3171 Lakeshore Road West, Town of Oakville

Functional Servicing and Stormwater Management Report

July 2019

Prepared for:

Vogue Wycliffe (Oakville) Limited

Submitted by:

SCS Consulting Group Ltd
30 Centurian Drive, Suite 100
Markham, ON, L3R 8B8
Phone 905 475 1900
Fax 905 475 8335

Project Number: 1930

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

SUBMISSION HISTORY

Submission	Date	In Support Of	Distributed To
1st	July 2019	Official Plan Amendment, Zoning Amendment, Draft Plan Approval	Town of Oakville, Halton Region

1.0 INTRODUCTION

SCS Consulting Group Ltd. has been retained by Vogue Wycliffe (Oakville) Limited to prepare a Functional Servicing and Stormwater Management Report for a proposed re-development of the Cudmore's Garden Centre at 3171 Lakeshore Road West, located in the Town of Oakville.

1.1 Purpose of the Functional Servicing Report

The Functional Servicing and Stormwater Management Report (FSSR) has been prepared in support of the Zoning By-Law Amendment, Official Plan Amendment, and Draft Plan of Subdivision applications for the proposed re-development. The FSSR will be used to support the Site Plan application at a later date pending any alterations to the proposed Draft Plan. The Draft Plan by Weston Consulting Group Inc. dated June, 2019 and the Site Plan by VA3 Design Inc. dated May, 2019 are provided in **Appendix A**. The proposed re-development consists of the following land uses:

- 8 Condominium Semi-Detached Lots;
- 3 Freehold Townhouses;
- 24 Condominium Townhouses;
- A Municipal Right-of-Way; and
- A private condominium Road.

The purpose of this report is to demonstrate that the re-development can be graded and serviced in accordance with the Town of Oakville, Halton Region, and the Ministry of Environment, Conservation and Parks (MECP) design criteria.

1.2 Study Area

For the purposes of this FSSR, the direction of Lake Ontario will be referred to as south with all other directions based on this assumption. The orientation of the site is such that Lake Ontario is actually east of the site based on true north. Both the true north and FSSR north arrows have been provided on all figures for reference.

The site area owned by Vogue Wycliffe (Oakville) Limited is approximately 0.99 ha in size and is bound by Lakeshore Road West to the south, unopened municipal right-of-way to the east, and existing residential to the north and west (refer to **Figure 1.1**). The site is an infill development. The existing residential development to the north and west of the site was designed and constructed in the 1980's and the existing residential development east of the site was designed and constructed in the early 1970's. The site is located predominantly within the Bronte Creek watershed in the Town of Oakville.

The site is currently operating as a garden centre (Cudmore's Garden Centre Inc.). The site is zoned as Residential Low (RL3-0).

1.3 Background Information

In preparation of the site servicing and SWM strategies, the following design guidelines and standards were used:

- Town of Oakville, Development Engineering Procedures and Guidelines Manual;
- Development Engineering Procedures and Guidelines Manual Addendum #1, January 2017;
- Halton Region, Water and Wastewater Linear Design Manual, Version 4.0, April 2019; and
- Ministry of Environment (MECP) Stormwater Management Planning and Design Manual (March 2003).

The following approved Engineering Drawings were used in preparation of the site servicing and SWM strategies:

- Realmar Subdivision Sanitary Drainage Area, Dwg 0-5252, Duncan Hopper and Associates Limited, 1983
- Realmar Subdivision Storm Drainage Area, Municipal Drawing No Dwg 0-5253, Duncan Hopper and Associates Limited, 1983
- Realmar Subdivision Victoria Street 0+00 North Limit to 1+17.330 South Limit, Sheet 6, Duncan Hopper and Associated Limited, 1983
- Realmar Subdivision Victoria Street North of West Street, Sheet 11, Duncan Hopper and Associates Limited, 1983
- Riverview Street Sta. 0+000 – Sta. 0+100 and Speyside Court Sta. 0+000 – Sta. 0+060, Dwg 0-4089, McConnell Maughan Limited, 1980
- Proposed 18" Diameter Storm Sewer on Victoria Street from Mississaga Street to West Street, Plan No ST-29-74-4. 1976
- Proposed Reconstruction of Mississaga Street Sta. 0.000.00 to Sta. 0+220.00, Municipal Drawing No R-242-88-1, 1989
- Lakeshore Road West from 155m East of Great Lakes Boulevard to Mississaga Street, Sheet 18, McCormick Rankin Corporation, 2010
- Lakeshore Road West from 155m East of Great Lakes Boulevard to Mississaga Street, Sheet 19, McCormick Rankin Corporation, 2010
- Lakeshore Road West from 155m East of Great Lakes Boulevard to Mississaga Street, Sheet 20, McCormick Rankin Corporation, 2010
- Proposed 15" Diameter Sanitary Sewer & Proposed 48" Diameter Storm Sewer on Lakeshore Road West from Sta. 421+00 to Sta. 423+49, Plan No ST-6-69-5, 1969
- Proposed 15" Diameter Sanitary Sewer & Proposed 60" & 48" Diameter Storm Sewer on Lakeshore Road West from Sta. 417+00 to Sta. 421+00, Plan No ST-6-69-4, 1969
- Proposed 15" Diameter Sanitary Sewer on Lakeshore Road West & West River Street from Sta. 4+00 to Sta. 6+10, Plan No ST-6-69-2, 1969
- Proposed 15" Diameter Sanitary Sewer on West River Street from STA. 1+10 to STA. 4+00, Plan No ST.-6-69-B, 1969

The following supporting studies were used in preparation of the site servicing and SWM strategies:

- Phase One Environmental Site Assessment, Proposed Residential Development Lakeshore Road West and West Street, Town of Oakville, Soil Engineers Ltd., December 20, 2016
- Geotechnical Investigation, 3171 Lakeshore Road West, Soil Engineers Ltd., May 2017

Excerpts from the above listed documents are included in **Appendix B**. These documents were utilized to establish the design criteria and servicing approach for the site.

2.0 STORMWATER MANAGEMENT

2.1 Stormwater Runoff Control Criteria

The following stormwater runoff control criteria have been established based on the design guidelines and standards listed in **Section 1.3**. The stormwater runoff criteria are summarized below in **Table 2.1**:

Table 2.1 – Stormwater Runoff Control Criteria

Criteria	Control Measure
Quantity Control	Control proposed peak flows to existing peak flows for the 2 through 100 year storm events. Where runoff is conveyed to an existing storm sewer, limit the maximum peak flow to the existing 5 year storm event peak flow.
Quality Control	MECP Enhanced Level Protection (80% TSS Removal).
Erosion Control	Detention of the 25 mm rainfall runoff for a minimum of 24 hours.

2.2 Existing Drainage

As shown in **Figure 2.1**, runoff from Catchment 101 is conveyed west to Victoria St. (hereby referred to as Victoria St. (west)), runoff from Catchment 102 is conveyed south to Lakeshore Road West, and runoff From Catchment 103 is conveyed east to Victoria St. (hereby referred to as Victoria St. (east)). Runoff from Catchment 101 is captured by an existing storm sewer on Victoria St. (west) or conveyed overland to Sheldon Creek. Runoff from Catchments 102 and 103 are captured by existing storm sewers on Lakeshore Road West and Victoria Street (east) or conveyed overland, ultimately to Bronte Creek.

There are no stormwater management controls on the existing site. Per Realmar Subdivison Sheet 6 (provided in **Appendix B**), it was anticipated that the storm servicing for the site would be provided by connecting to MH10 at the intersection of Victoria St. (West) and West St. However, the Realmar Subdivision storm drainage plan (Sheet 4, provided in **Appendix B**) does not account for the site area. Therefore, the proposed peak flow to the storm sewer will be conservatively limited to the existing 5 year storm event peak flow.

2.2.1 Hydrologic Modelling

Hydrologic modelling was undertaken using the Visual Otthymo Version 2.2.4 software (VO2) with the 24-hour Chicago storm distribution. The study area is located within the Town of Oakville, therefore, the IDF rainfall information for the Toronto Bloor Street station was used, per Table 3.1 of the Town guidelines, to determine the existing peak flows to outlet locations.

The existing peak flows from the study area to Victoria St. (east and west), and Lakeshore Road West are summarized in **Table 2.2**.

Table 2.2: Existing Peak Flow Rates

Return Period Storm	Peak Flow Rate (m^3/s)		
	Victoria St. (West)	Lakeshore Road West	Victoria St. (East)
2 Year	0.004	0.048	0.035
5 Year	0.006	0.083	0.062
10 Year	0.008	0.108	0.082
25 Year	0.010	0.153	0.113
50 Year	0.012	0.180	0.133
100 Year	0.013	0.207	0.155

A summary of modelling parameters and VO2 schematics are provided in **Appendix C**. A CD containing the VO2 hydrology model is also provided in **Appendix C**.

2.3 Best Management Practices

In accordance with the Ministry of Environment Stormwater Management Planning and Design Manual (2003), a review of stormwater management best practices was completed using a treatment train approach, which evaluated lot level, conveyance system and end-of-pipe alternatives.

The following site characteristics were taken into consideration:

- The topography is relatively flat with moderate grades;
- Based on the Geotechnical Investigation by Soil Engineers Ltd., dated May, 2017, the site soils consist of silt and silty clay overlying shale bedrock;
- A grain size analysis was completed and indicates that the native soils have a percolation rate ranging from 7.5 - 10 mm/hr;
- Within the installed site wells, groundwater was observed at depths ranging between 1.7 – 2.3 m below existing grade; and
- The study area is approximately 1.17 ha and consists of semi-detached and townhouse lots, and the northern Lakeshore Road West Boulevard.

2.3.1 Lot Level Controls

Lot-level controls are at-source measures that reduce runoff prior to stormwater entering the conveyance system. These controls are proposed on private properties. The following lot level controls have been reviewed for use on this site. The associated recommendations are described below:

Increased Topsoil Depth – An increase in the restored topsoil depth on lots can be used to promote lot level infiltration and evapotranspiration. Increased topsoil depth will contribute to lot level quality and water balance control.

A minimum depth of 0.3 m is proposed in all landscaped areas.

Passive Landscaping/Bio-Retention – Planting of gardens and other vegetation designed to minimize local runoff or use rainwater as a watering source can be used to reduce rainwater runoff by increasing evaporation, transpiration, and infiltration. By promoting infiltration through passive landscaping, water quality and quantity control is provided for the volume of water retained. Passive landscaping can provide significant SWM benefits as part of the overall treatment train approach for the re-development.

There is insufficient space in the municipal right-of-way or the private condominimum road to accommodate bio-retention, therefore it is not proposed.

Roof Runoff to Soak-away Pits – Directing roof runoff to subsurface soak-away pits can be used to promote infiltration. By promoting infiltration water quality and quantity control is provided for the volume of water retained. Infiltration of roof runoff can provide significant SWM benefits as part of the overall treatment train approach for the re-development. Infiltration techniques require loam or better soils (min. inf. Rate >= 15 mm/hr) and are recommended for drainage areas less than 0.5 ha.

The site soils are not suitable based on the 7.5 to 10 mm/hr infiltration rate and since a 1 m separation between the bottom of the infiltration trench and the groundwater elevation may not always be available, therefore soak-away pits are not proposed.

Roof Runoff to Retention Cisterns – Directing roof runoff to rainwater retention cisterns (i.e. rain barrels or greywater re-use) can contribute to water quality and water balance control. The retained rainwater can be harvested for re-use such as irrigation and/or greywater use. Rain barrels require the collected rainwater to be used between rain events making their effectiveness dependent on the user. Rain barrels have not been recognized as a retention device since their effectiveness is based on homeowner behaviour. However, rain barrels can be provided as a "best efforts" approach.

Runoff retention is not required, therefore rain barrels are not proposed.

Green Roofs – Best suited for flat roofs on high-rise buildings, green roofs provide rainwater retention in the growing medium where it is evaporated, evapo-transpirated, or slowly drains away after the rainfall event.

The proposed homes will have peaked roofs, therefore green roofs have not been proposed.

Rooftop and/or Parking Lot Detention Storage – Often employed with large rooftop or parking lot footprints, flow attenuation for quantity or extended detention control can be provided via a flow restriction with stormwater storage provided via ponding either on rooftops or parking lots.

The proposed homes will have peaked roofs and no parking lots are proposed, therefore rooftop and parking lot detention storage has not been proposed.

Roof overflow to Grassed Areas – Directing roof leaders to grassed areas can contribute to water quality and water balance control by encouraging stormwater retention.

Roof leaders will be directed to grassed areas where practical.

Pervious Pavement – By encouraging infiltration and filtration, pervious pavement can contribute to water quality, water balance and erosion control.

Pervious pavement is not proposed as part of the re-development.

Vegetated Filter Strip – At source filtration and infiltration can be achieved through the use of vegetated filter strips by directing sheet flow from impermeable areas to the strip prior to being collected via the storm system. Vegetated filter strips are best suited to parking lot areas with landscaped borders or islands.

There are no suitable areas for vegetative filter strips, therefore they are not proposed.

A summary of the suitability of potential lot level controls for the re-development is provided in **Table 2.3**.

2.3.2 Conveyance Controls

Conveyance controls provide treatment of stormwater during the transport of runoff from individual lots to the receiving watercourse or end-of-pipe facility and present opportunities to distribute stormwater management techniques throughout a development. The following conveyance controls have been reviewed for use on this site. The associated recommendations are described below:

Grassed Swales – A grassed swale can promote infiltration, filtration, and evapo-transpiration, contributing to water quality and quantity control. Grassed swales need an unimpeded and relatively wide stretch of landscaped area, such as within a wide boulevard with no driveways, to function properly.

There are no areas on the re-development for grassed swales based on the number of required driveways and the municipal road standard, therefore they are not proposed. It is noted that smaller grassed swales will be used at the individual lot grading level.

Exfiltration at Rear Lot Catchbasins – Where rear lot catchbasins are required due to grading constraints, a perforated pipe system could be incorporated into the rear lot catchbasin design to promote infiltration of 'clean' stormwater runoff. By promoting infiltration, water quality and quantity control can be provided for the volume of water retained. Infiltration techniques require loam or better soils (min. inf. Rate >= 15 mm/hr) for optimal operation and a separation of at least 1.0 m between the bottom of the infiltration trench and the seasonally high groundwater elevation.

Based on the low infiltration rates of the existing soil (7.5 to 10 mm/hr) and the measured groundwater level which would result in insufficient separation from the proposed surface elevations to support exfiltration at rear lot catchbasins, exfiltration at lot catchbasins is not proposed.

Infiltration Trench in Boulevard Behind Street Catchbasins – Street catchbasins can be uses as a pre-treatment system with the initial stormwater runoff directed to a pervious pipe and stone infiltration under the boulevard behind the catchbasins. Storm runoff in excess of the capacity of the infiltration trench can be directed to the storm sewer system. Sufficient width is required within the boulevard to accommodate the infiltration trench to avoid conflict with the watermain or utility trenches.

The private condominium Road and municipal right-of-way do not have sufficient space for an additional infiltration trench and additional infiltration is not required, therefore this system is not recommended.

A summary of the suitability of potential conveyance controls for the re-developments is provided in **Table 2.3**.

2.3.3 End-of-Pipe Controls

Stormwater management facilities at the end of pipe receive stormwater flows from a conveyance system and provide treatment of stormwater prior to discharging flows to the receiving watercourse. While lot level and conveyance system controls are valuable components of the overall SWM plan, on their own, they are typically not sufficient to meet the quantity and quality control objectives. The following end-of-pipe controls have been reviewed for use on this site. The associated recommendations are described below:

Stormwater Detention Facility – To meet quantity and erosion control targets, flow restrictors can be used to control stormwater release rates. To accommodate the reduced release rate, stormwater detention facilities are required to store stormwater runoff. Stormwater storage can be provided by large concrete storm sewers (superpipes) and controlled with flow restrictors prior to discharging to the receiving infrastructure.

A superpipe will be considered beneath the private condominium Road.

Wet Ponds, Wetlands, Dry Ponds – Sized in accordance with the MECP criteria, these end of pipe facilities can provide water quality, quantity, and erosion control treatment.

Based on the small size of the site, an end of pipe wet pond has not been proposed.

Oil-Grit Separator – A properly sized oil-grit separator (OGS) can provide MECP Enhanced (Level 1) treatment and contribute to the treatment train approach for water quality control. An OGS unit is required to have New Jersey Department of Environmental Protection (NJDEP) certification.

An OGS will be considered at the storm sewer outlet to the existing Victoria Street (east) storm sewer.

2.3.4 Selection of Best Management Practices

Table 2.3 summarizes the suitability of the various stormwater management controls identified for the re-developments.

Table 2.3 - Recommended Stormwater Best Management Practices

STORMWATER MANAGEMENT PRACTICE	**Feasible (Yes/No)**	**RECOMMENDED (Yes/No)**
Increased Topsoil Depth	Yes	Yes
Passive Landscaping/Bio-Retention	No	No
Roof Leader to Soak-away Pits	No	No
Roof Runoff to Retention Cisterns	Yes	No
Green Roofs	No	No
Rooftop and/or Parking Lot Detention Storage	No	No
Roof overflow to Grassed Areas	Yes	Yes
Pervious Pavement	Yes	No
Vegetated Filter Strips	No	No
Grassed Swales	No	No
Exfiltration at Rear Lot Catchbasins	No	No
Infiltration Trench in Boulevard Behind Street Catchbasins	No	No
Stormwater Detention Facility	Yes	Yes
Wet Ponds, Wetlands, Dry Ponds	No	No
Oil-Grit Separator	Yes	Yes

2.4 Proposed Storm Drainage

As shown in **Figure 2.2**, runoff from Catchment 201 will be conveyed to Victoria St. (west). Due to grading restraints, runoff from this area cannot be conveyed to the proposed superpipe described below. Runoff from Catchment 202 will be conveyed overland to Lakeshore Road West, which generally matches the existing drainage condition. Runoff from Catchment 203 will be captured in the proposed storm sewer system and outlet to the existing Victoria St. (east) storm sewer. A superpipe attenuation facility under the private condominium Road will provide quantity control for Catchment 203 before the flow is conveyed through an oil-grit separator (OGS) and released to the existing storm sewer on Victoria Street (east). Runoff from Catchment 204 will drain uncontrolled to Victoria St. (east).

Runoff from the 100 year storm event will be captured in two locations as shown on **Figure 2.2**. Runoff from the Victoria St. cul-de-sac and some private condominium development drainage will be captured in low points in the cul-de-sac gutter. Runoff from the private condominium development will be captured in a low point at the eastern edge of the study area.

2.5 Proposed Stormwater Management Plan

2.5.1 Quality Control

Runoff from Catchment 201 will not have quality control due to the small size of the catchment (0.01 ha).

Runoff from Catchment 202 and 204 will be from roofs and yards which is generally considered to be "clean", therefore no quality control is proposed for these catchments.

Quality control for runoff from Catchment 203 will be provided by a treatment train of Best Management (BMP) techniques which will include additional topsoil depth on all grassed areas and directing roofs to grass which will work together with an end-of-pipe oil-grit separator (OGS). The OGS will be sized to achieve Enhanced level quality control (80% TSS removal) as requested by the Town of Oakville (correspondence provided in **Appendix B**). The quality control provided by the grassed areas and roof leaders to grass has not been quantified.

2.5.2 Erosion Control

The 1.17 ha study area is too small to practically detain the runoff volume from the 25 mm storm event over 24 hours, and therefore, erosion control will not be able to be provided. It is typical that for relatively small sites of less than 2 ha, erosion control in the form of stormwater detention is not required.

2.5.3 Quantity Control

The proposed peak flow rates from Catchment 201 and Catchment 202 are less than the existing peak flow rates to Victoria St. (west) and Lakeshore Road West as shown in **Table 2.5**, therefore no quantity control is required.

Quantity control for runoff to Victoria St. (east) will be provided by approximately 164 m of 1500 mm wide by 1200 mm high concrete box culvert superpipe beneath the private condominium Road. The superpipe will release runoff from Catchment 203 to the existing Victoria St. (east) storm sewer, therefore the maximum release rate during the 100 year storm event from Catchment 203 will be limited to 0.053 m^3/s which is equal to the existing 5 year peak runoff rate entering the storm sewer (0.062 m^3/s) from Catchment 103, minus the 5 year peak runoff rate from Catchment 204 (0.009 m^3/s). Runoff will be detained by a 140 mm diameter orifice plate on the downstream side of a control manhole. The location of the control manhole is shown on **Figures 2.2** and **2.3**. Orifice plate and superpipe parameters are provided in **Appendix D**.

The 24-hour Chicago Storm distribution was modelled for the proposed conditions hydrology model per Town of Oakville standards. A summary of modelling parameters and VO2 schematics are provided in **Appendix C**. A CD containing the VO2 hydrology model is also provided in **Appendix C**. A summary of the resulting stage-storage-discharge characteristics for the superpipe are provided in **Table 2.4**.

Table 2.4: Superpipe Stage-Storage-Discharge Characteristics

Return Period Storm	Stage (m)	Storage (m^3)	Discharge (m^3/s)
2 Year	82.15	85	0.030
5 Year	82.34	134	0.035
10 Year	82.48	167	0.038
25 Year	82.66	212	0.043
50 Year	82.81	247	0.046
100 Year	83.02	279	0.051

As shown in **Table 2.4**, the proposed 100 year peak flow to the Victoria St. (east) storm sewer from the superpipe is 0.051 m^3/s, which is less than the allowable release rate of 0.053 m^3/s.

2.6 Comparison of Existing Peak Flows and Proposed Peak Flows

The study area was designed to control proposed peak flow rates to existing peak flow rates. A summary of the existing (target) and proposed conditions peak flow rates to Victoria St. (west), Victoria St. (east), and Lakeshore Road West are provided in **Table 2.5**.

Table 2.5: Comparison of Existing and Proposed Peak Flow Rates

Return Period Storm	Peak Flow Rate (m^3/s)					
	Victoria St. (West)		Lakeshore Road West		Victoria St. (East)	
	Existing	Proposed	Existing	Proposed	Existing	Proposed
2 Year	0.004	0.001	0.048	0.019	0.035	0.034
5 Year	0.006	0.002	0.083	0.035	0.062	0.043
10 Year	0.008	0.002	0.108	0.045	0.082	0.048
25 Year	0.010	0.003	0.153	0.061	0.113	0.055
50 Year	0.012	0.004	0.180	0.072	0.133	0.061
100 Year	0.013	0.004	0.207	0.087	0.155	0.066

As shown in **Table 2.5**, the proposed peak flows for all storm events are less than the existing condition for all outlets.

2.7 Storm Servicing

The storm sewer system (minor system) will be designed for the 5 year return storm as per the Town of Oakville standards where a superpipe is not proposed. The superpipe will be located only on condo lands, however it will service the freehold lots which front onto the Victoria St. (west) cul-de-sac (units 9-11).

Based on the required stormwater storage elevation in the superpipe, it is anticipated that all proposed lots will require sump pumps for foundation drainage. Sump pumps will outlet to the proposed storm sewer.

The storm sewer system will typically be designed with a slope of 0.5%. The storm sewer will be constructed at a minimum depth of 1.5 m where sump pumps are required. The storm sewer depth is limited by the invert elevation of the existing downstream sewer on Victoria St. (east). The preliminary layout for the proposed storm sewer is illustrated on **Figure 2.3**.

The storm drainage system will be designed in accordance with the Town of Oakville and MECP guidelines, including the following:

- Pipes to be sized to accommodate runoff from a 5 year storm event;
- Minimum Pipe Size: 300 mm diameter storm sewer
- Maximum Flow Velocity: 4.0 m/s;
- Minimum Flow Velocity: 0.75 m/s; and
- Minimum Pipe Depth: 1.2 m, 1.5 m where sump pumps are required.

The rainfall intensity will be using the values from Table 3.1 of the Town of Oakville Development Engineering Procedures and Guidelines Manual dated January 2011 as shown in **Table 2.6**.

Table 2.6: Rainfall Intensity Parameters

Return Period Storm	A	B	C
2 Year	725	4.8	0.808
5 Year	1170	5.8	0.843
10 Year	1400	5.8	0.848
25 Year	1680	5.6	0.851
50 Year	1960	5.8	0.861
100 Year	2150	5.7	0.861

2.8 Overland Flow

The major system flow drainage (up to the 100 year storm event) will generally be conveyed overland along the municipal right-of-way and Condo Road. In accordance with the Town of Oakville standards, the product of depth and velocity for all overland flow shall be < $0.65\ m^2/s$. The major system flow to Victoria St. (west), Lakeshore Road West, and Victoria St. (east) will be less than existing per **Table 2.5**, therefore the major system capacity downstream of the study area will improve.

3.0 SANITARY SERVICING

3.1 Existing Sanitary Sewer System

The sanitary sewer system in the Town of Oakville is owned and operated by Halton Region. The existing sanitary sewer system is illustrated on **Figure 2.3.** Existing sanitary sewers adjacent to the study area include:

- 200 mm diameter sewer on Victoria St. (west) flowing west;
- 250 mm diameter sewer on Victoria St. (east) flowing east; and
- 300 mm diameter sewer on Lakeshore Road West flowing east.

The Victoria St. (west) sanitary sewer conveys flows west to the Sheldon Creek Wastewater pumping station which pumps flows east into the Lakeshore Road West sanitary sewer located south of the proposed development.

3.2 Proposed Sanitary Sewer System

A 200 mm diameter sanitary sewer is proposed to service the freehold lots fronting onto the Victoria St. cul-de-sac which will connect to the existing 200 mm diameter sewer on Victoria St. (west). The lots within the condo development will be serviced by a 200 mm diameter sanitary sewer which will connect to the existing 200 mm diameter sewer on Victoria St. (east). The preliminary layout for the proposed sanitary sewer within the study area is provided on **Figure 2.3**.

An analysis of the existing downstream sanitary sewer system capacity was undertaken to confirm that there is sufficient capacity for the study area. The proposed design flow to the Victoria St. (west) sanitary sewer is approximately 0.3 L/s which is approximately 0.8% of the anticipated 2031 flow from the Sheldon Creek Wastewater pumping station. The proposed design flow to the Victoria St. (east) sanitary sewer is approximately 1.4 L/s. The downstream sanitary sewer was found to have sufficient capacity to convey the proposed flows. Calculations are provided in **Appendix E**.

The sanitary sewers within the site will have slopes ranging between 0.5% and 2% (typically) and will be provided at 3 m to 4 m deep. The sanitary sewer system will be designed in accordance with the Halton Region and MECP criteria, including but not limited to:

- Residential Sanitary Generation Rate: 275 L/c/d;
- Population Density: 135 persons/ha (Townhouse);
- Peaking Factor: Harmon (Min. 2.0)
- Infiltration Rate: 0.286 L/s/ha;
- Minimum Sewer Pipe Size: 200 mm dia (residential), 300mm dia (commercial/industrial);
- Minimum Pipe Cover: 2.75 m;
- Minimum Actual Velocity: 0.60 m/s; and
- Maximum full flow Velocity: 3.0 m/s.

Servicing allocation will be provided by Town Council through the planning approval process.

4.0 WATER SUPPLY AND DISTRIBUTION

4.1 Existing Water Distribution

The watermain system in the Town of Oakville is owned and operated by Halton Region. The existing watermain system is illustrated on **Figure 2.3.** Existing watermains adjacent to the study area include:

- 150 mm diameter pipe on Victoria St. (east);
- 150 mm diameter pipe on Victoria St. (west); and
- 300 mm diameter pipe on Lakeshore Road West.

4.2 Proposed Water System

The freehold lots fronting onto the Victoria St. cul-de-sac (units 9-11) will be serviced by a 50 mm diameter copper loop. The lots within the private condominium development will be serviced by a 200 mm diameter watermain connected to the existing 300 mm diameter watermain on Lakeshore Road West.

A watermain hydraulic analysis has been prepared by Municipal Engineering Solutions and included in **Appendix F**. Service pressures are expected to range between 501 kPa and 533 kPa. The available fire flow meets or exceeds the required fire flow at the minimum pressure of 140 kPa. The preliminary layout for the proposed watermain system is provided on **Figure 2.3**.

The watermain system will designed in accordance with the Halton Region and MECP criteria including:

- Residential water usage rate: 275 l/c/d;
- Max Day Demand Peaking Factor: 2.25;
- Max Hourly Demand Peaking Factor: 4.0 (Residential), 2.25 (Commercial/Industrial);
- Population Density: 55 persons/ha (single family), 135 person/ha (6 story apt. or less), 285 person/ha (greater than 6 story apt.), 90 person/ha (Commercial), 125 persons/ha (Light Industrial);
- Minimum Pipe Size: 150 mm diameter (Residential), 300 mm (Commercial/Industrial);
- Minimum Pipe Depth: 1.8 m; and
- Maximum Hydrant Spacing: 90 m.

5.0 SITE GRADING

5.1 Existing Grading Conditions

The majority of the study area topography has slopes in the range of 0.5% to 2%. The steepest slopes occur in the natural areas of the study area located at the east and north edges of the study area with a maximum slope of approximately 17%. The ground surface elevations through the study area range from approximately 86.75 m along the northern edge to approximately 84.50 m along the eastern edge.

5.2 Proposed Grading Concept

In general, the site will be graded in a manner which will satisfy the following general goals:

- Satisfy the Town of Oakville lot and road grading criteria including:
  - Minimum Road Grade: 0.5%
  - Maximum Road Grade: 7.0%
  - Minimum Lot Grade: 2%
  - Maximum Lot Grade: 5%
  - Maximum slope: 3:1
- Provide continuous road grades for overland flow conveyance;
- Minimize the need for retaining walls;
- Minimize the volume of earth to be moved and minimize cut/fill differential;
- Minimize the need for rear lot catchbasins; and
- Achieve the stormwater management objectives required for the site.

A preliminary grading plan is provided on **Figure 5.1**. At the detailed design stage, the preliminary grading will be subject to a more in-depth analysis in an attempt to balance the cut and fill volumes, minimize slopes, and preserve existing trees to the extent possible.

6.0 RIGHT-OF-WAYS AND SIDEWALKS

The proposed Victoria St. (west) right-of-way will continue the 17.0 m standard street section (Town of Oakville STD 7-22A). This right-of-way section includes sidewalk on both sides of the right-of-way. The 17.0 m standard right-of-way cross-section is provided in **Appendix B**.

The proposed private condominium road typical cross-section is shown on **Figure 6.1**. Sidewalk is proposed along the inside edge of the proposed private road. A 2.5 m servicing and utility easement is required to service the condominium development.

7.0 EROSION AND SEDIMENT CONTROL DURING CONSTRUCTION

During the detailed design stage, erosion and sediment control measures will be designed with a focus on erosion control practices (such as stabilization, track walking, staged earthworks, etc.) as well as sediment controls (such as fencing, mud mats, catchbasin sediment control devices, rock check dams and temporary sediment control ponds). These measures will be designed and constructed as per the "Erosion and Sediment Control Guideline for Urban Construction" document (December 2006). A detailed erosion and sediment control plan will be prepared for review and approval by the Municipality and Conservation Authority prior to any site grading being undertaken. This plan will address phasing, inspection and monitoring aspects of erosion and sediment control. All reasonable measures will be taken to ensure sediment loading to the adjacent properties are minimized both during and following construction.

8.0 SUMMARY

This Functional Servicing and Stormwater Management Report has been prepared in support of the Official Plan Amendment, Zoning By-Law Amendment, and Draft Plan of Subdivision applications for the proposed re-development of the Cudmore's Garden Centre at 3171 Lakeshore Road West in the Town of Oakville. This report outlines the means by which the site can be graded and serviced in accordance with the Town of Oakville, Halton Region, and the Ministry of Environment and Climate Change (MECP) design criteria.

<u>General Information</u>

- The existing land use is commercial lands operating as a garden centre;
- The study area is located predominantly within the Bronte Creek watershed; and
- The proposed development consists of, 8 Condo Semi-Detached Lots, 3 Freehold Townhouses, and 24 Condo Townhouses.

<u>Stormwater Management and Storm Servicing</u>

- Quality Control: MECP Enhanced (Level 1) water quality protection will be provided by a treatment train of best management practices including additional topsoil depth and roof leaders directed to grass, working together with an end-of-pipe oil-grit separator;
- Erosion Control: The study area is too small to practically detain the runoff volume from the 25 mm storm event over a minimum of 24 hours;
- Quantity Control: Quantity control will be provided via a 1500 mm wide by 1200 mm tall concrete box superpipe beneath the condo road;
- Storm Servicing:
  - Storm runoff will be conveyed by storm sewers designed in accordance with Municipality and MECP criteria;
  - Storm sewers will generally be designed for the 5 year storm event where superpipe is not proposed; and
  - Adequate 100 year overland flow routes will be provided.

<u>Sanitary Servicing</u>

- A 200 mm diameter sanitary sewer is proposed to service the freehold lots fronting onto the proposed Victoria St. cul-de-sac which will connect to the existing 200 mm diameter sewer on Victoria St. (west);
- A 200 mm diameter sanitary sewer is proposed to service the condo lots which will connect to the existing 200 mm diameter sewer on Victoria St. (east);
- An analysis of the existing sanitary sewer system capacity was undertaken to confirm that there is sufficient capacity for the study area.

<u>Water Servicing</u>

- There are existing municipal watermains on Victoria St. (east and west) and Lakeshore Road West;
- The freehold lots fronting onto the Victoria St. cul-de-sac will be serviced by a 50 mm diameter copper loop;
- The condo lots will be serviced by connecting a proposed 200 mm diameter watermain to the existing 300 mm diameter watermain on Lakeshore Road West;

- Municipal Engineering Solutions has completed a watermain hydraulic analysis to show that there is sufficient domestic and fire flows to service the development.

Site Grading

- The site grading has been developed to match to the existing surrounding grades, and provide conveyance of stormwater runoff; and
- The lot grading will be subject to further grading design at the architectural design stage prior to the building permit applications.

Right-of-Ways and Sidewalks

- The proposed Victoria St. right-of-way will continue the 17.0 m standard street section (Town of Oakville STD 7-22A); and
- A condo road right-of-way section has been prepared.

Erosion and Sediment Control during Construction

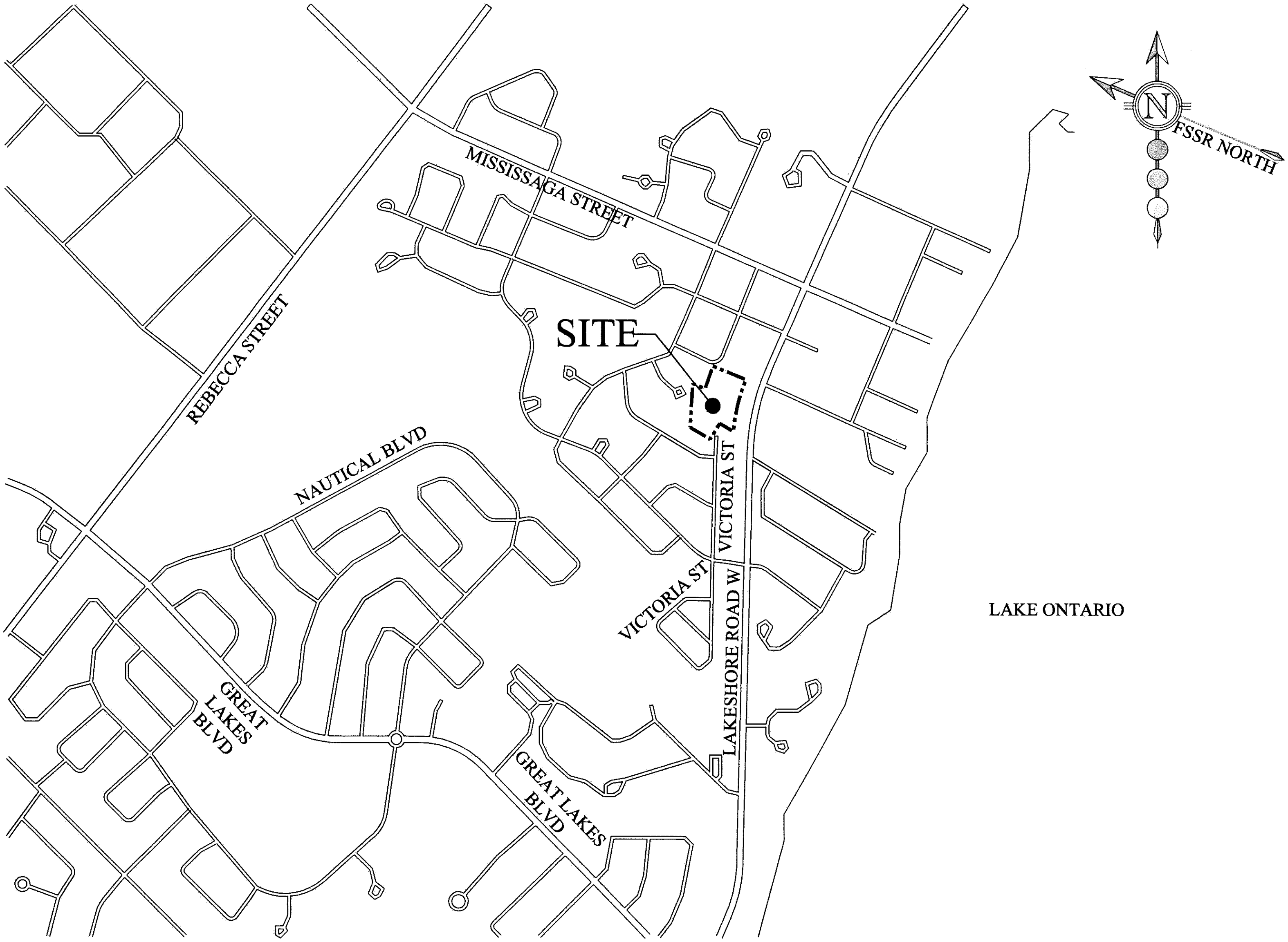
- An erosion and sediment control plan will be prepared at the detailed engineering stage, in accordance with the "Erosion and Sediment Control Guideline for Urban Construction" document (December 2006).

Respectfully Submitted:

SCS Consulting Group Ltd.

Nicholas McIntosh, M.A.Sc., P. Eng.
nmcintosh@scsconsultinggroup.com

Sarah Kurtz, P. Eng.
skurtz@scsconsultinggroup.com



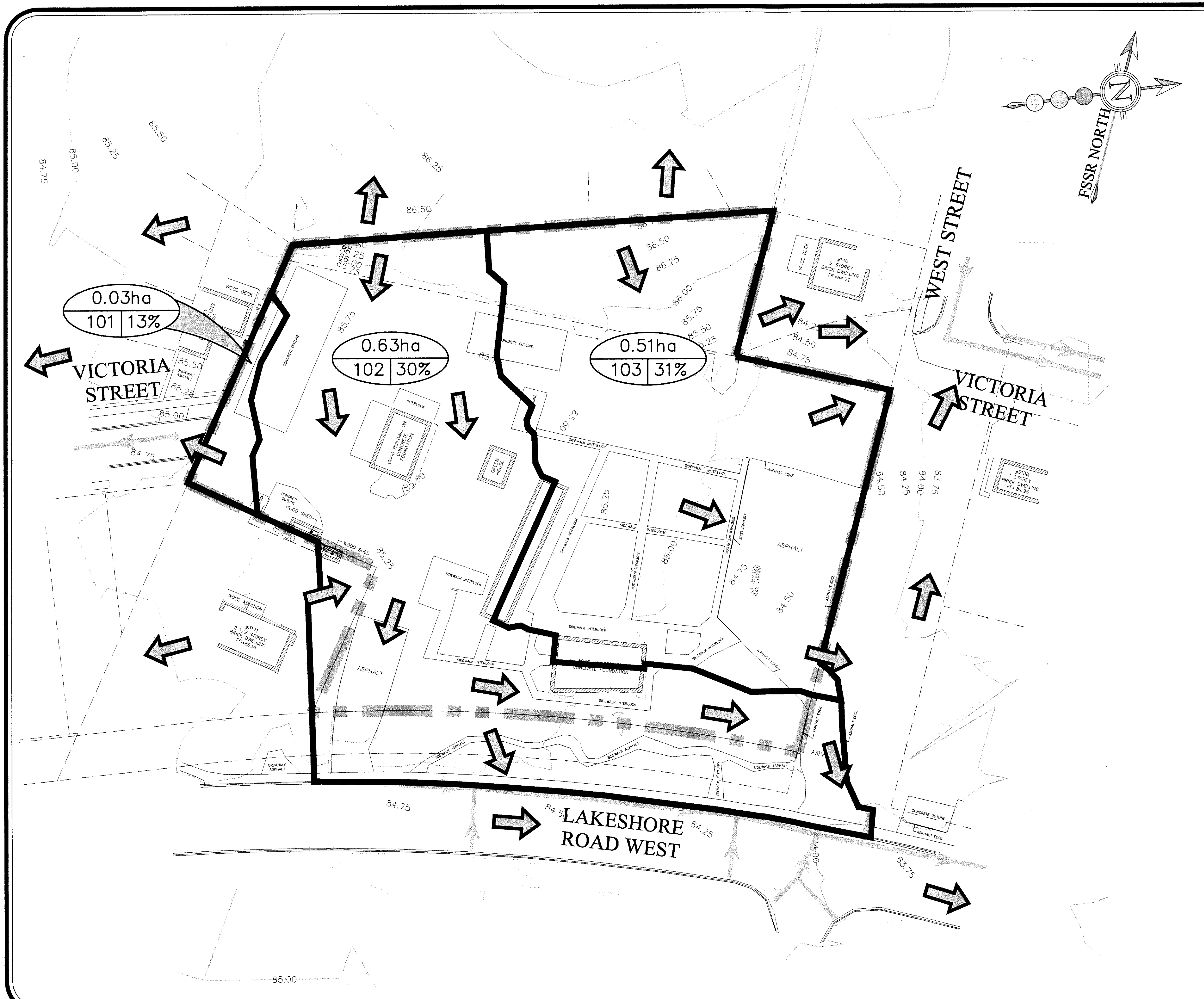
3171 LAKESHORE ROAD WEST, OAKVILLE

SITE LOCATION PLAN

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

DESIGNED BY: N.D.M.	CHECKED BY: S.M.S.	PROJECT No: 1930	FIGURE No: 1.1
SCALE: 1:1000	DATE: JULY 2019		

File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-LOCT-1.1.dwg – Revised by <FBISDA> : Wed, Jul 03 2019 – 10:53am



LEGEND:

- LIMIT OF PROPERTY
- STORM DRAINAGE BOUNDARY
- EXISTING STORM SEWER/MANHOLE
- 83.25 EXISTING CONTOUR
- MAJOR SYSTEM - OVERLAND FLOW
- 0.03ha DRAINAGE AREA (HECTARES)
- 101 | 13% PERCENT (%) IMPERVIOUS
- CATCHMENT ID

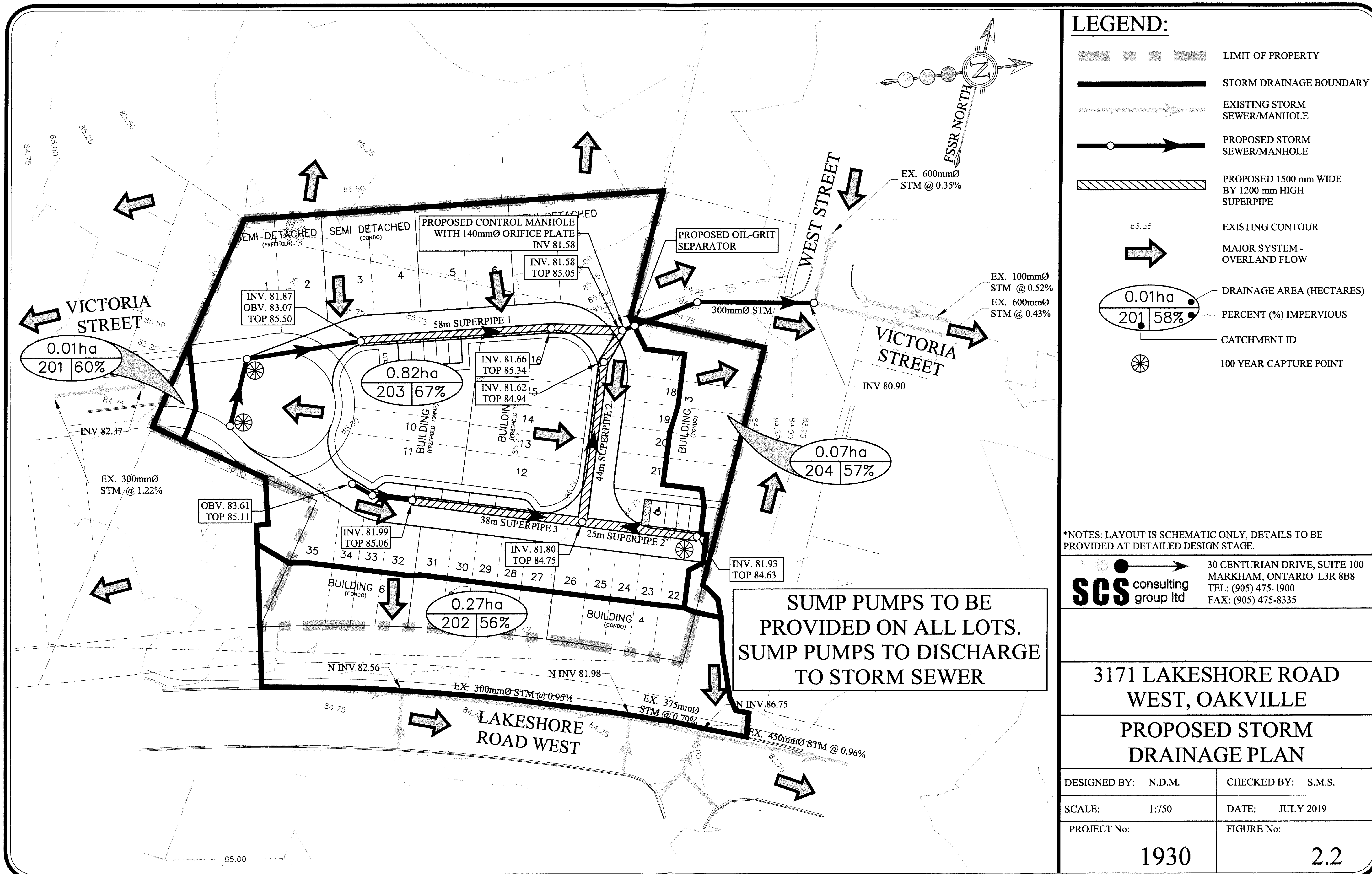
SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

3171 LAKESHORE ROAD WEST, OAKVILLE

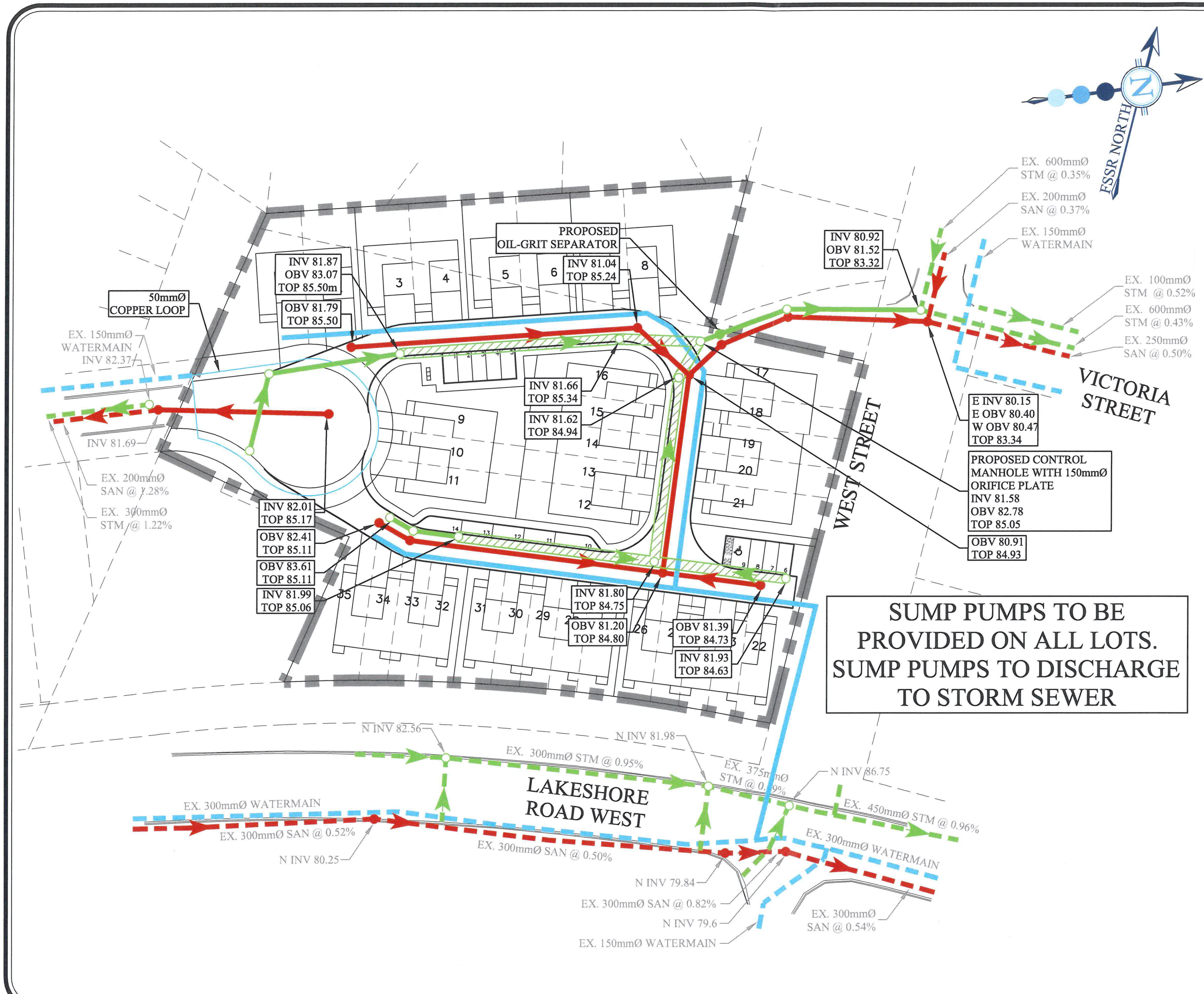
EXISTING STORM DRAINAGE PLAN

DESIGNED BY: N.D.M.	CHECKED BY: S.M.S.
SCALE: 1:750	DATE: JULY 2019
PROJECT No: 1930	FIGURE No: 2.1

File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-PRED-2.1.dwg — Revised by <FBISDA> : Wed, Jul 03 2019 — 10:54am



File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-POST-2.2.dwg - Revised by <FBISDA> : Wed, Jul 03 2019 - 10:54am



LEGEND:

- LIMIT OF PROPERTY
- EXISTING SANITARY SEWER / MANHOLE
- EXISTING STORM SEWER / MANHOLE
- EXISTING WATERMAIN
- PROPOSED 200mm DIAMETER SANITARY SEWER / MANHOLE
- PROPOSED STORM SEWER / MANHOLE
- PROPOSED 1500 mm WIDE BY 1200 mm HIGH SUPERPIPE
- PROPOSED 200mm DIAMETER WATERMAIN
- PROPOSED 50mm DIAMETER COPPER LOOP

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

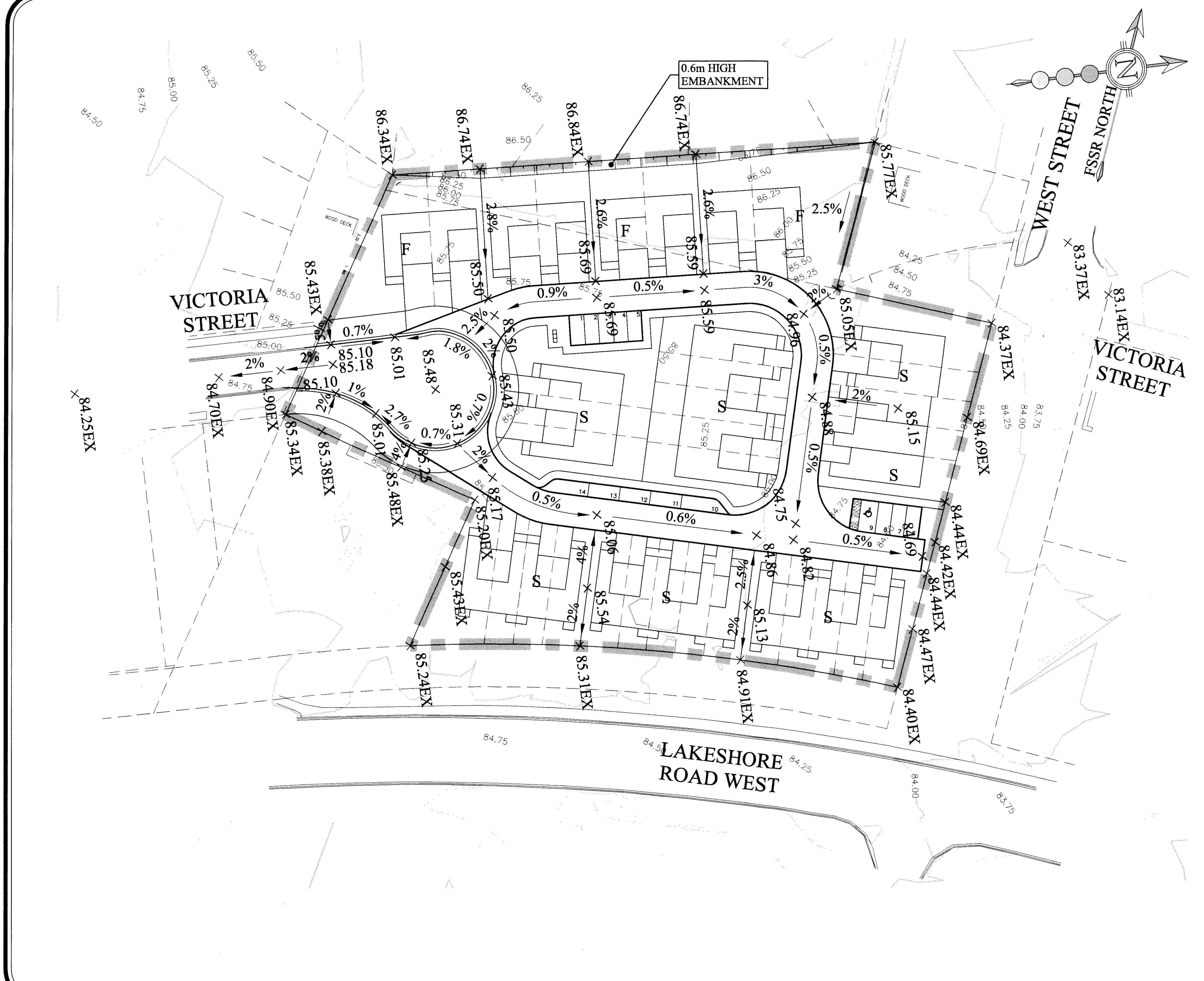
SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

3171 LAKESHORE ROAD WEST, OAKVILLE

PRELIMINARY SERVICING PLAN

DESIGNED BY:	N.D.M.	CHECKED BY:	S.M.S.
SCALE:	1:750	DATE:	JULY 2019
PROJECT No:	1930	FIGURE No:	2.3

File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-SERV-2.3(C3D).dwg - Revised by <FBISDA> : Wed, Jul 03 2019 - 11:00am



LEGEND:

LIMIT OF PROPERTY

265.00 EXISTING CONTOUR AND ELEVATION

× 265.50 PROPOSED ELEVATION

× 265.50 EX EXISTING ELEVATION

2.0% PROPOSED SLOPE

F FRONT DRAINING LOT

S SPLIT DRAINING LOT

3:1 SLOPE MAX

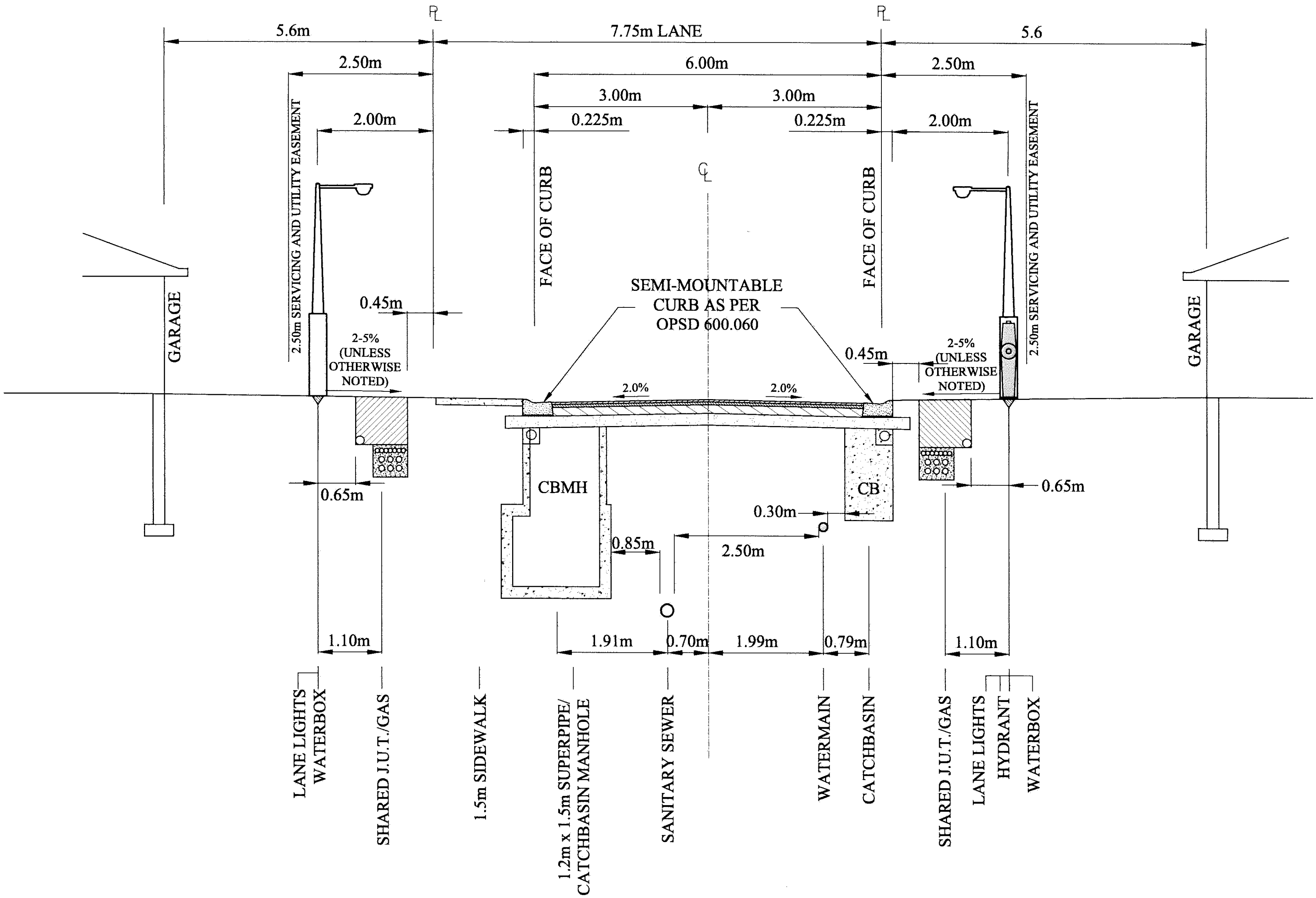
*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

3171 LAKESHORE ROAD WEST, OAKVILLE

PRELIMINARY GRADING PLAN

DESIGNED BY: N.D.M.	CHECKED BY: S.M.S.
SCALE: 1:750	DATE: JULY 2019
PROJECT No: 1930	FIGURE No: 5.1



	3171 LAKESHORE ROAD WEST, OAKVILLE		CONDO ROAD RIGHT-OF-WAY CROSS-SECTION
SCS consulting group ltd 30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335	DESIGNED BY: N.D.M.	CHECKED BY: S.M.S.	PROJECT No: 1930
	SCALE: N.T.S.	DATE: JULY 2019	FIGURE No: 6.1

File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-FIGS-ROW-6.1.dwg – Revised by <NMCINTOSH> : Thu, Jun 20 2019 – 11:47am

APPENDIX A

DRAFT PLAN AND SITE PLAN

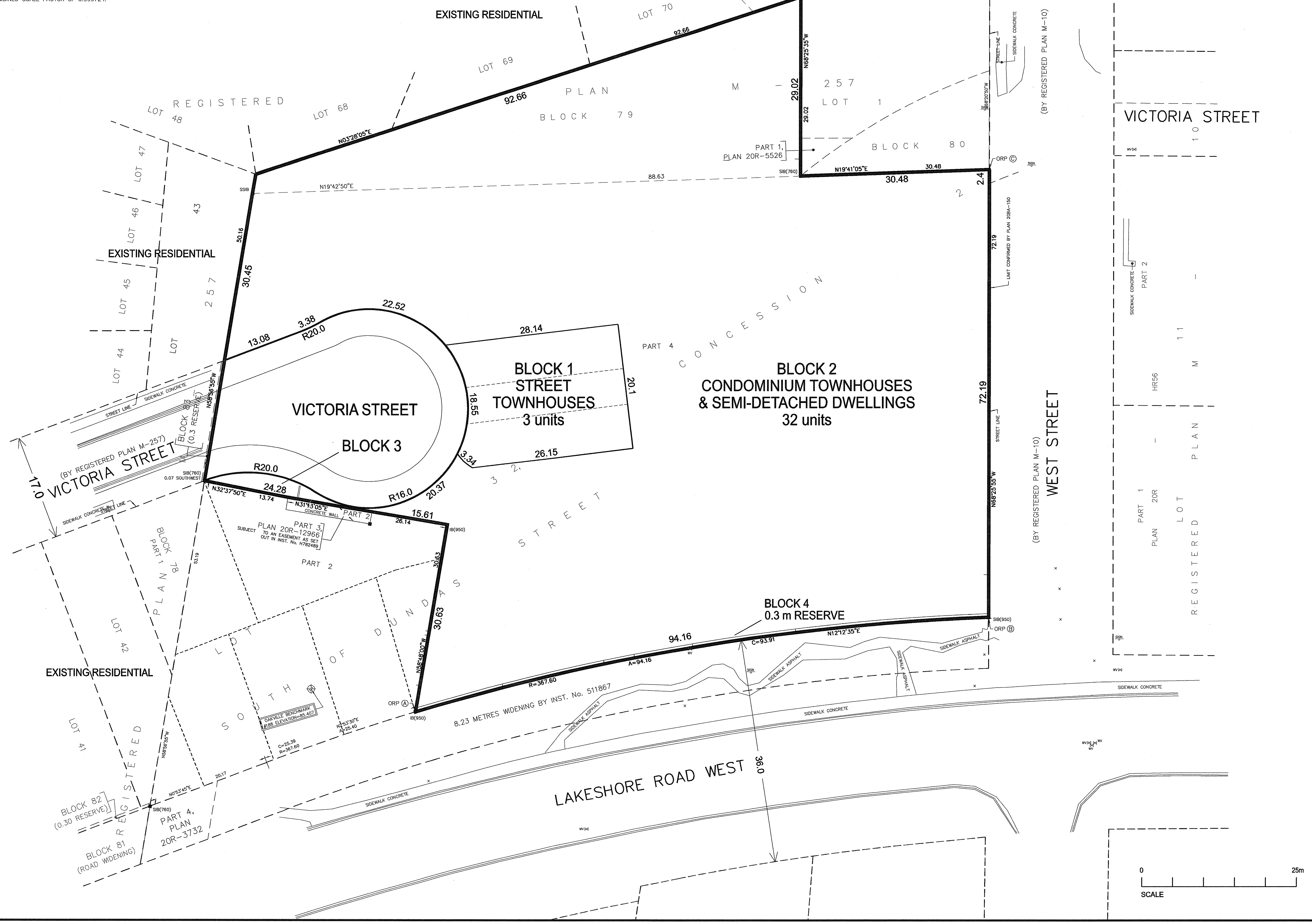
BEARING NOTE

BEARINGS ARE GRID, UTM ZONE 17, NAD 83 (CSRS)(2010.0)
COORDINATES ARE DERIVED FROM GPS OBSERVATIONS USING THE CAN-NET REAL TIME NETWORK.

COORDINATE VALUES ARE, TO URBAN ACCURACY PER SEC. 14 (2) OF O.REG. 216/10, AND CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OR BOUNDARIES SHOWN ON THIS PLAN.

POINT ID	NORTHING	EASTING
ORP A	4804561.31	603892.37
ORP B	4804653.06	603912.23
ORP C	4804679.59	603845.11

DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999724.

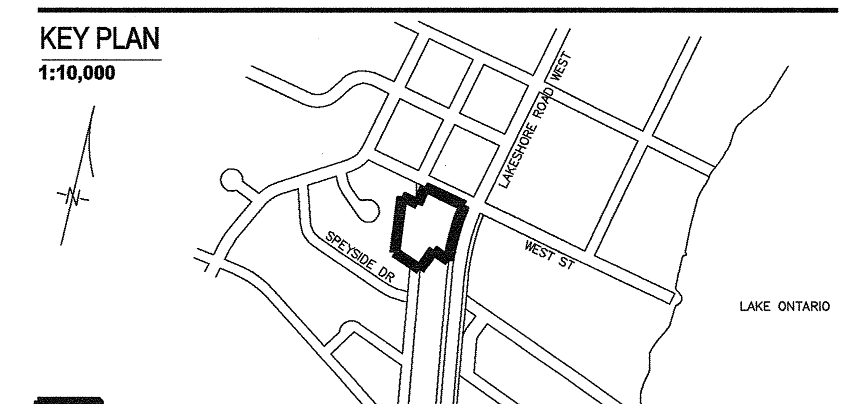


DRAFT PLAN OF SUBDIVISION

PART OF LOT 32,
CONCESSION 4,
SOUTH OF DUNDAS STREET
(GEOGRAPHIC TOWNSHIP OF TRAFALGAR)
AND BLOCK 79, REGISTERED PLAN M-257
TOWN OF OAKVILLE
REGIONAL MUNICIPALITY OF HALTON

WESTON CONSULTING
planning + urban design

KEY PLAN
1:10,000



SUBJECT PROPERTY

OWNER'S CERTIFICATE:

I authorize Weston Consulting Group Inc. to prepare and submit this plan for draft approval.

Date: ____________

GARY BENSKY
WYCLIFFE HOMES
34 DONCASTER AVENUE, SUITE 201
THORNHILL, ONTARIO L3T 4S1
TEL (905) 881-3040

Date: ____________

LOUIS GREENBAUM
VOGUE DEVELOPMENT GROUP INC.
1118 CENTRE STREET, SUITE 208
THORNHILL, ONTARIO L4J 7R9
TEL (905) 886-8500

SURVEYOR'S CERTIFICATE:

I hereby certify that the boundaries of the lands being subdivided and their correct relationship to the adjacent lands are accurately and correctly shown on this plan.

Date: ____________

RADY-PENTEK & EDWARD SURVEYING LTD.
ONTARIO LAND SURVEYORS
643 CHRISLEA ROAD, SUITE 7
WOODBRIDGE, ONTARIO L4L 8A3
TEL (416) 635-5000 FAX (416) 635-5001
TEL (905) 264-0881 FAX (905) 264-2099

ADDITIONAL INFORMATION:

[Section 51(17) of the Planning Act, R.S.O. 1990, c. P.13], as amended to May 29, 2019.
a), b), e), f), g), & j) - on plan.
c) - on key plan
d) - see statistics
h) - piped water to be installed by developer
i) - silt, silty clay, loam
k) - all services to be made available by developer
l) - nil

DEVELOPMENT STATISTICS:

	UNITS	AREA(HA)
Street Townhouses [Block 1]:	3	0.0534
Condominium Townhouses & Semi-Detached [Block 2]:	32	0.8306
Open Space [Block 3]:		0.0046
0.3 m Reserve [Block 4]:		0.0028
Roads:		0.1012
TOTAL	35	0.9926

Overall Density 39.6 uph (excl. Open Space & Road)

	Parking spaces provided
Street Townhouses (3 units):	6
Semi-Detached (8 units): Condo townhouses (24 units)	80 + 14

WESTON CONSULTING
planning + urban design

Vaughan: 201 Millway Ave. Suite 19
Vaughan, Ontario L4K 5K8
T. 905.738.8080 F. 905.738.6637
Toronto: 268 Berkeley St.
Toronto, Ontario M5A 2X5
T. 416.640.9917 F. 905.738.6637

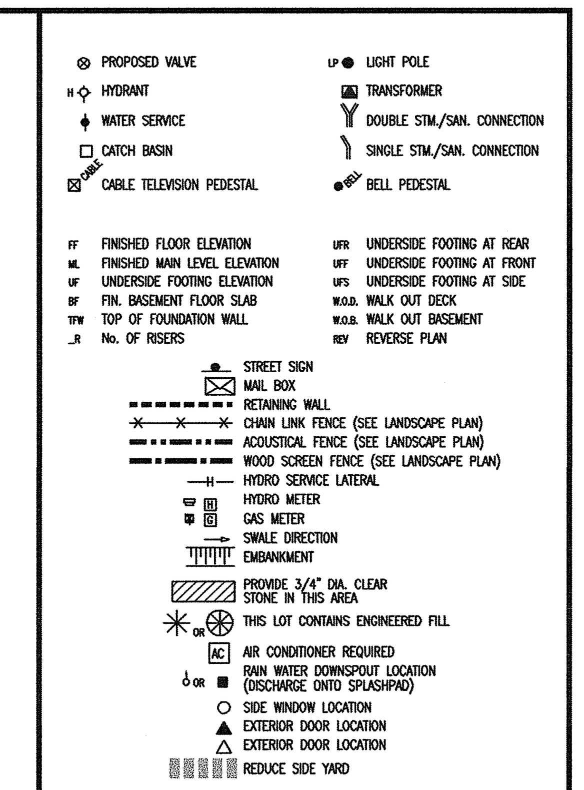
1-800.363.3558 westonconsulting.com

REVISIONS LIST

19 JUNE 2019	FIRST ISSUE

Drawing Number: D1

File Number:	8030
Drawn By:	SM
Planner:	RG
Scale:	1:300
CAD:	8030/draft plans/D1.dgn



SITE STATISTICS SUMMARY (WITH OVERALL LOT AREA

DRAFT PLAN OF SUBDIVISION PART OF LOT 32, CONCESSION 4, SOUTH OF DUNDAS STREET (GEOGRAPHIC TOWNSHIP OF TRAFALGAR) AND BLOCK 79, REGISTERED PLAN M-257 TOWN OF OAKVILLE REGIONAL MUNICIPALITY OF HALTON

ZONING :	–	
OVERALL LOT AREA: *(LOT AREA OWNED BY DEVELOPER)*		9,925.86m2 2.45 ACRES 1.00 Ha
SITE AREA: *(SITE AREA, NOT INCLUDING CUL-DE-SAC R.O.W)*	TOTAL LOT AREA:	8,866.05m2 2.19 ACRES 0.88 Ha
O.B.C. BUILDING CLASSIFICATION	PART 9, GROUP C 3 STOREY BUILDINGS/SEMIS	
BUILDING COVERAGE:	3,108.97m2 (35.07%)	
PAVED AREA:	3,253.97m2 (36.70%)	
LANDSCAPE AREA:	2,503.11m2 (28.23%)	
GROSS FLOOR AREA:	N/A	
AMENITY AREA	–	
DENSITY 35 UNITS TOTAL *(DENSITY CALCULATED FROM SITE AREA)*	39.77 PER Ha 15.98 PER ACRES	
PARKING :	*27 TOWN HOMES RESIDENCE PARKING PROVIDED (1 INTERNAL & 1 EXTERNAL) 54 SPACES TOTAL*	
	8 SEMI-DETACHED HOMES RESIDENCE PARKING PROVIDED (2 INTERNAL & 2 EXTERNAL) 32 SPACES TOTAL	
	VISITOR PARKING: 35 x 0.25 = 8.75 *(9 VISITOR PARKING REQUIRED)* *14 VISITOR PARKING PROVIDED (INCLUDES 1 ACCESSIBLE SPACE)*	

UNIT COUNT

10.35m SEMI DETACHED	8
5.80m STANDARD TOWN HOME	13
5.50m DUAL FRONTAGE TOWN HOME	14
TOTAL	**35 UNITS**

no.	description	date	by
10	.	.	.
9	.	.	.
8	.	.	.
7	.	.	.
6	.	.	.
5	.	.	.
4	.	.	.
3	.	.	.
2	.	.	.
1	ISSUED FOR CLIENT REVIEW	19-05-01	SS

Builder to verify location of all hydrants, street lights, transformers and other services. If minimum dimensions are not maintained, builder is to relocate at his own expense.

Builder to verify service connection elevations prior to constructing foundations.

VA3 DESIGN

255 Consumers Rd
Suite 120
Toronto ON M2J 1R4
t 416.630.2255
f 416.630.4782
va3design.com

VOGUE/WYCLIFFE

project name
—

municipality: OAKVILLE, ON
project no.: 17027

registered plan no.: —
lot/block no.: —

title: SITE PLAN

date: MAY 2016
scale: 1:250
drawing no.: 1
drawn by: STEVE SOSTARIC
checked by: –
file name: 17027-SP

STEVE - H:\ARCHIVE\WORKING\2017\17027.WYC\SITE\17027-SP.dwg - Wed - Jul 24 2019 - 10:51 AM

APPENDIX B

EXCERPTS FROM BACKGROUND INFORMATION

SCS consulting group ltd

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 • TEL: (416) 754-8515 • FAX: (416) 754-8516

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	PETERBOROUGH	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 440-2040	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (416) 754-8516	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

A REPORT TO
WYCLIFFE HOMES

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

3171 LAKESHORE ROAD WEST

TOWN OF OAKVILLE

REFERENCE NO. 1704-S067

MAY 2017

DISTRIBUTION

3 Copies - Wycliffe Homes
1 Copy - Soil Engineers Ltd. (Mississauga)
1 Copy - Soil Engineers Ltd. (Toronto)

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 4, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

The boreholes revealed that beneath a veneer of topsoil and an earth fill, the site is generally underlain by strata of silt and silty clay, overlying the shale bedrock.

4.1 **Topsoil** (Boreholes 1 and 2)

Topsoil was encountered at the ground surface of the landscaped areas. At Boreholes 1 and 2, the topsoil is 10 cm and 30 cm in thickness. The topsoil is dark brown in colour and permeated with roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value. It can be used for general landscaping purpose only.

Due to the humus content, the topsoil will generate an offensive odour under anaerobic conditions and may produce volatile gases; therefore, it must not be buried within the building envelope, or deeper than 1.2 m below the finished grade, as it may have an adverse impact on the environmental well-being of the development.

4.2 **Earth Fill** (All Boreholes)

A layer of earth fill, extending to depths ranging from 1.0 to 2.3 m from grade, was encountered in the boreholes. It consists of sandy silt or silty sand, with occasional rootlet and topsoil inclusions.

The obtained 'N' values ranged from 3 to 22 blows per 30 cm penetration, showing that the earth fill was generally loose, with non-uniform compaction. The natural water content of the earth fill samples range from 9% to 19%, indicating moist to very moist conditions.

In using the earth fill for structural backfill, it must be subexcavated, inspected, sorted free of any serious topsoil inclusions, or other deleterious materials, and properly compacted in layers.

<u>One must be aware that the samples retrieved from boreholes 10 cm in diameter may not be truly representative of the geotechnical and environmental quality of the fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.</u>

4.3 **<u>Silt</u>** (Boreholes 1, 2 and 4)

The silt deposit was generally encountered below the earth fill at 1.0 to 2.3 m from grade. Sample examinations show that the deposit is slightly cohesive, in a very moist condition. The natur6al water content values of the soil samples were determined at 17% to 21%.

The obtained 'N' values ranged from 6 to 26 blows, with a median of 16 per 30 cm of penetration, indicating a relative density of loose to compact, being generally compact.

A grain size analysis was performed on 1 representative sample of the silt and the result is plotted on Figure 5.

According to the above findings, the following engineering properties are deduced:

- High to high frost susceptibility and soil adfreezing potential.
- High water erodibility, susceptible to migration of soil particles through small openings under seepage pressure.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, an average percolation rate of 60 min/cm and runoff coefficients of:

Slope	
0% - 2%	0.15
2% - 6%	0.20
6% +	0.28

- A frictional soil, its shear strength is dependent on its internal friction angle and soil density. Its shear strength is susceptible to impact disturbance, i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and reduction of shear strength.
- In excavation, the wet silt will slough, run with seepage and boil with a piezometric head of about 0.4 m.
- A poor pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 3%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.4 <u>Silty Clay</u> (All Boreholes)

The silty clay was contacted below the silt or earth fill at depths of 1.5 to 3.3 m from grade. It is laminated with sand seams with shale fragments at the lower depth.

The consistency of the silty clay is stiff to hard, being generally very stiff, as confirmed by the obtained 'N' values between 11 and 32 blows, with a median of 16 blows per 30 cm of penetration.

The Atterberg Limits of 1 representative sample and the natural water content of all the clay samples were determined. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	25%
Plastic Limit	15%
Natural Water Content	5% to 21% (median 15%)

The above results show that the clay is cohesive material with low plasticity. The natural water content generally lies below and slightly above the plastic limits, confirming the consistency of the clay deposit as determined by the 'N' values.

A grain size analysis was performed on 1 representative sample of the silty clay; the result is plotted on Figure 6.

According to the above findings, the following engineering properties are deduced:

- Highly frost susceptible and low water erodible.
- Virtually impervious, with an estimated coefficient of permeability of 10^{-7} cm/sec, an average percolation rate of 80 min/cm, and runoff coefficients of:

Slope	
0% - 2%	0.15
2% - 6%	0.20
6% +	0.28

- A cohesive soil, its shear strength is derived from consistency and augmented by the internal friction of the silt. Its shear strength is moisture dependent and, due to the dilatancy of the silt, the overall shear strength of the silty clay is susceptible to impact disturbance, i.e. the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.
- It will generally be stable in a relatively steep cut; however, prolonged exposure will allow the sand seams to become saturated which may lead to localized sloughing.
- A poor pavement-supportive material, with an estimated CBR of 3%.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.5 **Shale** (All Boreholes)

Weathered shale was encountered beneath the silty clay in all the boreholes. It is reddish-brown in colour indicating a Queenston formation. The quality of the shale bedrock, is not proven by rock coring. The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the laminated limy and sandy layers would remain as rock slabs. The shale within the borehole depth can be penetrated by power-augering with some difficulty in grinding through the hard layers.

The shale has a low permeability and occasional pockets of groundwater trapped in its fissures have been encountered. This water may be under a moderate subterranean artesian pressure but, upon release through excavation, the water is likely to drain readily with a limited yield.

The weathered rock can be excavated with considerable effort by a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require the aid of blasting or pneumatic hammering.

The excavated spoil will contain a large amount of hard limy and sandy rock slabs, rendering it virtually impossible to obtain uniform compaction. Therefore, unless the spoil is sorted, it is considered unsuitable for engineering applications.
In sound shale excavation, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock

4.6 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 • TEL: (416) 754-8515 • FAX: (416) 754-8516

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	PETERBOROUGH	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 440-2040	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (416) 754-8516	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

A REPORT TO
WYCLIFFE HOMES OAKVILLE

PHASE ONE ENVIRONMENTAL SITE ASSESSMENT

PROPOSED RESIDENTIAL DEVELOPMENT

LAKESHORE ROAD WEST AND WEST STREET

TOWN OF OAKVILLE

Reference No. 1610-E074

December 20, 2016

DISTRIBUTION

3 Copies - Wycliffe Homes Oakville

8.0 CONCLUSIONS

Soil Engineers Ltd. was retained by Wycliffe Homes Oakville to carry out a Phase One ESA for a site located at Lakeshore Road West and West Street in the Town of Oakville. The Phase One Property has mainly been used as a garden centre. The neighbouring properties consist of a former auto body/repair shop to the northeast and residential properties in the remaining directions.

8.1 Phase Two Assessment Recommendation

Our Phase One Environmental Site Assessment has revealed the following items of environmental concern attendant to the Phase One Property:

- Possible use of pesticide as part of activity in the garden centre and in the area of the former orchard at the Phase One Property.
- A former auto body/repair shop is located adjacent to the northeast of the Phase One Property.

It is recommended that a Phase Two Environmental Site Assessment (Phase Two ESA) be conducted to assess the above concerns.

8.2 RSC Requirements

Based on the type of development proposed for the Phase One Property, an RSC is not required to be filed in accordance with O. Reg. 153/04, as amended. However, local and regional governments may require an RSC as part of the development approval process.

Please note that if there is an intent to file an RSC, in accordance with O. Reg. 153/04, any environmental reports including a Phase One ESA must be dated within 18 months of the date of filing.

8.3 Environmentally Sensitive Area, Body of Water, ANSI

No Environmentally Sensitive Area, Body of Water or ANSI is located on the Phase One Property or within 30 m of the Phase One Property.

8.4 O. Reg. 511/09 and Fill

Due to the potential economic consequences associated with the fill requirements should an RSC be filed for the Phase One Property, we recommend that all site works related to any development proceed prior to the filing of any RSC.

8.5 Legal Requirements

If an RSC has been submitted and filed, the property owner must retain a copy of this report for at least seven (7) years in accordance with O. Reg. 153/04, Section 18.

The objectives and requirements as set out in the O. Reg. 153/04, as amended, for a Phase One ESA were applied in carrying out the environmental site assessment and in the preparation of this report.

SOIL ENGINEERS LTD.

Kathryn Miles, B.Sc. (Eng.)

Eleni Girma Beyene, P.Eng. QP_{ESA}
KM/EGB:km

McIntosh, Nick

From: George Trenkler <george.trenkler@oakville.ca>
Sent: September-21-17 1:33 PM
To: McIntosh, Nick
Subject: Quality Control Criteria vs. OGS Performance History

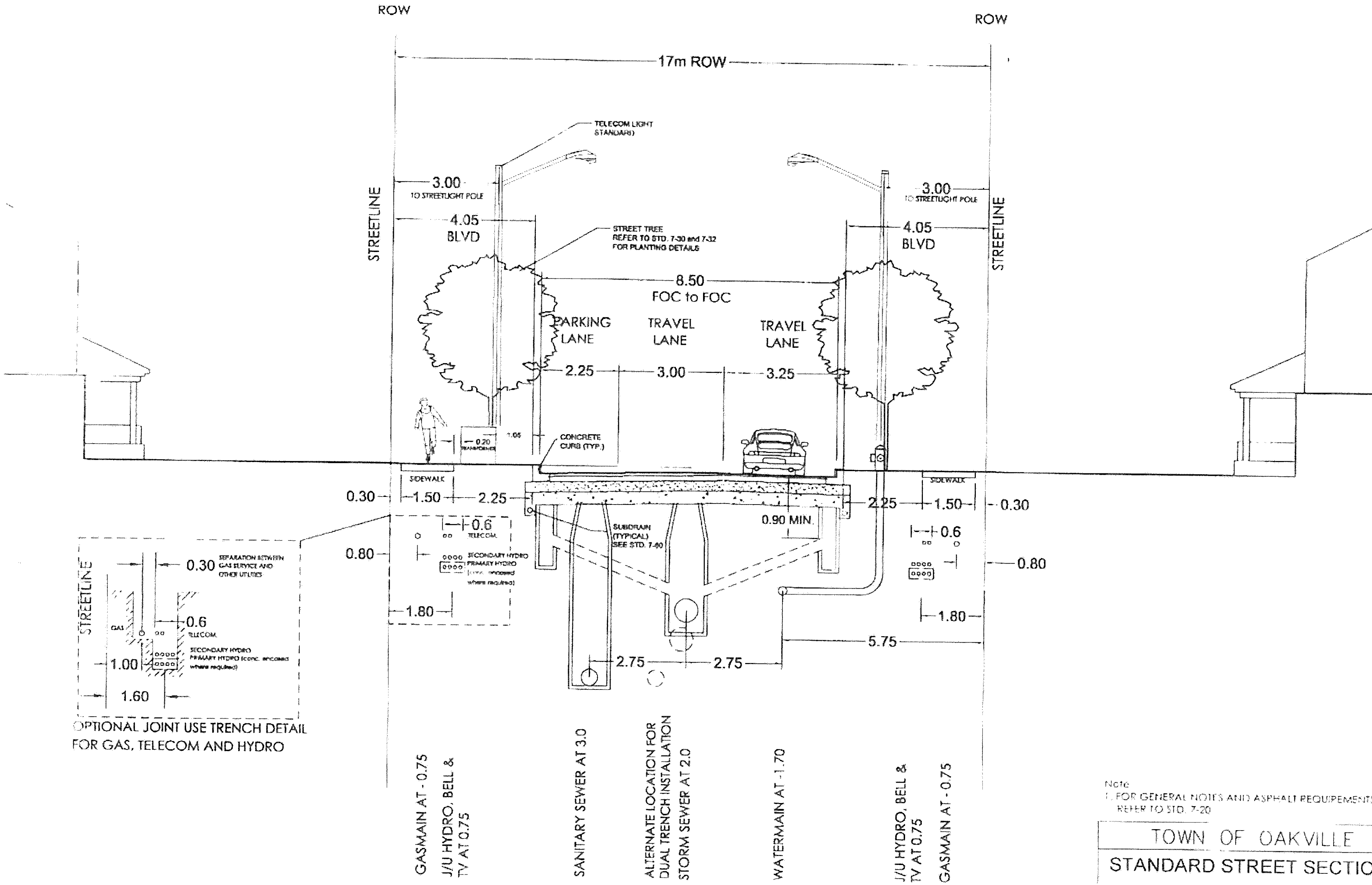
Nick, where we have sensitive creeks governed by watershed/EIR studies, OGS units are not supported and may be applied with a 50% performance credit. This is enforced by the conservation authority.

We will still support the use of OGS units which claim a 80% TSS reduction in the older parts of the Town where infill sites are draining to older storm networks which discharge directly to the lake.

If you are designing the Cudmore property site development the OGS unit can be applied in this case. The CA will not be reviewing this project.

George Trenkler, C.E.T.
Development Engineering Technologist
Development Engineering
Town of Oakville | 905-845-6601, ext.3343 | f: 905-338-4414 | www.oakville.ca

Complete our Community Development customer service survey



Note
1. FOR GENERAL NOTES AND ASPHALT REQUIREMENTS REFER TO STD. 7-20

TOWN OF OAKVILLE

STANDARD STREET SECTION

LOCAL ROADWAY
17.0m RIGHT OF WAY
MODIFIED BOULEVARD SPACE

APPROVED

DIRECTOR OF ENGINEERING AND CONSTRUCTION

STD 7-22A

REVISION DATE
July 3, 2012

APPENDIX C

HYDROLOGY MODELLING

SCS consulting group ltd

EXISTING CONDITIONS
VO2 MODEL SCHEMATIC

3171 Lakeshore Road West
Project No.: 1930
Date: June 2019
Designer: N.D.M.

Existing Conditions
VO2 Parameter Summary

STANDHYD			
Number	**101**	**102**	**103**
Description			
DT(min)	2	2	2
Area (ha)	0.03	0.63	0.51
XIMP[1]	0.01	0.11	0.01
TIMP	0.13	0.30	0.31
CN*	98.0	88.0	84.0
IA(mm)	3.0	3.7	4.2
SLPP(%)	2	2	2
LGP(m)	40	40	40
MNP	0.25	0.25	0.25
DPSI (mm)	2.0	2.0	2.0
SLPI(%)	1	1	1
LGI(m)	14.14	64.81	58.31
MNI	0.013	0.013	0.013

[1]Note that where there is NO directly connected area (ie: roof runoff to grassed areas), the hydrology program does not accept XIMP=0%, therefore, XIMP = 1% has been used

SCS consulting group ltd

Existing Conditions
CN Calculations

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: N.D.M.

Site Soils: (per Soil Engineers Ltd. Geotechnical Investigation dated May, 2017)

Soil Type	**Hydrologic Soil Group**
Silty Clay	CD

TABLE OF CURVE NUMBERS (CN's)**									
Land Use	Hydrologic Soil Type							Manning's 'n'	Source
	A	AB	B	BC	C	CD	D		
Meadow "Good"	30	44	58	64.5	71	74.5	78	0.40	MTO
Woodlot "Fair"	36	48	60	66.5	73	76	79	0.40	MTO
Gravel	76	80.5	85	87	89	90	91	0.30	USDA
Lawns "Good"	39	50	61	67.5	74	77	80	0.25	USDA
Pasture/Range	58	61.5	65	70.5	76	78.5	81	0.17	MTO
Crop	66	70	74	78	82	84	86	0.13	MTO
Fallow (Bare)	77	82	86	89	91	93	94	0.05	MTO
Low Density Residences	57	64.5	72	76.5	81	83.5	86	0.25	USDA
Streets, paved	98	98	98	98	98	98	98	0.01	USDA

1. MTO Drainage Manual (1997), Design Chart 1.09-Soil/Land Use Curve Numbers
2. USDA (1986), Urban Hydrology for Small Watersheds, Table 2.2-Runoff Curve Numbers for Urban Areas

HYDROLOGIC SOIL TYPE (%) - Existing Conditions								
	Hydrologic Soil Type							
Catchment	A	AB	B	BC	C	CD	D	TOTAL
101						100		100
102						100		100
103						100		100

LAND USE (%) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101							100.0			100.0
102				33.2			66.8			100.0
103				58.5			41.5			100.0

Note: Where STANDHYD command used (shaded), impervious fraction is not considered in CN determination, since %Imp directly input in STANDHYD command

CURVE NUMBER (CN) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
101	0.0	0.0	0.0	0.0	0.0	0.0	93.0	0.0	0.0	**93**
102	0.0	0.0	0.0	25.6	0.0	0.0	62.1	0.0	0.0	**88**
103	0.0	0.0	0.0	45.0	0.0	0.0	38.6	0.0	0.0	**84**

** AMC II assumed

Existing Conditions
CN Calculations

Step	Input Values				
	Subcatchment:		101	102	103
1	**CN (AMC II):**		**93**	**88**	**84**
2	CN (AMC III) =		98	95	93
3	100 Year Precipitation, P =	mm	89.46	89.46	89.46

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$

Q = rainfall excess or runoff, mm
S = potential maximum retention or available storage, mm

$$CN = \frac{25400}{S + 254}$$

CN* = modified SCS curve # that better reflects Ia conditions in Ontario

Step	Output Values				
	Subcatchment:		101	102	103
	S_{III} =	mm	5.18	13.37	19.12
	SCS Assumption of 0.2 S = Ia =	mm	1.04	2.67	3.82
4	Q_{III} =	mm	83.53	75.20	70.01
	Preferred Initial Abstraction, Ia =	mm	3.0	3.7	4.2
5	S^*_{III} =	mm	3.04	12.09	18.62
6	CN^*_{III} =	mm	98.82	95.46	93.17
	CN^*_{III}=	Rounded	**99**	**95**	**93**
7	**CN^*_{II}=**	convert	**98**	**88**	**84**

Explanation of Procedure

1 Determine CN based on typical AMC II conditions (attached)
2 Convert CN from AMC II to AMC III conditions (standard SCS tables)
3 Get precipitation depth P for 100 year storm
4 Using CN_{III} with Ia = 0.2S, compute Q_{III} for 100 year precipitation
5 For the same Q_{III}, compute S^*_{III} using Ia=1.5mm (or otherwise determined)
6 Compute CN^*_{III} using S^*_{III}
7 Calculate CN^*_{II} using SCS conversion table

SCS consulting group ltd

Existing Conditions
IA Calculations

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: N.D.M.

LAND USE (%)										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101							100.0			100.0
102				33.2			66.8			100.0
103				58.5			41.5			100.0

IA VALUES (mm)										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
IA (mm)	8	10	2	5	8	8	3	2	2	
101							3.0			3.0
102				1.7			2.0			3.7
103				2.9			1.2			4.2

SCS consulting group ltd

Existing Conditions
Percent Impervious Calculations

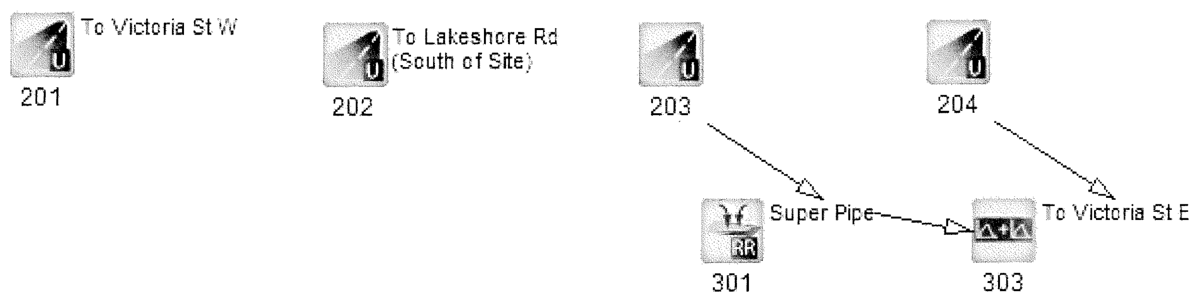
3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: N.D.M.

Land Use Areas (ha)	StandHyd IDs		
	101	102	103
Pavement	0.004	0.16	0.14
Sidewalk			
Driveway			
Roof		0.03	0.02
Total Impervious Area	0.004	0.19	0.16
Total Impervious Area Directly Connected	0.0	0.07	0.00
Total Area	0.03	0.63	0.51
Total Pervious Area	0.03	0.56	0.51
Timp	0.13	0.30	0.31
Ximp	0	0.11	0

SCS consulting group ltd

PROPOSED CONDITIONS
VO2 MODEL SCHEMATIC

3171 Lakeshore Road West
Project No.: 1930
Date: June 2019
Designer: N.D.M.



Proposed Conditions
VO2 Parameter Summary

STANDHYD				
Number	**201**	**202**	**203**	**204**
Description				
DT(min)	2	2	2	2
Area (ha)	0.01	0.27	0.82	0.07
XIMP[1,2]	0.50	0.11	0.66	0.01
TIMP[2]	0.60	0.56	0.67	0.57
CN*	75.0	75.0	75.0	75.0
IA(mm)	5.0	5.0	5.0	5.0
SLPP(%)	2	2	2	2
LGP(m)	40	40	40	40
MNP	0.25	0.25	0.25	0.25
DPSI (mm)	2.0	2.0	2.0	2.0
SLPI(%)	1	1	1	1
LGI(m)	8.16	42.43	73.94	21.60
MNI	0.013	0.013	0.013	0.013

SCS consulting group ltd

Proposed Conditions
CN Calculations

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: N.D.M.

Site Soils: (per Soil Engineers Ltd. Geotechnical Investigation dated May, 2017)

Soil Type	**Hydrologic Soil Group**
Silty Clay	CD

TABLE OF CURVE NUMBERS (CN's)**										
Land Use	Hydrologic Soil Type							Manning's	Source
	A	AB	B	BC	C	CD	D	'n'	
Meadow "Good"	30	44	58	64.5	71	74.5	78	0.40	MTO
Woodlot "Fair"	36	48	60	66.5	73	76	79	0.40	MTO
Gravel	76	80.5	85	87	89	90	91	0.30	USDA
Lawns "Good"	39	50	61	67.5	74	77	80	0.25	USDA
Pasture/Range	58	61.5	65	70.5	76	78.5	81	0.17	MTO
Crop	66	70	74	78	82	84	86	0.13	MTO
Fallow (Bare)	77	82	86	89	91	93	94	0.05	MTO
Low Density Residences	57	64.5	72	76.5	81	83.5	86	0.25	USDA
Streets, paved	98	98	98	98	98	98	98	0.01	USDA

1. MTO Drainage Manual (1997), Design Chart 1.09-Soil/Land Use Curve Numbers
2. USDA (1986), Urban Hydrology for Small Watersheds, Table 2.2-Runoff Curve Numbers for Urban Areas

HYDROLOGIC SOIL TYPE (%) - Proposed Conditions								
	Hydrologic Soil Type							
Catchment	A	AB	B	BC	C	CD	D	TOTAL
201						100		100
202						100		100
203						100		100
204						100		100

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
201				100.0						100.0
202				100.0						100.0
203				100.0						100.0
204				100.0						100.0

Note: Where STANDHYD command used (shaded), impervious fraction is not considered in CN determination, since %Imp directly input in STANDHYD command

CURVE NUMBER (CN) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
201	0.0	0.0	0.0	77.0	0.0	0.0	0.0	0.0	0.0	77
202	0.0	0.0	0.0	77.0	0.0	0.0	0.0	0.0	0.0	77
203	0.0	0.0	0.0	77.0	0.0	0.0	0.0	0.0	0.0	77
204	0.0	0.0	0.0	77.0	0.0	0.0	0.0	0.0	0.0	77

** AMC II assumed

SCS consulting group ltd

Proposed Conditions
CN Calculations

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: N.D.M.

Step	Input Values		201	202	203	204
	Subcatchment:		201	202	203	204
1	**CN (AMC II):**		**77**	**77**	**77**	**77**
2	CN (AMC III) =		89	89	89	89
3	100 Year Precipitation, P =	mm	89.46	89.46	89.46	89.46

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \qquad S = \frac{(P - Ia)^2}{Q} - (P - Ia)$$

Q = rainfall excess or runoff, mm
S = potential maximum retention or available storage, mm

$$CN = \frac{25400}{S + 254} \qquad S = \frac{25400}{CN} - 254$$

CN* = modified SCS curve # that better reflects Ia conditions in Ontario

Step	Output Values					
	Subcatchment:		201	202	203	204
	S_{III} =	mm	31.39	31.39	31.39	31.39
	SCS Assumption of 0.2 S = Ia =	mm	6.28	6.28	6.28	6.28
4	Q_{III} =	mm	60.39	60.39	60.39	60.39
	Preferred Initial Abstraction, Ia =	mm	5.0	5.0	5.0	5.0
5	S^*_{III} =	mm	33.66	33.66	33.66	33.66
6	CN^*_{III} =	mm	88.30	88.30	88.30	88.30
	CN^*_{III}=	Rounded	**88**	**88**	**88**	**88**
7	**CN^*_{II}=**	convert	**75**	**75**	**75**	**75**

Explanation of Procedure

1 Determine CN based on typical AMC II conditions (attached)
2 Convert CN from AMC II to AMC III conditions (standard SCS tables)
3 Get precipitation depth P for 100 year storm
4 Using CN_{III} with Ia = 0.2S, compute Q_{III} for 100 year precipitation
5 For the same Q_{III}, compute S^*_{III} using Ia=1.5mm (or otherwise determined)
6 Compute CN^*_{III} using S^*_{III}
7 Calculate CN^*_{II} using SCS conversion table

Proposed Conditions
IA Calculations

LAND USE (%)										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
201				100.0						100.0
202				100.0						100.0
203				100.0						100.0
204				100.0						100.0

IA VALUES (mm)										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
IA (mm)	8	10	2	5	8	8	3	2	2	
201				5.0						5.0
202				5.0						5.0
203				5.0						5.0
204				5.0						5.0

Proposed Conditions
Percent Impervious Calculations

Land Use Areas (ha)	StandHyd IDs			
	201	202	203	204
Road	0.005	0.00	0.20	0.00
Sidewalk	0.001	0.09	0.04	0.01
Driveway	0.000	0.00	0.10	0.00
Roof	0.000	0.06	0.21	0.03
Total Impervious Area	0.006	0.15	0.55	0.04
Total Impervious Area Directly Connected	0.005	0.03	0.54	0.00
Total Area	0.01	0.27	0.82	0.07
Total Pervious Area	0.005	0.24	0.28	0.07
Timp	0.60	0.56	0.67	0.57
Ximp	0.50	0.11	0.66	0.00

APPENDIX D

STORMWATER MANAGEMENT CALCULATIONS

SCS consulting group ltd

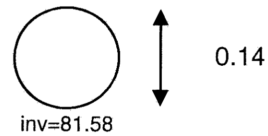
SCS consulting group ltd

Control Structure Summary

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: E.S.D.

Orifice 1

Invert =	81.58 m
Size =	0.140 m
Orifice Coefficient, C =	0.62
Obvert =	81.72 m



Starting Water Level (m) = 81.58
Elevation Increment (m) = 0.02

Shading represents Storage-Discharge pairings used in VO2 modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Storage (m^3)	24-Hour Chicago
81.58	0.000	0	
81.60	0.000	0	
81.62	0.001	0	
81.64	0.003	1	
81.66	0.004	1	
81.68	0.007	2	
81.70	0.009	3	
81.72	0.011	5	
81.74	0.013	6	
81.76	0.014	8	
81.78	0.015	10	
81.80	0.016	12	
81.82	0.017	15	
81.84	0.018	17	
81.86	0.019	21	
81.88	0.020	24	
81.90	0.021	28	
81.92	0.022	32	
81.94	0.023	36	
81.96	0.024	40	
81.98	0.024	45	
82.00	0.025	50	
82.02	0.026	54	
82.04	0.026	59	
82.06	0.027	64	
82.08	0.028	69	
82.10	0.028	74	
82.12	0.029	79	
82.14	0.030	84	2 Year
82.16	0.030	89	
82.18	0.031	93	
82.20	0.031	98	
82.22	0.032	103	
82.24	0.032	108	
82.26	0.033	113	
82.28	0.034	118	
82.30	0.034	123	
82.32	0.035	128	
82.34	0.035	133	5 Year
82.36	0.036	138	
82.38	0.036	143	
82.40	0.037	148	
82.42	0.037	153	
82.44	0.038	158	
82.46	0.038	163	
82.48	0.039	168	10 Year

Control Structure Summary

Starting Water Level (m) = 81.58
Elevation Increment (m) = 0.02

Shading represents Storage-Discharge pairings used in VO2 modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Storage (m^3)	24-Hour Chicago
82.50	0.039	173	
82.52	0.039	178	
82.54	0.040	183	
82.56	0.040	188	
82.58	0.041	192	
82.60	0.041	197	
82.62	0.042	202	
82.64	0.042	207	
82.66	0.042	212	25 Year
82.68	0.043	217	
82.70	0.043	222	
82.72	0.044	227	
82.74	0.044	232	
82.76	0.045	236	
82.78	0.045	241	
82.80	0.045	245	50 Year
82.82	0.046	250	
82.84	0.046	254	
82.86	0.047	257	
82.88	0.047	261	
82.90	0.047	264	
82.92	0.048	267	
82.94	0.048	270	
82.96	0.048	273	
82.98	0.049	275	
83.00	0.049	277	
83.02	0.049	279	100 Year
83.04	0.050	280	
83.06	0.050	281	
83.08	0.051	282	
83.10	0.051	283	
83.12	0.051	283	
83.13	0.051	283	
83.15	0.052	284	
83.17	0.052	284	
83.19	0.052	284	Maximum Storage Provided

SCS consulting group ltd

Superpipe Parameters

3171 Lakeshore Road West
Project Number: 1930
Date: June 2019
Designer Initials: E.S.D.

Control Structure Outlet Invert = 81.58 m

Superpipe 1 Parameters

Length = 58 m
Slope = 0.5 %
Rise = 1200 mm
Span = 1500 mm
Haunch height = 200 mm
D/S Superpipe Invert = 81.58 m
Cross-Sectional Area = 1.72 m^2
Total Storage Provided = 99.76 m^3

U/S Superpipe Invert = 81.87 m
U/S Superpipe Obvert = 83.07 m
D/S Superpipe Obvert = 82.78 m

Stage/Storage Table:

Stage (m)	Total Volume (m^3)
81.58	0.00
81.68	1.17
81.78	4.93
81.98	19.91
82.18	37.26
82.38	54.66
82.58	72.06
82.78	88.93
82.98	98.82
83.00	99.20
83.02	99.48
83.04	99.66
83.06	99.75
83.08	99.76
83.10	99.76
83.12	99.76
83.13	99.76
83.15	99.76
83.17	99.76
83.19	99.76

Superpipe 2 Parameters

Length = 69 m
Slope = 0.5 %
Rise = 1200 mm
Span = 1500 mm
Haunch height = 200 mm
D/S Superpipe Invert = 81.58 m

Total Storage Provided = 118.68 m^3

U/S Superpipe Invert = 81.93 m
U/S Superpipe Obvert = 83.13 m
D/S Superpipe Obvert = 82.78 m

Stage/Storage Table:

Stage (m)	Total Volume (m^3)
81.58	0.00
81.68	1.17
81.78	4.93
81.98	20.99
82.18	41.49
82.38	62.19
82.58	82.89
82.78	103.05
82.98	116.16
83.00	116.83
83.02	117.39
83.04	117.84
83.06	118.20
83.08	118.45
83.10	118.61
83.12	118.68
83.13	118.68
83.15	118.68
83.17	118.68
83.19	118.68

Superpipe 3 Parameters

Length =	38 m
Slope =	0.5 %
Rise =	1200 mm
Span =	1500 mm
Haunch height =	200 mm
D/S Superpipe Invert =	81.80 m
Total Storage Provided =	65.36 m^3
U/S Superpipe Invert =	81.99 m
U/S Superpipe Obvert =	83.19 m
D/S Superpipe Obvert =	83.00 m

Stage/Storage Table:

Stage (m)	Total Volume (m^3)
81.58	0.00
81.68	0.00
81.78	0.00
81.98	3.95
82.18	14.73
82.38	26.12
82.58	37.52
82.78	48.92
82.98	59.94
83.00	60.93
83.02	61.85
83.04	62.66
83.06	63.35
83.08	63.94
83.10	64.42
83.12	64.80
83.13	64.95
83.15	65.18
83.17	65.32
83.19	65.36

APPENDIX E

SANITARY SEWER CAPACITY CALCULATIONS

scs consulting group ltd

SCS consulting group ltd

Sanitary Design Sheet
3171 Lakeshore Road West

Oakville, Halton Region

Project: 3171 Lakeshore Road West
Project No. 1930
Date: 29-Jul-19
Designed By: N.D.M.
Reviewed By: S.M.S.

Minimum Sewer Diameter (mm) = 200
Mannings n = 0.013
Minimum Velocity (m/s) = 0.60
Maximum Velocity (m/s) = 3
Minimum Pipe Slope (%) = 0.50

Avg. Domestic Flow (l/cap/day) = 275
Infiltration Rate (l/s/ha) = 0.286
Max. Harmon Peaking Factor = 3.4
Min. Harmon Peaking Factor = 2.0
NOMINAL PIPE SIZE USED

Notes: Areas Derived From Sanitary Operating Maps as Shown on **Figure E.1** in this appendix
West River St. from MH6844 to MH6845 sewer size assummed to be 450mm diameter based on Wastewater Capital Program Table 46 in this appendix
Sheldon Creek WWPS flow rate from Sustainable Halton Water and Wastewater Master Plan Appendix 1-3 2031 Inflow in this appendix

P:\1930 3171 Lakeshore Road West, Oakville\Design\Pipe Design\Sanitary\[1930-Existing Sanitary Sewer Capacity.xlsm]Design

LOCATION			RESIDENTIAL							INDUSTRIAL/COMMERCIAL/INSTITUTIONAL					FLOW CALCULATIONS								PIPE DATA				
STREET	MANHOLE FROM	MANHOLE TO	AREA	ACCUM. AREA	UNITS	DENSITY PER UNIT	DENSITY PER HA	RESIDENTIAL POPULATION	ACCUM. RESIDENTIAL POPULATION	AREA	ACCUM. AREA	POPULATION DENSITY	FLOW RATE	ACCUM. EQUIV. POPULATION	INFILTRATION	TOTAL ACCUM. POPULATION	AVG. DOMESTIC FLOW	ACCUM. AVG. DOMESTIC FLOW	PEAKING FACTOR	PEAKED RESIDENTIAL FLOW	ICI FLOW	TOTAL FLOW	LENGTH	PIPE DIAMETER	SLOPE	FULL FLOW CAPACITY	FULL FLOW VELOCITY
			(ha)	(ha)	(#)	(p/unit)	(p/ha)			(ha)	(ha)	(p/ha)	(l/s/ha)		(L/s)		(L/s)	(L/s)		(L/s)	(L/s)	(L/s)	(m)	(mm)	(%)	(m3/s)	(m/s)
Victoria St. (West)	1	MH6676	0.17	0.17	0		135.0	23.0	23.0	0.00	0.00	0	0.00	0.0	0.0	23.0	0.1	0.1	3.38	0.2	0.0	0.3	100.0	200	1.00	32.8	1.04
Victoria St. (East)	2	MH6617	0.82	0.82	0		135.0	110.7	110.7	0.00	0.00	0	0.00	0.0	0.2	110.7	0.4	0.4	3.38	1.2	0.0	1.4	100.0	200	0.50	23.2	0.74
Victoria St.	MH6617	MH6603	3.62	4.44	0		55.0	198.8	309.5	0.00	0.00	0	0.00	0.0	1.3	309.5	0.6	1.0	3.38	3.3	0.0	4.6	100.0	250	0.50	42.0	0.86
Victoria St.	MH6603	MH6600	0.75	5.18	0		55.0	41.2	350.7	0.00	0.00	0	0	0.0	1.5	350.7	0.1	1.1	3.38	3.8	0.0	5.3	100.0	250	0.50	42.0	0.86
Missisauga St.	MH6600	MH6599	31.48	36.66	0		55.0	1731.2	2081.9	0.00	0.00	0	0	0.0	10.5	2081.9	5.5	6.6	3.38	22.4	0.0	32.9	100.0	300	0.33	55.5	0.79
Sheldon Creek WWPS	-	MH6917	0.00	0.00	0			0.0	0.0	1.00	1.00	0	36.5	0.0	0.3	0.0	0.0	0.0	3.38	0.0	36.5	36.8	-	-	-	-	-
Lakeshore Rd. W(1)	MH6917	MH6605	2.51	2.51	0		78.0	195.6	195.6	0.12	1.12	0	90	0.0	1.0	195.6	0.6	0.6	3.38	2.1	47.4	50.5	100.0	300	0.55	71.7	1.01
Lakeshore Rd. W	MH6605	MH6599	1.10	3.61	0		55.0	60.6	256.2	0.00	1.12	0	0	0.0	1.4	256.2	0.2	0.8	3.38	2.8	47.4	51.5	100.0	380	0.45	121.8	1.07
Lakeshore Rd. W	MH6599	MH7030	11.31	51.58	0		55.0	621.8	2959.9	0.78	1.90	125	0.29	98.0	15.3	3057.9	2.3	9.7	3.38	32.9	47.6	95.8	100.0	380	0.86	168.4	1.48
West River St.	MH7030	MH6844	2.76	54.34	0		55.0	151.8	3111.8	0.00	1.90	0	0	98.0	16.1	3209.7	0.5	10.2	3.38	34.6	47.6	98.2	100.0	380	1.30	207.0	1.83
West River St.	MH6844	MH6845	5.49	59.83	0		55.0	302.0	3413.8	0.00	1.90	0	0	98.0	17.7	3511.7	1.0	11.2	3.38	37.8	47.6	103.1	100.0	450	0.42	184.7	1.16

(1) - Population density based on area weighted calculation for Townhouses and Single Detached

Table 46 - Wastewater Capital Program

Region IPFS ID	Project Description	Municipality	Project Type	Total Estimated Cost (2012$)	EA Schedule	Project Start Year
Oakville						
3706	600 mm WWM crosing Dundas St and 600mm WWM on Dundas St from 900m west of Colonel William Parkway to Colonel William Parkway	OAK	WWM	$ 4,005,000	A+	2012-2016
4994	600 mm WWM on new North Oakville road from Burnhamthorpe Rd to Dundas Street	OAK	WWM	$ 5,880,000	A+	2012-2016
4995	New 37 ML/d WWPS on Dundas St E approximately 550m west of Ninth Line (428 L/s)	OAK	WWPS	$ 6,935,000	B (Satisfied through 2008 MP)	2012-2016
5062	600 mm WWM on new North Oakville road from Burnhamthorpe Rd West to Dundas Street	OAK	WWM	$ 2,579,000	A+	2012-2016
5063	525 mm WWM on new North Oakville road from Burnhamthorpe Rd West to Dundas Street	OAK	WWM	$ 2,087,000	A+	2012-2016
5095	Mid Halton WWTP Odour Control Studies (OAK) - $87,000/year for 20 years	OAK	STUDY	$ 1,740,000	N/A	2012-2031
5945	Mid-Halton new effluent sewer/outfall - Construction	OAK	WWTP	$ 90,000,000	3808 (EA/D), EA Completed	2012-2016
6215	600 mm WWM on Neyagawa Blvd from Burnhamthorpe Rd to new internal North Oakville road, north of Dundas St	OAK	WWM	$ 4,000,000	A+	2012-2016
6380	2400 mm WWM on new 3rd Line from 700 m north of Dundas St to Dundas St - Construction	OAK	WWM	$ 7,632,000	3794 (EA/D), EA Completed	2012-2016
6381	2400 mm WWM on new road alignment from Lower Base Line to 3rd Line (700 m north of Dundas St) - Construction	OAK	WWM	$ 49,923,000	3794 (EA/D), EA Completed	2012-2016
6383	Mid Halton WWTP Phase 4/5 expansion from 75 ML/d to 125 ML/d (OAK) - Construction	OAK	WWTP	$ 120,000,000	3808 (EA/D), EA Completed	2012-2016
6384	Mid Halton North Pumping Station Expansion - Construction	OAK	WWPS	$ 13,500,000	3808 (EA/D), EA Completed	2012-2016
6481	450 mm WWM on internal road parallel to Dundas St from west of 16 Mile Creek Bridge to 190 m east of Proudfoot Trail	OAK	WWM	$ 296,000	A+	2012-2016
6546	Construction of approx 100 m of new local sewer to eliminate Shepherd Rd WWPS sewer sized adequately to accommodate 2031 flows of approximately 21.2 L/s	OAK	WWPS	$ 400,000	A+	2027-2031
6551	525 mm WWM on new North Oakville road from Burnhamthorpe Rd to Project #5062	OAK	WWM	$ 1,519,000	A+	2012-2016
6588	Mid-Halton WWTP expansion from 125 ML/d to 175 ML/d	OAK	WWTP	$ 93,304,000	C	2012-2016
6526	450 mm WWM on service Rd for West River WWPS from West River Street to WWPS	OAK	WWM	$ 141,000	A+	2017-2021
6527	450 mm WWM on service road to Marine Drive WWPS from Marine Drive	OAK	WWM	$ 27,000	A+	2017-2021
6528	31.9 L/s upgrade of West River WWPS	OAK	WWPS	$ 467,000	A+	2012-2016
6529	375 mm WWM on Oak Park from Dundas Street East to Central Park then along Central Park to Georgian Drive	OAK	WWM	$ 838,000	A+	2012-2016
6530	300 mm WWM on Kerr Street between Forster Park and Rebecca Street	OAK	WWM	$ 864,000	A+	2017-2021
6531	250 mm WWM on Chisholm/Rebecca Street between Forsyth Street and Chisholm Street on Rebecca Street and on Chisholm Street between Rebecca Street and 45 m north of Lakeshore Rd West	OAK	WWM	$ 218,000	A+	2022-2026
6532	525 mm WWM on Stewart Street between Felan Drive and Kerr Street	OAK	WWM	$ 360,000	A+	2027-2031
6534	375 mm WWM on Lyons Lane between Cross Ave and due north up Lyons Lane 150 m	OAK	WWM	$ 206,000	A+	2022-2026
6535	450 mm WWM on Trafalgar Rd between 10 m north of Inglehart Street North and over Cornwall Rd and railway to connect to Cross Ave	OAK	WWM	$ 1,191,000	A+	2022-2026
6536	525 mm WWM on Cross Avenue between Argus Rd and Lyons Lane	OAK	WWM	$ 1,902,000	A+	2022-2026
6537	675 mm WWM on Trafalgar Rd between Spruce Steet until 60 m north of Cornwall Rd where it follows the side road crossing the railway line and through the Go Transit Stn car Park and heads due west and north up Argus Rd for 60 m	OAK	WWM	$ 3,276,000	A+	2022-2026
6538	Upgraded gravity sewer within UGC on North Service Rd from Truman Ave to east of Pearson Dr	OAK	WWM	$ 472,000	A+	2012-2016
6540	Twin 900 mm WWM on Trafalgar Rd and Randall Street/Rebecca Street from Lawson Street to Wilson Street	OAK	WWM	$ 8,574,000	A+	2012-2016
6541	Deep Trunk Sewer on Rebecca St and Lakeshore Rd W from Wilson St to Oakville SW WWTP	OAK	WWM	$ 45,502,000	B	2012-2016
6542	Decommissioning of 5 WWPSs	OAK	WWPS	$ 500,000	A+	2012-2016
6543	Gravity Sewers from Decommissioned WWPS's to New Deep Trunk	OAK	WWM	$ 1,311,000	A+	2012-2016
6544	6 L/s upgrade of Bronte Yacht Club WWPS	OAK	WWPS	$ 87,000	A+	2012-2016
Subtotal Oakville				**$ 469,736,000**		
Burlington						
5907	300 mm WWM North Aldershot Servicing	BURL	WWM	$ 4,268,000	Being Satisfied Under Separate Planning Study	2012-2016
6143	300mm WWM on North Service Rd from 440m east of Waterdown Rd to 360m north of North Service Rd - Part of 5907	BURL	WWM	$ 500,000	A+	2012-2016
6482	300 mm WWM on Plains Rd West from Grand View heading due north west	BURL	WWM	$ 128,000	A+	2012-2016
6483	325 mm WWM on Plains Rd West from Grand View heading due north west	BURL	WWM	$ 82,000	A+	2012-2016
6484	375 mm WWM on Plains Rd West from Grand View heading due north west	BURL	WWM	$ 91,000	A+	2012-2016
6485	375 mm WWM on Plains Rd West between Howard Rd and entrance to Spring Gardens	BURL	WWM	$ 588,000	A+	2017-2021
6486	200 mm WWM on Plains Rd West between Howard Rd and entrance to Spring Gardens	BURL	WWM	$ 61,000	A+	2017-2021
6487	525 mm WWM on Plains Rd West between Howard Rd and entrance to Spring Gardens	BURL	WWM	$ 2,287,000	A+	2017-2021
6488	250 mm WWM on Guelph Line between Woodward and 120m south of Prospect Street	BURL	WWM	$ 331,000	A+	2017-2021
6489	450 mm WWM on Appleby Line between Fairview Street and 151 m south of Harvester Rd	BURL	WWM	$ 1,445,000	A+	2017-2021
6490	300 mm WWM on Pearl Street between Old Lakeshore Rd and Pine Street	BURL	WWM	$ 195,000	A+	2017-2021
6491	525 mm WWM on Lakshore Rd between Old Lakeshore Rd entrance and Torrance Street	BURL	WWM	$ 133,000	A+	2017-2021

APPENDIX 1-3 WASTEWATER PUMPING STATION FLOWS AND CAPACITIES

PS ID	Description	Firm Capacity	2010 Inflow	2016 Inflow	2021 Inflow	2026 Inflow	2031 Inflow
		(L/s)	(L/s)	(L/s)	(L/s)	(L/s)	(L/s)
1	ARMSTRONG AVE WWPS	170.00	120.87	127.70	126.91	126.91	130.11
2	GOLLOP CR WWPS	63.10	22.18	34.92	35.34	35.90	48.67
3	LYNDEN CR WWPS	54.26	32.46	39.56	39.44	41.00	39.38
4	MOORE PARK WWPS	31.50	19.65	23.36	23.23	23.23	23.04
5	AGNES ST WWPS	140.00	79.77	106.71	114.96	115.04	130.17
6	KINGHAM HILL WWPS	41.61	13.27	17.98	18.29	18.29	19.09
7	WATER STREET WWPS	20.00	1.96	2.69	2.88	2.88	3.16
8	CEDARBERRY WWPS	7.50	0.68	1.43	1.41	1.41	1.40
9	CARRINGTON PLACE WWPS	22.50	34.24	36.40	36.17	36.17	35.98
10	CHANCERY LANE WWPS	16.70	7.38	7.76	7.70	7.70	7.65
11	ENNISCLARE DRIVE WWPS	6.50	3.85	3.87	3.85	3.85	3.85
12	NINTH LINE WWPS	439.00	905.05	985.32	989.27	1,031.76	1,004.46
13	BEL AIR ESTATES WWPS	3.00	3.12	4.17	4.13	4.13	4.11
14	ARGYLE DRIVE WWPS	15.10	6.14	6.98	6.89	6.89	7.02
15	RAYMAR PLACE WWPS	6.00	1.65	1.69	1.67	1.67	1.70
16	FIRST STREET WWPS	8.20	4.69	4.74	4.72	4.72	4.84
17	GAIRLOCH GARDENS WWPS	7.00	6.89	6.55	6.83	6.83	7.22
18	NAVY STREET WWPS	66.10	35.12	36.95	38.30	38.42	40.61
20	LAKEWOOD DRIVE WWPS	21.00	14.12	14.27	14.23	14.23	14.32
21	WALKER STREET WWPS	43.80	22.52	24.06	24.13	24.19	24.44
23	BIRCH HILL LANE WWPS	38.60	24.78	27.28	27.30	27.30	27.81
24	WESTDALE ROAD WWPS	30.00	20.00	22.09	21.99	21.99	22.15
25	WEST 18 WWPS / LAKESHORE RD	947.00	1,780.59	1,906.39	1,961.22	2,016.26	2,031.70
26	TIMBER LANE WWPS	2.42	2.22	2.24	2.24	2.24	2.30
27	HIXON STREET WWPS	10.00	0.69	1.03	1.02	1.02	1.04
28	BRONTE YACHT CLUB WWPS	2.00	3.30	4.95	4.58	4.58	7.92
29	WEST RIVER WWPS	66.90	75.09	93.76	97.08	97.08	98.80
30	PINEDALE WWPS	260.50	73.36	79.33	79.66	79.66	80.57
31	ELIZABETH GARDENS WWPS	594.00	362.29	454.60	468.19	498.47	473.89
32	LAKESHORE ROAD WWPS (#6)	133.00	80.45	83.61	84.31	84.35	85.85
33	JUNCTION STREET WWPS	104.00	112.87	131.81	133.98	153.10	144.44
34	LAKESHORE ROAD WWPS (#9)	255.00	115.06	119.16	119.65	119.65	121.89
35	LAKESHORE ROAD WWPS(#10)	75.00	132.91	137.92	137.56	137.57	138.07
37	EDGEWATER CRESCENT WWPS	4.00	0.78	0.82	0.82	0.82	0.82
38	SPRING GARDEN ROAD WWPS	6.30	1.47	1.76	1.78	1.78	1.79
40	OAKLAND PARK COURT WWPS	12.00	10.44	10.79	10.70	10.70	10.56
41	DANFORTH PLACE WWPS	4.80	4.64	4.70	4.74	4.74	4.76
42	CHARTWELL ROAD WWPS	6.40	50.49	52.11	53.11	53.11	54.95
43	MORRISON HEIGHTS WWPS	5.00	10.04	10.67	10.60	10.60	10.58
44	CUMNOCK WWPS	7.40	7.85	8.60	8.49	8.49	8.46
45	WEAVER WWPS	25.80	15.02	15.36	15.20	15.20	15.17
46	RIVERSIDE DRIVE WWPS	82.00	30.15	31.71	32.67	32.67	34.44
47	SHEPHERD ROAD WWPS	14.90	11.33	14.16	18.24	18.24	21.23
48	CARDINAL DRIVE WWPS	8.30	6.74	7.08	7.05	7.05	7.09
50	SHELDON CREEK WWPS	32.00	28.17	36.27	35.97	35.97	36.51
52	SHOREWOOD PLACE WWPS	8.00	5.62	6.63	6.63	6.63	6.71
53	LAKEVIEW WWPS	33.20	6.76	8.30	8.16	8.23	8.43
54	LASALLE PARK ROAD WWPS	100.00	62.99	67.85	69.69	70.25	72.05
55	3RD LINE WWPS	1,568.00	991.62	1,336.19	1,359.46	1,378.69	1,417.98
56	STIRLING DRIVE WWPS	18.70	6.71	7.13	7.09	7.09	7.13
57	NORTHSHORE BOULEVARD WWPS	44.20	51.92	52.59	52.39	52.39	52.50
59	LEACHATE STATION 2 WWPS	10.00	0.07	0.07	0.07	0.07	0.07
60	LEACHATE STATION 1 WWPS	10.00	0.07	0.07	0.07	0.07	0.07
62	GARDINER DR WWPS	14.70	3.76	4.02	4.13	4.48	4.33
63	CINDERBARKE TERR WWPS	10.00	2.28	2.55	2.56	2.56	2.59
64	MARINE DRIVE WWPS	108.00	170.87	201.16	206.23	211.62	233.49
65	CORONATION PARK WWPS	12.00	1.78	1.91	1.89	1.89	1.91
66	BELVEDERE WWPS	25.70	17.17	17.24	17.02	17.02	17.06

NOTE: SANITARY SERVICING PER HALTON REGION SANITARY OPERATING MAPS DATED OCTOBER 31, 2018.

LEGEND:

- SITE BOUNDARY
- SANITARY DRAINAGE CATCHMENT BOUNDARY
- 2.94ha SANITARY DRAINAGE CATCHMENT AREA
- SITE AREA
- MEDIUM DENSITY RESIDENTIAL
- INDUSTRIAL

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

3171 LAKESHORE ROAD WEST, OAKVILLE

EXTERNAL SANITARY DRAINAGE PLAN

DESIGNED BY:	N.D.M.	CHECKED BY:	S.M.S.	PROJECT No: 1930	FIGURE No: E.1
SCALE:	1:2000	DATE:	JULY 2019		

File: P:\1930 3171 Lakeshore Road West, Oakville\Drawings\FSP\Fig\Report Figures\1930P-SANI-E.1(C3D).dwg - Revised by <NMCINTOSH> : Mon, Jul 29 2019 - 10:12am

APPENDIX F

WATER DISTRIBUTION ANALYSIS

SCS consulting group ltd

3171 LAKESHORE ROAD WEST DEVELOPMENT

WATER ANALYSIS

PREPARED BY:

MUNICIPAL ENGINEERING SOLUTIONS

Municipal Engineering

Solutions

FOR:

SCS CONSULTING

July 2019

Project Number: 17002-14

TABLE OF CONTENTS

Section 1 – INTRODUCTION

Municipal Engineering Solutions ("MES") was retained by SCS Consulting to conduct a preliminary hydraulic water analysis for the proposed 3171 Lakeshore Road West development located in the Town of Oakville in the Region of Halton. As part of this hydraulic assessment MES was requested to undertake the following:

1. Calculate/verify water demands for the proposed development using Region of Halton, provincial and industry design standards;
2. Add the subject watermains/development to the Region's existing water model;
3. Run the model to size the subject mains to achieve service criteria during Average Day, Peak Hour and fire flow during Maximum Day demand; and
4. Prepare a Report summarizing the modeling results for agency review and design purposes.

1.1 Development Background

The 3171 Lakeshore Road West development is located at 3171 Lakeshore Road West East between King Road and Waterdown Road in the Town of Oakville. The development consists of 8 semi-detached and 27 townhomes to be constructed on existing commercial lands. All but 3 townhomes will be condominiums. Units 9, 10 & 11 will be freehold units. The proposed development is shown below on **Figure 1**.

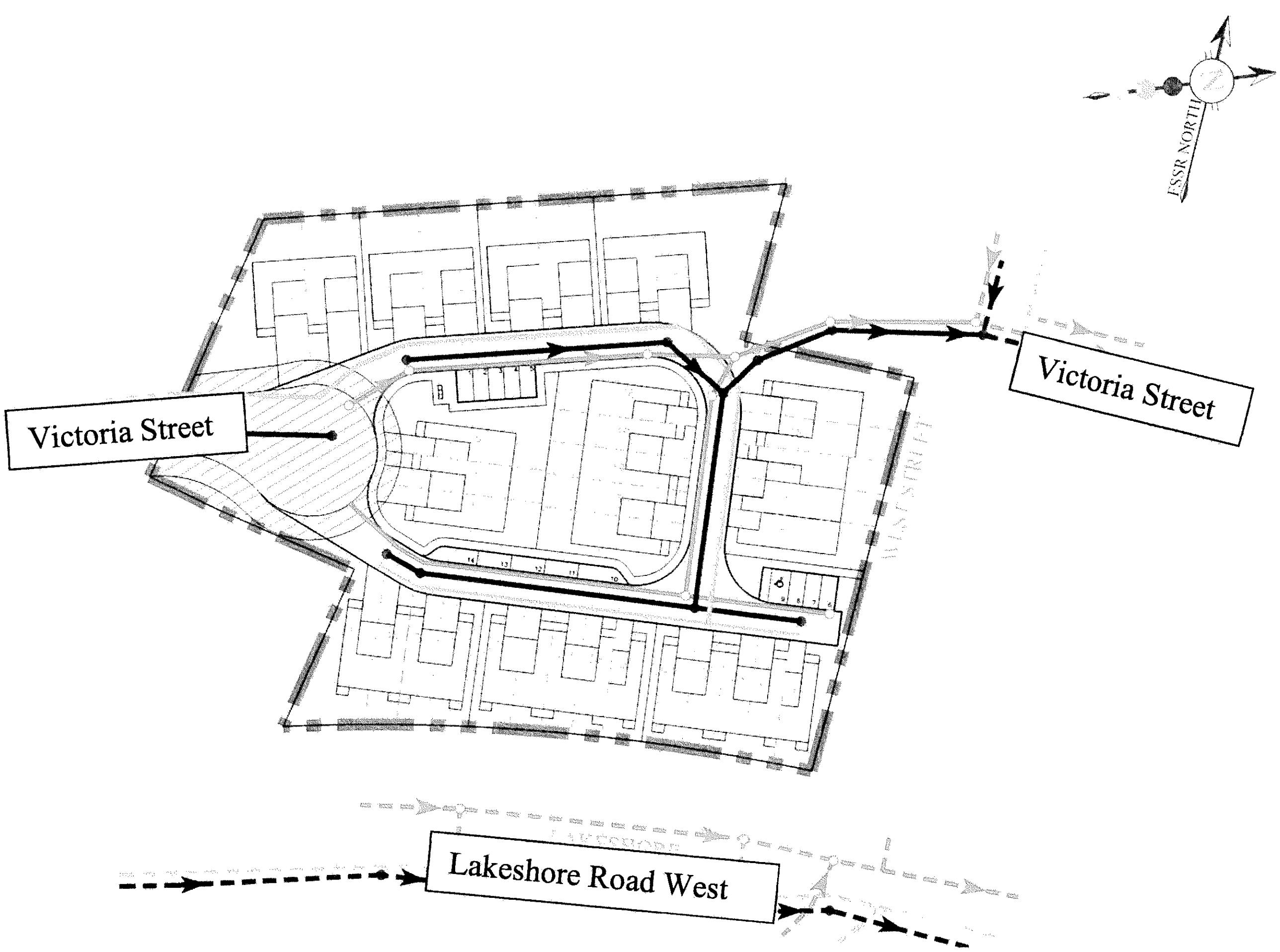


Figure 1 - Proposed 3171 Lakeshore Road West Development

Section 2 – WATERMAIN DESIGN CRITERIA

The design criteria utilized to estimate the water demands for the hydraulic water model follows general industry standards and is calculated using the design criteria and guidelines outlined in the Region of Halton's 2019 Water and Wastewater Linear Design Manual, the Ministry of the Environment, Conservation and Parks (MECP) Watermain Design Criteria, and the Fire Underwriters Survey.

The following sections summarize the specific design criteria used to carry out the hydraulic watermain assessment for this development.

2.1 Equivalent Population Densities & Water Design Factors

To calculate the equivalent population and water design factors for this development MES used Region of Halton standard population densities as noted in the *"Region of Halton Water and Wastewater Linear Design Manual, April 2019"*. **Table 1** summarizes the population densities and **Table 2** summarizes the average daily demand and peaking factors used for this analysis.

Table 1 – Equivalent Population Density

Type of Development	Equivalent Population (Persons/Ha)	Equivalent Population (Persons/Unit)
Single Family	55	3.52
Semi-Detached	100	3.52
Townhouse	135	2.664
Apartment	285	1.579
Light Commercial	90	
Community Services	40	

Source: Region of Halton Water and Wastewater Linear Design Manual, April 2019; 2017 Development Charges Update, October 2016

Table 2 - Water Design Factors

Type of Development	Average Daily Demand (m^3 per capita)	Maximum Daily Demand Peaking Factor	Peak Hourly Demand Peaking Factor
Residential	0.275	2.25	4.00
Industrial	0.275	2.25	2.25
Commercial	0.275	2.25	2.25
Community Services	0.275	2.25	2.25

Source: Region of Halton Water and Wastewater Linear Design Manual, April 2019

Section 3 –FLOW DEMANDS

Utilizing the equivalent population data from Table 1 and the corresponding Average Day, Maximum Day and Peak Hour data from Table 2 the water demands for this development were calculated.

3.1 Equivalent Population Flow Demands

The calculated demands for the supply watermain to the development are summarized in **Table 3.**

Table 3 – Water Demand for 3171 Lakeshore Road West Development

Development	Average Day Demand (L/S)	Maximum Day Demand (L/S)	Peak Hour Demand (L/S)
3171 Lakeshore Road West	0.32	0.72	1.27

3.2 Fire Flow Demands

The Region criteria requests that fire demands be calculated using the Fire Underwriter's Survey where possible. As this is a preliminary analysis and the specifics of the proposed development are unknown, the fire flows suggested in **Table 4** have been used for this assessment. Once building designs/configurations are known, the fire flows must be confirmed using the FUS formula. Building construction and sprinkler systems may need to be designed to suit the available flow and pressure. For details on the fire flow demands for each node used to model see **Appendix A**.

Table 4 - Typical Fire Flow Requirements

Type of Development	Fire Flow (L/S)
Residential Low-Density	91
Residential Medium-Density	136
Residential High-Density	273
Commercial	273
Institutional	273
Industrial	273

Source: Region of Halton Development Coordinator Fire Flow Certification, 2003

3.2 External Demands

The Region of Halton InfoWater models that was provided by the Region to MES included water demands for existing and known future developments within the Region.

Section 4 – OTHER SYSTEM REQUIREMENTS

4.1 System Pressure Requirements

In addition to meeting the various flow requirements, the system must also satisfy minimum and maximum pressure requirements as outlined by the Region of Halton. The Region's pressure requirements are outlined in the Water and Wastewater Linear Design Manual and stipulate the following:

1. The water system shall be designed to maintain as close as possible to a maximum working pressure of 690 kPa (100 psi) as a best management practice.
2. The minimum system pressure shall not be less than 140 kPa (20 psi) at any point in the water system under fire flow conditions.
3. Under normal operating conditions, the water system shall have a target minimum static pressure of 310 kPa. (45 psi). Under no operating conditions shall the static pressure within a distribution main fall below 275 kPa (40 psi).
4. The normal method of reduction of pressures to comply with the Ontario Building Code (reduction of pressures to 550 kPa, 80 psi) is by pressure reducing valves to be installed on individual services.

4.2 Watermain Sizing

The Region of Halton also stipulates minimum pipe sizes and requires that all watermains are adequately sized to maintain demand flows at the required pressures without causing excessive energy loss or result in water quality decay. The watermain system must therefore be designed to accommodate the greater of the following:

- Maximum day plus fire demand
- Peak hour demand

The minimum pipe size for commercial and industrial areas shall be 300 mm diameter and for residential areas the minimum pipe size shall be 150 mm diameter. For distribution systems providing fire protection the minimum pipe size shall be 150 mm diameter in accordance with Ministry of the Environment, Conservation and Parks (MECP) and NFPA requirements.

To provide appropriate fire protection, reliable supply and pressures the water distribution system should be looped wherever possible to improve supply security and water quality.

4.3 Watermain C-Factor

In designing and modeling of the pipes the Coefficient of Roughness (C-Factor) factors from the Region's design manual and as suggested by the MECP were utilized. The Coefficient of Roughness assigned to each pipe size in summarized in **Table 5** below.

Table 5 - Hazen-Williams Coefficient of Roughness (C-Factors)

Size of Pipe (Diameter in mm)	Coefficient of Roughness (C)
150 mm	100
200 mm to 250 mm	110
300 mm to 600 mm	120
Greater Than 600 mm	130

Source: Region of Halton Water and Wastewater Linear Design Manual, April 2019

Section 5 – ANALYSIS & MODELING RESULTS

In order to conduct the preliminary hydraulic water analysis for the proposed development the water demands were estimated by MES using the design criteria previously discussed and incorporated the demands into the existing Region of Halton InfoWater models (July 2018) which was provided by the Region in July 2018 and confirmed as the most recent in February 2019. The following sections discusses the model setup and results.

5.1 Model Setup

The 3171 Lakeshore Road West development is located within pressure zone O1 (TWL 140.2 m). Zone O1 is supplied by Oakville WPP with storage in the Kitchen Reservoir. The O1 zone services elevations from 76m to 100.7 m. The development site elevations are within this service range and therefore should be adequately supplied by Zone O1.

The Region provided 2 models, one with the existing pressure districts; the other with the revised proposed pressure districts. The development was modeled under the 2016 planning scenario in the existing pressure district model and under the 2031 planning scenario in the future pressure district model to confirm that the development could be serviced adequately under existing and ultimate conditions.

New nodes were created to add the flow demands and service elevation information from the development to the Region of Halton's Infowater hydraulic water distribution models and the system analysis was carried out. Friction factor for the pipes were assigned according to Table 5.

As the fire flow requirements are not yet known, a meter was not included in the modeling. The size of the meter will need to be determined when additional information is known about the unit construction to complete the FUS calculation.

5.2 Watermain Sizing and System Pressures

The analysis was conducted under 2016 with the existing zone boundaries and 2031 servicing conditions with the future zone boundaries for Average Day, Maximum Day, Peak Hour and Maximum day plus Fire demands to size the watermains and meet the pressure requirements. The pipe size and layout are shown in **Appendix B**.

The pipes in the surrounding existing developments supplying both ends of Victoria Street are all 150 mm diameter. These watermains are not capable of supplying the necessary fire flow for townhomes into the site. There are three options to bring additional flow to the area:

- Replacing the existing 150 mm along West Street to the existing 200 mm along Riverview Street;
- Replacing the existing 150 mm along Victoria Street from the dead end to the existing 200 mm at Willard Street;
- Connecting to the 300 mm along Lakeshore Road West.

To minimize the disruption to the existing homes, the Lakeshore Road West connection was considered the preferred option. To adequately supply the proposed townhome fire flows, the development watermains will be supplied from this 300 mm pipe. It should be noted that the fire flow demands are based on Halton Region's Fire Flow Certification and ,while the available flow is in excess of the required, when additional information of the home construction is available, the fire flows must be confirmed.

Three units of the development will have the domestic demands supplied from Victoria Street. A 50 mm loop will be constructed to supply units 9, 10, and 11. Fire flow can be provided to all units within the proposed development by the new hydrants within the development. Since three townhouse units will be freehold and not part of the condominium block the fire flows for the existing nearby hydrants, including the one located at the corner of Victoria Street and Speyside Drive, which could be utilized during a fire event, are included in the output.

The watermains were sized at 50 to 200 mm according to the results of average day, maximum day, maximum day plus fire, and peak hour scenarios, as shown in the schematics included in **Appendix B**. Modeled service pressures to the development are summarized in **Table 6**.

Detailed pipe and node tables for the various scenarios modelled are attached to this report in **Appendix B**.

Table 6 - Modeled Service Pressures

Scenario	Average Day	Maximum Day	Peak Hour	Max. Day + Fire
2016	72.6 to 74.6 psi (501 to 514 kPa)	75.3 to 77.4 psi (519 to 533 kPa)	72.9 to 74.9 psi (503 to 516 kPa)	163 to 487 L/s@ 20 psi
2031	69.8 to 71.9 psi (481 to 496 kPa)	71.0 to 73.1 psi (489 to 504 kPa)	69.8 to 71.8 psi (481 to 495 kPa)	183 to 553 L/s@ 20 psi

Section 6 – CONCLUSIONS/RECOMMEDATIONS

The proposed watermain for the 3171 Lakeshore Road West development can achieve hydraulic requirements as prescribed by the Region of Halton watermain design criteria as summarized below.

- The service pressures from the watermain are expected to range between 72.6 psi to 77.4 psi (501 kPa to 533 kPa) under 2016 conditions and between 69.8 psi to 73.1 psi (481 kPa to 504 kPa) under 2031 conditions.
- The proposed fire hydrants located at the dead end sections of the 200 mm watermain (i.e. at Units 1 and 35) must be located close enough to provide fire protection to all units within the development. If not, the proposed 200 mm watermains will need to be extended into the Victoria Street cul-de-sac and connected to the 150 mm

on Victoria Street. Fire flow protection for the three freehold townhouse units can be supplied by the internal condominium hydrants and/or an existing/proposed hydrant along the existing 150 mm watermain on Victoria Street (at the appropriate location) provided the final required FUS fire flow does not exceed the available flow along Victoria Street currently estimated by the model.

- The available fire flow meets or exceeds the preliminary fire flow demands utilized for this assessment at the minimum pressure of 140 kPa based on the proposed watermain supply and assumptions made within this report but should be confirmed when additional information becomes available. Should higher fire flows be required once the FUS calculations are completed and should those fire flows exceed the expected values available as identified by the model, an additional water supply will be required and/or the watermain system and building design will need to be adjusted to suit the fire flows ultimately available.
- The size of the meter will need to be determined when additional information is known about the unit construction to complete the FUS calculation. Once the meter has been sized and selected it will need to be added to the model and the modeling results will need to be updated to reflect the addition of the meter.
- As the development design is at the initial stages, the findings and recommendations in this report are considered preliminary. As more information is available on the watermain and building layout, all building assumptions and the fire flow demands must be reviewed and updated to size the water meter and confirm that the water supply is adequate.
- This report, including all modeling assumptions used, is to be submitted to and reviewed by the water operating authority (municipality) to confirm that the modeling parameters used are acceptable to the operating authority and/or confirm if modified domestic or fire flow requirements are required or should be implemented for this particular development.

Appendix A

Demands

Halton Design Criteria

Water & Wastewater Linear Design Manual, April 2019 Version 4 (unless otherwise stated)

Equivalent Population by Unit

(2017 Development Charges Update, October 2016)

Type of Development	Equivalent Population Density
	(Person/Unit)
Single Family or Semi-Detached	3.5
Townhouse	2.7
Apartment	1.6

Equivalent Population by Area

Type of Development	Equivalent Population Density	Average Day Demands
	(Person/Hectare)	*(m3/ha/day)*
Single Family	55	15.13
Semi-detached duplex and 4-plex	100	27.50
Townhouse, Maisonette (<6 stories)	135	37.13
Apartments (>6 stories)	285	78.38
Light Commercial Areas	90	24.75
Community Services	40	11.00
Light Industrial Areas	125	34.38
Hospitals (persons/bed)	4	

Water Design Factors

Average Daily Demand (m3/capita)	0.275
Maximum Daily Demand P.F.	2.25
Maximum Hourly Demand P.F.	
Residential	*4*
I/C/I	*2.25*

Cofficient of Roughness

Size of Pipe (mm Dia.)	Coefficient of Roughness (C)
150	100
200-250	110
300-600	120
Over 600	130

Minimum Pipe Size

Type of Development	Size of Pipe (mm Dia.)
Residential	150
Commercial/Industrial/Community	300

Working Pressures

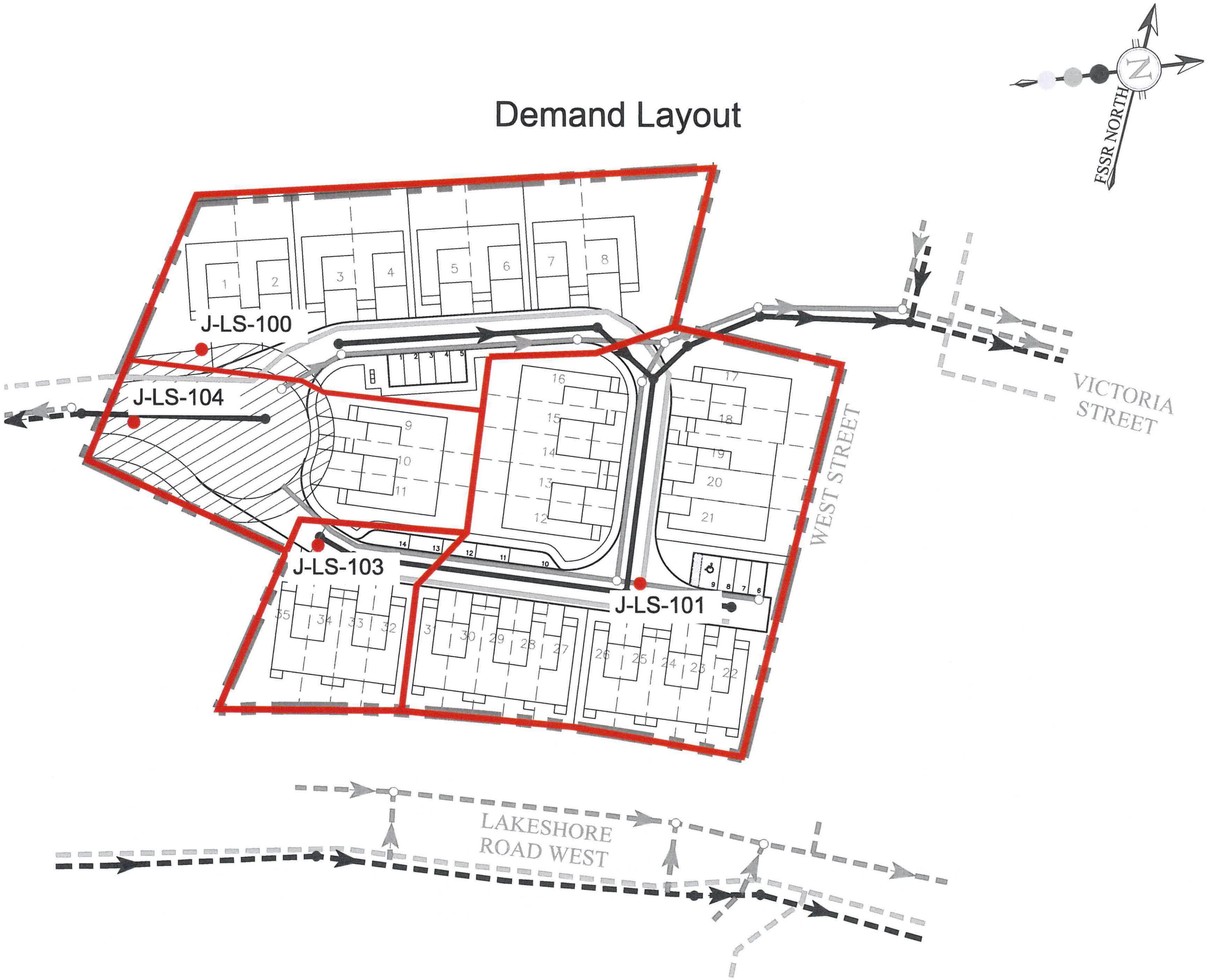
Parameter	Pressure
Normal Condition	
Minimum Pressure	275 kPa (40 psi)
Target Pressure	310 kPa (45 psi)
Maximum (Building Code)	550 kPa (80 psi)
Maximum (Halton)	690 kPa (100 psi)
Fire Flow Conditions	
Minimum Pressure	140 kPa (20 psi)

Water Demand
3171 Lakeshore Development, Oakville On
July 29, 2019

Municipal Engineering Solutions

Node	Phase	Elevation	Type of Development							Equivalent Population		Demands			Fire Flow Demands
			Single Family	Semi-Detached	Townhouse	Apartment	Commercial	Schools	Industrial	Total Population	Total Population	ADD	MDD	PHD	
		(m)	*(units)*	*(units)*	*(units)*	*(units)*	*(ha)*	*(ha)*	*(ha)*	*(Residential)*	*(ICI)*	*(L/s)*	*(L/s)*	*(L/s)*	*(L/s)*
J-LS-100		85.10		*8*						28	0	0.09	0.20	0.36	136
J-LS-101		84.80			*20*					53	0	0.17	0.38	0.68	
J-LS-102		84.73								0	0	0.00	0.00	0.00	
J-LS-103		85.11			*4*					11	0	0.03	0.08	0.14	136
J-LS-104		85.20			*3*					8	0	0.03	0.06	0.10	
WFT10235		83.46								0	0	0.00	0.00	0.00	136
WFT17860		85.08								0	0	0.00	0.00	0.00	136
WFT27583		84.57								0	0	0.00	0.00	0.00	136
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
										0	0	0.00	0.00	0.00	
Total			**0**	**8**	**27**	**0**	**0.00**	**0.00**	**0**	**100**	**0**	**0.32**	**0.72**	**1.27**	

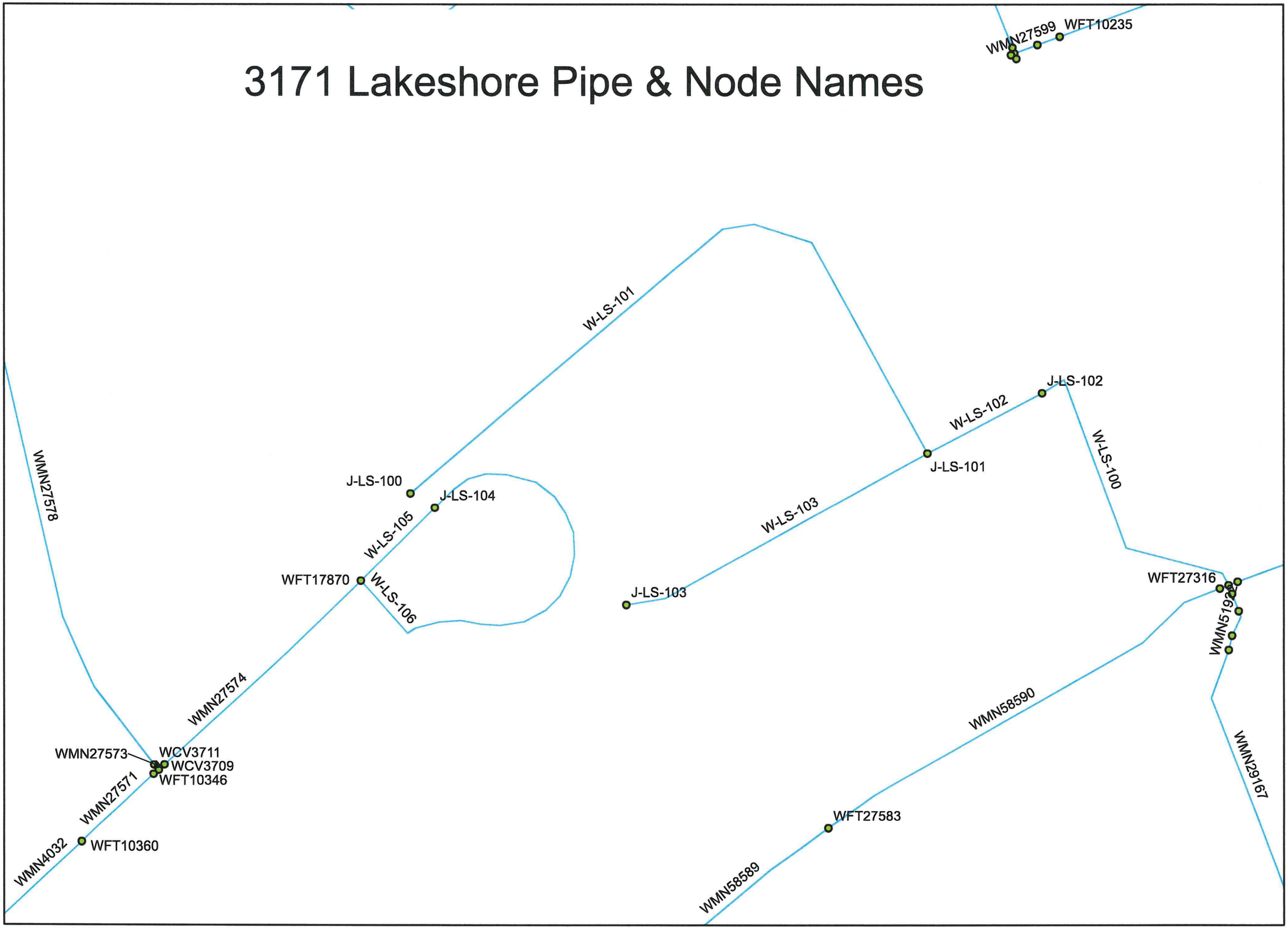
Demand Layout

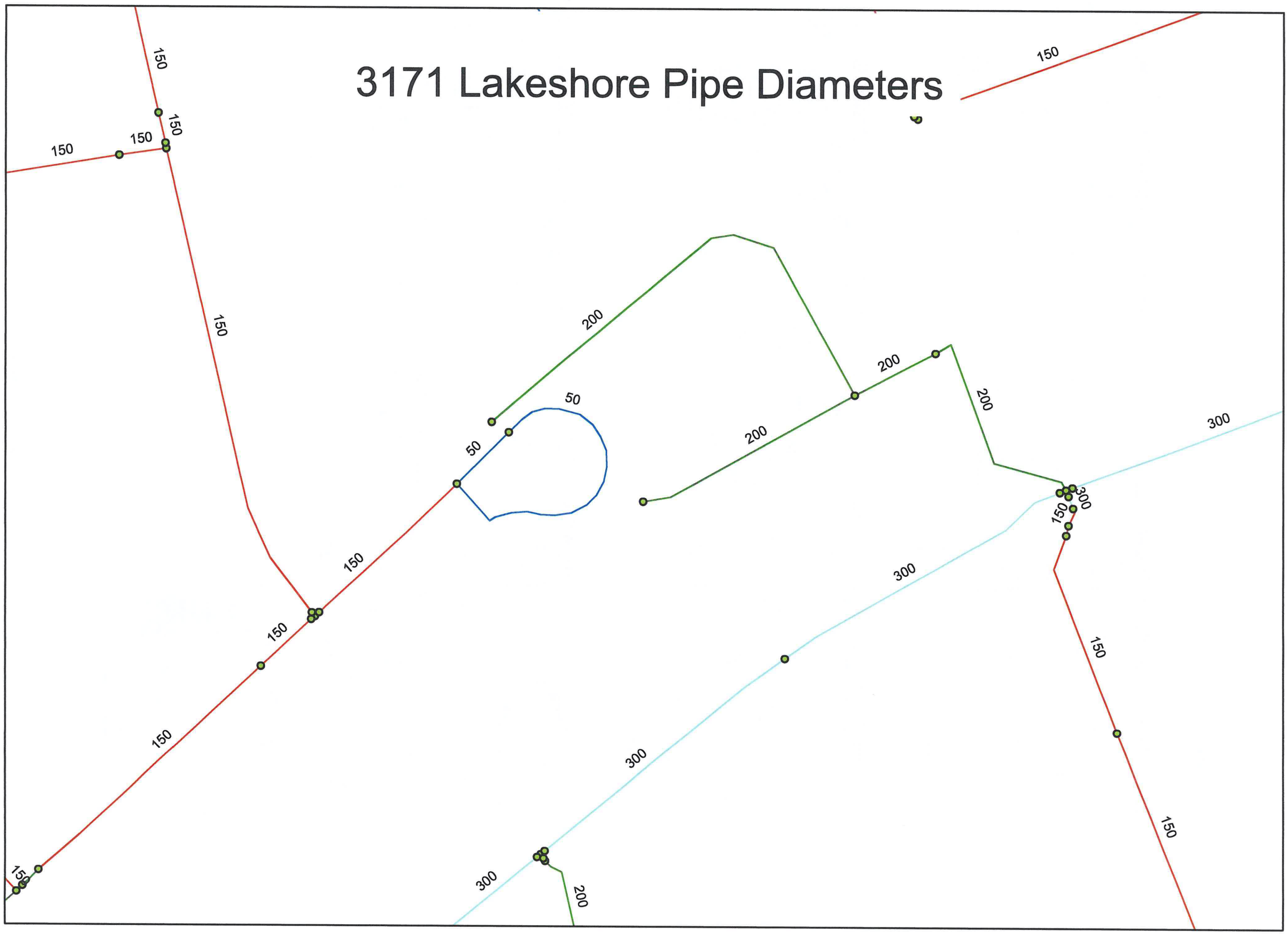


Appendix B

Model Results

3171 Lakeshore Pipe & Node Names




3171 Lakeshore Pipe Diameters
150
150
150
150
150
150
150
150
150
150
150
150
150
50
50
200
200
200
200
200
300
300
300
300
300
300

2016 Conditions
3171 Lakeshore Development, Oakville On
July 29, 2019

Municipal Engineering Solutions

Average Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.09	85.10	136.18	72.61
J-LS-101	0.17	84.80	136.18	73.04
J-LS-102	0.00	84.73	136.18	73.14
J-LS-103	0.03	85.11	136.18	72.60
J-LS-104	0.03	85.20	136.24	72.56
WCV3709	0.02	84.60	136.24	73.42
WFT10346	0.01	84.45	136.24	73.63
WFT17870	0.02	85.08	136.24	72.73
WFT27316	0.01	83.70	136.18	74.61
WFT27583	0.04	84.57	136.19	73.39
MIN		**83.70**		**72.56**
MAX		**85.20**		**74.61**

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (ML/d)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-0.03	0.01
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.01	0.00
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-0.03	0.01
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.00	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.00	0.01
W-LS-106	J-LS-104	WFT17870	83.57	50	100	0.00	0.00
WMN27573	WFT10346	WCV3709	1.41	150	100	0.01	0.00
WMN27574	WCV3709	WFT17870	46.96	150	100	0.00	0.00

Maximum Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.20	85.10	138.11	75.36
J-LS-101	0.38	84.80	138.11	75.79
J-LS-102	0.00	84.73	138.11	75.89
J-LS-103	0.08	85.11	138.11	75.35
J-LS-104	0.06	85.20	138.22	75.37
WCV3709	0.04	84.60	138.22	76.23
WFT10346	0.01	84.45	138.22	76.45
WFT17870	0.04	85.08	138.22	75.55
WFT27316	0.01	83.70	138.11	77.35
WFT27583	0.08	84.57	138.14	76.15
MIN		**83.70**		**75.35**
MAX		**85.20**		**77.35**

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (ML/d)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-0.06	0.02
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.02	0.01
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-0.06	0.02
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.01	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.00	0.02
W-LS-106	J-LS-104	WFT17870	83.57	50	100	0.00	0.01
WMN27573	WFT10346	WCV3709	1.41	150	100	0.01	0.01
WMN27574	WCV3709	WFT17870	46.96	150	100	0.01	0.01

Peak Hour

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.36	85.10	136.39	72.92
J-LS-101	0.68	84.80	136.39	73.35
J-LS-102	0.00	84.73	136.39	73.45
J-LS-103	0.14	85.11	136.39	72.90
J-LS-104	0.10	85.20	136.52	72.96
WCV3709	0.07	84.60	136.52	73.82
WFT10346	0.02	84.45	136.52	74.03
WFT17870	0.07	85.08	136.52	73.13
WFT27316	0.02	83.70	136.40	74.91
WFT27583	0.13	84.57	136.42	73.71
MIN		**83.70**		**72.90**
MAX		**85.20**		**74.91**

Pipe Table

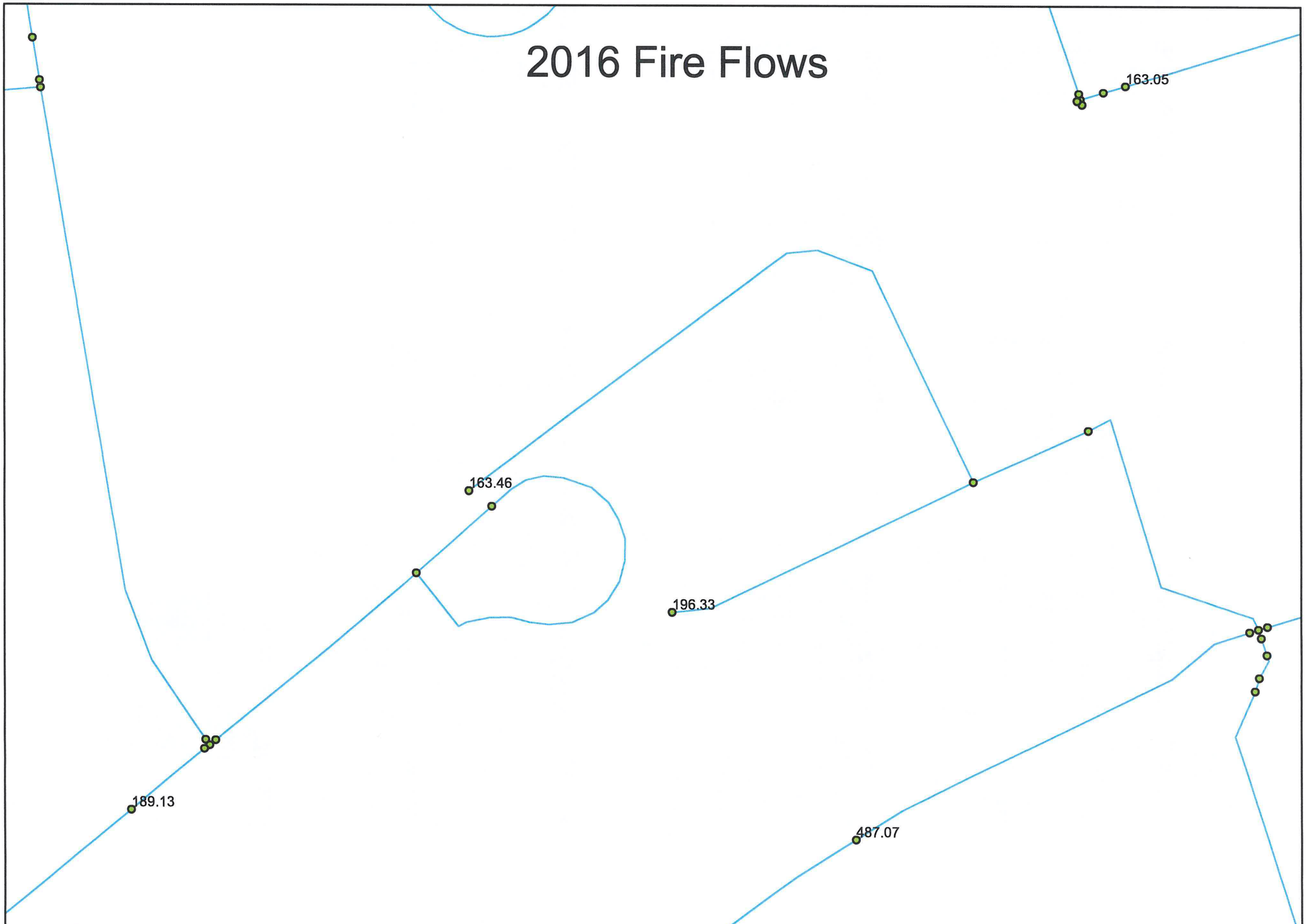
ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (ML/d)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-0.10	0.04
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.03	0.01
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-0.10	0.04
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.01	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.01	0.04
W-LS-106	J-LS-104	WFT17870	83.57	50	100	0.00	0.02
WMN27573	WFT10346	WCV3709	1.41	150	100	0.02	0.01
WMN27574	WCV3709	WFT17870	46.96	150	100	0.01	0.01

Fire Flow Table

ID	Total Demand (L/s)	Available Flow (L/s)	Fire Flow Met?
J-LS-100	136.20	163.46	TRUE
J-LS-103	136.08	196.33	TRUE
WFT10235	136.07	163.05	TRUE
WFT10360	136.07	189.13	TRUE
WFT27583	136.08	487.07	TRUE

MIN	**163.05**
MAX	**487.07**

2016 Fire Flows



2031 Conditions
3171 Lakeshore Development, Oakville On
July 29, 2019

Municipal Engineering Solutions

Average Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.09	85.10	134.26	69.89
J-LS-101	0.17	84.80	134.26	70.31
J-LS-102	0.00	84.73	134.26	70.41
J-LS-103	0.03	85.11	134.26	69.87
J-LS-104	0.03	85.20	134.28	69.78
WCV3709	0.02	84.60	134.28	70.63
WFT10346	0.01	84.45	134.28	70.85
WFT17870	0.02	85.08	134.28	69.95
WFT27316	0.01	83.70	134.26	71.88
WFT27583	0.04	84.57	134.27	70.65

MIN		83.70		69.78
MAX		85.20		71.88

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-0.29	0.01
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.09	0.00
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-0.29	0.01
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.03	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.02	0.01
W-LS-106	J-LS-104	WFT17870	83.57	50	100	-0.01	0.00
WMN27573	WFT10346	WCV3709	1.41	150	100	0.07	0.00
WMN27574	WCV3709	WFT17870	46.96	150	100	0.05	0.00

Maximum Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.20	85.10	135.10	71.07
J-LS-101	0.38	84.80	135.10	71.50
J-LS-102	0.00	84.73	135.10	71.60
J-LS-103	0.08	85.11	135.10	71.06
J-LS-104	0.06	85.20	135.15	71.01
WCV3709	0.02	84.60	135.15	71.86
WFT10346	0.01	84.45	135.15	72.08
WFT17870	0.02	85.08	135.15	71.18
WFT27316	0.01	83.70	135.10	73.07
WFT27583	0.04	84.57	135.11	71.85

MIN		83.70		71.01
MAX		85.20		73.07

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-0.66	0.02
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.20	0.01
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-0.66	0.02
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.08	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.04	0.02
W-LS-106	J-LS-104	WFT17870	83.57	50	100	-0.02	0.01
WMN27573	WFT10346	WCV3709	1.41	150	100	0.10	0.01
WMN27574	WCV3709	WFT17870	46.96	150	100	0.08	0.00

Peak Hour

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (psi)
J-LS-100	0.36	85.10	134.23	69.84
J-LS-101	0.68	84.80	134.23	70.27
J-LS-102	0.00	84.73	134.23	70.37
J-LS-103	0.14	85.11	134.23	69.83
J-LS-104	0.10	85.20	134.34	69.85
WCV3709	0.06	84.60	134.34	70.71
WFT10346	0.02	84.45	134.34	70.93
WFT17870	0.07	85.08	134.34	70.03
WFT27316	0.02	83.70	134.23	71.84
WFT27583	0.13	84.57	134.25	70.62

MIN		83.70		69.83
MAX		85.20		71.84

Pipe Table

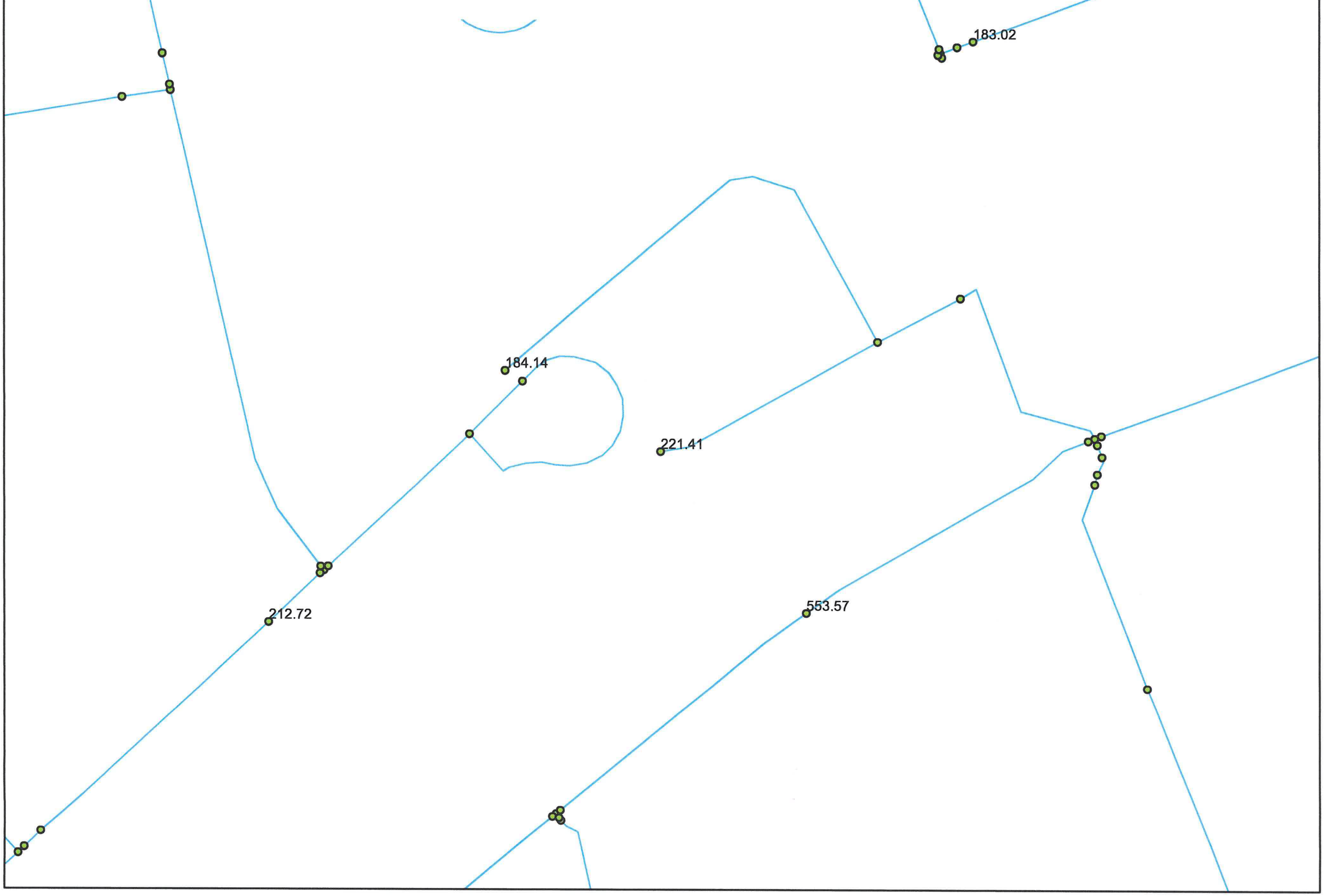
ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
W-LS-100	J-LS-102	WFT27316	55.47	200	110	-1.18	0.04
W-LS-101	J-LS-100	J-LS-101	129.50	200	110	-0.36	0.01
W-LS-102	J-LS-101	J-LS-102	22.63	200	110	-1.18	0.04
W-LS-103	J-LS-101	J-LS-103	59.36	200	110	0.14	0.00
W-LS-105	WFT17870	J-LS-104	18.15	50	100	0.07	0.04
W-LS-106	J-LS-104	WFT17870	83.57	50	100	-0.03	0.02
WMN27573	WFT10346	WCV3709	1.41	150	100	0.23	0.01
WMN27574	WCV3709	WFT17870	46.96	150	100	0.17	0.01

Fire Flow Table

ID	Total Demand (L/s)	Available Flow (L/s)	Fire Flow Met?
J-LS-100	136.20	184.14	TRUE
J-LS-103	136.08	221.41	TRUE
WFT10235	136.07	183.02	TRUE
WFT10360	136.07	212.72	TRUE
WFT27583	136.04	553.57	TRUE

MIN	183.02
MAX	553.57

2031 Fire Flows



SCS Consulting Group Ltd
30 Centurian Drive, Suite 100
Markham, ON, L3R 8B8
Phone 905 475 1900
Fax 905 475 8335