April 2024

UD23-045

Functional Servicing and Stormwater Management Report

Project: 4933 Victoria Avenue North
Court Holdings

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PROVINCE OF ONTARIO

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Principal

Identification	Date	Description of issued and/or revision
FSR/SWM Report	March 14th 2024	Issued for Zoning By-law Amendment
FSR/SWM Report	April 29th 2024	Issued for Site Plan Application
FSR/SWM Report	April 29th 2024	Issued for Zoning By-law Amendment

Statement of Conditions

Executive Summary

Lithos Group Inc. (Lithos) was retained by 4933 Vic Court Globizen LP (the "Owner") to prepare a Functional Servicing and Stormwater Management (FSR-SWM) Report in support of a Zoning By-law Amendment (ZBA) for a mixed-use development at 4933 Victoria Avenue North (L0R 2E0), in the Town of Lincoln (the "Town"). The following is a summary of our conclusions:

Storm Drainage

The site stormwater discharge will be controlled to the 5-year pre-development peak flow rate as specified by the Town's Design Standards and Criteria. The proposed development will be connected to the 600 mm diameter storm sewer on Victoria Avenue North at the west of the property, through a 300 mm diameter storm sewer lateral, with a minimum grade of 2% (or equivalent size). In order to achieve the target flow and meet the Town's criteria, quantity controls will be utilized and up to 371.82 m^3 of on-site storage will be required. The on-site storage will be achieved through two (2) underground storage tanks, located at P1 level of the proposed building. The stormwater management (SWM) system will be designed to provide enhanced level (Level 1) protection, as specified by the Ministry of the Environment, Conservation and Parks (MECP). Additional quality control measures will also be required by the MECP, provided by the proposed treatment device, for the driveway area, which is exposed to oil and grit, for a minimum total suspended solids (TSS) removal of 80%.

Sanitary Sewers

The proposed development will be connected to the existing 200 mm diameter sanitary sewer along Victoria Avenue North at the west of the property, through a 200 mm diameter sanitary sewer lateral, with a minimum grade of 2.00% (or equivalent pipe design). The post-development discharge flow from the site is anticipated at approximately 9.07 L/s. Furthermore, the additional net discharge flow from the proposed development is anticipated at approximately 8.04 L/s.

Water Supply

Water supply for the proposed development will be provided by the existing 200 mm diameter watermain on Victoria Avenue North at the west of the property. It is anticipated that a total design flow of 88.15 L/s will be required to support the proposed development. Following an assessment of the Town's provided boundary conditions, it is evident that the existing water infrastructure can sufficiently meet the demands of the proposed development, given that the required flow of 88.15 L/s falls below the specified threshold of 135.00 L/s. The results of the fire hydrant flow test, prepared by Lithos, dated April 24, 2024, reveal that the existing water infrastructure can support the proposed development.

Site Grading

The proposed grades will match current drainage pattern and will improve the existing drainage conditions to meet the Town's/Regional requirements. Grades will be maintained along the property line wherever feasible and overland flow will be directed towards the adjacent right of ways (ROW), as well as the Creek.

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1.0 Introduction

Lithos Group Inc. (Lithos) was retained by 4933 Vic Court Globizen LP (the "Owner") to prepare a Functional Servicing and Stormwater Management (FSR-SWM) Report in support of a Zoning By-law Amendment Application for a proposed mixed-use development at 4933 Victoria Avenue North (L0R 2E0), in the Town of Lincoln (the "Town").

The purpose of this report is to provide site-specific information for the Town's review with respect to infrastructure required to support the proposed development. More specifically, the report will present details on sanitary discharge, water supply and an outline of the storm drainage.

We contacted the Town's engineering department to obtain existing information in preparation of this report. The following documents were available for our review:

- Plan and Profile drawings of Victoria Avenue, drawing No. 02-014-C1, dated March 2003;
- Plan and Profile drawings of Victoria Avenue, drawing No. 02-014-C2, dated March 2003;
- Plan and Profile drawings of Victoria Avenue, drawing No. 92-042-C3, dated March 2003;
- Plan and Profile drawings of Victoria Avenue, drawing No. 92-042-C4, dated March 2003;
- Plan and Profile drawings of Victoria Avenue, drawing No. 92-042-C5, dated March 2003;
- Site Plan and Statistics prepared by gh3*, dated April 26, 2024;
- Topographical Survey prepared by J. D. Barnes., dated February 8, 2023;
- Geotechnical Investigation prepared by Landtek Limited, dated November 3, 2023;
- Slope Stability Assessment Letter report prepared by Landtek Limited, dated November 6, 2023; and,
- Preliminary Hydrogeological Investigation prepared by Landtek Limited, dated November 10, 2023.

2.0 Site Description

The existing site is approximately 1.935 hectares of industrial use land, located in the Town of Lincoln, and it is bounded by Victoria Avenue North to the west, Lake Ontario to the North, a stream to the east, and residential properties on Laurie Avenue to the south (refer to **Figures 1** and **2** following this report). It is currently occupied by a single storey industrial building, which has already been demolished. Please refer to site photographs in **Appendix A** and to the topographic survey in **Appendix B**.

3.0 Site Proposal

The proposed development will be consisted of four mixed-use buildings (a 15-storey Building A, a 15-storey Building B and a 14-storey Building C) and will be serviced by one (1) underground parking level and four (4) above grade parking levels at Building C. The proposed development will be comprised of 396 residential units, 0.897 hectares of hotel area and 0.561 hectares of commercial area. The development will include 46,941.0 m^2 of total Gross Floor Area (GFA). Furthermore, an area of approximately 3,049 m^2 will be dedicated to the Town, for land conveyance. Therefore, the future private property area will be approximately 1.629 ha. Please refer to **Appendix B** for the proposed site plan and building site statistics.

4.0 Terms of Reference and Methodology

4.1. Terms of Reference

The Terms of Reference used for the scope of this report were based on:

- Town of Lincoln “Municipal Design and Quality Standards”, Rev: 6, April 2023;
- Town of Lincoln “Terms of Reference: Stormwater Management Reports and Briefs”, August 2023;
- Town of Lincoln “Low Impact Development and Green Infrastructure Design Guidelines”, May 2020;
- Central Lake Ontario Conservation “Technical Guidelines for Stormwater Management Submissions”, version October, 2020; and,
- Ontario Building Code 2012 (O.B.C.).

4.2. Methodology: Stormwater Drainage and Management

This report will provide an overview of the pre and post-development conditions, and comments on opportunities to reduce peak flows. This is illustrated on a proposed servicing connection plan.

The proposed development will be designed to meet the standards of the Province of Ontario as set out in the Ministry of Environment, Conservation and Parks (MECP) 2003 Stormwater Management Planning and Design Manual (SWMPD). The following design criteria will be reviewed:

- Post-development peak flows from the site will be controlled to the 5-year pre-development peak flow rate;
- Post-development drainage areas to Prudhomme Creek will not match the pre-development drainage areas; and,
- A safe overland flow will be provided for all flows in excess of the 100-year storm event.

4.3. Methodology: Sanitary Discharge

The sanitary sewage discharge from the site is determined using sanitary sewer design sheets that incorporate the land use and building statistics as supplied by the design team. The calculated values provide peak sanitary flow discharge that considers infiltration.
The estimated sanitary discharge flows from the proposed site are calculated based on the criteria shown in **Table 4.1** below.

Table 4.1 – Sanitary Flows

Usage	Design Flow	Units	Population Equivalent
Residential	255	Litres / capita / day	High Density Units = 1.7 ppu
Commercial / Retail	5	Litres / 1.0 m^2 / day	-

4.4. Methodology: Water Usage

The fire flow requirements were estimated using the method prescribed by the Fire Underwriters Survey (FUS) 1999. This method is based on the fire protection of building floors, the type and combustibility of the structural frame and the separation distances with adjoining building units. The domestic water usage was calculated based on the Town's design criteria outlined in **Table 4.2**.

Table 4.2 – Water Usage

Usage	Water Demand	Units
Residential	240	Litres / capita / day
Non-residential	5	Litres / 1.0 m^2 / day

5.0 Stormwater Management and Drainage

The existing site is approximately 1.935 hectares and is currently occupied by a single storey industrial building, which has been already demolished. According to available records, there are three (3) existing storm sewers abutting the subject property. More specifically there are:

- One (1) 600 mm diameter storm sewer on Victoria Avenue North flowing north;
- One (1) 250mm diameter storm sewer on Victoria Avenue North flowing north; and,
- One (1) 250mm diameter storm sewer on Victoria Avenue North flowing south.

5.1. Existing Conditions

According to the Topographic survey provided by R. D. Barnes., dated February 8, 2023, under pre development conditions, the existing site drained towards the Creek east of the site and finally discharged to Lake Ontario.

The existing site is mostly covered by the existing building and an open parking space, therefore there is no significant infiltration on-site. The input drainage parameters, summarized in **Table 5.1** below, are illustrated in the pre-development drainage area plan in **DAP-1** in **Appendix C**.

Table 5.1 – Pre-development Input Parameters

Drainage Area	Drainage Area (ha)	Design "C"	Tc (min.)
A1 Pre (Towards Creek discharged to Lake Ontario)	1.723	0.45	10
A2 Pre (Towards Victoria Avenue North discharged to Lake Ontario)	0.212	0.50	10

Peak flows calculated for the existing conditions are shown in **Table 5.2** below. Detailed calculations are in **Appendix C**.

Table 5.2 – Target Peak Flows

Catchment	Peak Flow Rational Method (L/s)	
	5-year	100-year
A1 Pre	206.4	343.6
A2 Pre	28.2	46.9
Total A Pre	234.6	390.5

As shown in **Table 5.2**, post-development flows towards Victoria Avenue North will need to be controlled to the target flow of 234.6 L/s.

5.2. Stormwater Management

In order to meet the Town's Design Specifications for Storm Drainage, the post-development flow rate is to be controlled to the pre-development five (5)-year target flow as established in **Section 5.1.**

The post-development site will consist of four (4) internal drainage areas:

1. A1 Post – Storm runoff from Green Roof area, controlled in the underground storage tank 1;
2. A2 Post – Storm runoff from the driveway, driven to the OGS device and controlled in the underground storage tank 1;
3. A3 Post – Storm runoff from rooftop, terraces and asphalt areas, controlled in the underground storage tank 2;
4. A4 Post – Uncontrolled Area towards the Creek, discharged to Lake Ontario;
5. A5 Post – Uncontrolled Area towards the Victoria Avenue North, discharged to Lake Ontario.

The post-development drainage areas and runoff coefficients are indicated on **Figure DAP-2**, located in **Appendix C** and summarized in **Table 5.3** below.

Table 5.3 – Post-development Input Parameters

Drainage Area	Drainage Area (ha)	"C"	Tc (min.)
A1 Post	0.305	0.30	10
A2 Post	0.098	0.77	10
A3 Post	0.987	0.87	10
A4 Post	0.222	0.78	10
A5 Post	0.018	0.73	10

5.2.1. Water Balance

The City's WWFMG requires 5 mm of onsite runoff from any rainfall event to be retained over the entirety of the site. A 5 mm of rainfall over the entire site equates to a required water balance volume of 81.50 m^3. Based on the initial abstraction values, the site will provide 33.14 m^3 of initial abstraction in post-development conditions.

The remaining 48.36 m^3 will be provided within the two proposed underground storage tanks (Primary Underground Storage Tank and Secondary Storage Tank), located at P1 and will be used within 72 hours. Consequently, the proposed development will meet the water balance requirement. Detailed calculations about the irrigation requirements will be provided at a later Stage. Please refer to **Appendix C**, for more details. The results of the water balance analysis are summarized in **Table 5.4** below.

Table 5.4 – Water Balance Analysis Results

Total Site Area (m2)	Depth of Rainfall (mm)	Water Balance Requirement (m^3)	Water Balance Provided through Initial Abstraction (m^3)	Water Balance Provided in Tank 1 (m^3)	Water Balance Provided in Tank 2 (m^3)	Total Water Balance Volume Provided (m^3)
1,629	5.0	81.50	33.14	15.50	34.01	49.51

5.2.2. Quantity Controls

As established in **Section 5.1** of this report, storm runoff from the existing property, will be controlled to the 5-year pre-development target flow. Using the Town's intensity-duration-frequency (IDF) data, modified rational method calculations were undertaken to determine the maximum storage required during each storm event. Results for the 5-year and 100-year storm events are provided in **Table 5.5** below. The detailed post-development quantity control calculations are provided in **Appendix C**.

Table 5.5 – Post – Development Quantity Control as per Town's Requirements for 5-year event

Storm Event	Target Flow (L/s)	Uncontrolled Flow (L/s)	Total Site Release Rate (L/s)	Required Storage Tank 1 Volume (m^3)	Required Storage Tank 2 Volume (m^3)	Total Required Storage Tank Volume (m^3)
5 - year	234.6	49.3	87.4	39.02	135.17	174.19
100 - year		82.1	120.1	82.24	289.58	371.82

As shown in **Table 5.5** above, in order to control post-development flows to 5-year pre-development conditions, a target flow of 234.6 L/s is to be satisfied. The minimum required on-site storage is 371.82 m^3 for the 100-year storm event. The buffer zone's grading design aims to maintain the existing conditions to the extent possible. Additionally, potential implementation of supplementary mitigation measures (bioretention swales, infiltration trenches) might be explored at a later stage if deemed necessary.

5.2.3. Underground Storage Tank 1

An underground storage tank is proposed to meet the quantity control requirements set forth by the City's WWFMG. Stormwater from the Green Roof area (**Drainage Area A1 Post**) and from the Driveway and Landscaped areas (**Drainage Area A2 Post**), will be gravity driven into the proposed underground storage tank.

The underground storage tank 1, located at the south-east side of the property (refer to engineering drawing "**SS-01"**, submitted separately), will have a maximum storage of 82.24 m^3 with a maximum active storage depth of 1.88 m (1.26 m of active storage depth above the invert of the outlet pipe, accounting for the quantity control maximum storage of 82.24 m^3, another 0.52 m below the invert of the outlet pipe, accounting for 34.01 m^3 of storage for Water Balance purposes and 0.10 m for Sediment Control purposes), during the 100-year storm event. The total storm runoff after being infiltrated at the treatment device (Stormfilter SFPD 0608), will be pumped into the proposed storm control chamber and finally discharged to the sewer infrastructure along Victoria Avenue North, with a maximum release rate of 3.0 L/s. Refer to **Figure 3**, included in **Appendix C**, as well as to engineering drawing "**SS-01"** (submitted separately), for the storm tank design requirements and dimensions.

5.2.4. Underground Storage Tank 2

Stormwater from the rooftop, terraces and landscaped Areas (**Drainage Area A3 Post**) will be gravity driven into the proposed underground storage tank 2.

The underground storage tank 2, located on the north-east side of the property, (refer to engineering drawing "**SS-01"**, submitted separately), will have a maximum storage of 289.58 m^3 with a maximum active storage depth of 2.07 m (1.87 m of active storage depth above the invert of the outlet pipe, accounting for the quantity control maximum storage of 289.58 m^3, another 0.10 m below the invert of the outlet pipe, accounting for 15.50 m^3 of storage for Water Balance purposes and 0.10 m for Sediment Control purposes), during the 100-year storm event, and will be pumped into the proposed storm control sewer infrastructure along Victoria Avenue North, with a maximum release rate of 35.0 L/s.

Refer to **Figure 3**, included in **Appendix C**, as well as to engineering drawing "**SS-01"** (submitted separately), for the storm tank design requirements and dimensions. Additional details of the tank design and pump requirements will also be provided by the mechanical engineer.

5.2.5. Quality Controls

Stormwater treatment must meet Enhanced Protection criteria as defined by the Municipal Design and Quality Standards", Rev: 6, April 2023, including a minimum 70% of total suspended solids removal (TSS). Based on water quality calculations found in **Appendix C**, an overall TSS removal of 80% is achieved.

Stormwater discharged from the Green Roof area (**Drainage Area A1 Post**) and from the Driveway Area and Landscape Area along Lane (**Drainage Area A2 Post**) that will be directly driven into the underground tank, located at the south – east corner of the property.

The detailed quality control calculations are provided in **Appendix C**. A summary of the site quality control is included in below.

Table 5.6 – TSS Removal

Drainage Area	Drainage Area (ha)	Overall TSS Removal	Additional Quality Control Required
Rooftop / Terraces / Green Roof	1.292	74%	Inherent
Rooftop/Terraces/Landscaped/Hardscaped Areas	0.098	6%	Stormfilter SFPD 0608
Total	**1.390**	**80%**	

5.3. Proposed Storm Connection

The proposed development will connect to the existing 600 mm diameter storm sewer, running along Victoria Avenue North, through a proposed 300 mm diameter storm sewer service connection, with a minimum grade of 2% (or equivalent pipe design). Refer to **SS-01** submitted separately for more details.

The post-development 100-year storm flow will be designed to match the 5-year pre-development target flow; therefore, this development will not negatively affect flow conditions downstream and the existing infrastructure along Victoria Avenue North will be able to support the proposed development.

Flows above the 100-year storm event will be conveyed overland to the adjacent municipal right-of-way (ROW). Refer to engineering drawing "**SG-01**" (submitted separately) for overland flow design in excess of the 100-year storm event.

6.0 Sanitary Drainage System

6.1. Existing Sanitary Drainage System

The existing site is approximately 1.935 hectares, occupied by a single storey industrial building, which has already been demolished, located in the Town of Lincoln. Additionally, there is an outdoor parking space, located on the south of the property. According to available records, there is one (1) sanitary sewer, abutting the subject property. More specifically:

- A 200 mm diameter sanitary sewer located on Victoria Avenue North flowing south.

6.2. Total Pre – Development Flows

Sanitary Flow

Under pre-development conditions sanitary flow from the existing industrial use building, is discharged into the abutting 200 mm sanitary sewer on the east side of Victoria Avenue North.

Table 6.1 – Existing Flows into the sanitary sewer network

	Type of Flow	Existing Flow (L/s)
Victoria Avenue North Street	Existing Sanitary Flow	0.25
	Infiltration	0.77
	Foundation Allowance	-
	Existing Storm Flow (5-year)	-
	Total	**1.02**

6.3. Total Proposed Flows

As per City's design criteria the flow for the proposed development was calculated. **Table 6.1** below shows the total calculated sanitary flow, based on the unit count, retail area and hotel area in the proposed development's statistics. Refer to **Appendix D** for detailed calculations.

Table 6.2 – Proposed Sanitary Flows

Unit Type	Number of Units / Area	Design Guideline Flow Multiplier	Flow (L/sec.)	Total Flow Including Peaking Factor (L/sec.)
223 x 1 Bedroom Units & 153 x 2 Bedroom Units & 20 x 3 Bedroom Units	396	1.7ppu x 255 L /day	1.99	7.76
Commercial Area	5,607 m^2	5 L./m^2/day	0.32	0.32
Hotel Area	8,967 m^2	5 L./m^2/day	0.52	0.52
Infiltration area	1.630	0.286 L/s/ha	0.47	0.47
Total				**9.07**

According to the Site Statistics prepared by gh3*, dated April 26, 2024, as well as the design criteria outlined in **Section 4.3**, the new development will discharge 9.07 L/s (8.60 L/s of sanitary flow and 0.47 L/s of infiltration) into the Town's sanitary sewer network.

The additional flow will be considered within the sanitary discharge rate, therefore, there is an increase in sanitary flow of approximately 8.04 L/s. For detailed calculations, refer to the sanitary sewer design sheet in **Appendix D**. Further to our coordination with the Town, upgrades will be required in order to support the proposed development.

Table 6.3 – Proposed Flows into the sanitary sewer network

	Type of Flow	Total Flow (L/s)
Victoria Avenue North	Proposed Sanitary Flow	**8.60**
	Infiltration	**0.47**
	Foundation Allowance	-
	Proposed Storm Flow (5-year)	-
	Total	**9.07**

6.4. Proposed Sanitary Connection

The proposed development will connect to the existing 200 mm diameter sanitary sewer on Victoria Avenue North via a 200 mm diameter sanitary sewer connection with a minimum grade of 2.00% (or equivalent pipe design). Refer to engineering drawing "**SS-01**" (submitted separately) for details.

7.0 Groundwater Flow

According to the "Hydrogeological Investigation" prepared by Landtek Limited, dated November 10, 2023, the highest water level was determined to be 2.18 mbgs, at 75.52 masl, towards the north east of the property. The groundwater quality results indicate that, all analyzed parameters were within the Niagara Sanitary/Storm Sewers Discharge Limits Discharge Limits. The results of the "Hydrogeological Report" can be found in **Appendix B**.

7.1. Long Term Dewatering

Given that the underground construction will be partially submerged into the existing groundwater table, long-term groundwater discharge, along with the installation of a permanent dewatering system, will be required. According to the Hydrogeological Report prepared by Landtek Limited, dated November 10, 2023, a groundwater flow rate of 27,993 L/day (0.32 L/s) is estimated to be discharged on a permanent basis. The following two options are proposed to implement groundwater control measures for that volume: use of weeping tiles and perimeter drainage to avoid the potential inflow of groundwater into the underground parking level post-construction, subject the approval, or waterproof of the underground parking level below the established "seasonally high groundwater level" plus the required buffer zone (nominally 1.0 m to 1.5 m above). Details will be provided in a later Stage.

7.2. Short Term Dewatering

According to the Hydrogeological Report prepared by Landtek Limited, dated November 10, 2023, short-term groundwater discharge (during construction), outside of periods of active precipitation, is estimated at 27,993 L/day (0.32 L/s). An Environmental Activity and Sector Registry EASR registration and permit to take water (PTTW) will not be required for this volume of water taking, as the estimated water taking is less than 50,000 L/day, respectively. However, temporary discharge application to the Niagara Peninsula Conservation Authority (NPCA) is required and will be provided in a later Stage.

8.0 Water Supply System

8.1. Existing System

Based on plans provided by the Town, there are two (2) watermains abutting the subject site. More specifically:

- A 200 mm diameter watermain on the east side of Victoria Avenue North, starting approximately 150 meters south from the intersection between Victoria Avenue Street and Dustan Street; and

- An abandoned 200 mm diameter watermain on the east side of Victoria Avenue North, starting approximately 150 meters south from the intersection between Victoria Avenue Street and Dustan Street.

8.2. Proposed Water Supply Requirements

The estimated water consumption was calculated based on the occupancy rates shown on **Table 4.2**, according to the Town's Engineering Design Standards and Criteria revised in August 2019. Calculations were conducted to confirm that the proposed site can be supported by the existing water servicing infrastructure.

The fire flow requirements were estimated using the method prescribed by the Fire Underwriters Survey (FUS) be undertaken to assess the minimum requirement for fire suppression. The fire flow calculations are normally conducted for the largest storey, by area, and for the two immediately adjacent storeys.

It is anticipated that an average consumption of approximately 2.83 L/s (244,512 L/day), a maximum daily consumption of 5.65 L/s (488,160 L/day) and a peak hourly demand of 8.48L/s (30,528 L/hr) will be required to service this development with domestic water. Detailed calculations can be found in **Appendix E**.

Having selected the Levels 1, 2 and 3 as a worst-case scenario, we have determined the fire flow demand. **Table 8.1** illustrates the input parameters used. According to our calculations, a minimum fire suppression flow of approximately 82.50 L/s (1,308 USGPM) will be required. Refer to detailed calculations found in **Appendix E**.

Table 8.1 – Fire Flow Input Parameters

Parameter	Frame used for Building	Combustibility of Contents	Presence of Sprinklers	Separation Distance			
				North	West	South	East
Value according to FUS options	Fire Resistive Construction	Limited-Combustible	Yes	>45m	>45m	20.1m - 30m	>45m
Surcharge/reduction from base flow	0.6	25%	50%	0%	0%	10%	0%

In summary, the required design flow is the sum of 'the minimum fire suppression flow' and the total 'maximum daily demand' (82.50 + 5.65 = 88.15 L/s, 1397 USGPM). The results of the hydrant flow test carried out by Lithos Group Inc. on April 24, 2024, along Victoria Avenue North, indicate that 98.08 L/s (1554.38 USGPM) of water is available with a pressure of 138KPa (20.0 psi) revealing that the existing water infrastructure is capable to support the proposed development. The hydrant flow test can be found in **Appendix E.**

8.3. Proposed Watermain Connections

The proposed development will be serviced by one (1) 200 mm diameter fire and one (1) 150 mm diameter domestic water services on Victoria Avenue North. Refer to "**SS-01**" (submitted separately) for more details.

9.0 Site Grading

9.1. Existing Grades

The existing site was previous occupied by a single storey industrial building, which has since been demolished. Under pre-development conditions, no external drainage enters the site and the drainage within the site is conveyed towards the north to the Lake Ontario, as well towards the Creek, east of the site. Refer to **Figures 1** and **2** following this report, site photographs in **Appendix A** and to the topographic survey in **Appendix B**.

9.2. Proposed Grades

The proposed grades will improve the existing drainage conditions to meet the Town's requirements. Grades will be maintained along the property line wherever feasible, and overland flow will be directed towards the adjacent ROWs, as well as the Creek. Major overland flows from the proposed development will be from the two (2) storage perforated access hatches towards the Creek, north – east and south – east of the site; thus, pre-development major overland flow route will be maintained.

10.0 Conclusions and Recommendations

Based on our investigations, we conclude the following:

Storm Drainage

The site stormwater discharge will be controlled to the 5-year pre-development peak flow rate as specified by the Town's Design Standards and Criteria. The proposed development will be connected to the 600 mm diameter storm sewer on Victoria Avenue North at the west of the property, through a 300 mm diameter storm sewer lateral, with a minimum grade of 2% (or equivalent size). In order to achieve the target flow and meet the Town's criteria, quantity controls will be utilized and up to 371.82 m^3 of on-site storage will be required. The on-site storage will be achieved through two (2) underground storage tanks, located at P1 level of the proposed building. The stormwater management (SWM) system will be designed to provide enhanced level (Level 1) protection, as specified by the Ministry of the Environment, Conservation and Parks (MECP). Additional quality control measures will also be required by the MECP, provided by the proposed treatment device, for the driveway area, which is exposed to oil and grit, for a minimum total suspended solids (TSS) removal of 80%.

Sanitary Sewers

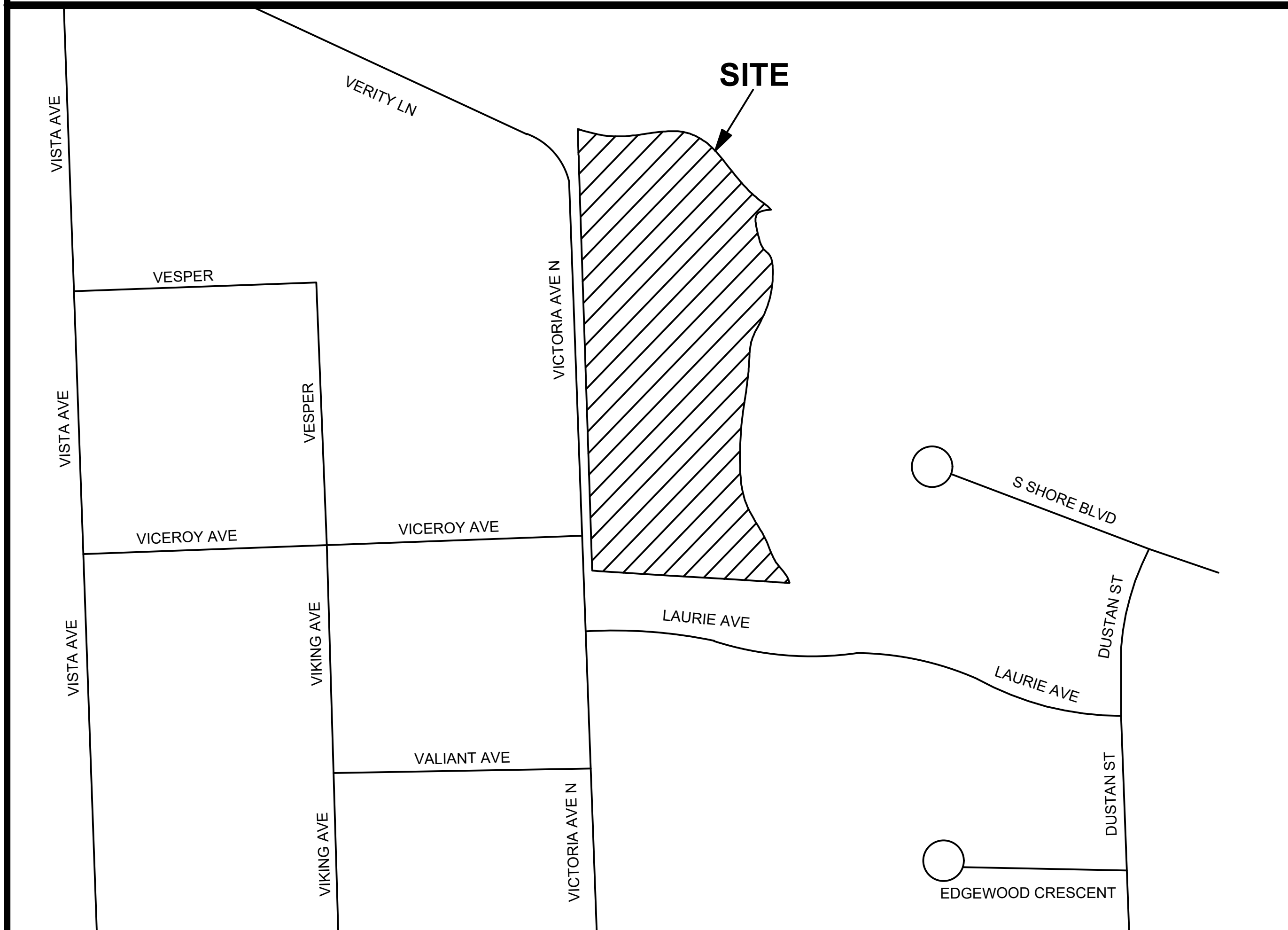
The proposed development will be connected to the existing 200 mm diameter sanitary sewer along Victoria Avenue North at the west of the property, through a 200 mm diameter sanitary sewer lateral, with a minimum grade of 2.00% (or equivalent pipe design). The post-development discharge flow from the site is anticipated at approximately 9.07 L/s. Furthermore, the additional net discharge flow from the proposed development is anticipated at approximately 8.04 L/s.

Water Supply

Water supply for the proposed development will be provided by the existing 200 mm diameter watermain on Victoria Avenue North at the west of the property. It is anticipated that a total design flow of 88.15 L/s will be required to support the proposed development. Following an assessment of the Town's provided boundary conditions, it is evident that the existing water infrastructure can sufficiently meet the demands of the proposed development, given that the required flow of 88.15 L/s falls below the specified threshold of 135.00 L/s. The results of the fire hydrant flow test, prepared by Lithos, dated April 24, 2024, reveal that the existing water infrastructure can support the proposed development.

Site Grading

The proposed grades will match current drainage pattern and will improve the existing drainage conditions to meet the Town's/Regional requirements. Grades will be maintained along the property line wherever feasible and overland flow will be directed towards the adjacent right of ways (ROW), as well as the Creek.



Lithos

150 Bermondsey Road, Toronto, Ontario M4A 1Y1

LOCATION PLAN

MIXED USE DEVELOPMENT
4933 VICTORIA AVENUE NORTH,
LINCOLN, ONTARIO

DATE:	APRIL 2024	PROJECT No:	UD23-045
SCALE:	N.T.S.	FIGURE No:	FIG 1

150 Bermondsey Road, Toronto, Ontario M4A 1Y1

AERIAL PLAN
MIXED USE DEVELOPMENT
4933 VICTORIA AVENUE NORTH,
LINCOLN, ONTARIO

DATE:	APRIL 2024	PROJECT No:	UD23-045
SCALE:	N.T.S.	FIGURE No:	FIG 2

Appendix A

Site Photographs

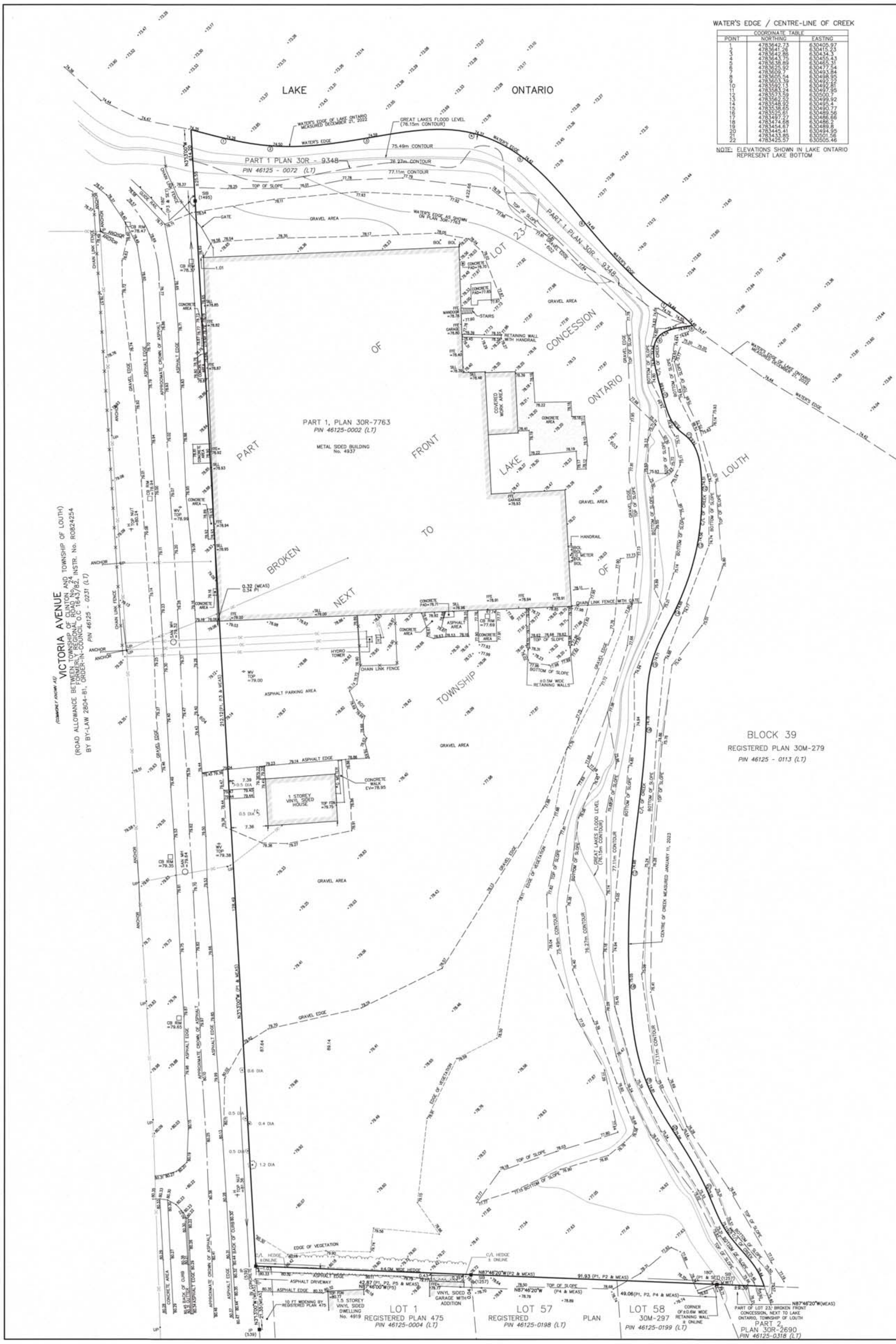
North West corner of the property along Victoria Avenue North facing South East

South West point of the property along Victoria Avenue North facing East

South West corner of the property along Victoria Avenue North facing North East

Appendix B

Background Information



WATER'S EDGE / CENTRE-LINE OF CREEK

COORDINATE TABLE		
POINT	NORTHING	EASTING
1	4783642.73	630405.97
2	4783641.26	630415.23
3	4783642.86	630434.3
4	4783643.75	630455.43
5	4783638.89	630465.31
6	4783625.92	630477.54
7	4783609.7	630493.84
8	4783605.54	630498.95
9	4783603.39	630492.22
10	4783592.13	630492.81
11	4783583.24	630497.95
12	4783573.59	630500.7
13	4783562.52	630499.92
14	4783548.92	630495.4
15	4783538.65	630490.77
16	4783525.61	630489.56
17	4783497.27	630486.66
18	4783474.68	630486.2
19	4783454.67	630489.8
20	4783445.41	630494.95
21	4783433.85	630501.56
22	4783425.57	630505.46

NOTE: ELEVATIONS SHOWN IN LAKE ONTARIO REPRESENT LAKE BOTTOM

PLAN OF SURVEY
WITH TOPOGRAPHIC DETAILS OF

PART OF THE BED OF LAKE ONTARIO IN FRONT OF LOT 23, BROKEN FRONT CONCESSION NEXT TO LAKE ONTARIO

AND

PART OF LOT 23 BROKEN FRONT CONCESSION, NEXT TO LAKE ONTARIO

GEOGRAPHIC TOWNSHIP OF LOUTH

TOWN OF LINCOLN

REGIONAL MUNICIPALITY OF NIAGARA

SCALE 1 : 300

5 0 5 10 20 metres

J.D. BARNES LIMITED

METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

NOTES

BEARINGS ARE UTM GRID, DERIVED FROM GNSS OBSERVATIONS, UTM ZONE 17, NAD83 (CSRS) (2010.0).

DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999802.

FOR BEARING COMPARISONS, A ROTATION OF 1°04'00" COUNTER-CLOCKWISE WAS APPLIED TO BEARINGS ON P1.
FOR BEARING COMPARISONS, A ROTATION OF 1°04'30" COUNTER-CLOCKWISE WAS APPLIED TO BEARINGS ON P2.
FOR BEARING COMPARISONS, A ROTATION OF 1°05'00" COUNTER-CLOCKWISE WAS APPLIED TO BEARINGS ON P5.

ELEVATION NOTE

ELEVATIONS ARE OF GEODETIC ORIGIN (CGVD-1928:78), AND ARE DERIVED FROM GNSS OBSERVATIONS AND NATURAL RESOURCES CANADA'S GEOID MODEL HT2.0.

TOPOGRAPHIC LEGEND

CB	DENOTES CATCHBASIN
SAN MH	DENOTES SANITATION MANHOLE
HT	DENOTES HYDRO TRANSFORMER
G METER	DENOTES GAS METER
BOL	DENOTES BOLLARD
UP	DENOTES UTILITY POLE
H	DENOTES FIRE HYDRANT
WV	DENOTES WATER VALVE
C/L	DENOTES CENTERLINE
OC	DENOTES OVERHEAD HYDRO CABLE
	DENOTES FENCE
	DENOTES CONIFEROUS TREE
	DENOTES DECIDUOUS TREE

LEGEND

■	DENOTES	SURVEY MONUMENT FOUND
SIB	DENOTES	STANDARD IRON BAR
IP	DENOTES	IRON PIPE
WIT	DENOTES	WITNESS
P1	DENOTES	PLAN 30R-7763
P2	DENOTES	PLAN 30R-2258
P3	DENOTES	PLAN 30R-9348
P4	DENOTES	PLAN 30M-297
P5	DENOTES	PLAN 475
539	DENOTES	KIRKUP & URE SURVEYING LTD. O.L.S.
1257	DENOTES	R. LAROCQUE, O.L.S.
1495	DENOTES	P.D. REITSMA SURVEYING LTD.
MEAS	DENOTES	MEASURED
——	DENOTES	PROPERTY LINE

N=NORTH / S=SOUTH / E=EAST / W=WEST

SURVEYOR'S CERTIFICATE

I CERTIFY THAT:
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON JANUARY 26, 2023.

February 8, 2023
DATE

DASHA PAGE
ONTARIO LAND SURVEYOR

THIS PLAN OF SURVEY RELATES TO AOLS PLAN SUBMISSION FORM #V-39660 AND IS NOT VALID UNLESS EMBOSSED BY A SURVEYOR'S SEAL

J.D.BARNES LIMITED
SURVEYING MAPPING GIS
LAND INFORMATION SPECIALISTS
4318 PORTAGE ROAD - UNIT 2, NIAGARA FALLS, ON L2E 6A4
T: (905) 358-3693 F: (905) 358-6224 www.jdbarnes.com

DRAWN BY:	CHECKED BY:	REFERENCE NO.:
SC	BM / DP	22-16-360-00

FILE: G:\2022\22-16-360\00\Drawing\22-16-360-POS-TOPO.dgn

PLOTTED: 2/8/2023

1.0 SUMMARY

TOTAL SITE AREA 19,348.3 m²
NET SITE AREA 16,299.1 m²
LAND CONVEYANCE PART 1 3,035.7 m²
LAND CONVEYANCE PART 2 13.5 m²

TOTAL GFA - 46,941 m²
FSI - 2.88

1.1 BUILDING AREA SUMMARY		
Total Non-Residential GFA (m²)	Residential GFA (m²)	Total GFA (m²)
13,058	33,883	46,941

**EXCLUDING ANY PARKING AREAS OR LOADING AREAS WITHIN THE BUILDING OR STRUCTURE.*

1.2 PARKING SUMMARY				
RESIDENTIAL	VISITOR	OUTDOOR VISITOR	STREET	Total Car Parking
427	197	13	21	658

DESIGNATED P.O.P.S.	1797 m²

2.0 BUILDING HEIGHTS

Building A: 50.8 m (15 Storeys)
Building B: 48.3 m (15 Storeys)
Building C: 47.8 m (14 Storeys)
Building C TOWN: 18.3 m (5 Storeys)

COVERAGE

BUILDING COVERAGE 8502m²
PERCENTAGE OF NET SITE AREA 52.1%

3.0 FLOOR AREA

3.1 GROSS FLOOR AREA	Total Non-Residential GFA		Residential GFA (m²)	TOTAL GFA (m²)
	Retail GFA (m²)	Hotel GFA (m²)		
NORTH PHASE				
BELOW-GRADE NON-PARKING				
Level P1	0	0	2,007	2,156
	0	**0**	**2,007**	**2,156**
BLDG 'A'				
Level 1	194	0	1,708	1,902
Level 2	0	0	1,524	1,524
Level 3	0	0	1,786	1,786
Level 4	0	0	1,308	1,308
Level 5	0	0	1,308	1,308
Level 6	0	0	1,138	1,138
Level 7	0	0	972	972
Level 8	0	0	972	972
Level 9	0	0	972	972
Level 10	0	0	801	801
Level 11	0	0	635	635
Level 12	0	0	635	635
Level 13	0	0	635	635
Level 14	0	0	465	465
Level 15	0	0	299	299
	194	**0**	**15,160**	**15,354**
BLDG 'B'				
Level 1	1,749	310	0	2,059
Level 2	0	1,801	0	1,801
Level 3	827	493	0	1,320
Level 4	799	47	0	846
Level 5	0	628	0	628
Level 6	0	628	0	628
Level 7	0	628	0	628
Level 8	0	628	0	628
Level 9	0	628	0	628
Level 10	0	628	0	628
Level 11	0	628	0	628
Level 12	0	628	0	628
Level 13	0	628	0	628
Level 14	0	628	0	628
Level 15	373	32	0	405
	3,748	**8,967**	**0**	**12,715**
SOUTH PHASE				
BLDG 'C'				
Level 1	0	0	1,392	1,392
BLDG C - Level 1M	0	0	1,452	1,452
Level 2	0	0	1,489	1,489
Level 3	0	0	1,425	1,425
Level 4	0	0	2,531	2,531
Level 5	0	0	2,303	2,303
Level 6	0	0	1,120	1,120
Level 7	0	0	972	972
Level 8	0	0	972	972
Level 9	0	0	764	764
Level 10	0	0	635	635
Level 11	0	0	635	635
Level 12	0	0	428	428
Level 13	0	0	299	299
Level 14	0	0	299	299
	0	**0**	**16,716**	**16,716**

GRAND TOTAL	Total Non-Residential GFA		Residential GFA (m²)	Total GFA (m²)
	Retail GFA (m²)	Hotel GFA (m²)		
	3,942	8,967	33,883	46,941

3.2 PARKING FLOOR AREA (EXCLUDED FROM GFA)	Total Excluded GFA		Total Excluded GFA (m²)
	Loading Area (m²)	Parking Area (m²)	
NORTH PHASE			
BELOW-GRADE PARKING			
Level P1	0	11,284	11,284
	0	**11,284**	**11,284**
SOUTH PHASE			
ABOVE-GRADE PARKING			
Level 1	249	2,284	2,533
BLDG C - Level 1M	0	2,074	2,074
Level 2	0	2,153	2,153
Level 3	0	2,490	2,490
	249	**9,001**	**9,250**
GRAND TOTAL	**249**	**20,285**	**20,534**

TOTAL GFA =
*WITH REFERENCE TO THE TOWN OF LINCOLN COMPREHENSIVE ZONING BY-LAW NO.2022-50, SECTION 2.110:
THE TOTAL FLOOR AREA OF ALL STOREYS OF A BUILDING OR STRUCTURE, INCLUDING THE FLOOR ARE OF ANY BASEMENT, WHICH FLOOR AREAS ARE MEASURED BETWEEN THE EXTERIOR FACES OF THE EXTERIOR WALLS OF A BUILDING AT EACH FLOOR LEVEL, BUT EXCLUDING ANY PARKING AREAS OR LOADING AREAS WITHIN THE BUILDING OR STRUCTURE.

ALL INTERIOR FLOOR AREAS ABOVE GRADE
INCL. CORES/CORRIDORS/VERTICAL SHAFTS
EXCL. PARKING / LOADING
EXCL. BALCONIES
EXCL. ROOF TERRACE / OUTDOOR AMENITY

TOTAL NON-RESIDENTIAL GFA =
RETAIL GFA + HOTEL GFA

RETAIL GFA=
RESTAURANT & KITCHEN
CAFE
GROCER
SPA + BOH
SKY BAR + BOH
LOBBY BAR
COMMON LOBBY

HOTEL GFA =
ALL INTERIOR HOTEL FLOOR AREAS
EVENT SPACE
ALL HOTEL BOH

4.0 UNITS

4.1 RESIDENTIAL UNITS

4.1 RESIDENTIAL UNITS	LEVEL	NEW RESIDENTIAL			UNIT COUNT
		1B	2B	3B	
BLDG 'A'					
NORTH PHASE	Level 1	9	2	0	11
NORTH PHASE	Level 2	16	4	1	21
NORTH PHASE	Level 3	16	6	1	23
NORTH PHASE	Level 4	13	6	1	20
NORTH PHASE	Level 5	10	6	2	18
NORTH PHASE	Level 6	11	5	1	17
NORTH PHASE	Level 7	7	6	1	14
NORTH PHASE	Level 8	7	6	1	14
NORTH PHASE	Level 9	5	5	2	12
NORTH PHASE	Level 10	4	7	0	11
NORTH PHASE	Level 11	2	6	0	8
NORTH PHASE	Level 12	2	6	0	8
NORTH PHASE	Level 13	2	6	0	8
NORTH PHASE	Level 14	0	4	1	5
NORTH PHASE	Level 15	0	2	0	2
		104	**77**	**11**	**192**
BLDG 'C'					
SOUTH PHASE	Level 1	2	12	0	14
SOUTH PHASE	Level 2	3	11	2	16
SOUTH PHASE	Level 3	3	11	2	16
SOUTH PHASE	Level 4	25	8	2	35
SOUTH PHASE	Level 5	29	7	0	36
SOUTH PHASE	Level 6	11	4	1	16
SOUTH PHASE	Level 7	10	3	1	14
SOUTH PHASE	Level 8	9	4	1	14
SOUTH PHASE	Level 9	9	3	0	12
SOUTH PHASE	Level 10	9	1	0	10
SOUTH PHASE	Level 11	9	1	0	10
SOUTH PHASE	Level 12	0	5	0	5
SOUTH PHASE	Level 13	0	3	0	3
SOUTH PHASE	Level 14	0	3	0	3
		119	**76**	**9**	**204**
TOTAL RESIDENTIAL UNITS		**223**	**153**	**20**	**396**

4.2 NON-RESIDENTIAL UNITS

4.2 HOTEL SUITES	LEVEL	NEW HOTEL SUITES		UNIT COUNT
		SINGLE SUITE	DOUBLE SUITE	
BLDG 'B'				
NORTH PHASE	Level 5	8	5	13
NORTH PHASE	Level 6	8	5	13
NORTH PHASE	Level 7	8	5	13
NORTH PHASE	Level 8	8	5	13
NORTH PHASE	Level 9	8	5	13
NORTH PHASE	Level 10	8	5	13
NORTH PHASE	Level 11	8	5	13
NORTH PHASE	Level 12	8	5	13
NORTH PHASE	Level 13	8	5	13
NORTH PHASE	Level 14	8	5	13
TOTAL HOTEL SUITES		**80**	**50**	**130**

4.2 COMMERCIAL UNITS		
Name	Level	AREA
* DOES NOT INCLUDE "BACK OF HOUSE" SPACES		
BLDG 'A'		
RETAIL	Level 1	194 m²
		194 m²
BLDG 'B'		
LOBBY-BAR	Level 1	118 m²
RESTAURANT	Level 1	818 m²
RETAIL	Level 1	133 m²
EVENT	Level 2	999 m²
SPA	Level 4	799 m²
SKY BAR	Level 15	197 m²
		3064 m²
TOTAL COMMERCIAL UNIT AREA		**3258 m²**

4.2 COMMERCIAL UNITS - BACK-OF-HOUSE		
Name	Level	AREA
NORTH PHASE		
BOH - HOTEL	<varies>	1070 m²
BOH - KITCHEN	Level 1	276 m²
BOH - SKY BAR	Level 15	176 m²
BOH - SPA	Level 3	279 m²
BOH - SPA MECHANICAL	Level 3	548 m²
		2349 m²
TOTAL COMMERCIAL BOH AREA		**2349 m²**

4.3 UNIT MIX

NEW RESIDENTIAL - UNIT MIX

BLDG 'A'

4.3 UNIT MIX SUMMARY - BUILDING 'A'	Name	No. Units	Unit Mix	Avg Area	Avg Area (SF)	Total Area
NORTH PHASE	1B	104	54%	50 m²	543 SF	5245 m²
NORTH PHASE	2B	77	40%	71 m²	764 SF	5463 m²
NORTH PHASE	3B	11	6%	103 m²	1111 SF	1135 m²

BLDG 'C'

4.3 UNIT MIX SUMMARY - BUILDING 'C'	Name	No. Units	Unit Mix	Avg Area	Avg Area (SF)	Total Area
SOUTH PHASE	1B	119	58%	51 m²	551 SF	6144 m²
SOUTH PHASE	2B	76	37%	69 m²	742 SF	6136 m²
SOUTH PHASE	3B	9	4%	91 m²	983 SF	919 m²

TOTAL RESIDENTIAL - UNIT MIX

4.3 TOTAL RESIDENTIAL UNIT MIX SUMMARY					
Name	No. Units	Unit Mix	Avg Area	Avg Area (SF)	Total Area
1B	223	56%	51 m²	547 SF	11336 m²
2B	153	39%	70 m²	753 SF	10699 m²
3B	20	5%	98 m²	1054 SF	1957 m²
TOTAL	396	100%			23992 m²

NEW HOTEL SUITES - UNIT MIX

BLDG 'B'

4.3 UNIT MIX SUMMARY - BUILDING 'B'	Name	No. Units	Unit Mix	Avg Area	Avg Area (SF)	Total Area
NORTH PHASE	HOTEL SUITE - DOUBLE	50	38%	40 m²	434 SF	2014 m²
NORTH PHASE	HOTEL SUITE - SINGLE	80	62%	36 m²	386 SF	2867 m²
TOTAL		130	100%			4881 m²

5.0 AMENITY

5.1 AMENITY	AMENITY		
TOTAL UNITS	OUTDOOR	INDOOR	TOTAL AREA
NORTH PHASE			
192	815 m²	979 m²	1794 m²
SOUTH PHASE			
204	165 m²	38 m²	203 m²

6.0 PARKING

6.1 CAR PARKING						
Level	RESIDENTIAL	VISITOR	OUTDOOR VISITOR	TOTAL ON-SITE PARKING	STREET	TOTAL CAR PARKING
BLDG C - Level 3	73	0	0	73	0	73
BLDG C - Level 2	58	0	0	58	0	58
BLDG C - Level 1M	54	0	0	54	0	54
BLDG C - Level 1	52	0	13	65	21	86
Level P1	190	197	0	387	0	387
TOTAL	**427**	**197**	**13**	**637**	**21**	**658**

6.2 BICYCLE PARKING	Bicycle Parking		
Level	Long-Term	Short-Term	TOTAL BICYCLE PARKING
BLDG C - Level 1	160	0	160
TOTAL	**160**	**0**	**160**

7.0 LOADING

7.1 LOADING		
Level	Count	Type
Level 1	1	TYPE 'G' LOADING
Level 1	1	TYPE 'B' LOADING
Level P1	3	TYPE 'C' LOADING

8.0 WASTE

8.1 WASTE STORAGE AREA			
Area Type	Level	Name	Area
NORTH PHASE	Level P1	RES. WASTE	150 m²
NORTH PHASE	Level P1	RES. WASTE	135 m²
NORTH PHASE	Level P1	NON-RES. WASTE	65 m²
			350 m²

GENERAL NOTES:
1. Drawings are not to be scaled. Contractor will verify all existing conditions and dimensions required to perform the Work and will report any discrepancies with the Contract Documents to the Architect before commencing work.
2. The Architectural Drawings are to be read in conjunction with all other Contract Documents including the Project Manuals and the Structural, Mechanical and Electrical Drawings. In cases of difference between the Consultants' documents with respect to the quantity, sizes or scope of work, the greater shall apply.
3. Positions of exposed or finished Mechanical or Electrical devices, fittings and fixtures are indicated on the Architectural Drawings. Locations shown on the Architectural Drawings shall govern over Mechanical and Electrical Drawings. Mechanical and Electrical items not clearly located will be located as directed by the Architect.
4. Dimensions indicated are taken between the faces of finished surfaces unless otherwise noted.
5. The architect has not been retained for supervision of construction and assumes no responsibility for means, methods and techniques of construction.
6. These documents are not to be used for construction unless specifically noted for such purpose.

Rev.	Date	Issued
2	2024/04/26	REISSUED FOR ZBA
1	2024/02/23	ISSUED FOR ZBA

gh3
gh3*
55 OSSINGTON AVE, SUITE 100
Toronto, ON, Canada M6J 2Y9
416 915 1791

Globizen
VINELAND

4933 VICTORIA AVENUE NORTH

SCALE 1 : 1
PROJECT NO. 202302
ISSUE DATE APRIL, 26, 2024

PROJECT STATISTICS

A001

PARKING LEGEND & NOTES

NOTE: ALL PARKING LEVELS MUST COMPLY WITH THE REQUIREMENTS OF THE TORONTO MUNICIPAL CODE CHAPTER 629, PROPERTY STANDARDS

GENERAL MAINTENANCE:
THE WALLS, FLOOR, CEILINGS AND COLUMNS SHALL BE MAINTAINED FREE OF HOLES, BREAKS, OR CRACKS, AND KEPT CLEAN BY PERIODIC BRUSHING, WASHING AND PAINTING, AND EVERY WALL AND COLUMN SHALL BE PAINTED WHITE FROM A LEVEL OF 24 INCHES ABOVE THE FLOOR TO THE CEILING AND EVERY CEILING SHALL BE PAINTED WHITE.
WALLS FROM SLAB LEVEL TO 24 INCHES SHALL BE PAINTED BLACK.

LIGHTING STANDARDS:
PROVIDE ILLUMINATION BY ARTIFICIAL MEANS AT A MINIMUM MAINTENANCE LEVEL OF ILLUMINATION OF FIVE FOOT CANDLES (54 LUX), AT A LEVEL OF ILLUMINATION IN WHICH THE UNIFORMITY RATIO DOES NOT EXCEED 4:1.

PROTECTION AND CLEANING OF LIGHTING FIXTURES:
ALL LIGHTING FIXTURES SHALL BE PROTECTED FROM DAMAGE BY THE PROVISION OF REQUIRED WIRED GLASS OR OTHER SUITABLE MEANS OF PROTECTION AND SHALL BE MAINTAINED IN A CLEAN CONDITION.

FASTENINGS ON REQUIRED EXIT DOORS:
EVERY EXIT DOOR SHALL OPEN IN THE DIRECTION OF EGRESS FROM THE GARAGE WITHOUT THE USE OF A KEY OR OTHER DEVICE AND EQUIPPED WITH A LATCHING DEVICE THAT WILL RELEASE THE LATCH AND ALLOW THE DOOR TO OPEN FULLY WHEN A FORCE OF NOT MORE THAN 20 POUNDS IS APPLIED TO THE DEVICE IN THE DIRECTION OF EXIT TRAVEL.

FASTENINGS ON ENTRANCE DOORS:
EVERY ACCESS DOOR SHALL BE EQUIPPED WITH A LATCH THAT WILL PREVENT ENTRY TO THE GARAGE EXCEPT BY THE USE OF A KEY, CODED CARD OR SIMILAR DEVICE AND A SELF-CLOSING DEVICE DESIGNED TO RETURN THE DOOR TO THE CLOSED AND LATCHED POSITION AFTER EACH USE.

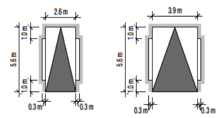
SIGNAGE:
LARGE SAFE-EXIT ARROWS (SEE DRAWING) SHALL BE DISPLAYED ON SAFE-EXIT DOORS 1,500 MILLIMETRES ABOVE THE FLOOR, MEASURED FROM THE CENTER OF THE ARROW TO THE FLOOR, WITH THE ARROW POINTING DOWN.

SMALL SAFE-EXIT ARROWS (SEE DRAWING) SHALL BE PROMINENTLY DISPLAYED ON COLUMNS OR WALLS 1,500 MILLIMETERS ABOVE THE FLOOR, MEASURED FROM THE CENTER OF THE ARROW TO THE FLOOR, AT LEAST EVERY TEN METERS ALONG THE SAFE-EXIT ROUTE, AT ALL SAFE-EXIT ROUTE DECISION POINTS ALONG THE SAFE-EXIT ROUTE, AND WHEREVER A SAFE-EXIT ROUTE CROSSES A TRAFFIC AISLE.

THE FOLLOWING SHALL BE COLOURED GREEN TO MATCH 1965 CANADIAN GOVERNMENT SPECIFICATION BOARD STANDARD PAINT COLOUR 503-326 FROM THE DOCUMENT ENTITLED "STANDARD PAINT COLOURS 1-GP-12c": THE SAFE-EXIT DOOR, THE FRAME OF THE SAFE-EXIT DOOR AND WALL ADJACENT TO THE SAFE-EXIT DOOR TO A DISTANCE OF ONE METRE ON EITHER SIDE OF THE FRAME, AND TO A HEIGHT OF 3,000 MILLIMETRES ABOVE THE FLOOR OR TO THE SOFFIT ABOVE THE BULKHEAD OVER THE DOOR.

ALERT SIGNS (SEE DRAWING) SHALL BE PROMINENTLY DISPLAYED ON COLUMNS OR WALLS 2,100 MILLIMETRES ABOVE THE FLOOR, MEASURED FROM THE TOP OF THE SIGN TO THE FLOOR, ON THE BASIS OF ONE ALERT SIGN FOR EVERY 25 PARKING STALLS IN THE GARAGE, WITH THE ALERT SIGNS BEING EVENLY DISTRIBUTED IN THE GARAGE AND IN ANY EVENT BEING NO CLOSER THAN 20 METRES APART FROM EACH OTHER.

TYPICAL PARKING SPOT / **TYPICAL ACCESSIBLE PARKING SPOT**



NOTE: PROVIDE AN ADDITIONAL 300MM FOR PARKING SPACE WIDTH WHEN OBSTRUCTIONS OCCUR BETWEEN THE FRONT AND REAR 1000mm

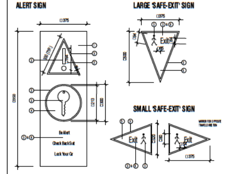
ABBREVIATIONS

R	RESIDENTIAL PARKING STALL
V	VISITOR PARKING STALL
C	NON-RESIDENTIAL PARKING STALL
EV	ELECTRIC VEHICLE SUPPLY EQUIPPED PARKING STALL

LEGEND

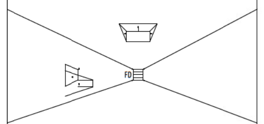
1. **GREY** - CANADIAN GOVERNMENT SPEC. BOARD STANDARD PAINT COLOUR NO. 501-210
2. **YELLOW** - CANADIAN GOVERNMENT SPEC. BOARD STANDARD PAINT COLOUR NO. 505-335
3. **WHITE** - STANDARD PAINT COLOURS 1 - GP - 12C
4. **BLACK** - STANDARD PAINT COLOURS 1 - GP - 12C
5. **GREEN** - CANADIAN GOVERNMENT SPEC. BOARD STANDARD PAINT COLOUR NO. 503-326
6. HELVETICA BOLD CONDENSED

ALERT SIGN / **LARGE 'SAFE-EXIT' SIGN** / **SMALL 'SAFE-EXIT' SIGN**



CONVEX MIRROR

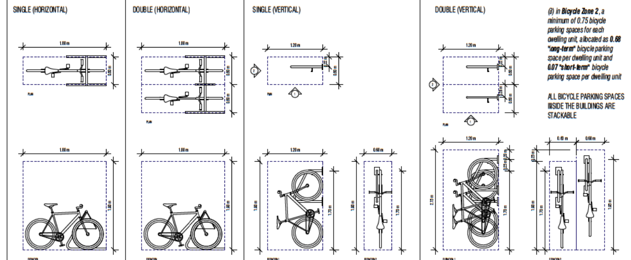
1. CONVEX MIRRORS, IN ADDITION TO THOSE SHOWN ON PLANS, ARE TO BE INSTALLED AND MAINTAINED BY THE OWNER(S) AT ALL RIGHT-ANGLED TURNS AND POSITIONED IN SUCH A MANNER AS TO PROVIDE MOTORISTS WITH CLEAR VIEWS OF ONCOMING TRAFFIC.
2. SLOPES AROUND BASE OF COLUMNS AT PARKING LEVELS



LOCAL SLOPE OF 2% IN THE SLAB AT THE BASE OF ALL INTERMEDIATE COLUMNS TO BE PROVIDED WITH MIN. RISE OF 30mm AT THE BASE AS SHOWN HERE

BICYCLE STORAGE

SINGLE (HORIZONTAL) / DOUBLE (HORIZONTAL) / SINGLE (VERTICAL) / DOUBLE (VERTICAL)

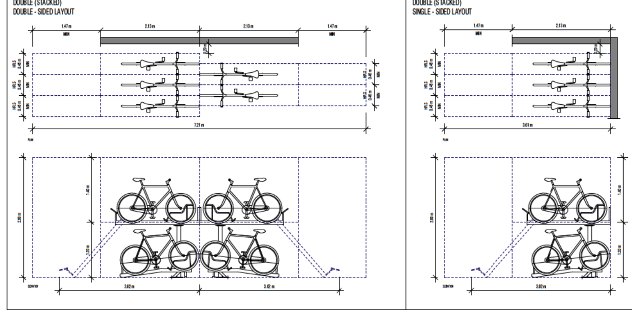


*(B) in **Bicycle Zone 2**, a minimum of 0.75 bicycle parking spaces for each dwelling unit, allocated as **0.68 "long-term"** bicycle parking space per dwelling unit and **0.07 "short-term"** bicycle parking space per dwelling unit*

ALL BICYCLE PARKING SPACES INSIDE THE BUILDINGS ARE STACKABLE

DOUBLE (STACKED)
DOUBLE - SIDED LAYOUT

DOUBLE (STACKED)
SINGLE - SIDED LAYOUT



From Table 230.5 10.1(1), City of Toronto Zoning By-law 569-2013, as amended

Long-Term Commercial Space Rates

*(B) in Bicycle Zone 2 is **0.13** for each 100 square metres of interior floor area used for an **eating establishment.***

*(B) in Bicycle Zone 2 is **0.13** for each 100 square metres of interior floor area used for an **office, other than a medical office.***

*(B) in Bicycle Zone 2 is **0.1** for each 100 square metres of interior floor area used for a **medical office.***

*(B) in Bicycle Zone 2 is **0.13** for each 100 square metres of interior floor area used for a **personal service shop.***

*(B) in Bicycle Zone 2 is **0.13** for each 100 square metres of interior floor area used for a **retail store.***

Short-Term Commercial Space Rates

*(B) in Bicycle Zone 2 is **3 plus 0.25** bicycle parking spaces for each 100 square metres of interior floor area used for an **eating establishment.***

*(B) in Bicycle Zone 2 is **3 plus 0.15** bicycle parking spaces for each 100 square metres of interior floor area used for an **office, other than a medical office.***

*(B) in Bicycle Zone 2 is **3 plus 0.1** bicycle parking spaces for each 100 square metres of interior floor area used for a **medical office.***

*(B) in Bicycle Zone 2 is **3 plus 0.25** bicycle parking spaces for each 100 square metres of interior floor area used for a **personal service shop.***

*(B) in Bicycle Zone 2 is **3 plus 0.25** bicycle parking spaces for each 100 square metres of interior floor area used for a **retail store.***

(4) Multiple uses on a lot
If Table 230.5 10.1(1) Bicycle Parking Space Rates, requires a bicycle parking space for one or more uses on a lot, the total number of bicycle parking spaces required is equal to the cumulative total of all bicycle parking spaces required for each use on the lot.

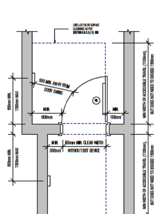
(7) Change and Shower Facilities
*If a building has uses, other than dwelling units, for which a "long-term" bicycle parking space is required, shower and change facilities must be provided for **each gender** at the following rate:*

(A) none if less than 5 required "long-term" bicycle parking spaces;
(B) 1 for 5 to 60 required "long-term" bicycle parking spaces;
(C) 2 for 61 to 120 required "long-term" bicycle parking spaces;
(D) 3 for 121 to 180 required "long-term" bicycle parking spaces; and
(E) 4 for more than 180 required "long-term" bicycle parking spaces.

BARRIER FREE DOORS

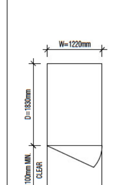
GENERAL NOTES:

1. ALL DOORS IN BARRIER FREE PATH OF TRAVEL MAINTAIN THE MINIMUM CLEARENCES NOTED IN THE DIAGRAM AS PER OBC SEC. 3.8.3.3. UNLESS EQUIPPED WITH AUTOMATIC DOOR OPENERS.
2. COORDINATE ALL EXIT DEVICES AND DOOR THICKNESS TO MAINTAIN A MINIMUM BARRIER FREE PATH OF TRAVEL AS NOTED IN THE DIAGRAM AS PER OBC SEC. 3.8.3.3.
3. ALL DOORS THAT OCCUR IN A BARRIER FREE PATH OF TRAVEL ARE TO RECIEVE A LEVER TYPE DOOR HANDLES THAT MEET THE REQUIREMENTS OF THE OBC.
4. THRESHOLD IN A BARRIER FREE PATH OF TRAVEL IS TO BE MAXIMUM 13mm HIGH.



STORAGE LOCKERS

*TYPICAL STORAGE LOCKER:
1830mm X 1220mm
UNLESS OTHERWISE NOTED



GENERAL NOTES:
1. Drawings are not to be scaled. Contractor will verify all existing conditions and dimensions required to perform the Work and will report any discrepancies with the Contract Documents to the Architect before commencing work.
2. The Architectural Drawings are to be read in conjunction with all other Contract Documents including the Project Manuals and the Structural, Mechanical and Electrical Drawings. In cases of difference between the Consultants' documents with respect to the quantity, sizes or scope of work, the greater shall apply.
3. Positions of exposed or finished Mechanical or Electrical devices, fittings and fixtures are indicated on the Architectural Drawings. Locations shown on the Architectural Drawings shall govern over Mechanical and Electrical Drawings. Mechanical and Electrical items not clearly located will be located as directed by the Architect.
4. Dimensions indicated are taken between the faces of finished surfaces unless otherwise noted.
5. The architect has not been retained for supervision of construction and assumes no responsibility for means, methods and techniques of construction.
6. These documents are not to be used for construction unless specifically noted for such purpose.

Rev.	Date	Issued
2	2024/04/26	REISSUED FOR ZBA
1	2024/02/23	ISSUED FOR ZBA

gh3
gh3*
55 OSSINGTON AVE, SUITE 100
Toronto, ON, Canada M6J 2Y9
416 915 1791

Globizen
VINELAND

4933 VICTORIA AVENUE NORTH

Project North True North

SCALE As indicated
PROJECT NO. 202302
ISSUE DATE APRIL, 26, 2024

PROJECT STATISTICS

A002

2 A301 A103

ROOF SITE PLAN

1 : 400

1.0 SUMMARY

TOTAL SITE AREA	19,348.3 m²
NET SITE AREA	16,299.1 m²
LAND CONVEYANCE PART 1	3,035.7 m²
LAND CONVEYANCE PART 2	13.5 m²

TOTAL GFA - 46,941 m²
FSI - 2.88

PROPOSED BUILDING SUMMARY		
BUILDING	BUILDING TYPE	TOTAL GFA (m²)
BELOW-GRADE NON-PARKING		2,156 m²
BLDG 'A'	RESIDENTIAL	15,354 m²
BLDG 'B'	HOTEL & RETAIL	12,715 m²
BLDG 'C'	RESIDENTIAL	16,716 m²
TOTAL		46,941 m²

2.0 BUILDING HEIGHTS

Building A: 50.8 m (15 Storeys)
Building B: 48.3 m (15 Storeys)
Building C: 47.8 m (14 Storeys)
Building C TOWN: 18.3 m (5 Storeys)

COVERAGE

BUILDING COVERAGE	8502m²
PERCENTAGE OF NET SITE AREA	52.1%

PARKING & LOADING

PARKING SUMMARY				
RESIDENTIAL	VISITOR	OUTDOOR VISITOR	STREET	Total Car Parking
427	197	13	21	658

LOADING SUMMARY	
Type	Count
TYPE 'G' LOADING	1
TYPE 'B' LOADING	1
TYPE 'C' LOADING	3

LANDSCAPED AREA

LANDSCAPED AREA SUMMARY	
	Area
DESIGNATED P.O.P.S.	1797 m²
PUBLIC LANDSCAPED AREA	2638 m²
TOTAL	4435 m²

PERCENTAGE OF NET SITE AREA 27.2%

*WITH REFERENCE TO THE TOWN OF LINCOLN COMPREHENSIVE ZONING BY-LAW NO.2022-50, SECTION 2.151:
LANDSCAPED AREA OR LANDSCAPED STRIP means a permeable area not built upon and not used for any purpose other than as a landscaped area which may include grass, shrubs, flowers, trees and similar types of vegetation and decorative paths, decorative walkways, fences and similar appurtenances, but does not include parking areas, driveways, service walkways or ramps. The words "landscaping" and "landscaped" shall have the same meaning.

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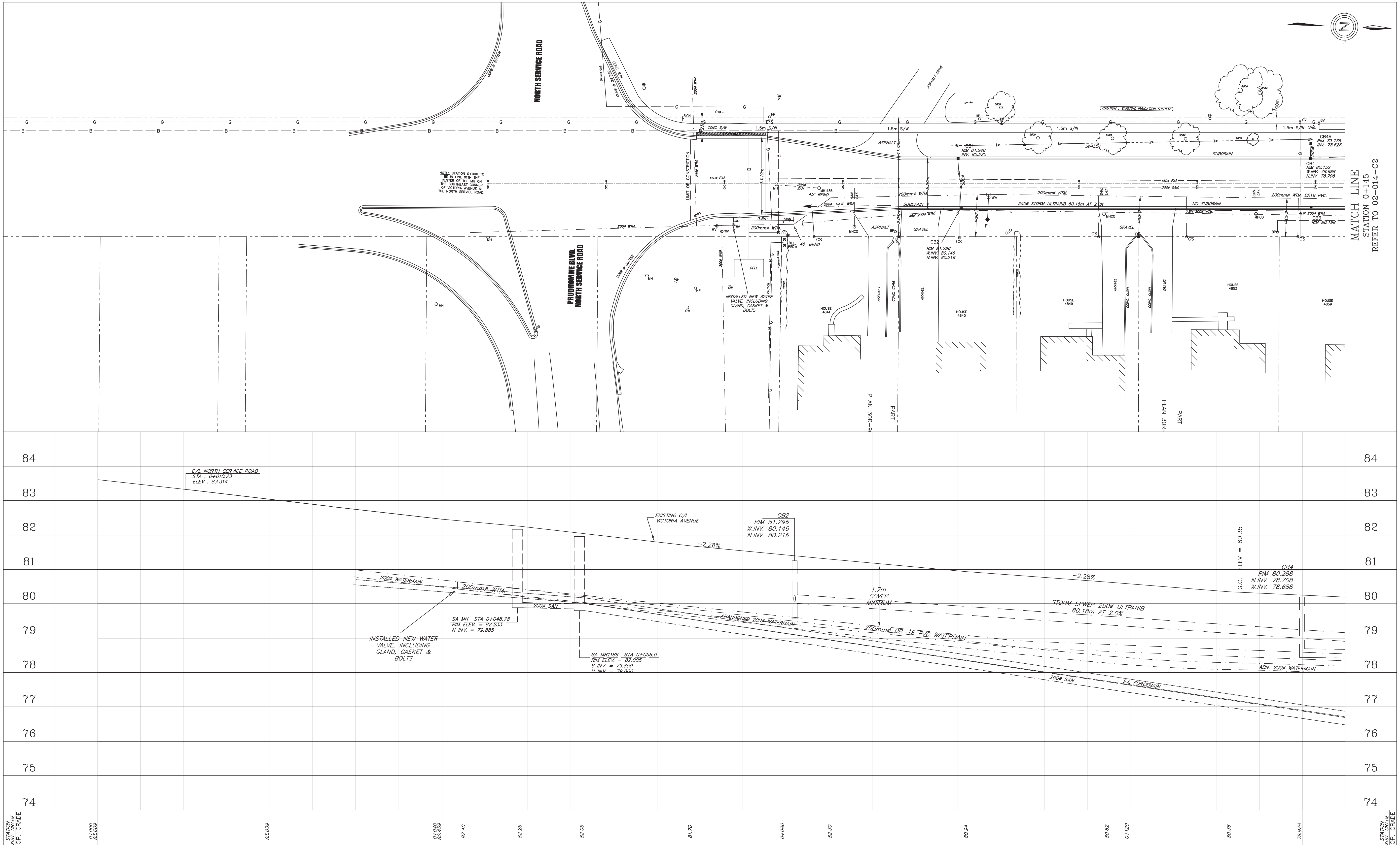
Project North True North

SCALE As indicated
PROJECT NO. 202302
ISSUE DATE APRIL, 26, 2024

ROOF SITE PLAN

A103



NO.	REVISION	DATE	INIT.
3	RECORD OF CONSTRUCTION	MAR. 03	RB
2	ISSUED FOR TENDER	MAY /02	JH
1	ISSUED FOR TOWN COMMENTS	MAY /02	JH

NOTES

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LEGEND

EXISTING SYMBOLS		PROPOSED SYMBOLS	
SIB□	STANDARD IRON BAR	CB1 ■	CATCH BASIN
IB■	IRON BAR	FH	FIRE HYDRANT
MH○	MAINTENANCE HOLE	WV	WATER VALVE
CBA□	CATCHBASIN	CS ⊗	CURB STOP
MHCO○	MANHOLE CLEANOUT		
FH	FIRE HYDRANT		
WV⊗	WATER VALVE		
CS⊗	CURB STOP		
GV	GAS VALVE		
GW HP○	HYDRO POLE & GUY WIRE		
LS○	LIGHT STANDARD		

DRAFTING: J.H.
DESIGN: R.Beaulieu C.E.T.
CHECKED BY: J.Jaeger P.Eng.
PROJ. SUPVR.: W.A.G.Mackay P.Eng.

KERRY T. HOWE ENGINEERING LTD.

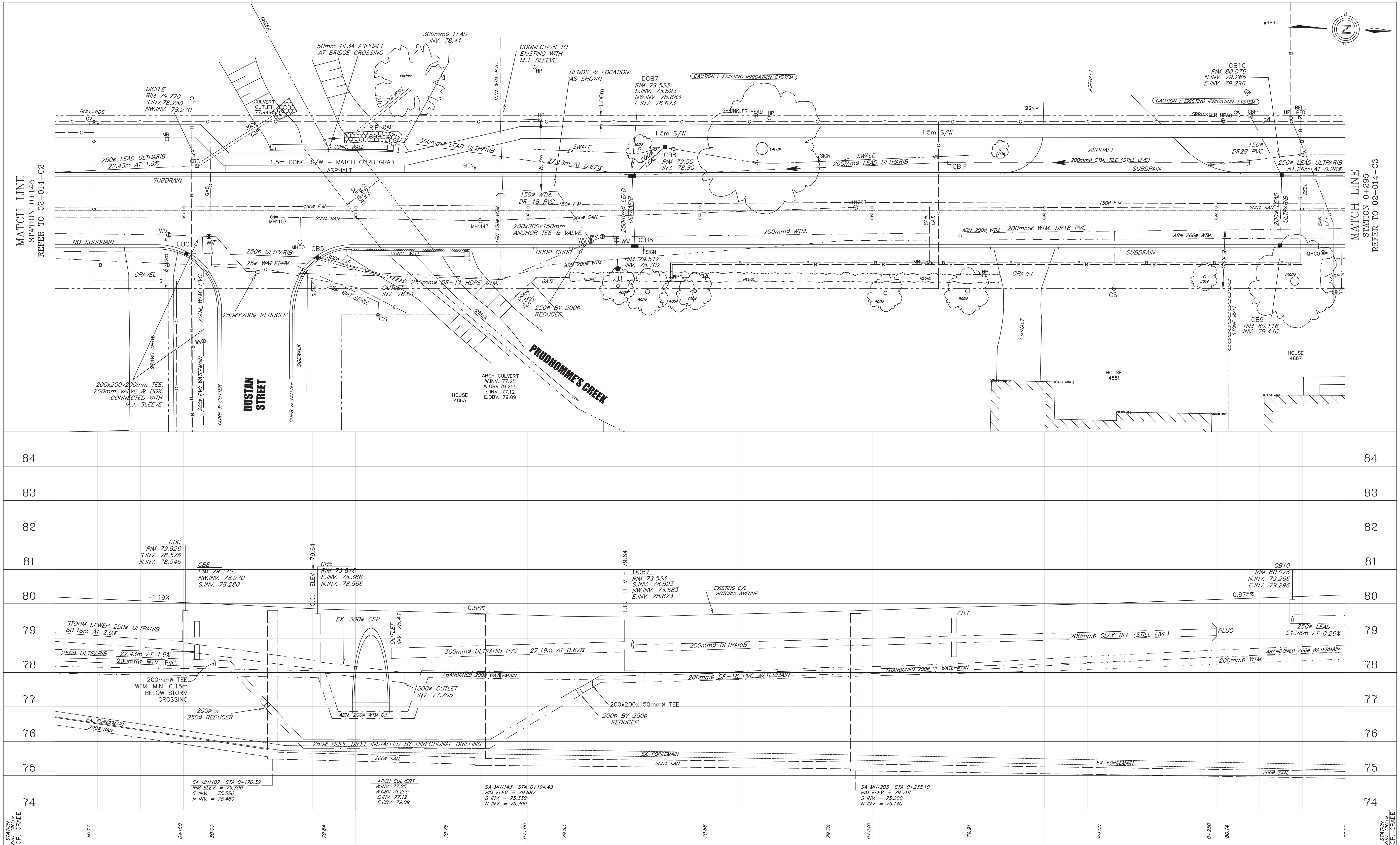
TOWN OF LINCOLN
1970

KERRY T. HOWE ENGINEERING LIMITED
CONSULTING ENGINEERS
St. Catharines ,Ontario
(905) 688-6550

BENCH MARK DATUM
TOP OF THE NORTH ARM OF FIRE HYDRANT NUMBER 303; ON THE EAST SIDE OF VICTORIA AVENUE, ±40m NORTH OF THE NORTH SERVICE ROAD. ELEVATION 82.051

TOWN OF LINCOLN
VICTORIA AVENUE
ROAD RECONSTRUCTION & WATERMAIN REPLACEMENT
NORTH SERVICE ROAD to ±150m NORTH OF LAURIE AVE.
STA. 0+000 to 0+145

FIELD NOTES: KTH Book No. 773
PLOT DATE: MARCH 18, 2003
SCALE: HOR. 1:200 VERT. 1:50
DWG. No. 02-014-C1
REV. 3
CAD FILE: 02014ASBUILT.dwg



NO.	REVISION	DATE	INIT.
3	RECORD OF CONSTRUCTION	MAR. 03	RB
2	ISSUED FOR TENDER	MAY /02	JH
1	ISSUED FOR TOWN COMMENTS	MAY /02	JH

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CS ⊗	CURB STOP		
GV	GAS VALVE		
GW HP○	HYDRO POLE & GUY WIRE		
LS○	LIGHT STANDARD		

DRAFTING J.H.
DESIGN R.Beaulieu C.E.T.
CHECKED BY J.Jaeger P.Eng.
PROJ. SUPVR. W.A.G.Mackay P.Eng.

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BENCH MARK DATUM
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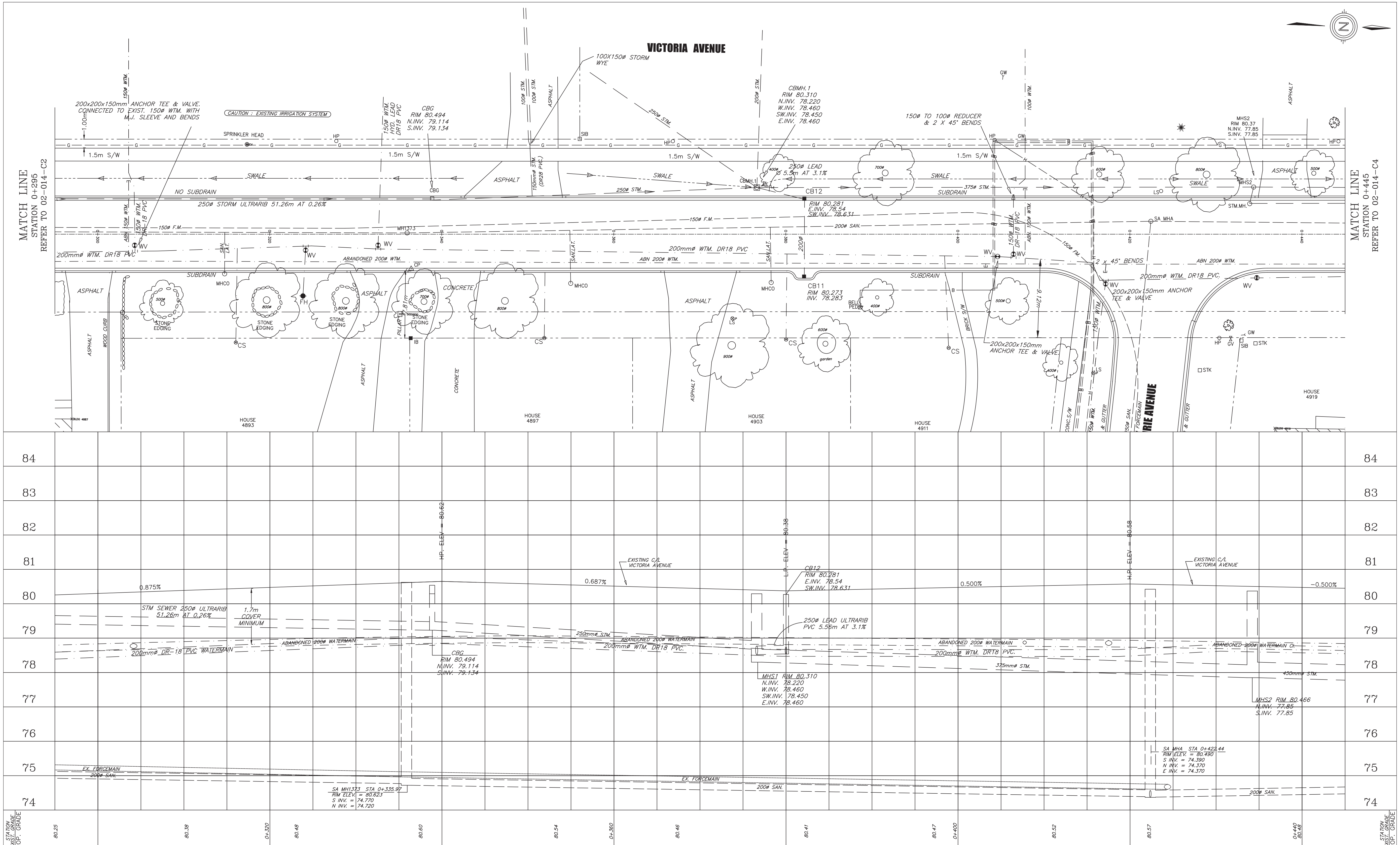
TOWN OF LINCOLN
VICTORIA AVENUE
ROAD RECONSTRUCTION & WATERMAIN REPLACEMENT
NORTH SERVICE ROAD to ±150m NORTH OF LAURIE AVE.
STA. 0+145 to 0+295

FIELD NOTES KTH Book No. 773
PLOT DATE MARCH 18, 2003
SCALE HOR. 1:200 VERT. 1:50
DWG. No. 02–014–C2
REV. 3
CAD FILE: 02014ASBUILT.dwg



STATION / EXIST. GRADE / PROP. GRADE	80.25	80.38	0+320	80.48	80.60	80.54	0+360	80.46	80.41	80.47	0+400	80.52	80.57	0+440 80.48

NO.	REVISION	DATE	INIT.
3	RECORD OF CONSTRUCTION	MAR. 03	RB
2	ISSUED FOR TENDER	MAY /02	JH
1	ISSUED FOR TOWN COMMENTS	MAY /02	JH

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LS	LIGHT STANDARD		

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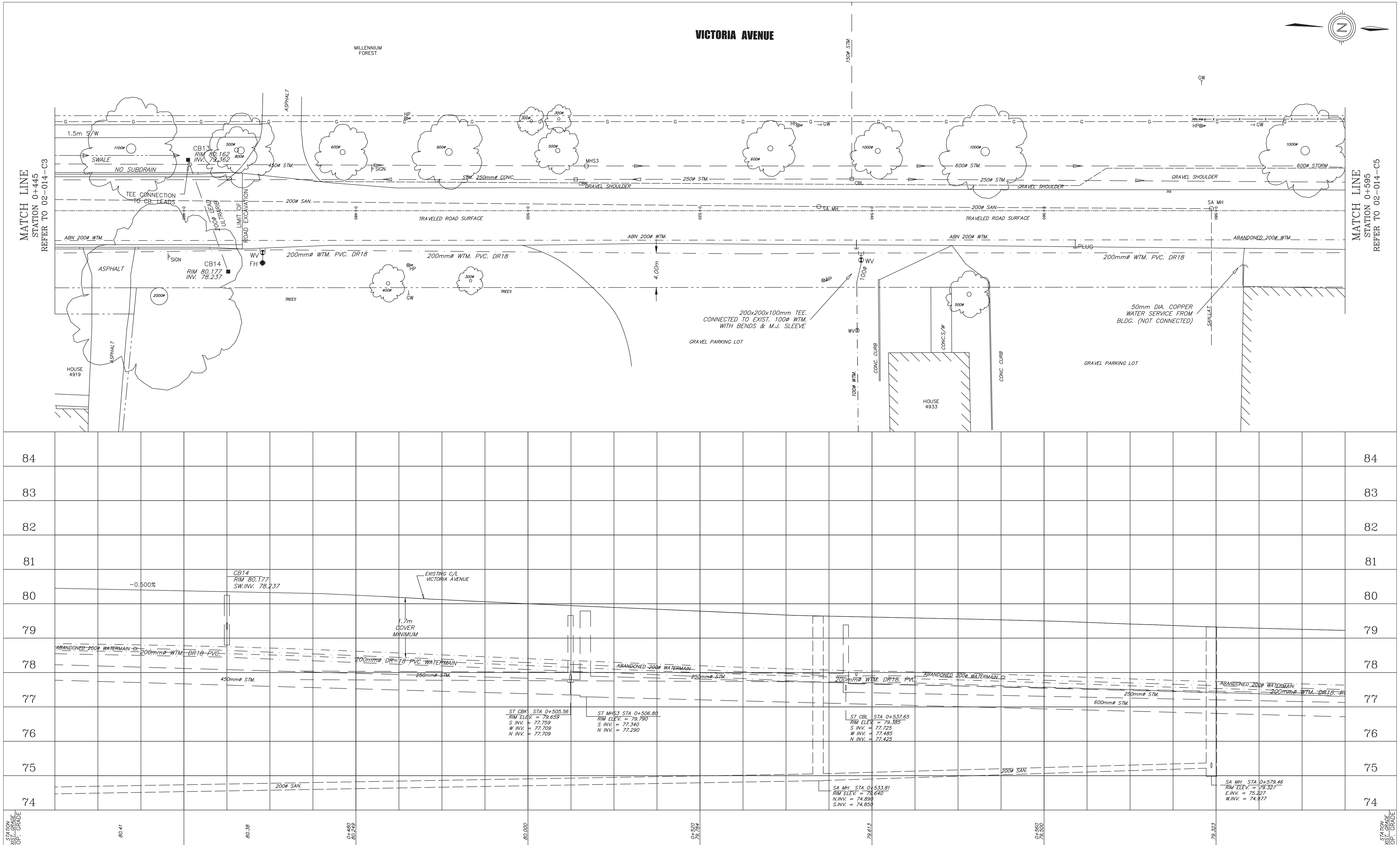
VICTORIA AVENUE
ROAD RECONSTRUCTION & WATERMAIN REPLACEMENT
NORTH SERVICE ROAD to ±150m NORTH OF LAURIE AVE.
STA. 0+295 to 0+445

FIELD NOTES KTH Book No. 773
PLOT DATE MARCH 18, 2003
SCALE HOR. 1:200 VERT. 1:50
DWG. No. 02-014-C3
REV. 3
CAD FILE: 02014ASBUILT.dwg



NO.	REVISION	DATE	INIT.
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2	ISSUED FOR TENDER	MAY /02	JH
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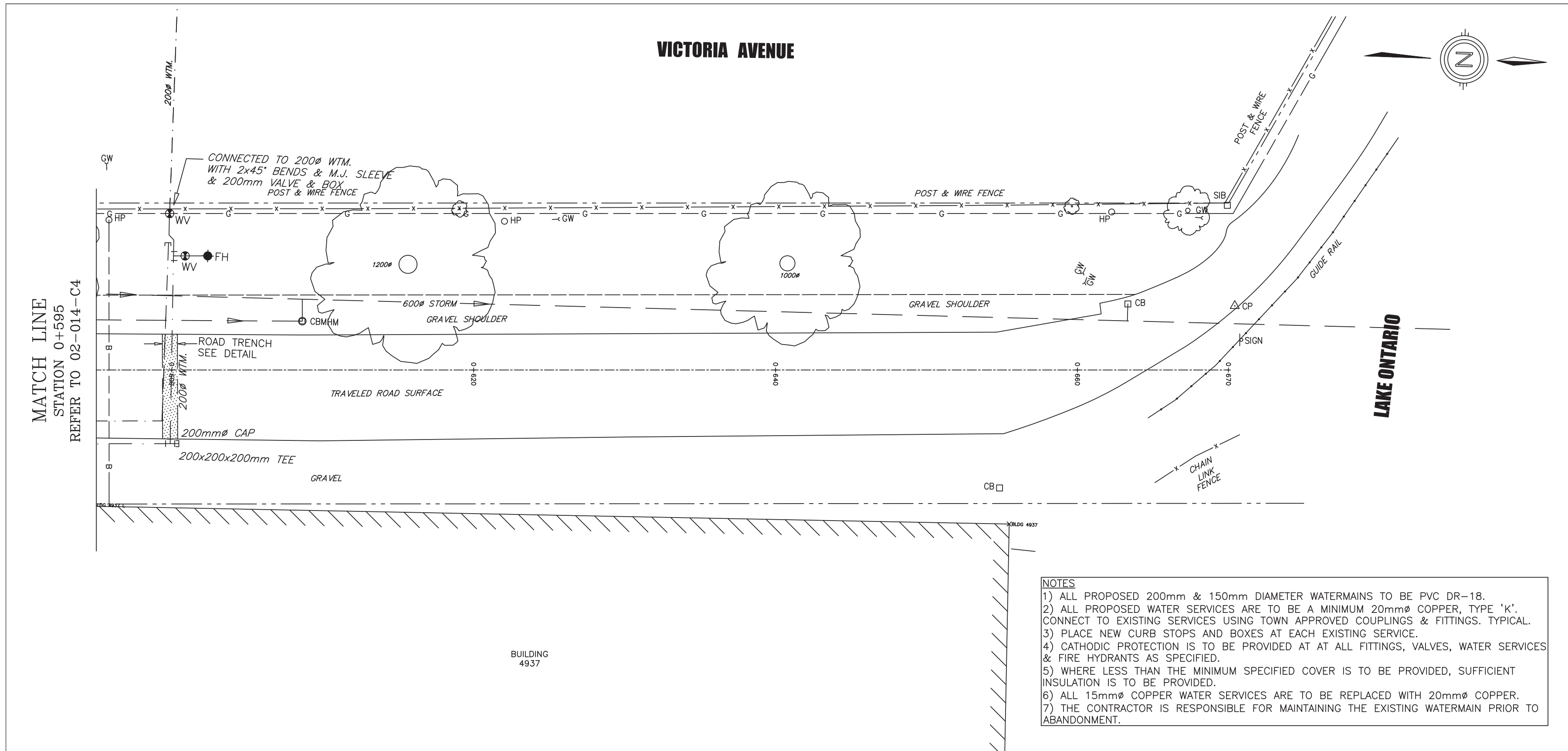
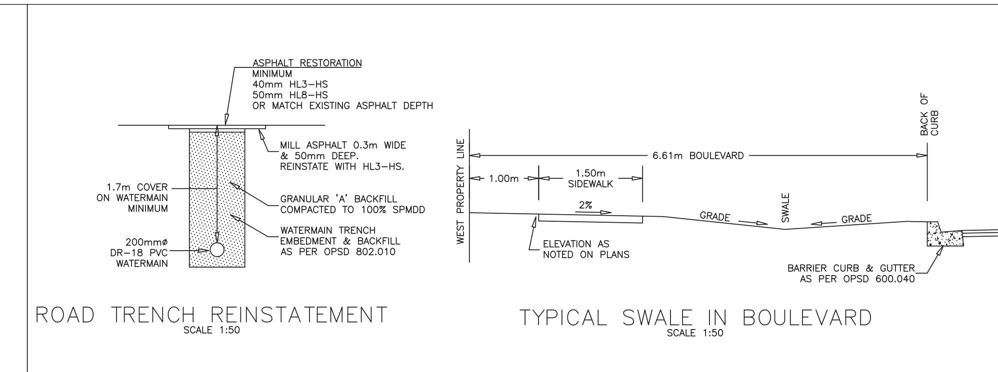
TOWN OF LINCOLN
VICTORIA AVENUE
ROAD RECONSTRUCTION & WATERMAIN REPLACEMENT
NORTH SERVICE ROAD to ±150m NORTH OF LAURIE AVE.
STA. 0+445 to 0+595

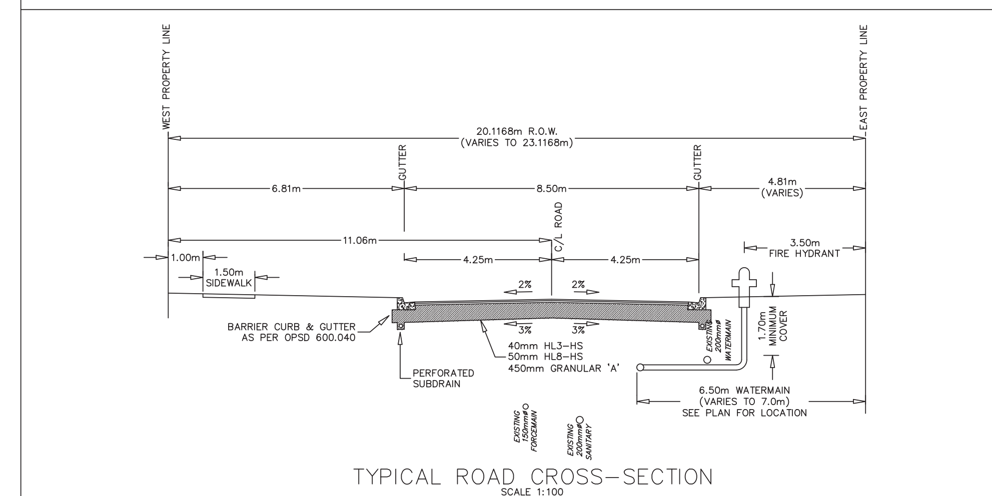
FIELD NOTES KTH Book No. 773
PLOT DATE MARCH 18, 2003
SCALE HOR. 1:200 VERT. 1:50
DWG. No. 02–014–C4
REV. 3
CAD FILE: 02014ASBUILT.dwg

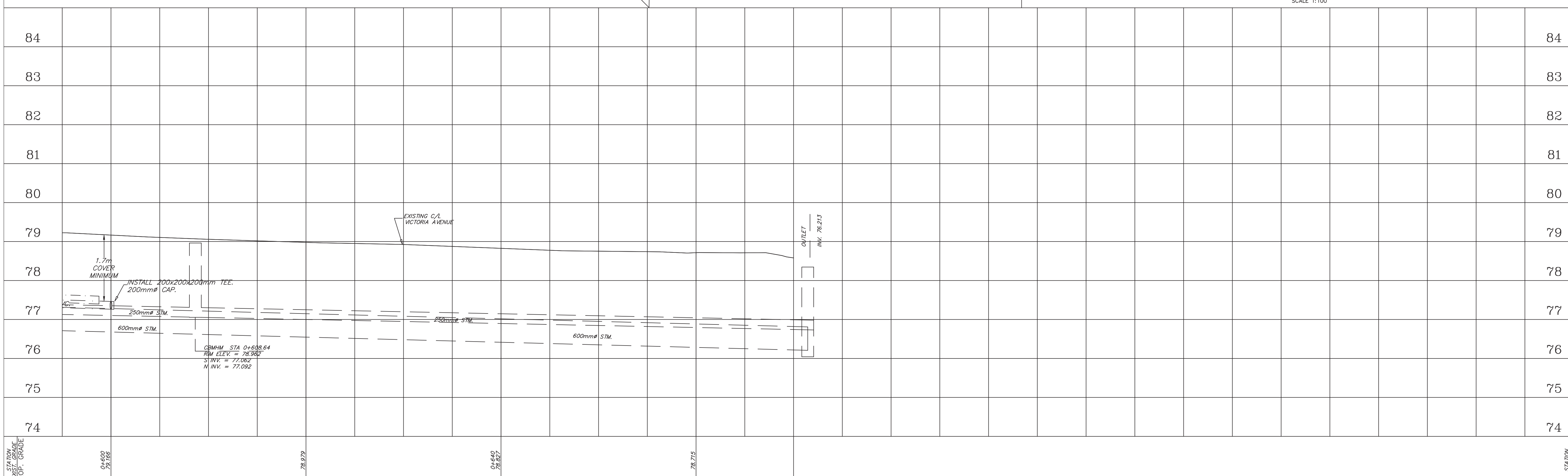


NOTES

1) ALL PROPOSED 200mm & 150mm DIAMETER WATERMAINS TO BE PVC DR–18.
2) ALL PROPOSED WATER SERVICES ARE TO BE A MINIMUM 20mmø COPPER, TYPE 'K'. CONNECT TO EXISTING SERVICES USING TOWN APPROVED COUPLINGS & FITTINGS. TYPICAL.
3) PLACE NEW CURB STOPS AND BOXES AT EACH EXISTING SERVICE.
4) CATHODIC PROTECTION IS TO BE PROVIDED AT AT ALL FITTINGS, VALVES, WATER SERVICES & FIRE HYDRANTS AS SPECIFIED.
5) WHERE LESS THAN THE MINIMUM SPECIFIED COVER IS TO BE PROVIDED, SUFFICIENT INSULATION IS TO BE PROVIDED.
6) ALL 15mmø COPPER WATER SERVICES ARE TO BE REPLACED WITH 20mmø COPPER.
7) THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING THE EXISTING WATERMAIN PRIOR TO ABANDONMENT.







3	RECORD OF CONSTRUCTION	MAR. 03	RB
2	ISSUED FOR TENDER	MAY /02	JH
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CAD FILE: 02014ASBUILT.dwg

gh3*
55 OSSINGTON AVENUE
SUITE 100
TORONTO, CANADA
M6J 2Y9
info@gh3.ca

Pat Hanson
Raymond Chow

December 19, 2023.

Brandon Donnely
Globizen

Reference: 4933 Victoria Avenue North, Lincoln, Ontario

Dear Sir:

Please be advised that the above-referenced building will be constructed in compliance with the 2015 Ontario Building Code (OBC), and equipped with a Fire Protection System conforming to the NFPA 13 Standards for Installation of Sprinkler Systems and specifically:

1. All structural members and floors will be of fire-resistive construction per the Fire Underwriters Survey (FUS) 2020 with 2-hour ratings per the OBC.
2. All vertical openings and exterior vertical communications will be constructed with a 1-hour fire rating.

Yours truly,

Raymond Chow OAA RAIC
Partner gh3* architects

From: Jeremy Korevaar
To: Dimitra Frysali
Cc: Melissa Shih; catherine@lithosgroup.ca
Subject: RE: 4933 Victoria Avenue, Lincoln - Data Collection
Date: November 27, 2023 11:47:16 AM
Attachments:

Good Morning,

Previous work by GM BluePlan for this secondary plan area has been undertaken the following population parameters were used in assigning sewer flows:

Residential Population – 673
Employment or Commercial Space – 500 (population 11)

Our design standards require flow calculations to use 255 L/cap/day for residential and 310 L/cap/day for employment.

The following information was previously provided as part of the formal Pre-Consultation Meeting - "This development proposes to use up sewer capacity that has been previously allotted for usage by the Prudhommes Development; That development would require pumping station upgrades to accommodate sewer capacity constraints which would be directly affected/increased by this development; therefore, cost sharing would be required."

With respect to water supply, GM BluePlan also completed analysis of the available flows on Victoria Avenue North in light of planned upgrades to the system. The Town currently has a call out for tenders for the upgrade of the watermain on Jordan Road from North Service to Fourth Avenue, North Service Road from Jordan Road to Victoria Avenue and Victoria Avenue from North Service Road to South Service Road. The table and summary below provides calculated fire flows at current conditions, after the previously noted upgrades and after future upgrades to Victoria Avenue from South Service Road to King Street. Information is also provided regarding capacity increases when the existing watermain is upgraded.

3.7. Victoria Avenue Watermain Trigger

The existing watermain along Victoria Avenue is a 200 mm. Fire flow was assessed along Victoria Avenue under existing system and proposed future upgrades with the flows outlined in Table 8.

Table 8: Victoria Avenue Watermain Fire Flow

		Victoria Avenue		Vineland Manufacturing	
		Extreme	Average	No Upgrades along Victoria Avenue north of North Service Road	Upgrade Victoria Avenue north of North Service Road to 300 mm
Available Fire Flow (L/s)	Existing System, no trunk upgrades	38	141	39	-
	Upgrade Jordan Road and North Service Road WM including Victoria Avenue QEW Crossing (Excluding Jordan Harbour)	107	186	97	119
	Upgrade Jordan Road, North Service Road, and Victoria Avenue WM to 300 mm (Excluding Jordan Harbour)	216	323	138	198
	Upgrade Jordan Road, North Service Road, and Victoria Avenue WM to 300 mm (Including Jordan Harbour)	221	326	139	202

If the required fire flow is greater than 135 L/s, the development will trigger upsizing Victoria Avenue north of North Service Road to 300 mm.

I trust this information will be of assistance. Please let me know if you need anything further.

Regards,

Jeremy Korevaar C.E.T, CAPM
Manager of Development Engineering
Town of Lincoln
Direct: 905-563-2799 ext. 504
Tel: 905-563-8205
jkorevaar@lincoln.ca
lincoln.ca
@TownofLincolnON

From: Dimitra Frysali <dimitraf@lithosgroup.ca>
Sent: Thursday, November 23, 2023 10:21 AM
To: Jeremy Korevaar <jkorevaar@lincoln.ca>
Cc: Melissa Shih <mshih@lincoln.ca>; catherine@lithosgroup.ca
Subject: RE: 4933 Victoria Avenue, Lincoln - Data Collection

Hello Jeremy,

I hope my email finds you well and safe.

Thank you for the information provided. We will await your consultant for the additional information regarding the sanitary flow.

Best Regards,

Dimitra Frysali, P.E., M.A.Sc.
Project Engineer
Lithos Group Inc.

LANDTEK LIMITED
Consulting Engineers

205 Nebo Road, Unit 4B
Hamilton, Ontario
L8W 2E1

Phone: 905-383-3733
engineering@landtek.ca
www.landtek.ca

Geotechnical Investigation
Proposed Tower Complex

4933 Victoria Avenue North
Vineland Station, Ontario
L0R 2E0

Prepared for:

4933 Vic Court Globizen LP
2720 Dundas Street West, Suite 608
Toronto, Ontario
M6P 0C3

Landtek File: 23016
November 3rd, 2023

EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	The proposed development is to comprise of the following: a stepped, five-storey to 17-storey residential tower, with three partial, above-ground parking levels and a three- and four- storey podium; a stepped, four-storey to 14-storey residential tower, with a four-storey podium courtyard; a 13- to 15-storey hotel with a rooftop pool; a central courtyard comprising public open space, trees, a pond and trellis-covered areas; and, a new deck, dock and access ramp.
Report Deliverables	The Preliminary Geotechnical Investigation Report is required to provide an understanding of the subsurface conditions underlying the site and to provide preliminary design and construction recommendations for the proposed new tower complex.

SITE DETAILS AND SETTING

Coordinates	630435, 4783500	**Geodetic Elevation**	73.0 m to 80.0 m

Site Description	The site is irregular in shape and is situated at the intersections of Verity Lane, Viceroy Avenue and Victoria Avenue North. The site is bound to the north by Lake Ontario, the west by Victoria Avenue North, the east by a forested area, and to the south by residential properties. The topography of the site is generally flat-lying and all existing buildings have been removed.
Geology	Existing pavement areas and/or fill material was encountered in all boreholes at the ground surface or underlying the existing pavement structure, and extends to depths between approximately 0.6 m and 4.5 m below existing ground level. Clayey silt, silty clay, silt till, clayey silt to silty clay till and completely to highly weathered red shale bedrock underlies the fill material to depths of between approximately 2.6 m and 12.1 m below existing ground level.
Groundwater	Groundwater, water seepages or saturated soils were not encountered during drilling but was reported at 2.2 m to 3.7 m depth during subsequent groundwater monitoring visits. Further information pertaining to groundwater conditions is provided in the Hydrogeological Assessment for the site, as completed by Landtek and reported under separate cover.

ENGINEERING CONSIDERATIONS

Foundations	It is considered by Landtek that the anticipated moderately- to highly-loaded tower structures and associated infrastructure can be supported by the shale bedrock underlying the site using conventional, concrete strip or pads foundations. It is anticipated that the foundations will be seated at depths of approximately 4.0 m to 5.0 m below surrounding ground level.
Settlements	The general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate. The SLS condition will not govern foundation design in bedrock as the stress required to induce the typical 25 mm settlement criteria at the SLS is anticipated to exceed the ULS. As such. settlements for foundations seated within bedrock are to be deemed negligible (i.e., less than 15 mm).
Earthquake Considerations	Based on the soil conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (*OBC*), the site is considered to be a 'C' Site Class.
At-grade Floor Slabs	It should be possible to construct the lowest (i.e., basement) concrete floor slab using slab-on-grade methods. The subgrade support condition is anticipated to be native clay, silt and till soils or bedrock, which should provide competent conditions for placing the vapour barrier material.

CONSTRUCTION CONSIDERATIONS

Excavations	The subsurface soils to be encountered during excavation at the site are expected to behave as "Type 2" and "Type 3" materials according to the OHSA classification in Part III. Type 2 soils are characteristic of the generally hard "clayey silt/silty clay till", while Type 3 soils are characteristic of the generally firm/compact "clayey silt/silty clay and silt till". The residual soils of completed weathered shale bedrock is considered to have strength characteristics that exceed Type 1 soils.
Subsurface Concrete	The native soils generally have a low to mild sulphate environment and are not aggressive to concrete (CSA criteria of less than 0.2 % water soluble sulphate in the soils). Therefore, normal General use (GU) hydraulic cement can be used for subsurface structures.
Construction Dewatering	It is expected that foundation elements for the proposed structure will be seated above the level at which groundwater was encountered. As such, temporary dewatering is not expected to be required during the construction process. Further construction dewatering considerations are provided in Landtek's Hydrogeological Assessment for the site, as reported under separate cover.

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SECTION PAGE(S)

1.0 INTRODUCTION

Landtek Limited (herein "*Landtek*") is pleased to submit this Preliminary Geotechnical Investigation report for the proposed new "*Vineland*" tower complex at civic address 4933 Victoria Avenue North in Vineland, Ontario. Authorization to proceed with the work was received from 4933 Vic Court Globizen LP, in January 2023.

Based on the Concept Plan drawing "*Site Plan – Ground Floor*", reference A103, it is understood that the proposed development is to comprise of the following:

- A stepped, five-storey to 17-storey residential tower in the east of the property, with three partial, above-ground parking levels and a three- and four- storey podium;
- A stepped, four-storey to 14-storey residential tower in the south of the property, with a four-storey podium courtyard;
- A 13- to 15-storey hotel in the northwest of the property, with a rooftop pool;
- A central courtyard comprising public open space, trees, a pond and trellis-covered areas; and,
- A new deck, dock and access ramp in the north of the property.

It is understood that one level of basement parking is also proposed and will cover the development footprint in full. Limited at-grade, deck parking is also proposed, with access leading from Victoria Avenue North.

No significant grade changes are anticipated, with foundations anticipated at depths of between approximately 4.0 m and 5.0 m below existing ground level. Elevator pits for the residential towers and hotel are expected to extend below foundation subgrades a further 1.5 m depth as a minimum.

The primary objectives of this investigation are:

- To confirm the subsurface soil and groundwater conditions for foundation design and construction;
- Provide design and construction recommendations with regards to building foundations, at-grade floor slabs, pavement structures, and subsurface drainage and utilities; and,
- Assess the characteristics of the soils to be excavated and their impact on excavatability, reuse and shoring systems.

This report has been prepared for the Client, the nominated engineers, designers, and project managers pertaining to the proposed residential tower complex at the site at civic address 4933 Victoria Avenue North in Hamilton, Ontario. Further dissemination of this report is not permitted without Landtek's prior written approval. Further details of the limitations of this report are presented in Appendix A.

2.0 SITE SETTING

2.1 Site Location and Description

The site is located in Vineland Station, Ontario, and is centered at approximate grid reference 630435, 4783500 (UTM 17T coordinates). The Geodetic elevation of the ground surface at the site is approximately 73.0 m to 80.0 m.

The site location is shown in Figure 2.1.1 below.

Figure 2.1.1: *Site Location and Surrounding Area*

The site is irregular in shape and is situated at the intersections of Verity Lane, Viceroy Avenue and Victoria Avenue North. The site is bound to the north by Lake Ontario, the west by Victoria Avenue North, the east by a forested area, and to the south by residential properties.

The topography of the site is generally flat-lying and has been cleared of all existing buildings that were once located on the site.

2.2 Published Geology

Based on previous geotechnical experience for the area and a review of the existing geological publications for the site area, Ontario Geological Survey (herein "*OGS*") Map P.0764 "*Quaternary*

Geology of the Niagara Area", the site is underlain by interbedded deposits of Lake Iroquois stratified sands and silt and clay till of the Halton Till Formation.

The Ontario Department of Mines (herein "*ODM*") Map 2344 "*Paleozoic Geology of the Niagara Area*" indicates that the superficial geology is underlain by red shale of the Queenston Formation.

Information provided by historical borehole records from within the vicinity of the site, and held by the OGS, generally confirms the anticipated geological conditions beneath the site. Based on the data from records for Borehole ID 852602, located approximately 500 m south of the site, the soil profile comprises of topsoil at the ground surface, followed by clay and silt till to approximately 6.6 m depth.

4.6 Clayey Silt to Silty Clay Till

Clayey silt to silty clay till was encountered **only** in boreholes BH1, BH/MW2, BH/MW3, BH/MW4, BH5, BH6, BH7, BH/MW8, BH/MW9A, and BH/MW11A underlying the fill and sand material and extends to depths of approximately 1.5 m and 3.0 m below existing pavement surface. The till is generally red and brown and contains traces of gravel, sand, iron staining and red shale fragments.

SPT ''N'' values ranging from 3 to 38 were reported, indicating the silty clay till deposits to be of a soft to hard, but generally hard consistency.

4.7 Bedrock

Red shale of the Queenston Formation was encountered in all boreholes at depths of between approximately 1.5 m to 4.5 m below existing ground level, equating to Geodetic elevations between approximately 79.6 m and 73.4 m. The shale is red and grey in colour, is very weak to weak, completely to highly weathered and was primarily recovered as "*residual soil*".

The Rock Quality Designation (RQD) values of the competent shale bedrock were in the order of 0 % to 77 % indicating the bedrock to be of a "*very poor to good*" quality, though improving with depth. The results of the rock strength parameter testing will be presented in Appendix D, once received.

4.8 Groundwater

Groundwater, water seepages or saturated soils were not encountered during augur drilling, with all boreholes remaining open and dry either on termination or on transition to rotary coring. Six subsequent groundwater monitoring well visits have been completed at the site to date, the most recent results of which are presented in Table 4.8.1.

Table 4.8.1: Summary of Water Level Measurements

MW ID	Well Details			Groundwater Monitoring Results	
	Depth	Screen	Water Strike	September 20, 2023	October 17, 2023
BH/MW1S-23	6.0 m	3.0 m – 6.0 m	-	-	3.42 m
BH/MW1D-23	10.6 m	7.6 m – 10.6 m	-	-	3.48 m
BH/MW2S-23	3.0 m	1.5 m – 3.0 m	-	-	3.33 m
BH/MW2D-23	4.5 m	1.5 m – 4.5 m	-	-	3.16 m
BH/MW3S-23	6.0 m	3.0 m – 6.0 m	-	-	3.48 m
BH/MW3D-23	10.6 m	7.6 m – 10.6 m	-	-	3.63 m
BH/MW4S-23	6.0 m	3.0 m – 6.0 m	-	-	3.22 m
BH/MW4-23	3.0 m	1.5 m – 3.0 m	-	-	2.35 m
BH/MW5S-23	6.0 m	3.0 m – 6.0 m	-	-	3.61 m
BH/MW6-23	3.0 m	1.5 m – 3.0 m	-	-	3.01 m
BH/MW8S-23	4.5 m	1.5 m – 4.5 m	-	-	2.74 m
BH/MW9S-23	4.5 m	1.5 m – 4.5 m	-	-	2.44 m
BH/MW9D-23	12.1 m	9.1 m – 12.1 m	-	-	3.43 m
BH/MW2	4.5 m	1.5 m – 4.5 m	-	2.02 m	-
BH/MW3	4.5 m	1.5 m – 4.5 m	-	2.22 m	-
BH/MW8	4.5 m	1.5 m – 4.5 m	-	2.25 m	-
BH/MW9A	4.5 m	1.5 m – 4.5 m	-	3.04 m	-
BH/MW10	4.5 m	1.5 m – 4.5 m	-	3.18 m	-
BH/MW11A	4.5 m	1.5 m – 4.5 m	-	2.21 m	-

It is noted that the boreholes were generally dry at the depths where water has been recorded during monitoring. This is indicative of a fracture-controlled groundwater regime with the bedrock responding to exposure by rising in the monitoring well through pressurization until it reaches a static equilibrium; what is referred to as the "*piezometric level*".

It should be noted that groundwater conditions and surface water flow conditions are expected to vary according to the time of the year and seasonal precipitation levels. Water seepage may be also anticipated from soil fissures and any fill material present at the site.

Further information pertaining to groundwater conditions is provided in the Hydrogeological Assessment for the site, as completed by Landtek and reported under separate cover.

9.0 EXCAVATION AND BACKFILL CONSIDERATIONS

9.1 Excavation Considerations for Soils

All temporary excavations and unbraced side slopes in the soils should conform to standards set out in the Occupational Health and Safety Act, Ontario Regulation 213/91 "*Construction Projects*" (herein "*OHSA*"). The subsurface soils to be encountered during excavation at the site are expected to behave as "*Type 2"* and "*Type 3"* materials according to the OHSA classification in Part III. Type 2 soils are characteristic of the generally hard "*clayey silt to silty clay till deposits"*, while Type 3 soils are characteristic of the generally firm "*clayey silt to silty clay deposits*", and the generally compact "*silt till deposits*".

The residual soils of completed weathered shale bedrock is considered to have strength characteristics that exceed Type 1 soils.

Excavations for new foundations should satisfy the criteria given in the example shown in Figure 9.1.1 to avoid overlapping stresses and minimize the risk of undermining existing adjacent structures, including utilities, and/or triggering additional settlements of the existing structures due to soil disturbance.

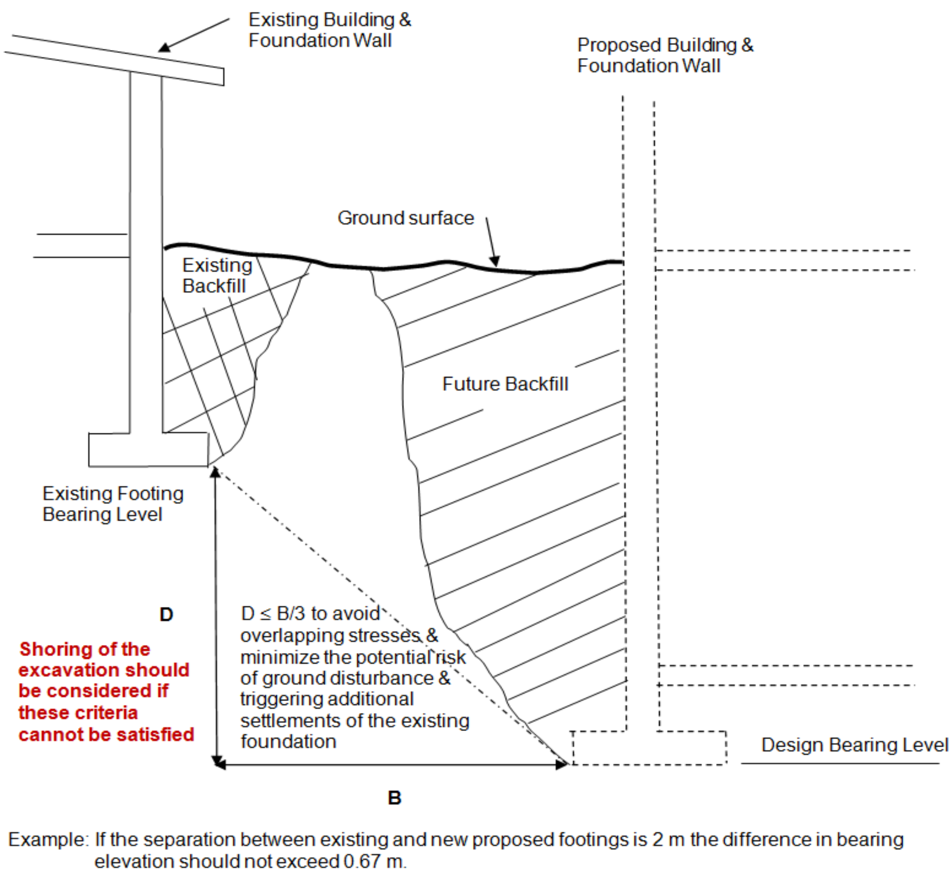

Figure 9.1.1: *Criteria for Assessing Excavation Shoring Requirements (Not to Scale)*

It should be possible to excavate the overburden soils with a hydraulic backhoe. Moist Type 2 and 3 soils are expected to be stable for short construction periods at slopes of approximately 45° to the horizontal (i.e., 1V:1H).

Consideration should be given to any existing trench excavations and associated backfill that may be present directly behind cut slopes within the native soils that may appear to be stable on first excavation. In these circumstances, slopes can suddenly slough or collapse due to the effects of the adjacent backfill.

Consequently, for excavation conditions that cannot satisfy the OHSA requirements for unbraced 1H:1V side slopes, a trench box system should be used, or temporary shoring should be installed to maintain safe working conditions. This may be more applicable to basement excavations, though may also apply to service trench excavations etc., particularly when in close proximity to new road pavements or associated infrastructure. Temporary shoring considerations are provided in more detail in Section 10.0 of this report.

9.2 Excavation Considerations for Bedrock

In accordance with the standards set out in the OHSA, the more competent "*shale bedrock*" encountered underlying the site has strength properties that exceed a Type 1 soil.

For any required bedrock excavation, a backhoe equipped with a hydraulic breaker and/or a bucket with rock-ripping 'tiger teeth' may be required in the shale bedrock, particularly where encountering harder siltstone or limestone bands. The blasting of bedrock will not be permitted by the Corporation of the Town of Lincoln (herein "*Town of Lincoln*"). Significant ground vibrations resulting from excavation works are not anticipated, though may be elevated above those associated with normal construction activities. As such, a period of ground vibration monitoring may be required to determine the peak vibration levels and any remedial measures or limitations required.

A backhoe equipped with a hydraulic breaker and/or a bucket with rock-ripping 'tiger teeth' may be required in the shale strata. Significant ground vibrations resulting from excavation works are not anticipated other than those associated with normal construction activities.

The shale is expected to remain relatively stable at near vertical slopes for short periods of time. It is recommended that any excavation slopes be scaled of loose rock pieces and overhang and cut back to about 10V:1H.

9.3 Short-Term (Construction) Dewatering Considerations

Based on the anticipated depths of excavation required for the one proposed basement parking level and associated elevator pits, it is expected that foundation elements for the proposed structure will be seated above the level at which groundwater was encountered. As such, temporary dewatering is not expected to be required during the construction process other than standard pumping of storm water or localized seepages from sumps at the base of excavations.

More detailed considerations regarding groundwater control and dewatering requirements during construction have been provided by the Hydrogeological Assessment for the site, as completed by Landtek and reported under separate cover.

9.4 General Backfill Considerations

Backfill next to foundation walls and in service trenches should be selected to be compactable in narrow trench conditions. The on-site clayey silt, sand and silty sand and completely to highly weathered shale are expected to be reusable as trench backfill and backfill around the proposed structures on the site. Any variation in the moisture contents of the soils encountered may require selective separation of material to avoid the use of wet soil.

Experience with shale indicates that any excavated bedrock material will not be suitable for reuse at the site without mechanical processing and grading to an Ontario Provincial Standard Specification (herein "*OPSS*") 1010-compliant product prior to its application.

Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. It is recommended that the target compaction specification for trench backfill be 97 % SPMDD with no individual test below 95 % SPMDD.

During inclement weather the native soils may become too wet to achieve satisfactory compaction. If construction is proposed for late in the year, a reduced level of trench compaction with a higher risk of future settlements is to be anticipated, and it is recommended that provisional contract quantities be established for the supply and placement of imported granular fill under such circumstances. The imported granular should meet the requirements of OPSS 1010 for Granular B Type I material as a minimum requirement.

10.0 TEMPORARY SHORING CONSIDERATIONS

The installation of temporary shoring is also recommended to maintain safe working conditions and eliminate the possibility of loss of ground and damage to nearby structures and buried utilities on the adjacent road allowances during excavation for the basement construction.

The requirement and application of shoring to support excavation side slopes will be dependent on the required excavation depth and the proximity of existing or newly constructed infrastructure adjacent to the excavation.

The preferred method of shoring will consist of a concrete caisson wall. This type of system is expected to provide the additional benefit of sealing the excavation from water penetration and loss of soil fines into the open excavation. Soldier piles and timber lagging may be considered as an option for a shoring system, though this type of system may require measures to prevent groundwater inflow into the excavation and any subsequent loss of soil between the spaces of lagging boards. Consideration may be also given to the application of shotcrete where groundwater is encountered and/or where shale bedrock is exposed in the excavation faces.

The shoring methods may provide lateral restraining force through the use of rakers or tieback anchors. Tieback anchors provide additional advantage since they do not protrude into the excavations as rakers would. However, the use of tieback anchors is also dependent upon whether permission is needed or whether it is physically possible to extend the anchors to the required distance into neighbouring properties.

It should be noted that the design of any temporary shoring system is the responsibility of the Contractor. Therefore, a specialist shoring contractor should be consulted to provide the most appropriate shoring type method and associated installation procedures. In any event, the shoring design should be based on the procedures outlined in the latest edition of the Canadian Foundation Engineering Manual. It is also recommended that lateral and vertical movement of the shoring system be monitored during construction to ensure that movements are within the acceptable range.

11.0 SITE SERVICING CONSIDERATIONS

There is no indication that special pipe bedding materials or procedures are required for the installation of services. All bedding cover and backfill materials should be selected in accordance with OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material.

The pipes should be placed with a minimum bedding thickness in conformance of Ontario Provincial Standard Drawing (herein "*OPSD*") 802.010, 802.013 and 802.014 for flexible pipe and OPSD 802.030, 031, 032, 033 and 034 for rigid pipes. The type of bedding shall be selected to suit the applicable pipe strength and site conditions.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 95 % of the SPMDD before a subsequent layer is placed. Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ by more than the 300 mm uncompacted layer. The remainder of the trench should be backfilled as per the requirements defined in Sections 9.0 of this report.

It is assumed all services will have a minimum of 1.2 m of soil cover for frost protection. For services installed at shallower depths, suitable insulation for frost protection is recommended.

13.0 PAVEMENT CONSIDERATIONS

13.1 Deck Pavement Design Considerations

It is understood that the footprint of the proposed basement will cover the site area in full. As such, any pavement structures are anticipated to be deck structures rather than standalone, at-grade pavement structures.

Such deck pavements should comprise a minimum 50 mm cover of OPSS HL 3 asphalt or minimum 80 mm cover of interlocking concrete pavers. The bedding or grading material to be placed between the concrete deck and the asphalt pavement surface or interlocking concrete pavers should comprise either blinding sand or OPSS Granular A material, depending on the thickness of the layer required.

Any tie-ins of the deck pavements to the road pavement structure of Victoria Avenue North should match existing as a minimum, in accordance with OPSS 310.

13.2 Pavement Materials

13.2.1 Granular Base Course

The granular base course material should meet OPSS Granular “A” specifications. Quarried 20 mm limestone crushed to Granular "A" gradation specifications is recommended.

13.2.2 Hot Mix Asphalt

The surface course asphalt should meet current specifications for HL 3, as prescribed by the Town of Lincoln or, alternatively, OPSS 1150.

13.2.3 Compaction

Granular base course and subbase course fill material should be compacted to 100 % SPMDD. Hot mix asphalt should be compacted to the criteria set out by the Town of Lincoln.

13.3 Sidewalk Considerations

The construction of the concrete sidewalks at the site should be completed to the satisfaction of the Town of Lincoln’s Engineering Standards, and as detailed in Table 13.3.1. The concrete and aggregates should be produced and placed to meet those standards also stipulated by the Town of Lincoln’s Engineering Standards.

Table 13.3.1: Recommended Minimum Concrete Sidewalk Specifications

Materials	Compaction Requirements	Layer Thickness
Normal Portland GU (32 MPa) (CAN3-CSA A23.1) - Class C-2	N/A	125 mm
Granular “A” Base	95 % SPMDD*	150 mm

* Standard Proctor Maximum Dry Density

Where finished sidewalks are on level ground, and to ensure that they remain free of ponding water, a final slope/gradient of the concrete sidewalk surface of at least 2 % should be maintained. In addition, construction joints in the sidewalk concrete should be properly sealed (e.g., bitumen filler) to minimize the water migration.

14.0 CLOSURE

The Limitations of Report, as stated in Appendix A, are an integral part of this report.

Soil samples will be retained and stored by Landtek for a period of three months after the report is issued. The samples will be disposed of at the end of the three-month period unless a written request from the client to extend the storage period is received.

We trust this report will be of assistance with the design and construction of the proposed development. Should you have any questions, please do not hesitate to contact our office.

Yours sincerely,

LANDTEK LIMITED

LICENSED PROFESSIONAL ENGINEER
R. DI CIENZO
Nov. 3/23
PROVINCE OF ONTARIO

James Dann, B.Eng. (Hons) ACSM
Manager, Geotechnical Projects

Ralph Di Cienzo, P. Eng.
Consulting Engineer

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Phone: 905-383-3733
engineering@landtek.ca
www.landtek.ca

November 6, 2023
File: 23016

4933 Vic Court Globizen LP
2720 Dundas Street West, Suite 608
Toronto, Ontario
M6P 0C3

Attention: Mr. Rick Sole, Globizen Group

Dear Mr. Sole,

Re: Slope Stability Assessment Letter Report
Proposed Residential Development, 4933 Victoria Avenue North, Vineland, Ontario

This letter is provided by Landtek Limited (herein "*Landtek*") in response to comments received from the Niagara Peninsula Conservation Authority (herein "*NPCA*") pertaining to the proposed residential development of the site identified as civic address 4933 Victoria Avenue North in Vineland, Ontario.

According to the NPCA, the property is located in part within a regulated slope area, and it is understood that the NPCA has requested a geotechnical review be undertaken to establish the location of the Long-Term Stable Top of Slope (herein "*LTSTS*") and appropriate construction setback distances relative to the proposed development.

Background

Based on the Concept Plan drawing "*Site Plan – Ground Floor*", reference A103, it is understood that the proposed development along the eastern property boundary is to comprise of the following:

- A stepped, five-storey to 17-storey residential tower with three partial, above-ground parking levels and a three- and four- storey podium;
- A stepped, four-storey to 14-storey residential tower with a four-storey podium courtyard; and,
- A new deck, dock and access ramp in the north.

It is understood that one level of basement parking is also proposed and will cover the development footprint in full. No significant grade changes are anticipated, with foundations anticipated at depths of between approximately 4.0 m and 5.0 m below existing ground level.

For the purposes of this letter, the evaluation study area is focused to the table land area within the existing property boundary and extending eastwards from civic address 4933 Victoria Avenue North in Vineland.

The slope assessment is required by the NPCA to, from a geotechnical perspective:

- Assess the condition and stability of the slope adjacent to the property when considering the slope in its current condition;
- Establish the LTSTS relative to the existing slope and the proposed development; and,
- Determine whether the proposed development and associated basement level will have a detrimental impact on the existing slope.

This letter-format report was prepared in general accordance with the guidelines of the Ministry of Natural Resources (herein "*MNR*") document "*Natural Hazards Technical Guides*", and the supporting "*Geotechnical Principles for Stable Slopes*" document.

Site Characterization

Site Location and Description

The site is located in Vineland Station, Ontario, and is centered at approximate grid reference 630435, 4783500 (UTM 17T coordinates). The Geodetic elevation of the ground surface within the property boundary ranges between approximately 73.0 m and 80.0 m. The topography of the site is generally flat-lying, with a shallow slope towards the creek to the east.

The site is bound to the north by Lake Ontario, the west by Victoria Avenue North and the Millenium Forest Park, the east by a wooded area and a river valley system of Prudhomme Creek, and to the south by residential properties.

Figure 1: *Site location and setting.*

The site location is presented in Figure 1.

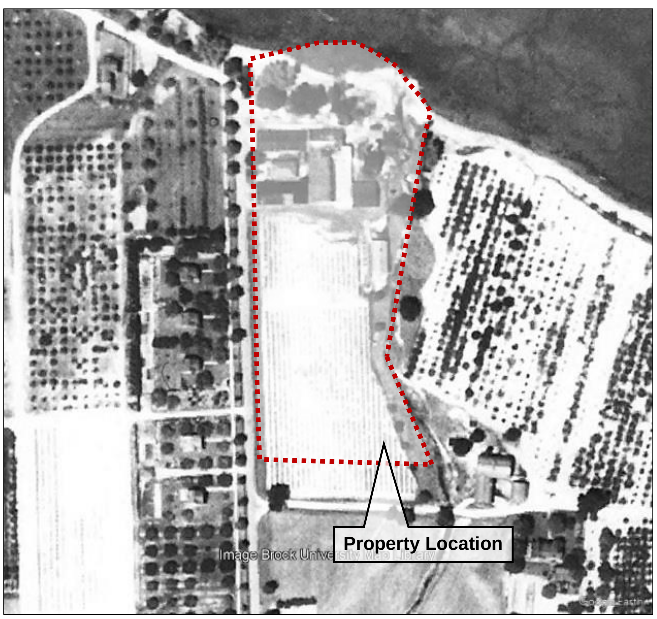

Figure 2: *1934 aerial photograph extract.*

Aerial photographs available for the property indicate the area to have been developed for a significant time. The slope area and slope crest are noted to be densely vegetated with mature trees and shrubs.

Prior to development in its current layout, the property appears to have been of agricultural use, with aerial photography from 1934 (see Figure 2) showing the site to be within an area of maintained farmland.

Prudhomme Creek is also noted to be in its current alignment and that vegetation across the eastern area of the property boundary and creek slope comprises dense vegetation and a notable tree canopy.

The majority of the property is generally flat-lying, being for the most part within the tableland area of Prudhomme Creek, with a minimal gradient (± 2° to 5°) that results in an approximately 1.0 m change in elevation across the tableland area. The top of the Martindale Pond slope, as identified by the NPCA, is inferred to pass through the existing residential structure to the north of the rear garden area.

<u>Published Geology</u>

According to the Ontario Geological Survey (herein "*OGS*") Map P.0764 "*Quaternary Geology of the Niagara Area*", the site is underlain by interbedded deposits of Lake Iroquois stratified sands and silt and clay till of the Halton Till Formation. The Ontario Department of Mines (herein "*ODM*") Map 2344 "*Paleozoic Geology of the Niagara Area*" indicates that the superficial geology is underlain at shallow depth (i.e., approximately 3.0 m) by and interbedded sequence of red shales, siltstones and sandstones of the Queenston Formation.

Landtek completed Geotechnical, Environmental and Hydrogeological Investigations at the site in 2022 and 2023 that included the drilling of a number of boreholes across the site area, and the installation of groundwater monitoring wells. Of these investigations, boreholes BHMW1D-23, BH7-23, BHMW8S-23 and BHMW9D were located along the eastern property boundary and have provided confirmation of the published geology recorded from historical boreholes records.

<u>Hydrology and Hydrogeology</u>

The nearest surface water feature is Prudhomme Creek that bounds the property to the east, as seen on Figure 3. Water flow rates are reported by the NPCA to be low to moderate. Prudhomme Creek outfalls into Lake Ontario that bounds the property to the north.

Except for the construction of coastline defense systems, the alignment of the Prudhomme Creek outfall and Lake Ontario shoreline shows no significant deviation since at least 1934, as shown in the aerial photograph extract presented as Figure 2.

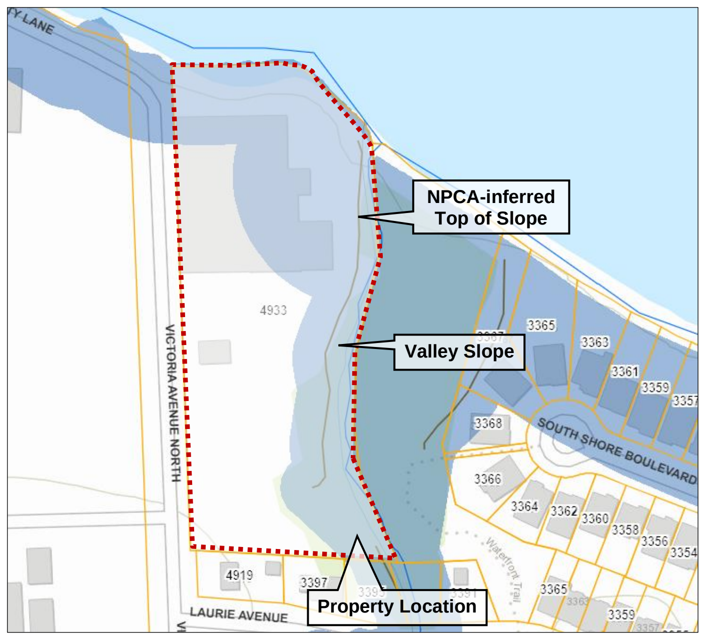

Figure 3*: NPCA Watershed Explorer extract.*

Aerial photography data shows there to be no evidence of erosion by surface water action within the tableland and slope area, indicating that water migration during heavy rainfall events within the site area is directed to topographically flatter or lower areas or through natural percolation.

According to the OGS, static groundwater levels in the vicinity of the site are generally associated with the Queenston Formation bedrock and are inferred to be in hydraulic continuity with Lake Ontario. It is also anticipated that Prudhomme Creek is also in hydraulic continuity with Lake Ontario by proximity.

No groundwater seepages were observed in the slope face, suggesting that any groundwater regime present beneath the property is likely to be within the bedrock and in hydraulic continuity with the water of Prudhomme Creek and Lake Ontario.

Monitoring wells installed in boreholes BHMW1D-23, BHMW8S-23 and BHMW9D, and subsequent phases of groundwater monitoring have identified groundwater presence within the bedrock underlying the site. Groundwater resting levels are reported to be at Geodetic elevations between approximately 74.8 m and 75.2 m. These levels are in direct correlation with Lake Ontario water levels, being in the order of Geodetic elevations 74.6 m to 75.3 m.

A copy of the borehole and groundwater monitoring well logs for boreholes BHMW1D-23, BH7-23, BHMW8S-23 and BHMW9D are attached as Enclosure 1.

Site Geomorphology

The tableland area is generally flat-lying to becoming a very shallow gradient (± 2° to 4°) in the south and west, and comprises primarily of gravel pavements, maintained and rough grassland and existing structures bordered in by mature trees to the east.

In the vicinity of the site, the Prudhomme Creek valley is classified by the Ontario Ministry of Natural Resources technical guide as a "*Confined Stream System*". From slope crest to water level, the slope within the property boundary is approximately 3.0 m to 4.0 m (calculated) in height, between approximately 10.0 m and 27.0 m in profile width, and appears to be generally uniform in profile, with overall slope angles of approximately 10° to 20° (calculated).

No areas of exposed, bare soils are noted, with trees, shrubs and detritus covering the slope area. The trees create a dense canopy and comprise of semi-mature and mature trees. Aerial photography for the site area indicates that the slope alignments have remained unchanged since at least 1934, and that slope vegetation has remained consistent, with some densification in places.

Field Observations

A site visit/reconnaissance was conducted on August 16, 2023, by a representative of Landtek. The visual assessment of the slope was conducted in accordance with the MNR's Technical Guide "*River and Stream Systems: Erosion Hazard Limit*".

As identified by the historical review, the property is located within the tableland associated with the western slope of Prudhomme Creek. The tableland is generally flat lying with local grading of approximately 2° towards the north and northwest. The tableland consists of a gravel cover edged by maintained and rough grass, and is fringed by dense, mature and semi-mature trees along the eastern boundary.

The transition between the tableland and the slope is clearly defined by the relatively flat tableland area abutting the approximately 3.0 m to 4.0 m high slope area. The crest of the slope is marked by clear changes in vegetation and topography. The field-measured angles of the slopes in their entirety, range between 16° and 21° (approximately 3H:1V) with local reduction to between 10° and 12° where the slope faces locally shallow to become more of a raised bench profile.

The slope is heavily vegetated with mature and semi-mature trees and low-level shrubs. Limited grass cover was noted, being due to the density of the tree canopy. The trees yield trunks of up to approximately 0.8 m diameter and are straight and true. No significant arcing or bowing to the trunks of the trees was noted within the study area, indicating there to be no active or historical ground movement occurring or having occurred.

Some leaning to trees was observed in the south of the site, where tree trunks lean towards the west on both sides of the valley. This is indicative of a prevailing wind influence during tree growth rather than soil subsidence. This is particularly as the trees on the site-side of the valley are uniformly leaning away from the slope instead of towards, as would be expected from soil creep or translational slope failure/erosion.

The toe of the slope is inferred as the waters edge of Prudhomme Creek. No evidence of active slope or toe erosion was noted during the site visit, with the slope faces observed being generally consistent in profile and appearance.

Shallow, surficial soils were exposed using a hand trowel and were noted to comprise a generally moist, brown, silt and clay till soils with variable fractions of sand and gravel. Red shale exposures were noted in the lower sections of the slopes and also locally exposed in the riverbed. This is consistent with the geology reported for the area and the measured slope angles are considered to be below the natural internal angle of friction (φ) of the exposed soils and completely weathered shale under natural moisture conditions.

The visual assessment of the tableland and slope area also identified no evidence of water seepage, spring activity or surface water runoff that would influence the moisture content of the soils of the slope. The slope area and associated vegetation cover of the western valley floor and slope areas also appear to have remained unchanged for a significant time-period, as is consistent with the information provided by the aerial photograph of 1934.

The Slope Stability Rating Chart completed for the slope, as included as Enclosure 2 for reference, assigned the slope a Stability Rating of 23, indicating a "…*stable slope with no toe erosion; no evidence of past instability*…" and "…*no structures within [the] slope height or crest*…". Selected photographs (Photographs 1 to 3) of the tableland area, slope crest and slope face are presented as Enclosure 3.

An initial review of the investigated slope was made using measurements on site and topographical information provided to Landtek. General features at the site are shown on Drawing 23016-01 "*Site Features Plan and Section*", attached as Enclosure 4, together with two representative cross-sections of the steepest and shallowest slope areas and their relationship to the current property footprint.

Discussion and Recommendations

Based on the findings of the site reconnaissance, it is considered by Landtek that the Top of Slope identified by the NPCA on their Watershed Explorer, as shown in Figure 3, is in general conformance with the available contour information and observations made during the site reconnaissance, as presented on Drawing 23016-01 "*Site Features Plan and Section*", attached as Enclosure 4.

Given the findings of the historical review, the site reconnaissance, slope rating and the measured slope angles being between 16° and 21° (approximately 3H:1V), it is considered that the full modelling of the slope is not necessary. On this basis, the actual Top of Slope presented on Drawing 23016-01 "*Site Features Plan and Section*" is also considered by Landtek to be the LTSTS.

<u>Development Impact Considerations and Construction Offsets</u>

In assessing the slope to determine construction offsets there are three principal requirements to consider for confined systems:

1. the '*Toe Erosion*' allowance;
2. the '*Stable Slope*' allowance; and,
3. the '*Erosion Access*' allowance.

These three requirements are addressed as follows:

Toe Erosion Allowance: Field observations made during the site reconnaissance identified no evidence of toe erosion to the slope face to the east of the development footprint. Though water flow, albeit very limited, is noted at the slope toe, exposed bedrock can be observed in the riverbed and lower slope areas. Slope angles noted during the site reconnaissance are also very shallow, reducing scour potential during higher waterflow events.

On this basis, it is considered by Landtek that a toe erosion allowance of 0.0 m may be applied, as is acceptable for bedrock in an erosive environment. As such, consideration for active toe erosion is to be zero in the application of the construction offset.

Stable Slope Allowance: The slope is considered to be stable in its current condition. As such, the actual slope crest is considered representative of the LTSTS.

It is therefore considered that, with the slope stable in its current form, a stable slope allowance equal to the width of the current slope profile (i.e., equating to a setback distance of 0.0 m) is to be applied.

Erosion Access Allowance: As defined by the NPCA, the accepted minimum construction offset from the LTSTS for residential development is to be considered as 7.5 m. However, given the presence of shallow bedrock and the stable conditions identified by this assessment, a reduction of the access allowance from 7.5 m to 6.0 m per MNR guidance can be both justified and supported.

This reduction in the access allowance will require NPCA approval.

Using the previously detailed parameters, the following construction offset from the LTSTS is to be applied to the proposed additions at the site:

‘*Toe Erosion*’ allowance + ‘*Stable Slope*’ allowance + ‘*Erosion Access*’ allowance
= 0.0 m + 0.0 m + 7.5 m
= 7.5 m (or 6.0 m, if approved) from the actual Top of Slope (i.e., the LTSTS).

Based on the information provided to Landtek, the proposed development is to be situated approximately 6.0 m away from the actual Top of Slope (i.e., the LTSTS) at it’s closest and therefore approximately 1.5 m inside of the NPCA’s construction offset requirements but in line with MNR construction offset requirements.

In considering the stable nature of the slope, it is considered by Landtek from a geotechnical perspective, that the proposed residential development and associated basement level at the property will not have any adverse affect on the existing slope condition such that the global stability of the slope is compromised.

As required by the NPCA, the actual Top of Slope and LTSTS have been defined on Drawing 23016-01 together with the required, calculated NPCA and MNR construction offsets.

<u>Development and Construction Drainage Considerations</u>

The erosional behaviour of fine sediments deposits in slope profiles is known to be influenced significantly by water flow, particularly from surface runoff, and not necessarily just by creek erosion at the toe of the slope.

A number of papers have been written that have evaluated the environments of such slopes, particularly during periods of heavy rainfall. “*A hydrochemical study of urban landslides caused by heavy rain: Scarborough Bluffs, Ontario, Canada*” (Eyles & Howard, 1988) particularly identifies the influence of surface water runoff resulting from snow melt and heavy rainfall in the summer of 1973 which “…*caused a rapid increase in hydrostatic pressure within fissures*...” that ultimately resulted in significant failures.

The most appropriate solution to reducing any potential for any future shallow-seated rotational or translational failures to the existing slope profile at the site is to control surface water flow from above the slope in order to prevent water from flowing over the slope face. It is therefore important to ensure that any surface water controls (roof drains etc.) associated with the proposed development are not directed towards the slope area. Such drainage is to be drawn away from the slope via a positive drainage system or directed to the front of the property.

It is also important to ensure that appropriate considerations and controls are applied at the construction stage of the development project. As with any construction adjacent to a slope face, controlling surface water and managing soil stockpiles will be essential to ensure that the slope is not subjected to increases in water volume or surface loads. On this basis, it is recommended that all excavation and construction activities, materials storage etc. remain outside of the regulation-required and/or agreed construction offset, as defined by this assessment.

Closure

We trust that this letter report is satisfactory for your purposes at this time, and please do not hesitate to call if you have any questions or would like to discuss the findings of this assessment in more detail.

Kind regards,
LANDTEK LIMITED

LICENSED PROFESSIONAL ENGINEER
R. DI CIENZO
Nov. 6/23
PROVINCE OF ONTARIO

James Dann, B.Eng. (Hons.) ACSM
Manager, Geotechnical Projects

Ralph Di Cienzo, P.Eng.
Consulting Engineer

Encls:

Enclosure 1: Landtek Limited Borehole Logs
Enclosure 2: Slope Stability Rating Chart
Enclosure 3: Site Photographs 1 to 3
Enclosure 4: Drawing 23016-01: "*Site Features Plan and Section*"

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Hydrogeological Investigation
Proposed Residential Development
4933 Victoria Avenue North
Vineland Station (Town of Lincoln), Ontario
L0R 2E0

Prepared for:

4933 Vic Court Globizen LP
2720 Dundas Street West, Suite 608
Toronto, Ontario
M6P 0C3

Landtek File: 23015
November 10, 2023

EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	The proposed development is to comprise of the following: a stepped, five-storey to 17-storey residential tower, with three partial, above-ground parking levels, one underground parking level extending across the Site, and a three- and four- storey podium; a stepped, four-storey to 14-storey residential tower, with a four-storey podium courtyard; a 13- to 15-storey hotel with a rooftop pool; a central courtyard comprising public open space, trees, a pond and trellis-covered areas; and, a new deck, dock and access ramp.
Report Deliverables	The Hydrogeological Investigation is required to assess the current site groundwater conditions, determine potential development/post development effects of the proposed development; and provide monitoring and mitigation plans for the development.

SITE DETAILS AND SETTING

Coordinates	630435, 4783500	**Geodetic Elevation**	83 m to 89 m
Site Description	The site is irregular in shape and is situated at the intersections of Verity Lane, Viceroy Avenue and Victoria Avenue North. The site is bound to the north by Lake Ontario, the west by Victoria Avenue North, the east by a forested area, and to the south by residential properties. The topography of the site is generally flat-lying and all existing buildings have been removed.		
Geology	Existing pavement areas and/or fill material was encountered in all boreholes at the ground surface or underlying the existing pavement structure and extends to depths between approximately 0.6 m and 4.5 m below existing ground level. Clayey silt, silty clay, silt till, clayey silt to silty clay till and completely to highly weathered red shale bedrock underlies the fill material to depths of between approximately 2.6 m and 12.1 m below existing ground level.		
Groundwater	Depths to groundwater in all monitoring wells were obtained manually by Landtek staff on July 13, August 18, September 20, October 6, and October 17, 2023. Based on the recorded groundwater levels, the highest water level was determined to be 2.18 mbgs on July 13, 2023, at MW9S-23. It should be noted that groundwater level monitoring is ongoing to determine the seasonal highest groundwater level which usually occurs in Spring. Groundwater samples were collected from three monitoring wells at the site and analyzed for the Niagara Sanitary/Storm Sewers Discharge Limits Discharge Limits. All analyzed parameters were within guideline values.		

DEWATERING CONSIDERATIONS

Short Term	Short-term dewatering rate outside periods of active precipitation, under normal conditions, was determined to be approximately 27,993 L/day (0.32 L/s. Normal conditions are considered to be weather conditions that should be expected during the operation of the construction dewatering. Normal operation does not include extreme weather events.
Long Term	Long-term dewatering volume was determined to be approximately 27,993 L/day (0.32 L/s). The following two options are proposed to implement groundwater control measures for this volume: use of weeping tiles and perimeter drainage to avoid the potential inflow of groundwater into the underground parking level post-construction, subject the approval, or waterproof of the underground parling level below the established "*seasonally high groundwater level*" plus the required buffer zone (nominally 1.0 m to 1.5 m above).
Monitoring and Mitigation Plans	Monitoring, mitigation, and contingency plans are provided. The monitoring plans include dewatering abstraction, construction, and settlement monitoring. Mitigation includes methods to limit adverse dewatering settlement.

PERMIT CONSIDERATIONS

Dewatering Permit	The dewatering rate for the proposed underground level excavation without rainfall was determined to be approximately 27,993 L/day (0.32 L/s). An Environmental Activity and Sector Registry EASR registration and permit to take water (PTTW) will not be required for this volume of water taking, as the estimated water taking is less than 50,000 L/day, respectively. However, temporary discharge application to the Niagara Peninsula Conservation Authority (NPCA) is required and should be completed.

IMPACTS CONSIDERATION

Construction	The radius of influence from the proposed dewatering was conservatively determined to be approximately 5.0 m. Potential geotechnical impacts are anticipated within 5.0 m of Site during dewatering at the Site. However, surrounding buildings and roads adjacent to Site should be monitored by geotechnical instrumentation to determine impact, if any.

Table of Contents

<u>Appendices:</u>

1.0 INTRODUCTION

1.1 Background

Landtek Limited (Landtek) has been retained by Globizen Developments Inc. and Court Holdings Limited to complete a Hydrogeological Investigation for the proposed development at 4933 Victoria Avenue North in Vineland, Ontario (the Site or development).

The Site is roughly rectangular in shape and comprises an area of approximately 1.6 hectares (4.0 acres) and is situated approximately 25 m north of the intersection of Laurie Avenue and Victoria Avenue North, in Vineland Station (Town of Lincoln), Ontario. It is bound by residential properties to the south (followed by Laurie Avenue); a conservation area (including a stream) followed by residential properties to the east, Lake Ontario to the north, and Victoria Avenue North (followed by parkland, residential, and institutional properties) to the west. The Site location is shown on Figure 1, in Appendix A.

It is understood that the proposed development is to consist of fourteen (14) to fifteen (15) storeys hotel/residential towers with one level of underground parking. The Site Plan and P1 Level Plan are shown in Figures 2 and 3 in Appendix A, respectively as provided by **gh3**.

The purpose of the Hydrogeological Investigation is to evaluate the groundwater conditions at the site, delineate possible development/post-development effects, and suggest mitigation measures to minimize the effects to the shallow groundwater system during and post-development. Specifically, the report provides the following:

- A description of the hydrogeologic setting of the Site and a summary of the existing soil and groundwater conditions at the site.

- Identification of hydrogeologic features such as zones of significant groundwater recharge and discharge.

- Assessment of the requirement for groundwater control during construction, if any.

1.2 Work Scope and Report Organization

The scope of work for this investigation includes the following:

- <u>Review of available background information.</u> A review of published works of available geologic and hydrogeologic information for the site including topographical and geological maps and water well records. A review of Meteorological data to assess the local climate.

- <u>Site Assessment.</u> A detailed visual inspection of the site and surrounding area to identify and document local topography, surface water drainage features, and the potential presence of significant hydrogeological features such as closed depressions (areas of ground water recharge), seeps, springs, or the presence of phreatophytic vegetation.

- <u>A subsurface investigation.</u> Drilling of boreholes and monitoring wells at the Site to characterize the subsurface soil and/or bedrock as well as assess the site-specific groundwater conditions.

- Hydraulic Conductivity Tests. In-situ rising head tests were completed in selected installed monitoring wells to assess the subsurface soil and/or bedrock hydraulic conductivity.

- Groundwater Monitoring. Groundwater level monitoring was conducted in all monitoring wells in order to assess the depth of groundwater level across the site.

The report is organized as follows:

Section 1 contains a brief introduction to the project and the scope of work undertaken by Landtek.

Section 2 outlines the methodologies followed during completion of the desktop study and the field investigation.

Section 3 summarizes the findings of the investigation. It includes:
- a description of the physical setting
- the results of the field investigation

Section 4 provides Water Taking Evaluation and Impact Assessment

Section 5 provides Monitoring Plan.

Section 6 provides Mitigation Plan.

Section 7 provides Summary and Conclusions.

Section 8 provides recommendations.

Section 9 provides Closure.

Section 10 provides References.

Section 11 provides Limitations.

2.0 METHODOLOGY

2.1 Desktop Study

A review of published works was done of available geological and hydrogeological information for the site including topographic and geologic maps.

The Ministry of Environment, Conservation and Park (MECP) water well database for the local area was also accessed and the individual well record obtained for wells located within 500 m radius of the Site.

2.2 Site Inspection to Assess Hydrogeologic Features

Landtek conducted a visual assessment of the Site on February 2, 2023, to assess the presence of features which may be significant from a hydrogeologic viewpoint. In particular, the site was inspected to assess the following:

- The presence of closed drainage features, depressions, or sandy areas which may allow for ponding and significant or enhanced infiltration of water.

- Assessment of the presence of phreatophytic vegetation which may indicate seasonally high groundwater levels and/or groundwater discharge and seepage.

- Identification of any zones of visible seepage or groundwater discharge.

2.3 Field Investigation

2.3.1 Drilling and Well Installation

Fieldwork undertaken at the site by Landtek included clearance of underground services, borehole layout, borehole drilling and soil sampling, and field supervision. A total of 11 boreholes (boreholes BH1 to BH11A) were drilled between April 14th and 27th, 2022. An additional total of nine boreholes (boreholes BH1-23 to BH9) were drilled between July 4th and 7th, 2023.

Full time supervision of drilling and soil sampling operations was carried out by a representative of Landtek. The boreholes were drilled using a Diedrich D-50 track mounted drill rig equipped with continuous flight, solid and hollow stem augers and were extended to depths of between approximately 2.6 m and 12.1 m below existing ground level. Boreholes encountering ultimate auger refusal were extended from bedrock refusal using NQ-gauge, rotary coring methodologies.

Boreholes BH2, BH3, BH8, BH9A, BH11A, BH1-23, BH2-23, BH3-23, BH4-23, BH5-23, BH6-23, BH8-23 and BH9-23 were completed as monitoring wells and renamed BH/MW2, BH/MW3, BH/MW8, BH/MW9A, BH/MW11A, BH/MW1S/D-23, BH/MW2S/D-23, BH/MW3S/D-23, BH/MW4/4S-23, BH/MW5S-23, BH/MW6-23, BH/MW8S-23, and BH/MW9S/D-23, respectively. The monitoring well consisted of new/sealed 50 mm polyvinyl chloride (PVC) screen with No.10 slots threaded onto a matching riser. The screens and risers were pre-threaded including o-ring seals such that no glues or solvents were used to connect the pipe sections. A J-Plug lockable air-tight cap was installed on the riser.

The annular space between the PVC riser pipes and each borehole wall was backfilled to at least 0.3 m above the top of the screen with selected silica sand. A bentonite seal was placed immediately above the sand pack to a height just below grade. Each monitoring well was finished with a monumental protective steel casing, which was cemented in-place.

The monitoring well installation details are presented on the respective borehole logs in Appendix B.

A summary of the monitoring well installation details is presented on the following page in Table 1. The locations of the monitoring wells are shown on Figure 4, in Appendix A.

Table 1. Construction Details

Monitoring Well ID	Easting* (NAD83)	Northing* (NAD83)	Ground Surface Elevation (masl)**	Well Depth (mbgs)	Stick-up (m)	Screened Interval (m)	Screened Material
BH/MW1S-23	630475	4783599	77.70	6.0	0.86	3.0–6.0	Shale
BH/MW1D-23	630475	4783599	77.70	10.6	0.85	7.6–10.6	Shale
BH/MW2S-23	630431	4783631	78.00	3.0	0.88	1.5–3.0	Fill
BH/MW2D-23	630431	4783631	78.00	4.5	0.87	1.5–4.5	Fill
BH/MW3S-23	630415	4783550	78.71	6.0	0.92	3.0–6.0	Shale
BH/MW3D-23	630415	4783550	78.71	10.6	0.84	7.6–10.6	Shale
BH/MW4S-23	630415	4783550	79.16	6.0	0.86	3.0–6.0	Shale
BH/MW4-23	630415	4783550	79.16	3.0	0.87	1.5–3.0	Shale
BH/MW5S-23	630422	4783495	79.38	6.0	0.86	3.0–6.0	Shale
BH/MW6-23	630419	4783470	79.96	3.0	0.87	1.5–3.0	Shale
BH/MW8S-23	630455	4783508	78.43	4.5	0.67	1.5–4.5	Clayey Silt/Shale
BH/MW9S-23	630469	4783557	78.37	4.5	0.87	1.5–4.5	Shale
BH/MW9D-23	630469	4783557	78.37	12.1	0.78	9.1–12.1	Shale

Notes:
masl = meters above sea level
mbgs = meters below ground level
m = meters
* Values are approximate by GPS +/- 4 m
** Values are approximate. Based on Topographical Survey Map by J.D. BARNES, Reference No. 22-16-360-360-00.

2.3.2 Monitoring Well Development

Well Development: Each of the installed monitoring wells MW1S-23, MW1D-23, MW2S-23, MW2D-23, MW3S-23, MW3D-23, MW4S-23, MW4-23, MW5S-23, MW6-23, MW8S-23, MW9S-23, MW9D-23 was developed to remove any sediment that may have been introduced during installation and to improve the hydraulic properties of the formation against which the wells were screened. The monitoring wells were developed by Landtek staff on staff on July 12, 2023. Development employed electric well pump/waterra tubing with foot valves and each well was developed until a visible decrease in turbidity and steady flow were observed.

2.3.3 Groundwater Monitoring

Depths to groundwater in monitoring wells MW1S-23, MW1D-23, MW2S-23, MW2D-23, MW3S-23, MW3D-23, MW4S-23, MW4-23, MW5S-23, MW6-23, MW8S-23, MW9S-23, MW9D-23 were obtained manually by Landtek staff on July 12, August 18, September 20, October 6, and October 17, 2023.

2.3.4 Groundwater Sampling

On October 24, 2023, groundwater samples were collected from monitoring wells MW23-2S, MW23-4S and MW23-8S after purging. All collected samples were stored in a cooler with freezer packs after collection and during transport to the PARACEL Laboratories Ltd. in Hamilton, Ontario. The samples were analyzed for the Niagara Region Sanitary and Storm Sewers Discharge analysis. PARACEL is accredited by the *Canadian Associations for Laboratory Accreditation Inc*. (CALA).

2.3.5 Hydraulic Conductivity Testing

Hydraulic conductivity tests were completed in monitoring wells MW23-3S, MW23-8S, and MW23-9S to provide estimates of the hydraulic conductivity for the zones against which the screens for the wells were set. Rising head tests were conducted by Landtek on October 23, 2023. The tests involved the extraction of a volume of groundwater to displace the water level. A datalogger programed at 2 second intervals were used to record the water level response during the tests.

Data Analysis: The rising head test data were analyzed using AqteSolve Professional Version 4.5 software package developed by Glenn M. Duffield of HydroSOLVE Inc. applying the Hvorslev analysis solutions, depending on hydrogeology.

3.0 FINDINGS

3.1 Topography, Drainage and Hydrology

The topography at the Site ranges from approximately 75 masl to 79 masl, with a gentle slope from the south to north portions.

The Site is located in the Niagara Peninsula Source Protection Area in a Highly Vulnerable Aquifer Area with a Score of 6.

According to the Karst Map of Southern Ontario, the Site is not located in a potential Karst area – areas of carbonate rock units identified as most susceptible to karst processes (Ontario Geological Survey).

3.2 Regional Physiography

The site is situated in the physiographic region known as the Iroquois Plain. The Iroquois Plain was formed in the late Pleistocene times by a body of water known as Lake Iroquois, which emptied eastward at Rome, New York (Chapman and Putnam, 1984). Lake Iroquois was characterized by higher water levels than the present-day Lake Ontario, caused by an ice sheet blocking the present-day St. Lawrence River valley. When the St. Lawrence valley became free of ice, the water level dropped to a level much lower than the present Lake Ontario levels (Karrow, 1959). The Iroquois Plain is characterized by sands deposited by Lake Iroquois.

3.3 Climate

The site is located in the Mixedwood Plains ecozone of Ontario (Natural Resources Canada, 2012). The general climate data presented below in Table 2 was obtained from Environment Canada publications and from the Environment Canada online database. Average climate data was taken from the St. Catharines A station for the period of 1981 to 2010.

Table 2. 1981 to 2010 Climate Normals for St. Catharines A Station (as averages)

	Daily Average Temperature (°C)	Average Rainfall (mm)	Average Snowfall (cm)	Average Precipitation (mm)
January	-3.8	30.8	38.6	65.2
February	-2.9	28.9	29.3	54.9
March	-1.1	39.3	23.2	61.7
April	7.4	71.2	5.8	77.0
May	13.7	76.3	0.4	76.8
June	19.0	86.0	0.0	85.9
July	21.9	77.8	0.0	77.8
August	20.8	70.3	0.0	70.3
September	16.6	90.6	0.0	90.6
October	10.4	67.0	0.1	67.0
November	4.6	72.1	9.6	81.6
December	-0.9	44.0	30.1	71.5
Year	9.0	754.2	137.1	880.1

3.4 Regional Geology

The Site is in the physiography of southern Ontario known as the Haldimand Clay Plain. The Haldimand Clay Plain is located between the Niagara Peninsula and Lake Erie occupying all of Niagara Peninsula except the fruit belt below the escarpment. The underlying rocks consists of a succession of Paleozoic beds dipping slightly southward under Lake Erie. Dolostone of the Lockport Formation form the vertical cliffs along the brow of the escarpment and underlies a narrow strip of the plain to be succeeded southward by dolostone of the Guelph Formation. The surficial geology of the site silty clay.

3.5 Local and Regional Hydrogeology

The hydrostratigraphic units are subdivided into two distinct groups based on their permeability, their ability to allow groundwater movement: an aquitard and an aquifer. An aquitard inhibits groundwater flow due to its low permeability, while an aquifer is permeable enough to allow flow of groundwater for sustainable use. The major regional hydrostratigrahic units that control groundwater at the Site are as follows:

Fine-Textured Glaciolacustrine

This comprised of silt and clay are the native surficial unit within the study area. These soils often occur at surface, or beneath the layer of fill, where present. This deposit forms a regional aquitard which limits groundwater flow and infiltration within the study area.

Silty Clay Till

This is present below the surficial glaciolacustrine deposits. This unit is found throughout the Region and acts as a surficial aquitard which limits groundwater flow and recharge to deeper bedrock aquifers.

Bertie Formation

This constitutes the bedrock and usually contains karstic porosity. The bedrock acts as an aquifer where solution enhanced fractures are present or along weathered bedding plans. The dolostone is expected to have a low hydraulic conductivity and restrict groundwater flow where unfractured. Shallow zones near the bedrock/overburden interface can have a relatively high permeability and hydraulic conductivity due to weathering. This zone acts as thin, unconfined aquifer with sufficient permeability to transit significant volumes of groundwater.

3.6 MECP Water Well Records and Groundwater Resources

The Ministry of Environment, Conservation and Park (MECP) Water Well Information System is a publicly available database which contains information such as groundwater well location, well construction details, static water level, geologic units encountered with depth, general water quality observations, water use, date of construction, and screened interval.

The MECP records for wells located within approximately 500 meters of the site were reviewed to assess the general nature and use of the groundwater resource in the area and to characterize local hydrogeologic conditions.

Desk Top Study

A search of the MECP water well records within approximately 500 m of the site, conducted on April 21, 2023, returned a total of thirty-five (35) wells comprising of three (3) water wells, one (1) abandoned water well, two (2) test/monitoring wells, nineteen (19) observation wells, and ten (10) wells with unknown uses. The records were reviewed to assess the general nature of the groundwater resource in the area and to characterize local hydrogeologic conditions. The locations of the wells are shown on Figure 5 in Appendix A. The well records summary is provided in Appendix C.

A summary of the data obtained from the well survey is presented below.

All Well Uses

- Water Well 3
- Abandoned Water Well 1
- Observation Wells 19
- Monitoring Wells 2
- Well without Information 10
- **Total..35**

Water Wells Construction

- Wells terminated in bedrock 3
- Wells terminated in overburden 1
- **Total..4**

Water Wells Depths

- Less than 15 m 2
- Between 15 m and 30 m..2
- **Total..4**

Based on the well records review, it was determined that there are three (3) water wells within a 500 m radius of the Site.

3.7 Results of Site Inspection

A detailed site inspection was conducted on February 2, 2023, to assess the presence of features which may be significant from a hydrogeologic viewpoint.

At the time of the Landtek's Site visit, the Site consisted of a vacant industrial property, with no above ground structures in place; the buildings were demolished in early 2023.

3.8 Results of Subsurface Investigation

The borehole information is generally consistent with the geological data identified from published geology of the area, with the predominant soils comprising sands, silts, clay and silt tills overlying red shale bedrock.

The detailed borehole/monitoring well logs are presented in Appendix B, with the ground conditions encountered by the boreholes discussed in the following sections.

Existing Pavement Structure

Boreholes BH1, BH/MW2, BH/MW3 and BH/MW8 were drilled within existing pavement areas, with a concrete thickness of approximately 150 mm to 475 mm. No pavement granular materials were encountered.

Fill Materials

Fill material was encountered in all boreholes from ground surface or underlying the existing pavement structure and extends to depths between approximately 0.6 m and 4.5 m below existing ground level. The fill comprises of sands, silts, clays and gravels, with varying proportions of orange brick fragments, gravel, concrete fragments, asphalt fragments, organics and limestone fragments, and is primarily brown, grey and red in colour.

Clayey Silt to Silty Clay

Clayey silt to silty clay deposits were encountered underlying the fill material in boreholes BH1, BH/MW4S-23, BH/MW7-23 and BH/MW8S-23 and extends to depths between approximately 1.4 m and 2.5 m below existing ground level. The clayey silt to silty clay was observed to be generally brown and red in colour and contains traces of gravel, sand, iron staining and peat.

Silt Till

Silt till was encountered in boreholes BH/MW1S/D-23, BH/MW3S/D-23 and BH/MW5S-23 underlying the fill materials and extends to depths between approximately 1.5 m to 2.3 m below existing ground surface. The silt till contains traces of gravel, iron staining and red shale fragments, and is generally brown in colour.

Clayey Silt to Silty ClayTill

Clayey silt to silty clay till was encountered **only** in boreholes BH1, BH/MW2, BH/MW3, BH/MW4, BH5, BH6, BH7, BH/MW8, BH/MW9A, and BH/MW11A underlying the fill and sand material and extends to depths of approximately 1.5 m and 3.0 m below existing pavement surface. The till is generally red and brown and contains traces of gravel, sand, iron staining and red shale fragments.

Bedrock

Red shale of the Queenston Formation was encountered in all boreholes at depths of between approximately 1.5 m to 4.5 m below existing ground level, equating to Geodetic elevations between approximately 79.6 m and 73.4 m. The shale is red and grey in colour, is very weak to weak, completely to highly weathered and was primarily recovered as “*residual soil*”.

3.9 Groundwater Monitoring

Depths to groundwater in monitoring wells MW1, MW2, MW3, MW4, MW5, MW6, and MW7 were obtained manually by Landtek staff on July 13, August 18, September 20, October 6, and October 17, 2023. The readings are presented on the following page in Table 3. It should be noted that groundwater level monitoring is ongoing to determine the seasonal highest groundwater level which usually occurs in Spring.

Table 3. Groundwater Monitoring Data

MW ID	Date	Total Depth (mbgs)	Stick-up (m)	Water Strike (m)*	Water Level (mbgs)	Water Level (masl)	Ground Elevation (masl)**
MW1S-23	13-Jul-23	6.0	0.86	None	4.19	73.51	77.70
	18-Aug-23				3.12	74.58	
	20-Sep-23				3.22	74.48	
	6-Oct-23				3.32	74.38	
	17-Oct-23				3.42	74.28	
MW1D-23	13-Jul-23	10.6	0.85	None	3.03	74.67	77.70
	18-Aug-23				3.45	74.25	
	20-Sep-23				3.29	74.41	
	6-Oct-23				3.38	74.32	
	17-Oct-23				3.48	74.22	
MW2S-23	13-Jul-23	3.0	0.88	None	2.90	74.8	78.00
	18-Aug-23				2.99	74.71	
	20-Sep-23				3.20	74.5	
	6-Oct-23				2.75	74.95	
	17-Oct-23				3.33	74.37	
MW2D-23	13-Jul-23	4.5	0.87	None	2.94	74.76	78.00
	18-Aug-23				Dry	NA	
	20-Sep-23				Dry	NA	
	6-Oct-23				2.78	74.92	
	17-Oct-23				3.16	74.54	
MW3S-23	13-Jul-23	6.0	0.92	None	3.15	74.55	78.71
	18-Aug-23				3.28	74.42	
	20-Sep-23				3.34	74.36	
	6-Oct-23				3.35	74.35	
	17-Oct-23				3.48	74.22	
MW3D-23	13-Jul-23	10.6	0.84	None	3.28	74.42	78.71
	18-Aug-23				4.39	73.31	
	20-Sep-23				3.50	74.2	
	6-Oct-23				3.50	74.2	
	17-Oct-23				3.63	74.07	
MW4S-23	12-Jul-23	6.0	0.86	None	4.11	73.59	79.16
	18-Aug-23				3.06	74.64	
	20-Sep-23				3.11	74.59	
	6-Oct-23				2.96	74.74	
	17-Oct-23				3.22	74.48	
MW4-23	13-Jul-23	3.0	0.87	None	2.42	75.28	79.16
	18-Aug-23				Dry	NA	
	20-Sep-23				2.56	75.14	
	6-Oct-23				2.64	75.06	
	17-Oct-23				2.35	75.35	
MW5S-23	13-Jul-23	6.0	0.86	None	3.35	74.35	79.38
	18-Aug-23				3.43	74.27	
	20-Sep-23				3.62	74.08	
	6-Oct-23				3.35	74.35	
	17-Oct-23				3.61	74.09	
MW6-23	13-Jul-23	3.0	0.87	None	3.09	74.61	79.96
	18-Aug-23				Dry	NA	
	20-Sep-23				2.95	74.75	
	6-Oct-23				3.03	74.67	
	17-Oct-23				3.01	74.69	
MW8S-23	12-Jul-23	4.5	0.67	None	3.09	74.61	78.43
	18-Aug-23				2.66	75.04	
	20-Sep-23				2.73	74.97	
	6-Oct-23				2.78	74.92	
	17-Oct-23				2.74	74.96	
MW9S-23	13-Jul-23	4.5	0.87	None	2.18	75.52	78.37
	18-Aug-23				2.25	75.45	

	20-Sep-23				2.48	75.22	
	6-Oct-23				2.42	75.28	
	17-Oct-23				2.44	75.26	
MW9D-23	13-Jul-23	12.1	0.78	none	3.43	74.27	78.37
	18-Aug-23				3.77	73.93	
	20-Sep-23				3.35	74.35	
	6-Oct-23				3.33	74.37	
	17-Oct-23				3.43	74.27	

Notes:
[*] water strike/groundwater seepage
masl = meters above sea level
mbtop = meters below top of pipe
mbgs = meters below ground level
m = meters
* Values are approximate by GPS +/- 4 m
** Values are approximate. Based on Topographical Survey Map by J.D. BARNES, Reference No. 22-16-360-360-00.

3.10 Hydraulic Gradients and Flow

Vertical Hydraulic Gradient

Groundwater generally flows from the shallow to deeper aquifers as leakage across the aquitards. However, this may vary locally, and the direction of vertical flow depends on the relative heads in the different layers. Leakage rates vary locally depending on the magnitude of the vertical gradients and on the thickness and hydraulic conductivity of the confining units.

Horizontal Hydraulic Gradient

Based on topography and mapping information of the area, the ground surface elevations indicate that the area generally slopes down to the north towards Lake Ontario and east towards NPCA regulated lands ultimately draining into an unnamed creek located adjacent to the east of the Site. The local groundwater flow direction has been inferred to be in a northerly direction towards Lake Ontario, located adjacent to the north of the Site. Shallow ground water direction may be influenced by trenches for municipal infrastructure, underground utilities, conduits, structures, variations in subsurface strata, and changes in local topography.

3.11 Estimated Hydraulic Conductivity

3.11.1 Hydraulic Conductivity Tests Analysis

The analyses were completed using the Hvorslev method (Fetter, 1994). The graphical results of the hydraulic conductivity analysis are presented in Appendix D, and the results are summarized below in Table 4.

Table 4. Hydraulic Conductivity Results

Monitoring Well	Hydraulic Conductivity (m/s)	Screened Material
MW23-3S	4.572×10^{-8}	Shale Bedrock
MW23-8S	7.468×10^{-9}	Clayey Silt/Shale Bedrock
MW23-9S	9.772×10^{-8}	Shale Bedrock

The results indicate that the hydraulic conductivity values of the screened clayey silt/shale bedrock at the site range from 7.468×10^{-9} m/s to 9.772×10^{-8} m/s, with a geometric mean of 3.219×10^{-8} m/s.

3.12 Groundwater Quality

Copies of the laboratory Certificates of Analysis are provided in Appendix E. The results of the analyzed groundwater samples collected from monitoring wells MW23-3S, MW3-4S and MW23-8S were compared to the Niagara Sanitary/Storm Sewers Discharge Limits Discharge Limit.

Based on the analysis, all analyzed parameters were within guideline values.

4.0 WATER TAKING EVALUATION & IMPACT ASSESSMENT

Proposed Development

The proposed development is to comprise of the following: a stepped, five-storey to 17-storey residential tower, with three partial, above-ground parking levels, one underground parking level extending across the Site. The underground parking level is shown on Figure 3 in Appendix A.

Underground Parking Level

Based on Figure 3, the dimensions of the equivalent rectangle of the underground parking level were determined to be approximately 210.0 m x 65.5 m

The maximum depth of the underground levels is estimated to be 4.1 mbgs. As a result, a dewatering depth of approximately 0.5 m below the excavation bottom (4.6 mbgs) is assumed in order to keep the bottom of the excavation dry during construction.

Static Water Levels

Depths to groundwater in all monitoring wells were obtained manually by Landtek staff on July 13, August 18, September 20, October 6, and October 17, 2023. The readings are presented in Table 3 of this report. Based on the recorded groundwater levels, the highest water level was determined to be 2.18 mbgs on July 13, 2023, at MW9S-23. It should be noted that groundwater level monitoring is ongoing to determine the seasonal highest groundwater level which usually occurs in Spring.

4.1 Groundwater Dewatering Requirements

Groundwater seepage will occur where excavations are made below the groundwater level. If groundwater levels are intercepted within the excavation, adequate pumping should be provided to prevent significant groundwater volumes from accumulating.

In order to evaluate the potential groundwater control requirements during construction of the proposed underground parking levels, depth to groundwater of 2.18 mbgs, (the highest groundwater level recorded on July 13, 2023, at MW9S-23, was assumed for the entire site.

The method suitable for dewatering an area depends on the locations, type, size and depth of the dewatering needs; and the hydrogeological conditions such as stratification, thickness, and hydraulic conductivity of the foundation soils below the water table into which the excavation extends or is underlain. It is assumed that any groundwater dewatering for the Site excavations would likely be completed with standard construction sump pump/well points or equivalent, depending on conditions encountered such as water table elevation and subsurface materials. The pumps must appropriately be used to prevent the pumping of fines and loss of ground during dewatering activities and the flow of water should be appropriately managed so that sediment is not pumped into the proposed discharge point.

For the purposes of this assessment, an open excavation was assumed. The use of conventional shoring could further reduce the amount of groundwater infiltration and should be determined in consultation with the selected subcontractor.

4.1.1 Dewatering Calculations

The potential groundwater flow rate to the underground parking excavation was estimated using the dewatering equation for a fully penetrated well of unconfined aquifer fed by circular source (Powers, et. al., 2007):

$$Q = \pi K\,(H^2 - h_w^2)/(\ln R_o/r_e)$$

Where: Q = pumping rate [m^3/s]
K = hydraulic conductivity [m/s]
H = saturated thickness of the aquifer before dewatering [m]
h_w = saturated thickness of the aquifer after dewatering [m]
R_o. = radius of cone of depression [m]
r_e = equivalent radius [m]

The radius of cone of depression R can be estimated using:

$$R_o = Ch*Sqrt(K)$$

Where: C = is a factor equal to 3000 for radial flow to a pumping well

h = H- h_w =required drawdown [m]
K = hydraulic conductivity [m/s]

Dewatering of a rectangular area can be accomplished by using an equivalent radius (r_e) to assess drawdown where r_e is given by the following equation:

$r_e = (a + b)/\pi)$ (applies when $a/b<1.5$ and $R_o >> r_e$)
r_e = Sqrt (length*width/π) (applies when $a/b>1.5$ and $R_o << r_e$)

Dewatering Estimate

The volume of groundwater required to be pumped for dewatering the excavation associated with the underground level construction, assuming there is no rainfall and applying a factor of safety of 1.5, was determined be approximately 27,993 L/day (0.32 L/s) and the radius of influence determined to be approximately 5.0 m with a factor of safety of 5. These calculations and associated assumptions are provided on Table 1, Appendix F.

4.2 Dewatering Considerations

4.2.1 Estimating Dewatering Volume

4.2.2 Short Term Dewatering Volume

The short-term dewatering rate outside periods of active precipitation, under normal conditions, was determined to be approximately 27,993 L/day (0.32 L/s).

Normal conditions are considered to be weather conditions that should be expected during the operation of the construction dewatering. Normal operation does not include extreme weather events.

4.2.3 Long Term Groundwater Control (Post Construction)

Long-term dewatering volume was determined to be approximately 27,993 L/day (0.32 L/s). The following two options are proposed to implement groundwater control measures for this volume: use of weeping tiles and perimeter drainage to avoid the potential inflow of groundwater into the underground parking level post-construction, subject the approval, or Waterproof of the underground parling level below the established "*seasonally high groundwater level*" plus the required buffer zone (nominally 1.0 m to 1.5 m above).

4.2.4 Dewatering Permit

The dewatering rate for the proposed underground level excavation without rainfall was determined to be approximately 27,993 L/day (0.32 L/s). An Environmental Activity and Sector Registry EASR registration and permit to take water (PTTW) will not be required for this volume of water taking, as the estimated water taking is less than 50,000 L/day, respectively. However, temporary discharge application to the Niagara Peninsula Conservation Authority (NPCA). is required and should be completed.

4.2.5 Dewatering Procedure

Based on the results of the hydraulic conductivity tests, seepage through the overburden and bedrock beneath the Site should be feasible to be handled by a sump and well point dewatering system. The type of dewatering system to be used should be discussed with a dewatering contractor and be evaluated based on anticipated low and high volumes estimates.

The following general construction practices should be implemented to minimize the volume of water to be extracted:

- Schedule construction outside the spring period when the water table is typically elevated and avoid constructing during period of active precipitation.
- Excavation should be staged or constructed in such a manner to be able to manage dewatering volume conveniently.
- Reduce the length of time during which the excavation cut remains open.

4.2.6 Water Management and Discharge Plan

Water extracted during construction dewatering is required to be discharged into an approved sewer near the Site.

As per the Sewers ByLaw, in order to issue a discharge approval, information relating to the quality and quantity of the discharge must be provided to the Niagara Region. It is strongly recommended that the applicant provide this information eight to twelve weeks prior to the proposed start of discharge.

The rate and total volume of the discharge during dewatering should be recorded. This would require that the discharge line be equipped with a flow meter capable of monitoring the discharge rate and a volume totalizer to record the total volume of water discharge. The discharge rate and total daily flow should be recorded with the records maintained on site.

If needed, a weir tank and filter bag should be utilized during dewatering to reduce total suspended solids (TSS) and turbidity prior to discharging of the water into either a sewer system or surface water.

A T-Coupling and valves should be installed downstream of the flow meter, which, if necessary, can be operated to divert flow for mitigation purposes.

4.3 Assessment of Potential Impacts and Water Management

4.3.1 Impact to Existing Groundwater Users

A search of the Ontario MECP within an area extending about 500 m outward from the site was completed.

A summary of the MECP Well Records is presented in Appendix C; and the approximate locations of the wells are shown on Figure 5 in Appendix A. Based on review, four (4) water wells was identified within 500 m radius of the Site.

The estimated radius of influence from the proposed basement level excavation dewatering was determined to be approximately 5 m. As a result, potential impacts on water wells located within 500 m radius of the Site are not anticipated, as none is within the radius of influence of 5 m.

4.3.2 Impact to Surface Water and Natural Functions of the Ecosystem

The nearest Surface Water/Natural Function of the Ecosystem to the Site are Lake Ontario located approximately 10 m to the north of the Site, and NPCA regulated lands which ultimately drains into an unnamed creek located adjacent to the east of the Site.

The estimated radius of influence due to proposed dewatering at the Site was determined to be 5 m. As a result, it is not anticipated that there will be impact to the Lake or the NPCA regulated lands, from the proposed development. However, it is recommended to monitor impacts to these identified Surface Water/Natural Function of the Ecosystem during construction, to determine impact, if any.

4.3.3 Contaminants Impacts

This occurs when pre-existing soil or groundwater contamination is mobilised and transported where transmission pathways are created.

A Phase Environmental Site Assessment (ESA) Report dated September 2023 was completed at the Site by Landtek.

Based on the results of the Phase One ESA, a Phase Two ESA was recommended to be completed for this Site to investigate the identified potential environmental concerns prior to the submission of a Record of Site Condition.

4.3.4 Geotechnical Impacts

Geotechnical impacts occur where the geotechnical properties or state of the ground are changed by groundwater dewatering activities. The most common type of impact in this category is ground settlement, with the corresponding risk of distortion and damage to structures, services and other sensitive infrastructure.

situated approximately 25 m north of the intersection of Laurie Avenue and Victoria Avenue North, in Vineland Station (Town of Lincoln), Ontario. It is bound by residential properties to the south (followed by Laurie Avenue); a conservation area (including a stream) followed by residential properties to the east, Lake Ontario to the north, and Victoria Avenue North (followed by parkland, residential, and institutional properties) to the west.

Based on the above, potential geotechnical impacts are anticipated during dewatering at the Site within a radius of influence of approximately 5.0 m. However, surrounding buildings and roads adjacent to Site should be monitored by geotechnical instrumentation to determine impact, if any.

Dewatering could be by pumping from a sump and well point dewatering system. These systems used for lowering the water table within the excavation should be properly screened and installed to ensure that pumping will not remove sediment from low permeability overburden aquifers. Removal of significant fines may result in the formation of voids and the loss of ground. It is anticipated that there will not be impact beyond the radius of influence of 5.0 m.

The proposed monitoring and mitigation plans are presented in Sections 5 and 6, respectively.

5.0 MONITORING PLAN

5.1 Construction Monitoring

Once construction dewatering is initiated it will be difficult to stop pumping or significantly reduce the rate of pumping without disrupting construction activities. It will however be possible to monitor the drawdown response at the construction site and to adjust the pumping rate to optimize drawdown and the associated pumping rate.

5.2 Management of Dewatering Abstraction

5.2.1 Monitoring, Trigger Levels and Management Responses

Abstraction management is critical to ensure target water levels within the construction zone are met, but that over-pumping does not occur.

Target groundwater levels in- and outside excavations should be set individually for each dewatering monitoring well based on location, aquifer and construction requirements, in-line with stated dewatering aims above.

Trigger levels for wells should typically be set 0.5 m above the dewatering target and 1.0 m below the dewatering target to give a 1.5 m target operational zone. These targets may be reviewed and adjusted to decrease size of the operational target zone and increase the factor of safety.

If monitoring indicates that dewatering zone groundwater levels exceed the upper trigger levels (i.e., required drawdown is not being achieved or maintained) the following management actions should be carried out (in order of preference):

- Adjust automatic pump start and stop water levels.
- Increase pumping rates within the constraints of the system; and/or
- Install additional abstraction capacity (well points, spears or sump pumps).

If monitoring indicates that excavation zone groundwater levels are below the lower trigger levels (i.e., excessive drawdown) the following management actions should be carried out (in order of preference):

- Adjust automatic pump start and stop water levels; and/or
- Decrease pumping rates; and/or
- Reduce the number of pumps operating.

5.2.2 Contingency Responses

If management responses prove to be insufficient to achieve and maintain the target levels, excavations should be slowed or suspended to enable contingencies to be implemented. Available contingency measures include the following (in order of preference):

- Construction of additional dewatering wells, spears or sumps.
- Construction of additional drains or groundwater control structures.

Excavation should resume when the required drawdown is obtained.

5.3 Settlement Monitoring

Ground settlement can be caused by two principal mechanisms:

- Increases in effective stress as a result of lowering of groundwater levels, resulting in compression and consolidation of the ground. Such settlements are the unavoidable consequence of lowering of groundwater level.
- Removal of fine particles from the ground (loss of fines) which can occur when poorly controlled sump pumping draws out soil particles with the pumped water. With good design and implementation, loss of fines (and the associated settlement risk) can be avoided.

Implementation of a settlement monitoring plan should be completed within an approximate radius of influence of 5.0 m of the Site, the estimated radius of influence from dewatering. Prior to commencing dewatering, condition surveys of adjacent properties that could potentially be affected by dewatering, considering anticipated effects and specific dewatering design, should be completed. However, it is recommended that surrounding buildings and roads adjacent to Site be monitored by geotechnical instrumentation to determine impact, if any.

Temporary access permit should be obtained from properties and utilities owners with the estimated radius of influence of the Site on a case-by-case basis prior to construction.

The following monitoring measures are recommended to be carried out before and during the temporary dewatering:

- Complete a pre-excavation condition survey and install settlement monitoring monuments and or markers at the existing buildings and roadways within the estimated zone of influence. This should be done to document existing ground elevations and building/structure conditions.

- The settlement monitoring monuments (markers) should be surveyed prior to the dewatering to establish a baseline and surveyed on a daily basis during the dewatering.

- A typical settlement monitoring system should comprise a series of settlement markers sited at various distances beyond and at the site, within the zone of influence of groundwater drawdown. Monitoring points should be surveyed to an accuracy of +/-2 mm. Note that the reference benchmark must be located beyond the extent of the anticipated influence of groundwater drawdown. For very high-risk projects, incorporation of piezometer standpipes will allow confirmation of the field groundwater drawdown and will enable calibration of field settlement observation with theoretical assessments.

- Alert and Action settlement thresholds should be set, selected through theoretical assessment of anticipated settlements and review of sensitivity of adjacent structures and infrastructures. It is prudent to implement staged groundwater drawdown, providing holding points to allow adequate time to enable observation of the delayed settlement response of the ground.

- The monitoring program will include review and alert levels. If instrument readings exceed “review” levels, the Proponent and its Contractor will jointly assess the necessity of altering the method, rate, or sequence of construction.

- The survey results should be provided to the project geotechnical engineer for evaluation. The estimated potential and actual settlements should also be reviewed by a structural engineer to assess the potential damage to the existing structures.

6.0 MITIGATION PLAN

The groundwater dewatering activities will result in localized depression of the groundwater table, and it is not anticipated that there will impact beyond the radius of influence of 5.0 m. However, it is recommended that surrounding buildings and roads adjacent to Site should be monitored by geotechnical instrumentation to determine impact, if any.

Mitigation would involve the reduction or elimination of the impacts induced by construction dewatering. As noted above, the potential exists for dewatering to cause ground settlement, with the corresponding risk of distortion and damage to structures, services and other sensitive infrastructure.

Methods to limit adverse dewatering settlement should include the following:

- Settlement associated with loss of fines should be mitigated through appropriate design of the dewatering system to control flow velocity and provide screens and/or filters matched to the grading of the in-situ soils. Entrainment of fines must be monitored during construction; actions could include analysis of TSS in discharge water and/or monitoring of accumulation of sediment in sedimentation tanks.

- Drawdown-induced ground settlement should be mitigated though pre-construction estimation of groundwater drawdown and settlement coefficients to identify risk prior to drawing the groundwater down, and water level monitoring in monitoring wells to check that larger drawdown than anticipated at distance from the excavation are not occurring.

- Differential settlement is most problematic. This should be reduced by managing the rate of drawdown and understanding where clear changes in soil type occur. Should potentially damaging settlement be indicated, these can be mitigated by installing groundwater cut-offs to stem or restrict groundwater flow and limit drawdown beyond the site.

- Sufficient temporary support should be provided for excavations to maintain stability, where seeps might otherwise induce progressive collapse of the sides of the excavation.

- During dewatering, staged drawdowns (where appropriate) should be implemented and field settlement and water level changes beyond the immediate site monitored, comparing against theoretical settlements and water levels to allow warning of potential dewatering settlement issues.

At “alert” levels, the dewatering should be reduced to a lower rate or ceased temporarily, and alternative measures considered for the excavation, which should be approved by the project geotechnical engineer and project team.

If the settlement monitoring indicates an undesirable deformation, the project manager should order construction operations to cease until the necessary mitigation measures are undertaken.

In the event that a property or infrastructure owner submits a claim for damages, the Developer should conduct further investigations and, if appropriate, negotiate a settlement.

7.0 SUMMARY AND CONCLUSIONS

The following summarizes the results of the investigation:

- The borehole information is generally consistent with the geological data identified from published geology of the area, with the predominant soils comprising sands, silts, clay and silt tills overlying red shale bedrock.

- The presence of significant hydrogeologic features such as closed depressions (areas of ground water recharge), seeps, springs, or the presence of phreatophytic vegetation were not observed during the visit and inspection.

- The topography at the Site ranges from approximately 75 masl to 79 masl, with a gentle slope from the south to north portions.

- The local groundwater flow direction has been inferred to be in a northerly direction towards Lake Ontario, located adjacent to the north of the Site. Shallow ground water direction may be influenced by trenches for municipal infrastructure, underground utilities, conduits, structures, variations in subsurface strata, and changes in local topography.

- Depths to groundwater in all monitoring wells were obtained manually by Landtek staff on July 13, August 18, September 20, October 6, and October 17, 2023. Based on the recorded groundwater levels, the highest water level was determined to be 2.18 mbgs on July 13, 2023, at MW9S-23. It should be noted that groundwater level monitoring is ongoing to determine the seasonal highest groundwater level which usually occurs in Spring.

- Groundwater samples were collected from three monitoring wells at the site and analyzed for the Niagara Sanitary/Storm Sewers Discharge Limits Discharge Limits. All analyzed parameters were within guideline values.

- The short-term dewatering rate outside periods of active precipitation, under normal conditions, was determined to be approximately 27,993 L/day (0.32 L/s. Normal conditions are considered to be weather conditions that should be expected during the operation of the construction dewatering. Normal operation does not include extreme weather events.

- Long-term dewatering volume was determined to be approximately 27,993 L/day (0.32 L/s). The following two options are proposed to implement groundwater control measures for this volume: use of weeping tiles and perimeter drainage to avoid the potential inflow of groundwater into the underground parking level post-construction, subject the approval, or waterproof of the underground parling level below the established "*seasonally high groundwater level*" plus the required buffer zone (nominally 1.0 m to 1.5 m above).

- The dewatering rate for the proposed underground level excavation without rainfall was determined to be approximately 27,993 L/day (0.32 L/s). An Environmental Activity and Sector Registry EASR registration and permit to take water (PTTW) will not be required for this volume of water taking, as the estimated water taking is less than 50,000 L/day, respectively. However, temporary discharge application to the Niagara Peninsula Conservation Authority (NPCA) is required and should be completed.

8.0 RECOMMENDATIONS

The following general construction practices are recommended to minimize the volume of water to be extracted:

- Schedule construction outside the spring period when the water table is typically elevated and avoid construction during period of active precipitation.

- Reduce, where practicable, the length of time during which the open cut remains open.

- Install valves on the individual well point to allow for the flow adjustment.

Potential geotechnical impacts are anticipated during dewatering at the Site within a radius of influence of approximately 5.0 m. However, surrounding buildings and roads adjacent to Site should be monitored by geotechnical instrumentation to determine impact, if any.

As per the Sewers ByLaw, in order to issue a discharge approval, information relating to the quality and quantity of the discharge must be provided to the Niagara Region. It is strongly recommended that the applicant provide this information eight to twelve weeks prior to the proposed start of discharge.

9.0 CLOSURE

We trust this report is satisfactory for your purposes. If you have any questions regarding our submission, please do not hesitate to contact Landtek.

Yours truly,

Landtek Limited

PROFESSIONAL GEOSCIENTIST
HENRY N. EREBOR
PRACTISING MEMBER
2792
ONTARIO

Henry Erebor, M.Sc., P.Geo.,

gh3*
55 OSSINGTON AVENUE
SUITE 100
TORONTO, CANADA
M6J 2Y9
info@gh3.ca

Pat Hanson
Raymond Chow

December 19, 2023.

Brandon Donnely
Globizen

Reference: 4933 Victoria Avenue North, Lincoln, Ontario

Dear Sir:

Please be advised that the above-referenced building will be constructed in compliance with the 2015 Ontario Building Code (OBC), and equipped with a Fire Protection System conforming to the NFPA 13 Standards for Installation of Sprinkler Systems and specifically:

1. All structural members and floors will be of fire-resistive construction per the Fire Underwriters Survey (FUS) 2020 with 2-hour ratings per the OBC.
2. All vertical openings and exterior vertical communications will be constructed with a 1-hour fire rating.

Yours truly,

Raymond Chow OAA RAIC
Partner gh3* architects

Determining Number of Cartridges for Flow Based Systems

Date	**4/22/2024**	**Black Cells =**	**Calculation**

Site Information

Project Name	**4933 Victoria Avenue**		
Project Location	**Lincoln, ON**		
OGS ID	**Stormfilter**		
Drainage Area, Ad	**0.24**	ac	(0.098 ha)
Impervious Area, Ai	**0.19**	ac	
Pervious Area, Ap	**0.05**		
% Impervious	**80%**		
Runoff Coefficient, Rc	**0.77**		
Treatment storm flow rate, Q_{treat}	**0.10**	cfs	(2.72 L/s)
Peak storm flow rate, Q_{peak}	**1.17**	cfs	(33.2 L/s)

Filter System

Filtration brand	**StormFilter**	
Cartridge height	**12**	in
Specific Flow Rate	**2.00**	gpm/ft^2
Flow rate per cartridge	**10.00**	gpm

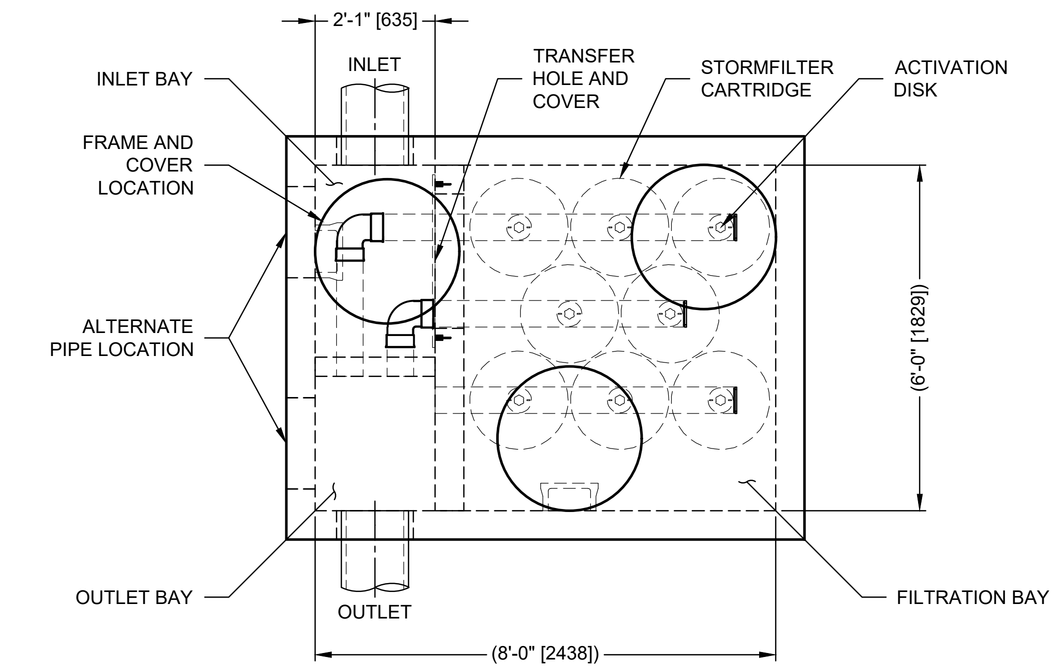
SUMMARY

Number of Cartridges	5
Media Type	Perlite

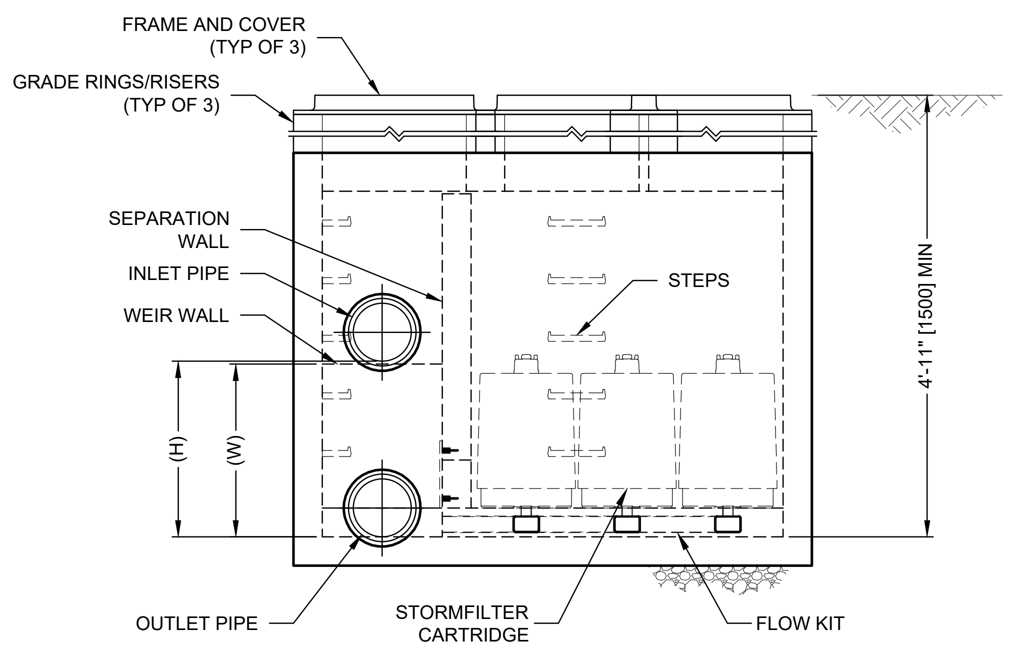
Event Mean Concentration (EMC)	**120**	mg/L
Annual TSS Removal	**80%**	
Percent Runoff Capture	**90%**	

Recommend SFPD 0608 vault or cast-in-place

conteches.com
200 Enterprise Drive
Scarborough, ME 04074
Phone 877-907-8676
Fax 207-885-9825



PLAN



ELEVATION

The Stormwater Management StormFilter®

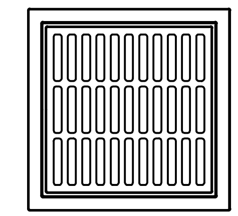
THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING U.S. PATENTS: 5,322,629; 5,524,576; 5,707,527; 5,985,157; 6,027,639; 6,649,048; RELATED FOREIGN PATENTS, OR OTHER PATENTS PENDING.

STORMFILTER DESIGN NOTES

- STORMFILTER TREATMENT CAPACITY VARIES BY CARTRIDGE COUNT AND LOCALLY APPROVED SURFACE AREA SPECIFIC FLOW RATE. PEAK CONVEYANCE CAPACITY TO BE DETERMINED BY ENGINEER OF RECORD
- A 6' x 8' [1829 x 2438] PEAK DIVERSION STYLE STORMFILTER IS SHOWN WITH THE MAXIMUM NUMBER OF CARTRIDGES (8) AND IS AVAILABLE IN A LEFT INLET (AS SHOWN) OR A RIGHT INLET CONFIGURATION
- ALL PARTS AND INTERNAL ASSEMBLY PROVIDED BY CONTECH UNLESS NOTED OTHERWISE

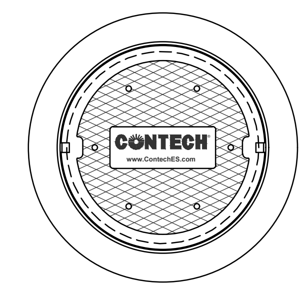
CARTRIDGE SIZE (in. [mm])	27 [686]			18 [457]			LOW DROP		
RECOMMENDED HYDRAULIC DROP (H) (ft. [mm])	3.05 [930]			2.3 [701]			1.8 [549]		
HEIGHT OF WEIR (W) (ft. [mm])	3.00 [914]			2.25 [686]			1.75 [533]		
SPECIFIC FLOW RATE (gpm/sf [L/s/m^2])	2 [1.36]	1.67* [1.13]*	1 [0.68]	2 [1.36]	1.67* [1.13]*	1 [0.68]	2 [1.36]	1.67* [1.13]*	1 [0.68]
CARTRIDGE FLOW RATE (gpm [L/s])	22.5 [1.42]	18.79 [1.19]	11.25 [0.71]	15 [0.95]	12.53 [0.79]	7.5 [0.47]	10 [0.63]	8.35 [0.53]	5 [0.32]

* 1.67 gpm/sf [1.13 L/s/m^2] SPECIFIC FLOW RATE IS APPROVED WITH PHOSPHOSORB® (PSORB) MEDIA ONLY



FRAME AND GRATE

(24" SQUARE)
(NOT TO SCALE)



FRAME AND COVER

(30" ROUND)
(NOT TO SCALE)

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID	
WATER QUALITY FLOW RATE (cfs [L/s])	
PEAK FLOW RATE (cfs [L/s])	
RETURN PERIOD OF PEAK FLOW (yrs)	
CARTRIDGE FLOW RATE	
CARTRIDGE SIZE (27, 18, LOW DROP (LD))	
MEDIA TYPE (PERLITE, ZPG, PSORB)	
NUMBER OF CARTRIDGES REQUIRED	
INLET BAY RIM ELEVATION	
FILTER BAY RIM ELEVATION	

PIPE DATA:	INVERT	MATERIAL	DIAMETER
INLET PIPE 1			
INLET PIPE 2			
OUTLET PIPE			

NOTES/SPECIAL REQUIREMENTS:

PERFORMANCE SPECIFICATION

FILTER CARTRIDGES SHALL BE MEDIA-FILLED, PASSIVE, SIPHON ACTUATED, RADIAL FLOW, AND SELF CLEANING. **RADIAL MEDIA DEPTH SHALL BE 7" [178].** FILTER MEDIA CONTACT TIME SHALL BE AT LEAST **37 SECONDS**. SPECIFIC FLOW RATE SHALL BE **2 GPM/SF [1.36 L/s/m^2] (MAXIMUM)**. SPECIFIC FLOW RATE IS THE MEASURE OF THE FLOW (GPM) DIVIDED BY THE MEDIA SURFACE CONTACT AREA (SF). MEDIA VOLUMETRIC FLOW RATE SHALL BE **6 GPM/CF [13.39 L/s/m^3] OF MEDIA (MAXIMUM).**

GENERAL NOTES

1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. ALTERNATE DIMENSIONS ARE IN MILLIMETERS [mm] UNLESS NOTED OTHERWISE.
4. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHTS, PLEASE CONTACT YOUR CONTECH REPRESENTATIVE. www.ContechES.com
5. STORMFILTER WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING. CONTRACTOR TO CONFIRM STRUCTURE MEETS REQUIREMENTS OF PROJECT.
6. STRUCTURE SHALL MEET AASHTO HS20 LOAD RATING, ASSUMING EARTH COVER OF 0' - 10' [3048] AND GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION. CASTINGS SHALL MEET AASHTO M306 AND BE CAST WITH THE CONTECH LOGO.

INSTALLATION NOTES

A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STORMFILTER STRUCTURE.
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH OUTLET PIPE INVERT WITH OUTLET BAY FLOOR.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT CARTRIDGES FROM CONSTRUCTION-RELATED EROSION RUNOFF.
F. CONTRACTOR TO REMOVE THE TRANSFER OPENING COVER WHEN THE SYSTEM IS BROUGHT ONLINE.

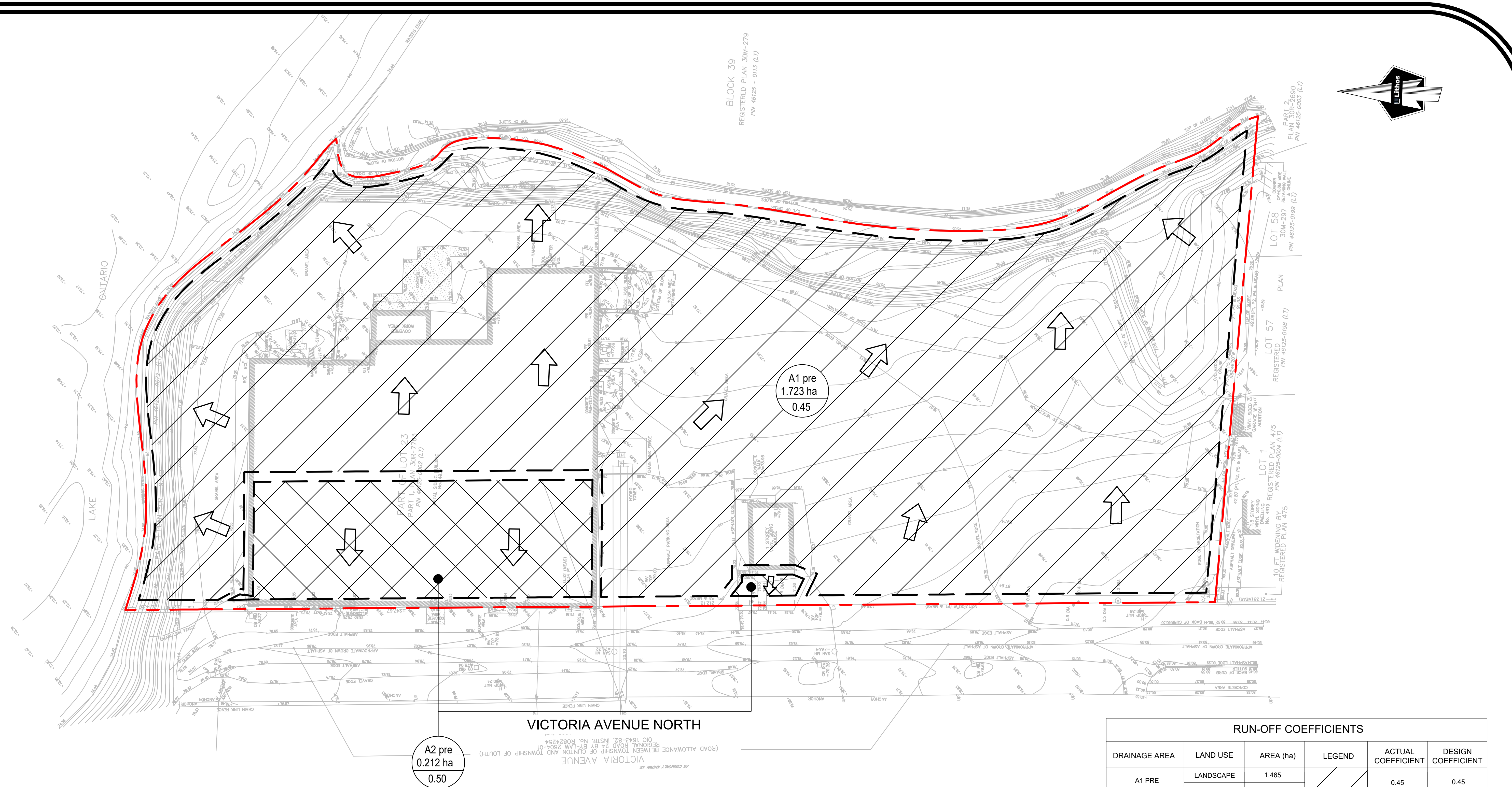
CONTECH® ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

SFPD0608 (6' x 8')
PEAK DIVERSION STORMFILTER
STANDARD DETAIL

I:\COMMON\CAD\TREATMENT\10 STORMFILTER\40 STANDARD DRAWINGS\SFPD\STANDARD\IN PROCESS\DWG\SFPD0608-DTL.DWG 10/20/2020 3:06 PM

Appendix C

Storm Analysis



RUN-OFF COEFFICIENTS

DRAINAGE AREA	LAND USE	AREA (ha)	LEGEND	ACTUAL COEFFICIENT	DESIGN COEFFICIENT
A1 PRE	LANDSCAPE	1.465		0.45	0.45
	HARDSCAPE	0.258			
A2 PRE	LANDSCAPE	0.000		0.90	0.50
	HARDSCAPE	0.212			

Lithos

150 Bermondsey Road, Toronto, Ontario M4A 1Y1

LEGEND

A1 pre — STORM DRAINAGE AREA NUMBER

0.04 ha — DRAINAGE AREA (ha)

0.50 — COMPOSITE RUNOFF COEFFICIENT

PRE-DEVELOPMENT STORM DRAINAGE AREA

PROPERTY LINE

OVERLAND FLOW ROUTE

PRE-DEVELOPMENT DRAINAGE AREA PLAN

MIXED USE DEVELOPMENT
4933 VICTORIA AVENUE NORTH
LINCOLN, ONTARIO

DATE: APRIL 2024

PROJECT No: UD23-045

SCALE: N.T.S.

FIGURE No: DAP1

Lithos

Prepared by: Dimitra Frysali, P.E., M.A.Sc.
Reviewed by: Catherine Agiou, P.E., M.A.Sc.

Rational Method
Pre-Development Flow Calculation
4933 Victoria Avenue North

File No. UD23-045

Town of Lincoln

Date: April 2024

Input Parameters

Area Number	Area (ha)	Actual "C"	Design "C"	Tc (min.)
A1 pre towards Creek discharged to Lake Ontario	1.723	0.45	0.45	10
A2 pre towards Victoria Avenue North discharged to Lake Ontario	0.212	0.90	0.50	10
Total	**1.935**			

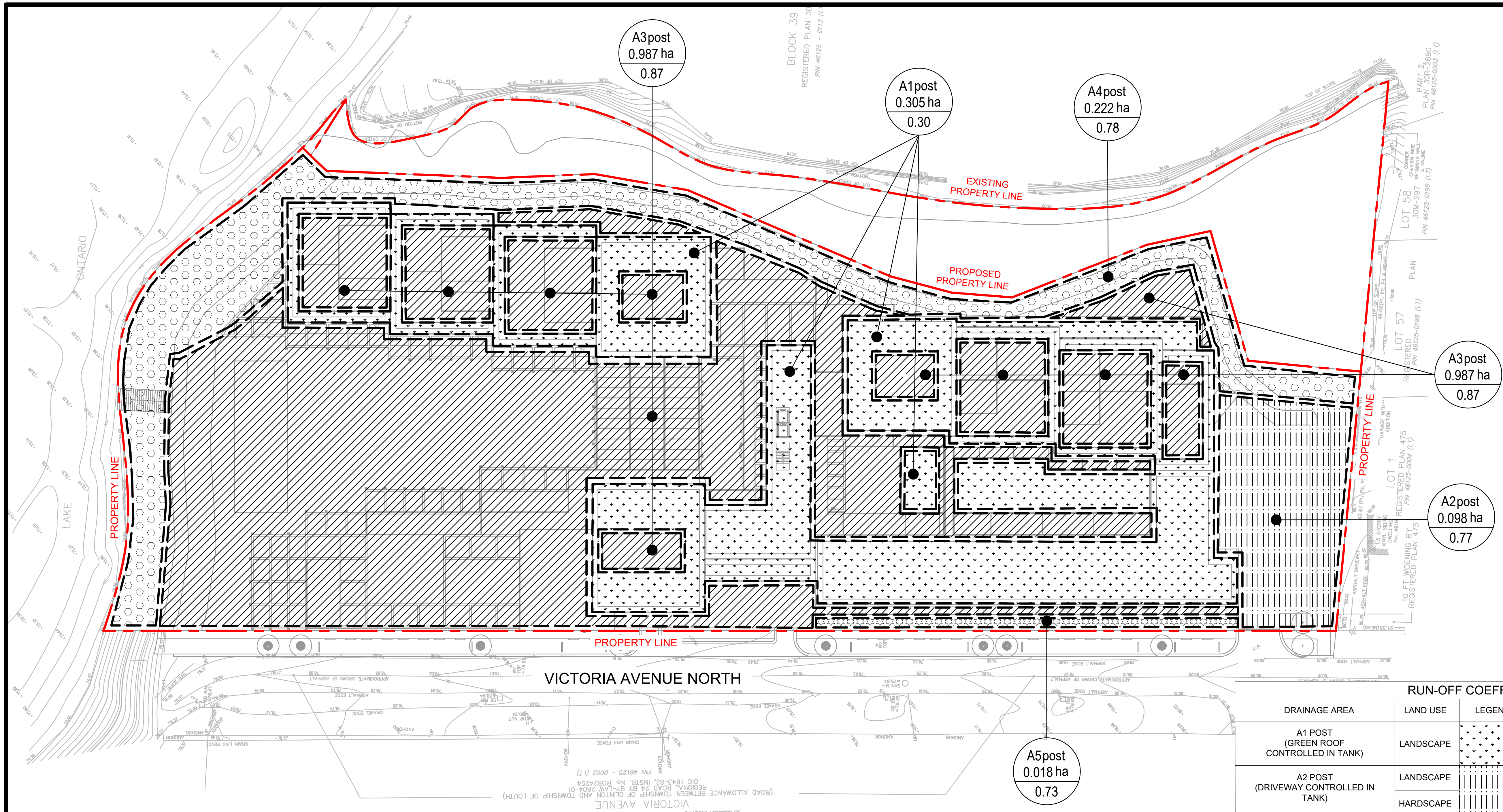
Rational Method Calculation

Event 5 yr
IDF Data Set City of Lincoln
a = 28.90
c = -0.669

Area Number	A (ha)	C	AC	Tc (min.)	I (mm/h)	Q (m^3/s)	Q (L/s)
A1 pre towards Creek discharged to Lake Ontario	1.723	0.45	0.78	10	95.8	0.206	206.4
A2 pre towards Victoria Avenue North discharged to Lake Ontario	0.212	0.50	0.11	10	95.8	0.028	28.2
Total	1.935	0.46	0.88	10	95.8	0.235	234.6

Event 100 yr
IDF Data Set City of Lincoln
a = 48.10
c = -0.669

Area Number	A (ha)	C	AC	Tc (min.)	I (mm/h)	Q (m^3/s)	Q (L/s)
A1 pre towards Creek discharged to Lake Ontario	1.723	0.45	0.78	10	159.5	0.344	343.6
A2 pre towards Victoria Avenue North discharged to Lake Ontario	0.212	0.50	0.11	10	159.5	0.047	46.9
Total	1.935	0.46	0.88	10	159.5	0.390	390.5



DRAINAGE AREA	AREA (ha)	TOTAL AREA (ha)
A1 POST	0.305	1.630
A2 POST	0.098	
A3 POST	0.987	
A4 POST	0.222	
A5 POST	0.018	

RUN-OFF COEFFICIENTS

DRAINAGE AREA	LAND USE	LEGEND	AREA (ha)	INITIAL COEFFICIENT	COMPOSITE COEFFICIENT
A1 POST (GREEN ROOF CONTROLLED IN TANK)	LANDSCAPE		0.305	0.30	0.30
A2 POST (DRIVEWAY CONTROLLED IN TANK)	LANDSCAPE		0.022	0.30	0.77
	HARDSCAPE		0.076	0.90	
A3 POST (ROOFTOP/ TERRACES/ LANDSCAPED AREA - CONTROLLED IN TANK)	LANDSCAPE		0.043	0.30	0.87
	HARDSCAPE		0.944	0.90	
A4 POST (UNCONTROLLED SITE AREA TOWARDS CREEK)	LANDSCAPE		0.046	0.30	0.78
	HARDSCAPE		0.176	0.90	
A5 POST (UNCONTROLLED SITE AREA TOWARDS VICTORIA AVENUE NORTH)	LANDSCAPE		0.005	0.30	0.73
	HARDSCAPE		0.013	0.90	

Lithos

150 Bermondsey Road, Toronto, Ontario M4A 1Y1

LEGEND

A1 pre — STORM DRAINAGE AREA NUMBER

0.04 ha — DRAINAGE AREA (ha)

0.50 — COMPOSITE RUNOFF COEFFICIENT

POST-DEVELOPMENT STORM DRAINAGE AREA

PROPERTY LINE

POST-DEVELOPMENT DRAINAGE AREA PLAN
MIXED USE DEVELOPMENT
4933 VICTORIA AVENUE NORTH
LINCOLN, ONTARIO

DATE: APRIL 2024 | PROJECT No: UD23-045

SCALE: N.T.S. | FIGURE No: DAP2

Lithos

Modified Rational Method - Five Year Storm
Site Flow and Storage Summary

4933 Victoria Avenue North
Town of Lincoln
Date: April 2024

5 Year Design Storm	
a=	28.90
c=	-0.67
I=	A(T)^c

Drainage Area A1 Post

Green Roof - Controlled in tank 1

Area (A1) = **0.305** ha
"C" = **0.30**
AC1 = **0.09**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Release Rate = **24.4** L/s

Drainage Area A2 Post

Driveway Area /Landscape/Hardscape Areas driven to OGS - Controlled in Tank 1

Area (A2) = **0.098** ha
"C" = **0.77**
AC2 = **0.08**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **20.0** L/s

Tributary Area	ha	"C"
Hardscape	0.076	0.90
Landscape	0.022	0.30
Total	**0.098**	**0.77**

Tank 1

A1+ A2

Controlled Release Rate Achieved (Pump from Tank 1) = **3.0** L/s

Max. Required Storage (Tank Size 1) = **39.02** m^3

Storage Tank 1 Footprint Area = 65.4 m^2

Drainage Area A3 Post

Rooftop/Terraces/Landscaped Area - Controlled in tank 2

Area (A3) = **0.987** ha
"C" = **0.87**
AC3 = **0.86**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **229.6** L/s

Tributary Area	ha	"C"
Hardscape	0.944	0.90
Landscape	0.043	0.30
Total	**0.987**	**0.87**

Tank 2

A3

Controlled Release Rate Achieved (Pump from Tank 2) = **35.0** L/s

Max. Required Storage (Tank Size 2) = **135.17** m^3

Storage Tank 2 Footprint Area = 155.0 m^2

Drainage Area A4 Post

Uncontrolled Site Area towards Creek

Area (A4) = **0.222** ha
"C" = **0.78**
AC4 = **0.17**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **45.8** L/s

Tributary Area	ha	"C"
Hardscape	0.176	0.90
Landscape	0.046	0.30
Total	**0.222**	**0.78**

Drainage Area A5 Post

Uncontrolled Site Area towards Victoria Avenue North

Area (A5) = **0.018** ha
"C" = **0.73**
AC5 = **0.01**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **3.5** L/s

Tributary Area	ha	"C"
Hardscape	0.013	0.90
Landscape	0.005	0.30
Total	**0.018**	**0.73**

Total Site

Area = A1+ A2 + A3 + A4 + A5

Total 5-yr Pre-Development Release Rate = **234.6** L/s

Total Controlled Release Rate Achieved (Pump) = **38.0** L/s

Total Site Uncontrolled Release Rate = **49.4** L/s

Total Site Release Rate = **87.4** L/s

Total Required Storage (Tank Size) = **174.19** m^3

(1) Time (min)	(2) Rainfall Intensity (mm/hr)	(3) Storm Runoff (A1 Post) (m^3/s)	(4) Runoff Volume (A1 Post) (m^3)	(5) Storm Runoff (A2 Post) (m^3/s)	(6) Runoff Volume (A2 Post) (m^3)	(7) Total Storm Runoff Volume (m^3)	(8) Released Volume (m^3)	(9) Storage Volume (m^3)	(10) Storage Depth of Tank (m)
10.0	95.8	0.024	14.61	0.020	11.98	26.59	1.80	24.79	0.38
15.0	73.1	0.019	16.71	0.015	13.70	30.41	2.70	27.71	0.42
20.0	60.3	0.015	18.38	0.013	15.07	33.45	3.60	29.85	0.46
25.0	51.9	0.013	19.79	0.011	16.22	36.01	4.50	31.51	0.48
30.0	46.0	0.012	21.02	0.010	17.23	38.25	5.40	32.85	0.50
35.0	41.4	0.011	22.12	0.009	18.13	40.26	6.30	33.96	0.52
40.0	37.9	0.010	23.12	0.008	18.95	42.08	7.20	34.88	0.53
45.0	35.0	0.009	24.04	0.007	19.71	43.75	8.10	35.65	0.55
50.0	32.6	0.008	24.89	0.007	20.41	45.30	9.00	36.30	0.56
55.0	30.6	0.008	25.69	0.006	21.06	46.75	9.90	36.85	0.56
60.0	28.9	0.007	26.44	0.006	21.68	48.12	10.80	37.32	0.57
65.0	27.4	0.007	27.15	0.006	22.26	49.41	11.70	37.71	0.58
70.0	26.1	0.007	27.83	0.005	22.81	50.64	12.60	38.04	0.58
75.0	24.9	0.006	28.47	0.005	23.34	51.81	13.50	38.31	0.59
80.0	23.8	0.006	29.09	0.005	23.84	52.93	14.40	38.53	0.59
85.0	22.9	0.006	29.67	0.005	24.32	54.00	15.30	38.70	0.59
90.0	22.0	0.006	30.24	0.005	24.79	55.03	16.20	38.83	0.59
95.0	21.3	0.005	30.79	0.004	25.24	56.02	17.10	38.92	0.60
100.0	20.5	0.005	31.31	0.004	25.67	56.98	18.00	38.98	0.60
105.0	19.9	0.005	31.82	0.004	26.09	57.91	18.90	39.01	0.60
110.0	19.3	0.005	32.32	0.004	26.49	58.81	19.80	39.01	0.60
115.0	18.7	0.005	32.80	0.004	26.88	59.68	20.70	38.98	0.60
120.0	18.2	0.005	33.26	0.004	27.26	60.53	21.60	38.93	0.60
125.0	17.7	0.004	33.72	0.004	27.64	61.35	22.50	38.85	0.59
130.0	17.2	0.004	34.16	0.004	28.00	62.15	23.40	38.75	0.59
135.0	16.8	0.004	34.59	0.003	28.35	62.93	24.30	38.63	0.59
140.0	16.4	0.004	35.00	0.003	28.69	63.70	25.20	38.50	0.59
145.0	16.0	0.004	35.41	0.003	29.03	64.44	26.10	38.34	0.59
150.0	15.7	0.004	35.81	0.003	29.35	65.17	27.00	38.17	0.58
155.0	15.3	0.004	36.20	0.003	29.68	65.88	27.90	37.98	0.58
160.0	15.0	0.004	36.59	0.003	29.99	66.57	28.80	37.77	0.58
165.0	14.7	0.004	36.96	0.003	30.30	67.26	29.70	37.56	0.57

(1) Time (min)	(11) Storm Runoff (A3 Post) (m^3/s)	(12) Runoff Volume (A3 Post) (m^3)	(13) Total Storm Runoff Volume (m^3)	(14) Released Volume (m^3)	(15) Storage Volume (m^3)	(16) Storage Depth of Tank (m)	(17) Storm Runoff (A4 Post) (m^3/s)	(18) Runoff Volume (A4 Post) (m^3)	(19) Storm Runoff (A5 Post) (m^3/s)	(20) Runoff Volume (A5 Post) (m^3)
10.0	0.230	137.75	137.75	21.00	116.75	0.75	0.046	27.50	0.004	2.11
15.0	0.175	157.53	157.53	31.50	126.03	0.81	0.035	31.45	0.003	2.41
20.0	0.144	173.27	173.27	42.00	131.27	0.85	0.029	34.59	0.002	2.65
25.0	0.124	186.56	186.56	52.50	134.06	0.86	0.025	37.25	0.002	2.86
30.0	0.110	198.16	198.16	63.00	135.16	0.87	0.022	39.56	0.002	3.03
35.0	0.099	208.53	208.53	73.50	135.03	0.87	0.020	41.63	0.002	3.19
40.0	0.091	217.96	217.96	84.00	133.96	0.86	0.018	43.52	0.001	3.34
45.0	0.084	226.62	226.62	94.50	132.12	0.85	0.017	45.25	0.001	3.47
50.0	0.078	234.66	234.66	105.00	129.66	0.84	0.016	46.85	0.001	3.59
55.0	0.073	242.19	242.19	115.50	126.69	0.82	0.015	48.35	0.001	3.71
60.0	0.069	249.26	249.26	126.00	123.26	0.80	0.014	49.77	0.001	3.81
65.0	0.066	255.95	255.95	136.50	119.45	0.77	0.013	51.10	0.001	3.92
70.0	0.062	262.31	262.31	147.00	115.31	0.74	0.012	52.37	0.001	4.01
75.0	0.060	268.37	268.37	157.50	110.87	0.72	0.012	53.58	0.001	4.11
80.0	0.057	274.16	274.16	168.00	106.16	0.68	0.011	54.74	0.001	4.20
85.0	0.055	279.72	279.72	178.50	101.22	0.65	0.011	55.85	0.001	4.28
90.0	0.053	285.06	285.06	189.00	96.06	0.62	0.011	56.91	0.001	4.36
95.0	0.051	290.21	290.21	199.50	90.71	0.59	0.010	57.94	0.001	4.44
100.0	0.049	295.18	295.18	210.00	85.18	0.55	0.010	58.93	0.001	4.52
105.0	0.048	299.99	299.99	220.50	79.49	0.51	0.010	59.89	0.001	4.59
110.0	0.046	304.64	304.64	231.00	73.64	0.48	0.009	60.82	0.001	4.66
115.0	0.045	309.16	309.16	241.50	67.66	0.44	0.009	61.72	0.001	4.73
120.0	0.044	313.54	313.54	252.00	61.54	0.40	0.009	62.60	0.001	4.80
125.0	0.042	317.81	317.81	262.50	55.31	0.36	0.008	63.45	0.001	4.86
130.0	0.041	321.96	321.96	273.00	48.96	0.32	0.008	64.28	0.001	4.93
135.0	0.040	326.01	326.01	283.50	42.51	0.27	0.008	65.09	0.001	4.99
140.0	0.039	329.96	329.96	294.00	35.96	0.23	0.008	65.88	0.001	5.05
145.0	0.038	333.81	333.81	304.50	29.31	0.19	0.008	66.65	0.001	5.11
150.0	0.038	337.58	337.58	315.00	22.58	0.15	0.007	67.40	0.001	5.17
155.0	0.037	341.26	341.26	325.50	15.76	0.10	0.007	68.13	0.001	5.22
160.0	0.036	344.87	344.87	336.00	8.87	0.06	0.007	68.85	0.001	5.28
165.0	0.035	348.40	348.40	346.50	1.90	0.01	0.007	69.56	0.001	5.33

Modified Rational Method - Hundred Year Storm
Site Flow and Storage Summary

4933 Victoria Avenue North
Town of Lincoln
Date: April 2024

Drainage Area A1 Post

Green Roof - Controlled in tank 1

Area (A1) = **0.305** ha
"C" = **0.30**
AC1 = **0.09**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Release Rate = **40.5** L/s

Drainage Area A2 Post

Driveway Area /Landscape/Hardscape Areas driven to OGS - Controlled in Tank 1

Area (A2) = **0.098** ha
"C" = **0.77**
AC2 = **0.08**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **33.2** L/s

Tributary Area	ha	"C"
Hardscape	0.076	0.90
Landscape	0.022	0.30
Total	**0.098**	**0.77**

Tank 1

A1+ A2

Controlled Release Rate Achieved (Pump from Tank 1) = **3.0** L/s

Max. Required Storage (Tank Size) = **82.24** m^3

Storage Tank 1 Footprint Area = 65.4 m^2

Drainage Area A3 Post

Rooftop/Terraces/Landscaped Area - Controlled in tank 2

Area (A3) = **0.987** ha
"C" = **0.87**
AC3 = **0.86**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **382.1** L/s

Tributary Area	ha	"C"
Hardscape	0.944	0.90
Landscape	0.043	0.30
Total	**0.987**	**0.87**

Tank 2

A3

Controlled Release Rate Achieved (Pump from Tank 2) = **35.0** L/s

Max. Required Storage (Tank Size) = **289.58** m^3

Storage Tank 2 Footprint Area = 155.0 m^2

Drainage Area A4 Post

Uncontrolled Site Area towards Creek

Area (A4) = **0.222** ha
"C" = **0.78**
AC4 = **0.17**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **76.3** L/s

Tributary Area	ha	"C"
Hardscape	0.176	0.90
Landscape	0.046	0.30
Total	**0.222**	**0.78**

Drainage Area A5 Post

Uncontrolled Site Area towards Victoria Avenue North

Area (A5) = **0.018** ha
"C" = **0.73**
AC5 = **0.01**
Tc = **10.0** min
Time Increment = **5.0** min

Max. Rel. Rate = **5.8** L/s

Tributary Area	ha	"C"
Hardscape	0.013	0.90
Landscape	0.005	0.30
Total	**0.018**	**0.73**

Total Site

Area = A1+ A2 + A3 + A4 + A5

Total 5-yr Pre-Development Release Rate = **234.6** L/s

Total Controlled Release Rate Achieved (Pump) = **38.0** L/s

Total Site Uncontrolled Release Rate = **82.1** L/s

Total Site Release Rate = **120.1** L/s

Total Required Storage (Tank Size) = **371.82** m^3

100 Year Design Storm	
a=	48.10
c=	-0.67
I=	A(T)^c

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Time (min)	**Rainfall Intensity** (mm/hr)	**Storm Runoff (A1 Post)** (m^3/s)	**Runoff Volume (A1 Post)** (m^3)	**Storm Runoff (A2 Post)** (m^3/s)	**Runoff Volume (A2 Post)** (m^3)	**Total Storm Runoff Volume** (m^3)	**Released Volume** (m^3)	**Storage Volume** (m^3)	**Storage Depth of Tank** (m)	**Storm Runoff (A3 Post)** (m^3/s)	**Runoff Volume (A3 Post)** (m^3)	**Total Storm Runoff Volume** (m^3)	**Released Volume** (m^3)	**Storage Volume** (m^3)	**Storage Depth of Tank** (m)	**Storm Runoff (A4 Post)** (m^3/s)	**Runoff Volume (A4 Post)** (m^3)	**Storm Runoff (A5 Post)** (m^3/s)	**Runoff Volume (A5 Post)** (m^3)
10.0	159.5	0.041	24.32	0.033	19.94	44.26	1.80	42.46	0.65	0.382	229.26	229.26	21.00	208.26	1.34	0.076	45.77	0.006	3.51
15.0	121.6	0.031	27.82	0.025	22.80	50.61	2.70	47.91	0.73	0.291	262.19	262.19	31.50	230.69	1.49	0.058	52.35	0.004	4.01
20.0	100.3	0.025	30.59	0.021	25.08	55.67	3.60	52.07	0.80	0.240	288.39	288.39	42.00	246.39	1.59	0.048	57.58	0.004	4.41
25.0	86.4	0.022	32.94	0.018	27.00	59.94	4.50	55.44	0.85	0.207	310.49	310.49	52.50	257.99	1.66	0.041	61.99	0.003	4.75
30.0	76.5	0.019	34.99	0.016	28.68	63.67	5.40	58.27	0.89	0.183	329.81	329.81	63.00	266.81	1.72	0.037	65.85	0.003	5.05
35.0	69.0	0.018	36.82	0.014	30.18	67.00	6.30	60.70	0.93	0.165	347.07	347.07	73.50	273.57	1.76	0.033	69.29	0.003	5.31
40.0	63.1	0.016	38.48	0.013	31.54	70.03	7.20	62.83	0.96	0.151	362.76	362.76	84.00	278.76	1.80	0.030	72.43	0.002	5.55
45.0	58.3	0.015	40.01	0.012	32.80	72.81	8.10	64.71	0.99	0.140	377.18	377.18	94.50	282.68	1.82	0.028	75.30	0.002	5.77
50.0	54.3	0.014	41.43	0.011	33.96	75.40	9.00	66.40	1.02	0.130	390.57	390.57	105.00	285.57	1.84	0.026	77.98	0.002	5.98
55.0	51.0	0.013	42.76	0.011	35.05	77.81	9.90	67.91	1.04	0.122	403.08	403.08	115.50	287.58	1.86	0.024	80.48	0.002	6.17
60.0	48.1	0.012	44.01	0.010	36.08	80.09	10.80	69.29	1.06	0.115	414.86	414.86	126.00	288.86	1.86	0.023	82.83	0.002	6.35
65.0	45.6	0.012	45.19	0.009	37.04	82.24	11.70	70.54	1.08	0.109	426.00	426.00	136.50	289.50	1.87	0.022	85.05	0.002	6.52
70.0	43.4	0.011	46.32	0.009	37.96	84.28	12.60	71.68	1.10	0.104	436.58	436.58	147.00	289.58	1.87	0.021	87.16	0.002	6.68
75.0	41.4	0.011	47.39	0.009	38.84	86.23	13.50	72.73	1.11	0.099	446.66	446.66	157.50	289.16	1.87	0.020	89.18	0.002	6.84
80.0	39.7	0.010	48.41	0.008	39.68	88.09	14.40	73.69	1.13	0.095	456.31	456.31	168.00	288.31	1.86	0.019	91.10	0.001	6.98
85.0	38.1	0.010	49.39	0.008	40.48	89.87	15.30	74.57	1.14	0.091	465.56	465.56	178.50	287.06	1.85	0.018	92.95	0.001	7.13
90.0	36.7	0.009	50.33	0.008	41.26	91.59	16.20	75.39	1.15	0.088	474.45	474.45	189.00	285.45	1.84	0.018	94.72	0.001	7.26
95.0	35.4	0.009	51.24	0.007	42.00	93.24	17.10	76.14	1.16	0.085	483.02	483.02	199.50	283.52	1.83	0.017	96.44	0.001	7.39
100.0	34.2	0.009	52.12	0.007	42.72	94.84	18.00	76.84	1.18	0.082	491.29	491.29	210.00	281.29	1.81	0.016	98.09	0.001	7.52
105.0	33.1	0.008	52.97	0.007	43.42	96.38	18.90	77.48	1.19	0.079	499.29	499.29	220.50	278.79	1.80	0.016	99.68	0.001	7.64
110.0	32.1	0.008	53.79	0.007	44.09	97.88	19.80	78.08	1.19	0.077	507.03	507.03	231.00	276.03	1.78	0.015	101.23	0.001	7.76
115.0	31.1	0.008	54.59	0.006	44.74	99.33	20.70	78.63	1.20	0.075	514.55	514.55	241.50	273.05	1.76	0.015	102.73	0.001	7.87
120.0	30.3	0.008	55.36	0.006	45.38	100.74	21.60	79.14	1.21	0.072	521.85	521.85	252.00	269.85	1.74	0.014	104.19	0.001	7.99
125.0	29.4	0.007	56.11	0.006	46.00	102.11	22.50	79.61	1.22	0.071	528.95	528.95	262.50	266.45	1.72	0.014	105.61	0.001	8.10
130.0	28.7	0.007	56.85	0.006	46.60	103.44	23.40	80.04	1.22	0.069	535.86	535.86	273.00	262.86	1.70	0.014	106.99	0.001	8.20
135.0	28.0	0.007	57.56	0.006	47.18	104.74	24.30	80.44	1.23	0.067	542.60	542.60	283.50	259.10	1.67	0.013	108.33	0.001	8.30
140.0	27.3	0.007	58.26	0.006	47.75	106.01	25.20	80.81	1.24	0.065	549.17	549.17	294.00	255.17	1.65	0.013	109.64	0.001	8.40
145.0	26.7	0.007	58.94	0.006	48.31	107.25	26.10	81.15	1.24	0.064	555.58	555.58	304.50	251.08	1.62	0.013	110.92	0.001	8.50
150.0	26.1	0.007	59.61	0.005	48.86	108.46	27.00	81.46	1.25	0.062	561.85	561.85	315.00	246.85	1.59	0.012	112.18	0.001	8.60
155.0	25.5	0.006	60.26	0.005	49.39	109.65	27.90	81.75	1.25	0.061	567.98	567.98	325.50	242.48	1.56	0.012	113.40	0.001	8.69
160.0	25.0	0.006	60.89	0.005	49.91	110.80	28.80	82.00	1.25	0.060	573.98	573.98	336.00	237.98	1.54	0.012	114.60	0.001	8.78
165.0	24.4	0.006	61.52	0.005	50.42	111.94	29.70	82.24	1.26	0.059	579.86	579.86	346.50	233.36	1.51	0.012	115.77	0.001	8.87

Water Balance Calculation

4933 Victoria Avenue North
Town of Lincoln
File No. UD23-045
Date: April 2024

Contributing Drainage Area	16299	m^2
Rainfall depth to be retained	5.0	mm
Total rainfall volume at 5mm	**81.50**	m^3

Initial Abstraction Calculations

Surface	**Area (m^2)**	**IA (mm)**	**Volume (m^3)**	
Green roof	3050.0	5.0	15.25	m^3
Landscaping	1160.0	5.0	5.80	
Roof/Terraces/Asphalt	12089.1	1.0	12.09	m^3
Total	**16299.1**		**33.14**	m^3

Water Volume provided by initial abstraction is	**33.14**	m^3
Therefore Water Balance Required is	**48.36**	$\mathbf{m^3}$

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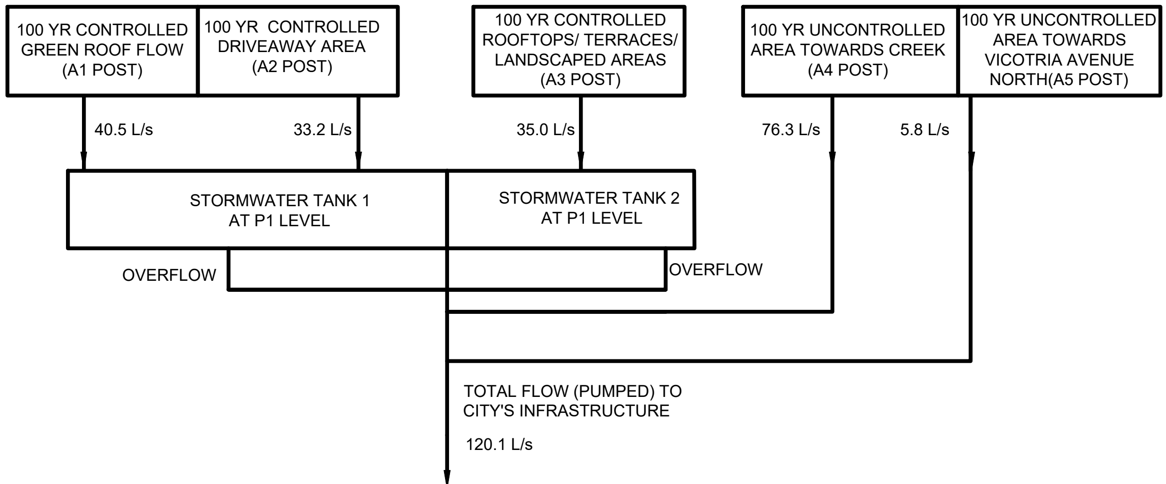
Water Quality Calculations

4933 Victoria Avenue North
File No. UD23-045
Date: April 2024

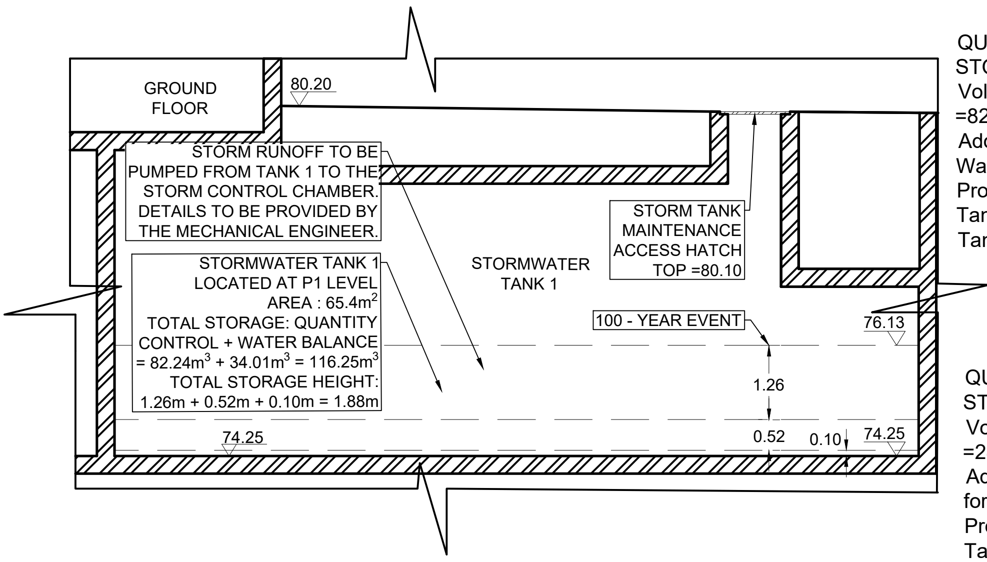
Surface	Method	Effective TSS Removal	Area (ha)	% Area of Controlled Site	Overall TSS Removal
Rooftop/Terraces/ Green Roof	Inherent	80%	1.292	93%	74%
Driveway Area /Landscape/Hardscape Areas driven to OGS - Controlled in Tank 1	SFPD0608 Treatment Device	80%	0.098	7%	6%
Total			1.390	100%	80%

Note: Uncontrolled water does not account in the above calculations



5-yr PRE-DEVELOPMENT CONDITION RELEASE RATE = 234.6 L/s

MAX SITE RELEASE RATE ACHIEVED = 120.10 L/s



QUANTITY CONTROL FOR STORMWATER TANK 1

Volume required for 100-year storm event =82.24 m³

Additional Volume required to be stored for Water Balance = 33.85 m³

Proposed Volume provided in Storage Tank for Water Balance = 34.01 m³

Tank Size:65.40sq.m

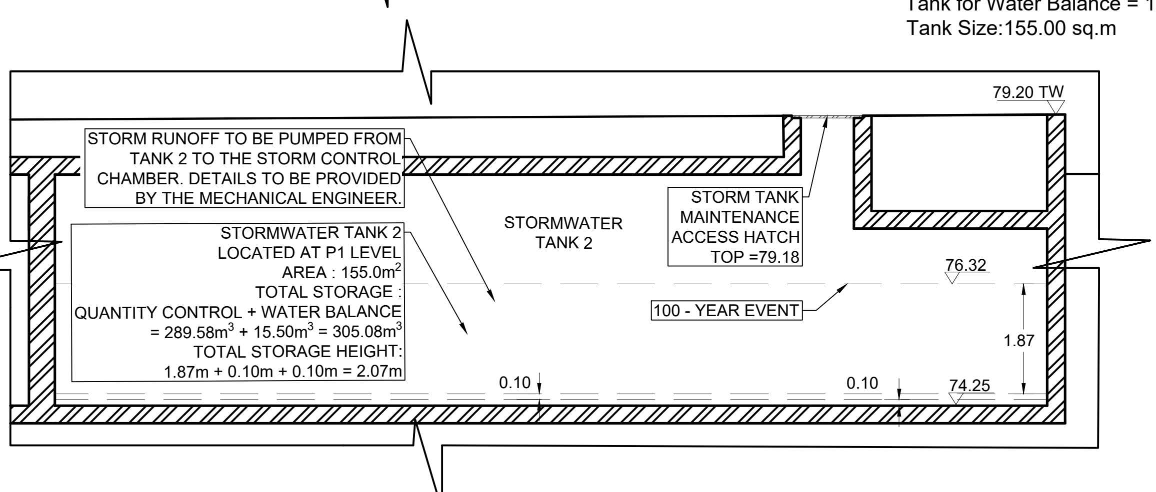
QUANTITY CONTROL FOR STORMWATER TANK 2

Volume required for 100-year storm event =289.58 m³

Additional Volume required to be stored for Water Balance = 14.50 m³

Proposed Volume provided in Storage Tank for Water Balance = 15.50 m³

Tank Size:155.00 sq.m



Lithos	CONCEPTUAL FLOE SCHEMATIC MIXED - USE DEVELOPMENT 4933 VICTORIA AVENUE NORTH LINCOLN, ONTARIO		
	DATE: APR 2024	PROJECT No: PUD23-045	
150 Bermondsey Road, Toronto, Ontario M4A 1Y1	SCALE: N.T.S.	FIGURE No: FIG 3	

Appendix D

Sanitary Data Analysis

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SANITARY SEWER DESIGN SHEET

4933 Victoria Ave. N.

TOWN OF LINCOLN

LOCATION	Site Area	RESIDENTIAL		COMMERCIAL	HOTEL	FLOW									SEWER DESIGN				
		NUMBER OF UNITS	SECTION Population @1.7ppu (persons)	COMM. AREA (ha)	HOTEL AREA (ha)	AVERAGE RESIDENTIAL FLOW @255 L/c/d (L/s)	HARMON PEAKING FACTOR	RES. PEAK FLOW (L/s)	AVERAGE COMMERCIAL FLOW @ 5 L/m^2/day or 310 L/c/day (L/s)	AVERAGE HOTEL FLOW @ 5 L/m^2/day or 310 L/c/day (L/s)	TOTAL ACCUM. AREA (ha.)	INFILT. @0.4 L/s/ha for ex. areas @0.286 L/s/ha for new dev. areas (L/s)	TOTAL SANITARY FLOW (L/s)	TOTAL DESIGN FLOW (L/s)	PIPE LENGTH (m)	PIPE DIA. (mm)	SLOPE (%)	FULL FLOW CAPACITY n = 0.013 (L/sec)	% of DESIGN CAPACITY (%)
column number	1	2	3	4	5	6	7	8	8	9	10	11	12	13	14	15	16	17	18
Existing Condition																			
Industrial Building	1.935	0	0	0.43	0.000	0.00	4.50	0	0.25	0.00	1.935	0.77	0.25	1.02					
Proposed Condition																			
Mixed-use Development	1.630	396	673	0.561	0.897	1.99	3.90	7.76	0.32	0.52	1.630	0.47	8.60	9.07		200	2.0%	46.38	20%

Net Flow (Towards Sanitary Network) **8.04**

Commercial Flow Rate - 310 litres/job/day

Residential Flow Rate - 255 litres/capita/day

Wet Weather Infiltration - 0.4 L/s/ha for existing areas - 0.286 L/s/ha for new development areas

Peaking Factor : (1 + [14 / (4 + P^0.5)], P=Population in thousands)

Site Area (ha): **1.630**

Lithos

Prepared by: Kouri Amaryllis Ioanna, P.E., M.A.Sc
Reviewed by: Catherine Agiou, P.E., M.A.Sc.
Date: April 2024

Project: 4933 Victoria Ave.N.
Project: UD23-045
Town of Lincoln — Sheet 1 OF 1

Appendix E

Water Data Analysis

WATER DEMAND

4933 Victoria Avenue North

Project No: UD23-045

Date: April 2024

Prepared by: Antonina Kokkinidou, P.E., M.A.Sc

Reviewed by: Catherine Agiou, P.E., M.A.Sc.

Fire Flow Calculation

1 $F = 220\ C\ (A)^{1/2}$

Where F= Fire flow in Lpm

C= construction type coefficient

= 0.6 for fire resistive construction

A = total floor area in sq.m. excluding basements, includes garage*

		Area Applied
Level 1	5353 m^2	25%
Level 2	4814 m^2	100%
Level 3	4531 m^2	25%
=	7,285 sq.m.	

Note: The levels indicated, reference the floors with the largest areas, which considers the total floor area of Buildings A, B, C (Please refer to building stats)

F = 11,266.49 L/min

F = 11,000 L/min Round to nearest 1000 l/min

2 Occupancy Reduction

25% reduction for Limited Combustible occupancy

F = 8250 L/min

3 Sprinkler Reduction

50% Reduction for NFPA Sprinkler System

F = 4125 l/min

4 Separation Charge

0%	East	>45m	
0%	North	>45m	
10%	South	20.1 to 30m	
0%	West	>45m	
10%	Total Separation Charge		825 L/min

F = 4,950.00 L/min

82.50 L/s

F = 1308 US GPM

Domestic Flow Calculations

Residential Population =	673	Persons	from Sanitary Calculations (Residential)
Commercial Flow =	0.32	L/s	from Sanitary Calculations (Commercial)
Hotel Flow =	0.52	L/s	from Sanitary Calculations (Hotel)
Average Day Demand (residential) =	255	L/cap/day	
Average Day Demand (commercial) =	310	L/cap/day	1 US Gallon=3.785 L
Average Day Demand (Total) =	2.83	L/s	
=	45	US GPM	1 US GPM=15.852L/s

Max. Daily Demand Peaking Factor = 2.00

Max. Daily Demand = 5.65 L/s = 90 US GPM

or

Max. Hourly Demand Peaking Factor = 3.00

Max. Hourly Demand = 8.48 L/s = 134 US GPM

Max Daily Demand = 5.65 L/s

Fire Flow = 82.50 L/s

Required 'Design' Flow = 88.15 L/s

1397 US GPM

Note: Required 'Design' Flow is the maximum of either:

1) Fire Flow + Maximum Daily Demand
2) Maximum Hourly Demand

Lithos	**WATER DEMAND** **4933 Victoria Avenue North** Project No: UD23-045 Date: April 2024 Prepared by: Antonina Kokkinidou, P.E., M.A.Sc Reviewed by: Catherine Agiou, P.E., M.A.Sc.

Pressure Losses

Hazen-Williams Formula

$V = kCR_h^{0.63} x S^{0.54}$

k= 0.85 - conversion factor (0.849 for SI units and 1.318 for US customary units)

C= 140 - roughness coefficient (PVC : 140-150)

S= h_f/L

Rh= D/4 - hydraulic radius (D/4 for full flow, A/P_W for partially flow)

Fire Fighting and Domestic Head Loss

Flow Requirements=	88.15	l/s
Diameter=	200	mm
Area=	3.14E-02	
L=	3.4	m
V=	2.81	m/s
S=	3.20E-02	
R_h=	0.05	
H_f=	0.11	m
=	0.15	psi

Flow Test *(dated: April 24, 2024)*

when:	Static Pressure =	100 psi	Flow (gpm) = 100	=	0	L/s
	Residual Pressure =	20 psi	Flow(gpm) = 1554.38	=	98.08	L/s

$$Q_R = Q_F \times \frac{h_r^{0.54}}{h_f^{0.54}}$$

Pressure (psi)	Flow (L/s)
100	0.0
20	98.08
28.1	*88.15*

Based on the Pressure/Flow relationship, we have to confirm that the flow requirement of 88.15 L/s can be provided at minimum pressure (20.3 psi + Losses) as set out by the FUS guidelines

Fire Flow is above minimum of 20.45 psi (20.3+Hf)

Since the flow of 88.15 L/s required for the proposed development is provided in the existing watermain at 28.1 psi (which is more than the minimum of 20.45 psi), we anticipate that the existing watermain infrastructure can support the proposed development.

Flow available at 20psi = 1,554.38 gpm = 98.08 L/s

$$Q_{avail} @ 20psi = Q_T ((P_S-P_A)/(P_S-P_R))^{0.54}$$
$$= 1554.38 \times ((100-20) / (100-28.1))^{0.54}$$
$$= 1,554 \text{ gpm}$$

Lithos

Fire Hydrant Flow Test Report

General Information

Report No. : **FHR-24-04-24-01** Date : **24-Apr-24**

Project No. : **PUD23-045**

Site Address/Location: **4933 Victoria Ave. North**

Region/Municipality: **Town of Lincoln**

Residual Fire Hydrant Location/description : **43.192424, -79.395093**

Flow Fire Hydrant Location/description : **43.193670, -79.395347**

Watermain Pipe Size (mm) : **200 mm**

Test Equipment Orifice Size (in) : **2.5**

Test Equipment Orifice coefficient : **0.9**

Date of test: **24-Apr-24**

Time of test: **9:00**

Temperature: **7°C**

Testing Method : **NFPA 291 (Recommended Practice for Fire Flow Testing and Marking of Hydrants)**

Attendants

	Name	Title	Contact Info.
Lithos Inspector	**Alma**	**Project Inspector**	**(647)-631-9602**
Lithos Inspector	**Peter**	**Project inspector**	**(437)-215-1144**
Town of Lincoln Rep.	**John**	**City Inspector**	**(905)-933-4497**

Site Plan/Sketch

Pressure Readings (PSIG)

Flow Hydrant's Outlet Condition	C-0 { Outlet #1 : Close Outlet #2 : Close	C-1 { Outlet #1 : Open Outlet #2 : Close	C-2 { Outlet #1 : Open Outlet #2 : Open
Residual Fire Hydrant	**100**	**54**	**32**
Flow Fire Hydrant	**-**	**38**	**18**

Fire Hydrant Flow Test Report

General Information

Report No. : **FHR-24-04-24-01** Date : **24-Apr-24**
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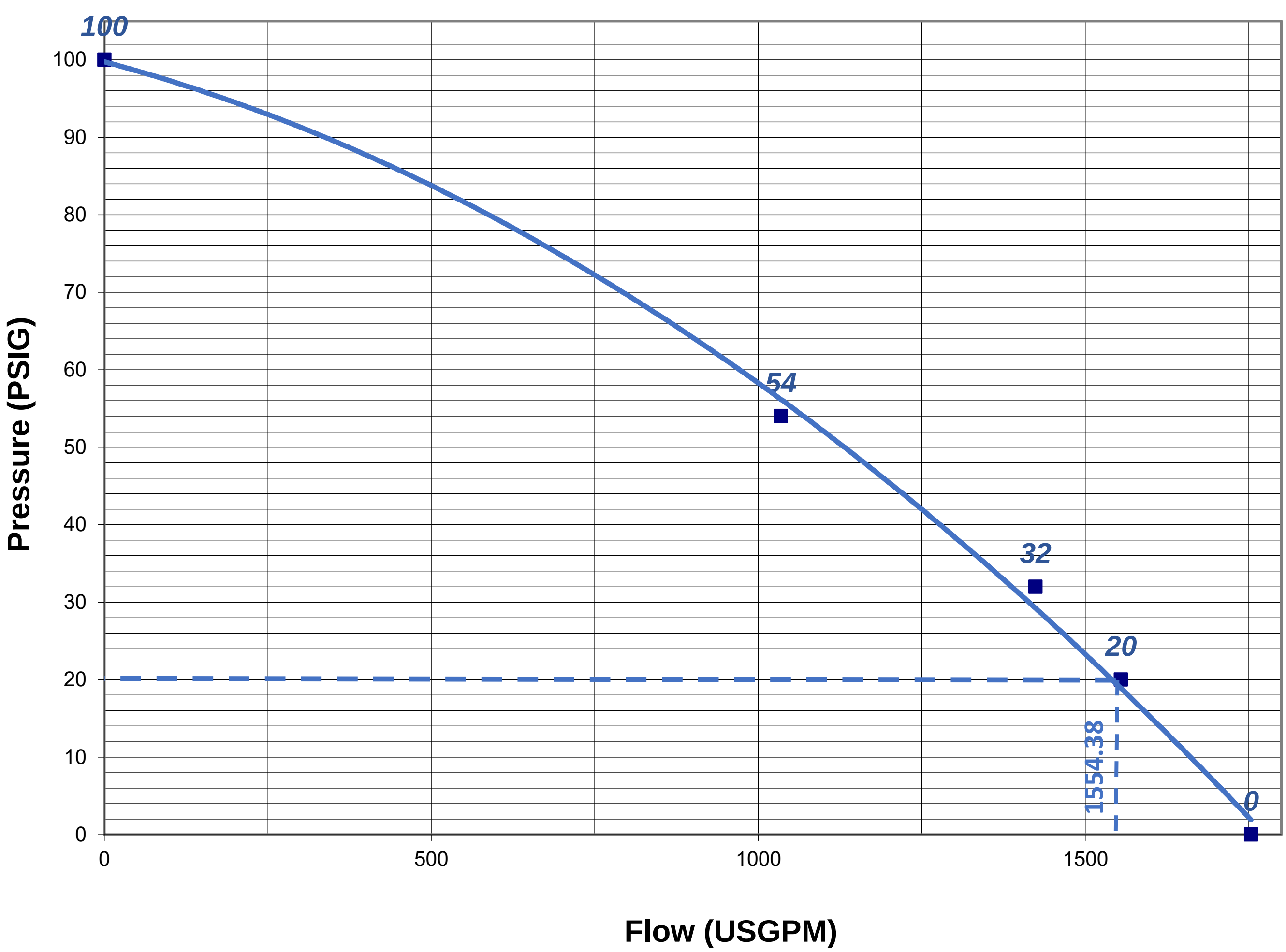
Pressure-Flow Table

Condition		C-0	C-1	C-2	C(20)	C(0)
Pressure (PSIG)		100	54	32	20	0
Flow	(USGPM)	0	1034.35	1423.78	1554.38	1753.43
	(L/S)	0.00	65.27	89.84	98.08	110.64

Pressure-Flow Graph



Result

Maximum available flow at 20PSI = 1554.38 USGPM or 98.08 L/s

Lithos

150 Bermondsey Road, Toronto, Ontario M4A 1Y1

SEPERATION DISTANCES
MIXED USE DEVELOPMENT
4933 VICTORIA AVENUE NORTH,
LINCOLN, ONTARIO

DATE:	APRIL 2024	PROJECT No:	UD23-045
SCALE:	N.T.S.	FIGURE No:	FIG 4