1101A, 1105 and 1163 Kingston Road, Pickering, Ontario

L1V1B5

Hydrogeological Investigation and Water Balance Assessment

Client:

Tribute (Brookdale) Limited

1815 Ironstone Manor, Unit 1, Pickering, Ontario, L1W3W9

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1 Introduction

1.1 Project Description

EXP Services Inc. (EXP) was retained by Tribute (Brookdale) Limited to prepare a Hydrogeological Investigation and Water Balance Assessment Report associated with the proposed development located at 1101A, 1105 and 1163 Kingston Road, Pickering, Ontario (hereinafter referred to as the 'Site').

The Site is currently occupied by the Brookdale Centre (containing five commercial buildings) and portion of a Walnut Lane at northern portion of the Site. It is our understanding that the proposed development plan is in preliminary stage and comprises of five parcels (A, , B, C, D and E) having seventeen (17) to thirty-five (35) storeys towers with one (1) to three (3) levels of underground parking. The Site location plan is shown on Figure 1.

EXP conducted a Preliminary Geotechnical Investigation in conjunction with this investigation. The pertinent information gathered from the noted investigations is utilized for this report.

1.2 Project Objectives

The main objectives of the Hydrogeological Investigation and Water Balance Assessment are as follows:

- Establish the local hydrogeological settings within the Site;
- Provide Preliminary recommendations on construction and long-term dewatering;
- Assess groundwater quality; and
- Prepare a Hydrogeological Investigation and Water Balance Assessment Report.

1.3 Scope of Work

To achieve the investigation objectives, EXP has completed the following scope of work:

- Reviewed available geological and hydrogeological information for the Site;
- Drilled and installed ten (10) monitoring wells (BH1, BH2S, BH2D, BH3S, BH3D, BH4, BH5S, BH5D, BH6, BH7) to an approximate depth ranging from 11 meter below ground surface (mbgs) to 19 mbgs and three monitoring wells (BH2S/2D, BH3S/3D and BH5S/5D) are in nested configurations;
- Installed 50 mm diameter monitoring wells in the geotechnical boreholes;
- Developed and conducted Single Well Response Tests (SWRT) on monitoring wells to assess hydraulic conductivities of the saturated soils at the Site;
- Completed two (2) rounds of groundwater level measurements at all monitoring wells;
- Collected one (1) groundwater sample for analyses of parameters, as listed in the Durham Region Sanitary and Storm Sewer Use By-Law;
- Evaluated the information collected during the field investigation program, including borehole geological information, Water Well Records (WWR), SWRT results, groundwater level measurements and groundwater water quality;
- Prepared site plans, cross sections, geological mapping and groundwater contour mapping for the Site;
- Provided preliminary recommendations on the requirements for construction and long-term dewatering;

- Provided recommendations on the Ministry of Environment, Conservation and Parks (MECP) Water Taking Permits and Durham Region Sewer Discharge Agreements (SDA) for the construction and post-construction phases;
- Conducted pre- and post-development water balance assessment (Thornthwaite and Mather approach); and
- Prepared a Hydrogeological Investigation and Water Balance Assessment Report.

The Hydrogeological Investigation and Water Balance Assessment was prepared in accordance with the Ontario Water Resources Act, Ontario Regulation 387/04, and Durham Region Sewer Use By-Lay No. 55-2013. The scope of work outlined above was made to assess dewatering and did not include a review of Environmental Site Assessments (ESA).

1.4 Review of Previous Reports

The following reports were reviewed as part of this Hydrogeological Investigation and Water Balance Assessment:

- EXP Services Inc. (July 12, 2023), Preliminary Geotechnical Investigation, 1101A and 1105 Kingston Road, Pickering, ON, prepared for Tribute (Brookdale) Limited.
- EXP Services Inc. (Revised October 18, 2023), Phase One Environmental Site Assessment, 1101A, 1105 and 1163 Kingston Road, Pickering, ON, prepared for Tribute (Brookdale) Limited.

Any past and/or future geotechnical, hydrogeological, environmental and risk assessments, and updated development/architectural plans should be provided to update this hydrogeological report prior to submission of permits and approvals by the municipalities and agencies.

2 Hydrogeological Setting

2.1 Regional Setting

2.1.1 Regional Physiography

The Site is within a physiographic region known as the Iroquois Plain as shown in Figure 2A. The physiographic landform is named Sand Plains on the west side and Clay Plains on the east side of the Site as shown in Figure 2B. The South Slope lies to the north of the Iroquois Plain (Chapman & Putnam, 2007).

The Iroquois Plain was created along the shores of former Lake Iroquois, an ancient glacial lake. The noted Plain primarily consists of shallow water sandy deposits.

The topography of the Iroquois Plain is relatively flat with a gradual slope to the south, toward Lake Ontario.

2.1.2 Regional Geology and Hydrogeology

The surficial geology can be described as fine-textured glaciolacustrine deposits consisting of silt and clay, minor sand and gravel and Till (5b) consisting of stone-poor sandy silt to silty sand-textures till on a small portion of northwest portion of the Site (Ministry of Northern Development and Mines, 2012). The surficial geology of the Site and surrounding areas is shown in Figure 2C. Figure 2D shows the bedrock geology of the Alignments which consists of Georgian Bay Formation comprising shale, limestone, dolostone and siltstone.

Based on the available regional geology maps, the subsurface stratigraphy of the Site from top to bottom is summarized in Table 2-1 (TRCA, 2008 and Oak Ridge Moraine Groundwater Program, 2018). The overburden thickness is approximately 18.2 m. Two cross sections obtained from the ORMGP are presented in Figure 5C and 5D.

Table 2-1: Summary of Subsurface Stratigraphy

Stratigraphic Unit	General Description	Top Elevation of Stratigraphic Unit
Undifferentiated Upper Sediments	fine-textured glaciolacustrine deposits consisting of silt and clay, minor sand and gravel on the east side and Till (5b) consisting of stone-poor sandy silt to silty sand-textures till on the small portion of west side of the Site	85.1
Lower Newmarket Till (Aquitard)	This lithologic unit typically consists of sandy silt to clayey silt till interbedded with silt, clay, sand and gravel.	82.1
Thorncliffe Formation (Aquifer)	This geology formation generally consists of glaciofluvial (sand, silty sand) or glaciolacustrine deposits (silt, sand, pebbly silt and clay).	81.7
Scarborough Formation (Aquifer)	This geology unit is interpreted as deposits of a fluvial-deltaic system fed by large braided melt-water rivers draining from an ice sheet. It consists of peat sand overlaying silt and clay deposits.	70.5
Georgian Bay Formation	Bedrock primarily consists of interbedded shale, limestone, dolostone and siltstone. It belongs to the Upper Ordovician, (Ministry of Northern Development and Mines, 2012).	66.9

Regional groundwater across the area flows southeast, towards Lake Ontario (Oak Ridge Moraine Groundwater Program, 2018). Local deviation from the regional groundwater flow pattern may occur in response to changes in topography and/or soils, as well as the presence of surface water features and/or existing subsurface infrastructure.

2.1.3 Existing Water Well Survey

Water Well Records (WWRs) were compiled from the database maintained by the Ministry of the Environment, Conservation and Parks (MECP) and reviewed to determine the number of water wells documented within a 500-m radius of the Site boundaries. The locations of the MECP WWRs within 500 m of the Site are shown on Figure 3. A summary of the WWR is included in Appendix A.

The MECP WWR database indicates that eighty-seven (87) records within a 500 m radius from the Site centroid where ten (10) well records are identified onsite (Figure 3 and Appendix A). Well distances are calculated relative to the Site centroid, therefore some distances in Appendix A exceed 500 m.

The database indicates that the offsite wells are at an approximate distance of one hundred twenty-four (124) m or greater from the Site centroid. All wells were reportedly identified as monitoring and test holes (33), water supply wells (5), abandoned (23) and/or listed with unknown use (26).

The Well Identification Numbers (Well ID No.) of the offsite water supply wells are 4601194, 4601195, 4601196, 4601197 4601889 where those are reportedly located ranging from 190 m to 491 m from the Site centroid.

The reported water found depths ranged from 0.9 m to 41.1 meters below ground surface (mbgs).

Based on the date of installation of the water supply wells (12/3/1959 to 12/11/1964) and since the area is municipally serviced, it is unlikely that the noted water supply wells are still active.

2.2 Site Setting

2.2.1 Site Topography

The Site is in an urban land use setting. The topography is considered relatively flat with a regional gradual southeasterly slope towards Pine Creek and Lake Ontario.

As indicated on the borehole logs included in Appendix B, the surface elevation of the Site ranges between approximately 84.89 to 86.38 meters above sea level (masl).

2.2.2 Local Surface Water Features

The Site is within the Lake Ontario Waterfront watershed. No surface water features exist onsite. The nearest surface water features are Pine Creek, approximately located 100 meters east of the Site boundary and a wetland associated with Pine Creek. Lake Ontario is approximately 2.2 kms from the Site boundary to the south.

2.2.3 Local Geology and Hydrogeology

A summary of subsurface soil stratigraphy is provided in the following paragraphs. The soil descriptions are based on the geotechnical investigation report (EXP, July 12, 2023). They are summarized for the hydrogeological interpretations. As such, the information provided in this section shall not be used for construction design purposes.

The detailed soil profiles encountered in each borehole and the results of moisture content determinations are presented on the attached borehole logs (Appendix B). The soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the Hydrogeological Investigation and Water Balance Assessment and shall not be interpreted as exact planes of geological change.

The "Notes on Sample Description" preceding the borehole logs form an integral part of and should be read in conjunction with this report. The following is a brief description of the soil conditions encountered during the investigation.

Based on the results of the geotechnical investigation, the general subsurface soil stratigraphy consists of the following units from top to bottom:

Pavement Structure

Pavement structure, comprising 50 to 75 mm asphaltic concrete and 360 to 580 mm granular material, was encountered surficially in all of the boreholes.

Fill

Fill was encountered below the pavement structure in Boreholes 1, 4, 5D, 6 and 7. The fill varied from dark brown to brown topsoil-stained sandy silt to silty sand or silty clay with some gravel and topsoil inclusions. The compactness of the fill varied from loose to compact. Moisture contents of the moist to very moist fill ranged from 8 to 30%. The fill extended to depths of approximately 0.45 to 0.65 m below existing grade.

Silty Sand

Silty sand was encountered below the pavement structure in Borehole 2D. The silty sand deposit was brown in colour and existed in a compact state of compactness. The silty sand had a moisture content of 10%, indicating a moist condition. The silty sand deposit extended to a depth of about 1.0 m below existing grade.

Silt

Silt was encountered at depths ranging from approximately 0.65 to 1.65 m below existing grade in Boreholes 2D, 3D and 5D. The silt stratum was brown in colour and existed in a loose to compact state of compactness. Moisture contents of this material ranged from 17 to 20%, indicating a saturated condition. The silt stratum extended to a depth of about 2.5 m below existing grade.

Clayey Silt

Clayey silt was encountered at depths ranging from approximately 1.0 to 2.5 m below existing grade in Boreholes 1, 2D, 5D, 6 and 7. The clayey silt stratum was brown and grey in colour and soft to very stiff in consistency. Field shear vane tests indicated undrained shear strengths ranging from 19 to 130 kPa. Moisture contents of this material ranged from 19 to 26%, indicating a saturated condition. The clayey silt stratum extended to depths of about 2.5 to 8.75 m below existing grade.

Sandy Silt Till

Sandy silt till was encountered at depths ranging from approximately 1.75 to 10.25 m below existing grade in all of the boreholes. The sandy silt till deposit was primarily grey in colour and contained wet sand/sand and gravel seams and scattered gravel and cobbles. Cobble and boulder layers were encountered in Boreholes 4 and 7. The compactness of the sandy silt till varied from loose to very dense. The sandy silt till was loose to depths of about 4.0 to 5.5 m in Boreholes 3D, 5D and 7. Moisture contents of the sandy silt till generally ranged from 7 to 13%, indicating a moist to saturated condition. The sandy silt till deposit extended to depths of approximately 7.0 m to 17.5 m below existing grade.

Coarse Sand

Coarse sand was encountered at a depth of about 7.0 m below existing grade in Borehole 5D. The coarse sand deposit was grey in colour, contained occasional gravel and existed in a very dense state of compactness. Moisture contents of the wet coarse sand ranged from 12 to 14%. The coarse sand deposit extended to a depth of about 11.75 m below existing grade.

Sand and Gravel

Sand and gravel was encountered below the coarse sand deposit in Borehole 5D. The sand and gravel deposit was grey in colour, wet with moisture contents ranging from 8 to 10%, and existed in a very dense state of compactness. The sand and gravel deposit extended to a depth of about 14.5 m below existing grade.

Clayey Silt (lower)

A lower clayey silt stratum was encountered at a depth of approximately 11.5 m below existing grade in Borehole 1. The clayey silt stratum was grey in colour, moist with moisture contents ranging from 16 to 18%, and hard in consistency. The lower clayey silt stratum extended to a depth of about 14.75 m below existing grade.

Silty Sand Till

Silty sand till was encountered at depths ranging from approximately 8.5 to 16.0 m below existing grade in Boreholes 1, 2D, 3D and 6. The silty sand till deposit was grey in colour, contained scattered gravel and cobbles, and existed in a very dense state of compactness. Cobble and boulder layers were encountered near the bottom of the deposit in Borehole 1. Moisture contents of the very moist to wet silty sand till ranged from 8 to 11%. The silty sand till deposit extended to depths of about 10.25 to 18.5 m below existing grade.

Bedrock

Shale bedrock was encountered at depths ranging from about 14.5 to 18.5 m below existing grade in Boreholes 1, 2D, 3D, 4, 5D, 6 and 7 (approximate Elevation 66.6 to 70.4 m), indicating variable depths to bedrock. The inferred bedrock boundaries should not be interpreted as exact planes of bedrock since the auger will frequently penetrate some distance into the weathered rock before noticeable resistance is encountered.

To confirm bedrock and to determine its quality, Boreholes 1 and 4 were extended about 3 m into the bedrock by coring in HQ size using diamond drilling equipment. The rock core logs are attached to Log of Boreholes 1 and 4. Based on the rock recovery and the Rock Quality Designation (RQD), the bedrock is poor to good quality rock with horizontal fractures and some vertical joints. Generally, the upper 1 to 2 m of the shale bedrock is weathered becoming more sound with depth. However, it should be noted that weathered shale bedrock extended to a depth of 30.55 m below existing grade in Borehole 5D based on auger resistance and recovered split spoon samples.

The bedrock encountered in the boreholes is of the Blue Mountain Formation and underlies this site to a significant depth. Based on our experience, the upper zone of the shale bedrock is typically weathered with isolated weathered zones extending to greater depth. The predominate rock type is shale, but this shale is interbedded with limestone and siltstone. Typically, EXP has found the shale component in this formation is in the order of 80 percent in Greater Toronto area excavations. The limestone and siltstone components are generally 50 to 300 mm thick; however, thicker layers of up to 1,000 mm have been encountered. Stress relief features such as folds and faults are common in the Blue Mountain Formation. In these fractures, the rock is heavily fractured and sheared. It can also contain layers of shale rubble and clay. Due to the fracturing, these features may also contain groundwater conduits, which could result in excessive water flow into excavations. Weathering is much deeper than the surrounding sound unweathered bedrock. The stress relief features are usually in the order of 4 to 6 m wide, but in depth can vary from 4 to 5 m to in excess of 10 m.

The borehole and monitoring well locations are shown on Figure 4. Geological cross-sections were generated based on the available borehole logs completed as part of the previous and current investigations and shown on Figure 5A (Cross section A-A') and on Figure 5B (Cross section B-B'). The cross section shows a simplified representation of soil conditions and soil deposits may be interconnected differently than represented. Borehole logs used to generate both cross-sections are provided in Appendix B.

3 Results

3.1 Monitoring Well Details

The monitoring well network was installed as part of the Geotechnical Investigations at the Site. It consists of the following:

- Installed ten (10) monitoring wells (BH1, BH2S, BH2D, BH3S, BH3D, BH4, BH5S, BH5D, BH6, BH7) to an approximate depth ranging from 11 meter below ground surface (mbgs) to 19 mbgs and three monitoring wells (BH2S/2D, BH3S/3D and BH5S/5D) are on nested configurations.

The diameter of all monitoring wells is 50 mm. All wells were installed with a flush mount protective casing. Borehole logs and monitoring well installation details are provided in Appendix B. The monitoring well locations are shown on Figure 4.

3.2 Water Level Monitoring

As part of the Hydrogeological Investigation and Water Balance Assessment, static water levels in the monitoring wells were recorded in two (2) monitoring events, including May 31 and June 6 of 2023. A summary of all static water level data as it relates to the elevation survey is given in Table 3-1 below.

The groundwater elevation recorded in the intermediate monitoring wells ranged from 81.04 masl (4.04 mbgs at BH/MW 3S on June 6, 2023) to 83.47 masl (2.91 mbgs at BH/MW 2S on June 6, 2023). The groundwater elevation recorded for the deep wells ranged from 78.51 masl (6.79 mbgs at BH/MW 6 on June 6, 2023) to 82.55 masl (3.83 mbgs at BH/MW 2D on May 31, 2023).

Table 3-1: Summary of Measured Groundwater Elevations

Monitoring Well ID	Ground Surface Elevation (masl)	Approximate Full Well Depth (mbgs)	Depth	31-May-23	6-Jun-23
BH/MW1	85.79	16.55	mbgs	3.42	3.37
			masl	82.37	82.42
BH/MW2S	86.38	12.27	mbgs	2.97	2.91
			masl	83.41	83.47
BH/MW2D	86.38	18.47	mbgs	3.83	3.98
			masl	82.55	82.40
BH/MW3S	85.08	11.41	mbgs	2.10	4.04
			masl	82.98	81.04
BH/MW3D	85.08	17.88	mbgs	4.04	4.04
			masl	81.04	81.04
BH/MW4	85.41	16.32	mbgs	3.97	4.19
			masl	81.44	81.22
BH/MW5S	84.89	10.78	mbgs	2.67	2.62
			masl	82.22	82.27
BH/MW5D	84.89	13.88	mbgs	2.54	2.61
			masl	82.35	82.28
BH/MW6	85.30	18.82	mbgs	3.11	6.79
			masl	82.19	78.51*l
BH/MW7	85.12	18.28	mbgs	3.10	3.59
			masl	82.02	81.53

*not static
mbgs - meters below ground surface
masl - meters above sea level

Two (2) maps were created for the Site to show groundwater contours of the intermediate and deep water-bearing zones (Figures 6 A and 6 B). Accordingly, the groundwater flow directions in the intermediate and deep zones are interpreted to be southeast of the Site, towards Pine Creek, respectively.

Groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing climate conditions. This may also affect the direction and rate of flow. It is recommended to conduct seasonal groundwater level measurements to provide more information on seasonal groundwater level fluctuations.

3.3 Hydraulic Conductivity Testing

Nine (9) Single Well Response Tests (SWRT's) were completed on monitoring wells BH/MW1, BH/MW2S, BH/MW2D, BH/MW3S, BH/MW3D, BH/MW4, BH/MW5S, BH/MW5D and BH/MW7 on June 6, 2023. The tests were completed to estimate the saturated hydraulic conductivity (K) of the soils at the well screen depths utilizing data loggers, preprogramed to take measurement on time in half second intervals.

The static water level within each monitoring well was measured prior to the start of testing. In advance of performing SWRTs, each monitoring well underwent development to remove fines introduced into the screens following construction. The development process involved purging of the monitoring wells to induce the flow of fresh formation water through the screen. Each monitoring well was permitted to fully recover prior to performing SWRTs.

Hydraulic conductivity values were calculated from the SWRT and constant rate test data as per Hvorslev's solution included in the Aqtesolv Pro. V.4.5 software package. The semi-log plots for normalized drawdown versus time are included in Appendix C.

A summary of the hydraulic conductivities (K-values) estimated from the SWRTs are provided in Table 3-2.

Table 3-2: Summary of Hydraulic Conductivity Testing

Monitoring Well ID	Measured Well Depth (mbgs)	Screened Interval (mbgs)	Formation Screened	Estimated Hydraulic Conductivity (m/s)
BH/MW1	16.55	13.55-16.55	Silty Sand Till/Clayey Silt	2.6E-05
BH/MW2S	12.27	9.27-12.27	Sandy Silt Till/Silty Sand Till	8.5E-06
BH/MW2D	18.47	15.47–18.47	Sandy Silt Till	9.1E-05
BH/MW3S	11.41	8.41-11.41	Silty Sand Till	9.6E-05
BH/MW3D	17.88	14.88-17.88	Silty Sand Till	1.1E-04
BH/MW4	16.32	13.32-16.32	Sandy Silt Till	7.9E-07
BH/MW5S	10.78	7.78-10.78	Coarse Sand	4.4E-05
BH/MW5D	13.88	10.88-13.88	Coarse Sand/Sand and Gravel	2.3E-05
BH/MW7	18.28	15.28-18.28	Sandy Silt Till	8.9E-06
			Highest Estimated K Value	1.1E-04
			Geometric Mean of Estimated K Values	3.4E-05
			Arithmetic Mean of Estimated K Values	5.1E-05

SWRTs provide K-estimates of the geological formation surrounding the well screens and may not be representative of bulk formation hydraulic conductivity. As shown in Table 3-2, the highest K-value of the tested water-bearing zone is 1.1E-4 m/s, and the geometric mean and arithmetic mean of the K-values are 3.4E-5 m/s and 5.1E-5 m/s respectively.

The silty sand Till, sand and gravel, and coarse sand deposits belong to the Thorncliffe and Scarborough formations which are regional aquifers. The Till denomination is based on a geotechnical soil description and does not reflect a low permeability deposit as is commonly expected from a Till deposit.

3.4 Groundwater Quality

To assess the suitability for discharging pumped groundwater into the sewers owned by the Durham Region during dewatering activities, one (1) groundwater sample was collected from monitoring well BH1 on June 6, 2020 using a peristaltic pump. Prior to collecting the noted water sample, approximately three (3) standing well volumes of groundwater were purged from the referred well. The samples 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 for analysis to Bureau Veritas Laboratory, a CALA certified independent laboratory in Mississauga, Ontario. Analytical results are provided in Appendix D.

Table 3-3 summarizes exceedance(s) of the Sanitary (Table 1) and Storm (Table 2) Sewer Use By-Law parameters.

When comparing the chemistry of the collected groundwater samples to the Durham Region Sanitary Sewer Discharge Criteria (Table 1), there were no parameter exceedances to be reported.

When comparing the chemistry of the collected groundwater samples to the Durham Region Storm Sewer Discharge Criteria (Table 2) the following parameters reported an exceedance: Total Suspended Solids (TSS).

Reporting detection limits (RDLs) were below the Sewer Use By-Law parameter criteria of Tables 1 and 2.

Table 3-3: Summary of Analytical Results

Parameter	Units	Durham Region Sanitary and Combined Sewer Discharge Limit (Table 1)	Durham Region Storm Sewer Discharge Limit (Table 2)	Concentration BH1 6-Jun-23
Total Suspended Solids (TSS)	mg/L	350	15	**59**

Bold – Exceeds Durham Region Storm Sewer Discharge Limit (Table 2).
Bold & underlined – Exceeds Durham Region Sanitary and Combined Sewer Discharge Limit (Table 1).

For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system. The specifications of the treatment system will need to be adjusted to the reported water quality results by the treatment contractor/process engineer.

For the short-term dewatering discharge to the sanitary sewer system and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the short-term dewatering discharge to the storm sewer system and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

For the long-term dewatering discharge to the sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

The water quality results presented in this report may not be representative of the long-term condition of groundwater quality onsite. As such, regular water quality monitoring is recommended for the post-construction phase, as required by the City.

An agreement to discharge into the sewers owned by the Durham Region will be required prior to releasing dewatering effluent.

The Environmental Site Assessment Report(s) shall be reviewed for more information on the groundwater quality conditions at the Site.

4 Dewatering Assessment

The dimensions of the proposed structure to support the dewatering assessment are summarized in Table 4-1 below.

Table 4-1 Building Dimensions for Dewatering Assessment

Input Parameter	Assumption					Units	Notes
	Parcel A	Parcel B	Parcel C	Parcel D	Parcel E		
Number of Subgrade Levels	3	2	3	3	1	-	
Ground Elevations	85.43	85.43	85.43	85.43	85.43	masl	Average of the borehole elevations on Site
Top of Slab Elevation	74.93	77.93	74.93	74.93	80.93	masl	Based on Underground level plans prepared by Turner Fleischer (November 22, 2024) and used10.5 mbgs for P3, 7.5 mbgs for P2 and 4.5 mbgs for P1 levels as per Turner Fleischer Building Sections (Drawing No. RZ401) dated December 13, 2024
Lowest Footing Elevation	73.43	76.43	73.43	73.43	79.43	masl	Assumed to be approximately 1.5 m below the top of slab elevation
Excavation Area (Length x Width)	(103 x 88)	(116 x 107)	(97 x 77)	(134 x 74)	(193 x 86)	m^2 (m x m)	Approximate area (length x width) based on underground plans prepared by Turner Fleischer (November 22, 2024)
Hydraulic Conductivity (permeability)		5.1 x 10^{-5} m/sec					Average K values for the site to be confirmed with pumping test.

4.1 Dewatering Flow Rate Estimate and Zone of Influence

The Dupuit-Forcheimer equation for radial flow to both sides of an excavation through an unconfined aquifer resting on a horizontal impervious surface was used to obtain a flow rate estimate. Dewatering flow rate is expressed as follows:

$$Q_w = \frac{\pi K(H^2 - h^2)}{Ln\left[\frac{R_o}{r_e}\right]}$$

$$r_e = \frac{a+b}{\pi} \qquad R_o = R_{cj} + r_e$$

Where:

Qw	= Rate of pumping (m^3/s)
X	= Length of excavation (m)
K	= Hydraulic conductivity (m/s)
H	= Hydraulic head beyond the influence of pumping (static groundwater elevation) (m)
h	= Hydraulic head above the base of aquifer in an excavation (m)
R_0	= Radius of influence (m)
R_{cj}	= Cooper-Jacob's radius of influence (m)
r_e	= Equivalent perimeter (m)
a	= Length of the excavation area (m)
b	= Width of the excavation area (m)

It is expected that the initial dewatering rate will be higher to remove groundwater from within the overburden formation. The dewatering rates are expected to decrease once the target water level is achieved in the excavation footprint as groundwater will have been removed, primarily from storage, resulting in lower seepage rates into the excavation.

4.2 Cooper-Jacob's Radius of Influence

The radius of influence (Rcj) for the construction dewatering was calculated based on Cooper-Jacob's equation. This equation is used to predict the distance at which the drawdown resulting from pumping is negligible.

The estimated radius of influence due to pumping is based on Cooper-Jacob's formula as follows:

$$\mathrm{R}_{cj} = \sqrt{2.25KDt/s}$$

Where:

Ro	= Estimated radius of influence (m)
D	= Aquifer thickness (original saturated thickness) (m)
K	= Hydraulic conductivity (m/s)
S	= Storage coefficient
t	= Duration of pumping (s)

4.3 Stormwater

Additional pumping capacity may be required to maintain dry conditions within the excavation during and following significant precipitation events. Therefore, the dewatering rates at the Site should also include removing stormwater from the excavation.

A 25 mm precipitation event was utilized for estimating the stormwater volume. The calculation of the stormwater volume is included in Appendix E.

The estimate of the stormwater volume only accounts for direct precipitation into the excavation. The dimensions of the excavation are considered in the dewatering calculations. Runoff which originated outside of the excavation's footprint is excluded and it should be directed away from the excavation.

During precipitation events greater than 25 mm (ex: 100-year storm), measures should be taken by the contractor to retain stormwater onsite in a safe manner to not exceed the allowable water taking and discharge limits, as necessary. A two (2) and a one hundred (100) year storm event over a 24-hour period are 55.4 and 121.0 mm (refer to Appendix E).

4.4 Results of Dewatering Rate Estimates

4.4.1 Preliminary Construction Dewatering Rate Estimate

Short-term (construction) dewatering calculations are presented in Appendix E.

Pits (elevator, sump pits) are assumed to have the same excavation depth and dewatering target as the main excavation; deeper pits may require localized dewatering and revised dewatering estimates.

Based on the assumptions provided in this report, the results of the dewatering rate estimate can be summarized as follows:

Table 4-2 Summary of Preliminary Construction Dewatering Rate

Description	Peak Dewatering Flow Rate Including Rain Collection Volume				
	Parcel A (3 levels UG) (m3/day)	Parcel B (2 levels UG) **(m^3/day)**	Parcel C (3 levels UG) **(m^3/day)**	Parcels D (3 levels UG) **(m^3/day)**	Parcel E (1 level UG) **(m^3/day)**
Total Volume (m^3/day) Short Term Discharge of Groundwater (Construction dewatering) with Safety Factor (including precipitation) for PTTW	6,537	6,108	6,182	6,869	5,322
Total Volume (m^3/day) Short Term Discharge of Groundwater (Construction dewatering) without Safety Factor (including precipitation)	3,382	3,209	3,185	3,558	2,869
Total Volume (m^3/day) Short Term Discharge of Groundwater (construction dewatering) with Safety Factor (excluding Precipitation)	6,310	5,798	5,996	6,621	4,907

These dewatering estimates are considered preliminary and are based on an average K value. Based on the soil type and highly permeable deposit encountered on site, a pumping test(s) is recommended to provide permeability on a broader scale for the final design of the dewatering system and for permitting.

Caisson walls around the full perimeter of the buildings may be required to reduce the groundwater inflows subject to final design.

The peak dewatering flow rates does not account for flow from utility beddings and variations in hydrogeological properties beyond those encountered during this investigation.

Local dewatering may be required for pits (elevator pits, sump pits, raft) and for localized areas with permeable, soft, or wet soil conditions. Local dewatering is not considered to be part of this assessment, but contractor should be ready to install additional system to manage such conditions. Dewatering estimates should be reviewed once the pit dimensions are available.

All grading around the perimeter of the excavation should be graded away from the shoring the systems and ramp/site access to redirect runoff away from excavation.

If groundwater cutoff systems (ex: caisson walls, sheet piles) are installed, these should be designed for maximal hydrostatic pressure for shallow and deep water levels, without dewatering on the outer side of the groundwater cutoff. Soldier pile and lagging and caisson wall systems should be designed to account for shallow groundwater conditions and take into consideration that dewatering systems may not provide fully dewatered soil conditions.

If groundwater cutoff systems are used for decreasing long-term dewatering rates, these should be designed as permanent structures to cutoff groundwater inflow in the long-term. All perforations should be sealed permanently (ex: tiebacks, breaches, and cold joints) with no leakages and inspected. Fillers should extend into low permeability deposits (ex: sound bedrock or till) to cutoff groundwater from water bearing zones. Inspections should be conducted to confirm the depth of low permeability deposits along shoring system and that fillers are keyed into low permeability soil deposits.

The contractor is responsible for the design of the dewatering systems (depth of wells, screen length, number of wells, spacing sand pack around screens, prevent soil loss etc.) to ensure that dry conditions are always maintained within the excavation at all costs.

Dewatering should be monitored using dedicated monitoring wells within and around the perimeter of the excavation, and these wells should be monitored using manual measurements and with electronic data loggers; records should be maintained on site to track dewatering progress. Discharge rates should be monitored using calibrated flow meters and records of dewatering progress, and daily precipitation as per MECP requirements should be maintained.

4.4.2 Post-Construction Dewatering Rate Estimate

It is our understanding that the development plan includes a permanent foundation sub-drain system that will ultimately discharge to the municipal sewer system if conventional footings are installed.

The long-term dewatering estimates are based on the same equations as construction dewatering shown in Section 4.1.

The calculation for the estimated flow to the future sub-drain system (with no cutoff walls) is provided in Appendix F. The dewatering target for the foundation drainage system is taken at 0.5 m below the lowest slab elevation.

The foundation drain analysis provides a flow rate estimate. Once the foundation drain is built, actual flow rate measurements of the sump discharge will be required to confirm the estimated flow rate.

Based on the assumptions provided in this report, the estimated sub-drain discharge volumes are summarized in Appendix F. Seasonal and daily fluctuations are expected. These estimates may be affected by hydrogeological conditions beyond those encountered at this time, fluctuations in groundwater regimes, surrounding Site alterations, and existing and future infrastructures.

Table 4-3: Summary of Long-Term Dewatering Rate

Long-Term Dewatering Flow Rate	Parcel A (3 levels UG) (m^3/day)	Parcel B (2 levels UG) (m^3/day)	Parcel C (3 levels UG) (m^3/day)	Parcels D (3 levels UG) (m^3/day)	Parcel E (1 level UG) (m^3/day)
Total Volume (m^3/day) Long-Term Drainage of groundwater (from foundation drainage, weeping tiles, sub slab drainage) with Safety Factor Included	2,384	1,975	2,301	2,465	1,369
Long-Term Dewatering Rate without Safety Factor	1,589	1,317	1,534	1,643	913

Intermittent cycling of sump pumps and seasonal fluctuation in groundwater regimes should be considered for pump specifications. A safety factor was applied to the flow rate to account for water level fluctuations due to seasonal changes.

These estimates assume that pits (elevator and/or sump pits) are made as watertight structures (without drainage), if their depths extend below the dewatering target, as previously stated.

The sub-drain rate estimate is based on the assumptions outlined in this report. Any variations in hydrogeological conditions beyond those encountered as part of this investigation may significantly influence the sub-drain discharge volumes.

4.5 MECP Water Taking Permits

4.5.1 Short-Term Discharge Rate (Construction Phase)

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m^3/day but less than 400 m^3 L/day, then an online registration in the Environmental Activity and Sector Registry (EASR) with the MECP will be required. If groundwater dewatering rates onsite exceed 400 m^3/day, a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

As of July 1, 2021, an amendment of O. Reg. 63/16 has come into effect and replaced the former subsection 7 (5) such that the EASR water taking limit of 400 m^3/day would apply to groundwater takings of each dewatered work area only, excluding stormwater.

The dewatering estimate including a safety factor is greater than 400 m^3/day as shown in Table 4-2. The MECP construction dewatering rate excludes the precipitation amount and is the rate used for the permit application. Based on the MECP construction dewatering a PTTW will be required to facilitate the construction dewatering program of the Site.

A Discharge Plan (dewatering sketch, sewer discharge agreement) must be developed and applied for any discharges from the Site. Monitoring of both water quantity and water quality must be carried out for the entire duration of the construction dewatering phase.

The PTTW, Discharge Plan, hydrogeological investigation report, and geotechnical assessment of settlements must also be available at the construction Site during the entire construction dewatering. EXP should be notified immediately about any changes to the construction dewatering schedule or design, since the dewatering rate will need to be updated to reflect these modifications. Altogether, the hydrogeological report, PTTW, Discharge Plan and geotechnical assessment need to be available onsite during the construction dewatering.

4.5.2 Long-Term Discharge Rate (Post Construction Phase)

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m^3/day, then an application for a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

Based on the dewatering estimate shown in Table 4-3 greater than 50 m^3/day, a Category 3 Permit to Take Water (PTTW) will be required to facilitate the post-development phase.

The safety factor for construction (short-term) dewatering is selected larger than for long-term to account for anticipated greater groundwater volumes during initial dewatering. The applied analytical formula is adequate for long-term (steady state) conditions as it omits specific yield and time dependency. When the formula is used for short-term conditions a larger safety factor is recommended to cover a larger initial dewatering rate, which is required to remove stored groundwater. Moreover, a large initial construction dewatering rate is favorable, as it supports reducing the time to reach the dewatering target elevation.

5 Water Balance Assessment (Site Specific)

5.1 Methodology

The Thornthwaite water balance (Thornthwaite, 1948; Mather, 1978; 1979) is a counting method used to analyze the allocation of water among various components of the hydrologic cycle. This methodology was used to complete the pre-construction (existing conditions) and post-development water balance. Inputs to the model are monthly temperature, precipitation, and Site latitude. Outputs include monthly potential and actual evapotranspiration, soil moisture storage, soil moisture storage change, surplus, infiltration, and runoff.

When precipitation (P) occurs, it can either runoff (R) through the surface water system, infiltrate (I) to the water table including an interflow component, or evapo-transpire (ET) from the earth's surface and vegetation. The difference between total precipitation (P) and the total of evaporation and evapotranspiration (ET) is defined to be the water surplus (S) which is available for both infiltration (recharge to the groundwater system including interflow) and for runoff. When long-term averages of P, R, I and ET are used, no net change in groundwater storage (ST) is assumed. Annually, however, there is a potential for small changes in ST. The annual water budget can be stated as follows:

$$\mathbf{P = ET + R + I + ST}$$

Where:

P = precipitation
ET = evapotranspiration
R = surface water runoff
I = infiltration
ST = change in groundwater storage

For this assessment, the Thornthwaite and Mather method was used to estimate average annual infiltration rates.

Infiltration is governed by the surficial soil types, topography, and land cover. If the water table is at surface, as measured in shallow monitoring wells, then the percolation rate of precipitation into the shallow soils is considered negligible.

For ease of calculation, a spreadsheet was used for the computation. A graphical user interface (Thornthwaite Monthly Water-Balance program, 2020) developed by the United Stated Geological Survey (USGS) was applied for the Thornthwaite and Mather Model.

5.2 Meteorological Data

Meteorological data including average monthly precipitation and average temperatures were obtained from the National Climate Data and Information Archive (Environment Canada) for the Oshawa WPCP (Station ID No. 6155878) climatic station (elevation 83.8 masl).

Meteorological data of 30 years from 1977 to 2006 was utilized for the assessment. Summary of input data is provided in Appendix G-1.

5.3 Pre- and Post-Development Site Characteristics

5.3.1 Pre-Development Site Characteristics

A summary of the existing (pre-development) landscape features is provided in Table 5-1:

Table 5-1: Pre-Development (Existing) Land Use

Description	Pre-Construction (Existing) (m^2)	Percentage %
Buildings	20,200	26.1%
Site Area Available for Infiltration (Landscaped)	10,700	13.9%
Impervious (Pavement, water bodies) Surfaces	46,400	60.0%
Total Site Area	77,300	100%

The areas provided in Table 5-1 above were determined based on a review of available Site plans. These estimates are considered appropriate for assessing the water balance. According to Table 5.1, under pre-development conditions 13.9% of the Site area is pervious and available for groundwater infiltration (Figure 7).

5.3.2 Post-Development Site Characteristics

Table 5-2 provides a summary of the post-development Site characteristics.

Table 5-2: Post-Development Site Characteristics

Description	Impervious Areas m^2	Pervious Areas available for Infiltration m^2	Total Areas Post-Construction (Proposed) m^2
Paved Surfaces (roads, sidewalks, parking)	22,900	0	22,900
Building roofs	36,100	0	36,100
Landscaped areas	0	18,300	18,300
Totals	59,000	18,300	77,300
Percentage %	76.3	23.7	100.0

Under post-development conditions, the total pervious area is increased from 13.9% to 23.7% of the total Site area (Tables 5-1 and 5-2, and Figure 8).

5.4 Pre-Development Water Balance Estimates

5.4.1 Climate Data Analysis

The mean annual water surplus was calculated by using the Thornthwaite and Mather (1955) method. Monthly average precipitation values were obtained for 30 years (1977 to 2006) from the National Climate Data and Information Archive (Environment Canada) for the Oshawa WPCP (Station ID No. 6155878) climatic station (elevation 83.8 masl).

Moisture storage of 135 mm/year was assumed for soils and considered to be representative of pre-construction Site conditions. The closest latitude to the Site is 44^0, which was used in the USGS model (2020).

Table 5.3 summarizes the climatic water balance analysis. Appendix G-1 and G-2 provide the model input and output, respectively.

Table 5.3: Climatic Water Balance Analysis in Pre-Development Conditions

Soil Moisture Storage (mm/yr)	Precipitation (mm/yr)	Actual ET (mm/yr)	Surplus (mm/yr)
135 mm/yr	880.6	431.7	448.9

Note: ET = Evapotranspiration

The results of the climatic water balance analysis for the Site suggest that a surplus of 448.9 mm/year of water is available for surface runoff and infiltration.

5.4.2 Infiltration

The infiltration is expected to be controlled by soil type, topography, and soil cover type. Surplus water is portioned between runoff and infiltration based on the controlling factors provided by MOE (1995). The controlling factors provided by the MOE were used for estimating infiltration factors.

Using this method, a total infiltration factor for the Site was estimated by using the individual sub-factors, which are representative of the topography, soil type and land cover conditions (Figures 2 and 7). Appendix G-3 provides a summary of the sub factors and total factor based on the Site conditions. The infiltration sub-factors were determined for estimating pre-development infiltration rates of the entire Site.

The estimated pre-development total infiltration factor of 0.33 (or 33%) represents the fraction of the water surplus available for infiltration. The complementary fraction of the available water for runoff is 0.67. The infiltration factor is utilized to calculate the amount of annual infiltration (in units of m^3/yr) at the Site by multiplying it with the average yearly water surplus estimate and with the Site area available for infiltration.

Applying the infiltration factor of 0.33 and a water surplus of 448.93 mm/yr, the estimated pre-development infiltration rate of the whole Site is 146.35 mm/yr.

In areas with water table at or above surface and less than approximately 1.0 m below surface, the infiltration rate would be considered negligible for existing and proposed grade. Water levels above ground surface or less than 1 m below ground surface were not reported.

5.4.3 Pre-Development Water Balance Analysis

The water balance analysis is based on available information on a regional scale and deemed representative for the Site. Table 5-4 provides the water balance analysis for the Site.

Table 5-4: Pre-Development Water Balance Results

Location	Total Site Area (m^2)	Area Available for Infiltration (m^2)	Total Precipitation (m^3/yr)	Actual Evapo-transpiration (m^3/yr)	Runoff (m^3/yr)	Infiltration (m^3/yr)
Total Site	77,300	10,700	68,072	33,370	33,136	1,566
Percentage of Total Precipitation			100%	49%	49%	2%

The total property area was used to estimate the annual precipitation volume of the Site (Appendix G-4). As summarized in Table 5-4, the breakdown of the pre-development water balance is as follows: 49% of the total precipitation is subject to evapotranspiration, 49% to runoff, and 2% to infiltration.

The pre-development water balance, on a weighted average depth basis (in mm/year) is as follows:

P (880.62) = ET (431.69) + R (428.67) + I (20.26) + ST (0)

5.5 Post-Development Water Balance Estimates

5.5.1 Post-Development Water Balance

Based on the proposed development drawings, the total area of pervious surfaces under post-development conditions is approximately 18,300 m^2, representing approximately 23.7% of the total Site area of 77,300 m^2 (Table 5-2). The remaining 59,000 m^2 will not contribute to infiltration in the post-development phase (approximately 76.3% of the total land area).

Post-development infiltration sub-factors were determined in a similar manner as for estimating infiltration sub-factors for pre-development Site conditions, both based on the method recommended by MOE (1995). For post-development infiltration sub-factors, the landscaped areas were assumed to be consistent with cultivated cover with an infiltration sub-factor of 0.13 (Appendix G-3). The estimated post-development total infiltration factor of 0.33 (or 33%).

Table 5-5 presents a summary of the overall post-development water balance assessment.

Table 5-5: Post-Development Water Balance Forecast

Location	Total Site Area (m^2)	Area Available for Infiltration (m^2)	Total Precipitation (m^3/yr)	Evapo-transpiration (m^3/yr)	Runoff (m^3/yr)	Infiltration (m^3/yr)
Total Site	77,300	18,300	68,072	7,900	57,494	2,678
Percentage of Total Precipitation			100%	11.6	84.5%	3.9%

It is expected that the annual infiltration volume will increase from approximately 1,566 m^3/year to 2,678 m^3/year in post-development, resulting in a surplus of 1,112 m^3/year (Appendix G-4).

The post-development water balance, on a weighted average depth basis (in mm/year) is as follows:

P (880.62) = ET (102.20) + R (743.77) + I (34.65) + ST (0)

Since there is a surplus of 1,112 m^3/year infiltration volume in the post development, no mitigation measure in terms of LID is required.

5.5.2 Mitigation Measures for Wetland Along Pine Creek

Based on the groundwater flow direction, the wetland associated with Pine Creek is located down gradient from the site. Although the proposed development will have more landscaped areas when compared to the existing conditions which is mostly paved, the groundwater contributions to the wetland and Pine creek may be altered if no remediation measures are implemented.

To assess the groundwater contribution to the wetland and Pine Creek, Darcy's equation was utilized which is expressed as follows:

Q=KIA;

where Q = Flow in m^3/s;

K=hydraulic conductivity 5.1 x 10-5 m/s;

I = slope = 0.01 m/m (based on shallow groundwater contours); and

A= cross sectional area 2,250 m^2 perpendicular to groundwater flow direction (Length = 150 m and aquifer thickness = 15m)

Based on above parameters, the groundwater contributions is estimated as 1.15×10^{-3} m^3/s which is approximately 100 m^3/day. In post construction, this volume of water will be returned to the wetland to mitigate potential impacts to the wetland. Caisson walls will be strategically placed along the east portion of the property to mitigate potential impacts from the proposed dewatering.

6 Environmental Impact

6.1 Surface Water Features

The Site is located within the Lake Ontario Waterfront watershed. No surface water features exist onsite. The nearest surface water features are Pine Creek, approximately located 100 meters east of the Site boundary and a wetland associated with Pine Creek. Lake Ontario is approximately 2.2 kms from the Site boundary to the south.

Due to the extent of zone of influence and the distance to the nearest surface water features, potential impacts on surface water features are expected during construction activities.

6.2 Groundwater Sources

Well Records from the MECP Water Well Record (WWR) Database were reviewed to determine the presence and number of water supply wells within a 500 m radius of the Site boundaries. Given that the dewatering zone of influence is limited, no dewatering related impact is expected on the water wells in the area. Based on the date of installation of the water supply wells (12/3/1959 to 12/11/1964) and since the area is municipally serviced, it is unlikely that the noted water supply wells are still active.

6.3 Geotechnical Considerations

As per the MECP technical requirement for PTTW, the geotechnical assessment of the stability of the soils due to water taking (ex: settlement, soil loss, subsidence, etc.) is required. The water taking should not have unacceptable interference on soils and underground structures (foundations, utilities, etc.).

A letter related to geotechnical issues as it pertains to the Site is required to be completed under a separate cover.

6.4 Groundwater Quality

It is our understanding that the potential effluent from the dewatering system during the construction will be released to the municipal sewer system. As such, the quality of groundwater discharge is required to conform the Durham Region Sewer Use By-Law.

Dewatering (short and long-term) may induce migration of contaminants within the zone of influence and beyond due to changing hydraulic gradients, hydrogeological conditions beyond Site boundaries and preferential pathways in utility beddings etc. The water quality sampling conducted as part of this assessment was performed under static conditions. As a result, monitoring may be required during dewatering activities (short and long-term) to monitor potential migration, and this should be performed more frequently during early dewatering stages.

For the short-term dewatering discharge to the sanitary sewer system and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the short-term dewatering discharge to the storm sewer system and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

For the long-term dewatering discharge to the sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

The water quality results presented in this report may not be representative of the long-term condition of groundwater quality onsite. As such, regular water quality monitoring is recommended for the post-construction phase as required by the City.

An agreement to discharge into the sewers owned by the Durham Region will be required prior to releasing dewatering effluent.

The Environmental Site Assessment Report(s) shall be reviewed for more information on the groundwater quality conditions at the Site.

6.5 Well Decommissioning

In conformance with Regulation 903 of the Ontario Water Resources Act, the installation and eventual decommissioning of any dewatering system wells or monitoring wells must be completed by a licensed well contractor. This will be required for all wells that are no longer in use.

7 Conclusions and Recommendations

Based on the findings of the Hydrogeological Investigation and Water Balance Assessment, the following conclusions and recommendations are provided:

- When comparing the chemistry of the collected groundwater samples to the Durham Region Sanitary Sewer Discharge Criteria (Table 1), there were no parameter exceedances to be reported.
- When comparing the chemistry of the collected groundwater samples to the Durham Region Storm Sewer Discharge Criteria (Table 2) the following parameters reported an exceedance: Total Suspended Solids (TSS).
- Based on the assumptions outlined in this report, the estimated peak preliminary dewatering rates for proposed construction activities at Parcels A, B, C, D and E are approximately 6,537 m^3/day, 6,108 m^3/day, 6,182 m^3/day, 6,869 m^3/day and 5,322 m^3/day respectively. These are the rates which will be required to be discharged to the municipal sewer system.
- As the dewatering flow rate estimate is greater than 400 m^3/day, a PTTW will be required to facilitate the construction dewatering program for the Site.
- The long-term flow rate of the foundation sub-drain is estimated to be approximately 2,384 m^3/day, 1,975 m^3/day, 2,301 m^3/day, 2,465 m^3/day and 1,369 m^3/day for Parcels A, B, C, D and E respectively. It is recommended that once the sub-drain system is in place, a flow meter be installed at the sump(s) to record daily discharge volumes during the commissioning stage of the system. Regular maintenance/cleaning of the sub-drain system is recommended to ensure its proper operation. A PTTW will be required for long-term discharge.
- These dewatering estimates are considered preliminary and are based on an average K value. Based on the soil type and highly permeable deposit encountered on site, a pumping test(s) is recommended to provide permeability on a broader scale for the final design of the dewatering system and for permitting.
- Caisson walls around the full perimeter of the buildings may be required to reduce the groundwater inflows subject to final design.
- The construction dewatering and long-term estimate of sub-drain discharge volumes is based on the assumptions outlined in this report. Any variations in hydrogeological conditions beyond those encountered as part of this preliminary investigation may significantly influence the discharge volumes.
- For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system. The specifications of the treatment system will need to be adjusted to the reported water quality results by the treatment contractor/process engineer.
- For the short-term dewatering discharge to the sanitary sewer system and based on the water quality test results, the water is suitable to be discharged without a treatment system.
- For the short-term dewatering discharge to the storm sewer system and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.
- For the long-term dewatering discharge to the sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to be discharged without a treatment system.
- For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

- As per the MECP technical requirement for PTTW, the geotechnical assessment of the stability of the soils due to water taking (ex: settlement, soil loss, subsidence etc.) is required. The water taking should not have unacceptable interference on soils and underground structures (foundations, utilities etc.). A letter related to geotechnical issues as it pertains to the Site is required to be completed under a separate cover.
- An agreement to discharge into the sewers owned by the Durham Region will be required prior to releasing dewatering effluent.
- A Discharge Plan (dewatering sketch, sewer discharge agreement) must be developed and applied for any discharges from the Site. The Discharge Plan and monitoring for both water quantity and water quality must be carried at the Site during the entire construction dewatering phase. The daily water taking records must be maintained onsite for the entire construction dewatering phase. The PTTW, Discharge Plan, hydrogeological investigation report, and geotechnical assessment of settlements must always also be available at the construction Site for the entire construction dewatering. EXP should be notified immediately about any changes to the construction dewatering schedule or design, since EASR will need to be updated to reflect these modifications. The hydrogeological report, PTTW, Discharge Plan and geotechnical assessment constitutes the Water Taking Plan which needs to be available onsite for the duration of construction dewatering.
- Based on water balance assessment for the Site there will be an infiltration surplus of 1,112 m^3/year in post-development. Since there is a surplus of 1,112 m^3/year infiltration volume in the post development, no mitigation measure in terms of LID is required.
- In conformance with Regulation 903 of the Ontario Water Resources Act, the installation and eventual decommissioning of any dewatering system wells or monitoring wells must be completed by a licensed well contractor. This will be required for all wells that are no longer in use.

The conclusions and recommendations provided above should be reviewed in conjunction with the entirety of the report. They assume that the present design concept described throughout the report will proceed to construction. This report is solely intended for the construction and long-term dewatering assessments. Any changes to the design concept may result in a modification to the recommendations provided in this report.

8 Limitations

This report is based on a limited investigation designed to provide information to support an assessment of the current hydrogeological conditions within the study area. The conclusions and recommendations presented within this report reflect Site conditions existing at the time of the assessment. EXP must be contacted immediately, if any unforeseen Site conditions are experienced during construction activities. This will allow EXP to review the new findings and provide appropriate recommendations to allow the construction to proceed in a timely and cost-effective manner.

Our undertaking at EXP, therefore, is to perform our work within limits prescribed by our clients, with the usual thoroughness and competence of the geoscience/engineering profession. No other warranty or representation, either expressed or implied, is included or intended in this report.

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We trust that this information is satisfactory for your purposes. Should you have any questions or comments, please do not hesitate to contact this office.

Sincerely,

EXP Services Inc.

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Figures

Appendix A – MECP WWR Summary Table

Appendix B – Borehole Logs

Appendix C – SWRT Procedures and Results

Appendix D – Laboratory's Certificates of Analysis

Appendix E – Construction Flow Rate Calculations

Appendix F - Post-Construction Flow Rate Calculations

Appendix G – Water Balance Assessment (Site Specific)

Appendix H -Architectural Drawings