



THURBER ENGINEERING LTD.

Preliminary Geotechnical Investigation Report

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

Client: HDR Inc.

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EXECUTIVE SUMMARY

This report presents the results of a preliminary geotechnical investigation completed for the proposed improvements and potential widening of Halton Regional Road 4 (James Snow Parkway) between 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. The approximately 5.8 km long section of James Snow Parkway addressed in this study comprises a two-lane roadway between Britannia Road and Derry Road, and a four-lane urban section North of Derry Road to Highway 401. The roadway crosses a bridge over a tributary of Sixteen Mile Creek approximately 450 m south of Louis St. Laurent Avenue, and an underpass below a Canadian Pacific Kansas City (CPKC) railway between Derry Road and Main Street East. An outlet pipe from a stormwater pond appears to cross under the roadway approximately 400 m south of Derry Road to outlet on the east side of the road. 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.

The geotechnical investigation comprised a review of existing subsurface data from along the corridor and drilling a total of fifteen boreholes to depths of 2.1 to 9.2 m. Monitoring wells were installed in five boreholes to permit monitoring of the groundwater levels along the corridor.

The subsurface stratigraphy encountered in the boreholes 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 contacted below the clay till locally.

Based on the results of the investigation, the following preliminary pavement designs are recommended for potential widening of the roadway:

Component	Thickness		
	Hwy 401 to Main	Main to Derry	Derry to Britannia
HL1	50	50	50
HDBC (2 lifts)	150	130	100
OPSS Granular A Base	150	150	150
OPSS Granular B Type II Subbase	500	450	450



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Pavement design analysis indicates that strengthening of the existing pavement structure is required between Derry Road and Main Street to support the future traffic demands. The structural requirements can be achieved by milling 100 mm of the existing asphalt and replacement with 100 mm of new asphalt (50 mm HL1 over 50 mm HDBC). Rehabilitation of the existing pavement structure in the remaining sections may entail milling 50 mm of the existing asphalt surface followed by replacement with a new 50 mm thick layer of HL1 surface course.

The existing CPKC railway underpass consists of a two span concrete structure with two lanes of roadway presently passing under each span. Based on the existing bridge configuration, it appears that the existing structure was designed to accommodate an additional lane in each direction. Construction of the new lanes will require shifting of existing toe walls towards the abutments. Standard toe wall design in accordance with OPSD 3120.100 is considered feasible.

Based on visual observation, modifications to the culverts/drains 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, and therefore support of these facilities is not expected to be an issue.

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1. INTRODUCTION

This report presents the results of a preliminary geotechnical 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 is proposed to consist of facility improvements for pedestrians, cyclists, mobility device users and other non-vehicular travel to create a safe and accessible network; and improvements to overall operations including intersections and widening to accommodate turn lanes, for example.

The purpose of the geotechnical investigation was to explore the subsurface conditions along the corridor and, based on the data obtained, provide a borehole location plan, record of borehole sheets, laboratory test results, a written description of the subsurface conditions, and preliminary geotechnical comments and recommendations for the proposed works.

A hydrogeological assessment was completed concurrently for this project. The results of hydrogeological assessment are reported under separate cover and should be read in conjunction with this report.

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

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 INFORMATION

2.1 Site and Project Description

Halton Regional Road 4 (James Snow Parkway) is a major north-south arterial road. The study corridor of this report is approximately 5.8 km in length extending from Regional Road 6 (Britannia Road) to Highway 401 in the Town of Milton. Between Britannia Road and Derry Road, the roadway presently comprises a two-lane cross section 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 roadway crosses a bridge over a tributary of Sixteen Mile Creek approximately 450 m south of Louis St. Laurent Avenue, and an underpass below a Canadian Pacific Kansas City (CPKC) railway between Derry Road and Main Street East. An outlet pipe from a stormwater pond appears to cross under the roadway approximately 400 m south of Derry Road to outlet on the east side of the road. 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.

The general topography of the study corridor is undulating, generally sloped downwards in a southeasterly direction towards Lake Ontario. The ground surface elevation ranges from near Elev. 207 at Highway 401 to near Elev. 192 at Britannia Road. Land use on the west side of the corridor primarily consists of residential subdivisions with occasional parkland, tree lots and stormwater ponds; residential development is currently underway south of Louis St. Laurent Avenue. The lands on the east side of the corridor generally consist of farmland with areas under development for employment purposes.

2.2 Geology

Based on the information in *The Physiography of Southern Ontario by Chapman and Putnam* (1984)¹, the study corridor is in the Peel Plain physiographic region. The Peel Plain physiographic region is described as a level to undulating tract of clay soils that gradually slopes towards Lake Ontario. The material of the plain is generally described as till containing large amounts of shale and limestone, typically modified by a veneer of clay.

Based on *Surficial Geology of Southern Ontario*² the surficial deposits in the study corridor are fine-textured glaciolacustrine deposits of silt and clay with minor sand and gravel, clay to silt textured till deposits derived from glaciolacustrine deposits or shale, and modern alluvial deposits of clay, silt, sand, gravel, which may contain organic remains.

¹ Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

² Ontario Geological Survey, 2010: Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV

According to *Paleozoic Geology of Southern Ontario*³, the bedrock geology in the study corridor consists of red shale of the Queenston Formation. Bedrock outcrops were not mapped within the project limits.

2.3 Previous Investigations

Several geotechnical investigations have been carried out previously along the corridor. The following sections provide a summary of the available geotechnical reports and drawings reviewed in preparation of this report. Selected geotechnical data is provided in Appendix E. Reference is made to the reports for additional details, including investigation procedures, laboratory testing results and detailed subsurface conditions, which are not provided herein.

2.3.1 James Snow Parkway Underpass at Highway 401

A foundation investigation was conducted by the Ontario Ministry of Transportation and Communications (MTC) for the James Snow Parkway Underpass at the Highway 401 interchange in 1979 (GEOCRES 30M12-138, report dated May 25, 1979). The field investigation consisted of three sampled boreholes advanced to depths of 9.6 m to 15.7 m (Elev. 197.1 to 189.4 m).

The subsurface stratigraphy encountered in the boreholes consisted of clayey silt till throughout the depth of investigation, locally overlain by a 0.6 m thick layer of granular roadway fill. The till contained some sand, trace gravel, occasional cobbles and boulders, and sand seams/layers. The consistency of the till was typically hard, locally very stiff. Groundwater was observed at depths of 6.1 and 6.4 m (Elev. 198.8 and 198.7 m) in two boreholes during drilling.

2.3.2 New Intersection at Derry Road and New James Snow Parkway

Thurber Engineering conducted a geotechnical investigation to support the design and construction of a proposed new intersection at Derry Road and James Snow Parkway in 2001/2002 (Ref. 14-145-15 dated January 24, 2002). The field investigation consisted of seven boreholes advanced to 5 m depth, two probe holes advanced to 0.5 m depth, and several other probes to estimate topsoil thickness.

The subsurface stratigraphy encountered in the sampled boreholes typically consisted of 1.0 to 1.2 m of shoulder granular material overlying native silty clay till. The clay till was encountered at the ground surface in two boreholes located off the road. The till was typically stiff to very stiff

³ Armstrong, D.K. and Dodge, J.E.P., 2007: Paleozoic geology of southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219.

within the upper 1.0 to 2.0 m and became hard with depth. Topsoil thicknesses of 120 to 250 mm or asphalt thicknesses of 50 to 85 mm were encountered in the probes.

Groundwater was measured at depths of 1.1 and 1.2 m in standpipe piezometers installed in two boreholes.

2.3.3 James Snow Parkway Grade Separation at CPR

A geotechnical investigation was conducted by Thurber Engineering Ltd. for the proposed James Snow Parkway grade separation at CPR Galt Subdivision Mileage 30.16 in Milton, Ontario (Ref. 14-145-20; report dated January 21, 2005). The field investigation consisted of twenty boreholes advanced to depths of 1.8 to 16.5 m (Elev. 202.12 to 187.80 m) at the underpass location and along the James Snow Parkway extension from Main Street to Waldie Avenue.

The subsurface stratigraphy encountered in the boreholes generally consisted of topsoil, asphalt and/or fill overlying a silty clay layer underlain by clayey silt till, sandy silt till, and shale bedrock. Where encountered, the topsoil thickness ranged from 75 to 250 mm and the asphalt thickness ranged from 75 to 100 mm. The fill thickness varied from 0.4 to 1.5 m. The silty clay layer was 0.5 to 1.5 m thick and was soft to very stiff.

The underlying till deposit consisted of clayey silt till containing trace to some sand and gravel, with occasional shale and siltstone fragments. Below 8.7 to 9.1 m depth, the till graded to non- to slightly plastic sandy silt with trace to some clay, trace gravel, and shale fragments. The consistency of the clayey silt till was typically very stiff to hard, and the relative density of the sandy silt till was very dense.

Weathered, reddish brown shale bedrock was encountered below the till in six boreholes at depths of 11.1 to 13.7 m (Elev. 193.1 to 190.5 m).

Groundwater was observed at depths of 0.3 to 11.6 m in ten of the boreholes upon completion of drilling. Piezometers were installed in four boreholes upon completion; groundwater levels measured approximately six weeks after installation were 2.2 m below ground surface (bgs) (Elev. 202.1 to 201.9 m).

2.3.4 Extension of James Snow Parkway and Louis St. Laurent Avenue

A geotechnical investigation was conducted by Thurber Engineering Ltd. for the proposed extension of James Snow Parkway between Derry Road and Britannia Road, and Louis St. Laurent Avenue between Fourth Line and the new James Snow Parkway in Milton, Ontario (Ref.

19-4854-0; report dated June 12, 2006). The field investigation comprised twenty-eight boreholes drilled to depths of 3.1 to 15.4 m (Elev. 196.0 to 175.9 m).

The subsurface stratigraphy encountered in the boreholes generally consisted of a surficial topsoil layer overlying silty clay and cohesive till deposits of silty clay and clayey silt. The till typically becomes less plastic with depth, and is underlain by sandy silt till, silt, and sand and gravel deposits. Shale bedrock was contacted locally.

The topsoil thickness typically ranged from 175 to 250 mm. The silty clay layer is typically very soft to stiff and extended to depths of 0.6 to 2.2 m (Elev. 198.5 to 189.0 m).

Cohesive till comprised of silty clay to clayey silt typically containing some sand, trace gravel, and occasional limestone and shale fragments was encountered below the surficial layers at depths of 0.6 to 2.2 m, locally at the ground surface. The till was typically very stiff to hard. Sixteen of the boreholes were terminated in the till at depths of 3.4 to 5.2 m. In ten boreholes, the cohesive till graded to sandy silt till at depths of 3.0 to 4.3 m.

The sandy silt till typically contained trace clay, trace gravel, and occasional boulders, limestone and shale fragments. Isolated silt to silt and sand layers up to 3.2 m thick and sand to sand and gravel layers up to 1.9 m thick were encountered within or below the till. The sandy silt till is generally very dense.

Weathered shale bedrock of the Queenston formation was encountered below the till and sand and gravel in six boreholes. The bedrock was contacted at depths of 2.2 to 14.3 m (Elev. 196.5 to 176.9 m). The bedrock was described as reddish brown shale with frequent hard grey limestone layers up to 130 mm thick.

Groundwater was observed in three boreholes at depths of 1.1 to 4.6 m (Elev. 186.7 to 197.0) upon completion of drilling. Piezometers were installed in ten boreholes; groundwater was measured in the piezometers at depths of 0.2 to 2.2 m (Elev. 197.6 to 191.3), locally 0.1 m above the ground surface (Elev. 191.4) at one location, approximately one month after installation.

2.3.5 James Snow Parkway Underpass Assessment, Highway 401

A report issued in April 2012 by Golder Associates Ltd. titled, "Preliminary Foundation Investigation and Design Report" (GEOCRE 30M12-335) presented a desktop assessment of the James Snow Underpass structure in connection with the proposed widening of Highway 401. The assessment relied upon the existing information collected by MTO during the 1979 foundation investigation and no new subsurface information was obtained.



3. INVESTIGATION PROCEDURES

3.1 Borehole Drilling Program

The drilling program was carried out from September 28th to October 3rd, 2023. The geotechnical investigation comprised a total of fifteen boreholes as summarized in Table 3.1 below. The approximate locations of the boreholes are shown in Drawings 35477-JSP-01 to 35477-JSP-03 in Appendix A. Borehole details are provided in the Record of Borehole sheets included in Appendix B.

Table 3.1: Borehole Details

Borehole No.	Ground Surface Elevation (m)	Termination Depth (m)	Termination Elevation (m)	Monitoring Well
JSP-01	194.9	5.2	189.7	☒
JSP-02	195.7	2.1	193.6	☐
JSP-03	194.5	5.2	189.3	☐
JSP-04	194.0	2.1	191.9	☐
JSP-05	196.2	4.6	191.6	☒
JSP-06	199.6	1.8	197.8	☐
JSP-07	198.7	2.1	196.6	☐
JSP-08	198.6	5.9	192.7	☒
JSP-09	200.8	2.1	198.6	☐
JSP-10	202.2	3.5	198.7	☐
JSP-11	202.4	1.9	200.5	☐
JSP-12	205.0	2.1	202.8	☐
JSP-13	206.6	3.7	203.0	☐
JSP-CULV-01	198.1	9.2	188.9	☒
JSP-CULV-02	203.7	5.2	198.5	☒

The borehole locations were established in the field by Thurber using a portable GPS receiver and verified relative to existing site features. The ground surface elevations at the borehole locations were determined using a Trimble R12 GNSS receiver. The ground surface elevations are orthometric derived from GNSS observations and Natural Resources Canada Geoid Model HT2.0, which is a height transformation model that was created by modifying a gravimetric geoid model (CGG2000) to fit the historical CGVD28:78 datum. Borehole location coordinates are presented in the Universal Transverse Mercator (UTM) system (UTM-17 NAD83).

The borehole locations were cleared of utilities prior to commencement of drilling. The boreholes were repositioned as necessary in consideration of surface features, underground and overhead utilities, and roadway traffic operations.

Boreholes were advanced using solid stem augers powered by a D50 truck mounted drill rig supplied and operated by DBW Drilling Ltd. Soil samples were obtained at selected intervals using a 50 mm outside diameter split-spoon sampler driven in conjunction with the Standard Penetration Test (SPT). Dynamic Cone Penetration Tests (DCPT) were completed where drilling methods were inferred to have disturbed the in-situ ground conditions. Field vane shear tests were conducted where soft cohesive materials were encountered.

Manually excavated test pits were advanced in the grassed shoulder adjacent to Boreholes JSP-02, JSP-04, JSP-06, JSP-08, JSP-10, and JSP-12 to determine topsoil thickness.

The field investigation was carried out under the full-time supervision of Thurber technical staff. All boreholes were logged in the field. Soil samples were identified, placed in labelled containers, and transported back to Thurber's laboratory for further examination and testing.

3.2 Monitoring Well Installation

Groundwater conditions were observed in the open boreholes throughout the drilling operations. Monitoring wells were installed in five boreholes to permit monitoring of the groundwater levels at along the corridor. The monitoring wells consisted of 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The installation details are summarized in Table 3.2 below.

Table 3.2: Monitoring Well Details

Borehole/ Monitoring Well No.	Ground Surface Elevation (m)	Monitoring Well Tip		Slotted Screen Length (m)	Mid-Screen Depth (m)	Mid-Screen Elevation (m)
		Depth (m)	Elevation (m)			
JSP-01	194.9	4.6	190.3	1.5	3.9	191.0
JSP-05	196.2	4.6	191.6	1.5	3.9	192.3
JSP-08	198.6	5.1	193.5	1.5	4.4	194.2
JSP-CULV-01	198.1	6.1	192.0	1.5	5.4	192.7
JSP-CULV-02	203.7	4.4	199.3	1.5	3.7	200.0

The boreholes in which no monitoring wells were installed were backfilled in general accordance with Ontario Regulation 903.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and the drill rods as the samples were retrieved and when appropriate by measurement of the water level in the open borehole.

Groundwater levels were collected at the monitoring well locations using a groundwater level meter. The water level meter was cleaned between uses at each monitoring well location.

4. LABORATORY TESTING

4.1 Geotechnical

Geotechnical laboratory testing was carried out at Thurber's laboratory. All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to grain size distribution analysis (hydrometer and/or sieve) and Atterberg Limits testing, where appropriate.

Results of the geotechnical soil laboratory testing are presented on the Record of Borehole sheets in Appendix B and in detail in Appendix C.

4.2 Corrosivity and Sulphates

Two (2) soil samples were submitted for chemical testing to assess the corrosion potential of soil to ductile iron and the potential for sulphate attack on subsurface concrete structures. The analyses were carried out by SGS Canada Inc. (SGS), an independent Canadian Association for Laboratory Accreditation (CALA) accredited laboratory. The laboratory Certificates of Analysis are included in Appendix D.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It should be recognized and expected that soil conditions will vary between and beyond borehole locations.

The subsurface stratigraphy encountered in the boreholes 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 contacted below the clay till locally.

5.1 Pavement Structure

A pavement structure was encountered in all boreholes except Borehole JSP-02 which was drilled on the gravel shoulder. The pavement structure consisted of 175 to 300 mm of asphalt (average 210 mm) overlying a granular base with thicknesses of ranging from 710 to 1150 mm (average 875 mm). A granular shoulder thickness of 910 mm was encountered in Borehole JSP-02.

The results of grain size distribution analyses carried out on samples of the granular fill materials are presented on Figure C1 in Appendix C. The results of the grain size distribution analyses indicated 39 to 49% gravel, 38 to 44% sand, and 13 to 17% silt and clay sized particles.

The samples tested do not meet the OPSS Granular A or B gradation specifications due to the percentage of silt and clay sized particles. The results may be impacted by the effects of compaction, auger sampling procedures, infiltration of fines with road runoff, or deterioration of the granular material over time.

5.2 Topsoil

A 100 to 150 mm thick layer of topsoil was encountered at the ground surface of the manually excavated test pits in the grassed shoulder adjacent to boreholes JSP-02, JSP-04, JSP-06, JSP-08, JSP-10, and JSP-12. The topsoil thicknesses are provided in Table 5.1 below.

Table 5.1: Topsoil Thickness

Borehole	Thickness (mm)
JSP-02	150
JSP-04	125
JSP-06	125
JSP-08	125
JSP-10	100
JSP-12	125

The topsoil thicknesses will vary between and beyond the borehole locations and the reported thicknesses are not meant to be used for estimating quantities.

5.3 Fill

A layer of fill was encountered below the pavement structure in all boreholes except JSP-06, JSP-07, JSP-11, JSP-12, JSP-13, and JSP-CULV-02 at depths of 0.9 to 1.4 m (Elev. 192.9 to 201.2 m). The fill was variously described as silty clay to clayey silt (cohesive) containing various amounts of sand (trace to sandy) and gravel (trace to some), and sand to silt (cohesionless) containing various amounts of clay and gravel (trace to some). Where penetrated, the fill extended

to depths of 2.2 to 3.2 m (Elev. 191.6 to 199.0 m). Boreholes JSP-02, JSP-04, and JSP-09 were terminated in the fill layer at 2.1 m depth (Elev. 191.9 to 198.6 m).

SPT N-values recorded in the fill varied from 11 to 54 blows per 0.3 m of penetration, indicating a compact to very dense relative density and a stiff to hard consistency. In borehole JSP-05, an SPT N-value of 4 blows per 0.3 m penetration was recorded at 0.8 to 1.4 m depth, indicating a soft to firm consistency. Measured moisture contents ranged from 4% to 23%.

The results of a grain size distribution analysis of a sample of the sandy silt fill indicated 0% gravel, 31% sand, 64% silt and 5% clay sized particles. The results are presented in Figure C2 in Appendix C.

5.4 Silty Clay

A layer of silty clay was encountered below the pavement structure in Boreholes JSP-06, JSP-07 and JSP-13, and below the fill in Borehole JSP-CULV-01, at depths of 0.9 to 2.6 m (Elev. 195.4 to 205.5 m). Boreholes JSP-06 and JSP-07 were terminated in the clay at depths of 1.8 and 2.1 m (Elev. 197.8 and 196.6 m), respectively. The clay layer was penetrated at depths of 1.8 and 3.0 m (Elev. 204.8 and 195.1 m) in Boreholes JSP-13 and JSP-CULV-01.

SPT N-values recorded in the clay ranged from 14 to 20 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. Measured moisture contents ranged from 7% to 26%.

5.5 Sands and Silts

Layers grading from silty sand, trace gravel, to silt, trace to some sand, then sand, trace to some gravel, were encountered below the silty clay layer in Borehole JSP-13. Borehole JSP-13 was terminated in the sand at 3.7 m depth (Elev. 203.0 m).

SPT N-values recorded in the sands and silts ranged from 14 to 40 blows per 0.3 m of penetration, indicating a compact to dense relative density. Measured moisture contents ranged from 9% to 16%.

5.6 Silty Clay Till

A deposit of silty clay till was encountered below the pavement structure and surficial layers at depths of 1.1 to 3.2 m (Elev. 191.6 to 203.8 m) in all boreholes except Boreholes JSP-02, JSP-04, JSP-06, JSP-07, JSP-09 and JSP-13. The till was penetrated at depths of 3.0 and 6.7 m (Elev. 191.4 and 191.4 m) in Boreholes JSP-05 and JSP-CULV-01, respectively. The remaining boreholes were terminated in the till at depths of 1.9 to 5.9 m (Elev. 189.3 to 202.8 m). The till

contained various amounts of sand (trace sand to sandy), gravel (trace), and occasional shale fragments, cobbles and boulders.

SPT N-values recorded in the clay till ranged from 13 to 65 blows per 0.3 m of penetration, with values up to 50 blows for 0.025 m of penetration, indicating a stiff to hard consistency. In general, the consistency of the till deposit increased with depth. Measured moisture contents ranged from 5% to 18%.

The results of grain size distribution analyses conducted on samples of the till indicated 1 to 2% gravel, 20 to 35% sand, 46 to 55% silt and 13 to 29% clay sized particles. The results are presented in Figure C3 in Appendix C. Atterberg limit analyses measured liquid limits, plastic limits, and plasticity indices of 28 to 34, 15 to 18 and 13 to 17, respectively. These results, which are plotted on Figure C4 in Appendix C, indicate that the samples tested consist of low plastic clay (CL).

5.7 Till/Shale Complex

Till/shale complex was encountered below the clay till in Borehole JSP-CULV-01 at a depth of 6.7 m (Elev. 191.4 m) to the exploration depth of 9.2 m (Elev. 188.9 m). The till/shale was described as sandy silt, some clay, with occasional to numerous shale fragments. SPT 'N' values recorded in the till/shale complex were 60 blows per 0.1 m and 50 blows per 0.075 m of penetration, indicating a very dense relative density. Moisture contents 8% and 9% were measured.

5.8 Shale Bedrock

Shale bedrock was encountered below the till in Borehole JSP-05 at a depth of 3.0 m (Elev. 193.2 m) to the exploration depth of 4.6 m (Elev. 191.6 m). The shale bedrock was described as highly weathered and red in colour. SPT 'N' values of 50 blows per 0.05 to 0.075 m of penetration were recorded in the shale. Moisture contents of 4% and 15% were measured.

5.9 Groundwater Observations

Upon completion of drilling, groundwater was observed at depths of 2.2 to 5.4 (Elev. 190.5 to 200.0 m) in and cave was noted at depths of 2.3 to 5.1 m (Elev. 191.9 to 199.9 m) in Boreholes JSP-01, JSP-05, JSP-08, JSP-10, JSP-CULV-01; the remaining boreholes were open and dry.

Monitoring wells were installed in selected boreholes to monitor groundwater levels. A summary of water levels measured are provided in Table 5.2 below.



Table 5.2: Groundwater Measurements

Borehole/ Monitoring Well No.	Ground Surface Elev. (m)	Mid- Screen Depth (m)	Mid- Screen Elev. (m)	Screened Soil Unit	Ground Water Level (m) Ground Water Elevation (m)		
					Oct. 25- 27, 2023	Nov. 28, 2023	Dec. 21, 2023
JSP-01	194.9	3.9	191.0	Silty Clay Till	2.4 192.4	2.5 192.4	3.0 191.9
JSP-05	196.2	3.9	192.3	Shale Bedrock	3.0 193.3	2.4 193.8	2.2 194.0
JSP-08	198.6	4.4	194.2	Silty Clay Till	2.5 196.1	1.9 196.7	1.9 196.7
JSP-CULV-01	198.1	5.4	192.7	Silty Clay Till	3.4 194.6	3.5 194.6	3.5 194.6
JSP-CULV-02	203.7	3.7	200.0	Silty Clay Till	3.5 200.2	2.0 201.7	1.8 201.9

The groundwater levels are short-term readings and fluctuations of the groundwater levels are to be expected. In particular, the groundwater levels may be at a higher elevation after periods of significant or prolonged precipitation.

6. ENGINEERING DISCUSSION AND RECOMMENDATIONS

This section of the report presents interpretation of the data obtained during the field and laboratory investigation and presents comments and preliminary recommendations for pavement reconstruction and widening as well as potential modification to existing structures and storm drains.

The comments and recommendations presented in this report are based on the subsurface soil and groundwater conditions encountered during the investigation and on the preliminary design concept available at the time of this report. In this regard, the comments and recommendations are considered preliminary and should be reviewed and/or revised, if required, during the detailed design stage. Subsurface conditions may vary between and beyond the borehole locations.

6.1 Pavement Design and Construction

6.1.1 Design Analysis

James Snow Parkway is a major north-south arterial road with a posted speed limit of 60 to 70 km/h. Between Britannia Road and Derry Road, the roadway presently comprises two-lanes 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 paved pathway (multi-use path) along the west side. Proposed improvements include widening of the road to a six-lane urban cross-section,

paved shoulders, priority transit (mixed traffic), and the development of vertical and horizontal alignments.

The traffic volumes, truck percentages and growth rate along James Snow Parkway, provided by HDR, , are presented in Table 6.1. For the analysis, it has been assumed that widening will be completed in 2029.

Table 6.1: James Snow Parkway Traffic Information

Segment	2020 AADT	Truck Volume	Average Annual Growth Rate without Widening	Average Annual Growth with Widening
Between Main Street and Hwy 401 EB Off-Ramp	27,129	5.3%	3.6	7.1
Between Waldie Avenue & Main Street	21,407	4.1%		
250 m North of Derry Road	17,191	3.8%		
Between Clark Boulevard & Derry Road	9,593	3.2%	1.3	8.2
Between Britannia Road & Louis St. Laurent	6,924	4.0%		

The traffic data was used to determine the pavement damage caused by the anticipated traffic volumes over the design life of the pavement. Using axle load equivalency factors, different axle loads and axle groups are converted to a standard axle load known as an Equivalent Single Axle Loads (ESALs). The Design ESALs calculation was completed in accordance with the MTO *Procedures for Estimating Traffic Loads for Pavement Designs*.

A truck factor of 1.87 was employed based on the following characteristic truck categories assessed by MTO for principal arterial highways in the Central Region:

Table 6.2: Truck Factor

MAJOR TRUCK CLASS	TYPICAL TRUCK FACTOR	AVERAGE % TRUCKS
2- and 3-axle trucks	0.47	25
4-axle trucks	2.04	5
5-axle trucks	1.26	45
6 and more axle trucks	4.33	25

Assuming an average truck factor of 1.87, directional factor of 0.5, lane distribution factor of 0.7 to 0.8, and the growth rates indicated above, the number of ESALs during a 20-year design period was computed to range from 4.0 to 19.6 million in the various sections of James Snow Parkway. These results are summarized in Table 6.3 below.

Table 6.3: Projected 20-Year ESALs on James Snow Parkway

Segment	Projected 20-Year ESALs (millions)
Between Main Street and Hwy 401 EB Off-Ramp	19.6
Between Waldie Avenue & Main Street	11.9
250 m North of Derry Road	10.2
Between Clark Boulevard & Derry Road	4.4
Between Britannia Road & Louis St. Laurent	4.0

The pavement design analysis was carried out using the methodology outlined in the 1993 AASHTO “*Guide for the Design of Pavement Structures*”, as modified by the Ministry’s “*Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions*”, and the MTO “*Pavement Design and Rehabilitation Manual*”. The AASHTO procedure determines a required Structural Number that characterizes the structural capacity of the pavement layers for a given set of inputs.

The following design inputs were used in the AASHTO design analysis.

- Design Period = 20 years
- Initial serviceability, (P_i) = 4.5
- Terminal serviceability (P_t) = 2.5
- Reliability level (R) = 90 percent
- Overall standard of deviation (S_o) = 0.44
- Mean soil resilient modulus (MR) = 30 MPa

The subgrade for the pavement structure is expected to consist primarily of discontinuous layers of fill and silty clay, localized silt and sand, and stiff to hard silty clay till.

Based on the range of calculated ESALs, the corridor has been divided into three sections for the purposes of preliminary design: Highway 401 to Main Street, Main Street to Derry Road, and Derry Road to Britannia Road. For the design input parameters and assuming the higher number of ESALs for each section, minimum design structural numbers (SN_{Des}) of 156, 146 and 128 mm are required on James Snow Parkway for the respective sections. The recommended pavement design thicknesses, based on the structural requirements, traffic projections, and subgrade conditions, are presented below.

6.1.2 Recommended Pavement Design

Based on the borehole data, the selected input values, calculated ESALs, and design SN_{Des} , the following preliminary pavement designs are recommended for widening of James Snow Parkway:

Table 6.4: Pavement Design for James Snow Parkway

Component	Thickness (mm)		
	Hwy 401 to Main	Main to Derry	Derry to Britannia
HL1	50	50	50
HDBC (2 lifts)	150	130	100
OPSS Granular A Base	150	150	150
OPSS Granular B Type II Subbase	500	450	450

It is noted that the average total depth of the existing pavement structure encountered in the boreholes was approximately 1.0 m. For the purposes of preliminary design and cost evaluation, the granular subbase thickness in widening areas should be increased from those noted in the table to 650 to 700 mm to match the existing total pavement depth to maintain lateral drainage of the existing granular materials.

Pavement design analysis indicates that strengthening of the existing pavement structure is required between Derry Road and Main Street to support the future traffic demands on the widened roadway. The structural requirements can be achieved by milling 100 mm of the existing asphalt and replacement with 100 mm of new asphalt (50 mm HL1 over 50 mm HDBC). Rehabilitation of the existing pavement structure in the remaining sections may entail milling 50 mm of the existing asphalt surface followed by replacement with a new 50 mm thick layer of HL1 surface course.

Full depth asphalt replacement should be considered for areas of pavement that will be significantly cut by construction activities such as installation of storm sewers and other underground utilities, replacement of culverts, shifting of lane alignments, or relocation of traffic islands, raised medians and curbs. Further, partial depth asphalt replacement may not be cost-effective or efficient to construct in areas requiring profile or grade changes.

The minimum PGAC grade of virgin asphalt cement in the surface and top binder course should be PG 64-28, and minimum PG 58-28 for the lower binder course. Consideration should be given to further upgrading of the PGAC grade to PG 70-28 if rutting has been experienced in other sections of this roadway due to truck traffic. Aggregates for the asphalt mixes should be in accordance with OPSS.MUNI 1003.

Should the Region consider using Superpave asphalt mixes for this project, the recommended HL1 material should be substituted with a Superpave 12.5 FC1 asphalt mix, and the HDBC asphalt material should be replaced with Superpave SP 19. As the maximum 20-year design ESALs for James Snow Parkway was estimated to be 19.6 million, a Traffic Category D designation should be used in preparing all Superpave asphalt mix designs.

All new granular subbase material should consist of OPSS Granular B Type II, while the granular base material should consist of OPSS Granular A. All new granular material should meet the requirements of OPSS.MUNI 1010, and be compacted to 100 percent of the Standard Proctor Maximum Dry Density (SPMDD) within 2 percent of Optimum Moisture Content (OMC). All granular material should be compacted in accordance with the requirements of OPSS.MUNI 501, and should be carried the entire width of the roadway platform to maintain appropriate drainage.

6.1.3 Pavement Subgrade Preparation

Pavement subgrade preparation should include removal of surficial vegetation, topsoil, organic or compressible material. Grading to the new top of subgrade should match or exceed the thickness of the existing pavement to maintain lateral drainage at the top of subgrade. The exposed subgrade should be compacted and proof-rolled with a heavy roller and examined to identify areas of unstable subgrade. Any soft/wet areas identified shall be subexcavated and replaced with approved material within 2% of Optimum Moisture Content (OMC), and compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD).

Bulk fill used to raise the road grade should be constructed as engineered fill, consisting of approved inorganic material, placed in maximum 200 mm thick lifts, within 2% of optimum moisture content, and compacted to at least 98% of SPMDD. Standard side slopes of 2H:1V or flatter should be suitable for embankment construction. Exposed embankment surfaces should be provided with a vegetation cover or otherwise protected against erosion in accordance with OPSS.MUNI 804 and 805.

The top of the compacted subgrade should be graded smooth with a minimum crossfall of 3% towards subdrains or side ditches. Continuity of drainage should be maintained at transitions from existing pavement to new pavement.

6.2 Canadian Pacific Kansas City 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 for approximate distances of 140 and 130 m along the northeast and southwest sides of the road cut, respectively. 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 an elevated grassed area about 6 to 7 m wide 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. Standard toe wall design in accordance with OPSD 3120.100 is considered feasible. Based on the existing borehole information, the relocated toe walls are expected to be founded on very stiff to hard clay till or granular engineered fill. Preliminary design of the toe wall foundations may be carried out assuming factored geotechnical resistances of 300 kPa at ULS and 200 kPa at SLS.

6.3 Bridge over Sixteen Mile Creek

The existing bridge over the Sixteen Mile Creek tributary located approximately 450 m south of Louis St. Laurent Avenue appears to have been recently constructed, to a width capable of carrying up to six lanes of traffic. The bridge is not expected to require modifications.

6.4 Culverts/Storm Drains

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. In addition, 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 either side of the road. Further details of these facilities were not available.

Based on visual observation, modifications to the culverts/drains 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, and therefore support of these facilities is not expected to be an issue.

Prior to placement of the pipe or culvert bedding, the base of the excavation should be maintained in a dry condition, free of loose or disturbed material. The culvert must be placed on a uniformly competent subgrade. Culvert bedding materials, compaction and cover should follow OPSD 802.030 to 803.034, and/or Halton Region specifications.

Backfill at the culverts should consist of non-frost susceptible, free-draining granular material conforming to OPS Granular A or Granular B Type II specifications. Reference should be made to the backfill arrangements stipulated in OPSD 802.031, 802.034, 803.010, 803.030 and 803.031 as applicable.

A hydrogeological assessment to provide recommendations for groundwater control during construction and determine the need for EASR registration or PTTW application was completed concurrently with the geotechnical investigation. A report documenting these findings will be issued under separate cover.

6.5 Frost Cover

The depth of frost penetration at this site is approximately 1.2 m. All spread footings should be provided with a minimum of 1.2 m of earth cover as protection against frost action.

6.6 Soil Aggressiveness

The results of corrosivity testing completed on samples of the overburden soil indicate the soil may be locally corrosive to steel. In this regard, consideration should be given to providing protective measures to resist corrosion if steel elements are used. The effects of road de-icing salts should also be considered when selecting the class of concrete and corrosion mitigation measures.

The measured sulphate concentrations indicate that buried concrete structures will not be subject to sulphate attack in the overburden soils.

6.7 Detailed Geotechnical Investigation

The information presented in this report is provided for preliminary design and planning purposes only. Detailed geotechnical investigation will be required to confirm the subsurface conditions and recommendations. This work should incorporate:

- A detailed pavement investigation including additional boreholes and asphalt cores within the existing roadway pavement and widening areas to further define the subgrade conditions, determine topsoil thickness, and confirm the pavement design recommendations;
- Boreholes to confirm the subsurface conditions at locations where structure modifications or extensions will be required to develop detailed geotechnical recommendations;
- Further assessment of dewatering requirements and the need for a PTTW; and
- Supplemental chemical testing to confirm the requirements for reuse or disposal of excess materials.



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7. 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.

Gregory Stanhope, B.A.Sc.
Geotechnical Engineering-In-Training



Karel Furbacher, P.Eng.
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Review Engineer

Date: February 28, 2025
File: 35477



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