flow till and rainout deposits. Till consisting of clay to silt-textured till (derived from glaciolacustrine deposits or shale) is observed approximately within the central portion of the Site, south of Derry Road. Modern alluvial deposits of clay, silt, sand, and gravel are also reported partially along the creeks. Figure 4 illustrates the mapped surficial geology of the Site.

The bedrock underlying the Site consists of the Queenston Formation, consisting of shale and siltstone with minor limestone and sandstone. A bedrock geology map is presented on Figure 5.

A regional north to south geological cross section using the ORMGP's mapping tools crossing the Site is provided on Figure 6. Based on a review of the regional geological cross section, the following units overlie the bedrock from youngest to oldest:

- Undifferentiated Upper Sediments;
- Halton Till (Aquitard);
- Oak Ridges Moraine (Aquifer);
- Newmarket Till (Aquitard);
- Thorncliffe Formation (Aquifer); and,
- Scarborough Formation (Aquifer).

A description of each hydrostratigraphic unit is provided below:

- Undifferentiated Upper Sediments – These deposits vary from near shore sand and gravel beach deposits of the Lake Iroquois shoreline to the fine sands, silts, and clays of glaciolacustrine pondings that occur north of the Lake Iroquois shoreline. Based on the regional cross section, the thickness of this unit in the vicinity of the Site ranged from approximately 1 m to 6 m.

- Halton Till – The Halton Till comprises of sandy silt to clayey silt till interbedded with silt, sand and gravel, with clay-rich deposits encountered in some areas. Based on the regional cross section, this unit is approximately up to 8 m thick in the Site area.
- Oak Ridges Moraine (ORM) – The uppermost aquifer underlying the Site is the ORM, which consists of sediments that include discontinuous sands and gravels. Based on the regional cross section, the ORM is absent/thin to the south of the Site and is approximately up to 10 m thick to the north of the Site.
- Newmarket Till – The Newmarket Till consists of a massive, stony, and consistently dense silty sand. It acts as a regional aquitard separating the ORM from the underlying Thorncliffe formation. The Newmarket Till is thin or absent in the vicinity of the Site.
- Thorncliffe Formation – The Thorncliffe Formation consists of glaciofluvial deposits of sand and silty sand within pre-existing valleys. The Thorncliffe Formation is typically considered an aquifer and is expected to be thin or absent in the Site area.
- Scarborough Formation – The Scarborough Formation is deposited as part of a large fluvial deltaic system fed by large, braided melt-water rivers draining from an ice sheet, flowing from Georgian Bay to Lake Ontario over an approximate area of 200 km². Based on the regional cross section, Scarborough Formation is mostly found to the north of the Study Area and is expected to be thin or absent in the Site area.

Based on the regional geologic cross section, the bedrock surface elevations at the Site ranged approximately from 195 m (north of the Site) to 180 m (south of the Site).

Information obtained from the ORMGP mapping shows that groundwater flows generally southeasterly toward tributaries of Sixteen Mile Creek, and ultimately to Lake Ontario.

2.5 MECP Water Well Records Review

A search of the Ministry of Environment, Conservation and Parks (MECP) well records database conducted for a 500 m radius around the Site returned a total of 86 records (Figure 7). Based on the MECP well records, the use of the majority of the well records was reported as observation, monitoring and test hole, abandoned, and unknown. Six (6) water supply wells were also found within the Study area, which were installed between the 1960s and 1990s. It is anticipated that most of the Study Area to the west of James Snow Parkway is serviced with municipal water. However, water supply wells could still be in use, particularly to the east of James Snow Parkway. A summary of the MECP's water well record database is provided in Appendix A.

2.6 Existing Water Taking Permits

A search of permitted water takers within the Study Area was conducted in July 2024. The search returned no active Permit to Take Water (PTTW) for construction dewatering, and no active Environmental Activity and Sector Registry (EASR) water taking registration within the Study Area.

2.7 Environmental Features

Based on the regional-scale source protection information Atlas mapping, the Site is not located within Wellhead Protection Areas (WHPAs), Significant Groundwater Recharge Areas (SGRAs), or Highly Vulnerable Aquifers (HVAs). Based on the Conservation Halton online mapping (planning and permits), the Site is partially located within the regulated areas.

A search on the Ministry of Natural Resources and Forestry (MNRF) online mapping accessed in July 2024 indicated that there are a number of woodlands, unevaluated wetlands, and watercourses within the Site and Study Area. Several tributaries of Sixteen Mile Creek cross the Study Area, and there are several ponds located within the Study Area. The nearby natural features are illustrated on Figure 8.

3. INVESTIGATION PROCEDURES

3.1 Geotechnical Investigation and Subsurface Drilling

Thurber conducted a geotechnical investigation for this project between September 28, 2023 and October 3, 2023, with a total of 15 boreholes (JSP-01 to JSP-13, JSP-CULV-01, and JSP-CULV-02) advanced to depths ranging from 1.8 m to 9.2 m below ground surface.

The geotechnical borehole logs were used to understand the local geology of the Site. The locations of the boreholes and monitoring wells are shown in Appendix B, and respective records of borehole logs are presented in Appendix C.

Based on the borehole logs, the subsurface stratigraphy encountered at the Site generally consisted of a pavement structure (asphalt over granular), overlying discontinuous layers of fill, silty clay, silt and sand overlying silty clay till. Sandy silt till/shale complex and shale bedrock were encountered below the clay till locally.

In general, the Site is underlain by a pavement structure and a granular base with thicknesses ranging from 710 mm to 1150 mm. The pavement structure and granular base are underlain by either fill or till deposits down to depths ranging from 0.9 m to 6.7 m. Fill was primarily encountered

in the boreholes located between Britannia Road and CPKC railway. A layer of silty clay was encountered below the pavement structure in boreholes JSP-06, JSP-07, and JSP-13, and below the fill material in Borehole JSP-CULV-01, at depths of 0.9 m to 2.6 m. Below the silty clay layer, alternating layers of silty sand to silt, and sand were also encountered in borehole JSP-13. Borehole JSP-13 was terminated within the sand deposits at 3.7 m depth. Till/shale complex was encountered below the clay till layer in borehole JSP-CULV-01 at a depth of 6.7 m to an investigated depth of 9.2 m. Shale bedrock was also encountered below the till in borehole JSP-05 at a depth of 3.0 m.

3.2 Hydrogeological Investigation and Monitoring Wells Installation

To support the hydrogeological investigation, Thurber installed a total of five (5) monitoring wells in selected boreholes. Monitoring wells were constructed with 50 mm diameter PVC pipes to depths ranging between approximately 4.4 m and 6.1 m below the ground surface. The monitoring wells were screened within the clay till and shale.

The monitoring wells were used to measure groundwater levels, collect samples for groundwater quality analyses, and estimate the hydraulic conductivity of the screened units. Monitoring well details are summarized in Table 3-1, and the locations of monitoring wells are provided on Figure 9.

Table 3-1: Monitoring Well Installation Details

Monitoring Well No.	Ground Surface Elev. (m)	Well Depth (m)	Screened Interval Elev. (m)	Screened Geologic Unit(s)
JSP-01	194.9	4.6	190.3 – 191.8	Clay Till
JSP-05	196.2	4.6	191.6 – 193.2	Shale
JSP-08	198.6	5.1	193.5 – 195.0	Clay Till
JSP-CULV-01	198.1	6.1	192.0 – 193.5	Clay Till
JSP-CULV-02	203.7	4.4	199.3 – 200.8	Clay Till

3.3 Single Well Response Tests

Rising head single well response tests (slug tests) were conducted on all monitoring wells. The tests were completed using the following method:

- In advance of conducting the slug tests, the monitoring wells (if not dried) were developed by withdrawing a minimum of three (3) well volumes of groundwater to remove excess sediment and to improve the transmissivity of the sand pack and well screen;
- Once the water level returned to a stabilized level, the static water level was measured and recorded. A datalogger was inserted into the well slightly above the bottom of the well. The datalogger was set to record water levels every 0.125 seconds to 10 seconds, depending on the anticipated rate of recovery of a well;
- A slug (or a known volume) of groundwater was removed from the well (rising head) to induce a change in the hydraulic head;
- Manual and electronic measurements of the water level were recorded until the water level in the well recovered sufficiently; and,
- Manual measurements were compared to electronic measurements for data quality control.

3.4 Groundwater Sampling

Groundwater quality samples were collected from selected monitoring wells (JSP-05, JSP-08, and JSP-CULV-01) for the purpose of considering disposal options and potential treatment needs at a preliminary level.

The monitoring wells were developed on October 24 and 26, 2023, prior to any sampling, by purging at least three (3) well volumes. The purpose of purging was to remove excess sediment that may have entered the well during installation and increase the representativeness of the natural groundwater in the well. Well development was assessed to be completed based on the number of well volumes purged, stabilization of general chemistry parameters of the purged groundwater (pH, temperature, electrical conductivity) over time, and qualitative observations such as a decrease in turbidity of the purged water.

Groundwater quality samples were collected on November 2 and 3, 2023. The collected samples were submitted to SGS Canada Inc. (SGS) for testing against the Regional Municipality of Halton Sewers By-Law No. 2-03, and the Provincial Water Quality Objectives (PWQO) for metals, inorganics, and general chemistry parameters. In addition to the unfiltered samples, field filtered metals samples were submitted to estimate the extent to which these components can be filtered.

4. INVESTIGATION FINDINGS AND ANALYSIS RESULTS

4.1 Groundwater Levels

Groundwater levels at monitoring wells were measured manually between October 24, 2023 and December 21, 2023. A summary of the water levels recorded in the monitoring wells is summarized in Appendix D.

The water level elevations for monitoring wells screened within the overburden ranged from 201.9 m to 191.9 m. The highest water level elevation (Elev. 201.9 m, depth 1.8 m) was measured in monitoring well JSP-CULV-02, and the lowest water level elevation (Elev. 191.9 m, depth 3.0 m) was measured in monitoring well JSP-01. The water level elevations at bedrock also ranged between 193.2 m and 194.0 m in monitoring well JSP-05 screened within the shale.

It should be noted that the groundwater levels, listed in Appendix D, are short-term readings and groundwater levels are expected to fluctuate seasonally. Higher groundwater levels may be expected during wet periods of the year such as spring or after periods of significant or prolonged precipitation. Perched water may be present in the fill material and/or more pervious sand pockets within the native soils.

4.2 Hydraulic Conductivity

Single well response tests (SWRTs) were conducted in all monitoring wells between October 25 and November 1, 2023. The groundwater analysis software *AquiferTest Pro* was used for analyzing the slug tests, and hydraulic conductivity (K) estimates were obtained using the Hvorslev method (1951). Hydraulic conductivity estimates are presented in Table 4-1, and plots of the slug test results are presented in Appendix E.

Table 4-1: Estimated Hydraulic Conductivity Values

Monitoring Well No.	Well Screen Bottom Elev. (m)	Well Screen Top Elev. (m)	Screened Geologic Unit(s)	Hydraulic Conductivity (K) (m/s)	Test Type
JSP-01	190.3	191.8	Clay Till	5.7×10^{-8}	Rising Head
JSP-05	191.6	193.2	Shale	2.9×10^{-5}	Rising Head
JSP-08	193.5	195.0	Clay Till	4.5×10^{-8}	Rising Head
JSP-CULV-01	192.0	193.5	Clay Till	6.7×10^{-6}	Rising Head
JSP-CULV-02	199.3	200.8	Clay Till	1.5×10^{-8}	Rising Head

The estimated hydraulic conductivity values for the clay till ranged between 6.7×10^{-6} m/s and 1.5×10^{-8} m/s, and the estimated hydraulic conductivity for the shale was estimated to be 2.9×10^{-5} m/s.

4.3 Groundwater Quality

Unfiltered groundwater quality sample results were compared with the Regional Municipality of Halton Sewers Use By-Law No. 2-03. Analytical results for metals, inorganics, and general chemistry parameters were also compared with the PWQO criteria. In addition to the unfiltered samples, field filtered samples were collected and tested for metals analysis. The certificates of analysis are provided in Appendix F.

The limits in the Regional Municipality of Halton Storm Sewer discharge include quantitative values for pH (between 6.5 and 8.5) and E. coli (less than 200 CFU/100 mL). The unfiltered groundwater samples met the storm sewer discharge limits for the parameters identified above. Storm sewer requirements provided in Section 5 of the Regional Municipality of Halton By-Law No. 2-03 should be followed.

The unfiltered groundwater samples met the Regional Municipality of Halton sanitary and combined sewers discharge limits except for total suspended solids (TSS). The exceedances of the Regional Municipality of Halton sanitary and combined sewers discharge limits are summarized in Table 4-2.

Table 4-2: Summary of Exceedances to the Halton Region Sanitary and Combined Sewers Limits

Sample ID	Parameter	Unit	Measured Concentration	Halton Region Sanitary and Combined Sewers Limits
JSP-05	Total Suspended Solids	mg/L	2,800	350
JSP-CULV-01	Total Suspended Solids	mg/L	410	350

The groundwater samples collected from monitoring wells JSP-05, JSP-08, and JSP-CULV-01 were also tested against the PWQO limits for general chemistry, selected metals, and inorganic parameters. A review of the analytical results indicated that multiple parameters exceeded the PWQO limits and interim PWQO limits. The exceedances of the PWQO limit and interim PWQO are summarized in Table 4-3.

Table 4-3: Summary of Exceedances to the PWQO and Interim PWQO Limits

Sample ID	Parameter	Unit	Measured Concentration	PWQO Limit Interim PWQO Limit
JSP-05	Total Phosphorous	mg/L	2.09	- 0.01
	Total Arsenic	mg/L	0.0132	0.1 0.005
	Total Cobalt	mg/L	0.0214	- 0.0009
	Total Copper	mg/L	0.0323	- 0.005
	Total Cadmium	mg/L	0.000229	0.0002 0.0005
	Total Iron	mg/L	34.0	0.3 -
	Total Lead	mg/L	0.0305	0.025 0.005
	Total Nickel	mg/L	0.0421	0.025 -
	Total Silver	mg/L	0.00018	0.0001 -
	Total Vanadium	mg/L	0.0401	- 0.006
	Total Zinc	mg/L	0.109	0.03 0.02
JSP-08	Total Arsenic	mg/L	0.0233	0.1 0.005
	Total Boron	mg/L	0.412	- 0.2
	Dissolved Boron	mg/L	0.521	- 0.2
	Total Cobalt	mg/L	0.0382	- 0.0009
	Dissolved Cobalt	mg/L	0.00298	- 0.0009
	Total Copper	mg/L	0.134	- 0.005
	Total Cadmium	mg/L	0.000422	0.0002 0.0005
	Total Iron	mg/L	81.6	0.3 -
	Total Lead	mg/L	0.0461	0.025 0.005
	Total Nickel	mg/L	0.0809	0.025 -

	Total Phosphorous	mg/L	3.3	- 0.01
	Total Silver	mg/L	0.00018	0.0001 -
	Total Thallium	mg/L	0.000374	- 0.0003
	Total Uranium	mg/L	0.0147	- 0.005
	Dissolved Uranium	mg/L	0.00684	- 0.005
	Total Vanadium	mg/L	0.0739	- 0.006
	Total Zinc	mg/L	0.238	0.03 0.02
JSP-CULV-01	Total Boron	mg/L	0.673	- 0.2
	Dissolved Boron	mg/L	0.731	- 0.2
	Total Cobalt	mg/L	0.00301	- 0.0009
	Total Copper	mg/L	0.0065	- 0.005
	Total Iron	mg/L	4.98	0.3 -
	Total Phosphorous	mg/L	0.147	- 0.01
	Total Uranium	mg/L	0.00673	- 0.005
	Dissolved Uranium	mg/L	0.00505	- 0.005
	Total Zinc	mg/L	0.026	0.03 0.02

Based on a review of the filtered (dissolved) groundwater sample results for metal parameters, filtering lowered the concentrations of some parameters below the PWQO and interim PWQO limits. However, dissolved boron, dissolved cobalt, and dissolved uranium remained above the interim PWQO limits.

5. DEWATERING ASSESSMENT

5.1 Construction Dewatering

Groundwater taking for construction dewatering is governed by the Ontario Water Resources Act (OWRA), Environmental Protection Act (EPA) and the Water Taking and Transfer Regulation 387/04, a regulation under the OWRA.

If the water taking rate will be greater than 50,000 L/day and less than 400,000 L/day, registration on the EASR is required. If the water taking rate will be greater than 400,000 L/day, a Category 3 PTTW is required.

A preliminary assessment of the need for a Category 3 PTTW or registration on the EASR is provided, based on dewatering estimates presented in this report; however, the final assessment will need to be determined during detailed design and investigation. For the purpose of water taking permitting, the estimated withdrawal rates are conservatively assessed in order to reduce the likelihood that actual pumping rates might exceed the permitted allowance thereby stopping work and delaying the project.

5.2 Dewatering Estimates

The following approach was used to estimate the budgeted peak water taking rate for the construction of proposed structures:

- A base groundwater extraction flow rate was estimated, and a factor of safety of three was applied to the base flow rate to provide an allowance for the removal of water from soil storage, variation in hydraulic conductivity values, actual excavation dimensions and geometry, and fluctuations of groundwater levels due to seasonality or other factors;
- An allowance for removal of rainfall into the excavation was included, assuming 24 hours is used to remove 50 mm of rainfall; and,
- Lowering groundwater to about 0.5 m below the excavation base to facilitate a dry and stable work area was assumed.
- The assumed water level elevation for each excavation was based on the highest recorded water level from a monitoring well in close proximity to the excavation.
- A conservative hydraulic conductivity of 6.7×10^{-6} m/s, estimated for clay till at monitoring well JSP-CULV-01, was used in assessing dewatering requirements for all excavations extended into clay till materials.

Dewatering rates were estimated using the Dupuit analytical solution provided in Powers et al. (2007). The zone of influence was estimated using the Sichardt equation. The calculations and equations for the peak flow rate and zone of influence are provided in Appendix G. Figure 11 shows the extent of the estimated zone of influence (ZOI).

For the purpose of estimating water taking flow rates, it is assumed that all excavations would be open cut and support of excavation would not be watertight. Surface water in the vicinity of the proposed excavations should be managed separately and directed away from the excavation. Temporary stream diversion measures should be provided to divert surface water runoff and stream flow away from excavations during construction.

Based on a review of the geotechnical investigation report (Thurber, 2024), it is Thurber's understanding that the structures may require dewatering including the following:

- CPKC Underpass Structure
- Culvert/Storm Drain, South of Derry Road
- Culvert/Storm Drain, South of Main Street East

It is also understood that James Snow Parkway crosses a bridge over a tributary of Sixteen Mile Creek, approximately 450 m south of Louis St. Laurent Avenue, which is not expected to require modifications. Therefore, no construction dewatering is anticipated for the existing bridge over this tributary of Sixteen Mile Creek. Furthermore, it is understood that there is an existing pumping station at CPKC underpass structure. If required, the capacity of the pumping station should be reviewed during detailed design should design changes warrant an assessment.

5.2.1 CPKC Underpass Structure

The existing CPKC railway underpass consists of a two-span concrete structure with two lanes of roadway presently passing under each span. Retaining walls extend from the bridge abutments along the northeast and southwest sides of the road cut. Wingwalls are present on the northwest and southeast quadrants. Concrete toe walls are present for a length of approximately 130 m under the bridge along both sides of the road, supporting elevated grassed areas in front of the bridge abutments. Based on the existing lane and toe wall configuration, it appears that the existing bridge was designed to accommodate an additional lane in each direction. Construction of the new lanes will require shifting of the toe walls towards the abutments. According to Thurber's geotechnical investigation report, the relocated toe walls are expected to be founded on very stiff to hard clay till or granular engineered fill (Thurber, 2024).

Based on the borehole information near the CPKC underpass (boreholes JSP-11 and JSP-CULV-02), the subsurface stratigraphy consisted of asphalt and granular overlying very stiff to hard clay till. The highest groundwater level, measured in monitoring well JSP-CULV-02, was at an approximate depth of 1.8 m (Elev. 201.9 m). Design information was not available at the time of preparing this report. It is assumed that the excavation is approximately 130 m long and 2 m wide, with a founding level for the toe walls at approximately 1.5 m.

Based on the above assumptions, the base of the excavation at toe walls will be above the ground water level and dewatering is expected to be minimal. To lower groundwater to 0.5 m below the base of the excavation in the clay till, the base groundwater flow is estimated to be approximately 39,000 L/day. With a safety factor of three on the base groundwater flow and a rainfall removal allowance of 50 mm in 24 hours, the estimated peak flow rate flow is approximately 130,000 L/day. The zone of influence is estimated to extend approximately 2 m from the edge of the excavation.

5.2.2 Culvert/Storm Drain, South of Derry Road

An outlet pipe from a stormwater pond appears to cross under the roadway approximately 400 m south of Derry Road, with a maintenance chamber located on both sides of the road. Based on Thurber's geotechnical investigation report, modifications to the culvert/drain do not appear to be necessary to accommodate road widening. Localized extensions or headwalls, if required, are expected to be founded on hard native clay till (Thurber, 2024).

Based on the borehole data at this culvert location (borehole JSP-CULV-01), the subsurface stratigraphy consisted of asphalt and granular, underlain by sand fill, followed by a thin (about 0.3 m thick) layer of stiff clay, overlying hard clay till encountered at 3.0 m depth (Elev. 195.1 m). The highest groundwater level measured in monitoring well JSP-CULV-01 was at an approximate depth of 3.4 m (Elev. 194.7 m). Design information was not available at the time of preparing this report. It is assumed that the excavation is approximately 6 m long and 3 m wide, with a founding level for the headwalls at approximately 3 m.

Based on the above assumptions, the base of the excavation at headwalls will be above the groundwater level and dewatering is expected to be minimal. To lower groundwater to 0.5 m below the base of the excavation in the clay till, the base groundwater flow is estimated to be approximately 4,000 L/day. With a safety factor of three on the base groundwater flow and a rainfall removal allowance of 50 mm in 24 hours, the estimated peak flow rate flow is approximately 13,000 L/day. The zone of influence is estimated to extend approximately 1 m from the edge of the excavation.

5.2.3 Culvert/Storm Drain, South of Main Street East

An outlet from a storm drain or road-crossing culvert is evident on the west side of the road, approximately 375 m south of Main Street East. Based on Thurber's geotechnical investigation report, modifications to the culvert/drain do not appear to be necessary to accommodate road widening. Localized extensions or headwalls, if required, are expected to be founded on hard native clay till (Thurber, 2024).

Based on the borehole information near this culvert (borehole JSP-CULV-02), the subsurface stratigraphy consisted of asphalt and granular, underlain by hard clay till encountered at 1.1 m depth (Elev. 202.6 m). The highest groundwater level measured in monitoring well JSP-CULV-02 was at an approximate depth of 1.8 m (Elev. 201.9 m). Design information was not available at the time of preparing this report. It is assumed that the excavation is approximately 6 m long and 3 m wide, with a founding level for the headwalls at approximately 2 m.

To lower groundwater to 0.5 m below the base of the excavation in the clay till, the base groundwater flow is estimated to be approximately 8,000 L/day. With a safety factor of three on the base groundwater flow and a rainfall removal allowance of 50 mm in 24 hours, the estimated peak flow rate flow is approximately 25,000 L/day. The zone of influence is estimated to extend approximately 5 m from the edge of the excavation.

5.3 Summary of Construction Dewatering Estimates

The estimated maximum construction dewatering pump rates and zone of influence for the analyzed excavations are summarized in Table 5-1.

Table 5-1: Estimated Construction Dewatering Volumes

Construction Element	Base Groundwater Flow (L/day)	Groundwater Flow with a Safety Factor of 3 (L/day)	Stormwater Allowance (L/day)	Estimated Peak Flow Rate (L/day)	Approx. Zone of Influence (m)
CPKC Underpass Structure (One Toe Wall)	39,000	117,000	13,000	130,000	2
One Headwall at Culvert South of Derry Road	4,000	12,000	1,000	13,000	1
One Headwall at Culvert South of Main Street East	8,000	24,000	1,000	25,000	5

The peak water taking rate including a safety factor of three on the base groundwater flow and a rainfall removal allowance of 50 mm in 24 hours ranged from 13,000 L/day at the headwall south of Derry Road to 130,000 L/day at the toe wall of CPKC underpass.

Some perched water may exist in the fill materials that may contribute additional groundwater into the excavation and need to be temporarily managed. It is anticipated the perched water in the fill will not be a source of continuous groundwater flow.

6. IMPACT ASSESSMENT

Within the construction dewatering zone of influence, potential impacts such as ground subsidence and reduction in groundwater flow to groundwater users and watercourses must be considered. The potential impacts and mitigation measures are discussed herein.

6.1 Geotechnical Impacts

Dewatering of open excavations for the CPKC underpass toe walls and headwalls at culverts would be within the clay till with an approximate drawdown of the water table up to 0.7 m and a maximum estimated zone of influence of 5 m from the edge of the excavation.

In general, the potential for ground settlement resulting from construction dewatering is greatest where the estimated drawdown is significant and compressible soils are located within the zone of influence of the dewatering. Structures or other infrastructure located within this zone may potentially be impacted by the ground settlement depending upon the magnitude of settlement and the sensitivity of the facility. The estimated maximum surface settlement values occur at the ground surface at the edge of the excavation. At distances away from the edge of the excavation, the estimated settlement decreases to almost zero at the outer limit of the respective zone of influence.

For the dewatering conditions assessed in this report, the magnitude of the drawdown is relatively low (less than 1 m), and the drawdown occurs primarily in very stiff to hard clay till. Under these conditions, settlement of the surrounding ground surface within the zone of influence as a result of dewatering is expected to be negligible (<10 mm).

A preconstruction survey of all structures and utilities within the zone of influence should be considered prior to dewatering activities, and a survey and periodic visual inspections should be considered during dewatering to assess if any undesirable deformation has occurred. Based on the site conditions, it is anticipated that the settlement of the overburden deposits will be negligible, however, consideration should be provided for the continuation of the settlement

monitoring post construction. A settlement monitoring program will need to be designed and implemented in accordance with the applicable requirements. The settlement monitoring program should be reviewed and approved by the responsible authorities.

If significant sediment and fines are removed during the dewatering due to improperly filtered extraction wells, then ground loss and settlement beyond that described above could occur.

6.2 Impacts to Surface Water and Natural Environment

Lowering the shallow groundwater level due to construction dewatering could potentially reduce the groundwater input into nearby environmental features. In addition, dewatering discharge that may be directed to nearby watercourses could potentially alter the physical, chemical and thermal regime of the receiving streams. Reduction of groundwater discharges to surface water flow, to some extent, may occur due to groundwater extraction; however, the magnitude of the impact is not expected to be significant due to the limited drawdown and low hydraulic conductivity of the creek beds and clay till.

The groundwater of the quality that was discussed in Section 4.3 cannot be discharged to the natural environment without pre-treatment due to exceedances of the PWQO limits based on the samples that were collected and analyzed. If discharge occurs to the natural environment or within 30 m of a surface water body, the discharge quality must be tested and routinely monitored to ensure that it meets the PWQO and interim PWQO requirements. Erosion and sediment control plans must also be implemented to ensure that there is no excess sediment being stirred up or into the surface water and to ensure that no erosion occurs. If discharging to land surface more than 30 m from a watercourse, the discharge must be monitored to ensure that the discharge does not reach a stream (unless it also meets the PWQOs).

6.3 Impacts to Water Well Users

The MECP well records review, as documented in Section 2.5, indicated that six (6) water supply wells are located within the Study Area. These water supply wells are situated outside of the estimated construction dewatering zone of influence for all assessed structures; therefore, impacts to groundwater users as a result of the construction dewatering are not expected.

6.4 Other Potential Impacts

With prolonged dewatering activities, there can be potential for inorganic or organic chemical compounds present within the zone of influence to migrate and enter open excavations where sufficient flow rate and time permit. If any contaminated groundwater is collected from the

dewatering operations, it must be treated to meet any discharge criteria or disposed of at a facility licensed to handle such materials.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Water Taking Permit

The peak dewatering rate for the construction elements discussed in Section 5.1 including a three (3) times safety factor on the base groundwater flow and a rainfall removal allowance of 50 mm in 24 hours was estimated to range approximately between 13,000 L/day and 130,000 L/day. Since the estimated peak water taking rate is greater than 50,000 L/day at the CPKC underpass toe wall, an EASR registration will be required for this structure. For culverts, the estimated peak water taking rate is less than 50,000 L/day for one headwall, which is below the EASR limit. If two (2) headwalls are to be constructed concurrently, particularly at the culvert south of Main Street East, it is recommended that the dewatering activities be registered under the EASR.

For each excavation, the water taking rate is expected to be highest at the beginning of dewatering when the water table is being actively lowered and during large rainfall events. The means and methods of sequencing of excavations will be primarily determined by the contractor.

For construction dewatering purposes, a qualified dewatering engineer retained by the contractor must complete an independent assessment based on their own assumptions and requirements (e.g., overall analysis, assumptions, predicted dewatering rates, etc.) using the final design specifications and the contractor's proposed construction methodologies. The design and operation of the dewatering activities is the sole responsibility of the contractor, to ensure the EASR limit is not exceeded. The excavation and dewatering methods that will be used in the field will be determined by the selected contractor in accordance with the construction contract documents.

7.2 Discharge of Groundwater

Groundwater of the quality that was observed among the tested wells met the discharge criteria of the Halton Sewer Use By-Law for sanitary and combined sewers except TSS, and thus pre-treatment is anticipated to be required.

Groundwater of the quality that was observed herein could not be discharged to the natural environment without pre-treatment due to exceedances of the PWQO and interim PWQO limits for various parameters including total phosphorous, total arsenic, total cobalt, total copper, total cadmium, total iron, total lead, total nickel, total silver, total vanadium, total zinc, total boron, total

thallium, and total uranium. Based on a review of the results of the filtered groundwater samples for metal parameters, filtering lowered some parameters concentrations below the PWQO and interim PWQO limits, except for dissolved boron, dissolved cobalt, and dissolved uranium which are above the interim PWQO limits. If considering discharge to the natural environment, additional treatment beyond sediment control is anticipated to be required. Also, approvals from external agencies such as CH, MNRF, and Fisheries and Oceans Canada (DFO) may be required.

The results obtained herein were representative of the water sampled from the selected wells at the time of sampling and provided a general understanding of groundwater quality under those conditions; however, the water quality may vary significantly from the results obtained based on location, time, meteorological conditions, and in particular based on construction and dewatering methods. The contractor would need to consult with its dewatering and water treatment specialists to develop methods and means to meet the applicable limits based on the results presented herein and on any additional testing taken by the contractor and/or its consultant during detailed design. An experienced dewatering contractor and water treatment contractor are recommended to be retained to design and operate dewatering and/or treatment operations as required.

The extent of suspended solids in the groundwater or in water that is collected during construction dewatering (for example from a sump in an open excavation) will significantly affect the concentrations of many parameters that may be regulated based on a discharge location. The value of testing groundwater quality during the investigation is primarily to identify the types of contaminants that may need to be managed, and the presence of anthropogenic contaminants that are listed in the given discharge criteria that may require specific treatment.

7.3 Control of Impacts and Monitoring Program

The following measures are recommended to mitigate the potential for the dewatering activities to cause negative impacts as assessed previously:

- If discharge to the natural environment is proposed, treatment of groundwater to meet acceptable levels is required. Suitable treatment would likely include measures to address suspended sediments and associated metals. The operation and monitoring of discharge facilities should be carried out by an experienced dewatering contractor and water treatment contractor familiar with fisheries and water quality requirements.



- Where discharge is to land surface, the discharge must be at least 30 m away from any water bodies including streams. Temporary erosion control measures should also be developed and installed to control erosion at the discharge points. If discharging near Sixteen Mile Creek tributaries is considered, it is recommended to develop a monitoring plan to identify, minimize, and mitigate potential impacts to the tributaries during the detailed design stage.
- During the detailed design stage, it will be necessary to refine the analysis of the hydrogeological conditions to estimate dewatering rates and zone of influence. These findings will be used to confirm the water takings requirements and the appropriate approvals from the MECP prior to the commencement of construction.
- Additional monitoring and terms and conditions will apply as determined by the water taking permits, by any discharge agreement or permit, and by other regulatory or jurisdictional bodies.



THURBER ENGINEERING LTD.

8. SIGNATURES/CLOSURE

We trust this report provides the information you require at this time. If you have any questions, please contact the undersigned at your convenience.



Mahmoud Meskar, Ph.D., P.Eng.
Hydrogeological Engineer



Karel Furbacher, P.Eng.
Senior Geotechnical Engineer



Alireza Hejazi, Ph.D., P.Eng.
Senior Hydrogeologist | Review Engineer

Date: February 28, 2025
File: 35477

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Town of Milton Official Plan, Schedule B, Urban Area Land Use Plan, Last Updated March 2023,
Office Consolidation March 2023.



STATEMENT OF LIMITATIONS AND CONDITIONS

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This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

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The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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