



THURBER ENGINEERING LTD.

Preliminary Geotechnical Investigation Report

**North Halton Coordinated Municipal Class Environmental Assessment Study
Halton Regional Road 25 From 5 Side Road to 10 Side Road
Regional Municipality of Halton, Ontario**

Client: HDR Inc.

Date: February 28, 2025

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EXECUTIVE SUMMARY

This report presents the results of a preliminary geotechnical investigation completed for the proposed improvements and widening of Halton Regional Road 25 between 5 Side Road and 10 Side Road in the Town of Halton Hills as part of the North Halton Coordinated Municipal Class Environmental Assessment (MCEA) Study, initiated by the Regional Municipality of Halton (Halton Region).

Regional Road 25 is a major north-south arterial road in Halton Region. The approximately 3.0 km long section of Regional Road 25 addressed in this study comprises a two-lane rural cross section. The roadway crosses over two tributaries of Sixteen Mile Creek. 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 Regional Road 25 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 localized widening to accommodate turn lanes, for example. The geotechnical investigation comprised a total of twelve boreholes drilled to depths of 2.1 to 11.3 m. Monitoring wells were installed in three boreholes to permit monitoring of the groundwater levels at the site.

The subsurface stratigraphy encountered in the boreholes generally consisted of a pavement structure (asphalt over granular) overlying clay fill and localized alluvial deposits, overlying native cohesive deposits of silty clay to clayey silt and cohesionless deposits of silt to silt and sand underlain by glacial clay till, locally grading to clayey silt till with depth.

Based on the results of the investigation, the following preliminary pavement design is recommended for reconstruction and widening of the roadway:

Component	Thickness
HL1	50 mm
HDBC (2 lifts)	135 mm
OPSS Granular A Base	150 mm
OPSS Granular B Type II Subbase	500 mm

If grades permit, strengthening and rehabilitation of the existing pavement structure involving milling 50 mm of existing asphalt and placing a 120 mm thick asphalt overlay could be considered.

The subgrade soils encountered at the culvert locations are considered capable of supporting culvert extensions and/or replacement box or open footing culverts.



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1. INTRODUCTION

This report presents the results of a preliminary geotechnical investigation completed for the proposed improvements and localized widening of Halton Regional Road 25 between 5 Side Road and 10 Side Road in the Town of Halton Hills as part of the North Halton Coordinated Municipal Class Environmental Assessment (MCEA) Study, initiated by the Regional Municipality of Halton (Halton Region).

Regional Road 25 is a major north-south two-lane 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 Regional Road 25 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 localized 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 are 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 Description

Regional Road 25 is a major north-south two-lane arterial road. The study corridor of this report is an approximately 3.0 km long portion of Regional Road 25 from 5 Side Road to 10 Side Road in the Town of Halton Hills.

The roadway crosses over two tributaries of Sixteen Mile Creek located approximately 1.1 and 1.6 km north of 5 Side Road.

The general surficial 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. 228 at 5 Side Road to near Elev. 245 at No. 10 Side Road. Land use in the general vicinity mainly consists of residential and agricultural land.

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 South Slope physiographic region. The South Slope region is characterized by low-lying, fine-grained, undulating ground moraine and knolls.

Based on *Surficial Geology of Southern Ontario*² the surficial deposits along the study corridor are typically described as clay to silt textured till derived from glaciolacustrine deposits or shale. Modern alluvial deposits of clay, silt, sand, gravel, which may contain organic remains, are mapped at the northern tributary to Sixteen Mile Creek, with a localized organic deposit of peat, muck, and marl shown extending approximately 200 m north of the creek. Ice contact stratified deposits of sand and gravel with minor silt and clay till extend from 10 Side Road to approximately 750 m south of 10 Side Road.

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

Thurber previously completed a geotechnical investigation for construction of a left turn lane along Regional Road 25 within the study corridor:

Geotechnical Investigation, Left Turn Lane at Chudleigh's Driveway, Regional Road 25, Town of Halton Hills, Ontario. Report Submitted to Totten Simms Hubicki Associates. Thurber Engineering Ltd., January 2, 2008, Thurber Ref: 14-145-2-1.

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

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

The investigation consisted of nine boreholes drilled to depths of 2.7 to 2.9 m in the roadway pavement and shoulders for a length of approximately 110 m north and 190 m south of the entrance to Chudleigh's orchard (9528 Regional Road 25). The subsurface stratigraphy encountered in the boreholes generally consisted of a pavement structure or granular shoulders overlying silty clay till south of the entrance and overlying embankment fill and alluvial deposits to the north. The pavement structure consisted of 175 to 225 mm of asphalt overlying 435 to 1125 mm of granular material. The total depth of pavement and granular materials typically ranged from 0.6 to 0.7 m in the cut section south of the entrance and 1.2 to 1.3 m in the fill area to the north.

The Record of Borehole sheets and Borehole Location Plan from the previous investigation are reproduced in Appendix A.

3. INVESTIGATION PROCEDURES

3.1 Borehole Drilling Program

The drilling program was carried out from September 11th to 14th, 2023 and comprised a total of twelve (12) boreholes as summarized in Table 3.1 below. The approximate locations of the boreholes are shown in Drawings 35477-RR25-01 and 35477-RR25-02 in Appendix B. Borehole details are provided in the Record of Borehole sheets included in Appendix C.

Table 3.1: Borehole Details

Borehole No.	Ground Surface Elevation (m)	Termination Depth (m)	Termination Elevation (m)	Monitoring Well
RR25-01	228.9	2.1	226.8	☐
RR25-02	235.1	6.7	228.4	☐
RR25-03	235.1	2.9	232.2	☐
RR25-04	233.8	3.5	230.3	☐
RR25-05	229.8	2.1	227.6	☐
RR25-06	230.7	6.7	224.0	☐
RR25-07	237.7	2.1	235.6	☐
RR25-08	241.8	3.7	238.1	☐
RR25-09	249.1	5.2	243.9	☒
RR25-10	244.2	2.1	242.1	☐
RR25-CULV-01	231.4	9.7	221.7	☒
RR25-CULV-02	226.7	11.3	215.4	☒

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 truck mounted CME55 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 conducted to further assess the strength characteristics of the substrata.

Manually excavated test pits were advanced in the grassed shoulder adjacent to Boreholes RR25-01, RR25-03, RR25-05, RR25-07, and RR25-09 to determine topsoil thicknesses.

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 three boreholes to permit monitoring of the groundwater levels at the site. 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)			
RR25-09	249.1	4.6	244.5	1.5	3.9	245.2
RR25-CULV-01	231.4	8.8	222.6	3.0	7.3	224.1
RR25-CULV-02	226.7	9.9	216.8	3.0	8.4	218.3

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 C and in detail in Appendix D.

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 E.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix C. 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 clay fill and localized alluvial deposits, overlying native

cohesive deposits of silty clay to clayey silt and cohesionless deposits of silt to silt and sand underlain by clay till, locally grading to clayey silt till with depth.

5.1 Pavement Structure

A pavement structure was encountered at the ground surface of the boreholes and typically consisted of 150 to 380 mm of asphalt (average 245 mm) overlying a sand and gravel base. The granular materials thickness ranged from 330 to 1480 mm (average 700 mm). Asphalt was not encountered in Borehole RR25-05 which was advanced through the gravel shoulder.

The results of grain size distribution analyses carried out on samples of the granular fill materials are presented on Figure D1 in Appendix D. The results of two grain size distribution analyses of the granular indicated 36 to 40% gravel, 45 to 47% sand and 15 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 125 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 RR25-01, RR25-03, RR25-05, RR25-07, and RR25-09. The topsoil thicknesses are provided in Table 5.1 below.

Table 5.1: Topsoil Thickness

Borehole	Thickness (mm)
RR25-01	125
RR25-03	150
RR25-05	125
RR25-07	125
RR25-09	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 Boreholes RR25-02, RR25-07, RR25-CULV-01 and RR25-CULV-02 at depths of 0.5 to 0.8 m (Elev. 226.0 to 237.2) and was penetrated at depths of 0.7 to 3.0 m (Elev. 224.5 to 235.9). The fill deposit was described as silty

clay, containing various amounts of sand and gravel (trace to some) with occasional organics and sand pockets.

SPT N-values recorded in the fill ranged from 5 to 24 blows per 0.3 m of penetration, indicating a firm to very stiff consistency. Measured moisture contents ranged from 8% to 19%.

5.4 Silt (Alluvial)

Locally, in Borehole RR25-CULV-02, a 0.8 m thick layer of grey to black alluvial silt was contacted below the fill at a depth of 2.2 m (Elev. 224.5) to a depth of 3.0 m (Elev. 223.7). The silt was sandy with trace to some clay and gravel, and wood fragments. An SPT N-value of 15 blows per 0.3 m of penetration was recorded in this layer, indicating a compact relative density. A moisture content of 32% was determined.

5.5 Gravel (Alluvial)

Locally, in Borehole RR25-CULV-02, a 3.1 m thick layer of gravel was contacted below the silt at a depth of 3.0 m (Elev. 223.7) and was penetrated at 4.1 m (Elev. 222.6). The gravel was sub-rounded to rounded with trace to some sand and trace silt. An SPT N-value of 20 blows per 0.3 m of penetration was recorded in this layer, indicating a compact relative density. Measured moisture contents of 9% and 10% were determined.

5.6 Silty Clay and Clayey Silt

A native cohesive deposit of silty clay and clayey silt was encountered below the pavement structure in Boreholes RR25-01, RR25-03 and RR25-10 and below the fill materials in RR25-07. This stratum was encountered at depths of 0.8 to 1.8 m (Elev. 227.5 to 242.9 m) and was contacted to the termination depth of 2.1 m (Elev. 226.8 to 242.1) in Boreholes RR25-01, RR25-07 and RR25-10. In Borehole RR25-03, the clay layer was penetrated at a depth of 2.1 m (Elev. 223.0). The samples were described as silty clay or clayey silt, containing various amounts of sand (trace to sandy) and trace to some gravel. Locally, in Borehole RR25-07, organic inclusions and lack staining observed in the sample may indicate the deposit is of alluvial origin.

SPT N-values recorded in the clay/silt typically ranged from 5 to 19 blows per 0.3 m of penetration, indicating a consistency of firm to very stiff. Locally an N-value of 57 per 0.3 m of penetration was recorded in Borehole RR25-10, indicating a hard consistency. Measured moisture contents ranged from 10% to 22%.

The results of a grain size distribution analysis carried out on a sample of the clayey silt is presented on Figure D2 in Appendix D. The results indicated 0% gravel, 20% sand, 66% silt and 14% clay sized particles.

Atterberg limit analyses of a sample of the clayey silt measured liquid limits, plastic limits, and plasticity indices of 19, 14 and 5, respectively. These results, which are plotted on Figure D5 in Appendix D, indicate that the sample tested consists of slightly plastic silt (ML-CL).

5.7 Sand to Silt

A deposit of cohesionless soil ranging from sand to silt was encountered below the fill material in Borehole RR25-02, below the gravel in Borehole RR25-CULV-02, and below the pavement structure in Boreholes RR25-04, RR25-06, RR25-08 and RR25-09 at depths ranging from 0.7 to 4.1 m (Elev. 222.6 to 248.4 m). The boreholes were terminated in this deposit at depths of 3.5 to 11.3 m (Elev. 215.4 to 243.9) with the exception of Borehole RR25-08 which penetrated this unit at 3.5 m depth (Elev. 238.3 m).

SPT N-values recorded in the silt and sand were highly variable, ranging from 4 to 23 blows per 0.3 m of penetration in Boreholes RR25-02, RR25-06, RR25-08 and RR24-CULV-02, indicating a loose to compact relative density, and from 25 blows for 0.3 m to 60 blows for 0.1 m of penetration in Boreholes RR25-04 and RR25-09, indicating a compact to very dense condition. Measured moisture contents ranged from 4 to 26%.

The results of grain size distribution analyses carried out on samples of the silt and sand deposits are presented on Figure D3 in Appendix D. The results of the grain size distribution analyses are summarized in Table 5.2 below.

Table 5.2: Grain Size Distribution for Sand to Silt

Soil Particle	Percentage (%)
Gravel	0
Sand	3 to 55
Silt	38 to 86
Clay	5 to 11

5.8 Silty Clay to Clayey Silt Till

A deposit of till was encountered below the granular material and fill in Boreholes RR25-05 and RR25-CULV-01 and below the native silty clay and silt and sand deposits in Boreholes RR25-03 and RR25-08. The till was encountered at depths of 0.6 to 3.5 m (Elev. 228.5 to 238.3). The boreholes were terminated in the till deposits at depths of 2.1 to 9.8 m (Elev. 221.7 to 238.1).

The till deposit was described as silty clay, locally grading to clayey silt at a depth of 5.6 m (Elev. 225.8) in Borehole RR25-CULV-01, with various amounts of sand (trace sand to sandy) and gravel (trace to some). It is noted that auger grinding was recorded at 7.9 m depth (Elev. 223.5) in Borehole RR25-CULV-01 which could indicate the presence of cobbles and/or boulders.

SPT N-values recorded in the clay till ranged from 13 to 32 blows per 0.3 m of penetration, indicating a stiff to hard consistency. N-values of 62 blows per 0.3 m to 50 blows for 75 mm of penetration were recorded in the clayey silt till, indicating a hard consistency. Measured moisture contents ranged from 8% to 14%.

The results of two grain size distribution analyses of the till indicated 2 to 3% gravel, 22 to 24% sand, 50 to 52% silt and 21 to 26% clay sized particles. The results are presented in Figure D4 in Appendix D. Atterberg limit analysis of the samples measured liquid limits, plastic limits, and plasticity indices of 28 to 35, 17 to 19 and 11 to 16, respectively. These results, which are plotted on Figure D6 in Appendix D, indicate that the samples tested consist of low plastic (CL) to intermediate plastic (CI) silty clay.

5.9 Groundwater Observations

Upon completion of drilling, groundwater was observed at depths of 1.7 to 4.4 (Elev. 224.2 to 244.7) in Boreholes RR25-06, RR25-08, RR25-09 and RR25-CULV-02. In addition, the boreholes sidewalls caved at depths of 2.1 to 9.9 m (Elev. 216.8 to 239.7) in Boreholes RR25-06, RR25-08, RR25-CULV-01 and RR25-CULV-02. 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 5.3 below.

Table 5.3: 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. 26, 2023	Nov. 27, 2023	Dec. 21, 2023
RR25-09	249.1	3.9	245.2	Sand	Dry	Dry	Dry
RR25-CULV-01	231.4	7.3	224.1	Clayey Silt Till	4.0 227.5	2.8 228.7	2.9 228.6
RR25-CULV-02	226.7	8.4	218.3	Silty Sand to Sand and Silt	1.6 225.1	1.5 225.2	1.5 225.2

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 design and construction of the two culverts.

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

Regional Road 25 is a major north-south arterial road with a posted speed limit of 80 km/h. The roadway presently has a two-lane rural cross section with gravel shoulders between 5 Side Road and 10 Side Road. Proposed improvements include increased facilities for pedestrians, cyclists, mobility device users and other non-vehicular travel, and improvements to overall operations such as intersections and localized widening to accommodate turn lanes and other facilities.

The traffic volumes, truck percentage and growth rates along Regional Road 25 provided by HDR, are presented in Table 6.1. For the analysis, it has been assumed that improvements will be completed in 2029.

Table 6.1: Regional Road 25 Traffic Information

2020 AADT	Truck Volume	Average Annual Growth Rate without Widening	Average Annual Growth Rate with Widening
11,516	12.1%	1.0	4.5

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 Load (ESAL). 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 Factors

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 0.9, and the growth rates indicated above, the number of ESALs during a 20-year design period was computed to be 14.7 million on Regional Road 25.

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) = 28 MPa

The subgrade for the pavement structure is expected to consist primarily of firm to stiff silty clay fill with localized areas of loose to compact silt and sand or stiff to very stiff silty clay till.

Based on the design input parameters and calculated ESALs, a design structural number (SN_{Des}) of 153 mm is required for the proposed improvements. The recommended pavement design thickness, based on the structural requirements, traffic projections, and subgrade conditions, is 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 design is recommended for widening and full reconstruction of Regional Road 25:

Table 6.3: Pavement Design

Component	Thickness
HL1	50 mm
HDBC (2 lifts)	135 mm
OPSS Granular A Base	150 mm
OPSS Granular B Type II Subbase	500 mm

If grades permit, strengthening and rehabilitation of the existing pavement structure for reuse in the final pavement could be considered. Based on the findings of the preliminary pavement analysis, the rehabilitation strategy could entail a hot mix overlay or partial depth reconstruction. An overlay is expected to be the most cost-effective strategy subject to the limitations discussed below.

Pavement design analysis indicates that the structural requirements can be achieved by milling 50 mm of the existing asphalt surface and placing a 120 mm thick asphalt overlay (50 mm HL1 over 70 mm HDBC). In conjunction with the overlay, remediation of existing distress manifestations such as asphalt cracking is essential to minimize premature pavement deterioration in the form of reflection cracking. This option is not recommended for areas of pavement that presently exhibit frequent cracks or 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, this option may not be cost-effective or efficient to construct in areas requiring profile or grade changes.

Partial depth reconstruction would involve removal of the existing asphalt followed by grading of the exposed granular and the placement of new asphalt. Pavement design analysis indicates that a new asphalt thickness of 215 mm is required over the existing granular materials to support the 20-year traffic loading. To improve grading and enhance drainage, a 100 mm thick layer of new Granular A material is recommended below the new asphalt layer. Removal and replacement of the asphalt layer will enable expedient grade revisions, eliminate reflection cracking, and improve the uniformity of pavement performance across the entire pavement platform. A grade raise of approximately 70 mm would be required.

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 20-year design ESALs for Regional Road 25 was estimated to be 14.7 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 the existing pavement structure and all 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 Culverts

It is anticipated that the existing concrete pipe culvert and concrete box culvert located approximately 1.1 and 1.6 km north of 5 Side Road, respectively, will require replacement and/or extension as part of the roadway widening. Proposed design details for the extension and/or replacement were not provided at the time of this report.

The subsurface stratigraphy encountered in Borehole RR25-CULV-01 drilled at the south culvert consisted of a pavement structure and silty clay embankment fill over very stiff to hard silty clay till encountered at 3.0 m depth (Elev. 228.5). The highest groundwater level was measured at 2.8 m depth (Elev. 228.7). Based on the borehole information, pipe culvert extensions or a

replacement pipe or box culvert should be placed on the very stiff to hard silty clay till below the level of the fill at or below Elev. 228.5. A minimum 300 mm thick layer of Granular A bedding material should be provided under the base of the culvert. If a new open footing culvert is planned, footings should be founded on the native very stiff to hard clay till at or below Elev. 228.5 and designed using factored geotechnical resistances of 350 kPa and 230 kPa at ultimate limit state (ULS) and serviceability limit state (SLS), respectively.

The subsurface stratigraphy encountered in Borehole RR25-CULV-02 drilled at the north culvert location consisted of a pavement structure and silty clay embankment fill overlying silt and gravel alluvial deposits underlain by compact silty sand encountered at 4.1 m depth (Elev. 222.6). The highest recorded groundwater level was measured at 1.5 m depth (Elev. 225.2). Extensions or a replacement concrete box culvert should be founded on the compact sand below the level of the fill and alluvial soils at or below Elev. 222.6. A minimum 300 mm thick layer of Granular A bedding material should be provided under the base of the box culvert. Alternatively, a new open footing culvert may be supported on spread footings founded on compact sand at or below 4.1 m depth (Elev. 222.6) and designed using factored geotechnical resistances of 200 kPa at ULS and 125 kPa at SLS.

In general, excavation for culvert installation will extend through the existing roadway pavement structure, fill materials and alluvial soils to the native silty clay till or silty sand. Cobbles and boulders may be present in till. Use of a hydraulic excavator should be suitable for excavation within these materials.

Prior to placement of the 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.

The installation of the culverts should be carried out in the dry. In general, excavation for the south culvert is expected to remain within the silty clay embankment fill and till. Considering the consistency and relatively low permeability of the clay soils, dewatering of the excavation should be feasible using sumps and pumping from within the excavation. At the north culvert, dewatering to lower the groundwater level below the base of the excavation will be required, potentially involving high-capacity pumps, well points and/or sheet pile cofferdams.

Temporary stream diversion measures such as impervious dikes should be provided to divert surface water runoff and stream flow away from excavations during construction.

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.

6.3 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.

6.4 Lateral Earth Pressures

Backfill to the culverts and wing walls should consist of non-frost susceptible, free-draining granular material conforming to OPS Granular A or Granular B Type II specifications.

The lateral earth pressures acting on the culvert walls, assuming full drainage, may be calculated from the following expression:

$$p_h = K (\gamma h + q)$$

Where:

- p_h = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
- γ = unit weight of retained soil (see table below)
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

Table 6.4 lists unfactored parameters for design purposes, assuming an essentially level ground surface around the culverts.

Table 6.4: Lateral Earth Pressure Parameters

Retained Material	Unit Weight (kN/m ³)	Friction Angle (degrees)	Earth Pressure Coefficient		
			Active (K_a)	At-rest (K_o)	Passive (K_p)
Granular A or B Type II	22.8	35	0.27	0.43	3.7
Granular B Type I	21.2	32	0.31	0.47	3.3

If lateral movement is not permissible and/or the walls are restrained from lateral yielding, the

at-rest earth pressure coefficient, K_o , should be used. If the wall design allows lateral yielding (non-rigid structure), the active earth pressure coefficient, K_a , may be used.

The earth pressure coefficients in the table above do not include potential compaction effects that must be included in the design. Compaction effects should be considered as per the CHBDC.

Design of the structures must incorporate measures such as weepholes to permit drainage of the backfill and avoid potential build-up of hydrostatic pressures behind the walls.

6.5 Soil Aggressiveness

The results of corrosivity testing completed on samples of the overburden soil indicate the soils 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.

6.6 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 for Detailed Design. This work should incorporate:

- A detailed pavement investigation including additional boreholes within the existing roadway pavement and widening areas to further define the subgrade conditions, determine topsoil thickness, and confirm the pavement design recommendations;
- Boreholes within the envelope of all structure foundation units to confirm the subsurface conditions at the structure locations and develop detailed geotechnical recommendations for design and construction of the culvert foundations;
- Further assessment of dewatering requirements and the need for a PTTW; and
- Supplemental chemical testing to confirm the requirements for reuse or disposal of excavated material.



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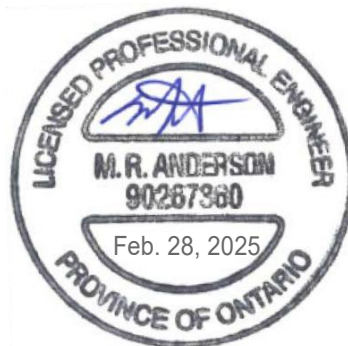
7. SIGNATURES/CLOSURE

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.
Senior Geotechnical Engineer



Murray Anderson, M.Eng., P.Eng.
Review Engineer

Date: February 28, 2025

File: 35477-20



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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b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

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d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

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