WEST VILLAGE (PORT CREDIT)

PORT CREDIT WEST VILLAGE PARTNERS INC.

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

City of Mississauga
70 Mississauga Road South / 181 Lakeshore Road West
Project Number 16-489
August 25, 2017

urbantech

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FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

This report provides functional servicing design and stormwater management information in support of proposed Official Plan and Zoning By-Law Amendment applications and Draft Plan of Subdivision for the subject lands. This report fulfils DARC 17-201 W1 submission requirements related to grading, servicing, drainage, stormwater management and LID measures. The servicing and development strategies presented in this report have been developed in conjunction with the greater consulting team and should be considered in conjunction with their work. The following studies are included in the appendices:

- *Supplementary Geotechnical Report - Stantec*
- *Watermain Hydraulic Modelling Analysis - AECOM*

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INTRODUCTION

Urbantech Consulting has been retained by Port Credit West Village Partners Inc. to prepare a preliminary engineering design and functional servicing report for the former Imperial Oil property located at 70 Mississauga Road South and 181 Lakeshore Road West in the City of Mississauga, Region of Peel.

This report is applicable to any future revisions to the Draft/Concept Plan, assuming the revisions are in general conformance with the land use, servicing and stormwater management concepts outlined herein. The design information presented in this report considers the following guidelines:

- *City of Mississauga Engineering Standard Drawings Manual*
- *Credit Valley Conservation Authority Stormwater Management Criteria Document (August 2012)*
- *Draft Ministry of the Environment and Climate Change LID SWM Guidance Manual (2017)*
- *Regional Municipality of Peel PW Design Specifications and Procedures*
- *Stormwater Management Planning and Design Manual by the Ministry of Environment and Climate Change; (March 2003)*

The subject property is approximately 29 hectares (72.0 acres) and is located in the City of Mississauga. The site was formerly used by Imperial Oil for refinery and other industrial uses (including a brickworks facility). Currently, the site is generally covered in low lying vegetation and some remnant roads, parking areas, a former service building and remnants of a gas service station. The site is bounded by:

- Lakeshore Road West to the north;
- Mississauga Road to the east;
- A strip of waterfront land to the south (not subject to this applications); and
- Existing residential lands with frontages on Pine Avenue to the west.

Figure 1 illustrates the location of the site. The legal description of the site is All of Lot 10, Part of Lots 9 and 11 and Water Lot Location in Front of Lot 9, Broken Front Range, Credit Indian Reserve (Geographic Township of Toronto), in the City of Mississauga, Regional Municipality of Peel.

The strip of waterfront lands abutting Lake Ontario are not part of this application.

The proposed development will proceed under an Official Plan Amendment, Rezoning and Plan of Subdivision processes. Subsequent site plan applications for the private blocks will be submitted once the process is further advanced.

SITE STATS

Location:
Lakeshore Road West & Mississauga Road

Existing Site / Drainage Area:
Approx. 29 ha

Subwatershed:
Credit River / Lake Ontario

Owner:
Port Credit West Village Partners Inc.

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EXISTING CONDITIONS

Land Use & Topography

The majority of the site is covered in vegetation with some areas of asphalt/concrete and remnants of the former industrial use. There is an existing shale pond located in the southern portion of the site which was the former extraction pit for the brickworks and then functioned as a stormwater management pond during oil refinery operations. Throughout the site there are multiple monitoring wells used to monitor the environmental conditions / quality of the groundwater.

A topographical survey of the subject lands was completed by JD Barnes in February of 2017. The site generally falls from Lakeshore Road to Lake Ontario with a maximum grade change of approximately 7m. Along the western boundary, an existing 3m high berm separates the rear yards of the existing residences on Pine Avenue South from the subject lands. The average slope from Lakeshore Road to the south property limit is approximately 1.5%.

Figure 3 illustrates the existing site features, topography and drainage patterns.

Shoreline

Lands adjacent to Lake Ontario are regulated by the Credit Valley Conservation Authority. Limits of the Regulated Area are shown on **Drawing GR-1**. The development will require new storm sewers discharging directly to Lake Ontario. All works within the regulated area will include appropriate shoreline protection, restoration and ESC measures required. CVCA permits will be required.

The waterfront lands directly to the south of the site adjacent to Lake Ontario are owned by the Crown and not subject to this application. Discussions with the City related to the shoreline will be held after the first submission is filed.

Soil Conditions

Stantec Consulting has been retained by West Village Partners LP to investigate the geotechnical conditions of the site. At this time a detailed geotechnical report is not available. Stantec has provided a summary letter "Supplementary Geotechnical Conditions – Preliminary Design, Imperial Oil Lands" (March 9, 2017) that provides general geotechnical site conditions. The letter prepared by Stantec regarding the background geology states that the site is located in the Iroquois Plain and that the soil stratigraphy in this area is generally characterized by clay till overlain by sand. Underlying bedrock comprises shale and limestone of the Georgian Bay Formation.

Numerous Environmental Site Assessments (ESAs) have been carried out on the subject lands. The letter summarizes geotechnical information that can be inferred from the boreholes and test pits carried out in these ESAs. The letter summarizes the findings as follows:

- The overburden consisted predominantly of brown and grey sandy silt with silty clay/clayey silt layers and localized (discontinuous) sand layers.
- The overburden was underlain by weathered shale bedrock.
- Depth of bedrock ranged from 0.7m to 11m below existing grade and certain areas may require rock-breaking equipment for excavation.

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Stantec (on behalf of Port Credit West Village Partners Inc.) has prepared a detailed environmental remediation program to be undertaken on site. This program will consist primarily of conventional excavation and disposal of impacted materials at approved facilities and the completion of Risk Assessments, as per Ontario Regulation 153/04, as amended. A significant quantity of the existing soils will be removed, which provides opportunities to construct the site with engineered fill suitable for construction and for low-impact development stormwater management measures / restoration.

Groundwater was encountered in both the overburden and the bedrock:

- Average depth of 1.8 m below existing grade, with a maximum depth of 6.8 m below existing grade in the overburden.
- Average of 3.8 m below existing grade; maximum depth of 11.4 m below existing grade in the bedrock.

Please refer to Appendix A for further information.

Existing Drainage

Drainage from the existing site is generally north to south, towards the lake. The majority of the site drainage is intercepted by the existing Shale Pond on the subject lands.

In terms of external drainage, Lakeshore Road West is urbanized and drains via storm sewers to the existing Mississauga Road storm sewer system. A 1050mm diameter storm sewer on Mississauga Road collects drainage from Lakeshore Road West and the existing developments east of Mississauga Road (approximately 13.65 ha). This sewer extends beneath the waterfront trail and discharges to the lake via a headwall.

Refer to Figure 3 for the existing site drainage.

SOILS

Topsoil Depth:

Varies

Predominant Soils:

Clay till / sand & Bedrock (0.7m -11.0m below ground)

Groundwater depth:

0.3m – 6.8m (overburden)

3.8m – 11.4m (bedrock)

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DEVELOPMENT CONCEPT

Draft Plan

As shown on **Figure 2**, the proposed 29.0 ha development consists of several public right-of-ways and private site plan blocks, including:

- Mixed use blocks including campus
- High density residential blocks
- A commercial development block
- Park blocks / Open space
- Public ROWs

The proposed development will be advanced through both Draft Plan of Subdivision approval process and the Site Plan approval process for the individual private site plan blocks. The Subdivision components will consist of the public ROW areas, open space blocks, and services. At the time of this report the proposed Right of Way cross sections have not been finalized. The intent is to create urban cross sections in consultation with the required approval agencies and utility companies, and in keeping with the developing master plan vision. Detailed cross sections will be provided in subsequent versions of this report.

Refer to Figure 4 – Proposed Draft Plan

Conceptual Development Phasing

Currently the project is proposed to be developed in 5 phases. Servicing infrastructure is designed to facilitate the proposed phasing and provide flexibility should the phasing be altered. The current phasing is based on the anticipated development schedule provided by Port Credit West Village Partners Inc. and may change through the approval process

External servicing works are required for the proposed development to proceed. These include a new sanitary sewer from the development lands to the existing Front Street Sanitary Pump Station (SPS), a new storm sewer within Mississauga Road to Lake Ontario and a storm outfall from the southwest limits of the site to Lake Ontario.

Refer to Figure 5 – Proposed Conceptual Phasing Plan

GRADING

The proposed conceptual grading for the development will be designed in accordance with City of Mississauga standards. Grading is generally governed by the existing boundary conditions. Site grading has also been designed to ensure that adequate cover over proposed services is maintained. No external grading works are proposed.

A preliminary grading concept plan has been prepared for the subject lands based on the following engineering constraints:

- Storm outlet elevations
- Major system drainage paths
- Provision of minimum cover over services
- Proposed road patterns and land use
- Elevations along boundary roads, property lines and waterfront trail
- Application of the City of Mississauga standards

The grading plans are consistent with the City standards. In general, grading of all proposed roads and site plan blocks adjacent to the surrounding development and roads matches the existing grades or the ultimate anticipated grades at the property line, as appropriate.

As noted in the preceding section, a considerable amount of soil will be removed from the lands as part of an environmental remediation program. The site grading design minimizes the overall site earthworks program once impacted soils are removed.

Refer to Drawing GR-1 for further details.

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SANITARY SERVICING

Existing & Future Infrastructure

There are two existing sanitary pump stations (SPS) in the vicinity of the subject lands.

The Ben Machree SPS is located to the south west of the subject site on Ben Machree Drive and services a relatively small drainage area representing approximately 140 residential lots. The Region of Peel identified that it has minimal excess service capacity available to increase its service area (refer to correspondence in **Appendix B)**.

The Front Street SPS is located to the east of the subject site at the southeast corner of Lakeshore Road and Front Street and services a 166 hectare drainage area representing a mixture of residential and commercial lands. The Region of Peel identified that this pump station has significant excess service capacity available to service the subject site (refer to correspondence in **Appendix B** for details).

The Region of Peel has identified the need to upgrade the wastewater infrastructure and has identified that a new large trunk sanitary sewer will be constructed along the frontage of the site. One of the outcomes is to remove the requirement for the existing SPS in the area. The proposed Lakeshore Trunk is currently in the Environmental Assessment (EA) process and is anticipated to be submitted to the MOE in late 2017 with approvals expected in 2018. The Region's Draft Master Plan identifies this as Project WW-ST-163 with a planned in service date of 2022.

There are existing sanitary mains surrounding the site which provide servicing to the existing drainage area, namely:

- the 350mm and 375mm sanitary sewers on Lakeshore Road West
- a 250 mm sanitary sewer on Mississauga Road
- a 250 mm sanitary sewer on Port Street
- a 250 mm sanitary sewer on Bay Street
- the 250, 300 and 375 mm sanitary sewers within Front Street

Refer to Drawing SAN-1 and Appendix B for further details.

Proposed Sanitary Drainage

A review of the 2013 Region of Peel Water and Wastewater Master Plan indicates that the Front Street SPS has excess/available capacity of approximately 200 L/s (i.e., the difference between the firm capacity of 276 L/s, and present-day peak wet weather flow of 76 L/s).

The existing sanitary sewers on Lakeshore Road, Mississauga Road, Port Street and Front Street do not have adequate capacity to convey the proposed sanitary flows to the Front Street SPS. It is proposed that a new 375mm sanitary sewer be constructed along Port Street and Front Street as an outfall for the subject lands. The existing sanitary sewer on Port Street would remain in place. Refer to **Drawing SAN-1** for the proposed sanitary sewer location. There is some available capacity in the surrounding network and the opportunity to utilize components of the existing sewer system will be further reviewed at the detailed design stage.

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SANITARY DESIGN CRITERIA

Average Dry Weather Flow:

302.8 L/c/day

Infiltration / Inflow:

0.2 L/s/ha

Peaking Factor:

Harmon Formula

(Section 2.2 in Region Design Criteria)

Population (people per ha):

Semi-detached – 70

Row Dwellings – 175

Apartment – 475

Commercial - 50

Based on a review of the available as-constructed information (refer to **Appendix B**), the subject lands can be serviced entirely by gravity sewers to the Front Street SPS (although private pumping within the site plan blocks may be necessary depending on evolving site plan concepts and depths of underground parking structures).

Wastewater infrastructure will be designed in accordance with the latest Region of Peel standards and specifications.

Sanitary sewer design sheets have been prepared and used to size proposed sanitary sewers for the proposed development. For apartments where the proposed population equivalent is greater than 475 persons/hectare based on a rate of 2.7 people per unit (ppu), the calculated population equivalent was used for design.

Population Estimates and Sanitary Design Sheets can be found in **Appendix B.**

Based on the above criteria and the proposed external improvements it has been determined that there is sufficient capacity to service the subject lands as the proposed development generates approximately **97.5 L/s** of additional peak flow to the Front Street SPS, which would result in a total peak wet weather flow of **173.5 L/s** (97.5 L/s proposed + 76 L/s existing).

A preliminary profile of the proposed 375 mm sanitary sewer and pictures of the proposed route are included in **Appendix B**.

Refer to Drawing SAN-1 for further details.

Timing Implications

With the exception of the proposed 375 mm sanitary outfall to the Front Street SPS which will be constructed by the proponent, all necessary sanitary infrastructure is in place and available to service the subject lands. 375 mm sanitary sewers qualify for development credits under the Region of Peel Capital Plan. Further discussions with the Region are required.

The Region's project WW-ST-163 is scheduled to be completed in 2022. This project is not required in order for the development of the subject lands to proceed.

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WATER DISTRIBUTION

Existing & Future Infrastructure

There are existing watermains on Lakeshore Road (300 mm and 400 mm) and Mississauga Road (300 mm).

The Region of Peel has identified the need to upgrade the water servicing in Pressure Zone 1 and has identified that a new 600 mm diameter watermain is to be constructed along Lakeshore Road from the subject lands easterly to the existing Lakeview Water Treatment Plant located south of Lakeshore Road on Cawthra Road. This new 600 mm diameter watermain is identified as Regional Project 18-1119 in the Region's 2015 Capital Budget and is funded through Development Charges. This watermain is expected to be in service by 2020.

Refer to Drawing WTR-1 for further details.

Proposed Water Infrastructure

AECOM was retained to carry out a detailed hydraulic analysis of the proposed developments impact on both existing and proposed infrastructure. The analysis includes design years of 2021, 2026, 2027 and 2041.

The analysis was based on Region of Peel 2016 design Criteria and the following criteria

All scenarios were modelled without the proposed 600mm watermain on Lakeshore Road.

A network of municipal watermains is proposed throughout the subject site. In accordance with AECOM's recommendations these have been proposed as 300 mm in diameter. The findings of the report indicate that the proposed development can be serviced without the proposed 600 mm watermain on Lakeshore Road, even under the 2041 maximum day demand conditions.

The Hydraulic Analysis Report is included in **Appendix C**.

Refer to Drawing WTR-1 and Appendix C for further details.

WATER DESIGN CRITERIA

Minimum Pressure:

275 kPa (40 psi)

Maximum Pressure:

700 kPA (100 psi)

Maximum Velocity:

2.0 m/s

Fire Flow:

25,020 L/minute

417 L/s

Minimum Pressure

(max. day + fire flow):

140 kPa (20 psi)

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STORM DRAINAGE

Minor & Major System

Storm servicing for the development will conform to City of Mississauga standards. Storm sewers will be designed to convey minor system flows resulting from the 10-year storm event for ultimate discharge to Lake Ontario.

The runoff coefficients were based on the proposed land use and the City standard runoff coefficients. The 100-year flows from the subject lands were calculated using the increased runoff coefficients (1.25 x $C_{10\text{-year}}$) as per the City requirements. At this preliminary stage of design, the storm sewers have been conservatively sized assuming no LID / stormwater management measures are in place.

Two separate outfalls are proposed to Lake Ontario. The West outfall will provide an outlet for the west and south portions of the site. We have provided for two possible alignments for the West outfall. The final location will be dependent on the configuration of the final development plan and the location of public easements and/or ROWs. We have included channel capacity calculations for Block 19 should the storm sewer not be located here.

Mississauga Road is low relative to the rest of the site and drainage naturally travels to the east. A storm sewer is proposed adjacent to the existing storm sewer on Mississauga Road to accommodate the post-development drainage at each intersection (i.e. Port Street West and Lake Street) as well as site plan drainage. **Drawing STM-1** shows two possible scenarios regarding the outfall of the proposed Mississauga Road storm sewer. Headwall HW-2 represents an additional headwall adjacent to the existing outfall. An alternative scenario would combine the proposed storm sewer with the existing storm sewer before the outfall into Lake Ontario in order to minimize the disturbance to the waterfront trail and shoreline. Preliminary analysis of this alternative drainage scenario shows that the proposed storm sewers on Mississauga Road would have to be increased in size in order to accommodate the additional external flows from the existing 1050mm storm sewer. The final leg of the Mississauga Road storm sewer would have to be increased from a 900mm x 3000mm box culvert to a 1200x3000mm box culvert. This upgrade would account for 100-year capture from the external lands shown on **Drawing STM-1**.

The proposed sewer will be designed to intercept both minor and major system flows to avoid spill onto the Mississauga Road ROW and will discharge to Lake Ontario via a proposed new outfall (East outfall). The result is the site drainage will effectively bypass the existing Mississauga Road 1050mm storm sewer. This eliminates any impacts on existing infrastructure resulting from the development of the subject lands. Opportunities to combine these storm sewers upstream of the Water Front Trail and one shared Mississauga Road outfall to Lake Ontario will be further explored.

Both outlets are protected with existing armour stone seawall structures. The seawalls will need to be modified to accommodate appropriate headwalls. The proposed invert of approximately 75.0m is expected to locate the pipes well above the existing lake bottom and will reduce the likelihood of any sediment entering the pipe. The design of the shoreline works including outfall protection will be undertaken by others and coordinated with future submissions.

The proposed ROWs within the subject lands have been evaluated and will provide conveyance capacity for the major system flows (evaluated as the greater of the 100-year less 10-year storm flows).

Refer to Drawing STM-1 and Appendix D for further details.

STORMWATER MANAGEMENT

Quantity Control

Due to the subject site's close proximity to Lake Ontario, quantity control is not required according to City and CVC guidelines. Major system flows in excess of the 10-year storm event will be conveyed within the site to the proposed storm sewer outfalls to Lake Ontario site via right-of-ways within the subject land. Major system flows will be captured upstream of the outfall pipes. The location and inlet capacity of the 100-year capture points are shown on Drawing STM-1. The outfalls beneath the Water Front Trail will be sized for the greater of the 100-year or Regional storm flows.

Quality Control

Although quantity control is not required for the development, the standard MOECC stormwater management quality criteria for TSS removal apply to this site. Controls will be designed to provide an Enhanced Level of water quality protection to ensure removal of 80% of suspended solids.

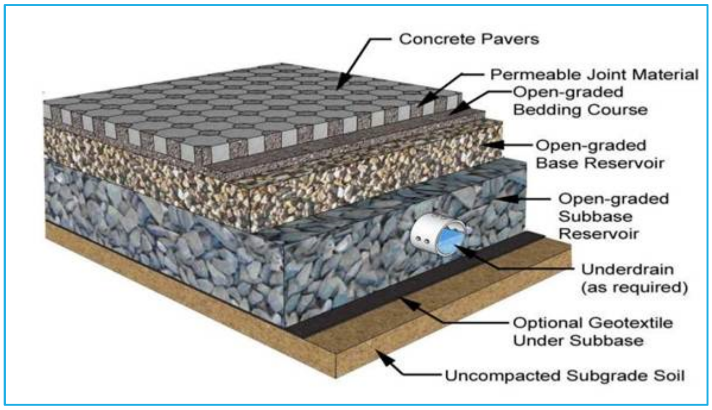
There is an opportunity to explore LID or other sustainable best management practices to provide water quality and erosion control since a conventional end-of-pipe facility is not required. A treatment train approach including possible LID measures and Oil Grit Separators (or other mechanical separators) will be implanted to provide quality control. The use of potential LID measures can also address the City's target infiltration volume (10mm), although it should be noted that opportunities for infiltration will be limited on the site plan areas due to underground parking structures. However, due to the nature of the soil removal and remediation required for the subject lands, there is unconventional flexibility to specify the new soil type/composition for the development in the open space or ROW areas. Since most LID practices are limited or defined by soil characteristics, there would be a wider range of practices available to achieve the stormwater management and infiltration objectives for the site. Potential LID measures are illustrated on the drawing LID-1 and are described below.

Potential LID Measures

The following LID practices are possible applications. All LID drawings/applications and graphics are conceptual and for illustrative purposes and will be further explored for feasibility. Note that in the site plans with underground parking, infiltrated flows will not be retained on site for recharge; flows will instead be temporarily attenuated in the soils above the underground parking structure but will ultimately be captured in the storm sewer system within the subsurface parking area. Infiltration measures in areas without underground parking will promote attenuation / retention within the native material. The potential LID measures will be designed to provide a minimum of 10mm runoff retention where feasible, or in the case of site plan areas, to provide attenuation, enhance quality / erosion control, and to promote evapotranspiration. The team is continuing to review the feasibility of incorporating some of these items in the proposed development.

SITE PLAN AREAS

- Permeable pavement for driveways and/or parking areas
- Increased topsoil depth in landscaped areas
- Rain gardens with sub-surface attenuation where feasible
- Green roofs
- Rainwater harvesting/rain barrels
- Tree pits



Conceptual LID Measures for Illustrative Purposes Only

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PUBLIC ROW AREAS

- Attenuation galleries/infiltration swales
- Bioswale/bioretention cells
- Green street boulevards
- Tree pits
- Increased topsoil depth in landscaped areas
- Permeable pavers for parking areas

Conceptual LID Measures for Illustrative Purposes Only

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OPEN SPACE AREAS

- Increased topsoil depth in landscaped areas
- Rain gardens with sub-surface attenuation
- Attenuation galleries/infiltration swales
- Tree pits
- Bioswale/bioretention cells

Refer to Drawing LID-1 D for further details.

Conceptual LID Measures for Illustrative Purposes Only

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E&SC

The erosion and sediment control plan for the site servicing program of the subject lands will be designed, approved, and implemented in conformance with the City of Mississauga, Credit Valley Conservation and MOECC recommendations.

Erosion and sediment control will be implemented for all construction activities including topsoil stripping, foundation excavation and stockpiling of materials. During construction, temporary sediment ponds may be required to treat pre-development drainage from stripped areas. The sediment control plan will be designed / coordinated with the soil remediation works and additional precautions will be taken due to the presence of contaminated soils on site.

The temporary ponds will be located at the low points of the site to detain sediment laden runoff and reduce peak flows and velocities prior to release into the receiving systems. The temporary silt ponds will maintain a permanent pool as per the MOE guidelines for temporary sediment control facilities. Forebay areas will be provided to enhance sediment removal.

The following erosion and sediment control measures will be installed and maintained during construction of the subdivision:

- A temporary sediment control fence will be placed prior to grading
- A construction plan will be implemented to limit the size of disturbed areas and to minimizing nonessential clearing
- Sediment traps will be provided
- Gravel mud mats will be provided at construction vehicle access points to minimize off-site tracking of sediments
- All temporary erosion and sediment control measures will be routinely inspected and repaired during construction. Temporary controls will not be removed until the areas they serve are restored and stable.

Recognizing that erosion and sediment control is a dynamic process, a detailed set of staging plans / construction sequencing will be required for the various stages of remediation, earthworks, servicing, site plan construction, and stabilization, coupled with the proposed development phasing.

CONCLUSIONS

The proposed Port Credit West Village Partners Inc. development can be adequately serviced through a combination of existing and proposed municipal infrastructure. In summary:

- Sanitary Servicing will be accomplished by the extension of a new municipal sanitary sewer from the existing Lake Street SPS to the subject lands and the construction of local sanitary sewers.
- Water servicing for domestic potable and fire protection will be through connections to the existing system and the construction of local watermains. The Region of Peel's proposed 600mm watermain is not required to service the subject lands.
- Storm drainage will include the construction of local storm sewers designed to convey the 10 year flow. Sections of storm sewer in close proximity to Lake Ontario and down Mississauga Road will be designed for the 100 year in order to prevent overland flow across the existing Lakefront Trail and to mitigate any potential concerns with directing drainage to Mississauga Road.
- Stormwater quantity control is not required due to the closer proximity to Lake Ontario. Major system flows will be captured in sewers directly upstream of the outlet pipe.
- Quality control will be provided through a treatment train approach to be further explored as the concept develops.
- Grading will be in accordance with City of Mississauga requirements and minimize on site earthworks and the need for retaining walls.
- Erosion and Sediment Control measures will be designed in accordance with City of Mississauga, MOECC and CVCA requirements.

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APPENDICES

Appendix A – Geotechnical Investigations (Stantec)
Appendix B – Sanitary Sewer Design Calculations
Appendix C – Hydraulic Modelling Analysis (AECOM)
Appendix D – Storm Servicing Design Calculations

Figures & Drawings:

Figure 1	*Site Location Plan*
Figure 2	*Concept Plan*
Figure 3	*Existing Conditions Plan*
Figure 4	*Draft Plan*
Figure 5	*Conceptual Phasing Plan*

Drawing GR-1	*Conceptual Grading Plan*
Drawing SAN-1	*Conceptual Sanitary Servicing Plan*
Drawing WTR-1	*Conceptual Water Servicing Plan*
Drawing STM-1	*Conceptual Storm Servicing Plan*
Drawing LID-1	*Preliminary Low Impact Development Plan*

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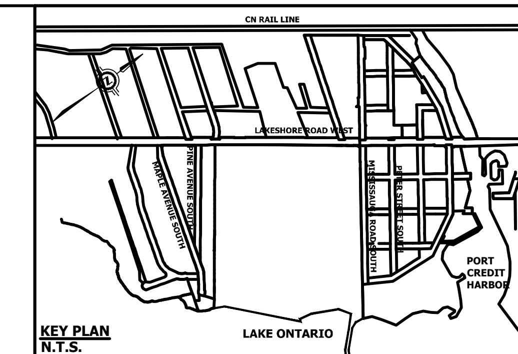

KEY PLAN
N.T.S.

LEGEND

PROPOSED SITE AREA

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

BENCHMARK

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Port Credit West Village Partners Inc.

urbantech
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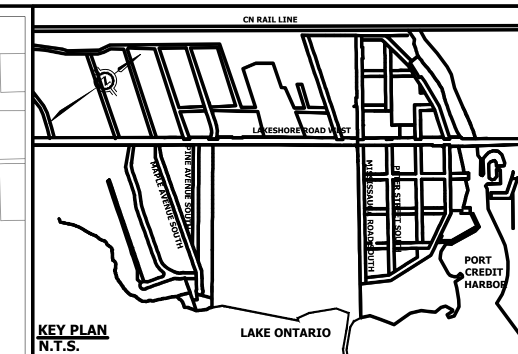
3760 14th Avenue, Suite 301, Markham, Ontario L3R 3T7
tel: 905.946.9461 fax: 905.946.9595
www.urbantech.com

SITE LOCATION PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.: FIGURE 1
SCALE: H 1:5000 (40.0m 20.0m 0m 20.0m 40.0m)		



KEY PLAN
N.T.S.

LEGEND

- PROPERTY BOUNDARY
- PROPOSED TREES
- PROPOSED PAVING
- PROPOSED GRASS
- CVC EROSION REGULATION LIMIT
- CVC HAZARD REGULATION LIMIT

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017

SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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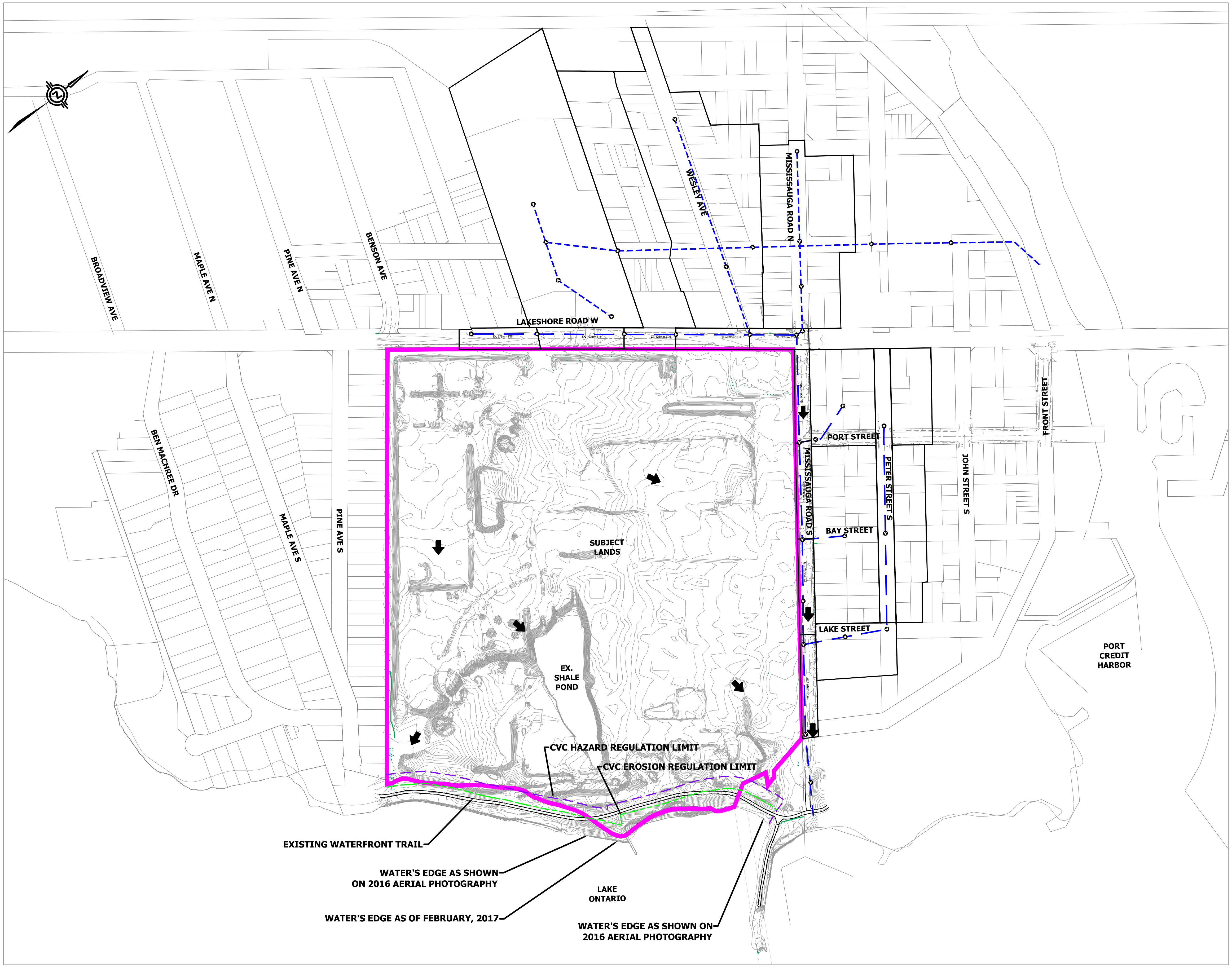
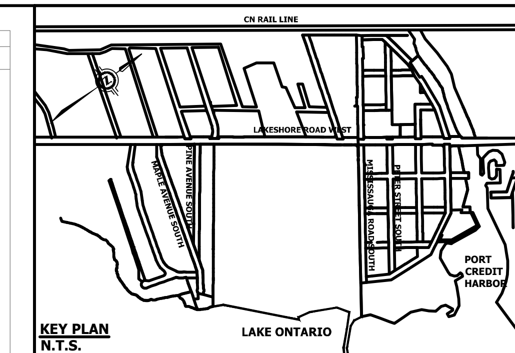
CONCEPT PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.:
SCALE: H 1:3000	40m 20m 0m 20m 40m	**FIGURE 2**





KEY PLAN
N.T.S.

LEGEND

PROPERTY BOUNDARY
EXISTING OVERLAND FLOW
INTERNAL DRAINAGE BOUNDARY
CVC EROSION REGULATION LIMIT
CVC HAZARD REGULATION LIMIT
EXISTING STORM SEWER

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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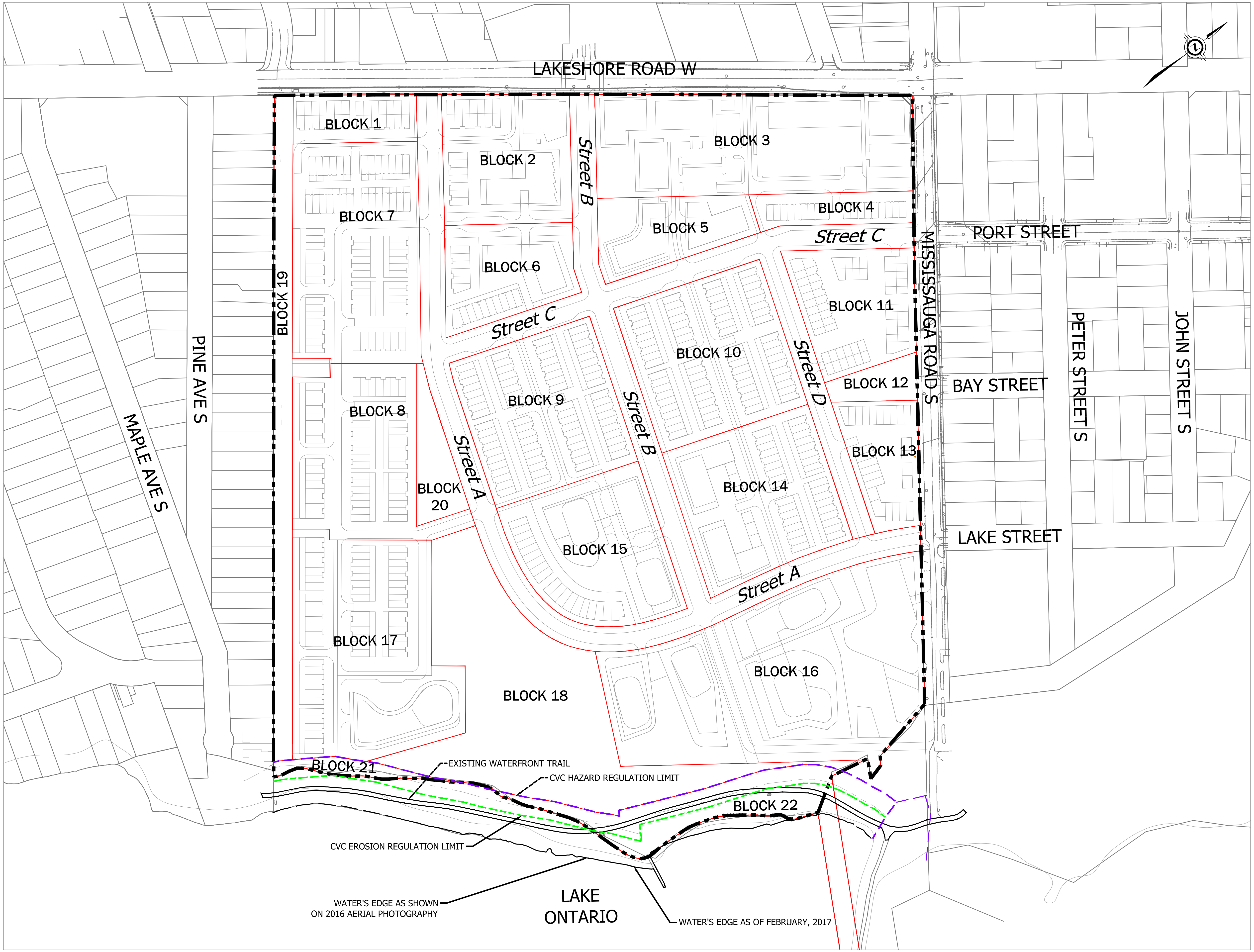
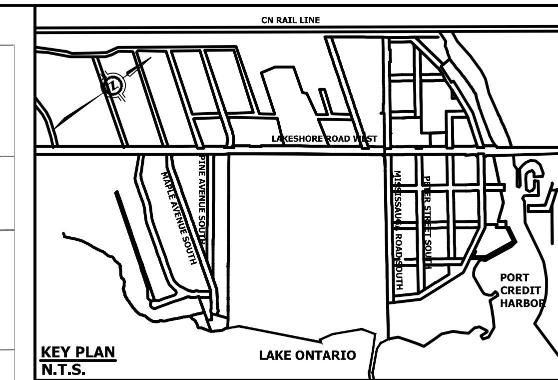
EXISTING CONDITIONS PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.:
SCALE: N.T.S.		FIGURE 3





KEY PLAN
N.T.S.

LEGEND

- PROPOSED SITE AREA
- PROPOSED LOT BOUNDARIES
- CVC EROSION REGULATION LIMIT
- CVC HAZARD REGULATION LIMIT

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
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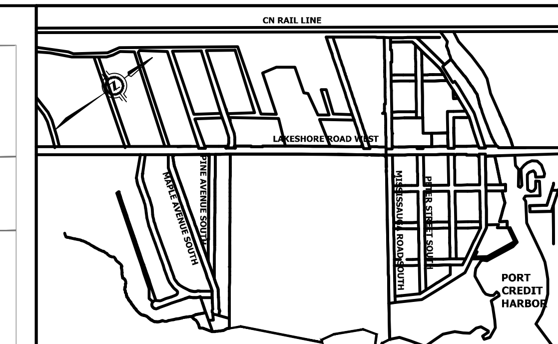
DRAFT PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.:
SCALE: H 1:3000	40m 20m 0m 20m 40m	FIGURE 4



KEY PLAN
N.T.S.

LEGEND

PROPOSED PHASING BOUNDARY

CVC EROSION REGULATION LIMIT

CVC HAZARD REGULATION LIMIT

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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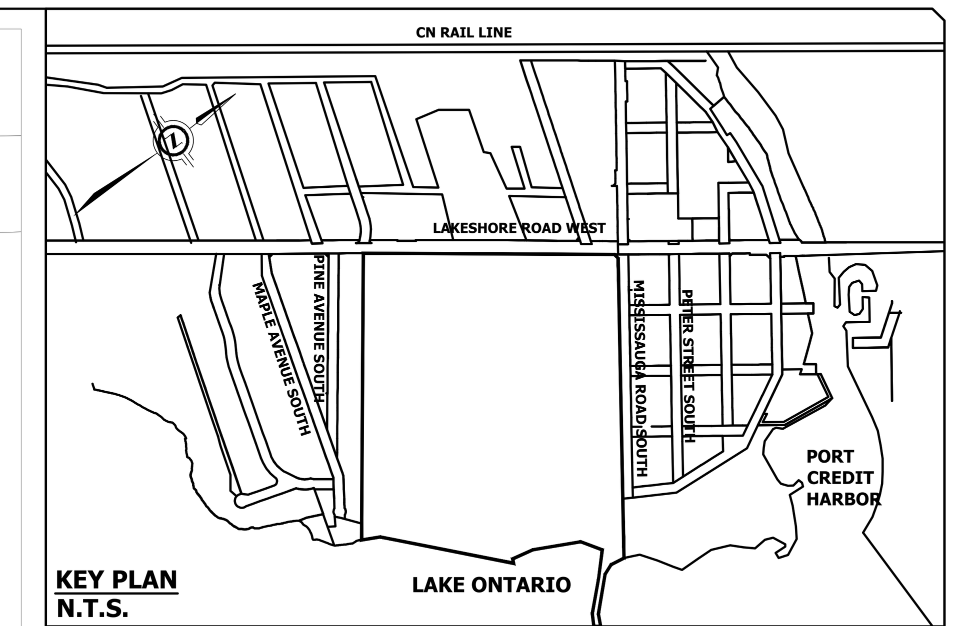
CONCEPTUAL PHASING

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.: FIGURE 5
SCALE: H 1:3000	40m 20m 0m 20m 40m	



KEY PLAN
N.T.S.

LEGEND

	PROPERTY BOUNDARY
90.0 ×	PROPOSED GRADE
90.0 ×	EXISTING GRADE
2.0%	PROPOSED SLOPE
	PROPOSED R.O.W.
	CVCA HAZARD REGULATION LIMIT
	CVCA EROSION REGULATION LIMIT
90.0	CONTOURS (1 m)
	PROPOSED OVERLAND FLOW ROUTE
FFF = 92.00	PROPOSED FIRST FINISHED FLOOR ELEVATION

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
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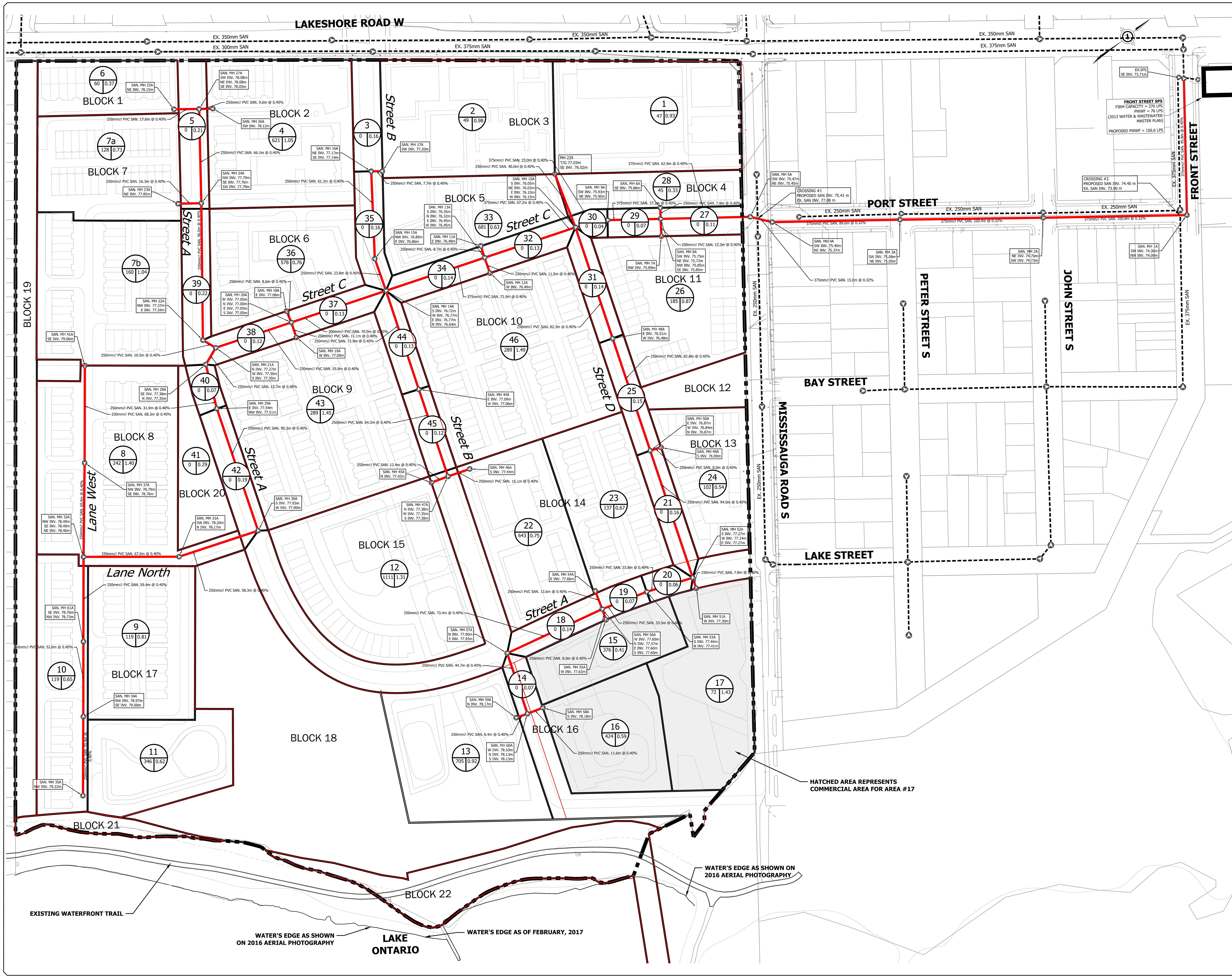
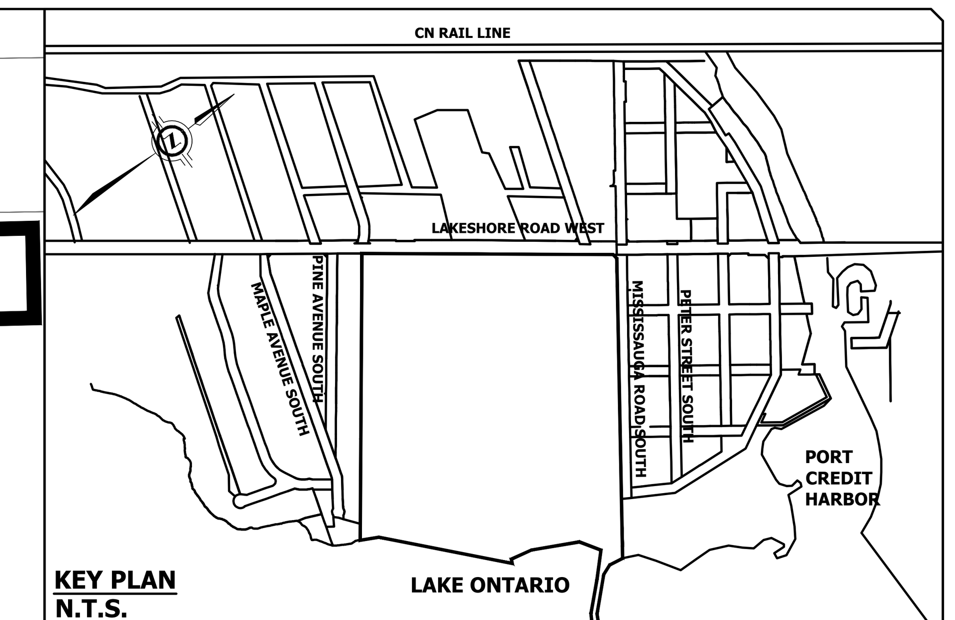
PRELIMINARY GRADING PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.:
SCALE: H 1:1250 (10m 0m 10m 20m 40m)		GR-1





KEY PLAN
N.T.S.

LEGEND

- PROPERTY BOUNDARY
- PROPOSED SANITARY SEWER AND MANHOLE
- EXISTING SANITARY SEWER AND MANHOLE
- SANITARY DRAINAGE AREA BOUNDARY
- DRAINAGE AREA I.D.
- DRAINAGE AREA (ha)
- POPULATION

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE, REFERENCE NO. 17-30-041-03-DP-A, DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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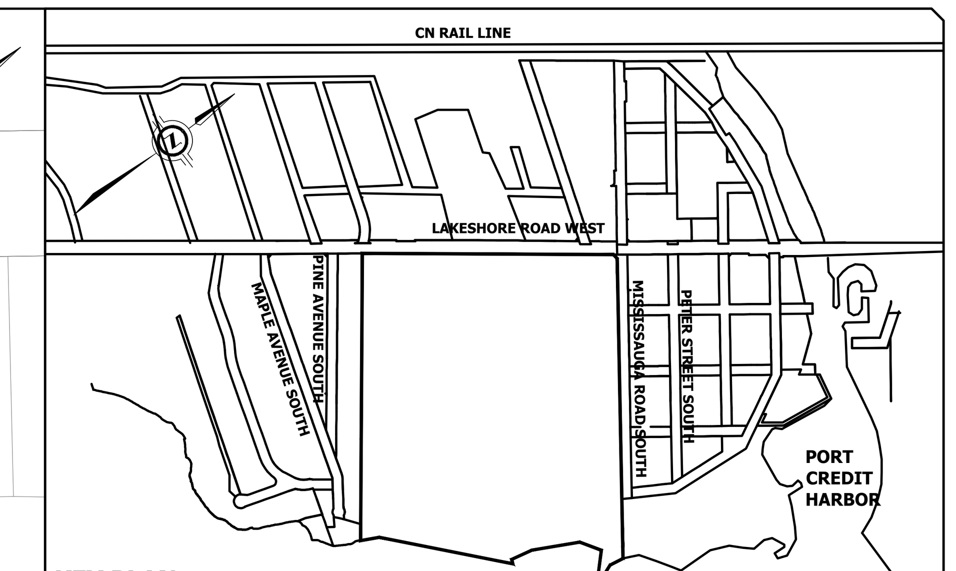
CONCEPTUAL SANITARY DRAINAGE AREA PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.:
SCALE: H 1:1250		SAN-1



KEY PLAN
N.T.S.

LEGEND

PROPOSED WATER MAIN

EXISTING WATER MAIN

HYDRANT

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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WATER SERVICING PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.: WTR-1
SCALE: H 1:1250	10m 0m 10m 20m 40m	

LAKESHORE ROAD W
PINE AVE S
JOHN STREET S
PETER STREET S
MISSISSAUGA ROAD S
PORT STREET
BAY STREET
LAKE STREET
Street A
Street B
Street C
Street D
BLOCK 1
BLOCK 2
BLOCK 3
BLOCK 4
BLOCK 5
BLOCK 6
BLOCK 7
BLOCK 8
BLOCK 9
BLOCK 10
BLOCK 11
BLOCK 12
BLOCK 13
BLOCK 14
BLOCK 15
BLOCK 16
BLOCK 17
BLOCK 18
BLOCK 19
BLOCK 20
BLOCK 21
BLOCK 22
Ext. 1 3.79ha 0.60
Ext. 2 2.41 ha 0.60
Ext. 3 0.97 ha 0.60
Ext. 4 0.61 ha 0.60
Ext. 5 1.78ha 0.60
Ext. 6 0.86 ha 0.60
Ext. 7 0.69 ha 0.90
Ext. 8 0.68 ha 0.75
Ext. 9 0.26 ha 0.90
Ext. 10 0.29 ha 0.90
Ext. 11 0.17 ha 0.90
Ext. 12 0.24 ha 0.90
Ext. 13 0.20 ha 0.90
Ext. 14 0.22 ha 0.90
Ext. 15 0.24 ha 0.90
Ext. 16 0.24 ha 0.90
Ext. 17 0.26 ha 0.90
EX. 375mm STM
EX. 450mm STM
EX. 525mm STM
EX. 600mm STM
EX. 675mm STM
EX. 900mm STM
EX. 1050mm STM
D 7.45 ha 1.166 m³/s
D 8.57 ha 1.075 m³/s
D 4.19 ha 0.551 m³/s
C 2.72 ha 0.431 m³/s
B-22 0.87 ha 0.120 m³/s
B-23 0.54 ha 0.075 m³/s
P-3 0.19 ha 0.012 m³/s
P-2 2.44 ha 0.156 m³/s
P-4 0.83 ha 0.055 m³/s
ALTERNATE STORM SERVICING FOR CATCHMENTS D-1 THROUGH D-6
ALTERNATE CONNECTION TO EXISTING OUTFALL
CVC HAZARD REGULATION LIMIT
CVC EROSION REGULATION LIMIT
EXISTING WATERFRONT TRAIL
WATER'S EDGE AS SHOWN ON 2016 AERIAL PHOTOGRAPHY
WATER'S EDGE AS OF FEBRUARY, 2017
LAKE ONTARIO
NOTE: ALL CATCHMENTS WILL RECEIVE QUALITY CONTROL AS PER CITY OF MISSISSAUGA AND CVC GUIDELINES
KEY PLAN N.T.S.
CN RAIL LINE
LAKESHORE ROAD WEST
PORT CREDIT HARBOR
LAKE ONTARIO
LEGEND
PROPOSED STORM SEWER
EXISTING STORM SEWER
ALTERNATE STORM SEWER SERVICING
INTERNAL DRAINAGE BOUNDARY
DRAFT PLAN BLOCK BOUNDARY
INTERNAL DRAINAGE AREA
AREA ID
DRAINAGE AREA (ha)
COMPOSITE RUNOFF COEFFICIENT
PROPOSED OVERLAND FLOW ROUTE
CVC EROSION REGULATION LIMIT
CVC HAZARD REGULATION LIMIT
WATER'S EDGE - LAKE ONTARIO
CAPTURE POINT ID
MAJOR SYSTEM CAPTURE AREA
100YR CONSTANT FLOW CAPTURED IN PIPE AT LOW POINT
REFERENCE DRAWINGS
DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015
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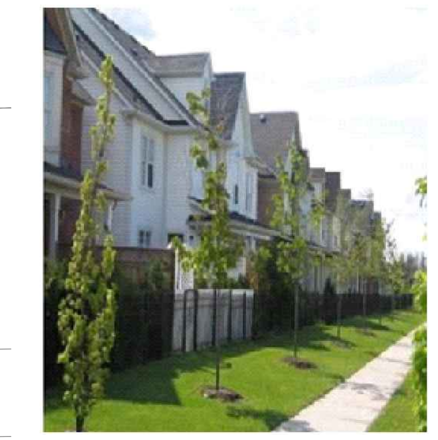
Port Credit West Village Partners Inc.
urbantech
CONCEPTUAL STORM DRAINAGE PLAN
PROJECT No.: 16-489
DATE: AUGUST 2017
DRAWING No.: STM-1
SCALE: H 1:1250

CONCEPTUAL LID MEASURES
FOR ILLUSTRATIVE PURPOSES


LID TREE PITS

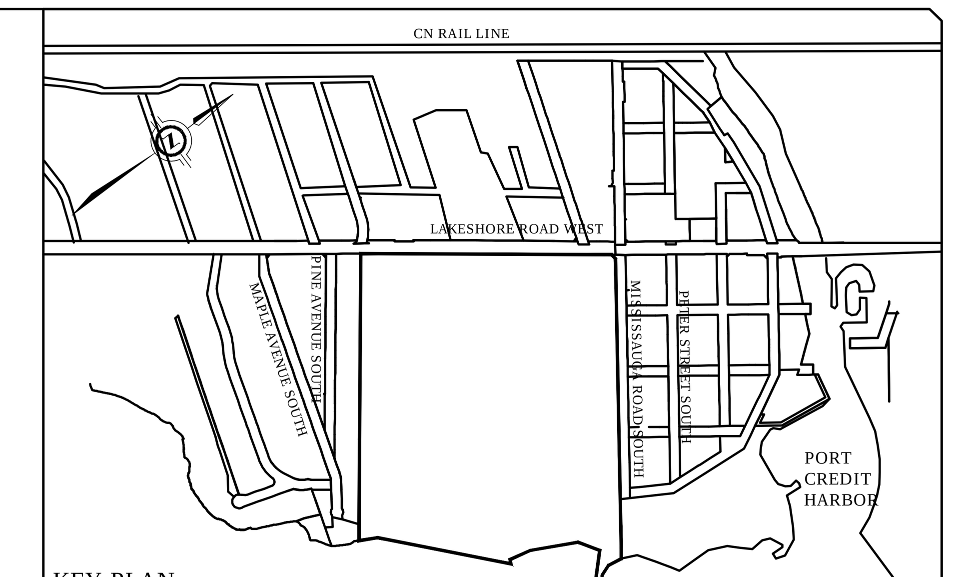

LID GREEN ROOF


LID RAIN BARRELS (RAINWATER HARVESTING)


LID GREEN STREET BOULEVARD (RAIN GARDENS)


LID PERMEABLE PAVEMENT


LID SWALE ALONG PEDESTRIAN PATH



KEY PLAN
N.T.S.

LEGEND

POTENTIAL LID APPLICATIONS

RIGHT-OF-WAYS
- LID GREEN STREET BOULEVARDS
- LID TREE PITS (10m SPACING)
- BIOSWALE/BIO-RETENTION CELLS
- GREEN STREET BOULEVARDS
- INCREASED TOPSOIL DEPTH IN LANDSCAPED AREAS
- CATCHBASIN SHIELDS/OIL-GRIT SEPARATORS

LOW-RISE BUILDINGS
- RAIN BARRELS
- PERMEABLE PAVEMENT FOR DRIVEWAYS/PARKING AREAS
- RAIN GARDENS WITH SUB-SURFACE ATTENUATION
- INCREASED TOP-SOIL DEPTH IN LANDSCAPED AREAS

MID/HIGH-RISE BUILDINGS
- GREEN ROOFS
- PERMEABLE DRIVEWAYS/PARKING
- CATCHBASIN SHIELDS/OIL-GRIT SEPARATORS
- INCREASED TOPSOIL DEPTH IN LANDSCAPED AREAS

OPEN SPACE AREAS
- RAIN GARDENS WITH SUB-SURFACE ATTENUATION
- INCREASED TOP-SOIL DEPTH IN LANDSCAPED AREAS
- ATTENUATION GALLERIES/INFILTRATION SWALES
- TREE PITS
- BIOSWALE/BIORETENTION CELLS
- INCREASED TOPSOIL DEPTH IN LANDSCAPED AREAS

REFERENCE DRAWINGS

DRAFT PLAN: J.D. BARNES, ALL OF LOT 10, PART OF LOTS 9 AND 11 AND WATER LOCATION IN FRONT OF LOT 9, BROKEN FRONT RANGE CREDIT INDIAN RESERVE. REFERENCE NO. 17-30-041-03-DP-A. DATE: AUGUST 11, 2017
SURVEY: MACKAY, MACKAY & PETERS LTD., PROJECT No. 14-102, DATE: JUNE 1ST, 2015

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PRELIMINARY
LOW-IMPACT
DEVELOPMENT PLAN

PROJECT No.: 16-489	DATE: AUGUST 2017	DRAWING No.: LID-1
SCALE: H 1:1250 (10m 0m 10m 20m 40m)		

File: P:\Projects\16-489\Reports\FSR\Drawings\16-489-FSR-LID-1.DWG - Revised by <SRIEMER> : Tue, Mar 07 2017 - 6:04pm

APPENDIX A

Geotechnical Investigations (Stantec)

Stantec Consulting Ltd.
300W-675 Cochrane Drive, Markham ON L3R 0B8

March 9, 2017
File: 122150207

Attention: Mr. David Harper, M.Sc., P. Geo
Port Credit West Village Partners Inc.
C/O
Scotia Plaza, Suite 2700, 40 King Street West,
Toronto, Ontario M5H 3Y2

Dear Mr. Harper,

Reference: Supplementary Geotechnical Considerations – Preliminary Design
Imperial Oil Lands
70 Mississauga Road South, Mississauga, ON

INTRODUCTION

It is understood that Port Credit West Village Partners Inc. (PCWVP) is considering the purchase of the Imperial Oil Limited (IOL) Lands located at the address captioned above (also referred to as "The Site"). The property is located immediately adjacent Lake Ontario and consists of approximately 30 hectares of land. The property was the site of historic oil refining operations but is currently vacant. It is understood that residential/commercial/institutional "mixed-use" development is contemplated for the property.

PCWVP previously requested that Stantec Consulting Ltd. (Stantec) provide preliminary input to assist in the evaluation of the existing site conditions and feasibility of the design and construction of the mixed-use development. Stantec subsequently prepared a letter dated June 20, 2016 that summarized the preliminary geotechnical considerations and constraints likely to be associated with redevelopment of the property (Stantec, 2016).

PWCVP has subsequently requested that Stantec prepare an additional letter discussing several specific locations/conditions that will have an implication for design and construction of the development.

For completeness, this letter includes the pertinent considerations and comments from the initial letter (Stantec, 2016) and the additional considerations and comments for specific geotechnical items as discussed herein.

This letter is not intended for use in detailed design and/or construction of the planned development. Additional geotechnical investigation and analysis will be required to support design of specific Development Blocks and associated infrastructure components.

INFORMATION SOURCES

BACKGROUND GEOLOGY

The property is located in the physiographic region known as the Iroquois Plain. The overburden is typically comprised of clay till overlain by sand (Physiography of Southern Ontario, Chapman and Putnam, 1984). The underlying bedrock in the region comprises shale and limestone of the Georgian Bay Formation (Geological Survey of Canada Map Sheet 30S, map 1355A).

ENVIRONMENTAL REPORTS

Environmental Site Assessments (ESA) have been completed at the Site by Exp Energy Services Ltd. (Exp) for IOL. These reports were provided without benefit of legal reliance. The reports included a Preliminary Environmental Assessment (Barenco, 2010), a Phase Two Environmental Site Assessment (EXP, 2015b) and a Supplementary Phase Two Environmental Site Assessment (EXP, 2015a). These documents were intended strictly for purposes of environmental characterization; geotechnical-specific investigation information, characterization or discussion was not included.

The ESAs included a large number of boreholes and test pits. A limited number of the boreholes included Standard Penetration Tests. There was no coring of the underlying bedrock included in the environmental work.

The overburden encountered in the boreholes and test pits consisted predominantly of brown and grey sandy silt with silty clay/clayey silt layers and localized (discontinuous) sand layers.

The overburden was underlain by weathered shale bedrock. Cross sections included in the Preliminary Assessment Report (Barenco, 2010) indicated the depth to the bedrock was typically in the range of 1 m to 6 m (Elevations ranging from 83.6 m above mean sea level (AMSL) in the northwest corner of the property to 68.9 m AMSL in the southeast corner of the property) though the data set was concentrated around the area of the former shale pit in the central portion of the property. The text of the Supplementary Phase Two ESA Report (EXP, 2015a) stated that bedrock was encountered at depths ranging from 0.7 m to 11 m.

The information provided in the Exp reports indicated the presence of groundwater in the overburden and in the underlying bedrock. The average depth to groundwater in overburden was 1.8 m below ground surface (BGS), with a maximum observed depth of 6.8 m BGS. The average depth to groundwater in the bedrock was 3.8 m BGS with a maximum depth of 11.4 m BGS.

Stantec completed supplemental soil and groundwater characterization in January and February 2017 as a component of due diligence. The areas of investigation were generally associated with the property boundaries where future construction activities are likely to be limited to grading and installation of services. Boreholes within the area of future development were for the purpose of additional characterization of bedrock groundwater in areas previously investigated by Exp.

A number of geotechnical reports for projects in the surrounding area were also reviewed but were considered without benefit of legal reliance. These reports provided information with respect to the geotechnical considerations and constraints associated with development on the property based on the conditions prevailing in the area.

GRADING PLAN

A Site Grading Plan dated December 15, 2016 was provided by Urbantech Consulting. The plan illustrated the existing topography across the site, and included an outline of the proposed development blocks and an indication of the anticipated depth of excavation on each block based on the intended number of levels of underground parking.

CUT/FILL REMEDIATION VOLUME ANALYSIS PLAN

A plan dated January 16, 2017 was prepared by Stantec in support of the anticipated scope of remediation required for the site. The plan illustrated the existing and proposed grades with the depth of intended excavation for the underground levels and the anticipated required depth of excavation to support the remediation work. The cut/fill calculated used a development plan dated November 28, 2016, prepared by Giannone Petricone Associates Inc.

GEOTECHNICAL CONSIDERATIONS AND CONSTRAINTS

EXCAVATIONS

The preliminary information provided indicates there will be a combination of one level and two level below-grade parking associated with the proposed building blocks. The excavations depths have been shown as 3.5 m and 7 m for the one level and two levels respectively. In addition, the remediation of the Site will require excavations at a variety of locations; a review of the Cut/Fill Remediation Volume Analysis Plan indicates the depths of the remedial excavations will vary considerably, and in cases such as for Blocks C, K, O1, O6, U1/2 and U3 will be similar to or exceed the depth of the required excavation for the proposed underground parking levels.

Excavations to the depths referenced above will encounter a combination of overburden and bedrock subject to the location on the property.

General comments on excavations in the overburden and bedrock are provided as follows:

- Excavation in the overburden (either fill materials or native soil strata) should be relatively straightforward using medium to large size excavating equipment. The borehole records reviewed indicated the presence of debris, waste, and organic inclusions in the fill materials. The recent drilling investigation by Stantec reported the presence of brick and concrete fragments in the fill materials in one borehole and this could potentially be more wide-spread. The presence of cobbles, boulders, and slabs of shale (particularly near the interface with the underlying bedrock) should be anticipated in the native soils.

- Excavation in the shale bedrock is common-practice in the Greater Toronto Area (GTA). The upper zone in the bedrock (typically to depths in the order of 1 m) is often weathered to a condition whereby the bedrock can be excavated using the same equipment used for excavation of the overburden. However, slightly deeper excavations in the shale bedrock or general mass excavation in the shale bedrock is typically undertaken using the heaviest/largest excavation equipment with a ripper tooth. Deeper excavations required for buildings that include underground parking levels or localized 'trench' excavations for services and utilities are typically undertaken using hydraulic rock breaking equipment. The presence of hard layers of rock (e.g. limestone and dolostone) within the shale is common and should be anticipated; these layers can be in the order of 300 mm thick or more and typically require the use of hydraulic rock breaking equipment to facilitate excavation. The shale bedrock in Southern Ontario is known to exhibit locked-in horizontal stresses. As a result, movement (both short-term and long-term) can occur in the rock face following excavation and slabs of the shale bedrock can be loosened during the excavation process. Appropriate care must be undertaken to address this with respect to design, construction, and Health & Safety.

Given the conditions noted in the preceding paragraph, full-depth excavation of any/all existing fill materials should be anticipated in all areas of the property where infrastructure is proposed. Excavation of existing fill materials may not be required in the areas of future parks and/or landscaped areas provided that the design grade is consistent with the existing grade or a relatively minor (e.g. less than approximately 1 m) grade raise is contemplated.

As summarized in a preceding section, the ESA reports recorded groundwater at depths ranging from as shallow as 0.3 m BGS to 6.75 m BGS (with several data outliers exceeding this range). A comparison of the intended depths of excavations outlined above with the groundwater levels recorded indicates that the excavations will penetrate below the static groundwater level. General comments with respect to the presence of groundwater in open excavations are provided as follows (these comments should not be construed as a hydrogeological analysis or recommendations):

- Seepage and infiltration from fill materials into open excavations is subject to the nature and condition of the fill and presence of perched groundwater therein. Volumes can vary from practically zero to major inflow although typically the flow decreases substantively over time as the volume of groundwater perched in the fill is limited.

- Seepage and infiltration from the prevailing overburden soils into open excavations is anticipated to be minor to moderate in part due to the predominantly fine-grain nature of these strata. However, more permeable seams and zones were observed in the investigation holes on the Site and are known to exist in the native soils. Where present these seams or zones will contribute to higher volumes of seepage and infiltration into open excavations.

- Seepage and infiltration from the shale bedrock into open excavations is typically minor to moderate; the exception to this is where the upper zone of the rock is extremely fractured contributing to the presence of preferential pathways for groundwater migration and where the excavation is located in proximity to the adjacent lake. For example, it is reasonable to presume that a hydraulic connection exists between the lake and the historic harbor slip in the southeast corner of the property. As a result, it is likely that excavations in the southern portion of the property may encounter large volumes of groundwater influx and as such will require the construction of groundwater cut-off walls (discussed further below).

- Where permeable seams and zones exist in the overburden, where the bedrock contact is particularly deep (the shale pit and the former harbor inlet) and/or where the bedrock is fractured, dewatering may be required to permit excavation to the required depths for remediation and/or construction.

REUSE OF EXISTING MATERIALS

The following comments are subject to the environmental condition and presence of contaminants on the property (discussed under separate cover).

Reuse of the existing fill materials as engineered fill or similar may be possible subject to the removal of any waste, debris, organics, oversize, or similar materials and confirmation that the moisture content of these materials is consistent with the requirements for placement and compaction.

Reuse of the native soil strata should be feasible subject to the moisture content being consistent with the requirements for placement and compaction.

Excavated shale bedrock has been used as engineered fill on projects in the GTA in the past. Typically, a more rigid and detailed placement and compaction program than would be considered typical is developed for this purpose.

It is understood that the required remediation on the property will include excavations in excess of 7 m deep (e.g. the former shale pit). As a result, there will be a requirement for zones of thick engineered fill. Typically, engineered fill is commonly placed up to 3 m thick provided that the approved material is placed and compacted in accordance with an engineered fill specification. It is not unusual that engineered fill is placed up to 5 m thick but this requires tighter and more rigid placement and compaction criteria and control. Engineered fill that is placed in excess of 5 m thick can result in future settlement which can pose a concern to the long-term serviceability of buildings and infrastructure. Placement of engineered fill in excess of 5 m thick will require a more rigid specification, a delay between the time of placement and the time of construction, and a settlement monitoring program to confirm that any potential future settlements will be within tolerable limits.

OPEN-CUT EXCAVATIONS & TEMPORARY SHORING

Excavations must meet the Occupational Health & Safety Act & Regulations (OH&S Act) in all cases.

The majority of excavations for one underground level should be feasible without the use of temporary shoring, subject to block and site-specific geometry and space constraints. The portion of the excavation in the overburden would be sloped consistent with the requirements of the OH&S Act. The portion of the excavation in the shale bedrock can often be undertaken with a vertical or near-vertical cut face. As noted in a preceding section, the shale bedrock in Southern Ontario is known to exhibit locked-in horizontal stresses. As a result, movement (both short-term and long-term) can occur in the rock face following excavation.

Excavations for one underground level that are located in proximity to the lake may require temporary shoring to prevent infiltration of groundwater.

Excavations for two underground levels are typically undertaken using a temporary shoring system.

- Where bedrock is present at a relatively shallow depth and groundwater infiltration is not a concern (e.g. more likely for the blocks in the north portion of the site), it is common to use a soldier pile and lagging system; the soldier piles are "toed" into the underlying bedrock to provide the necessary support.
- Where bedrock is present at a relatively shallow depth but groundwater infiltration is a concern (such as for the blocks in the south portion of the site particularly in proximity to the lake), the temporary shoring system would typically consist of a secant pile wall system which provides a water-tight seal against groundwater infiltration.

FOUNDATION COMMENTARY

Foundation Design Considerations

In consideration of the discussion provided in the preceding sections, it is anticipated that the development will likely include a combination of conventional spread and/or strip footing foundations and drilled piers (caissons).

Based on a comparison of the depth/elevation of the bedrock as encountered in the investigation holes advanced for the ESAs and the design depth/elevation for the Development Blocks, conventional spread and/or strip footings would be founded on one of three materials; engineered fill, native soils or the shale bedrock. These respective founding conditions are briefly described below with specific reference to the Serviceability Limit States Condition [SLS] (e.g. considering total settlements to be limited to the conventional 25 mm).

- Conventional foundations placed on the weathered shale bedrock are typically designed for bearing reactions in the range of 500 kPa to 1,000 kPa. Conventional foundations placed on the 'sound' shale bedrock (below the zone of weathering) are often designed for bearing reactions in the range of 2,500 kPa. Based on the comparison of the bedrock elevation with the founding elevation, it is conceivable that conventional foundations could be placed on the bedrock for Development Blocks A, B, C (part thereof), D, E, F2 and F3, G, H1, I, J, K1 and K2, O2, O3, O4, O5 (part thereof), O6 (part thereof), R2 (part thereof), T, U1, U2 and U3. In all of these cases, either the design founding elevation for the lowest level of underground is below the contact surface with the bedrock or within 1.2 m within the contact surface with the bedrock which should permit placement of conventional foundations on the bedrock. Confirmation of the bedrock elevation across each Development Block is recommended at the time of detailed design.

- Conventional foundations placed on the native soils are feasible but will likely be problematic in a number of cases given the N-values shown on the borehole records reviewed are lower than typical for the native Iroquois Plain till soil. There is a potential that conventional foundations placed on the native till soils could have relatively low bearing reactions at SLS in the order of 75 kPa to 150 kPa rendering this option either unviable or less efficient for medium rise development. Based on the comparison of the bedrock elevation with the founding elevation, this could apply for Development Blocks C (part thereof), H2, K2, L, M1, M2, O1, O5 (part thereof), O6 (part thereof), Q, R1, and R2 (part thereof). Evaluation and confirmation of the subsurface conditions across each individual Development Block is recommended at the time of detailed design.

- Conventional foundations placed on approved engineered fill (suitably placed and compacted) are typically designed using 150 kPa to 200 kPa under a SLS condition. The thickness of the engineered fill required to be placed should be uniform and is typically limited to approximately 5 m or less to avoid potential long-term settlements that could jeopardize the integrity of the foundations and building. Thicker fills will require additional consideration and analysis and may not prove viable for the use of conventional foundations. The placement of conventional foundations on engineered fill could apply to the following scenarios:

  - Those blocks where the depth of remedial excavation extends below the design founding elevation for a particular Development Block; and,
  - Those blocks where the native till soil is present but the condition of the native till soil is not consistent with providing a bearing reaction/resistance that supports a practical conventional foundation design (refer to the preceding bullet and discussion of conventional foundations placed on the native soils). Under this scenario, consideration could be given to sub-excavating the existing 'poorer quality' native soil and replacing the excavated material with engineered fill.

- Drilled piers (caissons) founded in the underlying sound shale bedrock (below the zone of weathering) could be considered for a number of scenarios:
  - Where higher bearing resistances/reactions are desired for use in design;
  - Where the subsurface conditions are not suitable for the use of conventional foundations. This was discussed in the preceding bullets and is described in the subsequent section of this report with reference to specific Development Blocks (e.g. Block P and the location of the former shale pit); and,
  - Where there is a large variation in the thickness of native soil or engineered fill below the founding level such that the use of conventional foundations could lead to concerns with respect to differential settlements.

Caisson foundations are commonly designed using bearing resistances at Ultimate Limit States (ULS) rather than Serviceability Limit States (SLS) as settlement is not typically the governing factor in design. Common ULS values are in the range of 2,500 kPa to 5,000 kPa for caissons founded in the sound shale bedrock (e.g. below the weathered zone).

Block P & The Former Shale pit

Block P is located in the south-central portion of the Site. The west portion of the block is located in the area of the excavation for the former shale pit. The existing data indicates the bedrock elevation within the limits of the former shale pit (e.g. in the west portion of Block P) is in the order of 70 m AMSL and the bedrock elevation to the immediate east of the former shale pit (e.g. in the east portion of Block P) could be as high as is in the order of 79 m AMSL to 80 m AMSL.

The grading plan indicated a proposed FFE of 82.0 m AMSL and a cut of 7 m for 2 levels of underground parking for this block, yielding a design founding elevation of the lowest underground level of 75.0 m AMSL.

In consideration of the elevations referenced above, at the intended founding elevation of 75 m AMSL the east portion of the block would be on the underlying shale bedrock whereas the west portion of the block would be on up to approximately 5 m of new fill material required to backfill the former shale pit (and any associated remedial excavation in the immediate area) to the design grade.

For this condition, differential settlements of the building foundations and floor slab would be a concern if conventional foundations and a slab-on-grade approach are considered. As a result, consideration could be given to supporting the west portion of this block on a deep foundation system (e.g. such as drilled piers extending into the underlying shale bedrock) and a structural floor slab and supporting the east portion of the block on a conventional spread/strip footing foundation system on the shale bedrock and a slab-on-grade floor slab. The differential settlements between the two building portions would be of such a small magnitude so as not to adversely affect the performance of the building.

Block U3 & The Former Harbor Inlet

Block U3 is located in the southeast corner of the site, overlying the location of the former harbor inlet that was used for loading/unloading for historic operations on the property.

The exact limits and depth of the original harbor inlet have not been confirmed. For reference a review of the Orthoimagery (circa 1977) indicates that the north-south boundary between Block U2 and Block U3 is likely in close proximity to the west limit of the former harbor inlet. In addition, a review of a record for a test hole located near the north limit of the former harbor inlet (Exp, 2015) indicates the bedrock was encountered at an elevation of approximately 72.4 m AMSL. A record for a test hole located on the east side of the former harbor inlet (Exp, 2015) indicates the bedrock was encountered at an elevation of approximately 73.7 m AMSL. In addition, two recently installed monitoring wells (Stantec, 2017) at the south end of the former harbor limit encountered bedrock at an elevation of 72.9 m AMSL and 73.5 m AMSL. There are noted variations in the depth to the bedrock recorded in the immediate area of the former harbor inlet and there is a possibility (though not confirmed based on the current data available) that the harbor inlet was excavated into the underlying shale bedrock.

The grading plan indicated a proposed FFE of 78.5 m AMSL and a cut of 7 m for 2 levels of underground parking, yielded a design founding elevation of the lowest underground level of 71.5 m AMSL.

In consideration of the current subsurface information available for this area, it is reasonable to anticipate that development of Block U3 could include the use of conventional spread/strip footing foundations placed in the underlying shale bedrock and the use of a conventional slab-on-grade, pending confirmation that the original excavation for the harbor inlet did not extend below approximately elevation 72+/- m AMSL. Subject to the specific extent of the building block and the bedrock elevation, filling with lean concrete could be required to level the bedrock surface. The volume of concrete required for this purpose may be substantial.

FLOOR SLAB CONSIDERATIONS

Conventional Slab-On-Grade Floor Slab

Conventional slab-on-grade floor slabs can be used under a number of scenarios provided that the elevation of the slab is above the prevailing static groundwater level. These scenarios include:

- Where the design elevation for the lowest level in a particular Development Block is at or below the contact surface with the bedrock
- Where the native till soils are present and the N-values obtained from the SPTs in the till soil are approximately ≥ 10;
- In areas where engineered fill has been placed to develop the design grade for the lowest underground level. Consistent with the comments provided in a preceding section, the engineered fill should have a uniform thickness not exceeding approximately 5 m.

In areas where the N-values in the native till soil are less than that indicated in the bullet above and/or there is a large variation in the thickness or condition of the native soils underlying the floor slab, localized sub-excavation and re-compaction may be required to facilitate a slab-on-grade floor slab approach.

Structurally Supported Floor Slab

A structurally supported floor slab will be required under the following scenarios:

- In areas where the N-values in the native till soil are less than that indicated in the bullet in the preceding sub-section and/or there is a large variation in the thickness or condition of the native soils underlying the floor slab and where sub-excavation and re-compaction to support a slab-on-grade is not considered practical, feasible or cost-effective.
- Where a portion of a block is underlain by a considerable thickness of fill or native soil and the adjacent portion of the building is underlain by a hard surface (e.g. the shale bedrock). For this condition, differential settlement could occur that would damage the serviceability of the floor slab. An example of this is Block P as discussed in detail above.

Design Considerations for the Presence of the Groundwater Table

As discussed in the preceding sections, the underground levels for a number of the blocks will extend below the static groundwater level. Typically, where such a condition occurs it is addressed in one of two ways; either the building includes a perimeter and underfloor drain system that collects groundwater seepage and discharges to the municipal sanitary sewer or the building is designed as a water-tight structure.

It is understood that subsequent to the remediation of the site, the groundwater will still exhibit environmental impacts, although concentrations will be below property specific standards derived for the Site through an Ontario Ministry of the Environment and Climate Change Risk Assessment. Depending on the development block, the groundwater seepage that would be collected in a perimeter and underfloor drain system could therefore require treatment to permit discharge to the municipal sanitary sewer system. This would be a permanent operational requirement and incur both effort and cost over the long-term.

The preferred approach is therefore to design the portion of the building that extends below the static groundwater level as a water-tight structure. The additional benefit of this approach is that the water-tight design will also serve as a barrier to prevent migration of vapors into the parking garage areas.

In designing the buildings as water-tight structures, the floor slab must be designed to resist the hydrostatic uplift force and the walls must be designed to resist the lateral hydrostatic forces.

LIMITATIONS

The Environmental Assessment Reports referenced herein (prepared by others) were provided without benefit of legal reliance. Stantec Consulting Ltd. assumes no responsibility for the accuracy or reliability of the documents and the information contained therein. Similarly, the plans and drawings referenced herein were prepared by others and provided to Stantec Consulting Ltd. for use in the preparation of this report. Stantec Consulting Ltd. assumes no responsibility for the accuracy or reliability of the plans and documents provided.

This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

The information, opinions and/or recommendations provided in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the project as described herein. If the scope of the project is modified from that described herein, this report may no longer be valid unless Stantec Consulting Ltd. is requested by the client to review and revise the report reflecting the modifications to the intended scope of development.

The information, opinions and/or recommendations provided in this report are based on the information sources referenced herein. The investigations completed to date were specific to environmental characterization of the Site and reflect the conditions encountered at the locations of the investigation holes. The interpretation of the information sources referenced has been conducted in accordance with industry standards and reasonable engineering practice. No 'geotechnical-specific' investigation has been completed. Geotechnical investigation and analysis will be required in support of the design and construction of the project.

CLOSURE

We trust that the information provided herein is of value to PCWVP at this time. If you have any questions or if we can be of further assistance in any regard, please do not hesitate to contact any of the undersigned at your convenience.

Regards,

STANTEC CONSULTING LTD.

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APPENDIX B

Sanitary Sewer Design Calculations

urbantech west

SANITARY SEWER DESIGN SHEET	PROJECT DETAILS	DESIGN CRITERIA
Port Credit Baseline conditions as per R.V Anderson Design Sheet City of Mississauga	Project No.: 16-489W Date: 9-Aug-17 Designed by: N.M Checked by: P.H	Min. Diameter= 250 mm Manning's 'n' = 0.013 Min. Velocity = 0.75 m/s Max. Velocity = 3.50 m/s Avg. Domestic Flow = 302.8 l/c/d Infiltration = 0.200 l/s/ha Max. Peaking Factor = 4.00 Min. Peaking Factor = 1.50

NOMINAL PIPE SIZE USED

			RESIDENTIAL							COMMERCIAL/INDUSTRIAL/INSTITUTIONAL						FLOW CALCULATIONS							PIPE DATA					
STREET	FROM MH	TO MH	AREA (ha)	ACC. AREA (ha)	UNITS (#)	DENISTY (P/ha)	DENSITY (P/unit)	POP	ACCUM. RES. POP.	AREA (ha)	ACC. AREA (ha)	EQUIV. POP. (p/ha)	FLOW RATE (l/s/ha)	EQUIV. POP.	ACCUM. EQUIV. POP.	INFILTRATION (l/s)	TOTAL ACCUM. POP.	PEAKING FACTOR	RES. FLOW (l/s)	COMM. FLOW (l/s)	ACCUM. COMM. FLOW (l/s)	TOTAL FLOW (l/s)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (l/s)	FULL FLOW VELOCITY (m/s)	ACTUAL VELOCITY (m/s)	PERCENT FULL (%)
Lane West	MH 35A	MH 34A	1.268	1.268				465	465							0.254	465	3.99	6.50			6.76	0.40	250	37.61	0.77	0.58	18.0%
Lane West	MH 34A	MH 61A	0.810	2.078				119	584							0.416	584	3.94	8.06			8.48	0.40	250	37.61	0.77	0.62	22.5%
Lane West	MH 61A	MH 32A		2.078					584							0.416	584	3.94	8.06			8.48	0.40	250	37.61	0.77	0.62	22.5%
Lane West	MH 41A	MH 37A	2.440	2.440				402	402							0.488	402	4.00	5.64			6.12	0.40	250	37.61	0.77	0.56	16.3%
Lane West	MH 37A	MH 32A		2.440					402							0.488	402	4.00	5.64			6.12	0.40	250	37.61	0.77	0.56	16.3%
Lane North	MH 32A	MH 31A		4.518					986							0.904	986	3.80	13.14			14.05	0.40	250	37.61	0.77	0.71	37.4%
Lane North	MH 31A	MH 30A	0.290	4.808					986							0.962	986	3.80	13.14			14.11	0.40	250	37.61	0.77	0.71	37.5%
Street A	MH 30A	MH 29A	0.190	4.998					986							1.000	986	3.80	13.14			14.14	0.40	250	37.61	0.77	0.71	37.6%
Street A	MH 29A	MH 28A	0.070	5.068					986							1.014	986	3.80	13.14			14.16	0.40	250	37.61	0.77	0.71	37.6%
Street C	MH 28A	MH 21A		5.068					986							1.014	986	3.80	13.14			14.16	0.40	250	37.61	0.77	0.71	37.6%
Street A	MH 25A	MH 27A	0.290	0.290				56	56	0.08	0.08	50		4	4	0.074	60	4.00	0.84			0.92	0.40	250	37.61	0.77	0.32	2.4%
Street A	MH 26A	MH 27A	0.720	0.720				604	604	0.33	0.33	50		17	17	0.210	621	3.92	8.54			8.75	0.40	250	37.61	0.77	0.62	23.3%
Street A	MH 27A	MH 24A	0.210	1.220					660		0.41				21	0.326	681	3.90	9.31			9.64	0.40	250	37.61	0.77	0.64	25.6%
Street A	MH 23A	MH 24A	0.730	0.730				128	128							0.146	128	4.00	1.79			1.94	0.40	250	37.61	0.77	0.4	5.2%
Street A	MH 24A	MH 22A	0.220	2.170					788		0.41				21	0.516	809	3.86	10.94			11.45	0.40	250	37.61	0.77	0.67	30.5%
Street A	MH 22A	MH 21A		2.170					788		0.41				21	0.516	809	3.86	10.94			11.45	0.40	250	37.61	0.77	0.67	30.5%
Street C	MH 21A	MH 20A	0.120	7.358					1774		0.41				21	1.554	1795	3.62	22.78			24.34	0.40	250	37.61	0.77	0.81	64.7%
Street C	MH 18A	MH 20A	0.650	0.650				572	572	0.11	0.11	50		6	6	0.152	578	3.94	7.98			8.14	0.40	250	37.61	0.77	0.61	21.6%
Street C	MH 19A	MH 20A	1.450	1.450				289	289							0.290	289	4.00	4.05			4.34	0.40	250	37.61	0.77	0.51	11.5%
Street C	MH 20A	MH 14A	0.130	9.588					2635		0.52				27	2.022	2662	3.49	32.52			34.54	0.40	300	61.16	0.87	0.89	56.5%
Street B	MH 17A	MH 16A								0.98	0.98	50		49	49	0.196	49	4.00	0.69			0.88	0.40	250	37.61	0.77	0.32	2.3%
Street B	MH 16A	MH 15A	0.160	0.160							0.98				49	0.228	49	4.00	0.69			0.91	0.40	250	37.61	0.77	0.32	2.4%
Street B	MH 15A	MH 14A	0.160	0.320							0.98				49	0.260	49	4.00	0.69			0.95	0.40	250	37.61	0.77	0.33	2.5%
Street B	MH 45A	MH 47A	1.310	1.310				1111	1111							0.262	1111	3.77	14.68			14.94	0.40	250	37.61	0.77	0.72	39.7%
Street B	MH 46A	MH 47A	0.750	0.750				643	643							0.150	643	3.92	8.82			8.97	0.40	250	37.61	0.77	0.63	23.9%
Street B	MH 47A	MH 44A	0.120	2.180					1754							0.436	1754	3.63	22.31			22.75	0.40	250	37.61	0.77	0.8	60.5%
Street B	MH 44A	MH 14A	0.130	2.310					1754							0.462	1754	3.63	22.31			22.77	0.40	250	37.61	0.77	0.8	60.5%
Street C	MH 14A	MH 13A	0.140	12.358					4389		1.5				76	2.772	4465	3.29	51.49			54.26	0.40	375	110.89	1.00	1	48.9%
Street C	MH 11A	MH 13A	0.520	0.520				675	675	0.11	0.11	50		6	6	0.126	681	3.90	9.31			9.44	0.40	250	37.61	0.77	0.64	25.1%
Street C	MH 12A	MH 13A	1.490	1.490				289	289							0.298	289	4.00	4.05			4.35	0.40	250	37.61	0.77	0.51	11.6%
Street C	MH 13A	MH 10A	0.130	14.498					5353		1.61				82	3.222	5435	3.21	61.17			64.39	0.40	375	110.89	1.00	1.04	58.1%
Street A	MH 59A	MH 60A	0.490	0.490				683	683	0.43	0.43	50		22	22	0.184	705	3.89	9.62			9.80	0.40	250	37.61	0.77	0.64	26.1%
Street A	MH 58A	MH 60A	0.590	0.590				424	424							0.118	424	4.00	5.94			6.06	0.40	250	37.61	0.77	0.56	16.1%
Street A	MH 60A	MH 57A	0.070	1.150					1107		0.43				22	0.316	1129	3.77	14.90			15.21	0.40	250	37.61	0.77	0.73	40.5%
Street A	MH 57A	MH 56A	0.140	1.290					1107		0.43				22	0.344	1129	3.77	14.90			15.24	0.40	250	37.61	0.77	0.73	40.5%
Street A	MH 54A	MH 56A	0.670	0.670				137	137							0.134	137	4.00	1.92			2.05	0.40	250	37.61	0.77	0.41	5.5%
Street A	MH 55A	MH 56A	0.410	0.410				376	376							0.082	376	4.00	5.27			5.35	0.40	250	37.61	0.77	0.54	14.2%
Street A	MH 56A	MH 53A	0.070	2.440					1620		0.43				22	0.574	1642	3.65	21.01			21.58	0.40	250	37.61	0.77	0.79	57.4%
Street A	MH 53A	MH 52A	0.060	2.500					1620		0.43				22	0.586	1642	3.65	21.01			21.59	0.40	250	37.61	0.77	0.79	57.4%
Street D	MH 51A	MH 52A								1.43	1.43	50		72	72	0.286	72	4.00	1.01			1.30	0.40	250	37.61	0.77	0.36	3.4%
Street D	MH 52A	MH 50A	0.160	2.660					1620		1.86				94	0.904	1714	3.64	21.85			22.75	0.40	250	37.61	0.77	0.8	60.5%
Street D	MH 49A	MH 50A	0.540	0.540				102	102							0.108	102	4.00	1.43			1.54	0.40	250	37.61	0.77	0.38	4.1%

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west

STREET	FROM MH	TO MH	RESIDENTIAL							COMMERCIAL/INDUSTRIAL/INSTITUTIONAL						FLOW CALCULATIONS							PIPE DATA					
			AREA (ha)	ACC. AREA (ha)	UNITS (#)	DENISTY (P/ha)	DENSITY (P/unit)	POP	ACCUM. RES. POP.	AREA (ha)	ACC. AREA (ha)	EQUIV. POP. (p/ha)	FLOW RATE (l/s/ha)	EQUIV. POP.	ACCUM. EQUIV. POP.	INFILTRATION (l/s)	TOTAL ACCUM. POP.	PEAKING FACTOR	RES. FLOW (l/s)	COMM. FLOW (l/s)	ACCUM. COMM. FLOW (l/s)	TOTAL FLOW (l/s)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (l/s)	FULL FLOW VELOCITY (m/s)	ACTUAL VELOCITY (m/s)	PERCENT FULL (%)
Street D	MH 50A	MH 48A	0.150	3.350					1722		1.86				94	1.042	1816	3.62	23.03			24.07	0.40	250	37.61	0.77	0.81	64.0%
Street D	MH 48A	MH 10A	0.140	3.490					1722		1.86				94	1.070	1816	3.62	23.03			24.10	0.40	250	37.61	0.77	0.81	64.1%
Street D	MH 229	MH 10A								1.02	1.02	50		47	47	0.204	47	4.00	0.66			0.86	0.40	250	37.61	0.77	0.32	2.3%
Street C	MH 10A	MH 9A	0.040	18.028					7075		4.49				223	4.504	7298	3.09	79.01			83.51	0.40	375	110.89	1.00	1.1	75.3%
Street C	MH 9A	MH 8A	0.070	18.098					7075		4.49				223	4.518	7298	3.09	79.01			83.53	0.40	375	110.89	1.00	1.1	75.3%
Street C	MH 6A	MH 8A	0.330	0.330				45	45							0.066	45	4.00	0.63			0.70	0.40	250	37.61	0.77	0.3	1.9%
Street C	MH 7A	MH 8A	0.870	0.870				185	185							0.174	185	4.00	2.59			2.77	0.40	250	37.61	0.77	0.45	7.4%
Street C	MH 8A	MH 5A	0.110	19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.40	375	110.89	1.00	1.11	77.5%
Port Street	MH 5A	MH 4A		19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.32	375	99.18	0.90	1.01	86.6%
Port Street	MH 4A	MH 3A		19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.32	375	99.18	0.90	1.01	86.6%
Port Street	MH 3A	MH 2A		19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.32	375	99.18	0.90	1.01	86.6%
Port Street	MH 2A	MH 1A		19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.32	375	99.18	0.90	1.01	86.6%
Front Street	MH 1A	156		19.408					7305		4.49				223	4.780	7528	3.08	81.15			85.93	0.40	375	110.89	1.00	1.11	77.5%

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Port Credit West Village Population Projections

Block A

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.32			0.08	
Units					
Population	55.13		0.00	4.20	59.33

Block B

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.16			0.04	
Units		0.00	219.00		
Population	27.56		591.30	2.10	620.96

Block C1

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				0.43	
Units					
Population	0.00		0.00	21.50	21.50

Block C2

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				0.70	
Units					
Population	0		0	35	34.85

Block C3

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				0.23	
Units					
Population	0		0	12	11.70

Block D

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)		0.25			
Units					
Population	0.00	44.10	0.00	0.00	44.10

Block E - Park

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units		0.00			
Population	0.00	0.00	0.00	0.00	0.00

Block F

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	1.65				
Units					
Population	288.75		0.00	0.00	288.75

Block G1

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.00				
Units					
Population	0.00	0.00	0.00	0.00	0.00

Block G2

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.22				
Units					
Population	39.03	0.00	0.00	0.00	39.03

Block G3

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				0.11	
Units			197.00		
Population	0.00	0.00	531.90	5.25	537.15

Block H1

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				0.11	
Units			124.00		
Population	0.00	0.00	334.80	5.50	340.30

Block H2

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units			126.00		
Population	0.00	0.00	340.20	0.00	340.20

Block I

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	1.38				
Units					
Population	241.15	0.00	0.00	0.00	241.15

Block J - Park

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units					
Population	0.00	0.00	0.00	0.00	0.00

Block K

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	1.65				
Units					
Population	288.75	0.00	0.00	0.00	288.75

Block L

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	1.65				
Units					
Population	288.75	0.00	0.00	0.00	288.75

Block M

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)		1.05			
Units					
Population	0.00	184.45	0.00	0.00	184.45

Block N - Park

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units					
Population	0.00	0.00	0.00	0.00	0.00

Block O1

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.68				
Units					
Population	118.30	0.00	0.00	0.00	118.30

Block O2

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.75				
Units					
Population	131.95	0.00	0.00	0.00	131.95

Block O3

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units			128.00		
Population	0.00	0.00	345.60	0.00	345.60

Block P1 - Towns

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.47				
Units					
Population	81.90	0.00	0.00	0.00	81.90

Block P2 - Highrises

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units			381.00		
Population	0.00	0.00	1028.70	0.00	1028.70

Block Q1

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units			238.00		
Population	0.00	0.00	642.60	0.00	642.60

Block Q2

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.78				
Units					
Population	136.59	0.00	0.00	0.00	136.59

Block R

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)	0.58				
Units					
Population	101.47	0.00	0.00	0.00	101.47

Block S - Park

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units					
Population	0.00	0.00	0.00	0.00	0.00

Block T

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)					
Units			253.00	0.43	
Population	0.00	0.00	683.10	21.50	704.60

Block U

	Towns	Stks &Backs	Apartments	Commercial/Retail	Total
Area (ha)				1.43	
Units			139.00		
Units			157.00		
Population	0.00	0.00	799.20	71.64	870.84

Total	7503.51

APPENDIX C

Hydraulic Modelling Analysis (AECOM)

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To:	Urbantech	Page	9
CC:			
Subject	Hydraulic Modelling Analysis – Imperial Oil, Region of Peel		
From	Benny Wan, P.Eng., Sogol Bandehali (EIT)		
Date	August 8, 2017	Project Number	60538792

INTRODUCTION

AECOM was retained to perform hydraulic analysis for determining the water infrastructure requirements for providing sustainable water service to the development located at the southwest corner of Mississauga Road and Lakeshore Drive West under the desired growth conditions. The purpose of this report is to summarize the findings of this analysis and confirm that the planning area may be serviced through the existing and future watermains, the sizing of the proposed watermains within the development and there are no significant off-site constraints, which may prohibit development.

Imperial Oil development includes 2,488 residential units and net site area of 200,056 m^2 (20 ha) located at the southwest corner of Mississauga Road and Lakeshore Drive West, Region of Peel. Figure 1 shows the location of the study area.

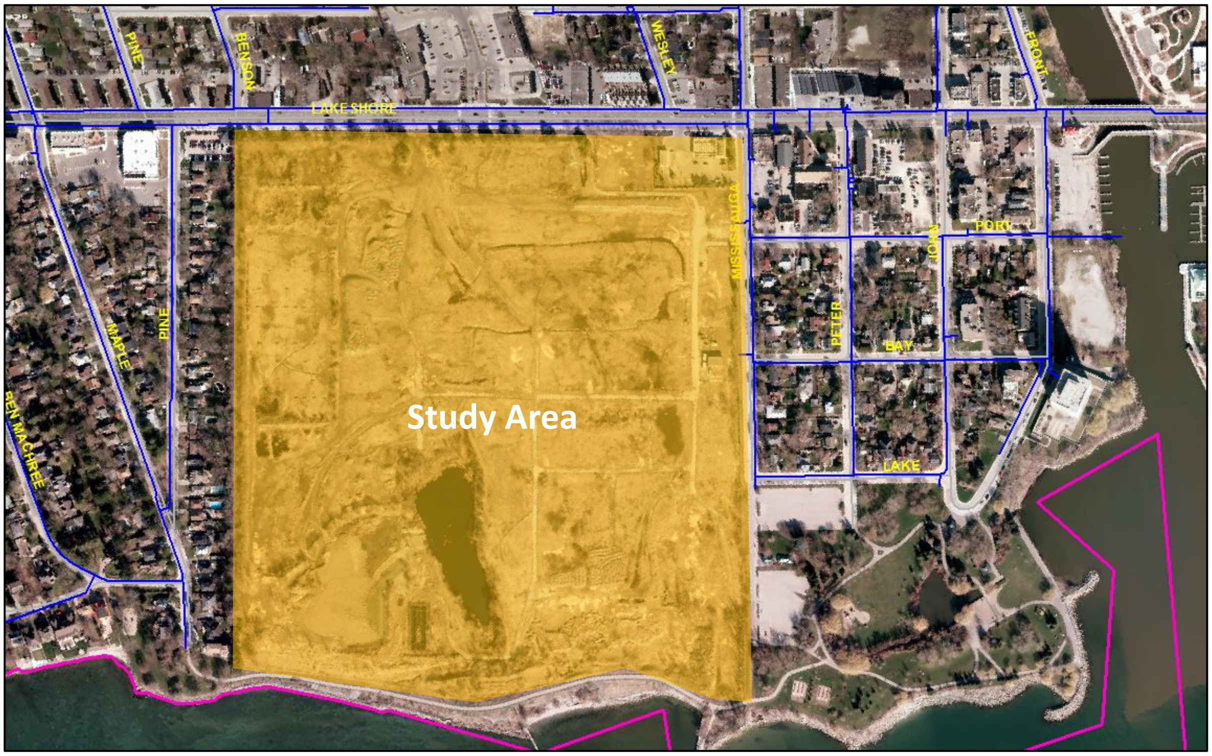

Figure 1 - Study Area

MODELLING PARAMETERS, CRITERIA, AND ASSUMPTIONS

AECOM received the necessary information provided by Urbantech on July 31, 2017. After a thorough review of the Peel water model and the information provided by Urbantech, the following subsections detailed the design criteria and the modelling methodology used for this analysis for requested design year of 2021, 2026, 2027 and 2041.

Connection to Existing Network

Based on the information provided, it was identified that the subdivision will obtain water service from the existing 300 mm watermain connecting to 150 mm watermain on Mississauga Road and Lake from east side of the development and to 300 mm watermain on Lakeshore Drive West from north side of the development. 300 mm watermain is used to simulate this development and the adequacy of this size can be confirmed under different condition such as fireflow.

The layout within the development is shown in Figure **2** based on topographical drawings provided by Urbantech. The modelling junctions that represented the Imperial Oil development are also shown in Figure **2**. The elevation for these junctions was updated in the hydraulic model based on the topology drawing.

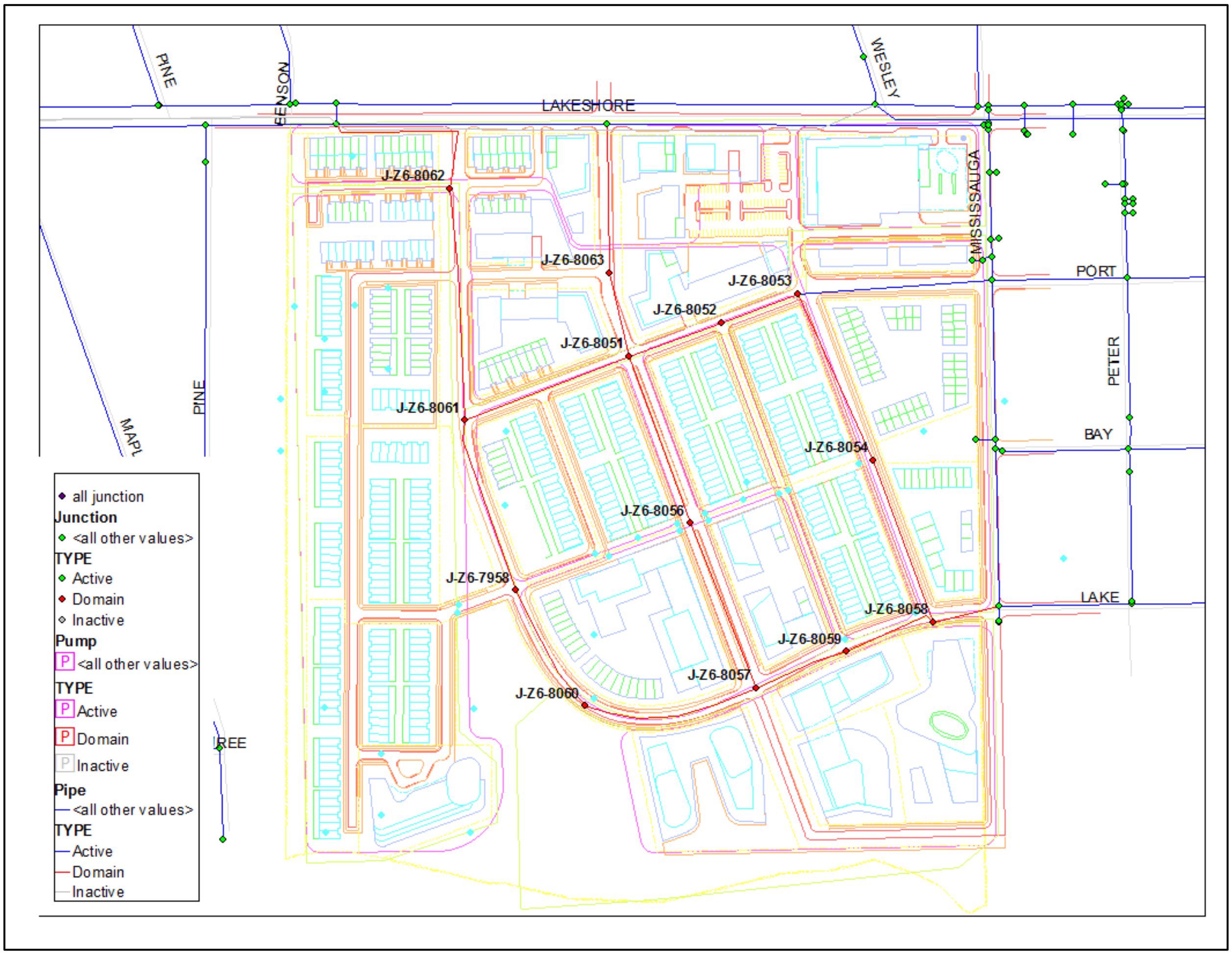

Figure 2 - Pipe Network in the study area

Design Criteria

The following design criteria and population information were used for the analysis.

Table 1 Region of Peel 2016 Masterplan Schematic Design Criteria

Criteria	Residential Population	Employment Force
Average Day Demand (ADD)(L/cap.day)	280	280
Maximum Day Factor (MDF)	2	1.4
Peak Hour Factor (PHF)	3	3

Table 2 Imperial Oil Population based on phasing

Design Year	Total Population
2021	1780
2026	7068
2027	7468

Water Demand

The total area of development is 20 ha; this is divided in 77% residential and 23% employment according to the master site plan drawing. The demand was calculated based on the population, which varied for each design year, and residential vs employment ratio. Subsequently, the demand was allocated to the assumed modelling junctions.

Table 3 Demand Summary

Design Year	Average Day Demand (ADD) (L/s)	Max day Demand (MDD) (L/s)	Peak Hour Demand (PHD) (L/s)
2021	5.8	10.7	17.3
2026	22.9	42.6	68.7
2027	24.2	45.0	72.6
2041	24.2	45.0	72.6

Table 4 Demand Allocation for 2021

2021		
	Residential Demand	**Employment Demand**
Average Day Demand (ADD) (L/s)	4.4	1.4
Maximum Day Demand (MDD) (L/s)	8.9	1.9
Peak Hour Demand (PHD) (L/s)	13.6	4.0

Table 5 Demand Allocation for 2026

2026		
	Residential Demand	**Employment Demand**
Average Day Demand (ADD) (L/s)	17.6	5.3
Maximum Day Demand (MDD) (L/s)	35.2	7.4
Peak Hour Demand (PHD) (L/s)	52.8	16.0

Table 6 Demand Allocation for 2027

2027		
	Residential Demand	**Employment Demand**
Average Day Demand (ADD) (L/s)	18.6	5.6
Maximum Day Demand (MDD) (L/s)	37.2*	7.9*
Peak Hour Demand (PHD) (L/s)	55.7	16.9

*The same demands calculated for 2027MDD within the development area was added to 2041MDD scenario which included the Region of Peel demands for the rest of the Region and it was assumed that there was no additional growth to the Imperial Oil lands Port Credit (West Village) area between 2027 and 2041.

The modelling results will be assessed based on the following criteria:

- Minimum acceptable pressure - 275 kPa (40 psi) (*Ministry of the Environment Design Guidelines for Drinking-Water Systems and Region of Peel Water System Design Criteria*)
- Maximum acceptable pressure - 700 kPa (100 psi) (*Ministry of the Environment Design Guidelines for Drinking-Water Systems and Region of Peel Water System Design Criteria)*
- Maximum acceptable velocity – 2 m/s (*Ministry of the Environment Design Guidelines for Drinking-Water Systems)*
- Fire demands – 25,020 L/min (417 L/s) (Region of Peel Public Works Watermain Design Criteria)
- Minimum pressure under maximum day demand plus fire flow - 140 kPa (20 psi) (*Ministry of the Environment Design Guidelines for Drinking-Water Systems and Region of Peel Water System Design Criteria*)

Scenarios

The following scenarios were used for the analysis

- **2021**
  - 2021 ADD/ MDD/ PHD / MDD + Fire Flow without the proposed 600 mm main on Lakeshore Road
- **2026**
  - 2026 ADD/ MDD/ PHD / MDD + Fire Flow without the proposed 600 mm main on Lakeshore Road
- **2027**
  - 2027 ADD/ MDD/ PHD / MDD + Fire Flow without the proposed 600 mm main on Lakeshore Road

- **2041**
    - 2041 MDD/ MDD + Fire Flow without the proposed 600 mm main on Lakeshore Road

The modelling analysis was completed based on the Region's all pipe water model. For each scenario, the minimum pressure for the areas that are within the vicinity of the development was reviewed under extended period simulation (EPS).

ANALYSIS OF MODELLING RESULTS

The following sections detail the results of the analysis completed for evaluating the impact of the Imperial Oil development on the Region's water system. According to the hydraulic modelling results, no serviceability issue within the development was indicated and there appeared to be no negative impact to the surrounding system after the growth. Under all scenarios, the development shows acceptable pressure and velocity using the 300 mm watermains within the development.

Serviceability to the Proposed Development

Table 7 demonstrates the average pressure at the junction representing the growth under all scenarios. Pressure within the development ranges between 74 psi and 87 psi; which is well within the 40 psi – 100 psi allowable range indicated that the development gets service using the 300mm main connecting to existing system and there will be no complication in velocity and pressure in this area.

Table 7 - Minimum Pressure Comparison in Different Scenarios within the Imperial Oil Development

Without 600 mm watermain on Lakeshore Drive West		
Scenarios		**Minimum Pressure (psi)**
2021	**ADD**	83.13
	MDD	86.1
	PHD	80.5
2026	**ADD**	81.21
	MDD	88.8
	PHD	83.9
2027	**ADD**	80.87
	MDD	88.8
	PHD	83.9
2041	**MDD**	93.4
	PHD	88.6

Hydraulic Implications to the Region's Water System

The following section summarizes the hydraulic implications in Zone 1 with the inclusion of the proposed development. Figure 3 displays modelling junctions in Zone 1, which the pressure was assessed during the analysis:

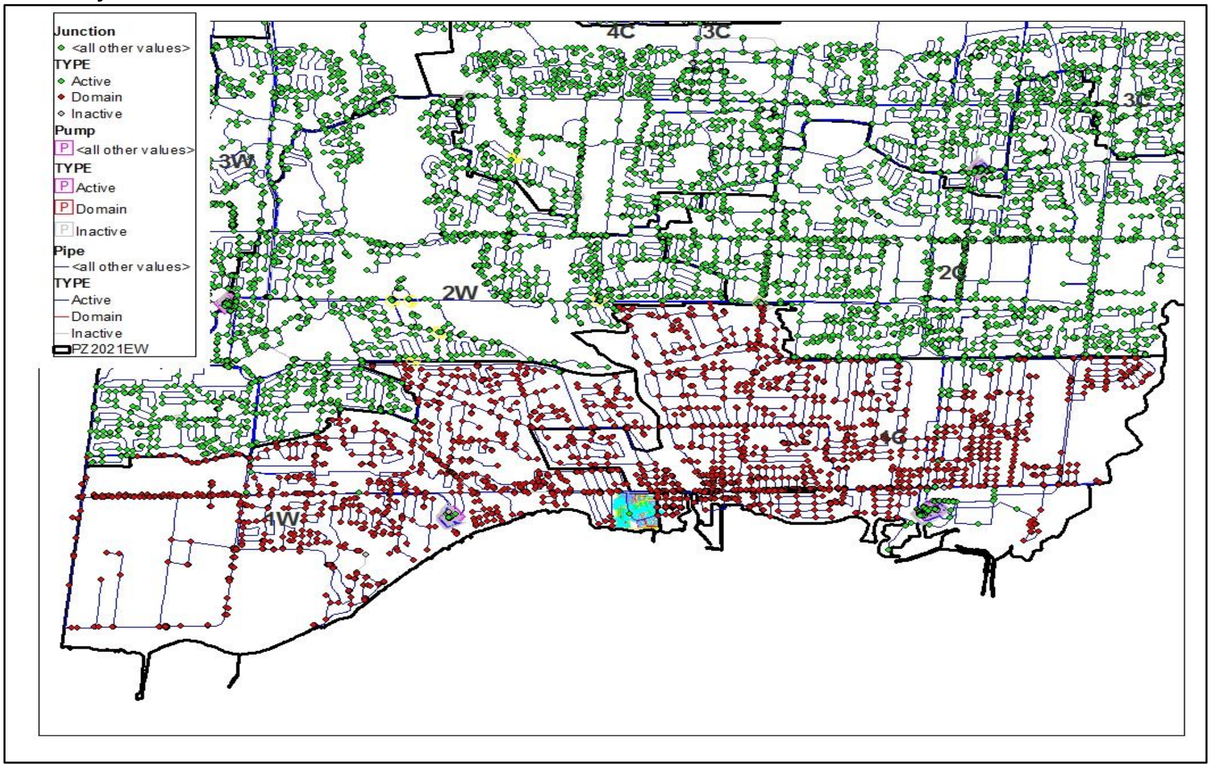

Figure 3 - Region of Peel Zone 1 Junctions

Table 8 - Minimum Pressure comparison in Different Scenarios for zone 1 Junctions

		BASE Scenario **Minimum Pressure without Proposed Development & without proposed 600 mm on Lakeshore Road (psi)**	**Minimum Pressure with Proposed Development (psi)** **(without 600 watermain on Lakeshore)**
2021	**ADD**	42.7	42.3
	MDD	37.9	37.8
	PHD	44.1	44.1
2026	**ADD**	39.8	39.5
	MDD	41.7	41.4
	PHD	39.6	39.1

2027	**ADD**	39.8	39.3
	MDD	41.7	41.4
	PHD	39.6	39.1
2041	**MDD**	44.9	44.7

According to the results stated in the above table, the growth has minimal effect on the minimum pressure (+/- 0.5 psi) in all of the scenarios in zone 1.

<u>Fire Flow Analysis</u>

Fire Flow analysis was completed to ensure the surrounding area of the development meets sufficient pressure and velocity during a fire event with the assumed size of 300 mm watermain within the development. The modelling results show that the assumed sizing of the watermains within the proposed development is sufficient to provide adequate supply during fire in this area. Table 9 summarizes the fire flow analysis results for the proposed development.

According to the fire flow analysis summary results, the Region's water system would provide adequate fire flow above 417L/s to proposed development while maintaining the minimum pressure at above 20 psi. In addition, the velocity in 300 mm watermains within the proposed development did not exceed the Region's design criteria of 2.0m/s.

Table 9 – Summary of Fireflow Results

Junction Within the Development	2021 MDD		2026 MDD		2027 MDD		2041 MDD	
	Residual Pressure (psi)	Available Flow at Hydrant (L/s)	Residual Pressure (psi)	Available Flow at Hydrant (L/s)	Residual Pressure (psi)	Available Flow at Hydrant (L/s)	Residual Pressure (psi)	Available Flow at Hydrant (L/s)
J-Z6-7958	81.92	751.73	86.06	779.45	85.99	775.52	92.51	910.66
J-Z6-8051	84.37	971.54	88.53	1003.78	88.46	999.03	94.99	1280.53
J-Z6-8052	86.69	915.62	90.85	962.27	90.78	957.88	97.31	1189.69
J-Z6-8053	87.07	950.02	91.24	994.80	91.17	990.35	97.70	1249.39
J-Z6-8054	88.28	843.43	92.44	892.51	92.37	888.23	98.90	1064.26
J-Z6-8056	84.71	814.20	88.86	841.75	88.79	837.53	95.32	1000.16
J-Z6-8057	86.95	887.55	91.10	913.77	91.03	909.31	97.56	1108.71
J-Z6-8058	88.33	946.16	92.49	991.71	92.42	987.24	98.95	1237.55
J-Z6-8059	87.69	872.36	91.85	916.48	91.78	912.20	98.31	1109.07
J-Z6-8060	86.13	781.78	90.27	803.17	90.20	798.94	96.72	935.51
J-Z6-8061	82.98	865.44	87.14	902.18	87.07	898.39	93.60	1104.54
J-Z6-8062	82.03	794.57	86.20	829.35	86.13	825.78	92.65	989.60
J-Z6-8063	84.15	908.18	88.32	946.11	88.25	940.63	94.77	1169.13

CONCLUSION

The hydraulic modelling results lead to the following conclusions:

- The hydraulic modelling results show that the Imperial Oil development can receive sufficient water service without 600 mm main on Lakeshore Drive West even under 2041 maximum day demand conditions.
- This development has minimal effect on the pressure in Zone 1 of the Region of Peel system and the Imperial Oil Development does not cause any negative impacts to the existing system.
- The assumed size for the watermains within the development is adequate to maintain the same level of service in Zone 1 area and the development can get adequate supply with a 300 mm watermain connecting to the existing system.

Recommendation:

This analysis shows proposed watermains are adequate to meet the growth in this area. However, it is recommended to include the proposed 300 mm watermain shown in Figure 4 to provide better system security.

Although the model used for this analysis was calibrated within the Region's acceptable accuracy, AECOM recommends hydrant flow test to be undertaken in order to further validate the hydraulic modelling results presented herein.

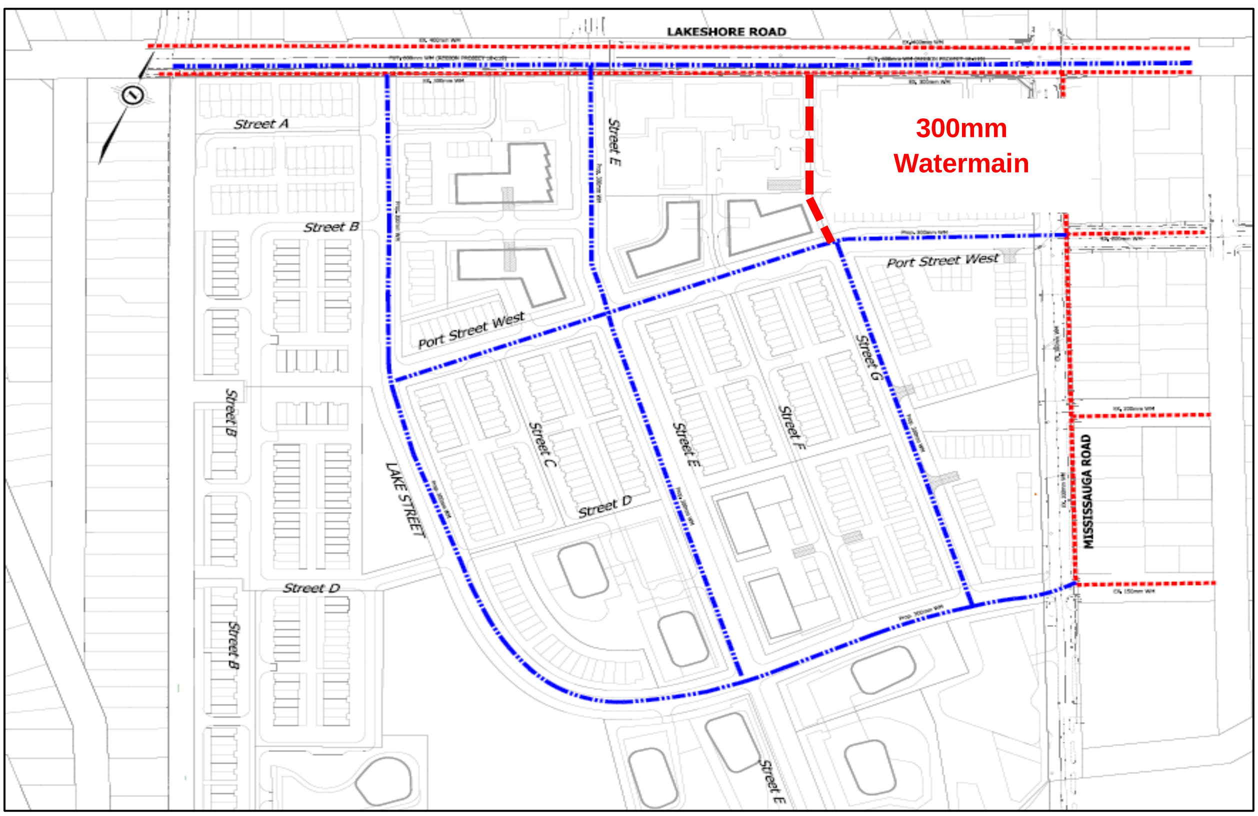

Figure 4 – Recommended 300 mm Watermain

APPENDIX D

Storm Servicing Design Calculations

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urbantech

STORM SEWER DESIGN SHEET

10 Year Storm

West Village Partners

City of Mississauga

PROJECT DETAILS

Project No: 16-489
Date: 1-Aug-17
Designed by: NM
Checked by: RM

JAN 6/16

DESIGN CRITERIA

Min. Diameter =	300	mm	Rainfall Intensity =	$\frac{A}{(Tc+B)^c}$	
Mannings 'n'=	0.013				
Starting Tc =	15	min	A =	1010	
			B =	4.6	
Factor of Safety =	10	%	c =	0.78	

NOMINAL PIPE SIZE USED

STREET	FROM MH	TO MH	AREA (ha)	RUNOFF COEFFICIENT "R"	'AR'	ACCUM. 'AR'	RAINFALL INTENSITY (mm/hr)	FLOW (m3/s)	CONSTANT FLOW (m3/s)	ACCUM. CONSTANT FLOW (m3/s)	TOTAL FLOW (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	INITIAL Tc (min)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONCENTRATION (min)	PERCENT FULL (%)
Street A	29	31	0.67	0.65	0.44	0.44	99.2	0.120			0.120	14.0	0.50	450	0.202	1.27	15.00	0.18	15.18	60%
	30	31	0.95	0.75	0.71	0.71	99.2	0.196			0.196	13.7	0.50	525	0.304	1.40	15.00	0.16	15.16	65%
	31	28	0.22	0.90	0.20	1.35	98.4	0.368			0.368	65.2	0.50	600	0.434	1.54	15.18	0.71	15.89	85%
	27	28	0.48	0.70	0.34	0.34	99.2	0.093			0.093	12.3	0.50	375	0.124	1.12	15.00	0.18	15.18	75%
	28	26	0.23	0.90	0.21	1.89	95.8	0.503			0.503	92.8	0.50	675	0.594	1.66	15.89	0.93	16.82	85%
	26	25				1.89	92.5	0.485			0.485	15.9	0.50	675	0.594	1.66	16.82	0.16	16.98	82%
Street C	25	24	0.25	0.90	0.23	2.11	92.0	0.540			0.540	79.5	0.50	675	0.594	1.66	16.98	0.80	17.78	91%
	23	24	0.66	0.80	0.53	0.53	99.2	0.145			0.145	12.5	0.50	450	0.202	1.27	15.00	0.16	15.16	72%
	24	22		0.90		2.64	89.4	0.656			0.656	43.6	0.50	750	0.787	1.78	17.78	0.41	18.19	83%
Street B	16	18	0.15	0.90	0.14	0.14	99.2	0.037			0.037	67.9	0.50	300	0.068	0.97	15.00	1.17	16.17	54%
	17	18	0.98	0.90	0.88	0.88	99.2	0.243			0.243	12.0	0.50	525	0.304	1.40	15.00	0.14	15.14	80%
	18	20				1.02	94.8	0.268			0.268	25.4	0.50	525	0.304	1.40	16.17	0.30	16.47	88%
	19	20	0.21	0.90	0.19	0.19	99.2	0.052			0.052	14.2	0.50	300	0.068	0.97	15.00	0.24	15.24	76%
	20	21	0.19	0.90	0.17	1.38	93.7	0.358			0.358	40.5	0.50	600	0.434	1.54	16.47	0.44	16.91	83%
	21	22		0.90		1.38	92.2	0.353			0.353	28.2	0.50	600	0.434	1.54	16.91	0.31	17.22	81%
Street C	22	15	0.13	0.90	0.12	4.14	88.2	1.013			1.013	73.1	0.50	900	1.280	2.01	18.19	0.61	18.79	79%
	14	15	0.62	0.90	0.56	0.56	99.2	0.154			0.154	12.7	0.50	450	0.202	1.27	15.00	0.17	15.17	76%
	15	13	0.13	0.90	0.12	4.81	86.4	1.154			1.154	67.0	0.50	900	1.280	2.01	18.79	0.55	19.35	90%
	12	13	0.95	0.90	0.86	0.86	99.2	0.236			0.236	47.6	0.50	525	0.304	1.40	15.00	0.56	15.56	77%
	13	11		0.90		5.67	84.8	1.335			1.335	25.8	0.50	975	1.585	2.12	19.35	0.20	19.55	84%
	11	9	0.24	0.90	0.22	5.88	84.3	1.377			1.377	75.0	0.50	975	1.585	2.12	19.55	0.59	20.14	87%
	9A	9	0.39	0.65	0.25	0.25	99.2	0.070			0.070	14.9	0.50	375	0.124	1.12	15.00	0.22	15.22	56%
	9	10				6.14	82.7	1.409	1.166	1.166	2.575	22.4	0.50	1200	2.757	2.44	20.14	0.15	20.29	93%
Mississauga Road	10	8		0.90		6.14	82.3	1.403		1.166	2.569	50.4	0.50	900x1500 (BOX)	3.152	2.33	20.29	0.36	20.65	81%
	7	8	0.87	0.65	0.57	0.57	99.2	0.156	0.120	0.120	0.276	9.5	0.50	525	0.304	1.40	15.00	0.11	15.11	91%
	8	6		0.90		6.70	81.4	1.515		1.286	2.801	75.6	0.50	900x1500 (BOX)	3.152	2.33	20.65	0.54	21.19	89%
	5	6	0.19	0.30	0.06	0.06	99.2	0.016	0.012	0.012	0.028	9.7	0.50	300	0.068	0.97	15.00	0.17	15.17	41%
	6	4		0.90		6.76	80.0	1.503		1.298	2.801	80.4	0.50	900x1500 (BOX)	3.152	2.33	21.19	0.57	21.77	89%
	3	4	0.54	0.65	0.35	0.35	99.2	0.097	0.075	0.075	0.172	8.8	0.50	450	0.202	1.27	15.00	0.12	15.12	85%
	4	2		0.90		7.11	78.7	1.554		1.373	2.927	32.2	0.50	900x1500 (BOX)	3.152	2.33	21.77	0.23	22.00	93%
Street A	32	33	0.72	0.65	0.47	0.47	99.2	0.129			0.129	15.6	0.50	450	0.202	1.27	15.00	0.21	15.21	64%
	33	35	0.26	0.90	0.23	0.70	98.4	0.192			0.192	73.7	0.50	525	0.304	1.40	15.21	0.87	16.08	63%
	34A	34	0.29	0.30	0.09	0.09	99.2	0.024			0.024	12.3	0.50	300	0.068	0.97	15.00	0.21	15.21	35%

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STORM SEWER DESIGN SHEET

10 Year Storm

West Village Partners

City of Mississauga

PROJECT DETAILS

Project No: 16-489
Date: 1-Aug-17
Designed by: NM
Checked by: RM

JAN 6/16

DESIGN CRITERIA

Min. Diameter = 300 mm
Mannings 'n'= 0.013
Starting Tc = 15 min
Factor of Safety = 10 %

Rainfall Intensity = $\frac{A}{(Tc+B)^c}$

A = 1010
B = 4.6
c = 0.78

NOMINAL PIPE SIZE USED

STREET	FROM MH	TO MH	AREA (ha)	RUNOFF COEFFICIENT "R"	'AR'	ACCUM. 'AR'	RAINFALL INTENSITY (mm/hr)	FLOW (m3/s)	CONSTANT FLOW (m3/s)	ACCUM. CONSTANT FLOW (m3/s)	TOTAL FLOW (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	INITIAL Tc (min)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONCENTRATION (min)	PERCENT FULL (%)
	34	35	0.22	0.90	0.20	0.29	98.3	0.078			0.078	10.9	0.50	375	0.124	1.12	15.21	0.16	15.37	63%
	35	36	0.16	0.90	0.14	1.13	95.1	0.299			0.299	77.9	0.50	600	0.434	1.54	16.08	0.85	16.93	69%
	36	37	0.10	0.90	0.09	1.22	92.2	0.313			0.313	47.5	0.50	600	0.434	1.54	16.93	0.52	17.44	72%
	37	39	0.10	0.90	0.09	1.31	90.5	0.330			0.330	50.1	0.50	600	0.434	1.54	17.44	0.54	17.98	76%
	38	39	1.30	0.80	1.04	1.04	99.2	0.286			0.286	11.5	0.50	600	0.434	1.54	15.00	0.12	15.12	66%
	39	46	0.10	0.90	0.09	2.44	88.8	0.602			0.602	51.8	0.50	750	0.787	1.78	17.98	0.48	18.47	76%
Street B	40	42	0.72	0.65	0.47	0.47	99.2	0.129			0.129	18.3	0.50	450	0.202	1.27	15.00	0.24	15.24	64%
	41	42	0.72	0.65	0.47	0.47	99.2	0.129			0.129	18.3	0.50	450	0.202	1.27	15.00	0.24	15.24	64%
	42	43	0.32	0.90	0.29	1.22	98.2	0.334			0.334	69.9	0.50	600	0.434	1.54	15.24	0.76	16.00	77%
	43	45		0.90		1.22	95.4	0.324			0.324	46.5	0.50	600	0.434	1.54	16.00	0.50	16.50	75%
	44	45	0.70	0.90	0.63	0.63	99.2	0.174			0.174	19.0	0.50	450	0.202	1.27	15.00	0.25	15.25	86%
	45	46	0.28	0.90	0.25	2.11	93.6	0.548			0.548	8.3	0.50	750	0.787	1.78	16.50	0.08	16.58	70%
Street A	46	48	0.13	0.90	0.12	4.66	87.3	1.131			1.131	73.6	0.50	900	1.280	2.01	18.47	0.61	19.08	88%
	47	48	0.41	0.90	0.37	0.37	99.2	0.102			0.102	19.0	0.50	375	0.124	1.12	15.00	0.28	15.28	82%
	48	49	0.07	0.90	0.06	5.10	85.6	1.211			1.211	37.3	0.50	975	1.585	2.12	19.08	0.29	19.37	76%
	49	54	0.05	0.90	0.05	5.14	84.8	1.210			1.210	29.8	0.50	975	1.585	2.12	19.37	0.23	19.61	76%
Street G	50	51	0.72	0.65	0.47	0.47	99.2	0.129			0.129	18.3	0.50	450	0.202	1.27	15.00	0.24	15.24	64%
	51	53	0.29	0.90	0.26	0.73	98.2	0.199			0.199	87.9	0.50	525	0.304	1.40	15.24	1.04	16.28	65%
	52	53	0.67	0.65	0.44	0.44	99.2	0.120			0.120	84.4	0.50	450	0.202	1.27	15.00	1.11	16.11	60%
	53	54	0.15	0.90	0.14	1.30	94.4	0.341			0.341	84.4	0.50	600	0.434	1.54	16.28	0.92	17.20	78%
Street A	54	2	0.09	0.90	0.08	6.52	84.1	1.524	1.075	1.075	2.599	47.8	0.50	900x1500 (BOX)	3.152	2.33	19.61	0.34	19.95	82%
Mississauga Road	2	1				13.63	78.2	2.959		2.448	5.407	166.3	0.50	900x2400 (BOX)	5.580	2.58	22.00	1.07	23.07	97%
Block 16	55	57	0.20	0.90	0.18	0.18	99.2	0.050			0.050	44.7	0.50	300	0.068	0.97	15.00	0.77	15.77	73%
	56	57	0.91	0.90	0.82	0.82	99.2	0.226			0.226	8.1	0.50	525	0.304	1.40	15.00	0.10	15.10	74%
	57	59	0.19	0.90	0.17	1.17	96.2	0.313			0.313	78.4	0.50	600	0.434	1.54	15.77	0.85	16.62	72%
	58	59	0.59	0.90	0.53	0.53	99.2	0.146			0.146	7.9	0.50	450	0.202	1.27	15.00	0.10	15.10	73%
	59	61		0.90		1.70	93.2	0.440			0.440	38.2	0.50	675	0.594	1.66	16.62	0.38	17.00	74%
	60	61	0.83	0.90	0.75	0.75	99.2	0.206			0.206	7.8	0.50	525	0.304	1.40	15.00	0.09	15.09	68%
	61	1		0.90		2.45	91.9	0.625	0.431	0.431	1.056	25.2	0.50	900	1.280	2.01	17.00	0.21	17.21	82%
Mississauga Road	1	HW2		0.90		16.08	75.8	3.385		2.879	6.264	65.0	0.50	900x3000 (BOX)	7.240	2.68	23.07	0.40	23.47	87%
Western Lane	62	63	0.95	0.65	0.62	0.62	99.2	0.170			0.170	53.0	0.50	450	0.202	1.27	15.00	0.70	15.70	84%
(D-1 through D-6)	63	64				0.62	96.5	0.166			0.166	51.7	0.50	450	0.202	1.27	15.70	0.68	16.38	82%
	64	65	1.27	0.65	0.83	1.44	94.1	0.377			0.377	53.2	0.50	600	0.434	1.54	16.38	0.58	16.95	87%
	65	66				1.44	92.1	0.369			0.369	59.7	0.50	600	0.434	1.54	16.95	0.65	17.60	85%

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STORM SEWER DESIGN SHEET

10 Year Storm

West Village Partners

City of Mississauga

PROJECT DETAILS

Project No: 16-489
Date: 1-Aug-17
Designed by: NM
Checked by: RM

JAN 6/16

DESIGN CRITERIA

Min. Diameter = 300 mm
Mannings 'n'= 0.013
Starting Tc = 15 min
Factor of Safety = 10 %

$$\text{Rainfall Intensity} = \frac{A}{(Tc+B)^{c}}$$

A = 1010
B = 4.6
c = 0.78

NOMINAL PIPE SIZE USED

STREET	FROM MH	TO MH	AREA (ha)	RUNOFF COEFFICIENT "R"	'AR'	ACCUM. 'AR'	RAINFALL INTENSITY (mm/hr)	FLOW (m3/s)	CONSTANT FLOW (m3/s)	ACCUM. CONSTANT FLOW (m3/s)	TOTAL FLOW (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	INITIAL Tc (min)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONCENTRATION (min)	PERCENT FULL (%)
	66	68	1.19	0.65	0.77	2.22	90.0	0.554			0.554	51.9	0.50	750	0.787	1.78	17.60	0.49	18.09	70%
	67	68	0.14	0.90	0.13	0.13	99.2	0.035			0.035	8.0	0.50	300	0.068	0.97	15.00	0.14	15.14	51%
	68	70	0.10	0.90	0.09	2.43	88.5	0.598			0.598	50.5	0.50	750	0.787	1.78	18.09	0.47	18.56	76%
	69	70	0.54	0.90	0.49	0.49	99.2	0.134			0.134	10.8	0.50	450	0.202	1.27	15.00	0.14	15.14	66%
	70	73				2.92	87.1	0.706	0.551	0.551	1.257	10.9	0.50	975	1.585	2.12	18.56	0.09	18.65	79%
	71	73	2.44	0.30	0.73	0.73	99.2	0.202	0.156	0.156	0.358	79.9	0.50	600	0.434	1.54	15.00	0.87	15.87	82%
	72	73	0.83	0.30	0.25	0.25	99.2	0.069	0.055	0.055	0.124	42.4	0.50	450	0.202	1.27	15.00	0.56	15.56	61%
	73	HW1				3.90	99.2	1.074		0.762	1.836	37.6	0.50	1200	2.757	2.44	15.00	0.26	15.26	67%

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F:\Projects\16-489W (West Village Partners - Port Credit)\Design\Storm\16-489 STM (10 yr):STM

PROJECT DETAILS	
Title1:	**STORM SEWER DESIGN SHEET**
Title2:	**100 Year Storm Capture**
Project Name:	**West Village Partners**
Municipality:	**City of Mississauga**
Project No:	**16-489**
Date:	**15-Aug-17**
Designed by:	**NM**
Checked by:	**RM**

IDF Parameters for Mississauga			
		10-yr	100-yr
$I=A/(T+b)^c$	A	1010	1450
	B	4.6	4.9
	C	0.78	0.78

Q=CiA

$Q_{100}=AR_{100}YR*(10000)*I*(1/(3600*1000))$ = $1/360*AR_{100}*I_{100}$

CAPTURE LOCATION	AREA ID	CAPTURE POINT	Area ha	R 10YR	R 100YR	AR 10YR	AR 100YR	Flow Length m	Velocity m/s	Tc* min	I_{10} mm/hr	I_{100} mm/hr	Q_{10} m3/s	Q_{100} m3/s	Q_{100}-Q_{10} m3/s	100YR Capture Flow m3/s
Lane (MH70 - MH72)	D-1 through D-6	D	4.19	0.70	0.88	2.93	3.67			18.56	87.1	123.7	0.709	1.260	0.551	0.551
Park (MH71)	P-2	P-2	2.44	0.30	0.38	0.73	0.92			15.00	99.2	140.7	0.202	0.358	0.156	0.156
Street C (MH9 - MH10)	A-1 through A-19	A	7.45	0.68	0.85	5.07	7.13			20.14	82.7	117.6	1.164	2.329	1.166	1.166
Mississauga Road (MH7)	B-22	B-22	0.87	0.65	0.81	0.57	0.71			15.00	99.2	140.7	0.156	0.276	0.120	0.120
Mississauga Road (MH5)	P-3	P-3	0.19	0.30	0.38	0.06	0.07			15.00	99.2	140.7	0.016	0.028	0.012	0.012
Mississauga Road (MH3)	B-23	B-23	0.54	0.65	0.81	0.35	0.44			15.00	99.2	140.7	0.097	0.171	0.075	0.075
Street A (MH54 - MH2)	B-1 through B-21	B	8.57	0.74	0.93	6.38	7.72			19.61	84.1	119.6	1.490	2.564	1.075	1.075
MH61 - MH1	C-1 through C-6	C	2.72	0.80	1.00	2.18	2.72			17.00	91.9	130.6	0.556	0.986	0.431	0.431
Southern End of Catchment P-4	P-4	P-4	0.83	0.30	0.38	0.25	0.32			15.00	99.2	140.7	0.069	0.123	0.055	0.055

*Where available, Tc is calculated from design sheet or overland flow calculation

Tc calcs *where Tc = starting Tc + flow length/velocity*
(starting Tc = 10min)

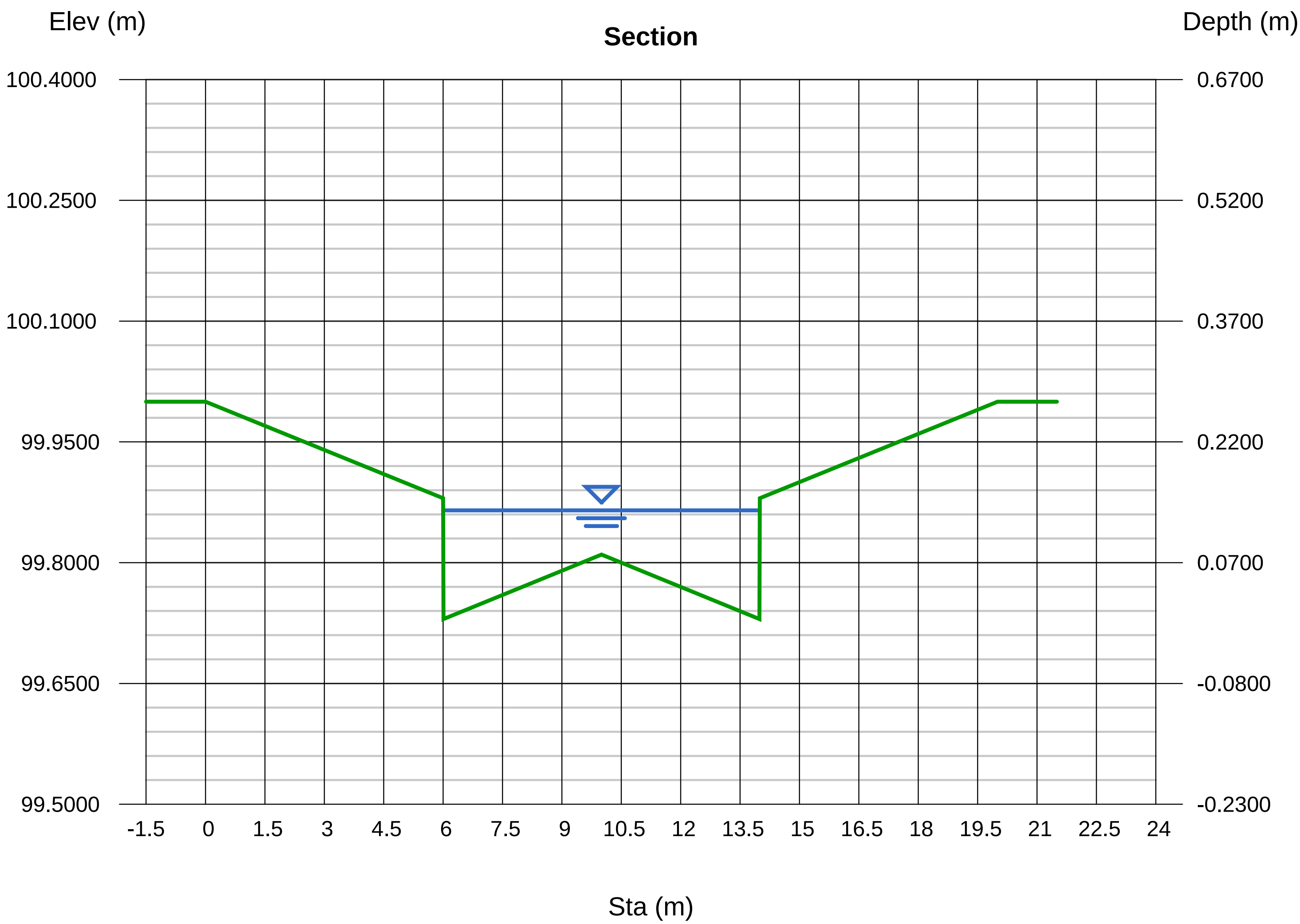
Assumed Velocities for Calculation of time of Concentration
Pipe Flow Velocity= 2.0 m/s
OLF Velocity= 1.5 m/s
External Flow Velocity= 0.25 m/s

Channel Report

20m ROW Capacity (8m pavement)

User-defined		**Highlighted**	
Invert Elev (m)	= 99.7300	Depth (m)	= 0.1350
Slope (%)	= 1.0000	Q (cms)	= 1.1926
N-Value	= Composite	Area (sqm)	= 0.7593
		Velocity (m/s)	= 1.5707
Calculations		Wetted Perim (m)	= 8.2522
Compute by:	Q vs Depth	Crit Depth, Yc (m)	= 0.1829
No. Increments	= 10	Top Width (m)	= 7.9980
		EGL (m)	= 0.2608

(Sta, El, n)-(Sta, El, n)...
(0.0000, 100.0000)-(6.0000, 99.8800, 0.025)-(6.0100, 99.7300, 0.013)-(10.0000, 99.8100, 0.013)-(13.9900, 99.7300, 0.013)-(14.0000, 99.8800, 0.013)-(20.0000, 10



Channel Report

Block 19 Swale

Triangular			**Highlighted**		
Side Slopes (z:1)	=	3.0000, 3.0000	Depth (m)	=	0.2700
Total Depth (m)	=	0.3000	Q (cms)	=	0.4273
			Area (sqm)	=	0.2187
Invert Elev (m)	=	5.0000	Velocity (m/s)	=	1.9540
Slope (%)	=	1.0000	Wetted Perim (m)	=	1.7076
N-Value	=	0.013	Crit Depth, Yc (m)	=	0.3000
			Top Width (m)	=	1.6200
Calculations			EGL (m)	=	0.4648
Compute by:		Q vs Depth			
No. Increments	=	10