ENVISION
CONSULTANTS LTD

HYDROGEOLOGICAL ASSESSMENT

Downtown Capitol Implementation Plan

Project #: 22-0120
Prepared for: City of Guelph
Date: January 5, 2024

Report Version: 02

6415 Northwest Drive
Units 37-40
Mississauga, ON L4V 1X1

envisionconsultants.ca

January 5, 2024

City of Guelph
1 Carden Street
Guelph, Ontario
N1H 3A1

Attention: Andrew Miller, P.Eng., PMP, Project Engineer, Design and Construction, Infrastructure, Development & Enterprise Services, City of Guelph

SUBJECT: HYDROGEOLOGICAL ASSESSMENT, DOWNTOWN CAPITOL IMPLEMENTATION PLAN, CITY OF GUELPH, ONTARIO

EnVision Consultants Ltd is pleased to present the enclosed Hydrogeological Assessment report for the above-noted property. The report describes the interpreted hydrogeological conditions based on our assessment and provides conclusions for your consideration.

We thank you utilizing EnVision for this assignment. If there are any questions regarding the enclosed report, please do not hesitate to contact us.

Yours sincerely,

Rob Byers, P.Geo.,
Senior Hydrogeologist
rbyers@envisionconsultants.ca

QUALITY MANAGEMENT

ISSUE	FIRST ISSUE	REVISION 1	REVISION 2
PROJECT NUMBER	22-0120	22-0120	
PROJECT REFERENCE	Hydrogeological Assessment, Downtown Capitol Implementation Plan	Hydrogeological Assessment, Downtown Capitol Implementation Plan	
VERSION NO.	01	02	
REMARKS	Draft Report	Final Report	
PREPARED BY	Sam Harding	Sam Harding	
SIGNATURE			
REVIEWED BY	Rob Byers	Rob Byers	
SIGNATURE			
DATE	May 19, 2023	January 5, 2024	

The original of this digital file will be kept by EnVision for a period of not less than 10 years. As the digital file transmitted to the intended recipient is no longer under the control of EnVision, its integrity cannot be assured. As such, EnVision does not guarantee any modifications made to this digital file subsequent to its transmission to the intended recipient.

TABLE OF CONTENTS

LIST OF FIGURES (AVAILABLE UPON REQUEST*)

LIST OF APPENDICES (AVAILABLE UPON REQUEST*)

*PLEASE CONTACT ENGINEERING@GUELPH.CA

1. INTRODUCTION

EnVision Consultants Ltd (EnVision) was retained by the City of Guelph (the 'Client') to conduct a hydrogeological assessment within the downtown area, in association with the Downtown Capitol Implementation Plan (the 'Site'). These services have been requested in support of the Downtown Infrastructure Revitalization Plan, distributed over approximately 7.0km of roadways at various locations in the City of Guelph downtown. A Site and Study Area Location plan is included as Figure 1.

Geotechnical and environmental studies were also carried out at the Site, which are presented in separate reports.

This report has been prepared for the City of Guelph and its civil engineering designer (R.V. Anderson Associates). Third party use of this report without EnVision consent is prohibited. The limitation conditions presented in this report form an integral part of the report and must be considered in conjunction with this report.

1.1. OBJECTIVES AND SCOPE OF WORK

The objective of this hydrogeological investigation is to characterize the geological and hydrogeological conditions at the Site and Study Area to:

- Review soil and groundwater data to understand any constraints to the project goals;
- Estimate the need for groundwater control during construction;
- Assess potential dewatering rates to determine the required permitting associated with water takings as per Ontario Water Resources Act;
- Assess any short- or long-term impacts on groundwater resources from the construction activities;
- Review mitigation measures to protect groundwater resources during the construction work;
- Determine management options for the handling of any groundwater collected and discharged during construction;
- Recommend a monitoring program for construction dewatering and discharge;

1.2. POLICY AND REGULATORY OVERVIEW

A review of the Source Water Protection Policy areas indicates that the site is located within the Grand River Source Water Protection Area. The Site is located in wellhead protection zone B with a score of 10 (Ministry of the Environment, Conservation and Parks, 2021). The Site does not fall under any intake protection zones and is not considered a highly vulnerable aquifer.

2. REGIONAL SETTING

2.1. GEOLOGY

2.1.1. Overburden Geology

Based on a review of the public geological mapping of the Study Area, the surficial material consists of glaciofluvial deposits of gravel along with sandy silt to silty sand textured till (Ministry of Northern Development, Mines and Forestry, 2013). Figure 2 highlights the surficial geology of the Study Area.

2.1.2. Bedrock Geology

Bedrock mapping of the Study Area identifies the bedrock as the Guelph formation; a mix of shale, siltstone, dolostone and sandstone (Sharpe, 1980). The depth to bedrock is expected to be between 3 and 13 meters below the ground surface of the Site.

2.2. HYDROGEOLOGICAL SETTING

2.2.1. Study Area Review of MECP Water Well Records

EnVision reviewed the online MECP Water Well Record (Ministry of Environment, Conservation and Parks, 2018) database to determine the number and reported use of water wells present within the Study Area.

The MECP WWR database indicated that there are one-hundred and forty-five (145) water wells in the Study Area. Of the well records returned in the search, nine of them were classified as water supply wells for domestic or commercial use. All the other well records were reported as monitoring wells, test holes, unclassified or abandoned. The results of this search have been plotted on Figure 3 and tabulated in Appendix A.

Water supply records were reviewed for the nine water supply wells in the area. All the wells have been constructed in bedrock using open hole construction with the depths to the top of the hole ranging from 3m-63.4m. The downtown area of Guelph is 100% serviced by municipal water and sewer utilities. The use of private groundwater wells for the supply of drinking or process water is considered unlikely.

2.2.2. Regional Hydrostratigraphy

The geological and hydrogeological conditions across the downtown area have been mapped and described previously by Matrix Solutions Inc. (2017), Golder and Associated (2011), Gamsby and Mannerow (1993), et. al. Matrix has also completed extensive groundwater modeling work as part of the Tier Three Risk Assessment, which has described the regional hydrostratigraphy as a sequence consisting of the following major units:

Upper Sand/Gravel Aquifer – shallow unconfined aquifer comprised of outwash deposits of moderate permeability. The unit ranges in thickness (where present) from 1 to 70 meters. Across the site and study area, this thickness is not expected to be beyond 5 m.

Lower Overburden – a combination of glacial till units that are of low to moderate permeability which overlies the bedrock contact.

Bedrock Contact Zone – the upper weathered or fractured portion of the Guelph Formation bedrock which can be hydraulically connected to the overburden outwash sand and gravel aquifer that has been historically used as a source of domestic water supply.

Guelph Formation – medium to thickly bedded dolostones that represent an important aquifer for the Cambridge and Guelph areas. Bedrock of the Guelph Formation outcrops along the Speed River.

Lower lying older units have been described in the previous modelling works but are not significant to this project scope.

2.2.3. Source Water Protection Policy Areas

A review of the Source Water Protection Policy areas indicates that the site is located within the Grand River Source Water Protection Area. The Site is located in wellhead protection zone B with a score of 10 (Ministry of the Environment, Conservation and Parks, 2021). The Site does not fall under any intake protection zones and is not considered a highly vulnerable aquifer. All activities involving water taking or discharge must abide by the Grand River protection plan.

2.2.4. Permit to Take Water and Construction Dewatering EASR Search

The MECP maintains an online database and GIS mapping service that contains all registered Permit to Take Water and Construction Dewatering EASR filings. A review of this service indicates that the following activities are currently reported for the Study Area.

- Permit to Take Water – Expired – For a pumping test on well PW1 Farquhar Street, Guelph Ontario (Total permitted volume of 1,080,000 L/day)
- Permit to Take Water – Expired – For construction dewatering Wellington Street East, Guelph Ontario (Total permitted volume of 396,000 L/day)
- Water Taking – Construction EASR – Active - for 55 Baker Street, Guelph (total permitted volume of 400,000 L/day)
- Water Taking – Construction EASR – Active - for 55 Arthur Steet South, Guelph (total permitted volume of 400,000 L/day)

Based on this review, the surrounding areas, particularly for locations along the river, have required temporary dewatering permits for construction activities.

3. SITE SETTING

3.1. TOPOGRAPHY AND DRAINAGE

Based on elevation survey completed by Envision at each of the borehole locations, the existing Site features a minor gradient dipping towards the north-northeast in the direction of the Speed River. The gradient slopes from Dublin Street at an elevation of approximately 338.2 meters above sea level (m ASL) to its lowest point of 322.6 m ASL at the west end of Cardigan Street.

The existing grounds are covered by impermeable asphalt or concrete and drainage of stormwater is controlled by the topography and directed to City of Guelph catch basins for discharge to nearby storm sewers, with ultimate release into the Speed River.

3.2. SURFACE WATER FEATURES

The northern border of the Site runs along the Speed River. The Speed River flows from Orton, Ontario south through Guelph where it meets the Eramosa River. The river then flows further south until uniting with the Grand River in north-west Cambridge approximately 20km away. Works associated with this program are not anticipated to include any tunnelling efforts below the river.

There are no mapped evaluated or unevaluated wetlands within the study area.

4. SUBSURFACE CONDITIONS

Geotechnical borehole drilling was completed over two distinct time intervals; 2022 and 2023. In 2022 the field investigation consisted of drilling a total of thirty-one (31) boreholes (BH22-1 through BH22-31) along various roads in the City of Guelph downtown, to depths varying from 1.9m to 5.2m below the existing ground surface. Ten (10) monitoring wells of 50mm diameter were installed in boreholes BH22-2, BH22-4, BH22-9, BH22-14, BH22-15, BH22-17, BH22-19, BH22-21, BH22-24, and BH22-26. In 2023, an additional five (5) boreholes were advanced, identified as BH23-1 to BH23-5. Three (3) of the boreholes, (BH23-1, BH23-3, and BH23-4) were instrumented as long-term groundwater monitoring wells. The 2023 series of boreholes reached depths from 2.9m to 7.6m below the ground.

The approximate locations of the boreholes/monitoring wells are presented on Figure 4 and record of borehole sheets are attached in Appendix B.

The subsurface conditions in the boreholes generally consisted of pavement structure overlying fill material consisting of sand and gravel, gravelly sand, sand, silty sand, clayey silt and silty clay. Native soils consisting of silty clay, clayey silt, silty clay till, clayey silt till, sandy silt till to silty sand till and cohesionless deposits of sand and gravel, sandy gravel, gravelly sand, sand, silty sand and sandy silt were encountered in the boreholes. Cobbles and boulders are expected in these deposits.

Bedrock of the Guelph Formation was encountered in six (6) boreholes (BH23-1, BH23-4, BH23-5, BH22-12, BH22-21, and BH22-30) at depths ranging from 2.3m to 3.8m below existing ground surface, corresponding to Elev. 307.3m to 322.5m. The subsurface conditions at each road/street are summarized in the following paragraphs.

4.1. DUBLIN STREET

Boreholes BH22-1 and BH22-2 were drilled along Dublin Street to depths of 2.1 and 4.4m below the existing ground surface. Borehole 22-2 was completed as a monitoring well.

4.1.1. Soil Conditions

Below pavement structure in the boreholes, fill material consisting of gravelly sand and sandy silt to silty sand was encountered which extended to depths of 1.2m to 1.5m below the existing ground surface. Fill was generally present in a compact state based on measured SPT ‘N’ values ranging from 14 to 20 blows per 300 mm of penetration.

SANDY SILT TILL

Below fill material in BH22-1, glacial till deposit of sandy silt were encountered which extended to the termination depth of 2.1m. Sandy silt till was found to be in a compact state based on measured SPT ‘N’ values of 14 to 29 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

COHESIONLESS DEPOSITS OF GRAVELLY SAND/SILTY SAND

Below fill material in BH22-2, cohesionless deposits of gravelly sand and sandy silt were encountered which extended to the termination depth of 4.2m. These cohesionless deposits were found to be in a dense state based on measured SPT ‘N’ values of 37 to 47 blows per 300 mm of penetration.

4.2. NORTHHUMBERLAND STREET

Borehole BH22-3 was drilled at Northumberland Street to a depth of 1.9m below the existing ground surface.

4.2.1. Soil Conditions

Below pavement structure in the borehole, fill material consisting of sandy silt was encountered which extended to 0.8m below the existing ground surface. Fill was present in a dense state based on measured SPT ‘N’ values ranging from 38 blows per 300 mm of penetration.

SILTY SAND TILL

Below fill material in BH22-3, glacial till deposit of silty sand was encountered which extended to the termination depth of 1.9m. Silty sand till was found to be in a very dense state based on measured SPT ‘N’ values of more than 50 blows per 300 mm of penetration.

Cobbles and boulders are expected in these cohesionless deposits.

4.3. NORFOLK STREET

Six (6) boreholes (BH22-4 through BH22-9) were drilled along Norfolk Street to depths varying from 2.1 to 4.4m below the existing ground surface. Monitoring wells of 50mm diameter were installed in two (2) boreholes (BH22-4 and BH22-9).

4.3.1. Soil Conditions

Below pavement structure in BH22-4 to BH22-8, fill material consisting of silty sand to sandy silt was encountered which extended to depths varying from 2.1m to 2.3m below the existing ground surface. Boreholes BH22-5, BH22-7 and BH22-8 were terminated in fill material at a depth of 2.1m. Traces of organics were present in BH22-6 at a depth of 1.6m. Fill was present in a loose to compact state based on measured SPT 'N' values ranging from 6 to 24 blows per 300 mm of penetration.

COHESIONLESS DEPOSITS OF SAND/SANDY SILT

Below fill material in Boreholes BH22-6 and BH22-9, cohesionless deposits of sand, sand with gravel and sandy silt were encountered which extended to depths of 3.1m to 4.4m below the existing ground surface. These deposits were found to be in a compact to very dense state based on measured SPT 'N' values of 21 to more than 50 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

SANDY SILT TILL

Below fill material in BH22-4 and below sandy silt in BH22-9, glacial till deposits of sandy silt were encountered which extended to a depth of 3.6m in BH22-8 and to the termination depth of 3.9m in BH22-9. Sandy silt till was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

SILTY CLAY

Below sandy silt till in BH22-4, cohesive deposits of silty clay were encountered which extended to the termination depth of the borehole. Silty clay was found to be in a hard consistency based on measured SPT 'N' value of 47 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 13%.

4.4. CARDIGAN STREET

Two (2) boreholes (BH22-10 and BH22-11) were drilled along Cardigan Street to depths of 2.1m below the existing ground surface.

4.4.1. Soil Conditions

Below pavement structure in the boreholes (BH22-10 and BH22-11), fill material consisting of silty sand and sandy silt was encountered which extended to the termination depths varying from 1.5m to 2.1m below the existing ground surface. Fill was present in a very loose to dense state based on measured SPT 'N' values ranging from 2 to 36 blows per 300 mm of penetration.

SILTY SAND TILL

Below fill material in BH22-11, glacial till deposit of silty sand was encountered which extended to the termination depth of 2.1m. Silty sand till was found to be in a dense state based on measured SPT 'N' values of 41 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

4.5. ERAMOSA ROAD

One (1) borehole (BH22-12) was drilled along Eramosa Road to a depth of 6.4m below the existing ground surface.

4.5.1. Soil Conditions

Below pavement structure in the borehole (BH22-12), fill material consisting of silty sand, sandy silt and silty clay was encountered which extended to 3.1m below the existing ground surface. Fill was present in a very loose to compact state or in a stiff consistency based on measured SPT 'N' values ranging from 2 to 9 blows per 300 mm of penetration.

BEDROCK OF GUELPH FORMATION

Below the fill material in BH22-12, bedrock of the Guelph Formation was encountered at a depth of 3.1m corresponding to Elev. 319.7m.

Because of the method of drilling and sampling, the actual surface elevations of the bedrock may be different than indicated on the borehole logs. With augering or setting of HW casing into rock, the auger/casing may penetrate some of the more weathered bedrock and the coring may therefore begin below the bedrock surface. As such, the inferred bedrock surface level should not be considered accurate to better than ±0.5m.

4.6. WYNDHAM STREET

Three (3) boreholes (BH22-13 through BH22-15) were drilled along Wyndham Street to depths varying from 2.1 to 4.4m below the existing ground surface. A monitoring well of 50mm diameter was installed in one borehole (BH22-14).

4.6.1. Soil Conditions

Below pavement structure in the boreholes (BH22-13 to BH22-15), fill material consisting of silty sand to sand was encountered which extended to depths varying from 1.4m to 2.6m below the existing ground surface. Fill was present in a loose to very dense state based on measured SPT 'N' values ranging from 9 to more than 50 blows per 300 mm of penetration.

SANDY SILT TILL / SILTY SAND TILL

Below fill material in Boreholes BH22-13 and BH22-15, native deposits of sandy silt till to silty sand till were encountered which extended to the termination depths of 2.1m to 4.4m below the existing ground surface. These deposits were found to be in a compact to very dense state based on measured SPT ‘N’ value of 13 to more than 50 blows per 300 mm of penetration. The moisture content in the tested samples was found to range from 5% to 13%. Cobbles and boulders are expected in the glacial till deposits.

COHESIONLESS DEPOSITS OF SAND / SILTY SAND

Below fill material in Borehole BH22-14, cohesionless deposits of sand and silty sand were encountered which extended to the termination depth of 4.2m below the existing ground surface. This deposit was found to be in a compact to dense state based on measured SPT ‘N’ value of 19 to 44 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

4.7. MACDONELL STREET

Three (3) boreholes (BH22-16, BH22-17 and BH22-30) were drilled along MacDonell Street to depth of 2.1m and 6.5m below the existing ground surface. A monitoring well of 50mm diameter was installed in one (1) borehole (BH22-17).

4.7.1. Soil Conditions

Below pavement structure in the boreholes (BH22-16, BH22-17 and BH22-30), fill material consisting of sand and gravel, gravelly sand and silty sand was encountered which extended to depths varying from 1.5m to 2.3m below the existing ground surface. Fill was present in a loose to dense state based on measured SPT ‘N’ values ranging from 7 to 32 blows per 300 mm of penetration.

SANDY SILT TILL

Below fill material in Borehole BH22-17, a glacial till deposit of sandy silt till was encountered which extended to 3.8m below the existing ground surface. This deposit was found to be in a dense to very dense state based on measured SPT ‘N’ values of 32 to more than 50 blows per 300 mm of penetration. The moisture content in the tested samples ranged from 5 to 8%.

CLAYEY SILT TILL

Below sandy silt till in Borehole BH22-17, glacial till deposits of clayey silt were encountered which extended to the termination depth of 4.4m below the existing ground surface. These deposits were found to be in a hard consistency based on measured SPT ‘N’ value of 42 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 11%.

BEDROCK OF GUELPH FORMATION

Below the fill material in BH22-30, bedrock of the Guelph Formation was encountered at a depth of 2.3m corresponding to Elev. 322.5m.

4.8. WOOLWICH STREET

Five (5) boreholes (BH22-18 to BH22-22) were drilled along Woolwich Street to depths of 2.1m to 6.3m below the existing ground surface. Monitoring wells of 50mm diameter were installed in two (2) boreholes BH22-19 and BH22-21.

4.8.1. Soil Conditions

Below pavement structure in the boreholes (BH22-18 to BH22-22), fill material consisting of gravelly sand, gravelly silty sand, silty sand and clayey silt was encountered which extended to the termination depths of 2.1m in BH22-18 and BH22-20 and to depths of 1.5m to 1.8m in BH22-19, Bh22-21 and BH22-22. Fill was present in a loose to dense state or in a firm consistency based on measured SPT 'N' values ranging from 4 to 37 blows per 300 mm of penetration.

SILTY CLAY TILL

Below fill material in Borehole BH22-21, a glacial till deposit of silty clay was encountered which extended to 2.6m below the existing ground surface. This deposit was found to be in a firm to hard consistency based on measured SPT 'N' values of 6 to more than 50 blows per 300 mm of penetration. The moisture contents in the tested samples were found to vary from 12% to 18%.

COHESIONLESS DEPOSITS OF SILTY SAND

Below fill material in Borehole BH22-19, a cohesionless deposit of silty sand was encountered which extended to 3.8m below the existing ground surface. This deposit was found to be in a compact state based on measured SPT 'N' value of 18 to 26 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

SANDY SILT TILL

Below silty sand in BH22-19 and below fill material in BH22-2, glacial till deposit of sandy silt was encountered which extended to the termination depths of 2.1m to 4.4m of the boreholes. Sandy silt till was found to be in a compact state based on measured SPT 'N' values of 15 to 24 blows per 300 mm of penetration. The moisture content in the tested samples was found to be 9 and 10%.

BEDROCK OF GUELPH FORMATION

Below silty clay till in BH22-21, bedrock of the Guelph Formation was encountered at a depth of 2.6m corresponding to Elev. 321.4m.

4.9. NORWICH STREET EAST

One (1) borehole (BH22-23) was drilled along Norwich Street East to a depth of 4.1m below the existing ground surface.

4.9.1. Soil Conditions

Below pavement structure in the borehole (BH22-23), fill material consisting of silty sand and silty clay was encountered which extended to 2.6m below the existing ground surface. Fill was present in a very loose to compact state or in a firm consistency based on measured SPT 'N' values ranging from 7 to 17 blows per 300 mm of penetration.

SANDY SILT TILL

Below fill material in BH22-23, a glacial till deposit of sandy silt was encountered which extended to the termination depth of 4.1m of the boreholes. Sandy silt till was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration. The moisture content in the tested samples was found to be 8 to 9%.

4.10. SUFFOLK STREET EAST

One (1) borehole (BH22-24) was drilled along Suffolk Street East to a depth of 4.2m below the existing ground surface. A monitoring well of 50mm diameter was installed within the borehole.

4.10.1. Soil Conditions

Below pavement structure in the borehole (BH22-24), fill material consisting of sand and gravel, silty sand and clayey silt was encountered which extended to 2.6m below the existing ground surface. Fill was present in a loose to compact state based on measured SPT 'N' values ranging from 6 to 28 blows per 300 mm of penetration.

SANDY SILT TILL/SILTY SAND TILL

Below fill material in BH22-24, a glacial till deposit of sandy silt to silty sand were encountered which extended to 3.8m below the existing ground surface. Sandy silt till was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 10%.

COHESIONLESS DEPOSIT OF SAND

Below the glacial till deposit in BH22-24, a cohesionless deposit of sand was encountered which was found to extend to the termination depth of 4.2m of the borehole. This deposit was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration.

4.11. YORMOUTH STREET

One (1) borehole (BH22-25) was drilled along Yarmouth Street to a depth of 2.1m below the existing ground surface.

4.11.1. Soil Conditions

Below pavement structure in the borehole (BH22-25), fill material consisting of sandy silt was encountered which extended to 1.5m. Fill was present in a very loose to a loose state based on measured SPT 'N' values ranging from 2 to 7 blows per 300 mm of penetration.

CLAYEY SILT TILL

Below fill material in BH22-25, a glacial till deposit of clayey silt was encountered which extended to 2.1m. Clayey silt till was found to be in a stiff consistency based on measured SPT 'N' value of 12 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 11%.

4.12. QUEBEC STREET

Two (2) boreholes (BH22-26 and BH22-27) were drilled along Quebec Street to depths of 5.2m and 2.1m below the existing ground surface. A monitoring well of 50mm diameter was installed in one (1) borehole (BH22-26).

4.12.1. Soil Conditions

Below pavement structure in the boreholes (BH22-26 and BH22-27), fill material consisting of silty sand was encountered which extended to the termination depth of 2.1m in BH22-27 and to 2.3m in BH22-26. Fill was present in a very loose to compact state based on measured SPT 'N' values ranging from 1 to 10 blows per 300 mm of penetration.

SILTY SAND TILL

Below fill material in BH22-26, glacial till deposits of sandy silt to silty sand were encountered which extended to the termination depth of 5.2m below the existing ground surface. Till was found to be in a compact to very dense state based on measured SPT 'N' values of 18 to more than 50 blows per 300 mm of penetration. The moisture content in the tested sample was found to range from 6 to 10%.

4.13. CORK STREET

Two (2) boreholes (BH22-28 and BH22-29) were drilled along Cork Street to depths of 2.1m and 4.4m below the existing ground surface.

4.13.1. Soil Conditions

Below pavement structure in the boreholes (BH22-28 and BH22-29), fill material consisting of gravelly sand, sand and sandy silt was encountered which extended to the termination depth of the boreholes. Fill was present in a very loose to compact state based on measured SPT 'N' values ranging from 2 to 13 blows per 300 mm of penetration.

SILTY SAND TILL

Below fill material in BH22-29, glacial till deposits of silty sand were encountered which extended to the termination depth of 4.4m below the existing ground surface. Till was found to be in a compact state based on measured SPT 'N' values of 15 to 26 blows per 300 mm of penetration.

4.14. DOUGLAS STREET

One (1) borehole (BH22-31) was drilled along Douglas Street to a depth of 2.1m below the existing ground surface.

4.14.1. Soil Conditions

Below pavement structure in the borehole (BH22-25), fill material consisting of silty sand was encountered which extended to 1m below the existing ground surface. Fill was present in a compact state based on measured SPT 'N' value of 13 blows per 300 mm of penetration.

SILTY SAND TILL

Below fill material in BH22-31, glacial till deposit of silty sand was encountered which extended to the termination depth of 2.1m below the existing ground surface. Till was found to be in a dense state based on measured SPT 'N' value of 44 blows per 300 mm of penetration.

4.15. WELLINGTON STREET

Three (3) boreholes (BH23-1 to BH23-3) were drilled along Wellington Street to depths ranging from 2.9m to 6.2m below the existing ground surface.

4.15.1. Soil Conditions

Below pavement structure in the boreholes (BH23-1 to BH23-3), fill material consisting of sand and gravel and silty sand was encountered which extended to depths ranging from 0.8m to 3.1m below the existing grade. Traces of cinders/slag were also present with fill material. Fill was present in a very loose to very dense state based on measured SPT 'N' values ranging from 3 to 52 blows per 300 mm of penetration.

COHESIONLESS DEPOSITS OF SAND AND GRAVEL/SANDY GRAVEL

Below fill material in all boreholes BH23-1 to BH23-3, cohesionless deposits of sand and gravel/sandy gravel were encountered which extended to bedrock surface in BH23-1 and to the termination depths of 2.9m to 4.3m in BH23-2 and BH23-3. These cohesionless deposits were found to be in a compact to very dense state based on measured SPT 'N' values of 12 to over 50 blows per 300 mm of penetration.

BEDROCK OF GUELPH FORMATION

Below sandy gravel in BH23-1, bedrock of the Guelph Formation was encountered at a depth of 2.7m corresponding to Elev. 307.3m.

4.16. WYNDHAM STREET

In August 2023, two (2) more boreholes (BH23-4 and BH23-5) were drilled along Wyndham Street east of the 22 series boreholes. The borehole depths ranged from 6.2m to 7.6m below the existing ground surface. A 50mm diameter well was installed in borehole (BH23-4).

4.16.1. Soil Conditions

Below pavement structure in the boreholes (BH23-4 and BH23-5), fill material consisting of sand and gravel and silty sand with gravel was encountered which extended to a depth of 1.5m below the existing grade. Fill was present in a compact to dense state based on measured SPT 'N' values ranging from 16 to 44 blows per 300 mm of penetration.

SANDY SILT TILL

Below fill material in the boreholes BH23-4 and BH23-5, glacial till deposits of sandy silt were encountered underlain by the bedrock of Guelph Formation. Sandy silt till was found to be in a loose to very dense state based on measured SPT 'N' values of 6 to over 50 blows per 300 mm of penetration.

BEDROCK OF GUELPH FORMATION

Below sandy silt till in BH23-4 and BH23-5, bedrock of the Guelph Formation was encountered at a depth of 2.3m and 3.8m corresponding to Elev. 312.2m and 316.2m.

4.17. BEDROCK OF GUELPH FORMATION

Bedrock of Guelph Formation, consisting of dolomite or dolomitic limestone was encountered at the location of boreholes BH23-1, BH23-4, BH23-5, BH22-12, BH22-21 and BH22-30 at depths varying from 2.3m to 3.8m below the existing ground surface, corresponding to Elev. 307.3m to 322.5m.

Because of the method of drilling and sampling, the actual surface elevations of the bedrock may be different than as indicated on the borehole log. With augering or setting of HW casing into rock, the auger/casing may penetrate some of the more weathered rock and the coring may therefore begin below the bedrock surface. As such, the inferred bedrock surface level should not be considered accurate to better than ±0.5m

Visual examination of the recovered rock cores indicates that the Guelph Formation typically consists of moderately weathered to slightly weathered, light brown to white, weak to strong dolostone or dolomitic limestone.

TOTAL CORE RECOVERY (TCR)

The total core recovery indicates the total length of rock core recovered expressed as a percentage of the actual length of the core run. The total core recovery ranged from 58% to 100%, with an average value of 86%.

SOLID CORE RECOVERY (SCR)

The solid core recovery is the total length of solid, full diameter rock core that was recovered, expressed as a percentage of the length of the core run. Solid core recovery generally ranged from 0% to 100% with an average value of 63% and appears to generally improve with depth.

ROCK QUALITY DESIGNATION (RQD)

The rock quality designation index is obtained by measuring the total length of recovered rock core pieces which are longer than 100mm and expressing their sum total length as a percentage of the length of the core run. RQD is a function of the frequency of joints, bedding plane partings and fractures in the rock cores. On the basis of the recorded RQD values which range between 0 and 100% with an average value of 54%, the rock quality is estimated to be “very poor” to “excellent” quality.

UNCONFINED COMPRESSIVE STRENGTH (UCS) AND POINT LOAD INDEX STRENGTH

To determine the unconfined compressive strength (UCS) of the intact rock, a total of three (3) rock samples of suitable length core were selected for uniaxial compressive strength testing.

The unconfined compressive strength (UCS) of the tested samples of dolomite ranged from 54.8MPa to 55.1MPa. Based on the above mentioned limited number of unconfined compressive strength test results, the dolomite rock samples can be classified “strong” rock under ISRM strength convention.

Point load index strength tests were performed on twelve (12) rock samples of the Guelph Formation. We have utilized the empirical approximate relationship between unconfined compressive strength (UCS) and point load index strength as follows:

UCS [MPa] ≈ 24.0 $I_{S(50)}$

where IS(50) is the point load index strength in MPa for a 50 mm equivalent diameter core. This is an approximate correlation after Franklin and Hoek, which may overestimate the UCS value.

For the Guelph Formation samples tested, the equivalent Point-Load derived unconfined compressive strength of the samples was inferred to range from 28 to 59MPa in the axial direction and 23 to 93MPa in the diametral direction. These values are indicative of generally “medium strong ” to “strong” rock under ISRM strength convention.

FRACTURE INDEX

When logging the rock cores, the fracture Index (i.e., the number of fractures for each 0.3m length of core) was also recorded. It was observed that the planes of weaknesses along which the cores tended to break. The Fracture Index is expressed as the number of discontinuities per 300 mm (1ft).

WEATHERING

The degree of weathering ranged generally from moderately weathered to slightly weathered as indicated on the Records of Rock Cores.

5. FIELD INVESTIGATION

5.1. MONITORING WELL INSTALLATION

In 2022, monitoring wells were installed in ten (10) of the thirty (30) boreholes upon completion of the drilling program for short term groundwater monitoring. The wells were screened across the shallow bedrock and overburden from 331.02 to 318.5m above sea level (m ASL).

In August 2023, five (5) more boreholes were drilled with three (3) of them being completed as monitoring wells. The additional wells were also screened across the shallow bedrock and overburden from 311.46 to 305.48m ASL.

Each monitoring well was installed by inserting the screen and casing assembly into the borehole to the designed depth and then packing a silica sand pack filter around the screen interval. Above the sand pack, a bentonite hole plug was installed to eliminate contamination from surface along the annulus space. All the installed monitoring wells were finished with a flush-mount protective casing. Ground levels at each of the monitoring well locations were surveyed to an elevation datum and reported on the borehole logs. Well installation details are also included on the logs in Appendix B.

5.2. GROUNDWATER LEVEL MONITORING

5.2.1. 2022 Groundwater Monitoring

Water levels at each of the newly installed monitoring wells were measured on completion of installation and at one week post installation. Ten (10) monitoring wells were installed within the site boundary. However, only three (3) of the monitoring wells showed the presence of water. The remaining seven (7) wells were noted as dry as they did not contain any water during site visits. During site visits, the groundwater level reached a maximum of 2.57m below ground surface corresponding to an elevation of 3.19.80m ASL.

5.2.2. 2023 Groundwater Monitoring

Water levels at each of the newly installed monitoring wells were measured on completion of installation and at one week post installation. The depth to groundwater ranged from 1.7 to 3.3m below ground surface, which corresponds to elevation 308.4 to 311.2m ASL.

Details on all completed monitoring wells, including manual groundwater level observations can be found in Table C-1 in Appendix C.

5.3. HYDRAULIC CONDUCTIVITY ASESSMENT

5.3.1. In-Situ Single Well Response Testing

EnVision conducted confirmatory SWRT at BH 22-4, and BH 22-17 in 2022 and at BH23-1, BH23-3 and BH23-4 in August 2023. In advance of performing SWRT, the monitoring wells were developed to remove the potential presence of fine sediments. The development process involved purging of the monitoring wells to induce the flow of fresh formation water through the screen. The monitoring well water levels were permitted to fully recover prior to performing SWRTs.

During the SWRT, a slug of water was near-instantaneously removed from the well and the response in water level was recorded. The K values for each of the tested wells were calculated from the SWRT data using Aqtesolv Software and the Bower-Rice solutions for unconfined conditions. The semi-log plots for normalized drawdown versus time are included in Appendix D. *Table 5-1* presents a summary of the In-Situ rising head test results.

Table 5-1: In-Situ Single Well Response Tests

		SCREEN DEPTH		K – BOUWER-RICE	
BH ID	**TESTING BY**	**From (m)**	**To (m)**	**(m/sec)**	**(m/day)**
BH 22-4	ENVISION	2.30	3.80	$7.92X10^{-8}$	$6.84X10^{-3}$
BH 22-17	ENVISION	2.30	3.80	$2.86x10^{-8}$	$2.47X10^{-3}$
BH23-1	ENVISION	3.05	4.57	$1.91x10^{-5}$	1.65
BH23-3	ENVISION	2.74	4.27	$2.49x10^{-7}$	$2.15x10^{-2}$
BH23-4	ENVISION	3.05	6.10	$1.67x10^{-5}$	1.44

The hydraulic conductivity all borehole locations was calculated through the Hazen Approximation using grain size analysis. The Hazen Approximation relates the hydraulic conductivity of a material to the 10th percentile grain size of the material and Hazen's empirical coefficient (Hazen, 1893). *Table 5-2* provides a summary of the Hazen approximation calculations.

Table 5-2: Hazen Approximation Summary

			GEOMEAN HYDRAULIC CONDUCTIVITY (K)	
BH ID	**SAMPLE ID**	**SOIL UNIT DESCRIPTION**	**(m/sec)**	**(m/day)**
BH 22-1	SS2	Sandy silt fill	5.76E-08	4.98E-03
BH 22-2	SS3	Silty sand fill	1.60E-05	1.38E+00
BH 22-3	SS1	Granular/fill	1.96E-06	1.69E-01
BH 22-4	SS4	Sandy silt till	9.00E-08	7.78E-03
BH 22-5	SS2	Silty sand fill	2.70E-07	2.34E-02
BH 22-6	SS3	Sandy silt fill	1.96E-08	1.69E-03
BH 22-7	SS2	Silty sand fill	1.60E-07	1.38E-02
BH 22-8	SS1	Granular	9.61E-06	8.30E-01
BH 22-9	SS2	Sandy silt	1.60E-07	1.38E-02
BH 22-10	SS1	Granular/fill	9.00E-06	7.78E-01
BH 22-11	SS2	Silty sand fill	4.41E-08	3.81E-03
BH 22-12	SS1	Granular	5.63E-05	4.86E+00
BH 22-13	SS3	Sandy silt	2.25E-08	1.94E-03
BH 22-14	SS4	Sand fill	2.50E-05	2.16E+00
BH 22-15	SS3	Silty sand fill	4.90E-07	4.23E-02
BH 22-16	SS1	Granular	2.03E-05	1.75E+00
BH 22-18	SS2	Sand fill	7.84E-06	6.77E-01
BH 22-19	SS1	Granular/fill	3.03E-05	2.61E+00

			GEOMEAN HYDRAULIC CONDUCTIVITY (K)	
BH ID	**SAMPLE ID**	**SOIL UNIT DESCRIPTION**	**(M/SEC)**	**(M/DAY)**
BH 22-20	SS1	Granular/fill	2.50E-05	2.16E+00
BH 22-21	SS3	Silty clay till	2.50E-09	2.16E-04
BH 22-22	SS1	Granular/fill	1.60E-05	1.38E+00

Based on the results of the Hazen approximations, the hydraulic conductivity ranges from 2.25×10^{-4} to 2.50×10^{-9}m/sec in the upper fill material and 1.60×10^{-7} to 1.69×10^{-8}m/sec in the native soils.

5.4. GROUNDWATER QUALITY ASSESSMENT

To assess the suitability for discharge of pumped groundwater to the City of Guelph sanitary sewer during dewatering activities, one (1) groundwater sample was collected from BH 22-4. One (1) sample of Routine Comprehensive Package (RCAp) was also collected from BH 22-17 to assess the general chemistry of the water.

From the additional boreholes drilled in August 2023, one (1) groundwater sample was collected from BH23-1 to assess the suitability for discharge of groundwater to the City of Guelph sanitary sewer. One (1) groundwater sample was collected from BH23-4 to assess the suitability for discharge of groundwater to the City of Guelph storm sewer. An additional two (2) Routine Comprehensive Packages (RCAp) were collected from BH23-3 and BH23-4 to assess the general chemistry of the water.

Prior to collection of the samples, approximately three (3) well volumes of standing groundwater were purged from each well.

The suites were collected unfiltered and placed into pre-cleaned laboratory-supplied vials and/or bottles provided with analytical test group specific preservatives, as required. Dedicated nitrile gloves were used during sample handling. The groundwater samples were submitted to an independent laboratory, Bureau Veritas Laboratories (BV), in Mississauga, Ontario, for analysis of parameters of the City of Guelph Sewer Use By-Law and general chemistry. BV is a certified laboratory by the Canadian Association for Laboratory Accreditation Inc.

For the assessment purposes, the analytical results were compared to Table 1 – Limits for Sanitary and Combined Sewer Discharge.

A summary of the analytical results and the laboratory Certificate of Analysis (CofA) are enclosed in Appendix E. A summary of the noted exceedances from 2022 of the Guelph Sanitary/Storm Sewer By-Law is included in *Table 5-3* below and the results from 2023 are included in *Table 5-4* below.

Table 5-3 Guelph Sanitary/Storm Sewer By-Law Exceedances 2022

PARAMETER	UNITS	LIMITS FOR STORM DISCHARGE	LIMITS FOR SANITARY DISCHARGE	RESULTS - BH22-4 (MAY 6, 2022)	RESULTS - BH22-17 RCAP (MAY 6, 2022)
TOTAL SUSPENDED SOLIDS (TSS)	mg/L	15	350	**150**	N/A
DISSOLVED CHLORIDE (CL-)	mg/L	-	1500	810	<u>6200</u>

Notes:

Double Underlined Bold = Exceeds both limits

UNDERLINE = EXCEEDS SANITARY SEWER RELEASE LIMITS ONLY

Bold = Exceeds the Storm Sewer Release limits only

N/A = Not Measured

Results from the monitoring well, BH22-4 indicate no exceedances for parameters outlined in the sanitary table. However, the results indicate that there is an exceedance in the concentration of total suspended solids (TSS) when compared to the parameters under the storm sewer release limits.

The general chemistry RCAP sample taken at BH 22-17 indicates one exceedance in dissolved chloride when compared to the parameters under the sanitary sewer release limits.

Table 5-4: Guelph Sanitary/Storm Sewer By-Law Exceedances 2023

PARAMETER	UNITS	LIMITS FOR STORM DISCHARGE	LIMITS FOR SANITARY DISCHARGE	RESULTS - BH23-1 (AUGUST 29, 2023)	RESULTS - BH23-3 (AUGUST 29, 2023)	RESULTS - BH23-4 (AUGUST 29, 2023)

TOTAL SUSPENDED SOLIDS (TSS)	mg/L	15	350	2600	N/A	830
DISSOLVED CHLORIDE (CL-)	mg/L	-	1500	2600	3500	1700
TOTAL CADMIUM (CD)	mg/L	0.001	1	**0.002**	N/A	0.00036
TOTAL COPPER (CU)	mg/L	0.01	3	**0.03**	<0.0009	0.0064
TOTAL LEAD (PB)	mg/L	0.05	5	**0.22**	<0.0005	0.004
TOTAL ZINC (ZN)	mg/L	0.05	3	**1.1**	<0.005	**0.13**

Notes:

Double Underlined Bold = Exceeds both limits

UNDERLINE = EXCEEDS SANITARY SEWER RELEASE LIMITS ONLY

Bold = Exceeds the Storm Sewer Release limits only

N/A = Not Measured

Results from the monitoring well BH23-1 indicate exceedances for total suspended solids and total dissolved chloride in the sanitary table. The results indicate that there are also exceedances in the concentration of total suspended solids (TSS), total cadmium, total copper, total lead and total zinc when compared to the parameters under the storm sewer release limits.

Results from the monitoring well BH23-3 indicate an exceedance in dissolved chloride when compared to the sanitary by-law table. When compared to the parameters under storm sewer release limits, no exceedances were found.

Results from the monitoring well BH23-4 indicate exceedances for total suspended solids and total dissolved chloride in the sanitary table. The results indicate that there are also exceedances in the concentration of total suspended solids (TSS) and total zinc when compared to the parameters under the storm sewer release limits.

The water quality results indicate that a form of treatment would be required prior to any discharge to the municipal sewer system. Treatment in the form of physical filtration or settlement is anticipated to control the total suspended solid concentrations. A contractor should provide further input into the selection of an appropriate treatment system.

6. CONSTRUCTION DEWATERING ASSESSMENT

Water takings within the Province of Ontario are governed by the Ontario Water Resources Act (OWRA), and the Water Taking and Transfer Regulation (O.Reg. 387/04). In addition, O.Reg. 63/16 regulates water takings for temporary activities, such as construction and road work dewatering. In Ontario, construction dewatering that exceed 50,000 L/day require either a Category 3 PTTW, or registration with the MECP EASR. The proposed work may fall within the following possible categories:

- Surface water diversions without pumping (i.e. non-earth cofferdam, sheet piles, sand bags designed to provide a dry work area) are exempt and do not require permitting.
- Surface water diversions with pumping out of an excavation designed to provide a dry working area is exempt from permitting, except that best management practices listed in the regulation must be followed.
- Pumping of groundwater (construction dewatering) to maintain a dry work area, which fall under one of three scenarios:
    - Volumes of a combination of groundwater and surface water (precipitation) that is below 50,000 L/day are exempt from permitting
    - Volumes of a combination of groundwater and surface water (precipitation) that is above 50,000 L/day but below 400,000 L/day require registration as an EASR
    - Volumes of groundwater that is above 400,000 L/day will require a Category 3 PTTW.

6.1. OPEN CUT TRENCHING METHODOLOGY

To estimate the amount of dewatering needed to drain the area for proposed construction along open-cut sections, the Powers expression (long narrow system equation) for unconfined and confined aquifer steady-state condition, was used (Powers, 2007):

$$Q = \frac{\pi K(H^2 - h^2)}{\ln {}^{R_0}/_{r_e}} + \frac{2(xK(H^2 - h^2))}{2L}$$

Where:

Q = Groundwater discharge (m^3/day)

H = Initial depth of water (static head) prior to dewatering (m)

h = Elevation of water beneath excavation while pumping (m)

K = Hydraulic Conductivity (m/day)

r_e = effective radius of excavation (m)

R_0 =2 L = estimated radius of influence (m)

The zone of influence (ZOI) is calculated using the empirical Sichardt equation, which can be stated as:

$$R_0 = C(H - h)\sqrt{K}$$

Where:

C = Coefficient constant, assumed 3000 for a line source;

6.2. DEWATERING ASSUMPTIONS

The following dewatering assessment has been created for preliminary purposes based on limited data collected. The assessment provides a water taking estimate for open cut trenching around the downtown area at a depth of 4m and a width of 3m. The water taking estimate is provided in L/day per 50m of trench length.

Groundwater levels and hydraulic conductivity values have been assigned based on groundwater level monitoring and in-situ single well response test results.

Many of the wells onsite were recorded dry as they did not intersect the water table below. To account for this, the maximum groundwater level at all dry wells is assumed to intersect the bottom of the well screen. The dry wells were also assigned a hydraulic conductivity (K) of 5.84X 10^{-8} m/sec. This was obtained by averaging K values across the site from grain size analysis for all soil samples that were taken below the granular/fill.

At well BH 22-21, a groundwater level was measured at 320.26m ASL. However, there was not enough water found in the well to perform an in-situ single well response test. Therefore, this well has also been assigned a K of 5.84X 10^{-8} m/sec.

To complete that dewatering assessment, the following assumptions have been included in *Table 6-1*.

Table 6-1: Summary of Dewatering Assumptions

DESCRIPTION	ASSUMPTION	NOTES
Dimensions of the excavation	4m depth, 3m width	Groundwater takings are calculated per 50m of trenching
Base of the aquifer	2m below excavation	Assumed based on borehole logs
Groundwater level of dry wells	3.81mbgs 2.90mbgs at BH 22-9	Assumed to be at the bottom of the screen in all dry wells
Hydraulic Conductivity for dry wells and BH 22-21	$5.84X\ 10^{-8}$ m/sec	Assigned as an average K over the site from grain size analysis.
Safety Factor	3	Assigned to account for unforeseen conditions
Stormwater Component	3000L/day	Assumed for a 20mm rain event during trenching

6.3. SUMMARY OF DEWATERING ASSESSMENT

Table 6-2 below provides a summary of estimated open cut trenching water takings required for a 50m length of generic 4m deep trench on each street/roadway.

Table 6-2: Summary of Short-Term Dewatering Assessment

SOURCE		GROUND ELEVATION (M ASL)	EXCAVATION DEPTH 4M TRENCH (M ASL)	INFERRED GW LEVEL (M ASL)	K (M/S)	Q MAX (M^3/ DAY)	SAFETY FACTOR	TOTAL WATER TAKINGS WITH SAFETY + STORMWATER INCLUDED (L/DAY)
2022 Investigation Results								
Dublin St	BH 22-2	325.3	321.3	321.49	$1.0x10^{-7}$	0.5	3	4,600
Norfolk St	BH 22-4	333.3	329.3	330.50	$7.92x10^{-8}$	1.2	3	6,600
Norfolk St	BH 22-9	325.6	321.6	322.70	$1.0x10^{-7}$	0.9	3	5,700
Wyndham St	BH 22-14	328.3	324.3	324.49	$1.0x10^{-7}$	0.5	3	4,600
Wyndham St	BH 22-15	324.1	320.1	320.29	$1.0x10^{-7}$	0.5	3	4,600
Macdonell St	BH 22-17	322.4	318.4	319.80	$2.86x10^{-8}$	0.6	3	4,900
Woolwich St	BH 22-19	330.5	326.5	326.69	$1.0x10^{-7}$	0.5	3	4,600
Woolwich St	BH 22-21	324.0	320.0	320.26	$1.0x10^{-7}$	0.6	3	4,700
Suffolk St W	BH 22-24	331.5	327.5	327.69	$1.0x10^{-7}$	0.5	3	4,600
Quebec St	BH 22-26	327.0	323.0	323.19	$1.0x10^{-7}$	0.5	3	4,600
2023 Investigation Results								
Wellington St	BH 23-1	310.1	306.1	308.37	$1.64x10^{-5}$	54.6	3	166,500
Wellington St	BH 23-3	313.4	309.4	310.07	$2.49x10^{-7}$	1.9	3	8,700

SOURCE		GROUND ELEVATION (M ASL)	EXCAVATION DEPTH 4M TRENCH (M ASL)	INFERRED GW LEVEL (M ASL)	K (M/S)	Q MAX (M3/ DAY)	SAFETY FACTOR	TOTAL WATER TAKINGS WITH SAFETY + STORMWATER INCLUDED (L/DAY)
Wyndham St	BH 23-4	314.5	310.5	308.5	$1.67x10^{-5}$	32.8	3	101,400

Based on the project setting, requirements, and findings from above, ***Table 6-2*** presents the short-term dewatering results for open cut trenching up to 4m deep across the downtown area. A factor of safety of 3.0 has been applied to account for unforeseen conditions, such as leakage along bedding planes, or perched groundwater within fill deposits. The safety factor has been included for appropriate contingency measures. As shown in ***Table 6-2***¸ the short-term dewatering rates, including stormwater/precipitation events is estimated to range from about 4,600 to 166,500 L/day The complete open cut trenching water taking assessment Is provided in Appendix F, Table F-1.

7. IMPACT ASSESSMENT

The process of groundwater control can introduce potential risks to nearby property owners, groundwater users, and to the environment. Based on the assumptions outlined in Section 6 above, the following impact assessment has been provided to aid in developing appropriate mitigation and monitoring activities.

7.1. ZONE OF INFLUENCE FROM DEWATERING

The concept of a zone of influence due to temporary dewatering is intended to be used in assessing on-site and off-site impacts to both the public and environment. Based on the preliminary dewatering assessment, the zone of influence from dewatering efforts from 50m sections of open cut trenching is not expected to extend beyond about 41.7 m from the edges of excavation. At distances beyond about 41.7m from this boundary, the expected groundwater lowering is anticipated to be negligible.

7.2. IMPACTS TO GROUNDWATER USERS

Any negative impacts to groundwater users within the Study Area is considered negligible based on the following:

1) Shallow earth works are anticipated and the well database indicates that the are few water supply wells in the area;
2) Area is 100% supplied by municipal water;
3) Temporary groundwater control measures are anticipated, which are not anticipated to effect any long-term users;
4) Relatively minor water takings have been estimated for the project, further lessening any risk to the groundwater quantity and quality for nearby groundwater users.

7.3. IMPACTS TO NEARBY STRUCTURES

There is always a possibility of inducing settlement to neighboring buildings, utilities and underground structures/infrastructure when lowering water levels or depressurizing an aquifer. It is considered a best practice to instigate a pro-active monitoring program to identify any potential areas of concern and the need and type of monitoring required. Utilities, and transit owners may have stringent monitoring requirements, which will have to be adhered to.

It is recommended that a geotechnical assessment for ground settlement be conducted based on the estimated pumping zone of influence and expected drawdowns outlined in Section 6.

7.4. IMPACTS TO CITY OF GUELPH SEWER SYSTEM

Negative impacts to the Municipal sewage works could potentially occur during dewatering, either due from quantity, or quality. The dewatering discharge rates provided in this report do not take into consideration the sewer capacity that the receiving system may hold. The capacity of any receiving system will generally be quantified during any future discharge permitting.

The quality of the discharge water must be controlled if it is to be conveyed to the regional sewer system. Controls must be put in place to ensure that the groundwater quality meets the allowable limits under the appropriate discharge permit. The groundwater sampling undertaken by EnVision indicates that the groundwater is of suitable quality for discharge to the sanitary sewer with treatment for TSS and dissolved chloride. A treatment plan will be required if the discharge of dewatering effluent is to be directed to any roadside ditch, land surface, or storm sewer system. A treatment contractor will be required to review the water taking plan and develop appropriate treatment based on the discharge receptor selected.

A monitoring program has been developed in Section 8.0 to ensure that negative impacts associated with quality and quantity are mitigated.

7.5. CONTAMINANT MIGRATION DURING DEWATERING

Changes to the hydraulic gradient could potentially influence migration of contaminants from off-site properties. The near site area is considered to feature many potential sources of groundwater contaminants. During dewatering activity, it is possible to alter the natural groundwater hydraulic gradient and cause dissolved contaminants to migrate onto the property. Although no impacts were noted during the sampling of groundwater, it is recommended that a contaminant monitoring program be implemented during any active dewatering. The existing monitoring well network can be utilized for this program.

7.6. LONG-TERM DRAINAGE

There are no long-term drainage systems being proposed for this project. Long-term impacts to the groundwater system is not anticipated.

8. WATER TAKING AND DISCHARGE PERMITS

8.1. MECP WATER TAKING PERMIT (EASR/PTTW)

The expected total daily water takings during open cut trenching for 50m open cut trenching sections are anticipated to be above 50,000 L/day on Wyndham St and Wellington St and therefore the activities will require a registration using the Ministry of the Environment's online Environmental Activity and Sector Registry for construction dewatering.

If necessary to expedite the work and it is determined that multiple trenching locations will be advanced at once, a registration under the EASR portal may be recommended. This will require that a dewatering and discharge plan be prepared by a qualified person to further outline the program and to conform with O.Reg. 63/16.

8.2. DISCHARGE PERMITTING AND TREATMENT

During construction, the discharge of construction dewatering effluent could be conveyed to the City of Guelph Sanitary sewer with treatment for dissolved chloride. However, a treatment plan will be required if the discharge of dewatering effluent is to be directed to any roadside ditch, land surface, or storm sewer system. A treatment contractor will be required to review the water taking plan and develop appropriate treatment, based on the discharge receptor selected. Alternatively, the water could be contained onsite in environmental tanks for haulage and removal to an offsite waste facility.

For discharge to the city storm sewer, a more rigorous treatment and monitoring plan would be required to ensure that the effluent meets the strict conditions for discharge release.

9. MONITORING AND MITIGATION

9.1. CONSTRUCTION DEWATERING MONITORING

The active construction dewatering stage will require monitoring designed to assess the potential for impacts to water levels in aquifers, water quality, and ground settlement. The monitoring program should include the following components:

- Discharge volume reporting
- Groundwater level monitoring
- Discharge water quality monitoring
- Ground settlement monitoring

9.1.1. Discharge Volume Reporting

During active dewatering, the contractor will be required to document discharge pumping rates as a required condition of the EASR, with regular reporting of water taking volumes via the MECP Water Taking Reporting System. A flow meter should be supplied, and all discharged ground and storm water should be discharged through the properly field calibrated device. A non-resettable flow meter that records discharge in both instantaneous and cumulative modes is recommended. Daily recording of the discharge volumes will be required for regular reporting. The total combined daily discharge must never exceed the limits as outlined in the EASR. Additional storage measures (such as Extra tank storage or temporary settling ponds) can be used to control large rain events and reduce the instantaneous discharge/pumping rates. Further restrictions or conditions may be imposed through the enforced discharge agreement issued by the municipality.

9.1.2. Groundwater Quality Monitoring

A monitoring program should be implemented that is based on the selected discharge option. The monitoring program should consist of daily visual examination of the construction effluent for the presence of any sheen, foam, or odour. Water clarity and sediment level should also be monitored to ensure that the quality is not degrading during construction. Filters should be examined on a regular basis, and any failures to equipment should be repaired immediately. Discharge permitting may also include specific water quality testing that must be adhered to.

Impacts to water quality can be controlled using safe construction practices that eliminate the potential for waste spills and other contamination events. Refueling should be performed in designated areas away from open excavations. In the event of a spill, remedial action must be undertaken immediately by the contractor, following all MECP and provincial spill guidelines.

In addition, the migration of contaminants from off-site properties should be monitored by periodic water quality sampling from monitoring wells located along the property boundary or from the discharge outlet. This periodic sampling should be done frequently during the first month of dewatering; daily for 3 days, weekly afterwards for the first month, and consist of analysis for gasoline by-products. If contaminant migration is noted, and based on the degrading water quality, a treatment system may be required to ensure discharge water continues to meet the limits of the discharge agreement for the proposed receptor.

9.1.3. Ground Settlement Monitoring

As discussed previously, structures located within the ZOI may be susceptible to potential settlement or subsidence during any temporary dewatering. The following monitoring and mitigation measures are recommended:

- Consider a pre-construction condition survey for the structures located within the ZOI;
- Install monitoring devices on nearby buildings and structures, and maintain scheduled monitoring during active dewatering;
- Prepare to reduce dewatering efforts if undesirable deformation conditions present.

A geotechnical engineer should review and provide further input for ground settlement impacts.

10. CLOSING

10.1. CONCLUSIONS

Based on the information obtained through this Hydrogeological Assessment, Envision presents the following conclusions and recommendations:

- The surficial material has been mapped and consists of glaciofluvial deposits of gravel along with sandy silt to silty sand textured till;
- Bedrock identified as the Guelph formation; a mix of shale, siltstone, dolostone and sandstone anticipated between 3 and 13m below ground surface;
- The MECP WWR database indicated that there are one-hundred and forty-five (145) water wells in the Study Area. Of the well records returned in the search, nine of them were classified as water supply wells for domestic or commercial use. All the other well records were reported as monitoring wells, test holes, unclassified or abandoned;
- Active groundwater monitoring has been completed and the groundwater level ranges from 308.37 to 330.46 m ASL for the site.
- Based on the results of twenty-one (21) Hazen approximations, the hydraulic conductivity ranges from $2.25x10^{-4}$ to $2.50x10^{-9}$ m/sec in the upper fill material and $1.60x10^{-7}$ to $1.69x10^{-8}$ m/sec in the native soils.
- Based on in-situ single well response tests, the estimated saturated hydraulic conductivity ranges from $1.67X10^{-5}$ to $7.92X10^{-8}$ m/sec ;
- Based on the assumptions outlined in this report, the dewatering assessment has been broken down into 50m sections of open cut trenching at 4m depth across the downtown area. The estimated short-term construction dewatering flow rate for each section of dewatering is expected to range from 4,900 to 166,500 L/day including the rainwater input after minor precipitation events;
- A short-term construction dewatering EASR is expected for temporary construction dewatering as daily water takings are expected to exceed 50,000 L/day;
- Approval and a discharge agreement with the City of Guelph will be required to discharge dewatering effluent into the municipal sanitary/combined sewer system;
- Based on groundwater sampling at two locations, pre-treatment for TSS, total cadmium, total copper, total lead and total zinc will be required for discharge to the region storm sewer;
- Based on groundwater sampling at two locations, pre-treatment for TSS and dissolved chloride will be required for discharge to the Regional sanitary sewer;

Based on the above conclusions, the following recommendations are provided:

1) All estimates provided should be revised during detailed design work as more information becomes available, including building footprint, depth of excavation, type of foundation, shoring selection, and other information;
2) A discharge agreement for any construction dewatering effluent discharge via City of Guelph sewer will require a permit and should be obtained 3 months prior to construction;

10.2. QUALIFICATION OF THE ASSESSORS

Robin Byers, P.Geo., B.Sc. is a Senior Hydrogeologist and is a practicing member of the Professional Geoscientists of Ontario with over 9 years of hydrogeological experience working in the Greater Toronto Area and Southern Ontario. He has experience in physical and chemical hydrogeology with foundational knowledge of well construction and design, groundwater modeling, pumping test analysis, and construction dewatering. Rob is also a qualified person as defined by O.Reg 63/16 for purposes of preparing water taking and discharge plans.

10.3. CERTIFICATION AND SIGNATURES

EnVision confirms the findings and conclusions of the Hydrogeological Investigation.

Prepared by

Sam Harding, B.Sc.,
Environmental Scientist
sharding@envisionconsultants.ca

Reviewed by

Rob Byers, B.Sc., P.Geo.,
Senior Hydrogeologist
rbyers@envisionconsultants.ca

10.4. QUALIFIER

EnVision prepared this report solely for the use of the intended recipient in accordance with the professional services agreement. In the event a contract has not been executed, the parties agree that the EnVision General Terms and Conditions, which were provided prior to the preparation of this report, shall govern their business relationship.

The report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment. The conclusions presented in this report are based on work performed by trained, professional and technical staff, in accordance with their reasonable interpretation of current and accepted engineering and scientific practices at the time the work was performed.

The content and opinions contained in the report are based on the observations and/or information available to EnVision at the time of preparation, using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by EnVision and other engineering/scientific practitioners working under similar conditions, and subject to the same time, financial and physical constraints applicable to this project.

EnVision disclaims any obligation to update this report if, after the date of this report, any conditions appear to differ significantly from those presented in this report; however, EnVision reserves the right to amend or supplement this report based on additional information, documentation or evidence.

EnVision makes no other representations whatsoever concerning the legal significance of its findings. The intended recipient is solely responsible for the disclosure of any information contained in this report. If a third party makes use of, relies on, or makes decisions in accordance with this report, said third party is solely responsible for such use, reliance or decisions. EnVision does not accept responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken by said third party based on this report.

EnVision has provided services to the intended recipient in accordance with the professional services agreement between the parties and in a manner consistent with that degree of care, skill and diligence normally provided by members of the same profession performing the same or comparable services in respect of projects of a similar nature in similar circumstances. It is understood and agreed by EnVision and the recipient of this report that EnVision provides no warranty, express or implied, of any kind. Without limiting the generality of the foregoing, it is agreed and understood by EnVision and the recipient of this report that EnVision makes no representation or warranty whatsoever as to the sufficiency of its scope of work for the purpose sought by the recipient of this report.

In preparing this report, EnVision has relied in good faith on information provided by others, as noted in the report. EnVision has reasonably assumed that the information provided is correct and EnVision is not responsible for the accuracy or completeness of such information.

Unless otherwise agreed in writing by EnVision, the Report shall not be used to express or imply warranty as to the suitability of the site for a particular purpose. EnVision disclaims any responsibility for consequential financial effects on transactions or property values, or requirements for follow-up actions /or costs.

This limitations statement is considered an integral part of this report.

11. REFERENCES

Hazen, A. (1893). Some physical properties of sands and gravels, with special reference to their use in filtration. *State Board of Health Annual Report 24*, 541-556.

Matrix Solutions Inc. (March 2017). *City of Guelph and Township of Guelph/Eramosa Tier Three Water Budget and Local Area Risk Assessment.*

Ministry of Environment, Conservation and Parks. (2018, October 1). *Well records*. Retrieved from Ontario.ca: https://www.ontario.ca/page/well-records

Ministry of Northern Development, Mines and Forestry. (2013, April). Surficial geology of southern ontario. Sudbury, Ontario.

Ministry of the Environment, Conservation and Parks. (2021, Nov). *Source Water Protection Atlas*. Retrieved from https://www.lioapplications.lrc.gov.on.ca/SourceWaterProtection/index.html?viewer=SourceWaterProtection.SWPViewer&locale=en-CA

Powers, J. C. (2007). *Construction Dewatering and Groundwater Control: New methods and applications.* Wiley and Sons.

Sharpe, D. (1980). Quaternary Geology of Toronto and Surrounding Area: Ontario Geological Survey Preliminary Map P2204.

FIGURES

APPENDIX A:

MECP Water Well Record Summary

APPENDIX B:

Borehole Logs

APPENDIX C:

Groundwater Monitoring

APPENDIX D: *Hydraulic Conductivity (Single Well Response Testing)*

APPENDIX E: *Laboratory Certificate of Analysis*

APPENDIX F: *Open Cut Trenching Dewatering Calculations*