Functional Servicing Report

Pinnacle International (Ontario) Limited

Part of Subdivision (Phase IV Part 2 and Phase V)
OPA/Rezoning for Intensification

Prepared for Pinnacle International
by IBI Group Professional Services (Canada) Inc.
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Table of Contents

1. INTRODUCTION 1

1.1 General 1

1.2 Subdivision Design 1

1.3 Subdivision Staging (Phase IV Part 2 and Phase V) 1

2. PROPERTY DESCRIPTION 2

3. EXISTING DATA COLLECTION/BACKGROUND REPORTS 2

4. SITE SERVICING 3

4.1 Proposed Site Servicing 3

4.1.1 Phase IV Part 2 3

4.1.2 Phase V 3

4.2 Sanitary Drainage System 4

4.2.1 WEST TRUNK 4

4.2.2 EAST TRUNK 5

4.3 Storm Water Management 5

4.3.1 SITE CHARACTERISTICS 5

4.3.2 STORMWATER MANAGEMENT DESIGN CRITERIA 6

4.3.3 STORMWATER MANAGEMENT PLAN 6

4.3.4 ON-SITE RETENTION AND DETENTION STORAGE REQUIREMENTS 7

4.3.5 EROSION AND SEDIMENT CONTROLS 11

4.4 Water Distribution System (see Figure 4) 13

4.5 Site Grading 13

5. CONCLUSION 14

Table of Contents (continued)

TABLES

Table 1. Comparison of Phase IV Part 2 Approved Subdivision VS Proposed Sanitary Flows
Table 2. Comparison of Phase V Approved Subdivision VS Proposed Sanitary Flows
Table 3: Results of Water Balance Analysis for Phase IV Part 2 Development
Table 4: Results of Water Balance Analysis for Phase V Development
Table 5: Pre-Development Design Peak Flows for Phase IV Part 2 and Phase V Developments
Table 6: Estimates of On-Site Detention Storage Requirements for Phase IV Part 2 and Phase V Developments
Table 7: Water Demand Table for Phase IV Part 2 and Phase V Developments

FIGURES

Figure 1: Site Plan Area/Proposed Roads Plan – Phase IV Part 2 and Phase V
Figure 2: Sanitary Drainage Areas and Sewer System – Phase IV Part 2 and Phase V
Figure 3: Minor Storm Drainage Areas and Sewer System – Phase IV Part 2 and Phase V
Figure 4: Watermain Distribution System
Figure 5: Road Grades and Overland Flow Route – Phase IV Part 2 and Phase V

APPENDICES

Appendix "A": Phase IV Part 2 and Phase V – Staging Plan of Subdivision
Appendix "B": Suite Summary
Appendix "C": Sanitary Sewer Design Sheet and Site Statistics
Appendix "D": Storm Sewer Design Sheet
Appendix "E": Pre-Development OTTHYMO Model Output
Appendix "F": Post-Development OTTHYMO Model Output

Table of Contents (continued)

Appendix "G": Hydraulic Modelling Analysis (prepared by AECOM), January 14, 2011

Appendix "H": Detailed Demand Calculations

Appendix "I": Approved Subdivision Sanitary Sewer Design Sheet and Drainage Area

Appendix "J" Functional Servicing and Storm Water Management Report Prepared by Masongsong Associates Engineering Ltd – Dec. 2017

1. INTRODUCTION

1.1 General

IBI Group has been retained by Pinnacle International to provide planning and engineering services for Blocks 16, 17, and portions of Blocks 1, 2, and 8 (site) (site illustrated on Figure 1) of Pinnacle lands located at the north-west corner of Hurontario Street and Eglinton Avenue West in the Hurontario District of Mississauga.

This Functional Servicing Report (FSR) has been prepared to demonstrate the servicing feasibility (storm, sanitary and water) of site in support of processing of OPA/ZBLA. The FSR will also be used to support any site alterations that may be required for the proposed site plan applications.

1.2 Subdivision Design

Engineering drawings were prepared and approved in July 2013. This FSR will take the approved engineering design and demonstrate the servicing feasibility of the proposed intensification will function under the current approved servicing.

1.3 Subdivision Staging (Phase IV Part 2 and Phase V)

Proposed site alterations are concentrated fully on Blocks 16 and 17 and on portions of Blocks 1 and 2. Figure 1 displays the proposed condominium towers. The proposed modification includes increased in Gross Floor Area.

The proposed subdivision concept contemplated development of 3 Development Blocks as shown in Appendix “A”. The original staging plan is still in effect and the proposed changes affects Stage 4 and a portion of Stage 3.

With the proposed changes, the revised roadway configuration consists of an internal road that runs from west to east, connecting Foursprings Avenue to Hurontario Street. The private access road runs along the north limit of Blocks 2 and 16. It provides internal access to the proposed condo tower in Block 16. The road can be seen in the site plan in Figure 1.

2. PROPERTY DESCRIPTION

The Pinnacle International total ownership comprises 14.78 ha (37 acres) of land located at the northwest corner of Hurontario Street (The Kings Highway No. 10) and Eglinton Avenue West in the City of Mississauga. The site is located approximately 1.5 km due north of the Mississauga City Centre within the Hurontario corridor which is planned for intensification and for higher order transit improvements.

The proposed intensification is concentrated fully on Blocks 16 and 17 and on portions of Blocks 1 and 2 in the above noted property holdings.

3. EXISTING DATA COLLECTION/BACKGROUND REPORTS

The drawings, reports, or detailed technical material which have been obtained from the City or other consultants are identified below. This information has been used in the development of this study and forms input to the servicing feasibility conclusions outlined in this report.

- IBI Group Phase II approved engineering drawings, approved July 2013.
- IBI Group Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision, dated January 2011
- Approved storm and sanitary sewer design sheets, approved July 2013. The approved storm sewer design sheet may be found in Appendix “D”.
- Suite summary and commercial areas, prepared by Richmond Architects in February 2020, used as parameters to update the sanitary sewer design sheet. This may be in found in Appendix “B”.
- Aecom Technical Memorandum, Jan. 14, 2011 prepared to update their previous assessment of off-site water and sewer capacity and availability now or in the future, of the water and sanitary infrastructure to meet this project’s demands. This includes updates to water modeling as requested by the Region of Peel in comments on the previous submission. This memorandum is attached as Appendix “G”. When referring to this memorandum, note that it was prepared for the original subdivision plan.
- Hydrant Flow Test prepared by GTA Waterworks on June 2017. Accompanied by Detailed Hydrant Flow calculation in Table F1 prepared by Masongsong Engineering Associates Engineering Limited on December 2017. This may be in found in Appendix “H”.

4. SITE SERVICING

4.1 Proposed Site Servicing

Based on the Region of Public Works Design, Specification and Procedures manual an equivalent population is calculated for sanitary sewerage and water servicing demand.

4.1.1 PHASE IV PART 2

This phase of the development is comprised of one Tower (35 stories - Building 3). The Tower will have a 6-storey podium at its base. The tower will have a total contemplated number of residential units of 406 units, with 0.09 ha of commercial/retail space area. This equivalent population for this phase is:

Site Specific Population = 1.68 ppu (1 bedroom) x 233 + 2.54 ppu (2 + bedroom) x 173
= 830 persons

Commercial Population = 917.2 m^2 x (50 persons/10,000m^2)
= 5 persons

Therefore, the total population for this Phase IV Part 2 development is **835 persons.**

Table 1. Comparison of Phase IV Part 2 Approved Subdivision VS Proposed Sanitary Flows

Approved Subdivision Sanitary Population - Phase IV

STREET	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population
TOWER	2.7	277		747.90
OFFICE/COMMERCIAL	50.00	0.518		25.90
AREA 2B				
TOWER	2.70	208		561.60
OFFICE/COMMERCIAL	50.00	0.049		2.45
AREA 2C				
TOWER	2.70	155		418.50
OFFICE/COMMERCIAL	50.0	0.051		2.55
AREA 2D				
TOWER	2.7	132		356.40
TH	3.80	8		30.40
Total Phase IV				1,371.90

Proposed Sanitary Population - Phased IV

STREET	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population
TOWER (1 bedroom)		1.68	233.0		391.44
TOWER (2+ bedrooms)		2.54	173.0		439.42
OFFICE/COMMERCIAL		50.00	0.1		4.59
INFILTRATION	1.25			1.25	835.45
AREA 2B					
TOWER		2.7	468.0		1,263.60
OFFICE/COMMERCIAL		50.0	0.0234		1.2
INFILTRATION	0.61			0.61	1,264.77
Total Phase IV					2,100.22

Revised Flows for Phase IV Part I as per approved Functional Servicing and Storm Water Management Report Prepared by Masongsong Associates Engineering Ltd – Dec. 2017 (see appendix “J”).

4.1.2 PHASE V

The total number of residential units contemplated for this phase 949 units, with 0.30 ha area of commercial/retail space. This results in an equivalent population:

Site Specific Population = 1.68 ppu (1 bedroom) x 458 + 2.54 ppu (2 + bedroom) x 491
= 2,017 persons

Commercial Population = 8,490.7 m^2 x (50 persons/10,000m^2)
= 42 persons

Therefore, the total population for this Phase IV Part development is **2,059 persons.**

Table 2. Comparison of Phase V Approved Subdivision VS Proposed Sanitary Flows

Approved Subdivision Sanitary Population Phase V

STREET	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population
AREA 1A				
TOWER	2.70	277		747.90
OFFICE/COMMERCIAL	50.00	0.80		39.95
AREA 1C				
TOWER	2.70	177		477.90
OFFICE/COMMERCIAL	50.00	0.04		2.18
Total Phase V - North				**1,267.93**
AREA 1B				
TOWER	2.70	277		747.90
OFFICE/COMMERCIAL	50.00	0.54		27.00
Total Phase V - South				**774.90**
Total Phase V				**2,042.83**

Proposed Sanitary Population - Phase V

STREET	A (ha)	Density Persons Per (ha) or Unit	Number of Units or Floor	Cummul. Area (ha)	Cummul. Population
AREA 1A (Phase V - BLDG 1 & 2)					
TOWER (1 bedroom)		1.68	304		510.72
TOWER (2+ bedrooms)		2.54	310		787.40
OFFICE/COMMERCIAL		50.00	0.32		16.23
Total Part V - North					**1,314.35**
AREA 1B (Phase V - BLDG 3)					
TOWER (1 bedroom)		1.68	154		258.72
TOWER (2+ bedrooms)		2.54	181		459.74
OFFICE/COMMERCIAL		50.00	0.52		26.23
Total Phase V - South					**744.69**
Total Phase V					**2,059.03**

Refer to Appendix “I” for Approved Subdivision Sanitary Sewer Design Sheet and Drainage Area.

4.2 Sanitary Drainage System

The approved subdivision sanitary sewer design was updated to incorporated as constructed information. The sewer design incorporated the additional proposed intensification flows to ensure the sanitary system has sufficient capacity. Sanitary flows from the subject subdivision (Phase IV Part 2 and Phase V), based on the site statistics identified can be found in Appendix B of this report.

4.2.1 WEST TRUNK

Currently the two phases are serviced by 250mm diameter sanitary sewer along Watergarden Drive, which ultimately discharges to the west trunk which consists of a 750 mm diameter sewer located at the north property limit in an easement on the west side of the creek. Phase IV Part 2 is serviced by one (1) control manhole. Alternatively, Phase V is serviced by three (3) control manholes. Two are serviced from Watergarden Drive and the third is serviced from the south via Littlecreek Road. Phase IV Part 2 will utilize the one connection, while Phase V will utilize one of the connections to Watergarden and the other from Littlecreek. The unused connection on Watergarden will be removed, and the leads grouted.

The current capacity of the west trunk is 212.2 L/s less the 76L/s which have been allowed for the ultimate design build out. The current design flows provide total development flow of 55.5L/s which is a slight increase from the design flows of 53L/s for the ultimate build out the subdivision. This is approximately 5% increase in flows but is still well below then the allowed 76L/s design flows. See Appendix “C” for details.

4.2.2 EAST TRUNK

As per the approved master sewerage plan part of the Phase V development is serviced via 250mm diameter sanitary sewer along Littlecreek Road. The sewer ultimately discharges into an existing 300 mm diameter sanitary sewer located in an easement on the east side of the creek on the south side of Eglinton Avenue West. The sewer allowance for the Pinnacle development is 37.5 l/sec. The flows will decrease from the 31 L/s to 30.4 l/s, as shown on the Sanitary Sewer Design Sheet (Appendix "C"). This will enable the servicing of the development (Phase I and a portion of Phase II) as shown on Figure 2.

4.3 Storm Water Management

4.3.1 SITE CHARACTERISTICS

Under the existing conditions, the Development Block 9 (formerly known as Development Block 4 in Phase II FSR dated January 2011) and Development Block 1 Phase III and Block 2 Phase IV Part 1 (formerly known as the western portions of the Development Block 2 and Block 3 respectively in Phase II FSR dated January 2011 and referred to in this report as Drainage Areas 2A and Block 3A as illustrated in **Figure 3**) are either being developed or have been approved by the regulatory agencies for development in accordance with the approved Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision (IBI Group, January 2011). The Phase IV Part 2 (legally described as Block 16 and part of Blocks 2 and 8) and Phase V (legally described as Block 17 and part of Block 1) developments are located to the north of the Phase I development in the Cooksville Creek Watershed and referred to as Drainage Area 3B & Drainage Area 2B respectively (see **Figure 3**) in this Report.

The Phase IV Part 1 (Drainage Area 3A, consisting of Drainage Area 3A1 & 3A2 as shown in **Figure 3**) development plan has been approved by regulatory agencies and is currently being constructed. According to the approved Functional Servicing and Stormwater Management Report for Phase IV Part 1 (Masongsong Associate Engineering Ltd., June 2017, on behalf of for Mondiale Development Limited), however, the Phase IV Part 1 stormwater management systems have not been designed to accommodate the existing/post-development runoff from the northeast corner of Phase IV Part 1 (Drainage Area 3A2). Instead, under the approved Phase IV Part 1 stormwater management plan, it is proposed to direct storm runoff from the northeast corner of Phase IV Part 1 into Phase IV Part 2 (Drainage Area 3B shown in **Figure 3**). As a result, the stormwater management plan for the Phase IV Part 2 development has to accommodate & provide all required stormwater management control for the post-development runoff from the northeast corner of Phase IV Part 1 (Drainage Area 3A2).

According to the findings of the Phase II Environmental Site Assessment for part of Lot 1, Conc. 1, WHS, designated as parts 2 to 6 on Plan 43R-24436 and Part 1 on Plan 43-R-24983 (Terraprobe Limited, Oct. 23, 2008), clayey silt is dominant soil with low hydraulic conductivity. Therefore, the subject site would not be considered as an effective groundwater recharge area. In addition, based on discussions with staff at the Credit Valley Conservation (CVC), the site is not officially identified by the CVC as a significant infiltration and groundwater recharge area.

The findings of the site geotechnical investigation indicate that the site hydrogeology is dominated by the Halton Till which has a very low hydraulic conductivity and the bedrock of the Georgian Bay Formation which similarly precludes the free flow of groundwater. Pockets of fluvial deposits of cohesionless sand or silt were found separating the glacial till from the bedrock. The cohesionless deposit is wet and dense to very dense where found. Observations in the boreholes indicated that groundwater is within 2 m below the existing ground surface although little flowing groundwater was observed. Where water entered boreholes, however, it was in limited quantity associated with the cohesionless sand and silt locally found over the bedrock.

4.3.2 STORMWATER MANAGEMENT DESIGN CRITERIA

The approved Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision (IBI Group, Jan. 2011) had included the intensification areas (Phase IV Part 2 and Phase V) as part of Phase II. In accordance with the approved Functional Servicing Report for Phase II and the City of Mississauga's comments on OZ/OPA 18 11 on August 29 & December 21, 2018 (also confirmed by Ghazwan Yousif, M. Sc., P. Eng. at the Transportation and Work Department on Jan. 13, 2019), the stormwater management design criteria are identified & refined as follows for the Phase IV Part 2 (including the northeast corner of Phase IV Part 1, i.e. Drainage Area 3A2 as illustrated in **Figure 3**) and Phase V developments:

1. *Water Quality Control*: the stormwater quality control is to be provided through a development charge payment to the City. Therefore, no additional on-site water quality control is required.

2. *Water Quantity Control*: to control the 1:100 year post-development runoff peak discharges from the development blocks to their corresponding 1:2 year pre-development levels.

3. *Runoff Volume Reduction*: to retain the first 5 mm of the stormwater runoff on site through the infiltration, evapotranspiration and rainwater reuse measures.

4. *Construction*: to provide necessary erosion and sediment controls for implementation during construction to prevent or minimize erosion potential and soil migration from the development site to adjacent lands and receiving waters.

4.3.3 STORMWATER MANAGEMENT PLAN

Since the City of Mississauga has adopted the Green Development Strategy (July 7, 2010) and the Stage One Green Development Standards (October 2012), it is required to implement sustainable technologies to manage stormwater runoff on site. To promote the sustainable and environmentally friendly development concept, low impact development (LID) measures were examined for potential applications within the Phase IV Part 2 and Phase V developments, in compliance with the City's Green Development Standards, although it is subject to the detailed design at the site plan/building permit application stage.

The LID measures mitigate potential negative impact of increased runoff / pollution by managing runoff as close to its source as possible through on-site infiltration, rainwater reuse / harvesting, and evapotranspiration control techniques. The LID control measures emphasize conservation and use of existing natural features integrated with distributed, small-scale water controls to closely mimic the natural hydrologic patterns, to the extent possible, under different land use settings.

The findings and conclusions of the recent research on the LID techniques (such as the Evaluation of an Extensive Greenroof, the Evaluation of Underground Stormwater Infiltration Systems, and the Performance Evaluation of Rainwater Harvesting Systems completed by the Toronto and Region Conservation Authority in July 2006, June 2011 and Feb. 2013 respectively) indicate that there are many positive environmental benefits associated with applications of the LID techniques such as protection of the downstream resources, abatement of pollution, water and energy conservation, improvement of water quality and natural habitat, improvement of flooding and erosion conditions downstream, groundwater recharge and improvement of aesthetics within the stream systems.

To promote the sustainable development concept and address the City's comments on OZ/OPA 18 11 on August 29 and December 21, 2018, the following stormwater management plan (subject to the detailed design at the site plan/building permit application stage) is proposed for the Phase IV Part 2 development (including the 0.14 ha northeast corner of Phase IV Part 1 / Drainage Area 3A2 as illustrated in **Figure 3**) and the Phase V development, in accordance with the approved Functional Servicing Report for Phase II OPA/Rezoning/Draft Plan of Subdivision:

a) To enhance runoff retention on site and reduce the amount of runoff leaving the development sites, efforts have been made during preparation of the current intensification plan not only to meet the development needs, but also to minimize total hard surface (impervious) area and to maximize total landscape (pervious) area.

   Under the current development plan, the landscape area occupies approximately 16.9% of the 0.9169 ha Phase IV Part 2 development (including the northeast corner of Phase IV Part 1), and 18.5% of the 0.9767 ha Phase V development. These percentages are relatively high, considering the nature of the proposed mixed use development.

b) Under the current development plan, a significant portion of the Phase IV Part 2 and Phase V developments is occupied by the proposed buildings (approximately 32.2% for the Phase IV Part 2 development and approximately 60.9% for the Phase V development). As currently proposed, the multi-level underground parking structures are proposed within the Phase IV Part 2 and Phase V developments. Since the parking structures are shallow in depth from the ground surface and their footprints occupy the entire Phase IV Part 2 (also into the northeast corner of Phase IV Part 1) and Phase V development blocks, it significantly limits types of LID measures that are physically feasible or suitable to be used within the developments.

   Considering high imperviousness and constraints of the proposed development, green roofs, landscaped terraces and outdoor recreation with landscape features are proposed within the Phase IV Part 2 and Phase V developments (in addition to the relatively large landscape area proposed) to increase evapotranspiration and runoff retention on site. As currently proposed, the building area that is covered by proposed green roofs, landscaped terraces and outdoor recreational landscape features is approximately 23.9% of total building area for the Phase IV Part 2 development and 13.3% for the Phase V development, subject to the detailed design.

c) To retain the first 5.0 mm of runoff for the balance of impervious areas, a rainwater harvesting tank (cistern) is proposed within the underground parking structures of the Phase IV Part 2 and Phase V developments (subject to the detailed design) for the purpose of irrigation, floor cleaning, toilet flushing, or other building maintenance functions such as mechanical cooling.

d) For the purpose of water quantity control, both major and minor runoff flows (up to the 1:100 year storm) from the Phase IV Part 2 development (including the northeast corner of Phase IV Part 1) and from the Phase V development will be directed to and attenuated through a stormwater detention facility (equipped with flow control devices together with the emergency overflow outlet and maintenance accesses) before discharging controlled peak flows (up to the 1:2 year pre-development levels under the 1:100 year design storm) into the municipal storm sewer system on Watergarden Drive (see **Appendix "D"**).

   As currently proposed, the rainwater harvesting tank within the Phase IV Part 2 development and within the Phase V development has priority to receive storm runoff flow (through internal storm pipes). When the rainwater harvesting tank is completely filled up, the excessive runoff (overflow) will spill into/toward the stormwater detention facility for peak flow attenuation before discharging controlled peak flows into the existing municipal storm sewer systems.

e) To provide erosion and sediment control, several erosion and sediment control measures are proposed to be implemented during construction.

4.3.4 ON-SITE RETENTION AND DETENTION STORAGE REQUIREMENTS

Stormwater Retention Storage Requirement for the Phase IV Part 2 Development: In addition to large landscape areas proposed, under the stormwater management plan, green roofs, landscaped terraces and outdoor recreational landscape features are also proposed within the Phase IV Part 2 development to increase runoff retention and evapotranspiration on site.

With the proposed green roofs, landscaped terraces and other landscape measures, water balance analysis was conducted to retain the first 5 mm runoff on site for the Phase IV Part 2 development (including the northeast corner of Phase IV Part 1), and the results are summarized and presented in Table 3.

Table 3. Results of Water Balance Analysis for Phase IV Part 2 Development

Type of Land Use within Expansion Area	Surface Area of Landuse (m^2)	Landuse Area as Percentage of Entire Area (-)	Initial Abstraction for Landuse (mm)	Initial Abstraction over Entire Area (mm)
	Part I: Water Balance Analysis with Proposed Green Roofs and Landscape Terraces/Features			
Roofs and Other Hard Surface Areas	6,913	75%	0.0	0.0
Green Roofs , Landscape Terraces/Features and Landscape Areas	2,256	25%	5.0	1.2
Total =	9,169	100%	NOT OK	1.2 < 5.0
Type of Landuse (Stormwater Retention Measures)	Retention Storage Required for Landuse (mm)	Retention Storage Required for Landuse (m^3)	Retention Storage Required over Entire Sarea (mm)	
	Part II: Additional Retention Storage Required to Meet Runoff Volume Reduction Design Criteria			
Roofs and Other Hard Surface Areas	5.0	34.6	3.8	
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	0.0	
Total =		34.6	3.8	
Type of Landuse (Stormwater Retention Measures)	Retention Storage Provided for Landuse (mm)	Retention Storage Provided for Landuse (m^3)	Initial Abstraction & Retention Storage Provided for Landuse (mm)	Initial Abstraction & Retention Provided over Entire Area (mm)
	Part III: Water Balance Analysis under Stormwater Management Plan			
Roofs and Other Hard Surface Areas to Rainwater Harvesting Tank 1	5.1	35.5	5.1	3.9
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	5.0	1.2
Total =		35.5	OK	5.1 > 5.0

The results of the water balance analysis indicate that, even with the proposed landscape areas, green roofs, landscaped terraces and outdoor recreational landscape features, the runoff volume reduction design criteria cannot be satisfied for the Phase IV Part 2 development without additional runoff retention measures (see Part I of Table 3), with retention storage shortage of approximately 34.6 m^3 (see Part II of Table 3).

To meet the runoff volume reduction design criteria, under the stormwater management plan, runoff from the Phase IV Part 2 development (including the northeast corner of Phase IV Part 1) will be collected and conveyed through internal storm pipes into a rainwater harvesting tank (referred to as Rainwater Harvesting Tank 1 in this Report) for purposes of irrigation, floor cleaning, toilet flushing, and/or other building maintenance functions (such as mechanical cooling among others), subject to the detailed design.

Under the current stormwater management plan, total design retention storage of 35.5 m^3 (see Part III of Table 3) is proposed for Rainwater Harvesting Tank 1, and it is higher than additional retention storage of 34.6 m^3 required for retaining the first 5.0 mm of runoff for the development. In another word, the desired runoff volume reduction control design criteria can be satisfied for the Phase IV Part 2 development with the proposed stormwater management plan in place.

Stormwater Retention Storage Requirement for the Phase V Development: To increase stormwater runoff retention on site and reduce runoff volume from leaving the development site, in addition to large landscape areas proposed within the development, under the stormwater management plan, green roofs, landscaped terraces and outdoor recreation with landscape features are also proposed within the Phase V development.

With the proposed green roofs, landscaped terraces and other landscape measures, water balance analysis was conducted for the purpose of retaining the first 5 mm of runoff on site within the Phase V development, and the results are summarized in Table 4.

Table 4. Results of Water Balance Analysis for Phase V Development

Type of Land Use within Expansion Area	Surface Area of Landuse (m^2)	Landuse Area as Percentage of Entire Area (-)	Initial Abstraction for Landuse (mm)	Initial Abstraction over Entire Area (mm)
	Part I: Water Balance Analysis with Proposed Green Roofs and Landscape Terraces/Features			
Roofs and Other Hard Surface Areas	7,173	73%	0.0	0.0
Green Roofs , Landscape Terraces/Features and Landscape Areas	2,594	27%	5.0	1.3
Total =	9,767	100%	NOT OK	1.3 < 5.0
Type of Landuse (Stormwater Retention Measures)	Retention Storage Required for Landuse (mm)	Retention Storage Required for Landuse (m^3)	Retention Storage Required over Entire Sarea (mm)	
	Part II: Additional Retention Storage Required to Meet Runoff Volume Reduction Design Criteria			
Roofs and Other Hard Surface Areas	5.0	35.9	3.7	
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	0.0	
Total =		35.9	3.7	
Type of Landuse (Stormwater Retention Measures)	Retention Storage Provided for Landuse (mm)	Retention Storage Provided for Landuse (m^3)	Initial Abstraction & Retention Storage Provided for Landuse (mm)	Initial Abstraction & Retention Provided over Entire Area (mm)
	Part III: Water Balance Analysis under Stormwater Management Plan			
Roofs and Other Hard Surface Areas to Rainwater Harvesting Tank 2	5.1	36.5	5.1	3.7
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	5.0	1.3
Total =		36.5	OK	5.1 > 5.0

Even with the proposed green roof, landscaped terraces and other landscaped features, the results of the water balance analysis indicate that the runoff volume reduction control design criteria cannot be satisfied for the Phase V development without additional runoff retention measures (see Part I of Table 4), with retention storage shortage of approximately 35.9 m^3 (see Part II of Table 4).

To satisfy the runoff volume reduction control design criteria, runoff from the Phase V development will be collected and conveyed (via internal storm pipes) into a rainwater harvesting tank (referred to as Rainwater Harvesting Tank 2 in this Report) for the purposes of irrigation, floor cleaning, toilet flushing, and/or other building maintenance functions, subject to the detailed design.

Under the current development and stormwater management plan, the design stormwater retention storage of 36.5 m^3 (see Part III of Table 4) is proposed for Rainwater Harvesting Tank 2, and it is slightly higher than the additional retention storage requirement of 35.9 m^3 to retain the first 5 mm of runoff within the development. Therefore, the storm runoff volume reduction control design criteria can be satisfied for the Phase V development with the proposed stormwater management plan.

Pre-Development Runoff Peak Flows: As part of the current regulatory floodplain mapping study for the Cooksville Creek, a detailed hydrologic analysis for the Watershed had been completed by R. V. Anderson Associates Ltd. in February 1996 in the study entitled “Cooksville Creek Floodline Mapping Study”, using the OTTHYMO computer program. The same hydrologic model established in the Cooksville Creek Floodline Mapping Study was applied in the approved Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision (IBI Group, Jan. 2011) and in this report to estimate the pre-development runoff peak flows for the development intensification blocks under the different design storms. A hard copy of the pre-development OTTHYMO model output is included in **Appendix “E”**.

The results of the detailed hydrologic analysis indicate that the pre-development unit runoff peak flows are 30.1, 83.1 and 151.5 l/s/ha under the 1:2, 1:10 and 1:100-year design storms respectively. Considering the development areas of 0.9169 ha and 0.9767 ha for the Phase IV Part 2 (Drainage Area 3B and 3A2 as shown in **Figure 3**, including the northeast corner of Phase IV Part 1) and Phase V (Drainage Area 2B) developments respectively, their corresponding pre-development runoff peak flows are calculated and summarized in Table 5 for the 1:2, 1:10 and 1:100-year design storms. To satisfy the desired water quantity control design criteria, it is required to control the 1:100 year post-development runoff peak discharges from the Phase IV Part 2 and Phase V developments to their corresponding 2-year pre-development levels of 0.028 m^3/s and 0.029 m^3/s respectively (see Table 5).

Table 5. Pre-Development Design Peak Flows for Phase IV Part 2 and Phase V Developments

Drainage Area Identification Number (-)	Drainage Area (ha)	Total Imperviousness (-)	Pre-Development Design Peak Flows (m^3/s)		
			2-Year Design Storm	10-Year Design Storm	100-Year Design Storm
Area 2B (Phase V)	0.9767	73.4%	0.029	0.081	0.148
Area 3B and 3A2 (Phase IV Part 2 and Northeast Corner of Phase IV Part 1)	0.9169	75.4%	0.028	0.076	0.139

Stormwater Detention Storage Requirements: To satisfy the desired water quantity control design criteria, under the proposed stormwater management plan, both major and minor storm runoff flows (up to the 1:100 year design storm) from the Phase IV Part 2 development and from the Phase V development will be directed to and attenuated via an on-site stormwater detention facility such as a stormwater detention tank located next to / directly above the rainwater harvesting tank within the underground parking structures, subject to the detailed design at the site plan and building permit application stage.

As a part of the stormwater detention facility within the Phase IV Part 2 and Phase V developments (referred to in this Report as Stormwater Detention Facility 1 and 2 respectively), flow control device (together with an emergency overflow outlet and the maintenance accesses among others) must be installed as the outlet of the stormwater detention facility to limit the 1:100 year post-development runoff peak discharges from the Phase IV Part 2 and Phase V developments into the municipal storm sewer systems on Watergarden Drive to their corresponding 1:2 year pre-development levels of 0.028 m^3/s and 0.029 m^3/s respectively (see Table 5).

The OTTHYMO computer program has been used to conduct the hydrologic analysis, determine runoff peak flows, and estimate stormwater detention storage requirements for the on-site detention facilities. The OTTHYMO computer program is an event-based hydrologic model. It has been widely used in similar analysis in Ontario and recognized as one of the reliable modeling tools to estimate the hydrologic response of rural and urban catchments to the different design and actual storms. The input of the OTTHYMO program mainly includes the meteorological and physiographic data to describe hydrologic and hydraulic characteristics of catchments, pipes/channels, reservoirs and the stormwater management facilities.

The main post-development hydrologic model parameters and the estimated design stormwater detention storage required for Phase IV Part 2 and Phase V developments are summarized and presented in Table 6. A copy of the post-development OTTHYMO model output is included in **Appendix "F"**.

Considering the development area of 0.9169 ha for the Phase IV Part 2 development (Drainage Areas 3B and 3A2) and 0.9767 ha for the Phase V development (Drainage Area 2B), the results of the post-development hydrologic analysis (see **Appendix "F"** for details) indicate that, under the current development plan, the design detention storage requirement is approximately 473 m^3 (see Table 6) for Stormwater Detention Facility 1 for the Phase IV Part 2 development (including the northeast corner of Phase IV Part 1) and 497 m^3 (see Table 6) for Stormwater Detention Facility 2 for the Phase V development to control the 1:100-year post-development runoff peak discharges to their corresponding 2-year pre-development levels of 0.028 and 0.029 m^3/s respectively.

Table 6. Estimates of On-Site Detention Storage Requirements for Phase IV Part 2 and Phase V Developments

Drainage Area Identification Number	Drainage Area (ha)	Total Imperviousness	2-Year Design Storm	10-Year Design Storm	100-Year Design Storm
Area 2B (Phase V)	0.9767	73.4%	Storage Requirement for Stormwater Detention Facility 2 (m^3)		
			190	340	497
			Uncontrolled Post-Development Peak Discharges (m^3/s)		
			0.211	0.365	0.552
			Controlled Post-Development Peak Discharges (m^3/s)		
			0.012	0.020	0.029
			Pre-Development Peak Flows (m^3/s)		
			0.029	0.081	0.148
Area 3B and 3A2 (Phase IV Part 2 and Northeast Corner of Phase IV Part 1)	0.9169	75.4%	Storage Requirement for Stormwater Detention Facility 1 (m^3)		
			190	320	473
			Uncontrolled Post-Development Peak Discharges (m^3/s)		
			0.203	0.350	0.526
			Controlled Post-Development Peak Discharges (m^3/s)		
			0.011	0.019	0.028
			Pre-Development Peak Flows (m^3/s)		
			0.028	0.076	0.139

It should be noted that, because the detailed intensification plans for the Phase IV Part 2 and Phase V developments are not finalized at this time, the above estimates of stormwater retention/detention storage requirements for rainwater harvesting tanks/stormwater detention facilities are preliminary in nature, and subject to the detailed design. When Phase IV Part 2 and Phase V are to be developed, it must proceed with the site plan/building permit application/approval process, including the detailed design and preparation of a stormwater management report in accordance with the stormwater management plan and design criteria outlined in this Report.

At the detailed design stage, the rainwater harvesting tanks and the stormwater detention facilities must be designed by qualified professional mechanical and structural engineers, and coordinated with qualified professional architect, landscape architect and civil engineer to satisfy the stormwater management design criteria specified in this Report.

4.3.5 EROSION AND SEDIMENT CONTROLS

The following erosion and sediment controls are proposed for implementation during construction to minimize erosion potential and soil migration from the site to adjacent lands and/or receiving waters:

- Install silt fence at the downslope side of disturbed areas and snow fence (if necessary) along the perimeter of the development site, prior to the start of construction.
- Install stone mud mats at all construction entrances.
- Stockpile topsoil at designated locations and at least 15 m away from the top bank of the watercourse. Stockpiles will be contained by silt fences on the downslope side.

- Accumulated silt shall be removed from all sediment control devices as required during construction and disposed of in locations approved by the City of Mississauga and CVC.

- All exposed soils are to be stabilized and vegetated as soon as possible using seed and mulch application on 100 mm of topsoil, as directed by the engineer.

- All catch basins are to be fitted with sediment control devices as directed by the engineer and in accordance with City standard requirements.

- Half bulk head to be installed in storm manholes immediately upstream from outfall structures and removed after all building construction and landscaping activity has been completed.

- Additional erosion/sediment controls may be required on site as determined by the engineer.

- No construction activity/machinery shall intrude beyond the silt/snow fence or property limit. All construction vehicles shall enter and leave the site via the designated entrances.

- All regraded areas that are not occupied by dwellings, roads, sidewalks, driveways, park, and other services shall be covered by 100 mm topsoil, and sodded/seeded immediately after completion of final grading operations, as directed by the engineer.

- All temporary erosion and sediment controls must be installed prior to the commencement of site grading, must be inspected on a regular basis and after every rainfall event, and must be cleaned and maintained as required to prevent the migration of sediment from the site.

- All temporary erosion and sediment controls must be removed after construction and once the site has been stabilized to the City's satisfaction. All areas disturbed by erosion/sediment control devices are to be restored with 100 mm topsoil and sodded/seeded after construction.

- The contractor shall keep public roadways free of debris during construction. Any material tracked from the site shall be promptly removed from roadways at the contractor's expenses.

- All material and workmanship shall conform to the current OPSD and standards endorsed by the City, the CVC and other regulatory agencies.

- The contractor is responsible to locate and protect existing all existing utilities and municipal services and make arrangements with utility companies prior to construction.

- All excavations shall be in accordance with the Ontario "Occupational Health and Safety Act", and other federal and provincial regulations related to construction projects.

4.4 Water Distribution System (see Figure 4)

The subject development is currently serviced with three (3) 200mm connection (one two the north for Phase IV – Part II and two to the south for Phase V off the existing 300mm watermain on Watergarden Drive.

The detailed demand calculations in the AECOM Hydraulic Modelling Analysis (dated January 14, 2011, see Appendix "G") was used as a basis to which the additional proposed intensification would contribute to the subdivision water demand. Using the Peel Region Water Demand Criteria, the estimated water demand is summarized in the Table 7 below and detailed in Appendix H.

Table 7. Water Demand Table for Phase IV Part 2 and Phase V Developments

	Typical Water Demand			Short Term Water Demand		
	Avg. Day (L/s)	Max Day (L/S)	Peak Hour (L/S)	Avg. Day	Max Day	Peak Hour
Phase IV Part II	4.26	8.49	12.79	6.20	8.53	12.79
Phase V	6.68	13.28	20.05	9.69	13.37	20.05
Total Water Demand	**10.95**	**21.77**	**32.84**	**15.89**	**21.89**	**32.84**

The Maximum day demand is based on the subdivision's residential water demand and commercial average day demand for both developments. We have also looked at the short-term water demand which is sometimes greater. In this instance the short term governs so the total Maximum day demand of 21.89 L/s will be used for domestic flows.

The required fire flow of 83.3 L/s for both Phase IV-Part II and Phase V have been obtained from the Fire Underwriter's Survey (FUS).

Further to the above a hydrant flow test, enclosed in Appendix H, was performed in June 2017 to obtain the available municipal flow on Watergarden Drive. The test confirms that the existing 300mm diameter watermain on Watergarden Drive is capable of delivering a fire flow of 14,358 L/min (239.3L/s) at a minimum pressure of 140 KPa, which satisfies both FUS and ISO fire flows superimposed on the max-day domestic consumption rate 5,802 L/min (96.70 L/s). The value for the maximum day is comprised from Phase V (being the larger of the two phases) with the fire flow from Phase V (13.37 + 83.33 = 96.70L/s).

As per the Ontario Building Code, structures greater than 84m are to be provided with two independent fire supply lines which can be isolated on separate system. As both developments exceed this height (with the smallest being 32 storeys) a second connection to Phase V is required. Therefore, we have proposed a new fire connection from Little Creek for Phase V (see Figure 4 for proposed location). This connection is to be detailed as part of Site Plan Application.

4.5 Site Grading

The subdivision is currently constructed. The site development will have to be graded to conform to the subdivision general grading. There are no issues anticipated with grading.

5. CONCLUSION

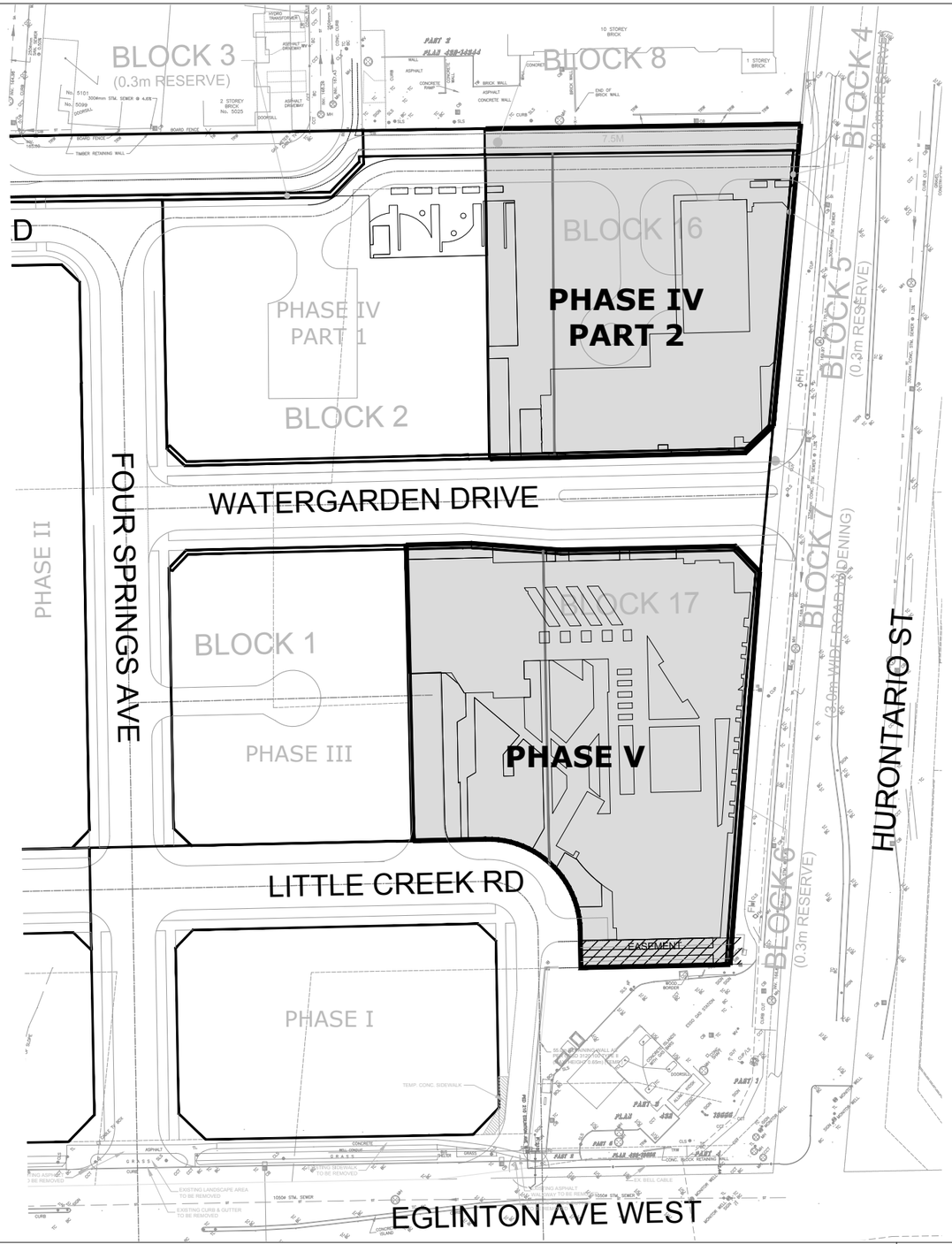
The foregoing clearly demonstrates that the proposed intensification for Phase IV – Part II (Blocks 16, 17), and Phase V (portions of Block 1, 2, and 8) development of the Pinnacle International site located at the north-west corner of Hurontario Street and Eglinton Avenue West can be adequately serviced with sanitary sewer, storm drainage and watermain systems to support the proposed OPA/ZBLA. Proposed servicing, road grades and Block grading will also be compatible with the existing development to the North and Phase I lands owned by the applicant to the South.

Respectfully submitted by:

IBI Group Professional Services (Canada) Inc.

Angelo Covello, P. Eng.
Associate Director - Practice Lead, Civil Engineering

Nicky Chang, Ph. D.
Water Resources Group



LEGEND:

PROPOSED INTENSIFICATION AREA

EASEMENT

20 10 0 20 40 60

1:1500 (m)

PINNACLE
INTERNATIONAL

TITLE:

SITE PLAN AREA / PROPOSED ROADS PLAN
PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	FEBRUARY, 2020
FIGURE No:	FIGURE 1

IBI
9133 Leslie Street
Suite 200
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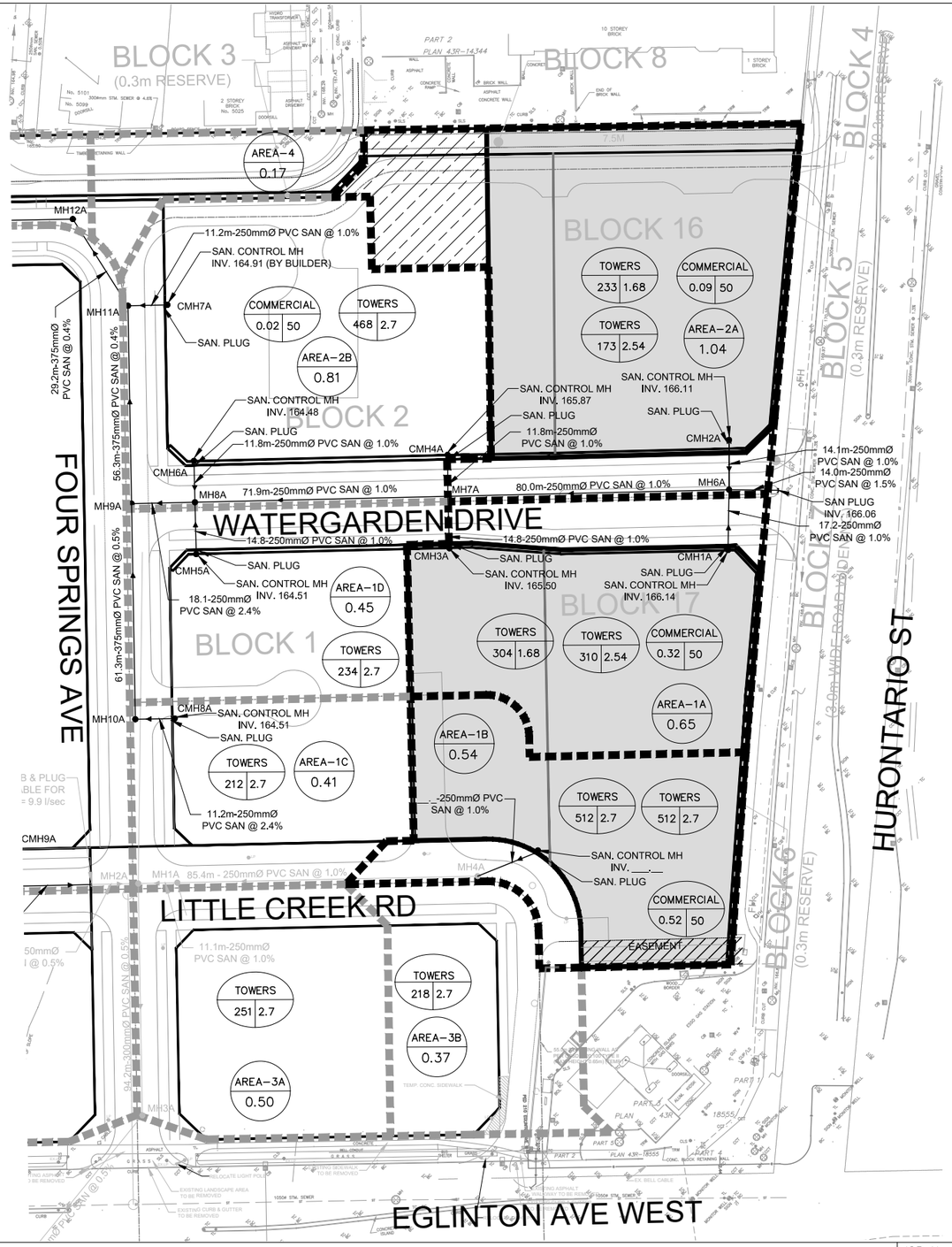
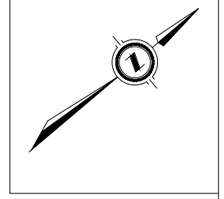
J:\108686_Pinn_Uptown\10.0 Reports\10.11 FSR\Working Files - Appendices and Figures\14604-FSR-PHASE II-FIG-1.dwg 2020-02-19 - 2:45pm





LEGEND:

- SANITARY DRAINAGE AREA DIVIDE (PROPOSED)
- SANITARY DRAINAGE AREA DIVIDE (EXISTING)
- MH1A — PROPOSED SANITARY SEWER & MH.
- SANITARY SEWER & MH. (FUTURE PHASES)
- EASEMENT
- EX. MH4 — EX. SANITARY SEWER & MH.
- CMH7A — PROPOSED SANITARY SEWER BLOCK CONNECTION & CONTROL MANHOLE
- TH 2.5 3 — DENOTES TYPE OF BUILDING; POPULATION PER UNITS; NUMBER OF UNITS OR FLOOR AREA
- B4 1.14 — DENOTES BLOCK NAME/ NUMBER; AREA IN HECTARES
- PROPOSED INTENSIFICATION AREA
- PORTION OF PHASE IV-PART I

20 10 0 20 40 60
1:1500 (m)

PINNACLE
INTERNATIONAL

TITLE:

SANITARY DRAINAGE AREAS AND SEWER SYSTEM PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	FEBRUARY, 2020
FIGURE No:	FIGURE 2

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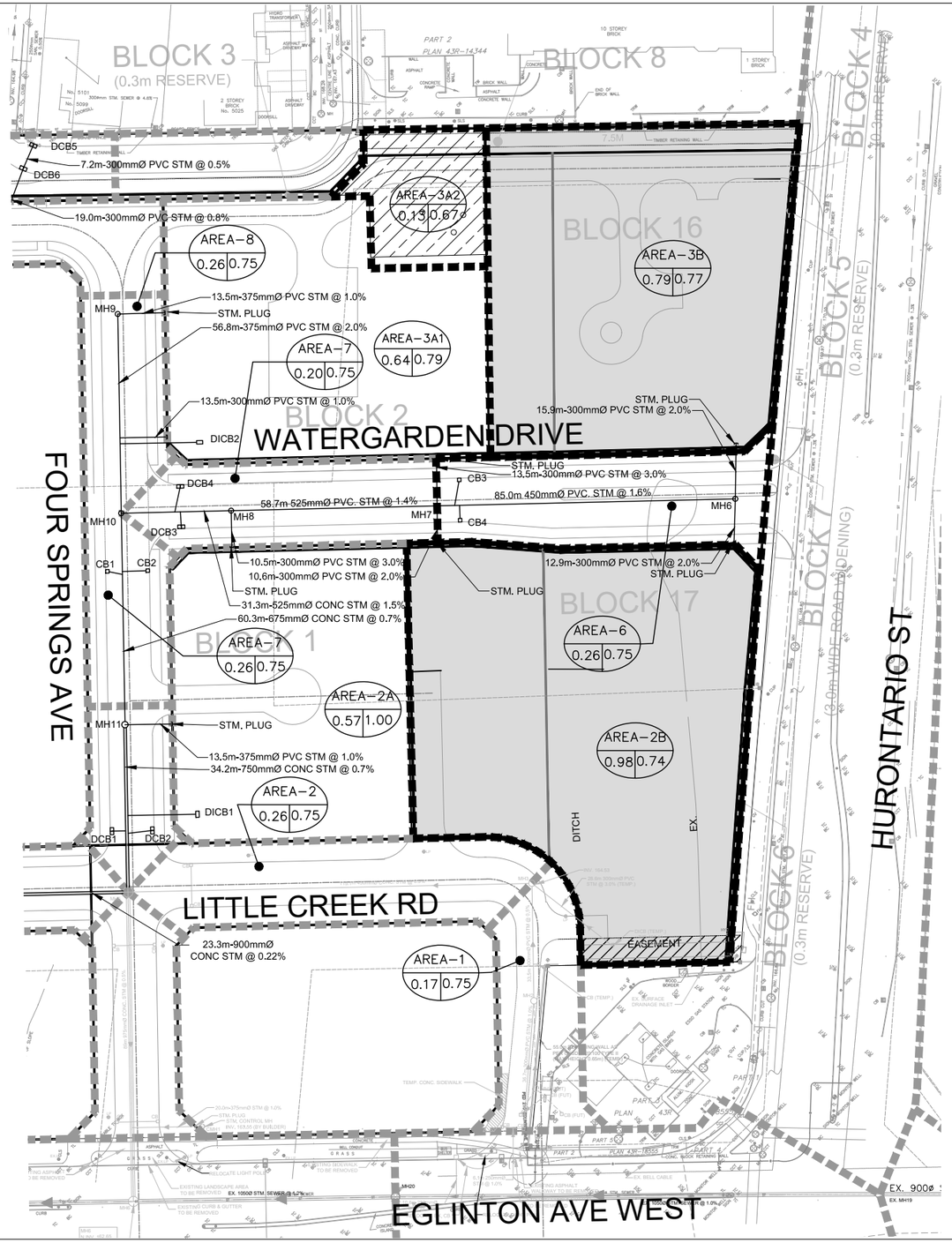
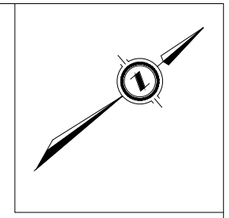
J:\108686_Pinn_Uptown\10.0 Reports\10.11 FSR\Working Files - Appendices and Figures\14604-FSR-PHASE II-FIG-2.dwg 2020-02-15 - 4:02pm





LEGEND:

- STORM DRAINAGE AREA DIVIDE (EXISTING)
- STORM DRAINAGE AREA DIVIDE (PROPOSED)
- EX. MH1 — EXISTING STORM SEWER
- MH1 — PROPOSED STORM SEWER SERVICING
- CONTROL MH (BY BLOCK DEVELOPER)
- EASEMENT
- 1 / 0.55 | 0.55 — DENOTES STORM DRAINAGE AREA / DENOTES STORM RUNOFF COEFFICENT / DENOTES DRAINAGE AREA IN HECTARES
- PROPOSED INTENSIFICATION AREA
- PORTION OF PHASE IV–PART I
- MH1 — STORM SEWER SERVICING (FUT. PHASES)

20 10 0 20 40 60
1:1500 (m)

PINNACLE
INTERNATIONAL

TITLE:

MINOR STORM DRAINAGE AREA AND SEWER SYSTEM PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	FEBRUARY, 2020
FIGURE No:	FIGURE 3

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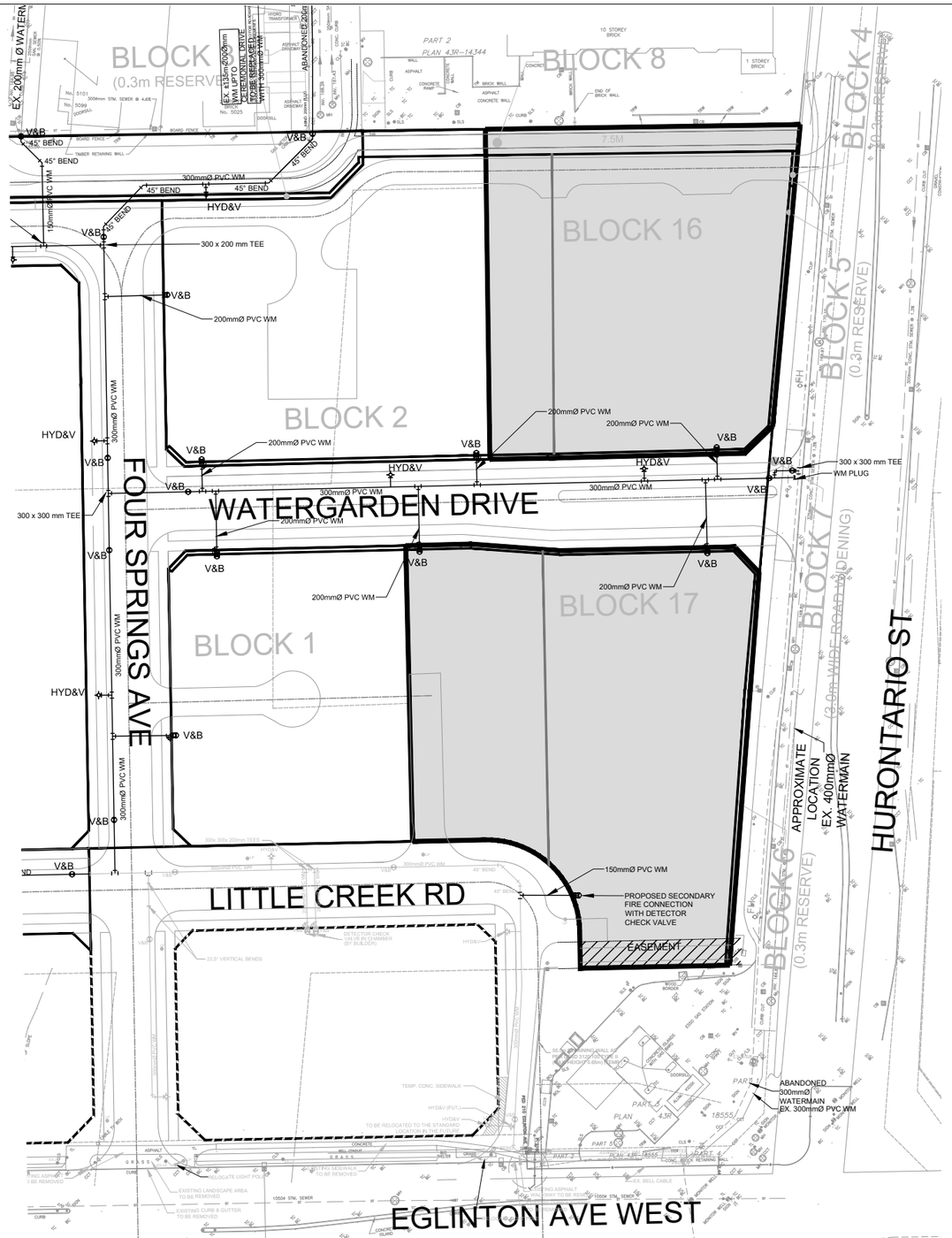

LEGEND:

- PROPOSED WATER MAIN
- WATER MAIN (FUT. PHASES)
- EX. WATERMAIN
- EASEMENT
- DENOTES HYDRANT
- PROPOSED INTENSIFICATION AREA

20 10 0 20 40 60

1:1500 (m)

PINNACLE
INTERNATIONAL

TITLE:

WATER DISTRIBUTION SYSTEM
PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	FEBRUARY, 2020
FIGURE No:	FIGURE 4

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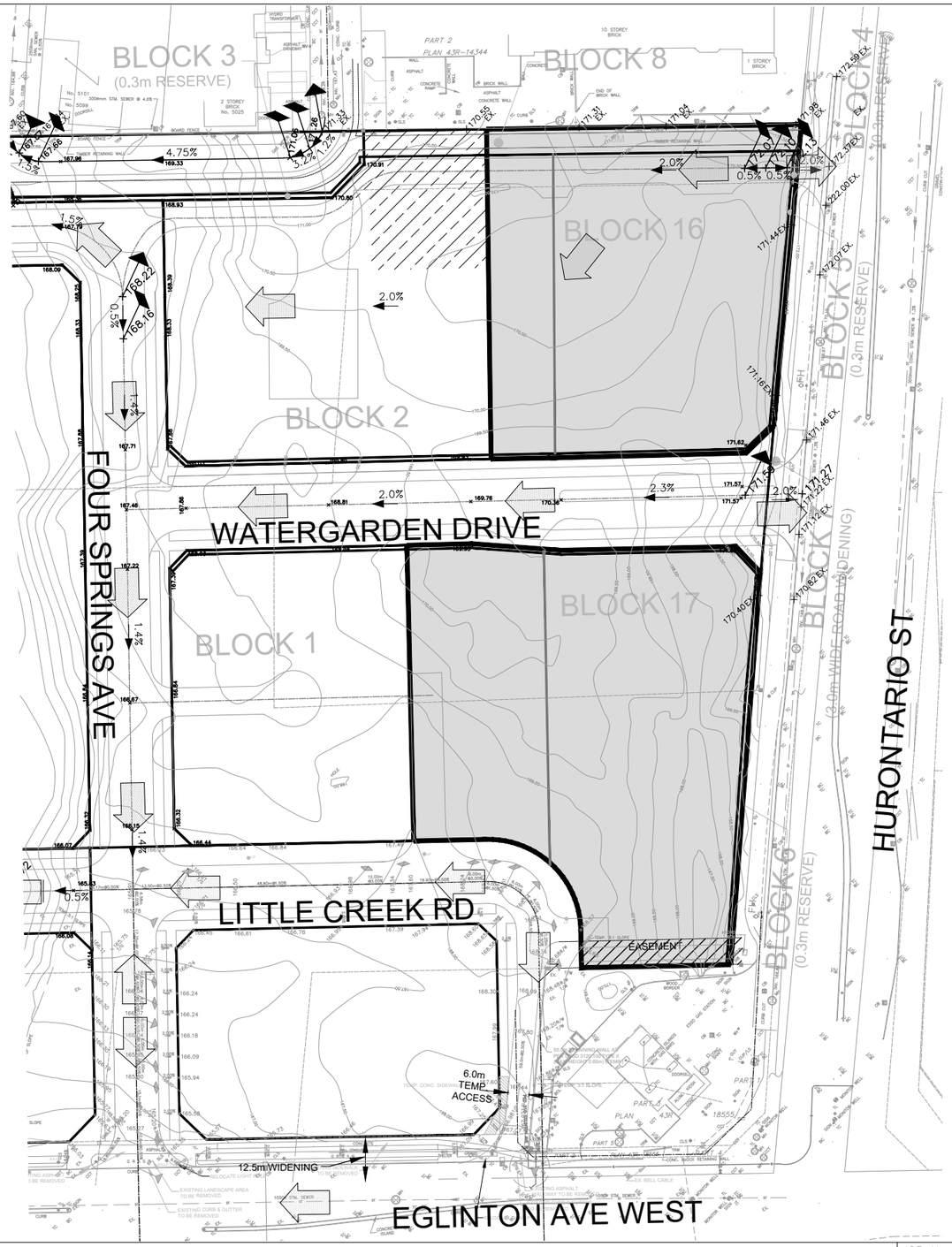

LEGEND:

- MAJOR STORM OVERLAND FLOW
- ×170.95 PROPOSED ELEVATION
- ×171.20 Ex. EXISTING ELEVATION (REMAIN)
- 3.9% PROPOSED ROAD GRADE
- PROPOSED INTENSIFICATION AREA
- PORTION OF PHASE IV–PART I
- EASEMENT

20 10 0 20 40 60
1:1500 (m)

PINNACLE
INTERNATIONAL

TITLE:

ROAD GRADES AND OVERLAND FLOW ROUTING
PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	FEBRUARY, 2020
FIGURE No:	FIGURE 5

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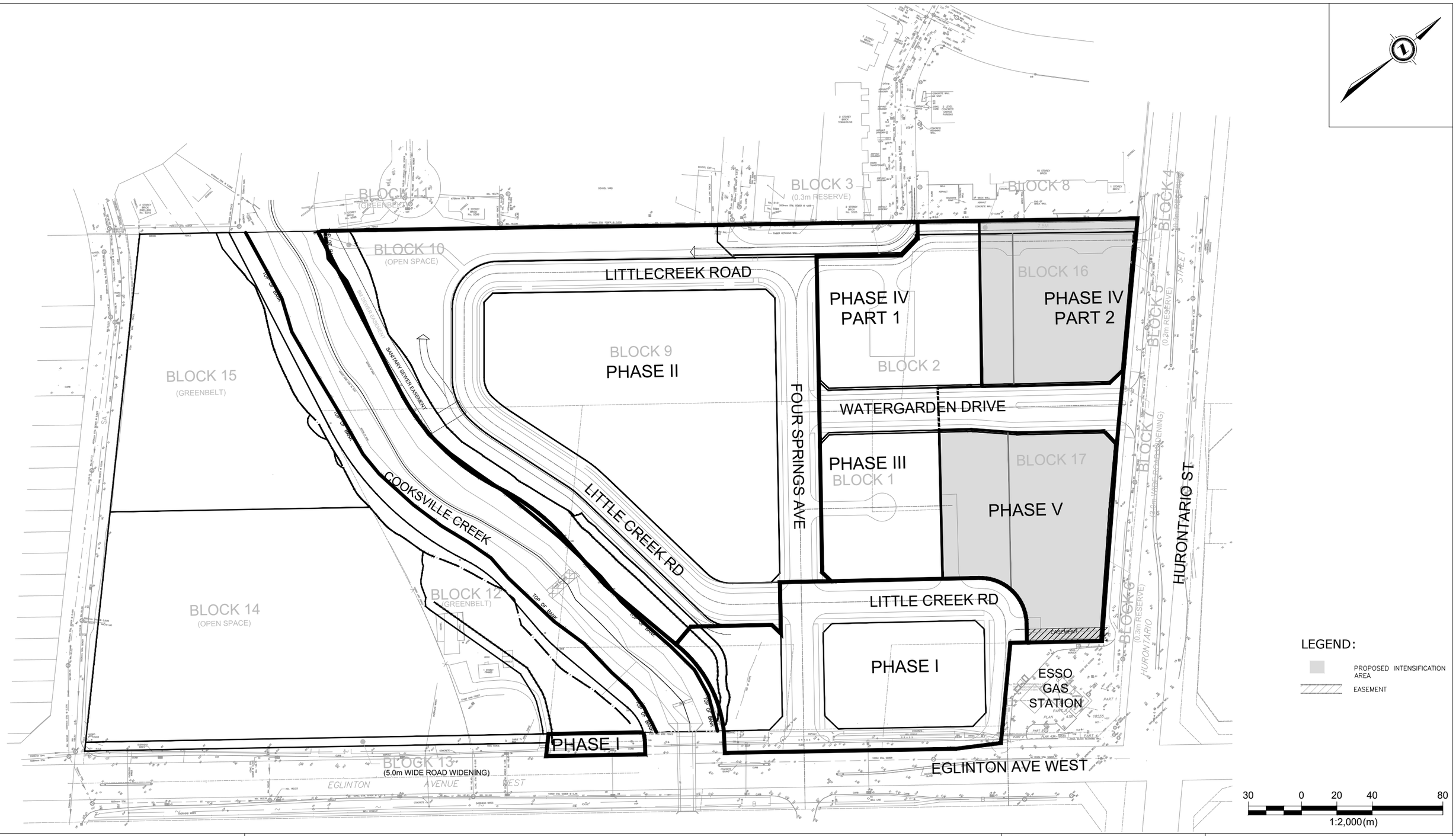
APPENDIX A

SUBDIVISION – STAGING PLAN



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PINNACLE
INTERNATIONAL

TITLE:

STAGING PLAN
SUBDIVISION

JOB No:	108686
SCALE:	1:2000
DATE:	MARCH,2019
FIGURE No:	APPENDIX 'A'

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APPENDIX B

SUITE SUMMARY

EXISTING ZONING CATEGORY: "DEVELOPMENT ZONE" PROPOSED ZONING CATEGORY: ALL SUBJECT TO OPA/ REZONING APPLICATION OZ 16/10

PHASE 4 , PART 1 AND 2 SITE DATA (BY LAW# 0275-2012)

LOT AREA/ COVERAGE

LOT AREA (INCLUDE RESERVE AREA) — 14,604.1 SM

	PART 1		PART 2	TOTAL FSI - PART (1 + 2)
LOT AREA	7327.2 SM (100%)	LOT AREA	7229.0 SM (100%)	14,556.2 SM (100%)
COVERED AREA/ FLOOR AREA	3494.2 SM (46.8%)	COVERED AREA/ FLOOR AREA	2952.5 SM (41%)	6446.7 SM (45%)
PAVED AREA	2005.8 SM (28.1%)	PAVED AREA	1388.7 SM (19%)	3392.5 SM (23%)
LANDSCAPED AREA	1827.5 SM (25.1%)	LANDSCAPED AREA	2889.8 SM (40%)	4717.3 SM (32%)

MINIMUM LANDSCAPED AREA — 25% OF LOT AREA

GFA AND FSI

MINIMUM ALLOWED FLOOR SPACE INDEX (2.9 X LOT AREA) — 42213.0 SM

MAXIMUM ALLOWED FLOOR SPACE INDEX (5.19X LOT AREA) — 75546.7 SM

PROPOSED FLOOR SPACE INDEX (FSI)

FSI - PART 1		FSI - PART 2		TOTAL FSI - PART (1 + 2)
FLOOR SPACE INDEX (FSI)	5.12X	FLOOR SPACE INDEX (FSI)	4.33X	4.73X

PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA)

PART 1		PART 2		TOTAL - PART (1 + 2)
PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA)	37,349.1 SM	PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA)	30,386.70 SM	68,003.00 SM

PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA)

PART 1		PART 2		TOTAL - PART (1 + 2)
PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	233.0 SM	PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	917.20 SM	1,150.20 SM
TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	37,582.1 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	31,303.90 SM	68,886.00 SM

REQUIRED NON-RESIDENTIAL GROSS FLOOR AREA AS PER OZ 16/10 (INCLUDED IN MAX. 5.19X F.S.I)

AREA	MINIMUM GROSS FLOOR AREA NON-RESIDENTIAL
A	230.0 SM
B	0.0 SM
C	4770.0 SM

BUILDING HEIGHT

BUILDING HEIGHT ALLOWED AS PER OZ 16/10

AREA	MIN. BLDG. HEIGHT	MAX. BLDG HEIGHT	MINIMUM HEIGHT OF PODIUM	MAXIMUM HEIGHT OF PODIUM
A	5 STOREYS	15 STOREYS	1 STOREY	4 STOREYS
B	5 STOREYS	34 STOREYS	1 STOREY	4 STOREYS
C	10 STOREYS	25 STOREYS	3 STOREYS	6 STOREYS

BUILDING HEIGHT PROPOSED

	PART 1 (ZONE A AND B)			PART 2 (ZONE C)
	BUILDING 1	BUILDING 2		BUILDING 3
BUILDING HEIGHT	15 STOREYS	34 STOREYS	BUILDING HEIGHT	35 STOREYS

UNITS

NUMBER OF UNITS ALLOWED — 748

NUMBER OF UNITS PROPOSED — 874

	PART 1				PART 2	TOTAL - PART (1 + 2)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BUILDING 3	
1 BEDROOM	57	88	145	1 BEDROOM	82	227
1 BEDROOM + DEN	47	44	91	1 BEDROOM + DEN	151	242
2 BEDROOM	5	2	7	2 BEDROOM	54	61
2 BEDROOM + DEN	64	152	216	2 BEDROOM + DEN	109	325
3 BEDROOM	–	–	–	3 BEDROOM	9	9
3 BEDROOM + DEN	–	–	–	3 BEDROOM + DEN	1	1
2 STOREY	5	4	9	2 STOREY	–	9
TOTAL SUITES PROPOSED	178	290	468	TOTAL SUITES PROPOSED	406	874

AMENITY

AMENITY REQUIRED:(GENERAL BY-LAW-CITY OF MISSISSAUGA)

MINIMUM AMENITY AREA ALLOWED (5.65M² PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)	2620.8 SM	2,273.6 SM	4,894.4 SM
MINIMUM AMENITY AREA ALLOWED TO BE PROVIDED OUTSIDE AT GRADE			55.0 SM

AMENITY PROVIDED:

	PART 1			PART 2	TOTAL - PART (1 + 2)
	BUILDING 1	BUILDING 2	TOTAL (1+2)	BLDG 3	
INDOOR AMENITY AREA PROVIDED	340.1	600.9	941.0 SM	914.0	1,855.0 SM
OUTDOOR AMENITY AREA PROVIDED	633.3	1090.1	1723.4 SM	2,009.7	3,733.1 SM
TOTAL AMENITY AREA PROVIDED	973.4	1691.0	2664.4 SM	2,923.7	5,588.1 SM
TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	N/A	N/A	2620.8 SM	2,273.6	4,894.4 SM
AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT					8.2 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	490.0 SM	820.0 SM	1310.0 SM	1,461.85	2,771.85 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	371.5 SM	371.5 SM	743.0 SM	2,612.20	3,355.20 SM
MINIMUM AMENITY AREA PROHIBITED OUTSIDE AT GRADE			55.0 SM	55.00 SM	110.00 SM
AMENITY AREA PROVIDED OUTSIDE AT GRADE			1036.0 SM	2,145.50	3,181.50 SM

PARKING

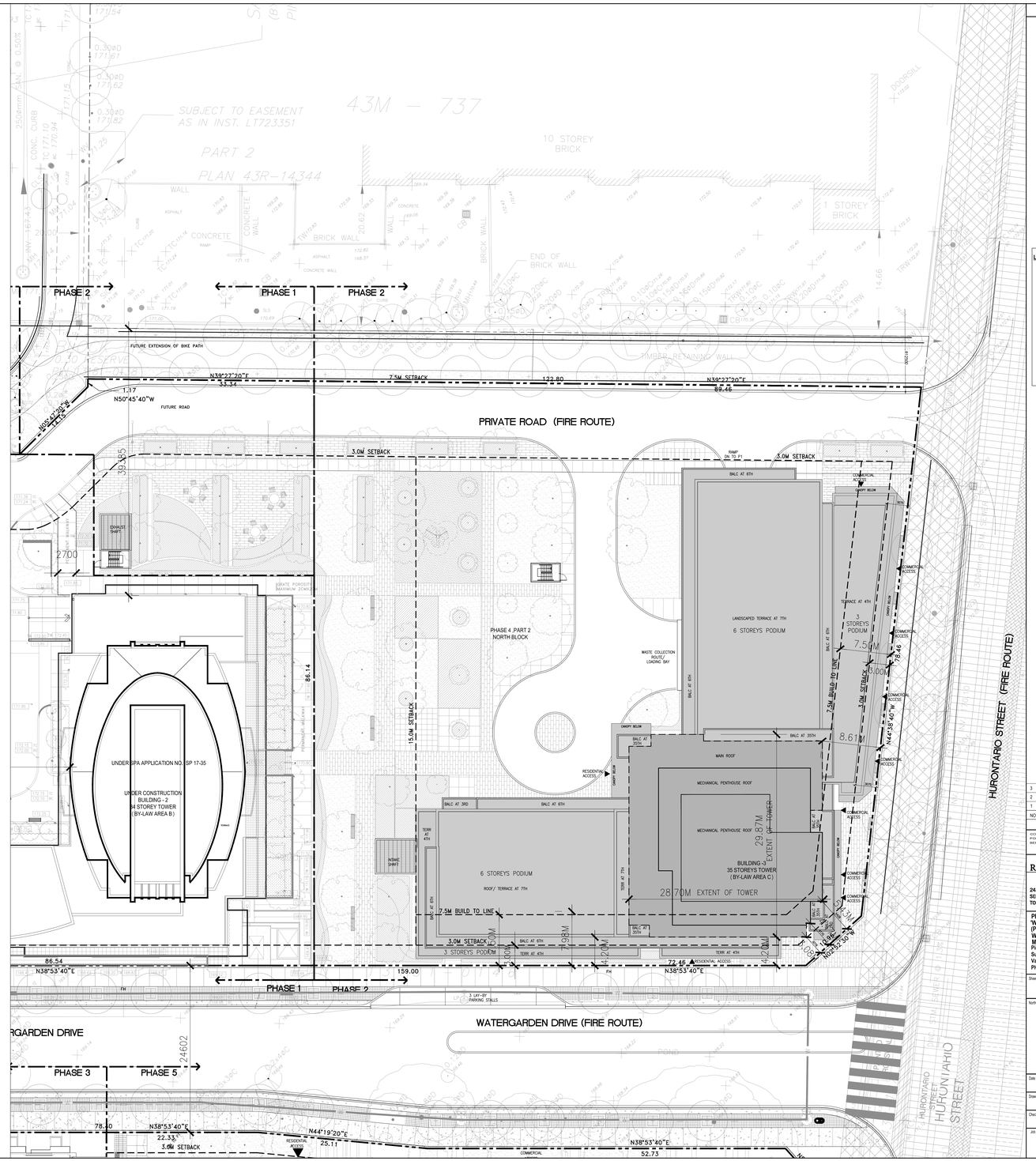
PARKING REQUIRED:(BY-LAW # 0225-2007)

	PART 1			PART 2	TOTAL - PART (1 + 2)
	BUILDING 1	BUILDING 2	TOTAL (1+2)	BLDG 3	
RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM: 1.1 SPACE/DWELLING UNIT	187	318	505	436	941
RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	6	5	11	12	23
SHARED PARKING REQUIRED: *20%RATE OF VISITOR PARKING REQUIRED OR ALL NON RESIDENTIAL PARKING REQUIRED:	–	–	–	–	–
VISITOR PARKING REQUIRED: 0.15 SPACES/DWELLING UNIT	27*	44	71	61*	132*
COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	13*	–		39	–
TOTAL PARKING REQUIRED:	220	367	587	509	1096
PARKING SPACES FOR PERSONS WITH DISABILITIES REQUIRED(4% OF TOTAL NON-RESIDENTIAL PARKING SPACES- AS PER TABLE 3.1.3.1-ZONING BY LAW 0225-2007)	1	2	3	3	6

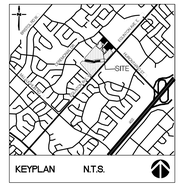
PARKING PROVIDED:

	PART 1					PART 2			TOTAL - PART (1 + 2)
	BUILDING 1		BUILDING 2		TOTAL (1+2)	BLDG 3			
	# OF CARS		# OF CARS			# OF CARS			
	VISITORS	TENANTS	VISITORS	TENANTS	TOTAL	VIS./COMM	TENANTS	TOTAL	
PARKING LEVEL 1	27	16	44	17	104	61	89	150	254
PARKING LEVEL 2	0	72	0	93	165	0	177	177	342
PARKING LEVEL 3	0	72	0	102	174	0	182	182	356
PARKING LEVEL 4	0	72	0	102	174	0	0	0	174
PARKING LEVEL 5		20			20	0	0	0	20
TOTAL PARKING PROPOSED	27	252	44	314	637	61	448	509	1146

SHARED PARKING SPACES WILL NOT BE RESERVED FOR CERTAIN USES AT SPECIFIC TIMES.
* THE PARKING SPACES WILL BE ACCESSIBLE TO ALL USERS IN THE SHARED PARKING ARRANGEMENT.
THE SHARED SPACES WILL ALL BE PART OF THE SAME CONDOMINIUM.



GENERAL NOTES



LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

PLAN OF SURVEY OF BLOCKS 1, 2 ,16 AND 17 REGISTERED PLAN 43M-1957 CITY OF MISSISSAUGA REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

R. AVIS SURVEYING INC.
235 YORKLAND BLVD., SUITE 203
NORTH YORK, ONTARIO
M2J 4Y8
TEL.: (416) 490-8581 FAX: (416) 491-6206
www.ravis.com
DATE:
FEBRUARY 23, 2018

NO	DATE	REMARKS
3	14 FEB 2020	RE-ISSUED FOR RE-ZONING SUBMISSION
2	1 APR 2019	RE-ISSUED FOR RE-ZONING SUBMISSION
1	31 MAY 2018	RE-ZONING SUBMISSION

REVISIONS

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME REPORTING ANY DISCREPANCIES TO THE ARCHITECT BEFORE COMMENCING THE WORK.

PRINTS ARE NOT TO BE SCALED.

RICHMOND | Architects Ltd.

243 COLLEGE STREET
SECOND FLOOR
TORONTO, ONTARIO M5T 1R5

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PROPOSED CONDOMINIUM DEVELOPMENT
'WATERGARDEN' (PHASE 4, PART 2)(H-RA5-43) & (PHASE 5)(H-RA5-42)
WATERGARDEN DRIVE
MISSISSAUGA, ONTARIO
Pinnacle International (Ontario) Limited
Suite 300- 911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 688 7749

Sheet Title:
SITE PLAN & STATS
NORTH BLOCK PHASE 4, PART 2

North:	Seal:
Date Plotted: 18 FEB 2020	Date: 25 MAY 2018
Drawn By: AA	Scale: 1:200
Checked By:	Revision Number: 3
Job No.: 2017-002-N	Sheet No.: SP A102-N

PHASE 3+5 BLOCK 2

EXISTING ZONING CATEGORY: "DEVELOPMENT ZONE" PROPOSED ZONING CATEGORY: ALL SUBJECT TO OPA/ REZONING APPLICATION OZ 16/10

PHASE 3 AND 5 SITE DATA (BY LAW# 0275-2012)

	BLOCK-2 PHASE 3 EXISTING RESIDENTIAL RA5-42	BLOCK-2 PHASE 5 H- RA5-42	
LOT AREA/ COVERAGE	LOT AREA (INCLUDE RESERVE AREA)		15,466.1 SM
	PHASE 3	PHASE 5	TOTAL - PHASE (3+5)
	LOT AREA 5,166.0 SM (100%)	LOT AREA 10,300.1 SM (100%)	LOT AREA 15,466.1 SM (100%)
	COVERED AREA/ FLOOR AREA 3014.0 SM (58.4%)	COVERED AREA/ FLOOR AREA 5881.1 SM (57.1%)	COVERED AREA/ FLOOR AREA 8875.1 SM (57.4%)
	PAVED AREA 390.6 SM (7.56%)	PAVED AREA 545.2 SM (5.3%)	PAVED AREA 933.8 SM (6.3%)
	LANDSCAPED AREA 1727.4 SM (33.4%)	LANDSCAPED AREA 3895.8 SM (37.8%)	LANDSCAPED AREA 5623.2 SM (36.3%)
	MINIMUM LANDSCAPED AREA		25% OF LOT AREA
GFA AND FSI	MINIMUM ALLOWED FLOOR SPACE INDEX (2.9 X LOT AREA)		44851.69 SM
	MAXIMUM ALLOWED FLOOR SPACE INDEX (7.11X LOT AREA)		109963.9 SM
	PROPOSED FLOOR SPACE INDEX (FSI)		
	FSI - PHASE 3	FSI - PHASE 5	TOTAL FSI - PHASE (3+5)
	FLOOR SPACE INDEX (FSI) 6.94X	FLOOR SPACE INDEX (FSI) 7.84X	7.54X
	PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA)		
	PHASE 3	PHASE 5	TOTAL - PHASE (3+5)
	PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA) 35,865.0 SM	PROPOSED RESIDENTIAL GROSS FLOOR AREA (GFA) 72,268.9 SM	108,133.9
	PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA)		
	PHASE 3	PHASE 5	TOTAL - PHASE (3+5)
	PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA) –	PROPOSED NON-RESIDENTIAL GROSS FLOOR AREA (GFA) 8,490.7 SM	–
	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA) 35,865.0 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA) 80,759.6 SM	116,624.6 SM
	REQUIRED NON-RESIDENTIAL GROSS FLOOR AREA AS PER OZ 16/10 (INCLUDED IN MAX. 7.11X F.S.I)		
	AREA	MINIMUM GROSS FLOOR AREA NON-RESIDENTIAL	
	A	1,000.0 SM	
	C	1,000.0 SM	
	D	4,000.0 SM	
BUILDING HEIGHT	BUILDING HEIGHT ALLOWED AS PER OZ 16/10		

AREA	MINIMUM BUILDING HEIGHT	MAXIMUM BUILDING HEIGHT	MINIMUM HEIGHT OF PODIUM	MAXIMUM HEIGHT OF PODIUM
A	5 STOREYS	15 STOREYS	3 STOREYS	4 STOREYS
B	5 STOREYS	20 STOREYS	–	–
C	5 STOREYS	20 STOREYS	3 STOREYS	4 STOREYS
D	10 STOREYS	34 STOREYS	3 STOREYS	6 STOREYS

BUILDING HEIGHT PROPOSED

	PHASE- 3 (ZONE C)		PHASE 5 (ZONE D)			
	BUILDING 1	BUILDING 2		BUILDING 1	BUILDING 2	BUILDING 3
BUILDING HEIGHT	26 STOREYS	23 STOREYS	BUILDING HEIGHT	35 STOREYS	32 STOREYS	38 STOREYS

UNITS	NUMBER OF UNITS ALLOWED	1121
	NUMBER OF UNITS PROPOSED	1395

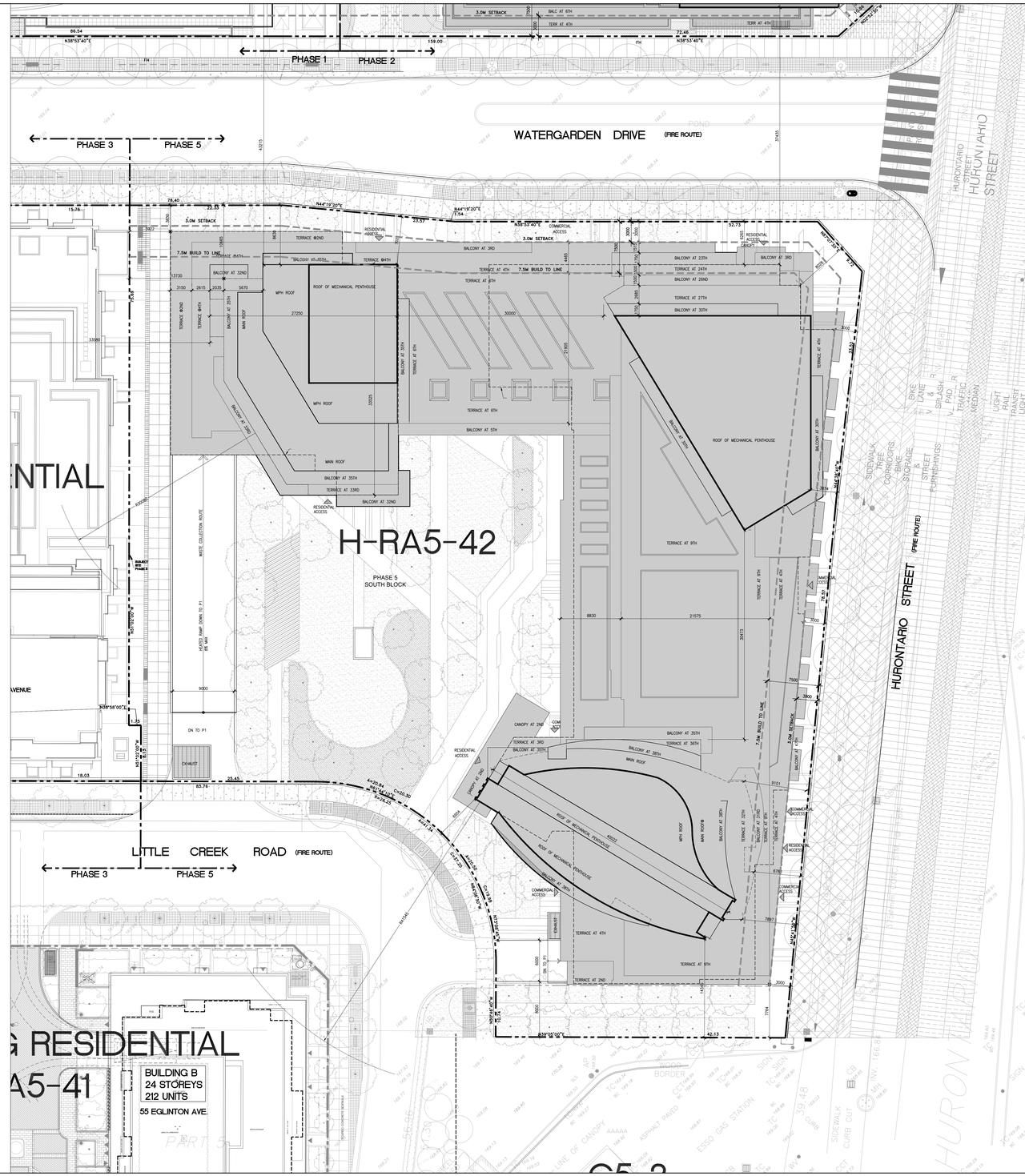
	PHASE 3			PHASE 5				TOTAL - PHASE (3+5)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 1	BLDG. 2	BLDG. 3	TOTAL	
1 BEDROOM	–	–	–	1 BEDROOM	71	29	98	98	–
1 BEDROOM + DEN	–	–	–	1 BEDROOM + DEN	156	76	128	360	–
2 BEDROOM	–	–	–	2 BEDROOM	92	53	72	217	–
2 BEDROOM + DEN	–	–	–	2 BEDROOM + DEN	70	51	84	205	–
3 BEDROOM	–	–	–	3 BEDROOM	4	40	25	69	–
2 STOREY	–	–	–	2 STOREY	–	–	–	–	–
TOTAL SUITES PROPOSED	234	212	446	TOTAL SUITES PROPOSED	323	291	335	949	1395

AMENITY	AMENITY REQUIRED:(GENERAL BY-LAW-CITY OF MISSISSAUGA)							
	MINIMUM AMENITY AREA ALLOWED (5.65M PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)		2497.6 SM				5314.4 SM	7,812.0 SM
	MINIMUM AMENITY AREA ALLOWED TO BE PROVIDED OUTSIDE AT GRADE							55.0 SM
	AMENITY PROVIDED:							
	PHASE 3			PHASE 5				TOTAL - PHASE (3+5)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 1	BLDG. 2	BLDG. 3	TOTAL	
	INDOOR AMENITY AREA PROVIDED 831.0	177.0	1,008.0 SM	INDOOR AMENITY AREA PROVIDED	566.0	969.5	1,415.2	2,949.8	3,957.8 SM
	OUTDOOR AMENITY AREA PROVIDED 1,060.0	612.0	1,672.0 SM	OUTDOOR AMENITY AREA PROVIDED	1,500.0	2,056.0	2,490.0	6,046.0	7,718.0 SM
	TOTAL AMENITY AREA PROVIDED 1,891.0	789.0	2,680.0 SM	TOTAL AMENITY AREA PROVIDED	2,065.7	3,025.0	3,905.2	8,995.8	11,675.8 SM
	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA) –	–	2,497.6 SM	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)				7,212.8	9,710.4 SM
	AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT								5.60 SM
	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA 655.0 SM	593.6 SM	1,248.6 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	1032.8	1533.7	2007.0	4573.6	5913.6 SM
	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA 945.5 SM	394.5 SM	1,340.0 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	1032.8	1533.7	2007.0	4573.6	5913.6 SM
	MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE		55.0 SM	MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE				55.0	110.0 SM
	AMENITY AREA PROVIDED OUTSIDE AT GRADE 402.0	321.0	723.0 SM	AMENITY AREA PROVIDED OUTSIDE AT GRADE	1,500.0	715.3	881.4	3,096.9	3819.9 SM

PARKING	PARKING REQUIRED:(BY-LAW # 0225-2007)							
	PHASE 3			PHASE 5				TOTAL - PHASE (3+5)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 1	BLDG. 2	BLDG. 3	TOTAL	
	RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM: 1.1 SPACE/DWELLING UNIT –	–	–	RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM 1.1 SPACE/DWELLING UNIT	351	276	341	968	–
	RESIDENTIAL PARKING REQ'D PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT –	–	–	RESIDENTIAL PARKING REQ'D PER 3 BEDROOM 1.2 SPACE/DWELLING UNIT	5	48	30	83	
	SHARED PARKING REQUIRED: *GREATER OF VISITOR PARKING REQUIRED OR ALL NON RESIDENTIAL PARKING REQUIRED:			SHARED PARKING REQUIRED: *GREATER OF VISITOR PARKING REQUIRED OR ALL NON RESIDENTIAL PARKING REQUIRED:					
	VISITOR PARKING REQUIRED: 0.15 SPACES/DWELLING UNIT –	–	–	VISITOR PARKING REQUIRED: 0.15 SPACES/DWELLING UNIT	48	44	50	142	–
	COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM –	–	–	COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	–	140*	225*	365*	–
	TOTAL PARKING REQUIRED: –	–	–	TOTAL PARKING REQUIRED:	356	464	596	1416	–
	PARKING SPACES FOR PERSONS WITH DISABILITIES REQUIRED(4% OF TOTAL NON-RESIDENTIAL PARKING SPACES- AS PER TABLE 3.1.3.1-ZONING BY LAW 0225-2007) –	–	–	PARKING SPACES FOR PERSONS WITH DISABILITIES REQ'D (1 SPACE+3% OF NON-RESIDENTIAL SPACES @ 1 SPACE+3% OF VISITOR SPACES REQ'D- AS PER TABLE 3.1.3.1-ZONING BY LAW 0225-2007)	–	–	–	15	–
	PARKING PROVIDED:							

	PHASE 3				PHASE 5						TOTAL - PHASE (3+5)
	BUILDING 1		BUILDING 2		BLDG. 1		BLDG. 2		BLDG. 3		TOTAL
	# OF CARS		# OF CARS		# OF CARS		# OF CARS		# OF CARS		
	VISITORS	TENANTS	VISITORS	TENANTS	VIS./COMM	TENANTS	VIS./COMM	TENANTS	VIS./COMM	TENANTS	TOTAL
PARKING LEVEL 1	–	–	–	–	54	0	50	0	90	0	194 –
PARKING LEVEL 2	–	–	–	–	0	0	90	0	95	0	185 –
PARKING LEVEL 3	–	–	–	–	0	50	0	89	40	100	279 –
PARKING LEVEL 4	–	–	–	–	0	90	0	89	0	100	279 –
PARKING LEVEL 5	–	–	–	–	0	90	0	89	0	100	279 –
PARKING LEVEL 6	–	–	–	–	0	73	0	57	0	71	201 –
TOTAL PARKING PROPOSED	–	–	–	–	54	303	140	324	225	371	1417 –

SHARED PARKING SPACES WILL NOT BE RESERVED FOR CERTAIN USES AT SPECIFIC TIMES.
* THE PARKING SPACES WILL BE ACCESSIBLE TO ALL USERS IN THE SHARED PARKING ARRANGEMENT. THE SHARED SPACES WILL ALL BE PART OF THE SAME CONDOMINIUM.



KEYPLAN N.T.S.

GENERAL NOTES

LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

PLAN OF SURVEY OF
BLOCKS 1, 2 ,16 AND 17
REGISTERED PLAN 43M-1957
CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

R. AVIS SURVEYING INC.
235 YORKLAND BLVD., SUITE 203
NORTH YORK, ONTARIO
M2J 4Y8
TEL: (416) 490-8581 FAX: (416) 491-6206
www.ravis.com
DATE:
FEBRUARY 23, 2018

NO	DATE	REMARKS
3	14 FEB 2020	RE-ISSUED FOR RE-ZONING SUBMISSION
2	1 APR 2019	RE-ISSUED FOR RE-ZONING SUBMISSION
1	31 MAY 2018	RE-ZONING SUBMISSION

REVISIONS

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME. REPORT ANY DISCREPANCIES TO THE ARCHITECTS BEFORE COMMENCING THE WORK.

PRINTS ARE NOT TO BE SCALED.

RICHMOND | Architects Ltd.

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SECOND FLOOR
TORONTO, ONTARIO M5T 1R5

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fr. 416 961 1321
RICHMONDARCH.COM

PROPOSED CONDOMINIUM DEVELOPMENT
'WATERGARDEN' (PHASE 4, PART 2)(H-RA5-43) & (PHASE 5)(H-RA5-42)
WATERGARDEN DRIVE
MISSISSAUGA, ONTARIO
Pinnacle International (Ontario) Limited
Suite 305-911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

Sheet No.: SITE PLAN&STATS
PHASE 5: (BLDG. 1,2&3)

North

Seal

Date Plotted: 18 FEB 2020
Date: 25 MAY 2018
Drawn By: –
Scale: 1:200
Checked By: –
Revision Number: 3
Job No.: 2017-002
Sheet No.: SP A103-S

APPENDIX C

SANITARY SEWER DESIGN SHEET AND SITE STATISTICS

DEVELOPMENT 14604 PINNACLE INTERNATIONAL (ONTARIO) LTD.

CONSULTANT IBI GROUP

MAJOR DRAINAGE AREA

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN CHART

Last Updated : February 3, 2020

Designed By: AC

Checked By: AC

LOCATION			DRAINAGE AREA			RUNOFF						PIPE SELECTION						
STREET	FROM MH. No.	TO MH. No.	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population	(1) Sewage Flow m3/sec	(2) Infiltration Flow m3/sec	(3) Foundation Drains m3/sec	Total Flow (1)+(2)+(3) m3/s	Pipe Length L m	Cumm. Pipe Length m	Pipe Dia> mm	Pipe Gradient % m/m	Actual Capacity (full) m3/s	Velocity (full) V m/s	Time Of Flow min.
TO EX. MH T4																		
AREA 2A (Phase IV - Part II)	STUB	MH6A																
TOWER (1 bedroom)				1.68	233.0		391											
TOWER (2+ bedrooms)				2.54	173.0		439											
OFFICE/COMMERCIAL				50.0	0.09		5											
INFILTRATION			1.25			1.25	835	0.0113	0.0003		0.0115	10.0		250	1.00%	0.059	1.211	0.138
AREA 1A (Phase V - BLDG 1 & 2)	STUB	MH6A																
TOWER (1 bedroom)				1.68	304.0		511											
TOWER (2+ bedrooms)				2.54	310.0		787											
OFFICE/COMMERCIAL				50.0	0.32		16											
INFILTRATION			0.65			0.49	1314	0.0171	0.0001		0.0172	15.5		250	1.00%	0.059	1.211	0.213
EXTERNAL AREA	STUB	MH6A																
Plaza East HWY #10				50.0	2.24	2.24	112	0.0017	0.0004		0.0021	18.4		250	1.50%	0.073	1.484	0.206
WATERGARDEN DRIVE	MH6A	MH7A				3.98	2262	0.0281	0.0008		0.0289	80.0		250	1.00%	0.059	1.211	1.101
WATERGARDEN DRIVE	MH7A	MH8A				3.98	2262	0.0281	0.0008		0.0289	72.0		250	1.00%	0.059	1.211	0.990
AREA 2B	STUB	MH8A																
TOWER				2.7	468.0		1264											
OFFICE/COMMERCIAL				50.0	0.02		1											
INFILTRATION			0.81			0.81	1265	0.0165	0.0002		0.0167	11.8		250	1.00%	0.059	1.211	0.162
AREA 1D	STUB	MH8A																
TOWER				2.7	234.0		632											
INFILTRATION			0.45			0.45	632	0.0087	0.0001		0.0088	14.8		250	1.00%	0.059	1.211	0.204
WATERGARDEN DRIVE	MH8A	MH9A				5.24	4158	0.0484	0.0010		0.0494	18.1		250	2.38%	0.092	1.869	0.161
AREA 1C	STUB	MH10A																
HIGH RISE				2.7	212.0		572											
INFILTRATION			0.41			0.41	572	0.0079	0.0001		0.0080	11.2		250	1.00%	0.059	1.211	0.154
FOURSPRINGS AVE.	MH10A	MH9A				0.41	572	0.0079	0.0001		0.0080	61.3		250	0.50%	0.042	0.857	1.192
	MH9A	MH11A				5.65	4731	0.0542	0.0011		0.0553	56.3		375	0.40%	0.111	1.004	0.934
LITTLE CREEK RD																		
AREA 7A	MH11A	MH12A	0.17			5.82	4731	0.0542	0.0012		0.0553	29.2		375	0.40%	0.111	1.004	0.485
AREA 5A	MH12A	MH13A	0.23			6.05	4731	0.0542	0.0012		0.0554	85.9		375	0.40%	0.111	1.004	1.426
AREA 5B	MH13A	MH14A	0.20			6.25	4731	0.0542	0.0013		0.0554	77.4		375	0.59%	0.135	1.219	1.058
EASEMENT																		
AREA 5C	MH14A	MH15A	0.08			6.33	4731	0.0542	0.0013		0.0554	28.3		375	1.50%	0.215	1.944	0.243
AREA 5D	MH15A	MH16A	0.23			6.56	4731	0.0542	0.0013		0.0555	69.2		375	2.00%	0.248	2.245	0.514
CREEK CROSSING																		
AREA 5E	MH16A	EX. MHT4	0.05			6.61	4731	0.0542	0.0013		0.0555	48.9		375	0.30%	0.096	0.870	0.937
TO EX. MH 9A																		
AREA 1B (Phase V - BLDG 3)																		
TOWER (1 bedroom)				1.68	154.0		259											
TOWER (2+ bedrooms)				2.54	181.0		460											
OFFICE/COMMERCIAL				50.0	0.52		26											
INFILTRATION	STUB	MH4A	0.54			0.54	745	0.0101	0.0001		0.010	13.5		200	4.00%	0.066	2.088	0.108
AREA 3B (PHASE I)	STUB	MH4A																
HIGH RISE				2.7	218.0		589											
INFILTRATION			0.37			0.37	589	0.0081	0.0001		0.008	14.5		200	2.00%	0.046	1.476	0.164
LITTLE CREEK RD	MH4A	MH1A				0.91	1333	0.0174	0.0002		0.018	85.4		250	1.00%	0.059	1.211	1.175
AREA 3A (PHASE I)	STUB	MH1A																
HIGH RISE				2.7	251.0		678											
INFILTRATION			0.50			0.50	678	0.0093	0.0001		0.009	15.5		200	4.00%	0.066	2.088	0.124
LITTLE CREEK RD	MH1A	MH2A				1.41	2011	0.0253	0.0003		0.026	11.1		250	1.00%	0.059	1.211	0.153
BLOCK 9																		
TH				3.8	100.0		380											
INFILTRATION	STUB	MH5A	2.68			2.68	380	0.0054	0.0005		0.006	11.2		250	1.00%	0.059	1.211	0.154
AREA 6	MH5A	MH2A	0.40			3.08	380	0.0054	0.0006		0.006	34.6		250	0.50%	0.042	0.857	0.673
Foursprings Avenue	MH2A	MH3A				4.49	2391	0.0295	0.0009		0.0304	65.9		300	0.50%	0.068	0.967	1.135
Foursprings Avenue	MH3A	EX.MH9A				**4.49**	**2391**	0.0295	0.0009		**0.0304**	68.4		**300**	0.50%	0.068	0.967	1.178

For Population < 1000 people Min. designed sewage flow = 0.013 m^3/sec as per Std. 2-5-2

For infiltration flow 0.0002 m^3/sec/ha was used as Min. designed flow

Population for Tower/ Midrise building is considered as 2.7 persons /unit

Population for Town Houses is considered as 3.8 persons /unit

Population for Commercial is considered as 50 persons/ Hectare of land

APPENDIX D

STORM SEWER DESIGN SHEET

PROJECT 14604 PINNACLE UPTOWN MISSISSAUGA PHASE II

SHEET No. 1 **OF** 1 **DATE** 21 JAN. 2020

CONSULTANT IBI GROUP.

Designed By: AC

MAJOR DRAINAGE AREA PHASE II

STORM DRAINAGE DESIGN CHART
FOR CIRCULAR DRAINS FLOWING FULL
FINAL DESIGN

Checked By: NC

LOCATION					DRAINAGE AREA				RUNOFF					PIPE SELECTION						Overland Flow along Road	Minor Losses	PROFILE		
	FROM		TO																				UPSTREAM	
STREET (DRAINAGE AREA LABEL)	MH. No.	Sta.	MH. No.	Sta.	A ha	Coeff. C	A*C	ACCUM. A*C	Cummul. Tc (Ti-__) min.	i_{10} mm/h	i_{100} mm/h	Q_{10} m^3/sec	Q_{100} m^3/sec	Pipe Length L m	Pipe Slope S %	Pipe Dia> mm	Actual Capacity (full) m^3/s	Velocity (full) V m/s	Time Of Flow min.	Overland Flow along Road Q_{Rd} m^3/sec	**Minor Losses** **m**	Fall In Sewer m	Surface Elev. m	Inv. Elev. m
INITIAL TC=15 min																								
FROM EAST OF HURONTARIO ST.																								
EGLINTON AVE. (EXT-1)	EX. MH 8		EX. MH 18		2.70	0.55	1.485								External Tc=15min. + 440m / 2.0m/s /60				18.667					
EXT-2					5.20	0.55	2.860																	
EXT-3					0.80	0.75	0.600	4.945	18.667	86.750	123.304	1.192	1.694	225.0	1.00%	**900**	1.889	2.876	1.304					
EGLINTON AVE. (EXT-4)	EX. MH18		EX. MH 19		2.03	0.80	1.624																	
EXT-5					0.55	0.75	0.413	6.982	19.971	83.138	118.231	1.612	2.293	122.0	1.00%	**900**	1.889	2.876	0.707					
HORONTARIO ST. (EXT-6)	TO		EX. MH 19		2.30	0.75	1.725								External Tc=15min. + 420m / 2.0m/s /60				18.500					
EGLINTON AVE. (EXT-7)	EX. MH 19		EX. MH20		0.62	0.75	0.465	9.172	20.678	81.318	115.674	2.072	2.947	138.0	1.00%	**1050**	2.849	3.187	0.722					
EGLINTON AVE. (EXT-8)	EX. MH20		MH6		0.60	0.75	0.450	9.622	21.399	79.552	113.191	2.126	3.025	74.0	1.20%	**1050**	3.121	3.491	0.353					
FROM PINNACLE PHASE II																								
WATERGARDEN DRIVE	MH 6		MH 7		0.98	0.74	0.725	0.725	15.000	99.166	140.690	**0.029***	**0.029***	12.9	2.00%	**300**	0.143	1.955	0.110					
					0.79	0.77	0.608	0.608	15.000	99.166	140.690	**0.028***	**0.028***	15.9	2.00%	**300**	0.143	1.955	0.136					
					0.26	0.75	0.195	0.195	15.000	99.166	140.690	0.054	0.076	5.0	1.00%	**300**	0.101	1.383	0.060					
								1.529	15.136	98.634	139.947	0.111	0.133	85.0	1.00%	**450**	0.297	1.812	0.782					
	MH 7		MH 8					1.529	15.917	95.690	135.829	0.109	0.131	90.0	1.40%	**525**	0.531	2.376	0.631					
	MH 8		MH 10		0.20	0.75	0.150	1.679	16.549	93.454	132.700	0.147	0.184	31.3	1.50%	**525**	0.549	2.459	0.212					
LITTLE CREEK	MH 9		MH 10		0.64	0.79	0.506	0.506	15.000	99.166	140.690	**0.019***	**0.019***	56.8	2.00%	**375**	0.259	2.269	0.417					
	MH 10		MH 11		0.26	0.90	0.234	2.418	16.761	92.729	131.686	0.225	0.288	60.3	0.70%	**675**	0.734	1.986	0.506					
	MH 11		MH 4		0.52	0.75	0.390	0.390	17.267	91.051	129.335	0.321	0.424	34.2	0.70%	**750**	0.972	2.131	0.268					
SALISHAN CIRCLE	DCB5		MH 12		0.32	0.75	0.240	0.240	15.000	99.166	140.690	0.066	0.094	55.0	0.80%	**300**	0.090	1.237	0.741					
LITTLE CREEK	MH 12		MH 13		0.07	0.75	0.053	0.293	15.741	96.336	136.733	0.078	0.111	65.1	0.80%	**300**	0.090	1.237	0.877					
	MH 13		MH 14		0.13	0.90	0.117	0.410	16.61865	93.214	132.365	0.106	0.151	74.6	0.90%	**375**	0.174	1.522	0.817					
	MH 14		MH 15					0.410	17.49604	90.314	128.302	0.103	0.146	16.0	0.50%	**375**	0.129	1.134	0.235					
LITTLE CREEK	MH 15		MH 16					0.410	18.31293	87.793	124.767	0.100	0.142	52.5	0.50%	**450**	0.210	1.281	0.683					
	MH 16		MH 17		0.17	0.90	0.153	0.563	18.996	85.804	121.976	0.134	0.191	19.0	0.50%	**450**	0.210	1.281	0.247					
	MH 17		MH 18		0.02	0.90	0.018	0.581	19.243	85.109	121.001	0.137	0.195	66.1	0.30%	**525**	0.246	1.100	1.002					
	MH 18		MH 19					0.581	20.245	82.421	117.224	0.133	0.189	87.2	0.30%	**525**	0.246	1.100	1.322					
	MH 19		MH 20		0.02	0.90	0.018	0.599	21.566	79.155	112.633	0.132	0.187	27.2	0.30%	**525**	0.246	1.100	0.412					
	Plug		MH 20		1.96	0.75	1.470	1.470	15.000	99.166	140.690	**0.154***	**0.154***	13.5	2.00%	**450**	0.421	2.562	0.088					
	MH 20		MH 4					2.069	21.979	78.196	111.283	0.284	0.339	23.3	0.22%	**900**	0.886	1.349	0.288					
FROM PINNACLE PHASE I																								
FUT. LITTLECREEK ROAD (AREA-1)	MH1(FUT.)		MH2		0.17	0.75	0.128	0.128	15.000	99.166	140.690	0.035	0.050	36.7	1.00%	**300**	0.101	1.383	0.442					
LITTLECREEK ROAD	MH2		MH3					0.128	15.442	97.454	138.297	0.035	0.049	33.5	0.50%	**300**	0.071	0.978	0.571					
LITTLECREEK ROAD	MH3		MH4		0.16	0.75	0.120	0.248	16.014	95.342	135.343	0.066	0.093	115.1	0.50%	**450**	0.210	1.281	1.497					
	MH 4		MH 5		0.70	0.75	0.525	3.231	22.267	77.542	110.362	1.376	1.237	89.0	0.50%	**975**	1.653	2.145	0.692					
TO EXISTING CULVERT																								
EGLINTON AVE. WEST	MH6		EX.CULVERT		0.60	0.75	0.450	12.853	22.958	76.020	108.220	2.714	3.864	60.0	1.20%	**1050**	3.121	3.491	0.286					
																	3.121							
NOTE: 0.029* - Represents Controled Flows																								

APPENDIX E

PRE-DEVELOPMENT OTTHYMO MODEL OUTPUT

Appendix "E": Pre-Development OTTHYMO Model Output

```
================================================================================

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    I N T E R H Y M O
       O   O    T      T    H   H   Y Y   MM MM  O   O   * * * 1989b * * *
       O   O    T      T    HHHHH    Y    M M M  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO    cA-313261302892

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  LICENSED TO:  Cumming Cockburn Limited, Ottawa

  Input   filename: 14604PRE.DAT
  Output  filename: 14604PRE.OUT
  Summary filename: 14604PRE.SUM

DATE: 02-20-2009                     TIME: 14:29:15

COMMENTS: _____________________________________________________________

--------------------------------------------------------------------------------
**************************************************************************
*                                                                        *
* PRE-DEVELOPMENT CONDITIONS                                             *
* PINNACLE DEVELOPMENT - PHASE I                                         *
* PART OF LOT 1, CONCESSION 1, WEST OF HURONTARIO STREET                 *
* NORTHWEST CORNER OF EGLINTON AVENUE EAST AND HURONTARIO STREET         *
* CITY OF MISSISSAUGA                                                    *
*                                                                        *
* IBI GROUP, FEBRUARY, 2009                                              *
* PROJECT REFERENCE NUMBER: 14604                                        *
*                                                                        *
**************************************************************************
*
**************************************************************************
*                                                                        *
* DESIGN STORMS: CITY OF MISSISSAUGA'S STANDARD DRAWING NO. 2111.010     *
*                                                                        *
**************************************************************************
************************* 1:2 YEAR DESIGN STORM ***************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 610.000
| Ptotal= 36.75 mm |                          B=   4.600
--------------------                          C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
                         Storm time step    =  5.00 min
                         Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    1.47 |  1.58    4.00 |  3.08    4.94 |  4.58    2.06
                 .17    1.52 |  1.67    4.51 |  3.17    4.54 |  4.67    2.00
                 .25    1.57 |  1.75    5.19 |  3.25    4.21 |  4.75    1.94
                 .33    1.63 |  1.83    6.14 |  3.33    3.93 |  4.83    1.89
                 .42    1.69 |  1.92    7.59 |  3.42    3.69 |  4.92    1.84
                 .50    1.76 |  2.00   10.07 |  3.50    3.47 |  5.00    1.80
                 .58    1.83 |  2.08   15.32 |  3.58    3.29 |  5.08    1.75
                 .67    1.91 |  2.17   33.85 |  3.67    3.12 |  5.17    1.71
                 .75    2.00 |  2.25  104.51 |  3.75    2.97 |  5.25    1.67
                 .83    2.09 |  2.33   40.43 |  3.83    2.84 |  5.33    1.64
                 .92    2.20 |  2.42   21.69 |  3.92    2.72 |  5.42    1.60
                1.00    2.32 |  2.50   14.85 |  4.00    2.61 |  5.50    1.57
                1.08    2.46 |  2.58   11.34 |  4.08    2.51 |  5.58    1.54
```

1.17	2.62	2.67	9.23	4.17	2.42	5.67	1.51
1.25	2.81	2.75	7.80	4.25	2.34	5.75	1.48
1.33	3.03	2.83	6.79	4.33	2.26	5.83	1.45
1.42	3.29	2.92	6.02	4.42	2.19	5.92	1.42
1.50	3.61	3.00	5.42	4.50	2.12	6.00	1.40

```
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=   2.410 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  11.311
     TOTAL RAINFALL   (mm)=  36.744
     RUNOFF COEFFICIENT   =    .308

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
```

```
*
************************* 1:5 YEAR DESIGN STORM ***************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 820.000
| Ptotal= 49.40 mm |                          B=   4.600
--------------------                          C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
                         Storm time step    =  5.00 min
                         Time to peak ratio =   .38
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.08	1.98	1.58	5.38	3.08	6.64	4.58	2.76
.17	2.04	1.67	6.06	3.17	6.11	4.67	2.69
.25	2.11	1.75	6.97	3.25	5.66	4.75	2.61
.33	2.19	1.83	8.25	3.33	5.28	4.83	2.54
.42	2.27	1.92	10.20	3.42	4.95	4.92	2.48
.50	2.36	2.00	13.53	3.50	4.67	5.00	2.42
.58	2.46	2.08	20.59	3.58	4.42	5.08	2.36
.67	2.56	2.17	45.50	3.67	4.20	5.17	2.30
.75	2.68	2.25	140.49	3.75	4.00	5.25	2.25
.83	2.81	2.33	54.34	3.83	3.82	5.33	2.20
.92	2.96	2.42	29.16	3.92	3.66	5.42	2.15
1.00	3.13	2.50	19.96	4.00	3.51	5.50	2.11
1.08	3.31	2.58	15.25	4.08	3.38	5.58	2.06
1.17	3.53	2.67	12.40	4.17	3.26	5.67	2.02
1.25	3.78	2.75	10.49	4.25	3.14	5.75	1.98
1.33	4.07	2.83	9.12	4.33	3.04	5.83	1.95
1.42	4.42	2.92	8.09	4.42	2.94	5.92	1.91
1.50	4.85	3.00	7.29	4.50	2.85	6.00	1.88

```
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
```

```
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=   4.490 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  19.581
     TOTAL RAINFALL   (mm)=  49.394
     RUNOFF COEFFICIENT   =    .396

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
--------------------------------------------------------------------------------
*
********************** 1:10 YEAR DESIGN STORM ***************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1010.000
| Ptotal= 60.84 mm |                          B=   4.600
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  6.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    2.44 |  1.58    6.63 |  3.08    8.18 |  4.58    3.41
                 .17    2.52 |  1.67    7.47 |  3.17    7.52 |  4.67    3.31
                 .25    2.60 |  1.75    8.59 |  3.25    6.97 |  4.75    3.22
                 .33    2.70 |  1.83   10.17 |  3.33    6.50 |  4.83    3.13
                 .42    2.80 |  1.92   12.56 |  3.42    6.10 |  4.92    3.05
                 .50    2.91 |  2.00   16.67 |  3.50    5.75 |  5.00    2.98
                 .58    3.03 |  2.08   25.36 |  3.58    5.44 |  5.08    2.90
                 .67    3.16 |  2.17   56.04 |  3.67    5.17 |  5.17    2.84
                 .75    3.30 |  2.25  173.04 |  3.75    4.93 |  5.25    2.77
                 .83    3.47 |  2.33   66.93 |  3.83    4.71 |  5.33    2.71
                 .92    3.65 |  2.42   35.91 |  3.92    4.51 |  5.42    2.65
                1.00    3.85 |  2.50   24.58 |  4.00    4.33 |  5.50    2.60
                1.08    4.08 |  2.58   18.78 |  4.08    4.16 |  5.58    2.54
                1.17    4.35 |  2.67   15.27 |  4.17    4.01 |  5.67    2.49
                1.25    4.65 |  2.75   12.92 |  4.25    3.87 |  5.75    2.44
                1.33    5.01 |  2.83   11.24 |  4.33    3.74 |  5.83    2.40
                1.42    5.45 |  2.92    9.97 |  4.42    3.62 |  5.92    2.35
                1.50    5.97 |  3.00    8.97 |  4.50    3.51 |  6.00    2.31


--------------------------------------------------------------------------------
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=   6.644 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  27.946
     TOTAL RAINFALL   (mm)=  60.839
```

```
     RUNOFF COEFFICIENT   =    .459

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
--------------------------------------------------------------------------------
*
*********************** 1:25 YEAR DESIGN STORM **************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1160.000
| Ptotal= 69.88 mm |                          B=   4.600
--------------------                          C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
                         Storm time step    =  5.00 min
                         Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    2.80 |  1.58    7.61 |  3.08    9.39 |  4.58    3.91
                 .17    2.89 |  1.67    8.57 |  3.17    8.64 |  4.67    3.80
                 .25    2.99 |  1.75    9.86 |  3.25    8.01 |  4.75    3.70
                 .33    3.10 |  1.83   11.68 |  3.33    7.47 |  4.83    3.60
                 .42    3.21 |  1.92   14.43 |  3.42    7.01 |  4.92    3.50
                 .50    3.34 |  2.00   19.14 |  3.50    6.61 |  5.00    3.42
                 .58    3.48 |  2.08   29.12 |  3.58    6.25 |  5.08    3.33
                 .67    3.63 |  2.17   64.36 |  3.67    5.94 |  5.17    3.26
                 .75    3.79 |  2.25  198.74 |  3.75    5.66 |  5.25    3.18
                 .83    3.98 |  2.33   76.87 |  3.83    5.40 |  5.33    3.11
                 .92    4.19 |  2.42   41.25 |  3.92    5.18 |  5.42    3.04
                1.00    4.42 |  2.50   28.23 |  4.00    4.97 |  5.50    2.98
                1.08    4.69 |  2.58   21.57 |  4.08    4.78 |  5.58    2.92
                1.17    4.99 |  2.67   17.54 |  4.17    4.60 |  5.67    2.86
                1.25    5.34 |  2.75   14.84 |  4.25    4.44 |  5.75    2.81
                1.33    5.76 |  2.83   12.90 |  4.33    4.30 |  5.83    2.75
                1.42    6.25 |  2.92   11.45 |  4.42    4.16 |  5.92    2.70
                1.50    6.86 |  3.00   10.31 |  4.50    4.03 |  6.00    2.65


--------------------------------------------------------------------------------
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=   8.476 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  34.974
     TOTAL RAINFALL   (mm)=  69.875
     RUNOFF COEFFICIENT   =    .501

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
--------------------------------------------------------------------------------
*
*********************** 1:50 YEAR DESIGN STORM **************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1300.000
| Ptotal= 78.30 mm |                          B=   4.700
--------------------                          C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
```

```
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    3.14 |  1.58    8.54 |  3.08   10.55 |  4.58    4.39
                 .17    3.24 |  1.67    9.63 |  3.17    9.70 |  4.67    4.26
                 .25    3.35 |  1.75   11.08 |  3.25    8.99 |  4.75    4.15
                 .33    3.47 |  1.83   13.12 |  3.33    8.39 |  4.83    4.03
                 .42    3.60 |  1.92   16.22 |  3.42    7.87 |  4.92    3.93
                 .50    3.75 |  2.00   21.53 |  3.50    7.42 |  5.00    3.83
                 .58    3.90 |  2.08   32.76 |  3.58    7.02 |  5.08    3.74
                 .67    4.07 |  2.17   72.28 |  3.67    6.67 |  5.17    3.65
                 .75    4.26 |  2.25  220.93 |  3.75    6.35 |  5.25    3.57
                 .83    4.47 |  2.33   86.28 |  3.83    6.07 |  5.33    3.49
                 .92    4.70 |  2.42   46.39 |  3.92    5.81 |  5.42    3.42
                1.00    4.96 |  2.50   31.76 |  4.00    5.58 |  5.50    3.34
                1.08    5.26 |  2.58   24.27 |  4.08    5.36 |  5.58    3.28
                1.17    5.60 |  2.67   19.73 |  4.17    5.17 |  5.67    3.21
                1.25    6.00 |  2.75   16.69 |  4.25    4.99 |  5.75    3.15
                1.33    6.46 |  2.83   14.50 |  4.33    4.82 |  5.83    3.09
                1.42    7.02 |  2.92   12.86 |  4.42    4.67 |  5.92    3.03
                1.50    7.70 |  3.00   11.58 |  4.50    4.52 |  6.00    2.98


--------------------------------------------------------------------------------
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=  10.228 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  41.778
     TOTAL RAINFALL   (mm)=  78.291
     RUNOFF COEFFICIENT   =    .534

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
--------------------------------------------------------------------------------
*
*********************** 1:100 YEAR DESIGN STORM *************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1450.000
| Ptotal= 87.30 mm |                          B=   4.900
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  6.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    3.51 |  1.58    9.57 |  3.08   11.82 |  4.58    4.90
                 .17    3.62 |  1.67   10.79 |  3.17   10.87 |  4.67    4.76
                 .25    3.75 |  1.75   12.42 |  3.25   10.07 |  4.75    4.63
                 .33    3.88 |  1.83   14.71 |  3.33    9.39 |  4.83    4.51
                 .42    4.03 |  1.92   18.20 |  3.42    8.81 |  4.92    4.39
                 .50    4.18 |  2.00   24.18 |  3.50    8.30 |  5.00    4.28
                 .58    4.36 |  2.08   36.82 |  3.58    7.85 |  5.08    4.18
                 .67    4.55 |  2.17   80.92 |  3.67    7.46 |  5.17    4.08
```

```
                .75    4.76 |  2.25  242.53 |  3.75    7.10 |  5.25    3.99
                .83    4.99 |  2.33   96.48 |  3.83    6.79 |  5.33    3.90
                .92    5.25 |  2.42   52.11 |  3.92    6.50 |  5.42    3.82
               1.00    5.55 |  2.50   35.69 |  4.00    6.24 |  5.50    3.74
               1.08    5.88 |  2.58   27.26 |  4.08    6.00 |  5.58    3.66
               1.17    6.26 |  2.67   22.15 |  4.17    5.78 |  5.67    3.59
               1.25    6.71 |  2.75   18.73 |  4.25    5.58 |  5.75    3.52
               1.33    7.23 |  2.83   16.27 |  4.33    5.39 |  5.83    3.45
               1.42    7.86 |  2.92   14.42 |  4.42    5.22 |  5.92    3.39
               1.50    8.62 |  3.00   12.98 |  4.50    5.05 |  6.00    3.32


--------------------------------------------------------------------------------
*
* COMPUTE HYDROGRAPH FOR BASIN CC11P
* SOURCE: THE COOKSVILLE CREEK FLOODLINE MAPPING STUDY CONDUCTED BY
*         R. V. ANDERSON ASSOCIATES LIMITED IN 1996
--------------------
| COMPUTE          |
| HYD       (0301) |   Area    (ha)=  80.00   Curve Number   (CN)= 83.0
| ID= 1 DT= 5.0 min|   Ia      (mm)=   6.10   Recession const.(K)=  .17
--------------------   U.H. Tp(hrs)=    .28

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

     U.H. peak       (cms)=  15.006

     PEAK FLOW       (cms)=  12.120 (i)
     TIME TO PEAK    (hrs)=   2.499
     RUNOFF VOLUME    (mm)=  49.270
     TOTAL RAINFALL   (mm)=  87.287
     RUNOFF COEFFICIENT   =    .564

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
--------------------------------------------------------------------------------
*

 FINISH
================================================================================
```

APPENDIX F

POST-DEVELOPMENT OTTHYMO MODEL OUTPUT

Appendix "F": Post-Development OTTHYMO Model Output

```
================================================================================

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    I N T E R H Y M O
       O   O    T      T    H   H   Y Y   MM MM  O   O   * * * 1989b * * *
       O   O    T      T    HHHHH    Y    M M M  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO    cA-313261313516

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  LICENSED TO:  Cumming Cockburn Limited, Ottawa

  Input   filename: 108686PS.DAT
  Output  filename: 108686PS.OUT
  Summary filename: 108686PS.SUM

DATE: 02-21-2020                      TIME: 09:06:37

COMMENTS: _____________________________________________________________

--------------------------------------------------------------------------------
*******************************************************************************
*                                                                             *
* POST-DEVELOPMENT CONDITIONS                                                 *
* PRELIMINARY ESTIMATE OF STORMWATER DETENTION STORAGE REQUIREMENT            *
* PINNACLE PHASE IV PART 2 AND PHASE V (OZ/OPA 18 11)                         *
* NORTHWEST OF EGLINTON AVENUE WEST AND HURONTARIO STREET                     *
* CITY OF MISSISSAUGA                                                         *
*                                                                             *
* IBI GROUP, REVISED ON FEBRUARY 2020                                         *
* PROJECT: 108686                                                             *
*                                                                             *
*******************************************************************************
*
*******************************************************************************
*                                                                             *
* DESIGN STORMS: CITY OF MISSISSAUGA'S STANDARD DRAWING NO. 2111.010          *
*                                                                             *
*******************************************************************************
*
************************ 1:2 YEAR DESIGN STORM ****************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 610.000
| Ptotal= 36.75 mm |                          B=   4.600
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  6.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    1.47 |  1.58    4.00 |  3.08    4.94 |  4.58    2.06
                 .17    1.52 |  1.67    4.51 |  3.17    4.54 |  4.67    2.00
                 .25    1.57 |  1.75    5.19 |  3.25    4.21 |  4.75    1.94
                 .33    1.63 |  1.83    6.14 |  3.33    3.93 |  4.83    1.89
                 .42    1.69 |  1.92    7.59 |  3.42    3.69 |  4.92    1.84
                 .50    1.76 |  2.00   10.07 |  3.50    3.47 |  5.00    1.80
                 .58    1.83 |  2.08   15.32 |  3.58    3.29 |  5.08    1.75
                 .67    1.91 |  2.17   33.85 |  3.67    3.12 |  5.17    1.71
                 .75    2.00 |  2.25  104.51 |  3.75    2.97 |  5.25    1.67
                 .83    2.09 |  2.33   40.43 |  3.83    2.84 |  5.33    1.64
                 .92    2.20 |  2.42   21.69 |  3.92    2.72 |  5.42    1.60
                1.00    2.32 |  2.50   14.85 |  4.00    2.61 |  5.50    1.57
                1.08    2.46 |  2.58   11.34 |  4.08    2.51 |  5.58    1.54
                1.17    2.62 |  2.67    9.23 |  4.17    2.42 |  5.67    1.51
                1.25    2.81 |  2.75    7.80 |  4.25    2.34 |  5.75    1.48
```

```
                1.33    3.03 |  2.83    6.79 |  4.33    2.26 |  5.83    1.45
                1.42    3.29 |  2.92    6.02 |  4.42    2.19 |  5.92    1.42
                1.50    3.61 |  3.00    5.42 |  4.50    2.12 |  6.00    1.40

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  76.60   Dir. Conn.(%)=  76.60
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .75          .23
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     104.51        21.91
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.95 (ii)    6.66 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .31          .14
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .21          .01           .21 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33          2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16         30.05
     TOTAL RAINFALL   (mm)=      36.63        36.63         36.63
     RUNOFF COEFFICIENT   =        .98          .30           .82

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .0497

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)        .98       .21      2.25      30.05
     OUTFLOW: ID= 8 (2025)        .98       .01      3.17      29.62

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.453
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .019

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 74 |   QPEAK     (cms)=    .21 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  30.05
  Filename: C:H2Bi_2.HYD
  Comments:
```

```
     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=272 |   QPEAK     (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  29.62
  Filename: C:H2Bo_2.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*-----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*-----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
--------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     104.51        21.91
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.92 (ii)    6.43 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .31          .14
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .20          .01           .20 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33          2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16         30.50
     TOTAL RAINFALL   (mm)=      36.63        36.63         36.63
     RUNOFF COEFFICIENT   =        .98          .30           .83

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)        .92       .20      2.25      30.50
     OUTFLOW: ID=10 (2035)        .92       .01      3.17      30.03

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.350
                   TIME SHIFT OF PEAK FLOW          (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .019

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 74 |   QPEAK     (cms)=    .20 (i)
```

```
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  30.50
  Filename: C:H3Bi_2.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=272 |   QPEAK     (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  30.03
  Filename: C:H3Bo_2.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
************************ 1:5 YEAR DESIGN STORM ***************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 820.000
| Ptotal= 49.40 mm |                          B=   4.600
--------------------                          C=    .780
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  6.00 hrs
                       Storm time step    =  5.00 min
                       Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    1.98 |  1.58    5.38 |  3.08    6.64 |  4.58    2.76
                 .17    2.04 |  1.67    6.06 |  3.17    6.11 |  4.67    2.69
                 .25    2.11 |  1.75    6.97 |  3.25    5.66 |  4.75    2.61
                 .33    2.19 |  1.83    8.25 |  3.33    5.28 |  4.83    2.54
                 .42    2.27 |  1.92   10.20 |  3.42    4.95 |  4.92    2.48
                 .50    2.36 |  2.00   13.53 |  3.50    4.67 |  5.00    2.42
                 .58    2.46 |  2.08   20.59 |  3.58    4.42 |  5.08    2.36
                 .67    2.56 |  2.17   45.50 |  3.67    4.20 |  5.17    2.30
                 .75    2.68 |  2.25  140.49 |  3.75    4.00 |  5.25    2.25
                 .83    2.81 |  2.33   54.34 |  3.83    3.82 |  5.33    2.20
                 .92    2.96 |  2.42   29.16 |  3.92    3.66 |  5.42    2.15
                1.00    3.13 |  2.50   19.96 |  4.00    3.51 |  5.50    2.11
                1.08    3.31 |  2.58   15.25 |  4.08    3.38 |  5.58    2.06
                1.17    3.53 |  2.67   12.40 |  4.17    3.26 |  5.67    2.02
                1.25    3.78 |  2.75   10.49 |  4.25    3.14 |  5.75    1.98
                1.33    4.07 |  2.83    9.12 |  4.33    3.04 |  5.83    1.95
                1.42    4.42 |  2.92    8.09 |  4.42    2.94 |  5.92    1.91
                1.50    4.85 |  3.00    7.29 |  4.50    2.85 |  6.00    1.88

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  76.60   Dir. Conn.(%)=  76.60
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .75          .23
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.
```

```
                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     140.49        37.11
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.74 (ii)    5.92 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .32          .15
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .28          .02           .29 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33          2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50         41.43
     TOTAL RAINFALL   (mm)=      49.24        49.24         49.24
     RUNOFF COEFFICIENT   =        .98          .38           .84

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .0497

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)       .98       .29      2.25      41.43
     OUTFLOW: ID= 8 (2025)       .98       .02      3.17      41.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.448
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .027

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .29 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  41.43
  Filename: C:H2Bi_5.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=290 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  41.00
  Filename: C:H2Bo_5.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*-------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*-------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
--------------------
                              IMPERVIOUS    PERVIOUS (i)
```

```
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     140.49        37.11
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.70 (ii)    5.71 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .32          .15
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .27          .02          .28 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33         2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50        41.97
     TOTAL RAINFALL   (mm)=      49.24        49.24        49.24
     RUNOFF COEFFICIENT   =        .98          .38          .85

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                  AREA     QPEAK     TPEAK       R.V.
                                  (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)         .92       .28      2.25      41.97
     OUTFLOW: ID=10 (2035)         .92       .01      3.17      41.50

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.343
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .026

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 74 |   QPEAK     (cms)=    .28 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  41.97
  Filename: C:H3Bi_5.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=290 |   QPEAK     (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  41.50
  Filename: C:H3Bo_5.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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*
********************** 1:10 YEAR DESIGN STORM **************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1010.000
```

| Ptotal= 60.84 mm | B= 4.600
C= .780
used in: INTENSITY = A / (t + B)^C

Duration of storm = 6.00 hrs
Storm time step = 5.00 min
Time to peak ratio = .38

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.08	2.44	1.58	6.63	3.08	8.18	4.58	3.41
.17	2.52	1.67	7.47	3.17	7.52	4.67	3.31
.25	2.60	1.75	8.59	3.25	6.97	4.75	3.22
.33	2.70	1.83	10.17	3.33	6.50	4.83	3.13
.42	2.80	1.92	12.56	3.42	6.10	4.92	3.05
.50	2.91	2.00	16.67	3.50	5.75	5.00	2.98
.58	3.03	2.08	25.36	3.58	5.44	5.08	2.90
.67	3.16	2.17	56.04	3.67	5.17	5.17	2.84
.75	3.30	2.25	173.04	3.75	4.93	5.25	2.77
.83	3.47	2.33	66.93	3.83	4.71	5.33	2.71
.92	3.65	2.42	35.91	3.92	4.51	5.42	2.65
1.00	3.85	2.50	24.58	4.00	4.33	5.50	2.60
1.08	4.08	2.58	18.78	4.08	4.16	5.58	2.54
1.17	4.35	2.67	15.27	4.17	4.01	5.67	2.49
1.25	4.65	2.75	12.92	4.25	3.87	5.75	2.44
1.33	5.01	2.83	11.24	4.33	3.74	5.83	2.40
1.42	5.45	2.92	9.97	4.42	3.62	5.92	2.35
1.50	5.97	3.00	8.97	4.50	3.51	6.00	2.31

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*

*--------------------
* PHASE V (AREA 2B) |
*--------------------

| DESIGN |
| STANDHYD (2020) | Area (ha)= .98
|ID= 2 DT= 5.0 min | Total Imp(%)= 76.60 Dir. Conn.(%)= 76.60

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.75	.23
Dep. Storage	(mm)=	.80	1.50
Average Slope	(%)=	1.50	1.50
Length	(m)=	80.69	40.00
Mannings n	=	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

		IMPERVIOUS	PERVIOUS (i)	*TOTALS*
Max.eff.Inten.	(mm/hr)=	173.04	52.90	
over	(min)	5.00	10.00	
Storage Coeff.	(min)=	1.60 (ii)	5.44 (ii)	
Unit Hyd. Tpeak	(min)=	5.00	10.00	
Unit Hyd. peak	(cms)=	.32	.16	
PEAK FLOW	(cms)=	.35	.03	.36 (iii)
TIME TO PEAK	(hrs)=	2.25	2.33	2.25
RUNOFF VOLUME	(mm)=	59.85	26.00	51.93
TOTAL RAINFALL	(mm)=	60.65	60.65	60.65
RUNOFF COEFFICIENT	=	.99	.43	.86

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 77.1 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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*
* STORMWATER DETENTION FACILITY #2

```
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000   |     .029       .0497

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)       .98       .36      2.25      51.93
     OUTFLOW: ID= 8 (2025)       .98       .02      3.17      51.49

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.450
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .034

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .36 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  51.93
  Filename: C:H2Bi_10.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=303 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  51.49
  Filename: C:H2Bo_10.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*-------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*-------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     173.04        52.90
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.57 (ii)    5.26 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .16
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .34          .02           .35 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33          2.25
     RUNOFF VOLUME    (mm)=      59.85        26.00         52.54
     TOTAL RAINFALL   (mm)=      60.65        60.65         60.65
     RUNOFF COEFFICIENT   =        .99          .43           .87

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
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*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                  AREA     QPEAK     TPEAK       R.V.
                                  (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)         .92       .35      2.25      52.54
     OUTFLOW: ID=10 (2035)         .92       .02      3.17      52.07

                    PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.344
                    TIME SHIFT OF PEAK FLOW         (min)= 55.000
                    MAXIMUM  STORAGE   USED       (ha.m.)=   .032

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 75 |   QPEAK     (cms)=    .35 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME      (mm)=  52.54
  Filename: C:H3Bi_10.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=303 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME      (mm)=  52.07
  Filename: C:H3Bo_10.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
********************** 1:25 YEAR RAINFALL STORM ************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1160.000
| Ptotal= 69.88 mm |                          B=   4.600
--------------------                          C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
                         Storm time step    =  5.00 min
                         Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    2.80 |  1.58    7.61 |  3.08    9.39 |  4.58    3.91
                 .17    2.89 |  1.67    8.57 |  3.17    8.64 |  4.67    3.80
                 .25    2.99 |  1.75    9.86 |  3.25    8.01 |  4.75    3.70
                 .33    3.10 |  1.83   11.68 |  3.33    7.47 |  4.83    3.60
                 .42    3.21 |  1.92   14.43 |  3.42    7.01 |  4.92    3.50
                 .50    3.34 |  2.00   19.14 |  3.50    6.61 |  5.00    3.42
                 .58    3.48 |  2.08   29.12 |  3.58    6.25 |  5.08    3.33
                 .67    3.63 |  2.17   64.36 |  3.67    5.94 |  5.17    3.26
                 .75    3.79 |  2.25  198.74 |  3.75    5.66 |  5.25    3.18
                 .83    3.98 |  2.33   76.87 |  3.83    5.40 |  5.33    3.11
                 .92    4.19 |  2.42   41.25 |  3.92    5.18 |  5.42    3.04
                1.00    4.42 |  2.50   28.23 |  4.00    4.97 |  5.50    2.98
                1.08    4.69 |  2.58   21.57 |  4.08    4.78 |  5.58    2.92
```

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                1.17    4.99 |  2.67   17.54 |  4.17    4.60 |  5.67    2.86
                1.25    5.34 |  2.75   14.84 |  4.25    4.44 |  5.75    2.81
                1.33    5.76 |  2.83   12.90 |  4.33    4.30 |  5.83    2.75
                1.42    6.25 |  2.92   11.45 |  4.42    4.16 |  5.92    2.70
                1.50    6.86 |  3.00   10.31 |  4.50    4.03 |  6.00    2.65

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*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  76.60   Dir. Conn.(%)=  76.60
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .75          .23
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     198.74        66.45
               over (min)         5.00        10.00
     Storage Coeff.  (min)=       1.51 (ii)    5.15 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .16
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .40          .04           .42 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33          2.25
     RUNOFF VOLUME    (mm)=      68.86        32.35         60.31
     TOTAL RAINFALL   (mm)=      69.66        69.66         69.66
     RUNOFF COEFFICIENT   =        .99          .46           .87

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000   |     .029       .0497

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)        .98       .42      2.25      60.31
     OUTFLOW: ID= 8 (2025)        .98       .02      3.17      59.88

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.453
                  TIME SHIFT OF PEAK FLOW         (min)= 55.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .039

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*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .42 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  60.31
  Filename: C:H2Bi_25.HYD
```

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  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=311 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  59.88
  Filename: C:H2Bo_25.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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*------------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*------------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
--------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     198.74        66.45
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.48 (ii)    4.97 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .22
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .39          .03          .42 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25         2.25
     RUNOFF VOLUME    (mm)=      68.86        32.35        60.97
     TOTAL RAINFALL   (mm)=      69.66        69.66        69.66
     RUNOFF COEFFICIENT   =        .99          .46          .88

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)       .92       .42      2.25      60.97
     OUTFLOW: ID=10 (2035)       .92       .02      3.08      60.50

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.145
                   TIME SHIFT OF PEAK FLOW         (min)= 50.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .037

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| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 74 |   QPEAK     (cms)=    .42 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  60.97
  Filename: C:H3Bi_25.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=311 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  60.50
  Filename: C:H3Bo_25.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
********************** 1:50 YEAR RAINFALL STORM ************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1300.000
| Ptotal= 78.30 mm |                          B=   4.700
--------------------                          C=    .780
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  6.00 hrs
                       Storm time step    =  5.00 min
                       Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    3.14 |  1.58    8.54 |  3.08   10.55 |  4.58    4.39
                 .17    3.24 |  1.67    9.63 |  3.17    9.70 |  4.67    4.26
                 .25    3.35 |  1.75   11.08 |  3.25    8.99 |  4.75    4.15
                 .33    3.47 |  1.83   13.12 |  3.33    8.39 |  4.83    4.03
                 .42    3.60 |  1.92   16.22 |  3.42    7.87 |  4.92    3.93
                 .50    3.75 |  2.00   21.53 |  3.50    7.42 |  5.00    3.83
                 .58    3.90 |  2.08   32.76 |  3.58    7.02 |  5.08    3.74
                 .67    4.07 |  2.17   72.28 |  3.67    6.67 |  5.17    3.65
                 .75    4.26 |  2.25  220.93 |  3.75    6.35 |  5.25    3.57
                 .83    4.47 |  2.33   86.28 |  3.83    6.07 |  5.33    3.49
                 .92    4.70 |  2.42   46.39 |  3.92    5.81 |  5.42    3.42
                1.00    4.96 |  2.50   31.76 |  4.00    5.58 |  5.50    3.34
                1.08    5.26 |  2.58   24.27 |  4.08    5.36 |  5.58    3.28
                1.17    5.60 |  2.67   19.73 |  4.17    5.17 |  5.67    3.21
                1.25    6.00 |  2.75   16.69 |  4.25    4.99 |  5.75    3.15
                1.33    6.46 |  2.83   14.50 |  4.33    4.82 |  5.83    3.09
                1.42    7.02 |  2.92   12.86 |  4.42    4.67 |  5.92    3.03
                1.50    7.70 |  3.00   11.58 |  4.50    4.52 |  6.00    2.98

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  76.60   Dir. Conn.(%)=  76.60
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .75          .23
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250
```

```
        NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                                IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     220.93        79.34
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.45 (ii)    4.94 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .22
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .45          .05           .50 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25          2.25
     RUNOFF VOLUME    (mm)=      77.25        38.55         68.19
     TOTAL RAINFALL   (mm)=      78.05        78.05         78.05
     RUNOFF COEFFICIENT   =        .99          .49           .87

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .0497

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)        .98       .50      2.25      68.19
     OUTFLOW: ID= 8 (2025)        .98       .03      3.08      67.76

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.255
                   TIME SHIFT OF PEAK FLOW         (min)= 50.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .044

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*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 74 |   QPEAK     (cms)=    .50 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  68.19
  Filename: C:H2Bi_50.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=318 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  67.76
  Filename: C:H2Bo_50.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*-------------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*-------------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
```

```
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     220.93        79.34
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.42 (ii)    4.77 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .22
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .43          .04           .47 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25          2.25
     RUNOFF VOLUME    (mm)=      77.25        38.55         68.89
     TOTAL RAINFALL   (mm)=      78.05        78.05         78.05
     RUNOFF COEFFICIENT   =        .99          .49           .88

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)        .92       .47      2.25      68.89
     OUTFLOW: ID=10 (2035)        .92       .02      3.08      68.42

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.169
                   TIME SHIFT OF PEAK FLOW         (min)= 50.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .042

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 74 |   QPEAK     (cms)=    .47 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  68.89
  Filename: C:H3Bi_50.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=318 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  68.42
  Filename: C:H3Bo_50.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
********************** 1:100 YEAR RAINFALL STORM ************************
```

```
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1450.000
| Ptotal= 87.30 mm |                          B=   4.900
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  6.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .38

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08    3.51 |  1.58    9.57 |  3.08   11.82 |  4.58    4.90
                 .17    3.62 |  1.67   10.79 |  3.17   10.87 |  4.67    4.76
                 .25    3.75 |  1.75   12.42 |  3.25   10.07 |  4.75    4.63
                 .33    3.88 |  1.83   14.71 |  3.33    9.39 |  4.83    4.51
                 .42    4.03 |  1.92   18.20 |  3.42    8.81 |  4.92    4.39
                 .50    4.18 |  2.00   24.18 |  3.50    8.30 |  5.00    4.28
                 .58    4.36 |  2.08   36.82 |  3.58    7.85 |  5.08    4.18
                 .67    4.55 |  2.17   80.92 |  3.67    7.46 |  5.17    4.08
                 .75    4.76 |  2.25  242.53 |  3.75    7.10 |  5.25    3.99
                 .83    4.99 |  2.33   96.48 |  3.83    6.79 |  5.33    3.90
                 .92    5.25 |  2.42   52.11 |  3.92    6.50 |  5.42    3.82
                1.00    5.55 |  2.50   35.69 |  4.00    6.24 |  5.50    3.74
                1.08    5.88 |  2.58   27.26 |  4.08    6.00 |  5.58    3.66
                1.17    6.26 |  2.67   22.15 |  4.17    5.78 |  5.67    3.59
                1.25    6.71 |  2.75   18.73 |  4.25    5.58 |  5.75    3.52
                1.33    7.23 |  2.83   16.27 |  4.33    5.39 |  5.83    3.45
                1.42    7.86 |  2.92   14.42 |  4.42    5.22 |  5.92    3.39
                1.50    8.62 |  3.00   12.98 |  4.50    5.05 |  6.00    3.32

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  76.60   Dir. Conn.(%)=  76.60
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .75          .23
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                            IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     242.53        93.14
               over (min)          5.00         5.00
     Storage Coeff.  (min)=       1.40 (ii)    4.75 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .22
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .49          .06           .55 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25          2.25
     RUNOFF VOLUME    (mm)=      86.22        45.43         76.67
     TOTAL RAINFALL   (mm)=      87.02        87.02         87.02
     RUNOFF COEFFICIENT   =        .99          .52           .88

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
```

```
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .0497

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)        .98       .55      2.25      76.67
     OUTFLOW: ID= 8 (2025)        .98       .03      3.08      76.24

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.3190
                  TIME SHIFT OF PEAK FLOW         (min)= 50.0000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0497

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 74 |   QPEAK     (cms)=    .55 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  76.67
  Filename: C:H2Bi_100.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=325 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  76.24
  Filename: C:H2Bo_100.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*------------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*------------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .92
|ID= 3 DT= 5.0 min |   Total Imp(%)=  78.40   Dir. Conn.(%)=  78.40
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .72          .20
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.18        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     242.53        93.14
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.37 (ii)    4.59 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .23
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .48          .05          .53 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25          2.25
     RUNOFF VOLUME    (mm)=      86.22        45.43         77.41
     TOTAL RAINFALL   (mm)=      87.02        87.02         87.02
     RUNOFF COEFFICIENT   =        .99          .52           .89

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
```

```
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .0473

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)        .92       .53      2.25      77.41
     OUTFLOW: ID=10 (2035)        .92       .03      3.08      76.94

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.2320
                  TIME SHIFT OF PEAK FLOW         (min)= 50.0000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0473

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .92
| ID= 3   PCYC= 74 |   QPEAK     (cms)=    .53 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME      (mm)=  77.41
  Filename: C:H3Bi_100.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .92
| ID=10   PCYC=325 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME      (mm)=  76.94
  Filename: C:H3Bo_100.HYD
  Comments:

     (i)  PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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 FINISH
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APPENDIX G

HYDRAULIC MODELLING ANALYSIS
(PREPARED BY AECOM), JANUARY 14, 2011

AECOM
105 Commerce Valley Drive West, Floor 7 905 886 7022 tel
Markham, ON, Canada L3T 7W3 905 886 9494 fax
www.aecom.com

Technical Memorandum

To	Chris Ho – Mondiale Development Ltd.	Page 1
Subject	Consulting Engineering Services – Hydraulic Modelling Analysis for Proposed Development – Uptown Mississauga Phase 2	
From	AECOM Canada Limited.	
Date	January 14, 2011	Project Number 60190634

1. Introduction

a. Purpose

The purpose of this study was to undertake an update of previous assessments for the water and sanitary servicing requirements for the Pinnacle Uptown Mississauga Development which incorporates both the reduced density found in the revised servicing plan as well as the comments provided by the Region for the previous submission. Mondiale Development Ltd. retained AECOM (formerly Earth Tech) to undertake the detailed hydraulic analysis for evaluating the serviceability of the proposed Mississauga Uptown development via existing and proposed infrastructure for both the water and sanitary sewer system. The project was completed and delivered to Mondiale Development Ltd. in October 2008. Subsequent to the completion of the hydraulic study, Mondiale Development Ltd. reviewed the proposed plan with the Region of Peel. The discussions resulted in the approval of the Phase I development and that the remaining phasing of the development should be re-evaluated to address the Region's comments. The Region of Peel's comments for the initial submission of the hydraulic analysis (October 2008) are found in the **Appendix**. A drawing of the Preliminary Master Plan was provided by IBI and is displayed as Drawing No. SK-01 in the **Appendix**. A drawing of the General Below-Ground Services Plan was also provided and is displayed as Drawing No. C-100B in the **Appendix**.

To perform the hydraulic analyses, AECOM used the Region of Peel's existing hydraulic model developed in InfoWater. AECOM had previously developed this model and delivered it to Peel Region around mid-November 2002. The model was subsequently converted to InfoWater in 2006 and further updated in 2007 by AECOM. In estimating the water demand required for this proposed development area, AECOM was provided with a Concept Plan that displayed the proposed land use

for the Pinnacle Uptown Mississauga Development. The Concept Plan can be found in the **Appendix**.

Mondiale Development Ltd. retained AECOM to undertake the hydraulic analyses and to evaluate the impact of the new proposed development on the existing Region of Peel water distribution and sanitary sewer systems. In the original study conducted in 2008, the estimated total number of units for condominiums and townhouses was 3,530. According to the updated estimates for the proposed development received for this study, the total projected number of units is reduced to 2,408. Due to the significant reduction in the projected units, a high level assessment was required for the sanitary sewer system, as it was determined that the sanitary loads will be reduced since the original analysis. Therefore, sanitary sewer modelling analysis was not necessary. This Technical Memorandum presents the results of the analyses conducted.

During the initial submission, Hetek Solutions conducted hydrant testing to validate the model performance. Hydrant tests were completed between Eglinton Avenue and Ceremonial Drive on the 200mm watermain on Fairwind Drive and on the 300mm watermain on Hurontario Street. The observed and simulated pressures had an accuracy between 96.7% and 99.4%. Flow monitoring was also undertaken at the intersection of the 300mm sewermain along Cooksville Creek and the 375mm sewermain on Kingsbridge Garden Circle. Based on this exercise, it was determined that the average DWF along this sewermain was 15.7 L/s and the peak DWF was 29L/s. In the hydraulic model, the 2009 peak flow along this sewermain was 21 L/s, and therefore the modeled peak flow was adjusted accordingly.

b. Background

The Region of Peel encompasses three local municipalities, the City of Mississauga, the City of Brampton and the Town of Caledon. The majority of water service to the Region relies on the lake-based water supply from two major water treatment plants, Lorne Park Water Treatment Plant and Lakeview Water Treatment Plant. The Region's lake-based water supply system can generally be divided into 7 pressure zones. The Pinnacle Uptown Development is located just inside Pressure Zone 4 at the edge of the pressure boundary between Zone 4 and Zone 3. The water service to the proposed Pinnacle Uptown Development would be predominantly supplied by Hanlan Pumping Station – Z4HL.

The proposed development site is bounded by Eglinton Avenue West to the south, Ceremonial Drive to the north, Cooksville Creek to the west and Hurontario Street to the east. Phase 1 of the

development, which consists of Block 1, has already been approved by the Region. Phase 2 of the development consists of Blocks 2 to 4. Phase 1 and 2 are located entirely within the east side of the development.

Figure 1 presents a general overview of the proposed Pinnacle Uptown Mississauga Development.

Figure 1 - Pinnacle Uptown Mississauga Development Location

c. Scope of Work

Based on discussions and a review of the drawings provided to AECOM, the following works were to be completed as part of this study:

Hydraulic Analysis of Water Distribution System

- Perform a demand analysis to allocate the development water demand to the hydraulic model based on the following:
    - Latest Region's population projection within the system,
    - Population projection for the proposed development as per the Pinnacle Uptown Mississauga Concept Plan that was provided by IBI via email on January 4th, 2011, and

        - Region's design criteria.
- Confirm existing/proposed infrastructure and projected population within the site with the IBI Group at the start of the project. If any changes exist, the hydraulic model will be updated accordingly.
- Update the existing Region's hydraulic model with the required demand for the proposed development.
- Review existing and planned infrastructure with the Region of Peel and confirm water main locations and timing of proposed works and update the existing hydraulic model accordingly.
- Perform hydraulic analyses for the following scenarios to confirm the serviceability of the Region of Peel's existing and proposed infrastructure.
    - Average Day Demand (ADD), Maximum Day Demand (MDD), MDD plus Fire Flow and Peak Hour Demand (PHD) conditions for the following design years:
        - Year 2016
        - Year 2031
- Determine the necessary improvement upgrades and/or operational changes in the Region's water distribution system to accommodate desired growth within the proposed development.

Sanitary Sewer Capacity Assessment

- Undertake a high level assessment for the Region's sanitary sewer system based on the hydraulic modelling analysis results identified in the previous study dated October 2008 and the sanitary sewer loadings that will be calculated from the revised projected population within the proposed development (Phase 1 and 2). The sanitary sewer assessments will be based on the Region's recommended capacity limit of 80% as per the Region's comments from the October 2008 analysis.

Reports

- Submit final Technical Memorandum summarizing results of the hydraulic analysis.

2. Demand Results

a. Water System

It is recognized that several future developments will be located in the eastside of Hurontario Street in conjunction with the Uptown Mississauga Development; the projected demands for the developments located outside the Uptown Mississauga development will be addressed through the Region's Small Geographic Units (SGU's). The projected water demands within the proposed development were calculated using the projected population / units as per the Pinnacle Uptown Mississauga Concept Plan. The Concept Plan included the proposed number of units for each block of the development. The demand from each of these units was calculated based on the following standards for population density and water demand criteria:

- Average long term residential water demand = 280 L/ca/d
- Average short term residential water demand = 409 L/ca/d
- Industrial, Commercial and Institutional (ICI) Water Demand = 300L/Employee/d
- Population Density for Semi Detached Home = 3.8 person / unit[1]
- Population Density for Apartments = 2.4 person / unit
- ICI Population Density = 50 employees/ha
- Residential Maximum Day Demand Factor = 2 x Average Day Demand
- ICI Maximum Day Demand Factor = 1.4 x Average Day Demand
- Residential/ICI Peak Hour Demand Factor = 3 x Average Day Demand
- Fire flow for residential 4 storey buildings = 250 L/s (15000L/min as per FUS)

Appendix 1 displays a detailed chart of the demand calculations. A summary of the calculated demands is displayed below in Table 2.1. Also, all calculations have been completed using a short term water demand criteria for new developments of 409L/cap/d. This value is specified in the Region of Peel "ETPS Watermain Design Criteria". **Appendix 2** displays a map of the model node ID's.

[1] *Region of Peel Design Criteria Manual Section IV – Small Groundwater Supply System for Residential Developments, October 2000*

Table 2.1 - Water Demand Summary

Model Node ID	Ave. Day (L/s)	Max. Day (L/s)	Peak Hour (L/s)	Fire Flow (L/s)
N-102	7.8	15.5	23.3	250
N-105	14.1	28.0	42.2	250
N-101	5.0	9.9	14.9	250
N-100	1.6	3.3	4.9	250
Total	28.4	56.7	85.3	250*
*Fire Flow is applied one node at a time				

b. Sanitary System

In the October 2008 report for the Pinnacle Uptown Mississauga Development (Report No. 105591R01C), it was determined that the peak 2031 sanitary load (including infiltration and inflow) would be equal to 90.8L/s for the development. These calculations can be found below in Table 2.2.

Table 2.2 - October 2008 Report – Sanitary Load Allocation

Location	MHID	2009			2016 and 2031		
		Population	Peak Flow (L/s)	Peak Flow + II (L/s)	Population	Peak DWF (L/s)	Peak DWF + I/I** (L/s)
East Development	1793663	1680	17.5	17.8	1680	17.5	17.8
	1796277	0	0	0	6240	65	66.2
	Total	1680	17.5	17.8	7920	82.5	84
West Development	1796430	585	6.1	6.9	585	6.1	6.9
Total		2265	23.6	24.6	8505	88.6	90.76
* Based on a total population of 8,505, a Harmon Peaking Factor of 3.0 was applied to the average DWF.							
** I/I = Inflow and Infiltration							

Figure 2 displays an overview of the sanitary system with possible connection points. Connection point 1793663 links the development to the 300mm east trunk along Cooksville Creek and the 375mm sewermain on Kingsbridge Garden Circle. Connection point 1796277 links the development to the 750mm west trunk along Fairwind Drive.

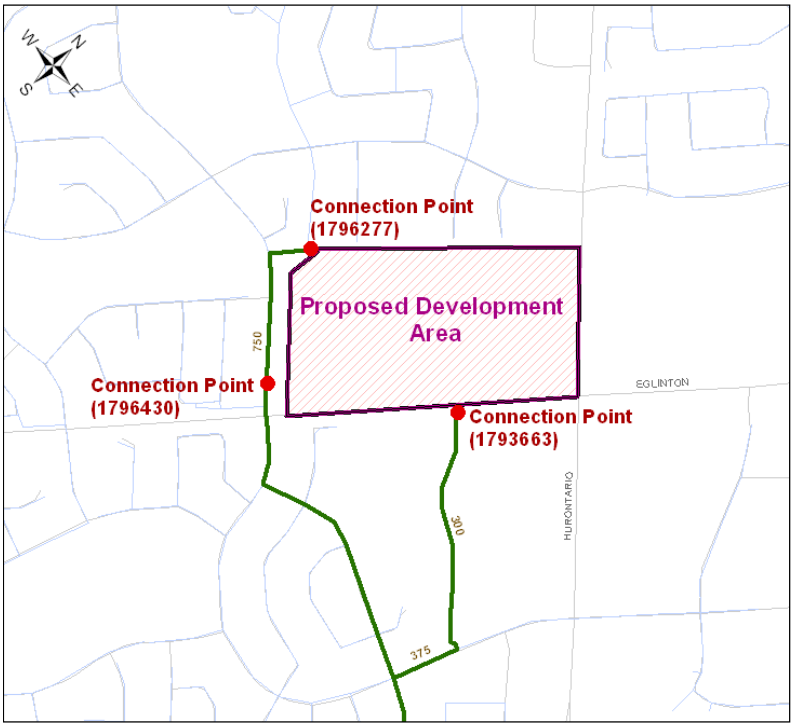


Figure 2 - Sanitary System Overview & Possible Connection Points

Using the population projections, the total sanitary load for the development was calculated based on the Region of Peel's Master Plan Design Criteria as follows:

Within Development

- Sanitary load generation rate: 300L/ca/d
- Peaking Factor: Harmon Peaking Factor
  - $M = 1 + 14/(4+p^{0.5})$
    - M = Ratio of peak flow to average flow
    - P = the tributary equivalent population in thousands
- Inflow/Infiltration rate: 17,280 L/Ha/d

Other Areas within Peel Region

- Sanitary loads located outside the new development are based on the latest Region's population projection data as per the Region's Small Geographic Units (SGU's).

The sanitary load generation rate criteria of 300 Lpcd already includes the Inflow and Infiltration (I/I) component. However, in accordance with the Region's Master Plan criteria, design of local collection system should project I/I flows over and above the 300 Lpcd base criteria. Therefore, an Inflow/Infiltration rate of 17,280 L/Ha/d was applied to the development study area. Table 2.3 below

presents the sanitary loads based on the updated population data provided by Mondiale Development Ltd.

Table 2.3 - Updated Sanitary Load Allocation

Location	Phase	Block	Commercial SQM	Commercial Equiv. Population	Residential Population	2016 and 2031					
						Equivalent Population	Area (Ha)	DWF (L/s)	Peak DWF (L/s)	I/I** (L/s)	Peak DWF + I/I** (L/s)
East Development	1	1	0	0	1097	1097	0.54	3.8	12.1	0.1	12.2
	2	2A	7990	40	665	705	1.54	2.4	7.8	0.3	29.6
		2B	0	0	665	665		2.3	7.3		
		2C	435	2	425	427		1.5	4.7		
		2D	457	2	314	317		1.1	3.5		
		2E	0	0	542	542		1.9	6.0		
		3	6179	31	1883	1914	1.53	6.6	21.1	0.3	21.4
		4	0	0	361	361	1.96	1.3	4.0	0.4	4.4
		Other	0	0	0	0	4.29	0.0	0.0	0.9	0.9
	Total		15061	75	5953	6028	9.86	20.9	66.4	2.0	68.4
West Development	2		0	0	0	0	4.92	0.0	0.0	1.0	1.0
Total			15061	75	5953	6028	14.78	20.9	66.4	3.0	69.4
Harmon peaking factor =		3.174									
Inflow/Infiltration rate =		17280	L/ha/d								
** I/I = Inflow and Infiltration											

3. Hydraulic Impact Assessment Basis

a. Water System

The hydraulic analysis results were assessed in accordance with the water distribution design guidelines provided by the Ministry of the Environment (MOE). As stated in the MOE Guidelines, water supply systems should be designed to satisfy the greater of either of the following demands:

- Maximum day plus fire flow; or,
- Peak hour (maximum hourly) demand

The maximum day demand is the average usage rate on the maximum day. The fire demand varies with the size of the municipality and the nature of the development. The level of fire protection to be provided in a municipality owned potable water system is the decision of the municipality. The peak rate demand is the short-term demand placed on the system by usages other than fire fighting. The peak rate demand is usually taken as the average water usage over the maximum hour. According to MOE Guidelines, water distribution systems should be designed so that the normal operating pressure ranges between 350 kPa and 700 kPa (50-100 psi) under conditions of maximum day demand. Similarly, under conditions of peak hour demand, the system pressures should not be less than 275 kPa (40 psi). Under conditions of simultaneous maximum day and fire flow demands, the pressure should not be less than 140 kPa (20 psi).

b. Sanitary System

The high level assessment of the sanitary system involved a comparison between the sanitary load allocation from the October 2008 report and the adjusted sanitary load allocation based on the data provided in the Concept Plan. In addition to the update of the sanitary load from the development, a review of the sanitary sewer results was required. This review was required because, in the previous report, the analysis was conducted for pipes that exceeded 90% of its full capacity. However, as per the Region's comments from the previous report, sanitary sewers within existing residential areas should not be more than 80% full under peak flow conditions. Therefore, a review of the sanitary sewer results was undertaken to ensure that q/Q (flow over full pipe capacity) does not exceed the recommended 80% limit.

4. Hydraulic Analysis Results

a. Steady State Analysis

Steady state hydraulic simulations were performed under different demand conditions to assess the serviceability of the proposed development via the Zone 4 water distribution system in Peel Region. For the purposes of the analysis, only watermains along major roads were added to the hydraulic model and demands were added to the node closest to where the demand would occur. A map of the development with pipe ID's and diameters is shown in **Appendix 3**. The analysis was undertaken to confirm serviceability for Average Day, Maximum Day and Peak Hour conditions. The pressure within the system was reviewed with and without the proposed development area. The principal analysis was undertaken with servicing to the development being provided via the 300mm diameter watermain on Salishan Circle (Model ID P-101) and the 300mm diameter watermain on Hurontario Street (Model ID P-107). The results of the analyses can be found in Table 4.1 and 4.2 for the 2016 and 2031 years, respectively.

Table 4.1 - 2016 Pressure Summary

2016	Pressure (psi)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	*100.49	99.88	89.18	87.60	80.97	77.50
595470	*101.99	*101.37	90.67	89.07	82.44	78.94
N-100	*104.06	*103.44	92.75	91.14	84.54	81.00
N-101	*101.93	*101.30	90.62	88.95	82.41	78.76
N-102	*104.63	*103.99	93.32	91.63	85.12	81.42
N-103	*107.19	*106.55	95.88	94.19	87.68	83.98
N-104	*104.92	*104.27	93.61	91.92	85.41	81.71
N-105	99.09	98.45	87.79	86.09	79.60	75.88
N-106	*103.78	*103.14	92.47	90.78	84.28	80.57
* Above MOE recommended operating pressure						

Note that a different number of pumps were turned on in the different demand conditions (ADD, MDD and PHD) and in the different years (2016 and 2031), in order to maintain the required hydraulic grade line (HGL) in the system.

Table 4.2 - 2031 Pressure Summary

2031	Pressure (psi)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	*101.00	*100.37	*105.94	*104.38	80.82	77.11
595470	*102.50	*101.86	*107.42	*105.85	82.28	78.54
N-100	*104.57	*103.93	*109.50	*107.91	84.38	80.60
N-101	*102.44	*101.78	*107.37	*105.73	82.26	78.37
N-102	*105.14	*104.48	*110.08	*108.41	84.96	81.02
N-103	*107.70	*107.03	*112.64	*110.97	87.52	83.58
N-104	*105.43	*104.76	*110.37	*108.70	85.26	81.31
N-105	99.60	98.93	*104.54	*102.87	79.44	75.49
N-106	*104.29	*103.62	*109.23	*107.56	84.12	80.17
* Above MOE recommended operating pressure						

Note that a different number of pumps were turned on in the different demand conditions (ADD, MDD and PHD) and in the different years (2016 and 2031), in order to maintain the required HGL in the system.

From these results, it can be concluded that the minimum pressure guideline is satisfied within the development region; however, the maximum pressure in the development region is often higher than the MOE maximum pressure guideline. This is anticipated because the development is on the edge of

the Zone 3 and Zone 4 pressure boundary. According to the hydraulic modelling results, the hydraulic grade line for these high pressure problem areas was between 241m and 244m. This is consistent with the design hydraulic grade line for Zone 4 of 243.8m. So the high pressure issue is mainly contributed by the elevation of the proposed development. A hydraulic grade line summary can be found in **Appendix 4**. **Appendix 5** displays the elevations of each node within the development. It should also be noted that these high pressures are not caused by the addition of the new development. This can be seen when comparing pressures before and after development because they were only slightly altered by the addition of the new infrastructure and demands. It should also be remarked that there is adequate water service security to the system, due to the looping provided in the development.

b. Pipe Velocity and Head Loss Gradient

Velocities within the surrounding watermains were checked during the analyses. It was found that the velocities remained within acceptable limits and therefore would not have a significant impact on the Peel water system. Even under peak hour conditions, velocity in the pipes was observed to not exceed 1m/s and the head loss gradient was observed not to exceed 2m/km. Therefore, the proposed sizing and routing of watermains in the development is considered appropriate. Maps of velocity, flow and headloss gradients in the 2031 peak hour demand scenario can be found in **Appendices 6, 7** and **8**, respectively.

c. Fire Flow Analysis

The system's response under Maximum Day plus Fire Flow demand conditions was also assessed to confirm the serviceability of the proposed development. As mentioned in Section 2, the fire flow used for the analysis was 250L/s for a typical residential building greater than 4 storeys.

For the analysis, a fire flow of 250L/s was applied to each of the new demand nodes (one node at a time) under maximum day demand conditions for each design year. The hydraulic model then calculated the residual pressure at the nodes within the development under these demand conditions. The minimum pressure should not be lower than 20 psi as per the MOE Design Guidelines. Detailed fire flow results are presented in **Appendix 9**.

Based on these results, the system is considered to have the required capacity to service the additional water demand associated with the new proposed development. Significantly more than 250L/s can be supplied to the new development nodes without causing pressures to drop below the minimum 20psi. So, the maximum available fire flow that is still able to maintain at least 20psi within the development was calculated and is provided in Table 4.3 below.

Table 4.3 - Fire Flow Summary

Year	Critical Node ID	Available Flow at Hydrant (L/s)
2016	N-105	629.6
2031	N-102	724.2

5. Sanitary Sewer System Assessment

The sanitary system, in particular along the trunk sewers downstream of the connection points up to Lakeview WWTP, was assessed and the capacity within the system was reviewed for pre and post development conditions. A sewermain was considered to be 'full' and flagged once it reached 80% of its full pipe capacity.

The maximum allowable conveyance capacity is evaluated for the East and West trunks that are directly connected to the proposed development. Table 5.1 and Table 5.2 summarize the analysis results for year 2016 and year 2031 system conditions, respectively. According to the results, the maximum allowable conveyance capacity to the east trunk is 38.1 L/s and 37.5 L/s in 2016 and 2031, respectively. The maximum allowable conveyance capacity to the west trunk is 200.9 L/s and 212.2 L/s in 2016 and 2031, respectively.

Now referring back to Table 2.3, it can be seen that Phase 1 of the development (Block 1) has a peak sanitary load of 12.2 L/s. Therefore, Phase 1 of the development can be serviced by the east trunk (through connection point 1793663); which is consistent with the Region's approval.

Phase 2 of the development (Blocks 2, 3, 4 and open space) has an additional peak flow contribution of 57.2 L/s. Therefore, Phase 2 of the development can be partially serviced by the East trunk (capacity of 25.3L/s is available for Phase 2). This means that the East trunk can also service the additional 20.7L/s load from Block 4 and Towers 2B and 2E of Block 2. The remaining sanitary sewer loadings must be serviced by the west trunk (through connection point 1796277).

Table 5.1 – 2016 – East and West Trunk Sewermains

ID	Location	Diameter (mm)	Length (m)	Slope	Full Flow (L/s)	Without Development		Maximum Available Capacity while maintaining 80% Full (L/s)
						Total Flow (L/s)	q/Q	
238305	East Trunk	375	136	0.003	108.5	48.8	0.45	38.1
239906	East Trunk	300	56	0.004	84.2	27.7	0.33	39.7
239905	East Trunk	300	66	0.005	89.1	28.6	0.32	42.7
239907	East Trunk	300	63	0.004	82.5	26.2	0.32	39.8
239904	East Trunk	300	11	0.005	93.1	29.0	0.31	45.4
239908	East Trunk	300	73	0.004	83.7	22.9	0.27	44.1
239909	East Trunk	300	46	0.004	83.6	21.2	0.25	45.6
239910	East Trunk	300	77	0.004	83.8	19.7	0.24	47.3
239903	East Trunk	300	12	0.012	139.1	29.3	0.21	82.0
239911	East Trunk	300	69	0.004	83.4	14.2	0.17	52.5
238389	West Trunk	750	40	0.002	571.5	256.3	0.45	200.9
238303	West Trunk	750	185	0.003	665.4	261.0	0.39	271.3
238302	West Trunk	750	67	0.003	663.2	259.8	0.39	270.8
238037	West Trunk	750	49	0.003	669.1	260.2	0.39	275.1
238301	West Trunk	750	90	0.003	662.4	256.9	0.39	273.0
238300	West Trunk	750	47	0.003	663.6	256.7	0.39	274.1
238299	West Trunk	750	66	0.003	665.7	256.5	0.39	276.1
238378	West Trunk	750	170	0.003	655.9	251.8	0.38	272.9
238379	West Trunk	750	74	0.003	673.3	252.5	0.38	286.1
238390	West Trunk	750	52	0.005	820.0	251.4	0.31	404.6
247127	West Trunk	750	74	0.005	846.4	245.2	0.29	431.9
238038	West Trunk	750	89	0.01	1212.8	262.5	0.22	707.7

Table 5.2 – 2031 – East and West Trunk Sewermains

ID	Location	Diameter (mm)	Length (m)	Slope	Full Flow (L/s)	Without Development		Maximum Available Capacity while maintaining 80% Full (L/s)
						Total Flow (L/s)	q/Q	
238305	East Trunk	375	136	0.003	108.5	49.3	0.45	37.5
239906	East Trunk	300	56	0.004	84.2	27.1	0.32	40.3
239905	East Trunk	300	66	0.005	89.1	28.0	0.31	43.3
239907	East Trunk	300	63	0.004	82.5	25.8	0.31	40.3
239904	East Trunk	300	11	0.005	93.1	28.4	0.31	46.1
239908	East Trunk	300	73	0.004	83.7	22.6	0.27	44.4
239909	East Trunk	300	46	0.004	83.6	21.0	0.25	45.8
239910	East Trunk	300	77	0.004	83.8	19.6	0.23	47.4
239903	East Trunk	300	12	0.012	139.1	28.6	0.21	82.7
239911	East Trunk	300	69	0.004	83.4	14.4	0.17	52.4
238389	West Trunk	750	40	0.002	571.5	245.0	0.43	212.2
238303	West Trunk	750	185	0.003	665.4	249.6	0.38	282.7
238302	West Trunk	750	67	0.003	663.2	248.4	0.38	282.2
238037	West Trunk	750	49	0.003	669.1	248.8	0.37	286.5
238378	West Trunk	750	170	0.003	655.9	243.1	0.37	281.6
238301	West Trunk	750	90	0.003	662.4	245.5	0.37	284.4
238300	West Trunk	750	47	0.003	663.6	245.3	0.37	285.5
238299	West Trunk	750	66	0.003	665.7	245.1	0.37	287.4
238379	West Trunk	750	74	0.003	673.3	243.3	0.36	295.3
238390	West Trunk	750	52	0.005	820.0	242.9	0.30	413.1
247127	West Trunk	750	74	0.005	846.4	236.8	0.28	440.3
238038	West Trunk	750	89	0.01	1212.8	251.1	0.21	719.1

The sanitary sewermains capacity for the downstream common connection of the West Trunk and the East Trunk to the Lakeview Wastewater Treatment Plant is evaluated. Table 5.3 and Table 5.4 show the lists of the trunk sewers with q/Q (flow over full pipe capacity) larger or equal to 0.8.

Table 5.3 – 2016 – Sewermains with q/Q > 0.8

ID	Diameter (mm)	Length (m)	Slope	Existing Capacity (L/s)	Without Development		With Development*		Proposed Upgraded Diameter **	Proposed Installed Year **
					Total Flow (L/s)	q/Q	Total Flow (L/s)	q/Q		
243292	1,350	170	0	344.5	741.0	2.15	810.4	2.35	1350	2006
487280	2,400	356	0	3946.8	7526.6	1.91	7596.0	1.92	N/A	N/A
243293	1,200	203	0	841.8	741.6	0.88	811.0	0.96	1350	2026
243295	1,200	116	0	881.2	742.6	0.84	812.0	0.92	1350	2031
243285	1,200	193	0	861.4	704.9	0.82	774.3	0.90	1350	2031
243209	1,050	116	0	1,030.1	830.6	0.81	900.0	0.87	N/A	N/A
243287	1,200	143	0	902.0	705.8	0.78	775.2	0.86	N/A	N/A

* "With Development" flows are calculated by adding the peak development flow of 69.4L/s to the existing flow in each main

** Proposed according to the previously undertaken study: *Updated MCC Water and Wastewater Servicing Report – October 2005*

Table 5.4 – 2031 – Sewermains with q/Q > 0.8

ID	Diameter (mm)	Length (m)	Slope	Pipe Capacity (L/s)	Without Development		With Development*	
					Total Flow (L/s)	q/Q	Total Flow (L/s)	q/Q
243292	1,350	170	0	344.5	785.2	2.28	854.6	2.48
487280	2,400	356	0	3946.8	8220.7	2.08	8290.1	2.10
243293	1,350	203	0	1,152.4	785.8	0.68	811.0	0.74
243295	1,350	116	0	1,206.4	786.8	0.65	856.2	0.71
243285	1,350	193	0	1,179.2	749.7	0.64	819.1	0.69
243209	1,200	116	0	1,470.7	876.3	0.60	945.7	0.64
243287	1,350	143	0	1234.9	750.4	0.64	819.8	0.66

* "With Development" flows are calculated by adding the peak development flow of 69.4L/s to the existing flow in each main

** Proposed according to the previously undertaken study: *Updated MCC Water and Wastewater Servicing Report – October 2005*

As shown in Table 5.3, an additional five pipes will exceed 80% capacity with the proposed development in the 2016 peak flow scenario. Four of these sewermains exceed 80% capacity without the proposed development. These sewermains (model pipe ID 243293, 243295, 243285, 243209 and 243287) exceed 80% capacity, but can be mitigated when the Region implements the recommended upgrades as proposed in the "*Updated MCC Water and Wastewater Servicing Report – October 2005*".

In 2031 peak flow scenario, five of the seven sewermains that are identified as constraints in the 2016 scenario can be mitigated by future system upgrades. The two sewermains (model pipe ID 243292 and 487280), which are located south of Mississauga Valley Boulevard (between Lolita Gardens and Silver Creek Boulevard) and on Fergus Avenue north of Lakeshore Boulevard respectively, were identified as system constraints for both the 2016 and 2031 scenarios. The results also show that the capacity constraints of these sewermains were not caused by the addition of the proposed development since the q/Q is already exceeding the limit under pre development conditions. This is

consistent with the previous detailed report. In the previous report, it was recommended that the Region further evaluate these sewermains. This recommendation still stands.

6. Conclusions

a. Water System

Hydraulic analysis was performed using the latest updated Region of Peel's water model and the following results have been obtained:

- There is no significant hydraulic impact to the system when adding the proposed development. Some pressures were observed to exceed the maximum pressure based on MOE guidelines in the average and maximum day demand scenarios. However, as described in Section 4a) this is not caused by the addition of the proposed new development.
- Servicing to the development can be supplied via the 300mm diameter watermain along Salishan Circle and the 300mm diameter watermain along Hurontario Street. These watermains also provide sufficient water service security to the system because of the looping that would exist within the development.
- The velocity and head loss gradients within the pipes in the development would be within the recommended range under average day, maximum day and peak hour conditions. Therefore, the sizing and routing of the pipes is considered acceptable.
- Allowable fire flow that maintains the minimum required pressure of 20psi was calculated. The critical fire flow location was observed to be at node N-105 in the 2016 scenario and N-102 in the 2031 scenario. The required 250L/s fire flow could be supplied within the recommended pressures. The maximum available fire flows were calculated as follows:
    - 2016: 433.2 L/s
    - 2031: 492.9 L/s
- It is recommended to undertake a hydrant flow test at the existing connection for the proposed development to further validate the hydraulic modelling results presented herein.

b. Sanitary System

A high level assessment of the sanitary sewer system was performed using the results from the previous report and the updated sanitary loads and the following results have been obtained:

- Two sewermains were identified as system constraints for both the 2016 and 2031 scenarios, due to the q/Q being greater than 0.8. This is not caused by the addition of the new development. These results are consistent with the previous report's findings. As suggested in the previous report, it is recommended for the Region to evaluate these sewermains in further detail.
- In the 2016 scenario, an additional five pipes will exceed 80% capacity with the development. Four of these exceed 80% capacity without the development. The capacity issue with these watermains can be mitigated when the Region implements the recommended upgrades as proposed in the "*Updated MCC Water and Wastewater Servicing Report – October 2005*". This is demonstrated in the 2031 analysis where these five pipes are within the recommended q/Q limits.
- The maximum allowable conveyance capacities remaining in the east and west trunks are the following:
    - 2016 Scenario
        - East Trunk: 38.1 L/s
        - West Trunk: 200.9 L/s
    - 2031 Scenario
        - East Trunk: 37.5 L/s
        - West Trunk: 212.2 L/s
- Phase 1 of the development (12.2 L/s) can be serviced by the east trunk (through connection point 1793663); which is consistent with the Region's approval.
- Phase 2 of the development (57.2 L/s) can be partially serviced by the East trunk (capacity of 25.3L/s is available for Phase 2) and the remaining sanitary sewer loadings (31.9 L/s) must be serviced by the West trunk (through connection point 1796277).

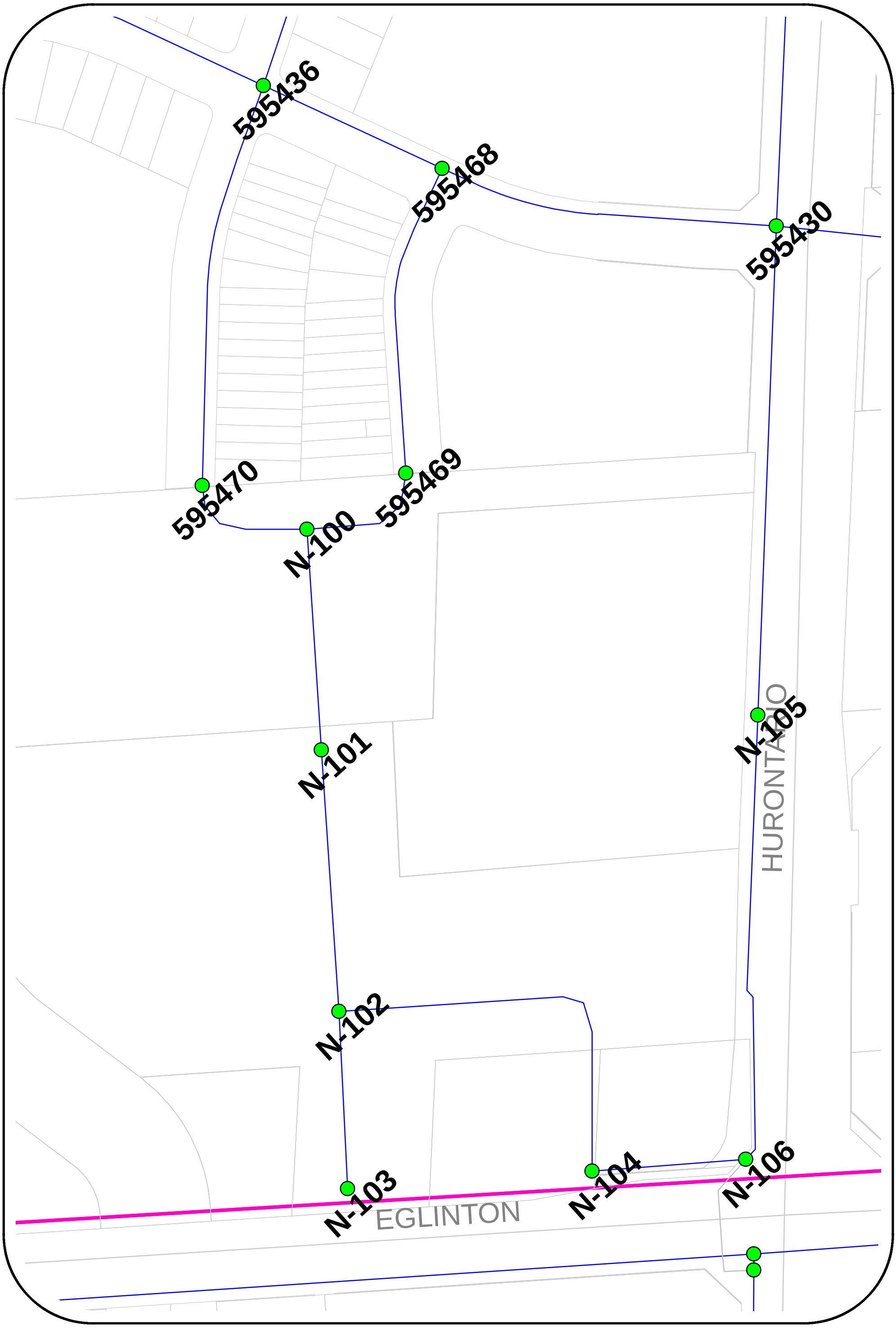
Appendices

Appendix 1: Detailed Demand Calculations

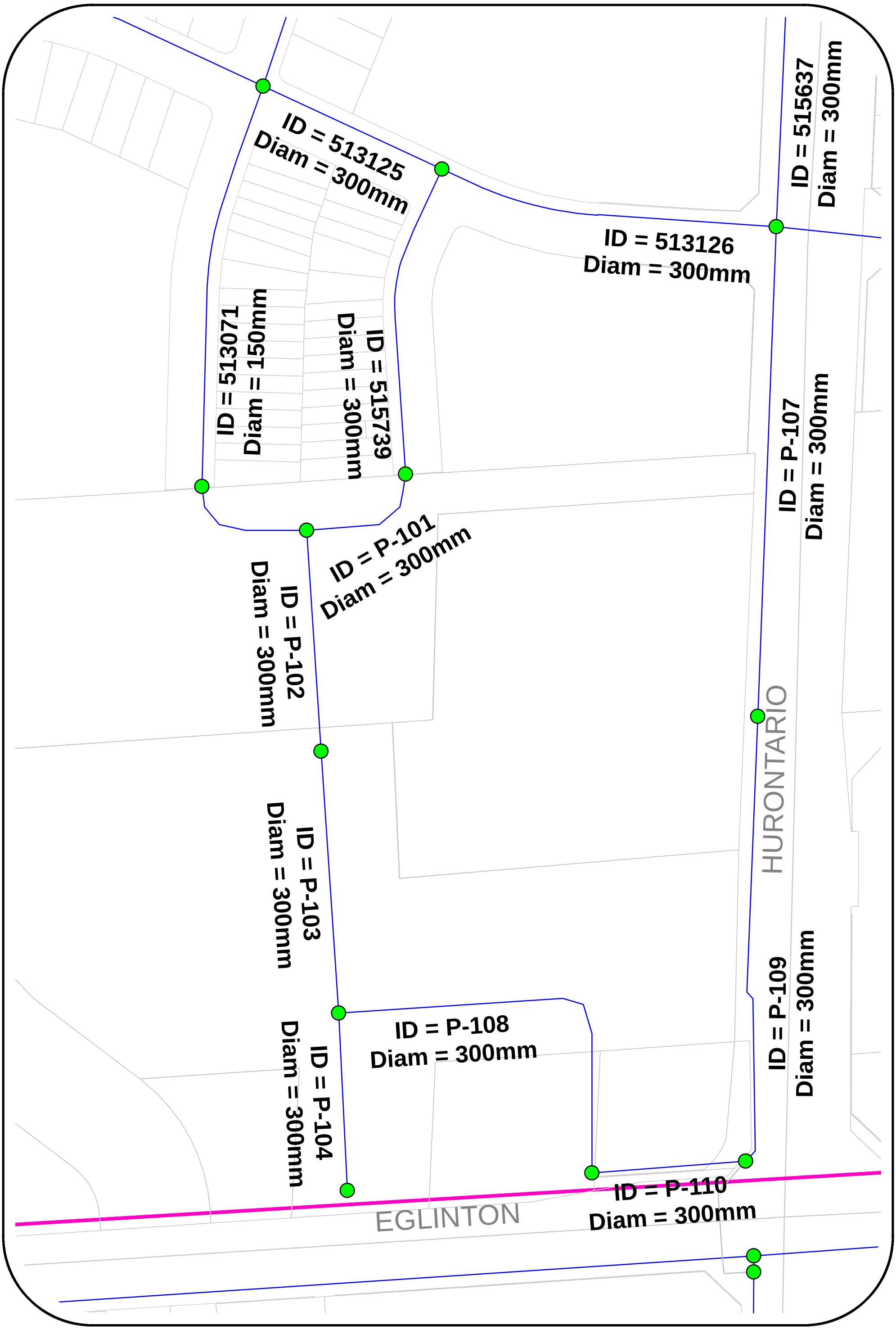
East Parcel	Block	Area (ha)	BLDG No.	Model Node ID	Node Description	Commercial			# of Units			Population	Residential ADD (L/s)	
						Retail SQM	Office SQM	Comm. ADD	Condo @ 85m2	Town	BLK Total		Short Term	Long Term
Phase 1	Block 1 -Mixed Use	0.54	1A	N-102	Little Creek Rd. and Four Springs Ave. Intersection	0	0	0	230	10	445	590	2.8	1.9
			1B	N-102					194	11		507	2.4	1.6
	Phase Subtotal	0.54							424	21		1097	5.2	3.6
Phase 2	Block 2 -Mixed Use	1.54	2A	N-105	Hurontario and Street "B" Intersection	3995	3995	0.1	277	0	1088	665	3.1	2.2
			2B	N-105					277	0		665	3.1	2.2
			2C	N-105		435		0.0	177	0		425	2.0	1.4
			2D	N-101	Four Springs Ave. and Street "B" Intersection	457		0.0	131	0		314	1.5	1.0
			2E	N-102	Little Creek Rd. and Four Springs Ave. Intersection				226	0		542	2.6	1.8
	Block 3- Mixed Use	1.53	3A	N-105	Hurontario and Street "B" Intersection	2590	2590	0.1	277	0	780	665	3.1	2.2
			3B	N-105		490		0.0	208	0		499	2.4	1.6
			3C	N-101	Four Springs Ave. and Street "B" Intersection	509		0.0	155	0		372	1.8	1.2
			3D	N-100	Four Springs and Salishan Circle Intersection				132	0		317	1.5	1.0
			3E	N-100					0	8		30	0.1	0.1
	Block 4 - Residential	1.96		N-101	Four Springs Ave. and Street "B" Intersection				0	95	95	361	1.7	1.2
	Phase Subtotal	5.03							1860	103		4855	23.0	15.7
Other	Open Space	1.53							0	0		0	0.0	0.0
	Public Roads	2.5							0	0		0	0.0	0.0
	Road Widenings	0.26							0	0		0	0.0	0.0
East Parcel Total		9.86				15061			2284	124	2408	5953	28.2	19.3
West Parcel	Block 8 - Hazard Land	0.58												
	Block 9 -Open Space	4.14												
	Road	0.2												
West Parcel Total		4.92												

Longterm Residential Water Demand =	280	L/cap*d	
Short Term Residential Water Demand =	409	L/cap*d	
Fire Flow Demand (4 storey residential)=	15000	L/min	250 L/s
ICI Water Demand	300	L/Employee*d	
Semi- Detached Home: Population Density =	3.8	population/unit	
Apartments: Population Density =	2.4	population/unit	
Commercial Density =	50	employees/ha	
Residential Maximum Day Demand Factor =	2	x Ave. Day Demand	
Residential Peak Hour Demand Factor =	3	x Ave. Day Demand	
ICI Max. Day Demand Factor =	1.4	x Ave. Day Demand	
ICI Peak Hour Demand Factor =	3	x Ave. Day Demand	

Pinnacle Uptown Mississauga Development
Appendix 2: Node Model ID's



Pinnacle Uptown Mississauga Development
Appendix 3: Pipe Model ID's and Diameters



Appendix 4: Hydraulic Grade Line Summary

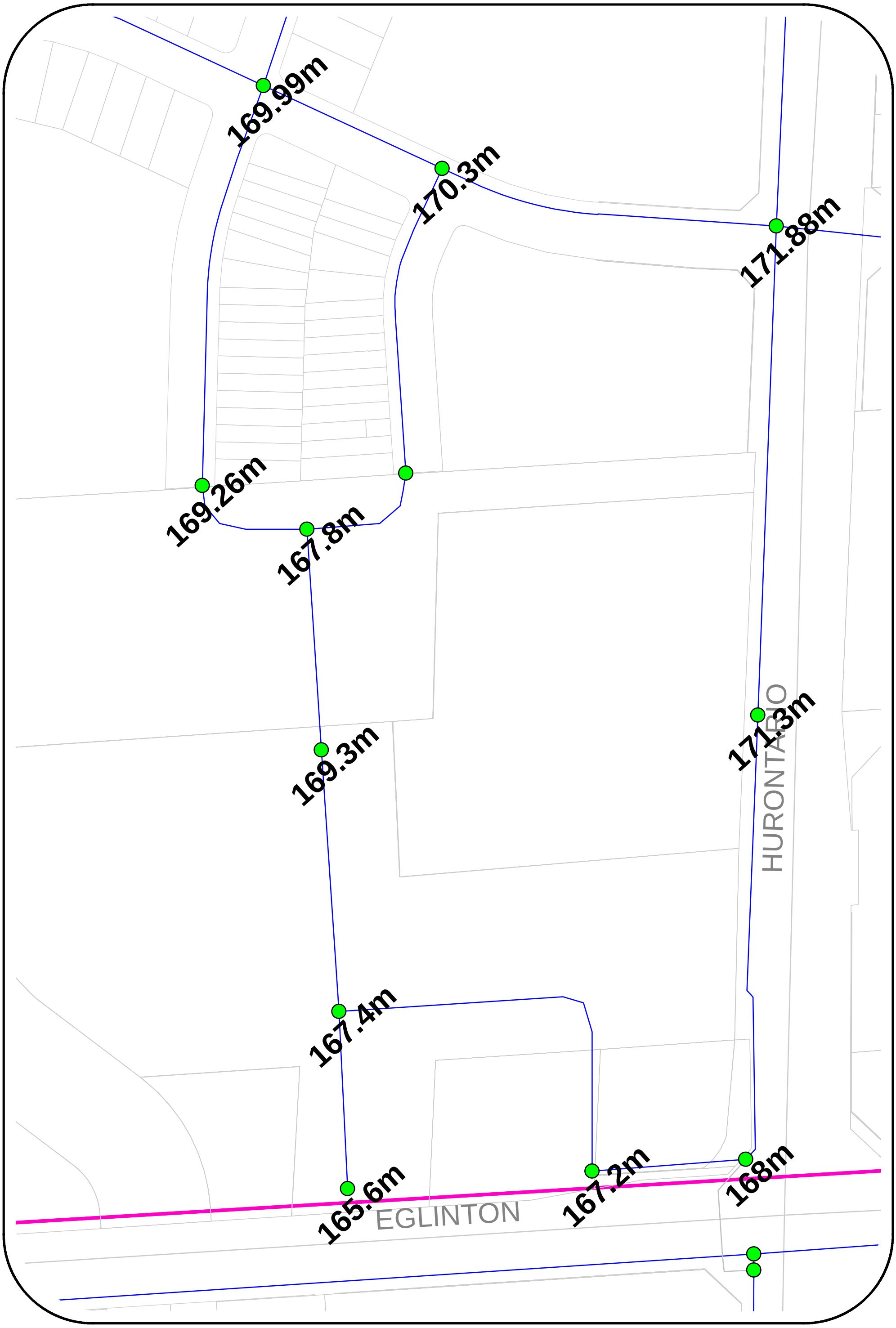
2016	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.00	240.57	233.04	231.93	227.27	224.83
595470	241.00	240.57	233.03	231.91	227.24	224.79
N-100	241.00	240.57	233.04	231.91	227.27	224.78
N-101	241.00	240.56	233.05	231.87	227.27	224.70
N-102	241.00	240.55	233.05	231.86	227.27	224.67
N-103	241.00	240.55	233.05	231.86	227.27	224.67
N-104	241.00	240.55	233.05	231.86	227.28	224.68
N-105	241.00	240.55	233.05	231.86	227.29	224.68
N-106	241.00	240.55	233.05	231.86	227.28	224.68
Zone 4 HGL = 243.8m						
Zone 3 HGL = 213.3m						

Note that a different number of pumps were turned on in the different demand conditions (ADD, MDD and PHD) and in the different years (2016 and 2031), in order to maintain the required HGL in the system.

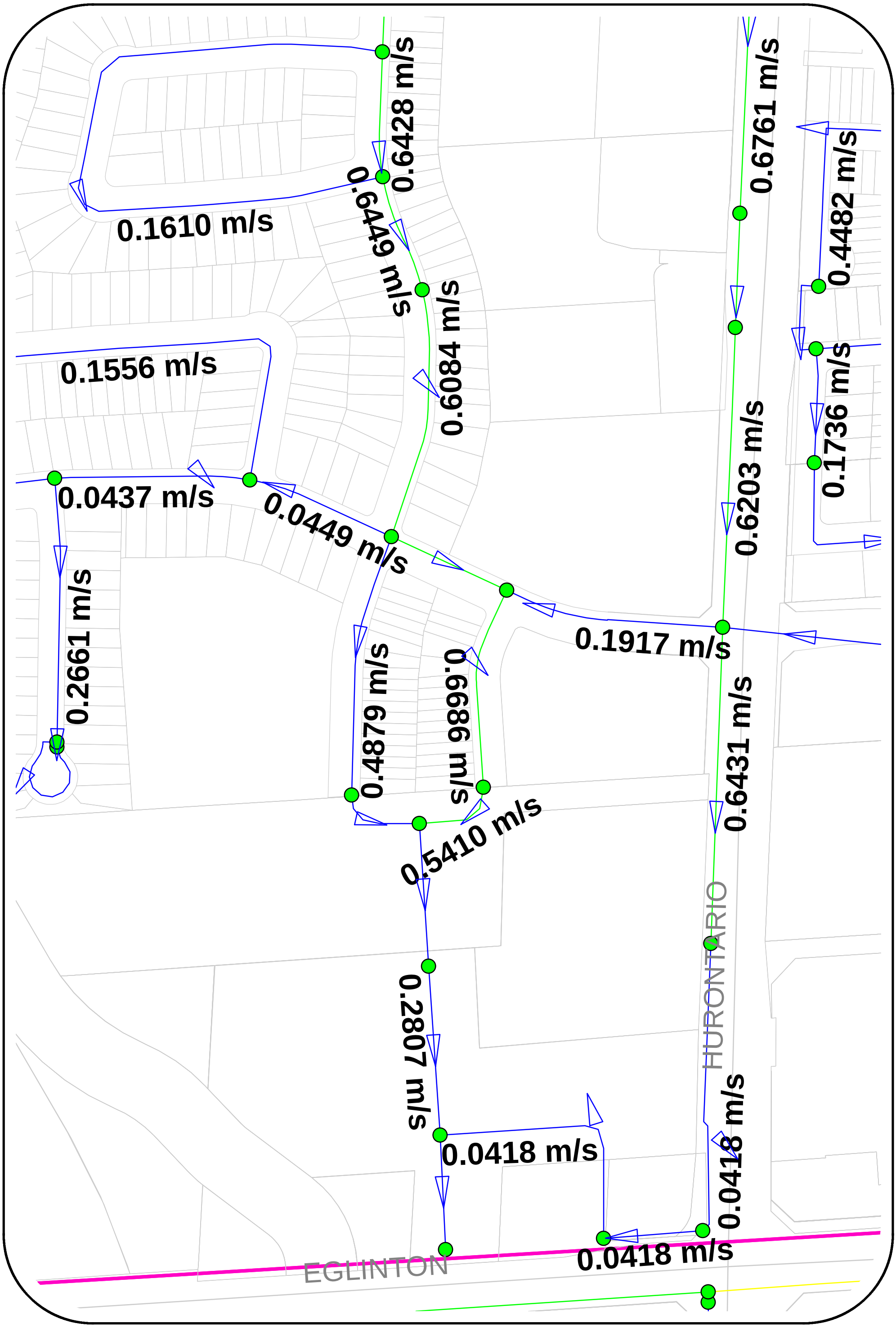
2031	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.36	240.91	244.83	243.73	227.16	224.55
595470	241.36	240.91	244.82	243.71	227.13	224.50
N-100	241.36	240.91	244.83	243.71	227.16	224.50
N-101	241.36	240.90	244.83	243.68	227.16	224.43
N-102	241.36	240.89	244.83	243.66	227.17	224.40
N-103	241.36	240.89	244.83	243.66	227.17	224.40
N-104	241.36	240.89	244.84	243.66	227.17	224.40
N-105	241.36	240.89	244.84	243.66	227.18	224.40
N-106	241.36	240.89	244.84	243.7	227.17	224.4
Zone 4 HGL = 243.8m						
Zone 3 HGL = 213.3m						

Note that a different number of pumps were turned on in the different demand conditions (ADD, MDD and PHD) and in the different years (2016 and 2031), in order to maintain the required HGL in the system.

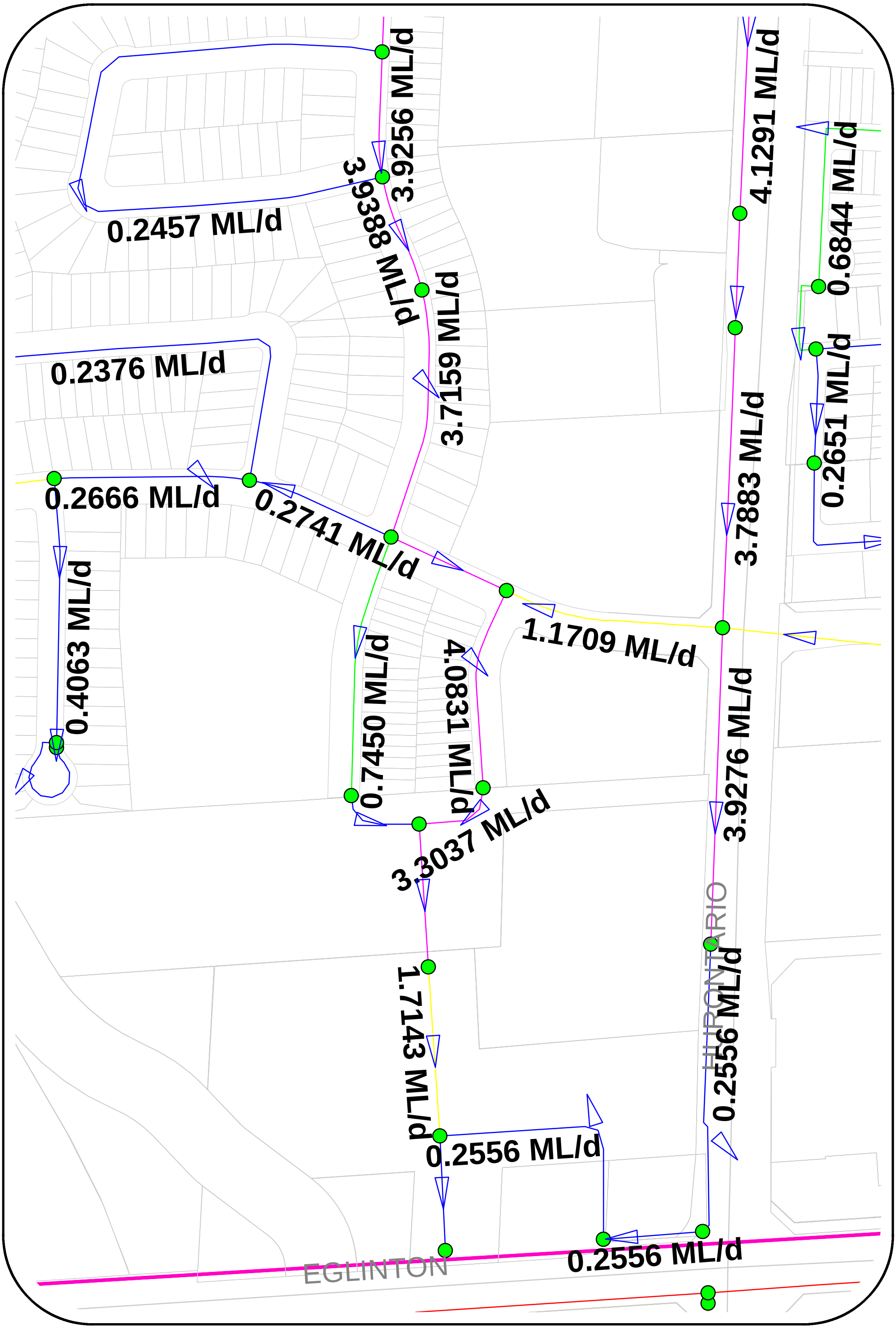
Pinnacle Uptown Mississauga Development
Appendix 5: Node Elevations



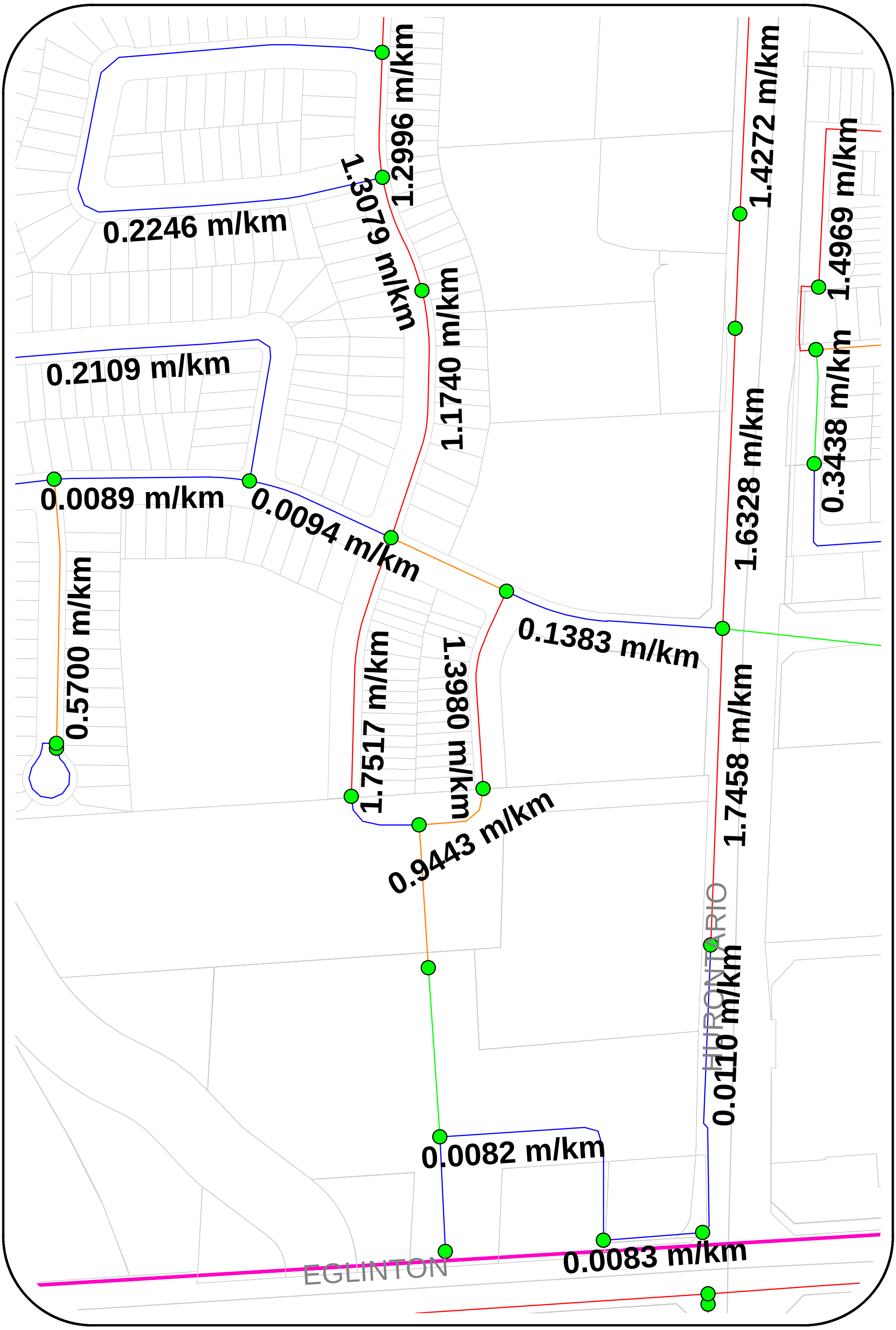
Pinnacle Uptown Mississauga Development
Appendix 6: 2031 Peak Hour Demand - Velocity



Pinnacle Uptown Mississauga Development
Appendix 7: 2031 Peak Hour Demand - Flow



Pinnacle Uptown Mississauga Development
Appendix 8: 2031 Peak Hour Demand - Headloss Gradient



Appendix 9: Detailed Fire Flow Results

Analyses With All Pipes Active								
2016 -MDD + Fire Flow								
ID	Static Demand (ML/d)	Static Pressure (psi)	Static Head (m)	Fire-Flow Demand (ML/d)	Residual Pressure (psi)	Available Flow at Hydrant (ML/d)	Available Flow Pressure (psi)	Available Flow at Hydrant (L/s)
N-100	0.29	91.14	231.91	21.60	75.93	60.52	20.05	700.47
N-101	0.88	88.95	231.87	21.60	72.16	55.62	20.05	643.77
N-102	1.31	91.63	231.86	21.60	73.64	54.49	20.05	630.72
N-105	2.44	86.09	231.86	21.60	68.70	54.40	20.05	629.58
2031 - MDD + Fire Flow								
ID	Static Demand (ML/d)	Static Pressure (psi)	Static Head (m)	Fire-Flow Demand (ML/d)	Residual Pressure (psi)	Available Flow at Hydrant (ML/d)	Available Flow Pressure (psi)	Available Flow at Hydrant (L/s)
N-100	0.29	107.91	243.71	21.60	93.14	69.96	20.05	809.74
N-101	0.88	105.73	243.68	21.60	89.38	64.33	20.05	744.54
N-102	1.31	108.41	243.66	21.60	90.85	62.57	20.05	724.16
N-105	2.44	102.87	243.66	21.60	85.93	62.98	20.05	728.93

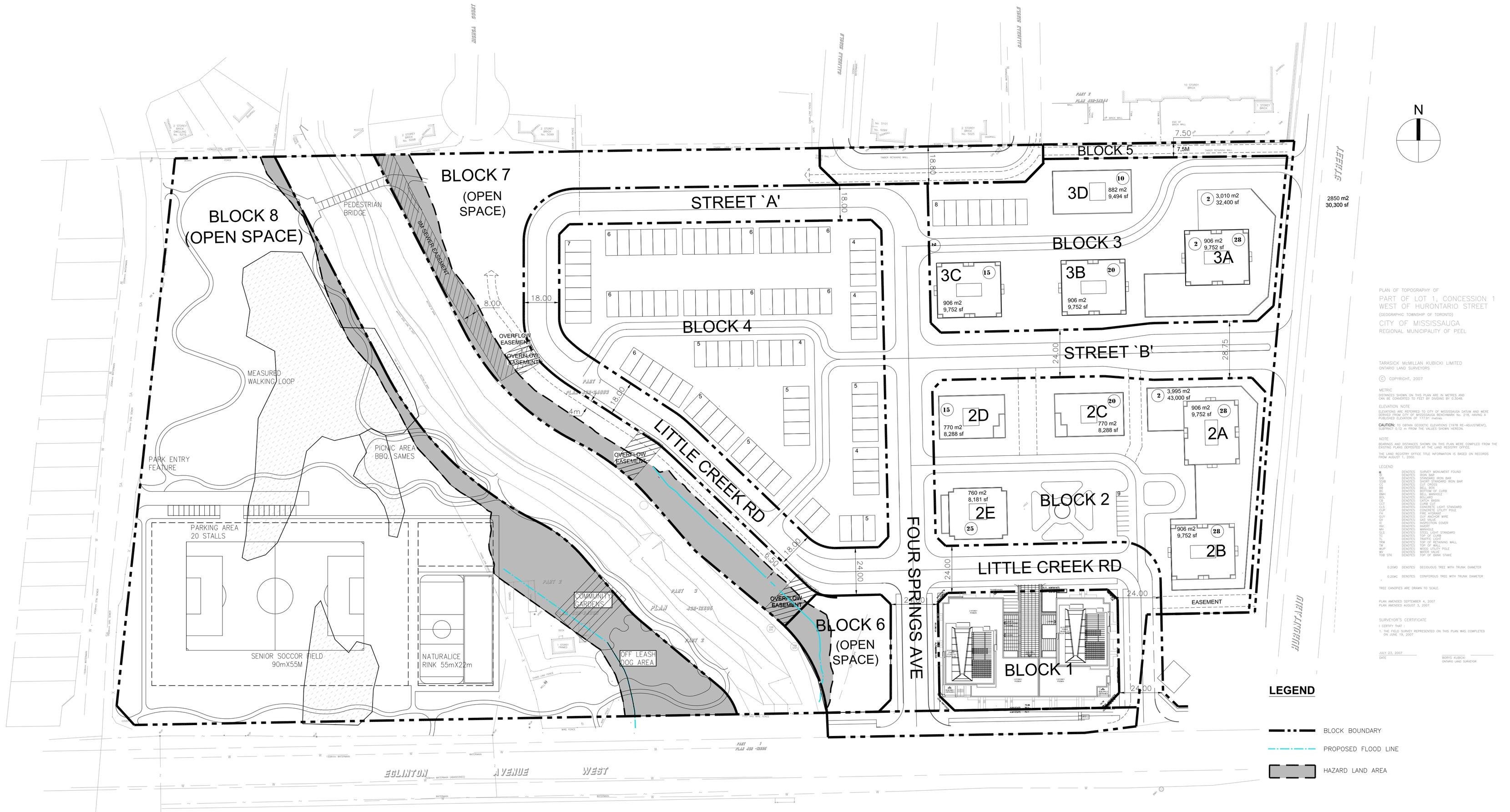
Note that a different number of pumps were turned on in the different demand conditions (ADD, MDD and PHD) and in the different years (2016 and 2031), in order to maintain the required HGL in the system.

REGION OF PEEL

REGION OF PEEL

Contact: Christina Iulianetti Tel.: (905) 791-7800 x4710

1	NOTE:	Regional staff has reviewed this application and are pleased to provide the following comments and conditions of approval. **Created :** 2009-06-22 15:05:26 **Last Modified :** -
2	NOTE:	An existing 300mm diameter water main is located on Hurontario Street and an existing 150mm diameter watermain is located on Salishan Circle. Connection to the existing 600mm diameter watermain located on Eglinton Avenue will not be permitted due to pressure zone boundary restrictions. **Created :** 2009-06-22 15:05:28 **Last Modified :** -
3	NOTE:	An existing 250mm diameter sanitary sewer is located on Salishan Circle and an existing 300mm diameter sanitary sewer is located in a 7.5m easement south of Eglinton Avenue West. **Created :** 2009-06-22 15:05:28 **Last Modified :** -
4	NOTE:	Region of Peel staff have reviewed the Functional Servicing Report dated October 2008, and subsequent report for Phase 1 of the development dated April 2009 and offer the following comments. **Created :** 2009-06-22 15:05:29 **Last Modified :** -
5	NOTE:	Phase One: The Region of Peel has no objection to the proposed water and wastewater servicing for Phase 1 of this development. Water Servicing for Phase 1 is possible through the existing 300mm diameter watermain on Hurontario Street and the proposed upgrade of the watermain on Salishan Circle to 300mm. The 300mm diameter sanitary sewer section can service the Phase 1 of the development with a population of 1065 person and 15.1L/s flows. **Created :** 2009-06-22 15:05:29 **Last Modified :** -
6	INFO REPORT	Phase Two: A revised Functional Servicing Report illustrating modelling results, showing flow rates and head losses in the watermains as well as a flow test on the existing 300mm watermain on Hurontario Street demonstrating it's ability to supply the required flows is required. Additionally, sanitary sewers within existing residential areas should not be at more than 80% full under peak flow conditions; this should also be reflected in the revised Functional Servicing Report. The revised report should consider servicing needs for the lands east of Hurontario Street. The report is to be based on Region of Peel design criteria and not the recommendations made in the Master Plan since these are only suggestions and have not been included in the design criteria to date. **Created :** 2009-06-22 15:05:29 **Last Modified :** -
7	SERVICING AND/OR DEV AGREEMENT	Easements will be required for Water and Sanitary services in order to service phase 1 of the development. **Created :** 2009-06-22 15:05:29 **Last Modified :** -
8	NOTE:	For clarifications regarding the comments above please contact Orest Jacyla at 905-791-7800 extension 7809. **Created :** 2009-06-22 15:05:30 **Last Modified :** -



PLAN OF TOPOGRAPHY OF
PART OF LOT 1, CONCESSION 1
WEST OF HURONTARIO STREET
(GEOGRAPHIC TOWNSHIP OF TORONTO)
CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

TARASICK McMILLAN KUBICKI LIMITED
ONTARIO LAND SURVEYORS

© COPYRIGHT, 2007

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

ELEVATION NOTE
ELEVATIONS ARE REFERRED TO CITY OF MISSISSAUGA DATUM AND WERE DERIVED FROM CITY OF MISSISSAUGA BENCHMARK No. 278, HAVING A PUBLISHED ELEVATION OF 177.91 metres.

CAUTION: TO OBTAIN GEODETIC ELEVATIONS (1978 RE-ADJUSTMENT), SUBTRACT 0.12 m FROM THE VALUES SHOWN HEREON.

NOTE
BEARINGS AND DISTANCES SHOWN ON THIS PLAN WERE COMPILED FROM THE EXISTING PLANS DEPOSITED AT THE LAND REGISTRY OFFICE.

THE LAND REGISTRY OFFICE TITLE INFORMATION IS BASED ON RECORDS FROM AUGUST 1, 2000.

LEGEND

■	DENOTES	SURVEY MONUMENT FOUND
IB	DENOTES	IRON BAR
SIB	DENOTES	SHORT IRON BAR
SSIB	DENOTES	SHORT STANDARD IRON BAR
CC	DENOTES	CUT CROSS
BB	DENOTES	BELL BOX
BC	DENOTES	BOTTOM OF CURB
BMH	DENOTES	BELL MANHOLE
BOL	DENOTES	BOLLARD
CB	DENOTES	CATCH BASIN
CBM	DENOTES	CANADA BOY
CLS	DENOTES	CONCRETE LIGHT STANDARD
CUP	DENOTES	CONCRETE UTILITY POLE
CV	DENOTES	CURB VALVE
GUY	DENOTES	GUY ANCHOR WIRE
GV	DENOTES	GAS VALVE
HC	DENOTES	HYDRO ACCESS COVER
INV.	DENOTES	INVERT
MH	DENOTES	MANHOLE
SLS	DENOTES	STEEL LIGHT STANDARD
TC	DENOTES	TOP OF CURB
TL	DENOTES	TRAFFIC LIGHT
TRW	DENOTES	TOP OF RETAINING WALL
TW	DENOTES	TOP OF WALL
UP	DENOTES	UTILITY POLE
WV	DENOTES	WATER VALVE
TOB STK	DENOTES	TOP OF BANK STAKE
	0.204D	DENOTES DECIDUOUS TREE WITH TRUNK DIAMETER
	0.204C	DENOTES CONIFEROUS TREE WITH TRUNK DIAMETER

TREE CANOPIES ARE DRAWN TO SCALE.

PLAN AMENDED SEPTEMBER 4, 2007
PLAN AMENDED AUGUST 3, 2007

SURVEYOR'S CERTIFICATE
I CERTIFY THAT :
1. THE FIELD SURVEY REPRESENTED ON THIS PLAN WAS COMPLETED ON JUNE 19, 2007

JULY 23, 2007
DATE

BORYS KUBICKI
ONTARIO LAND SURVEYOR

LEGEND

- BLOCK BOUNDARY
- PROPOSED FLOOD LINE
- HAZARD LAND AREA

No.	Date	Revisions	By	Signed
10				
9				
8				
7	2010-11-19	revisions to street A		
6	2010-11-17	revisions to block 3 & block 4		
5	2010-11-12	additional revisions		
4	2010-10-21	additional revisions resulting client's comments		
3	2010-10-21	revisions resulting client's comments		
2	2010-10-20	Additional revisions resulting from Oct.14th meeting with City		
1	2010-10-15	Revisions resulting from Oct.14th meeting with City		

Information

Seal:

Seal:

IBI GROUP
230 Richmond Street West
5th Floor
Toronto, Ontario
Canada M5V 1V6
Tel (416)596-1930
FAX(416)596-0644

Pinnacle International (Ontario) Limited

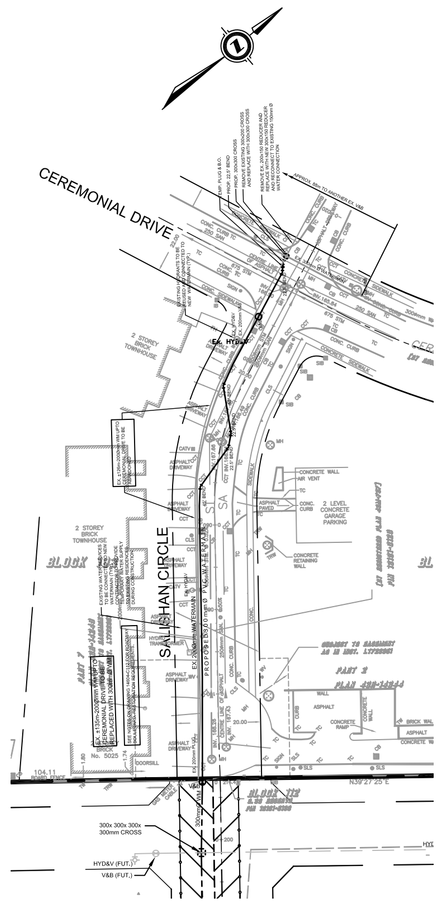
PINNACLE UPTOWN MISSISSAUGA

PRELIMINARY MASTER PLAN

Design:		Drawn:	GG	Checked:	PL
Scales:	HOR: 15 0 10 20 40 1:1000 (m)			Date:	SEPT 2010
				Drawing Number	SK-01
				Sheet Set No.	

TO-9368 (14604)

N:\14604\5.0 Drawings\Planning\Draft Plans\2010-10-15 Terms of Development\CONCEPT PLAN.dwg by gbarcelos at 2010-Nov-19 Attention: edition-day Sheet Set: A1 - 21 x 30 INCHES

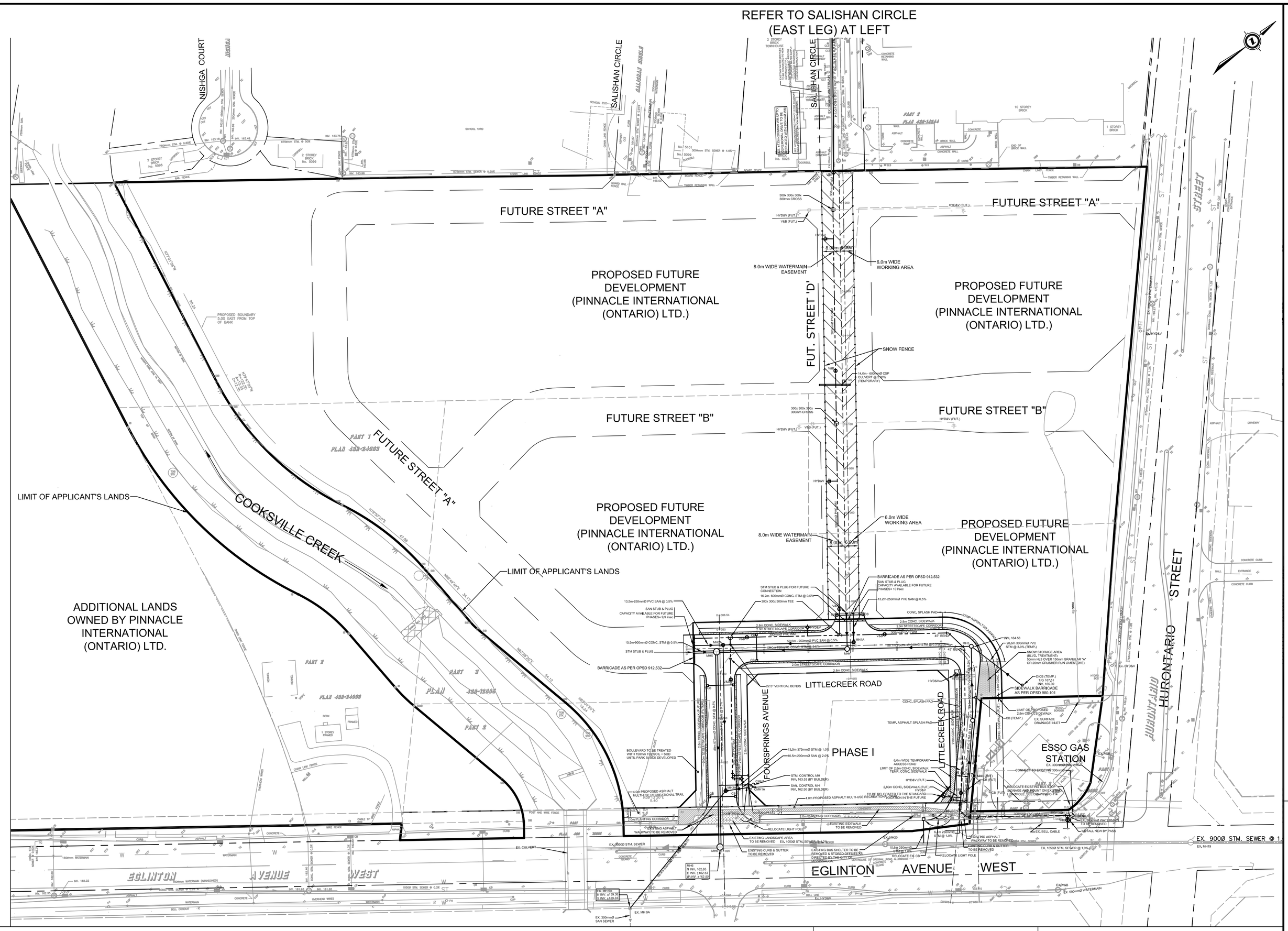


SALISHAN CIRCLE (EAST LEG)

SCALE 1:1000

NOTE:

1. ALL RESIDENTS ON SALISHAN CIRCLE TO BE NOTIFIED IN ADVANCE OF THE NEW WATERMAIN INSTALLATION.
2. THE CONSTRUCTION IMPACT TO BE KEPT TO MINIMUM.



NOTE:

PHASE 2 LANDS HAVE NOT BEEN DRAFT APPROVED. ROAD LAYOUT MAY BE SUBJECT TO CHANGE.

QUANTITIES OF PROPOSED WATERMAIN & SANITARY

NO	ITEM	QUANTITY
1	FIRE HYDRANT	6
2	WATER SERVICE	1
3	300mmØ VALVE & BOX	9
4	300mmØ WATERMAIN	656.4m
5	SANITARY MANHOLE 1200mmØ	3
6	200mmØ SANITARY SEWER	10.5m
7	250mmØ SANITARY SEWER	85.7m
8	300mmØ SANITARY SEWER	135.6m

LEGEND:

- PROPERTY BOUNDARY
- PHASE I BOUNDARY
- EX. MH1 — EXISTING STORM SEWER
- MH1 — PROPOSED STORM SEWER SERVICING
- CONTROL MH (BY BLOCK DEVELOPER)
- EASEMENT
- MH5A — PROPOSED SANITARY SEWER & MH.
- EX. MH — EX. SANITARY SEWER & MH.
- CMH7A — PROPOSED SANITARY SEWER BLOCK CONNECTION & CONTROL MANHOLE
- VAB — PROPOSED WATER MAIN
- EX. WATERMAIN
- HYDAV — DENOTES HYDRANT
- DEAD END BARRICADE (OPSD 912.532)
- 0.75m CONC. SPLASH PAD

FINAL SUBMISSION

2010-09-20

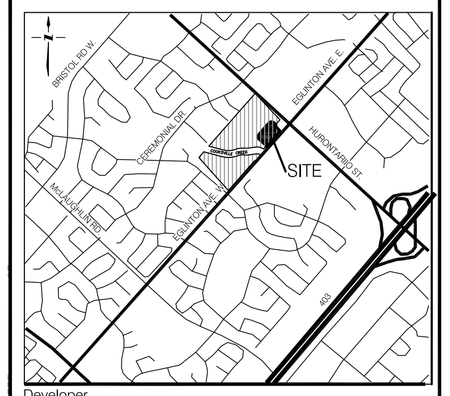
15 0 10 20 40

1:1,000(m)

This drawing, as an instrument of service, is the property of the Architect/Engineer and may not be reproduced without their permission and unless the reproduction carries their name. All design and other information shown on this drawing are for the use on the specified project only and shall not be used otherwise without written permission of the Architect/Engineer.

Written dimensions shall have precedence over scaled dimensions. Contractors shall verify and be responsible for all dimensions and conditions on the job and the Architect/Engineer shall be informed of any variations from the dimensions and conditions shown on the drawing. Shop drawings shall be submitted to the Architect/Engineer for approval before proceeding with fabrication.

Key Plan



Developer

PINNACLE INTERNATIONAL (ONTARIO) LTD.

BENCH MARK No. 218

ELEVATIONS ARE REFERRED TO CITY OF MISSISSAUGA DATUM AND WERE DERIVED FROM CITY OF MISSISSAUGA BENCHMARK No. 218, HAVING A PUBLISHED ELEVATION OF 177.91 metres.

LOCATION: TOWNSHIP - LIMEHOUSE
BELL TELEPHONE EXCHANGE STATION NO. 5253 ON NORTHEAST SIDE OF HURONTARIO STREET (HIGHWAY NO. 10), 0.8 KM NORTHWEST OF BASE LINE ROAD (EGLINTON AVE.), PLATE ON SOUTHWEST FACE, 7.6 M FROM SOUTH CORNER. ESTABLISHED BY TOWNSHIP OF TORONTO.

Revision / Submission

No.	Date	By	Comment

FIRST	SECOND	INTERIM	PRE-SER	FINAL
DATE SEP. 17/2009	DATE JUN. 18/2010	DATE AUG. 11/2010	DATE	DATE

SUBMISSION	DWG No

DESIGN BY — CHD — APPROVED BY

IBI GROUP
9133 Leslie Street
Suite 200
Richmond Hill, Ontario
Canada L4B 4N1
Tel (905)763-2322
FAX(905)763-9983

CITY OF MISSISSAUGA

PLANNING, DESIGN & DEVELOPMENT

DIRECTOR OF ENGINEERING & DEVELOPMENT SERVICES

Project Title

PINNACLE UPTOWN MISSISSAUGA
PHASE 1
MISSISSAUGA, ONTARIO

REGION FILE NO.: T-07006M

Sheet Title

GENERAL BELOW GROUND SERVICES PLAN

FILE : 02 07/025
REF. PLAN No.

CAD File
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Scale	Date
1:1000	AUGUST 2009
Drawn	Checked
UP/WS	GM/DB
Project No.	Drawing No.
TO-14604	C-100B

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Plot Style: AIA Standard-Full.ctb Plot Scale: 1:1 Printed By: Geza Marton Last Saved By: acovello Saved Date: 2010/11/19

Pinnacle Uptown Mississauga Concept Plan , Development Yields

Janurary 4,2011

EAST PARCEL	Block	Area (ha.)	BLDG No.	Residential			Commercial			Total	Floor Space Index	Units			U/G Parking @1.2/unit Residential	U/G Parking @3/100m2 Commercial	Parking Levels @32.5m2/stall
				Footprint (m2)	Levels	SQM	Footprint (m2)	Levels	Retail SQM			Condo@ 85m2	Town	BLK Total			
Phase I	**Block 1** - Mixed Use	0.54	**1A**	730	28	21,719	0	0	0	**40,088**	**7.49**	230	10	**445**	**532**		**4**
			1B	730	24	18,369						194	11				
	Block Subtotal	**0.54**				40,088						**424**	**21**				
Phase II	**Block 2** - Mixed Use	1.54	**2A**	906	26	23,556	3,995	2	7,990	101,424	6.59	277		**1,089**	1,306	266	**3.3**
			2B	906	26	23,556						277					
			2C	770	19.5	15,015	870	0.5	435			177					
			2D	770	14.5	11,165	915	0.5	457			131					
			2E	770	25	19,250						226					
	Block 3- Mixed Use	1.53	**3A**	906	26	23,556	2,590	2	5,180	73,199	4.78	277		**780**	926	185	**2.4**
			3B	906	19.5	17,667	980	0.5	490			208					
			3C	906	14.5	13,137	1,018	1	509			155					
			3D	1122	10	11,220						132					
			3E	576	2.5	1,440						0	8				
	Block 4 - Residential	1.96		7,410	2.5	18,525		0	0	18,525	0.95		95	**95**			
	Sub-Total	**5.03**				**178,087**			**15,061**	**193,148**	**3.84**	**1,860**	**103**	**1,963**			
	Block 5 Open Space	**0.85**															
	Block 6-Open Space	**0.19**															
	Block 6-Hazard land	**0.07**															
	Block 7-Hazard land	**0.42**															
	Public Roads	**2.5**															
	Road Widenings	**0.26**															
East Parcel Total		**9.86**		**218,175**			**15,061**			**233,236**	**2.37**	**2,284**	**124**	**2,408**			

WEST PARCEL	**Block 8 -Hazard Land**	**0.58**	
	Block 9 -Open Space	**4.14**	
	Road Widening	**0.2**	
West Parcel Total		**4.92**	

TOTAL UPTOWN MISSISSAUGA		**14.78**			**218,175**		**15,061**	**233,236**	**1.58**	**2,408**		

Open Space Breakdown	**Parkland Area**	**Easement Area** (outside Hazard Area)	**Hazard Area**	**Total**
Block 5	0.81	0.04		**0.85**
Block 6	0.19	0.00	0.07	**0.26**
Block7			0.42	**0.42**
East Parcel Sub-Total	**1.00**	**0.04**	**0.49**	**1.53**
West Parcel Block 8			**0.58**	**0.58**
West Parcel Block 9	4.14			**4.14**
West Parcel Sub-Total	4.14		0.58	**4.72**
TOTAL	**5.14**	**0.04**	**1.07**	**6.25**

APPENDIX H

DETAILED DEMAND CALCULATIONS

Connection Demand Table - Phase IV - Part II

WATER CONNECTION

Connection point [3]			
Watergarden Drive			
Pressure zone of connection point			
Total equivalent population to be serviced [1]			*5*
Total lands to be serviced			*1.04 ha*
Hydrant flow test			
Hydrant flow test location	*35 Watergraden Drive*		
(Northeast of Watergraden and Foursprings Ave.)			
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	*165*	*48.2*	*10:00 AM*
Maximum water pressure	*607*	*0*	*10:00 AM*

No.	Water demands			
	Demand type	Demand (in l/s)		
		Use 1 [5]		Total
1	Average day flow	*6.20*		*6.20*
2	Maximum day flow	*8.53*		*8.53*
3	Peak hour flow	*12.79*		*12.79*
4	Fire flow [2]			*83.33*
Analysis				
5	Maximum day plus fire flow			*91.86*

WASTEWATER CONNECTION

				Total
Connection point [4]				
Total equivalent population to be serviced [1]				*5*
Total lands to be serviced				*1.04 ha*
6	Wastewater sewer effluent (in l/s)			*0.07*

1) Please refer to design criteria for population equivelncies
2) Please reference the Fire Underwriters Survey Document
3) Please Specify the connectionpoint ID
4) Please specifiy the connection point (wastewater line or manhole ID). Also, the "total equivalent population to be serviced" and the "total lands to be serviced" should reference the connection point.
5) Please complete as many uses are nescessary for the development.

Please include the graphs associated withteh hydrant flow test information table
Please provide Professionsl Engineers signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

Connection Demand Table - Phase V

WATER CONNECTION

Connection point [3]			
Watergarden Drive			
Pressure zone of connection point			
Total equivalent population to be serviced [1]			*2,966*
Total lands to be serviced			*1.03 ha*
Hydrant flow test			
Hydrant flow test location	*35 Watergraden Drive*		
(Northeast of Watergraden and Foursprings Ave.)			
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	*165*	*48.2*	*10:00 AM*
Maximum water pressure	*607*	*0*	*10:00 AM*

No.	Water demands			
		Demand (in l/s)		
	Demand type	Use 1 [5]	Use 2 [5]	Total
1	Average day flow	-	*6.20*	*6.20*
2	Maximum day flow	-	*8.53*	*8.53*
3	Peak hour flow	-	*12.79*	*12.79*
4	Fire flow [2]			*83.33*
Analysis				
5	Maximum day plus fire flow			*91.856435*

WASTEWATER CONNECTION

				Total
Connection point [4]		Population	Area (ha)	
Watergraden Drive		1,700.80	0.49	
Little Creek Road		904.50	0.54	
Total equivalent population to be serviced [1]				*2,605*
Total lands to be serviced				*1.03 ha*
6	Wastewater sewer effluent (in l/s)			*31.90*

1) Please refer to design criteria for population equivelncies
2) Please reference the Fire Underwriters Survey Document
3) Please Specify the connectionpoint ID
4) Please specifiy the connection point (wastewater line or manhole ID). Also, the "total equivalent population to be serviced" and the "total lands to be serviced" should reference the connection point.
5) Please complete as many uses are nescessary for the development.

Please include the graphs associated withteh hydrant flow test information table
Please provide Professionsl Engineers signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

Detailed Water Demand Calculations

February 3, 2020

East Parcel	Block	Area (ha)	BLDG No.	Bldg Description	Commercial			# of Units			Population		Residential ADD (L/s)	
					Retail/Office ha	Comm. ADD	Condo Units (1 Bedroom)	Condo Units (2 + Bedrooms)	Town	BLK Total	Residential	ICI	Short Term	Long Term
Phase IV - Part II	Block 16	0.72	3	East building	0.32	0	304	310	0		1298	16	6.1	4.2
Sub - Total Phase IV Part II					**0.32**	**0**		**310**	**0**	**310**	**1298**	**16**	**6.1**	**4.2**
Phase V	Block 1	1.03	1	North building	-	0	157	166	0	614	685	0	3.2	2.2
	Block 17		2	North building	0.32	0	147	144	0		613	16	2.9	2.0
			3	South building	0.52	0	154	181	0	335	718	26	3.4	2.3
Sub - Total Phase V		1.03			**0.85**	**0**		**491.00**	0	**949**	**2017**	**42**	**9.5**	**6.5**

Longterm Residential Water Demand =	280	L/cap*d
Short Term Residential Water Demand =	409	L/cap*d
Fire Flow Demand (4 storey residential)=	15000	L/min
ICI Water Demand	300	L/Employee*d
Semi- Detached Home: Population Density =	3.8	population/unit
Apartments: Population Density ((2+ bedrooms)=	1.68	population/unit
Apartments: Population Density (1 Bedroom)=	2.54	population/unit
Commercial Density =	50	employees/ha
Residential Maximum Day Demand Factor =	2	x Ave. Day Demand
Residential Maximum Day Short Term Demand Factor =	2	x Ave. Day Demand
Residential Peak Hour Demand Factor =	3	x Ave. Day Demand
ICI Max. Day Demand Factor =	1.4	x Ave. Day Demand
ICI Max. Day Short Term Demand Factor =	2.0	x Ave. Day Demand
ICI Peak Hour Demand Factor =	3	x Ave. Day Demand

	Typical Water Demand			Short Term Water Demand		
	Avg. Day (L/s)	Max Day (L/S)	Peak Hour (L/S)	Avg. Day	Max Day	Peak Hour
Phase IV Part II	4.26	8.49	12.79	6.20	8.53	12.79
Phase V	6.68	13.28	20.05	9.69	13.37	20.05
Total Water Demand	**10.95**	**21.77**	**32.84**	**15.89**	**21.89**	**32.84**

IBI GROUP
7th Floor – 55 St. Clair Avenue West
Toronto ON M4V 2Y7 Canada
tel 416 596 1930 fax 416 596 0644
ibigroup.com

Preliminary Fire Flow Estimate

Based on Fire Underwriters Survey Method, Insurance Advisory Organization

Item **Pinnacle - Phase IV Part II**

1 **Fire Flow given by:** **F = 220 x C x A^ 0.5**

INPUT

Where:
F = the required fire flow in Litres per minute (L/min)
C = Coefficient related to the type of construction — C= 0.6
1) 1.5 For wood fram construction
2) 1.0 for ordinary construction
3) 0.8 for noncombustible constuction
4) 0.6 for fire-resistive construction (two largest successive floors, plus 50% of each of any floors immediately above them up to 8, if vertical openings are inadequately protected
5) 0.6 for fire-resistive construction (largest floor, plus 25% each of 2 adjoining floors, if vertical openings are adequately protected (1-hour rating).

A = the total floor area in the building being considered (in m2)

Mixed USE INPUT
Floor Area 2,604.40 m2

Total building floor area (m2) 2,604.40 m2 A= 2,604 m2

F = 6,736 Litres/minute

Round to nearest 1000 Litres/min **F = 7,000 Litres/minute**

	Increases or Reductions	INPUT	
2	Low/high fire hazard occupancy (open garage credit 25%)		-
3a	Automatic sprinkler protection credit (conforming to NFPA standards)	-15%	(1,050)
3b	If sprinkler system, an additional 10% credit if water supply is standard for both system and fire department hose lines.	-30%	(2,100)
3c	If sprinkler system, an additional 10% credit if fully supervised (water flow and control valve alarm system).	0%	-
4	Exposure (% sum of all sides of building)	20%	1,400
			-
	Note: Fire flow shall not exceed 45,000L/min nor be less than 2,000L/min.	**F =**	**5,250 Litres/minute**
	Round to nearest 1000 Litres/min	**FINAL F =**	**5000 Litres/minute**

Conversion 1 US Gallon = 3.785 L **conversion** **1,321 USGPM**
Conversion 1 Imperial Gallon = 4.546 L **conversion** **1,100 IGPM**

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ibigroup.com

Preliminary Fire Flow Estimate

Based on Fire Underwriters Survey Method, Insurance Advisory Organization

Item **Pinnacle - Phase V**

1 **Fire Flow given by:** **F = 220 x C x A^ 0.5**

INPUT

Where:
F = the required fire flow in Litres per minute (L/min)
C = Coefficient related to the type of construction — C= 0.6
1) 1.5 For wood fram construction
2) 1.0 for ordinary construction
3) 0.8 for noncombustible constuction
4) 0.6 for fire-resistive construction (two largest successive floors, plus 50% of each of any floors immediately above them up to 8, if vertical openings are inadequately protected
5) 0.6 for fire-resistive construction (largest floor, plus 25% each of 2 adjoining floors, if vertical openings are adequately protected (1-hour rating).

A = the total floor area in the building being considered (in m2)

Mixed USE INPUT
Floor Area 2,835.96 m2

Total building floor area (m2) 2836.0 m2 A= 2,836 m2

F = 7,029 Litres/minute

Round to nearest 1000 Litres/min **F = 7,000 Litres/minute**

	Increases or Reductions	INPUT	
2	Low/high fire hazard occupancy (open garage credit 25%)		-
3a	Automatic sprinkler protection credit (conforming to NFPA standards)	-15%	(1,050)
3b	If sprinkler system, an additional 10% credit if water supply is standard for both system and fire department hose lines.	-30%	(2,100)
3c	If sprinkler system, an additional 10% credit if fully supervised (water flow and control valve alarm system).		-
4	Exposure (% sum of all sides of building)	20%	1,400
			-
	Note: Fire flow shall not exceed 45,000L/min nor be less than 2,000L/min.	**F =**	**5,250 Litres/minute**
	Round to nearest 1000 Litres/min	**FINAL F =**	**5000 Litres/minute**

Conversion 1 US Gallon = 3.785 L **conversion** **1,321 USGPM**
Conversion 1 Imperial Gallon = 4.546 L **conversion** **1,100 IGPM**

PHASE IV - PART II GFA BREAKDOWN

BUILDING 3

Level	GFA (SF)	GFA (SM)
1	12,113	1125.40
2	21,499	1,997.30
3	24,175	2,246.00
4	16,113	1497.00
5 to 6	39,847	3,701.90
7	6,807	632.40
8 to 32	192,428	17,877.00
33	7,734	718.50
34	7,734	718.50
35	7,734	718.50
TOTAL GFA	**336,184.00**	**31,232.500**

Maximum Floor GFA =	24,175.0	2,246.0
25% of each of the adjoing floors =	3,857.8	358.4
Total Floor Area =	28,032.8	2,604.4

PHASE V - GFA BREAKDOWN

BUILDING 1

Level	GFA (SF)	GFA (SM)
1	3,929	365.0
2	7,267	675.1
3	8,063	749.1
4	6,863	637.6
5	6,863	637.6
6	6,244	580.1
7 to 31	186,990	17,371.9
32	7,480	694.9
33	5,995	557.0
34	5,995	557.0
35	5,995	557.0
TOTAL GFA	**251,684.00**	**23,382.2**

Maximum Floor GFA =	8,063.0	749.1
25% of each of the adjoing floors =	3,532.5	328.2
Total Floor Area =	**11,595.5**	**1,077.3**

PHASE V - GFA BREAKDOWN

BUILDING 2

Level	GFA (SF)	GFA (SM)
1	16,310	1,515.2
2	15,347	1,425.8
3	20,827	1,934.9
4	11,099	1,031.1
5	15,729	1,461.3
6	8,466	786.5
7	9,855	915.6
8	9,855	915.6
9	5,190	482.2
10 to 22	96,692	8,983.0
23	7,438	691.0
24	6,718	624.1
25 to 29	33,592	3,120.8
30 to 31	10,952	1,017.5
32	5,476	508.7
TOTAL GFA	**273,546.00**	**25,413.3**

Maximum Floor GFA =	20,827.0	1,934.9
25% of each of the adjoing floors =	6,611.5	614.2
Total Floor Area =	**27,438.5**	**2,549.1**

PHASE V - GFA BREAKDOWN

BUILDING 3

Level	GFA (SF)	GFA (SM)
1	19,603	1,821.2
2	20,703	1,923.4
3	19,689	1,829.2
4	2,528	234.9
5	13,880	1,289.5
6	13,880	1,289.5
7	13,880	1,289.5
8	13,880	1,289.5
9	4,913	456.4
10 to 30	160,900	14,948.1
31	7,662	711.8
32	6,746	626.7
33 to 35	20,238	1,880.2
36	6,319	587.1
37	6,319	587.1
38	6,319	587.1
TOTAL GFA	**337,459.00**	**31,351.0**

Maximum Floor GFA =	20,703.0	1,923.4
25% of each of the adjoing floors =	9,823.0	912.6
Total Floor Area =	**30,526.0**	**2,836.0**

APPENDIX I

APPROVED SUBDIVISION SANITARY SEWER DESIGN SHEET AND DRAINAGE AREA

DEVELOPMENT 14604 PINNACLE INTERNATIONAL (ONTARIO) LTD. (PHASE 2)

CONSULTANT IBI GROUP

MAJOR DRAINAGE AREA

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN CHART

Last Updated : January 22, 2013

Designed By: AC

Checked By: ND

LOCATION			DRAINAGE AREA			RUNOFF						PIPE SELECTION					
STREET	FROM MH. No.	TO MH. No.	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population	(1) Sewage Flow m3/sec	(2) Infiltration Flow m3/sec	(3) Foundation Drains m3/sec	Total Flow (1)+(2)+(3) m3/s	Pipe Length L m	Cumm. Pipe Length m	Pipe Dia> mm	Pipe Gradient % m/m	Actual Capacity (full) m3/s	Velocity (full) V m/s
TO EX. MH T4																	
AREA 1A	STUB	MH6A															
TOWER				2.7	277.0		748										
OFFICE/COMMERCIAL				50.0	0.80		40										
INFILTRATION			0.47			0.47	788	0.0107	0.0001		0.011	16.6		250	1.00%	0.059	1.211
AREA 2A	STUB	MH6A															
TOWER				2.7	277.0		748										
OFFICE/COMMERCIAL				50.0	0.52		26										
INFILTRATION			0.57			0.57	774	0.0105	0.0001		0.011	13.5		250	1.00%	0.059	1.211
EXTERNAL AREA	STUB	MH6A															
Plaza East HWY #10				50.0	2.24	2.24	112	0.0017	0.0004		0.002	18.4		250	1.50%	0.073	1.484
UPTOWN DRIVE	MH6A	MH7A				3.28	1674	0.0214	0.0007		0.022	80.0		250	1.00%	0.059	1.211
AREA 2B	STUB	MH7A															
TOWER				2.7	208.0		562										
OFFICE/COMMERCIAL				50.0	0.05		2										
INFILTRATION			0.32			0.32	564	0.0078	0.0001		0.008	11.8		250	1.00%	0.059	1.211
AREA 1C	STUB	MH7A															
TOWER				2.7	177.0		478										
OFFICE/COMMERCIAL				50.0	0.04		2										
INFILTRATION			0.28			0.28	480	0.0067	0.0001		0.007	14.1		250	1.00%	0.059	1.211
UPTOWN DRIVE	MH7A	MH8A				3.88	2718	0.0331	0.0008		0.034	72.0		250	1.00%	0.059	1.211
AREA 2C	STUB	MH8A															
TOWER				2.7	155.0		419										
OFFICE/COMMERCIAL				50.0	0.05		3										
INFILTRATION			0.36			0.36	421	0.0059	0.0001		0.006	11.2		250	1.00%	0.059	1.211
AREA 1D	STUB	MH8A															
TOWER				2.7	131.0		354										
OFFICE/COMMERCIAL				50.0	0.05		2										
INFILTRATION			0.35			0.35	356	0.0050	0.0001		0.005	14.2		250	1.00%	0.059	1.211
UPTOWN DRIVE	MH8A	MH9A				4.59	3495	0.0415	0.0009		0.042	18.1		250	2.38%	0.092	1.869
AREA 1E	STUB	MH10A															
HIGH RISE				2.7	226.0		610										
INFILTRATION			0.54			0.54	610	0.0084	0.0001		0.009	11.2		250	1.00%	0.059	1.211
FOURSPRINGS AVE.	MH10A	MH9A				0.54	610	0.0084	0.0001		0.009	61.3		250	0.50%	0.042	0.857
	MH9A	MH11A				5.13	4105	0.0478	0.0010		0.049	56.3		375	0.40%	0.111	1.004
AREA 2D	STUB	MH11A															
TOWER				2.7	132.0		356										

DEVELOPMENT 14604 PINNACLE INTERNATIONAL (ONTARIO) LTD. (PHASE 2)

CONSULTANT IBI GROUP

MAJOR DRAINAGE AREA

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN CHART

Last Updated : January 22, 2013

Designed By: AC

Checked By: ND

LOCATION			DRAINAGE AREA			RUNOFF						PIPE SELECTION					
STREET	FROM MH. No.	TO MH. No.	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population	(1) Sewage Flow m3/sec	(2) Infiltration Flow m3/sec	(3) Foundation Drains m3/sec	Total Flow (1)+(2)+(3) m3/s	Pipe Length L m	Cumm. Pipe Length m	Pipe Dia> mm	Pipe Gradient % m/m	Actual Capacity (full) m3/s	Velocity (full) V m/s
TH				3.8	8.0		30										
INFILTRATION			0.61			0.61	387	0.0055	0.0001		0.006	11.2		250	1.00%	0.059	1.211
LITTLE CREEK RD																	
AREA 7A	MH11A	MH12A	0.17			5.91	4492	0.0518	0.0012		0.053	29.2		375	0.40%	0.111	1.004
AREA 5A	MH12A	MH13A	0.23			6.14	4492	0.0518	0.0012		0.053	85.9		375	0.40%	0.111	1.004
AREA 5B	MH13A	MH14A	0.20			6.34	4492	0.0518	0.0013		0.053	77.4		375	0.59%	0.135	1.219
EASEMENT																	
AREA 5C	MH14A	MH15A	0.08			6.42	4492	0.0518	0.0013		0.053	28.3		375	1.50%	0.215	1.944
AREA 5D	MH15A	MH16A	0.23			6.65	4492	0.0518	0.0013		0.053	69.2		375	2.00%	0.248	2.245
CREEK CROSSING																	
AREA 5E	MH16A	EX. MHT4	0.05			6.70	4492	0.0518	0.0013		0.053	48.9		375	0.30%	0.096	0.870
TO EX. MH 9A																	
AREA 1B																	
TOWER				2.7	277.0		748										
INFILTRATION	STUB	MH4A	0.41			0.41	748	0.0102	0.0001		0.010	13.6		200	4.00%	0.066	2.088
AREA 1B (PHASE I)	STUB	MH4A															
HIGH RISE				2.7	201.0		543										
TH				3.8	11.0		42										
INFILTRATION			0.37			0.37	585	0.0081	0.0001		0.008	14.5		200	4.00%	0.066	2.088
LITTLE CREEK RD	MH4A	MH1A				0.78	1332	0.0174	0.0002		0.018	85.4		250	1.00%	0.059	1.211
AREA 1A (PHASE I)	STUB	MH1A															
HIGH RISE				2.7	237.0		640										
TH				3.8	12.0		46										
INFILTRATION			0.50			0.50	686	0.0094	0.0001		0.009	15.5		200	4.00%	0.066	2.088
LITTLE CREEK RD	MH1A	MH2A				1.28	2018	0.0253	0.0003		0.026	11.1		250	1.00%	0.059	1.211
BLOCK 9																	
TH				3.8	100.0		380										
INFILTRATION	STUB	MH5A	2.68			2.68	380	0.0054	0.0005		0.006	11.2		250	1.00%	0.059	1.211
AREA 6	MH5A	MH2A	0.40			3.08	380	0.0054	0.0006		0.006	34.6		250	0.50%	0.042	0.857
Foursprings Avenue	MH2A	MH3A				4.36	2398	0.0296	0.0009		0.030	65.9		300	0.50%	0.068	0.967
Foursprings Avenue	MH3A	EX.MH9A				**4.36**	**2398**	0.0296	0.0009		**0.030**	68.4		**300**	0.50%	0.068	0.967

For Population < 1000 people Min. designed sewage flow = 0.013 m^3/sec as per Std. 2-5-2

Population for Tower/ Midrise building is considered as 2.7 persons /unit

DEVELOPMENT 14604 PINNACLE INTERNATIONAL (ONTARIO) LTD. (PHASE 2)

CONSULTANT IBI GROUP

MAJOR DRAINAGE AREA

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN CHART

Last Updated : January 22, 2013

Designed By: AC

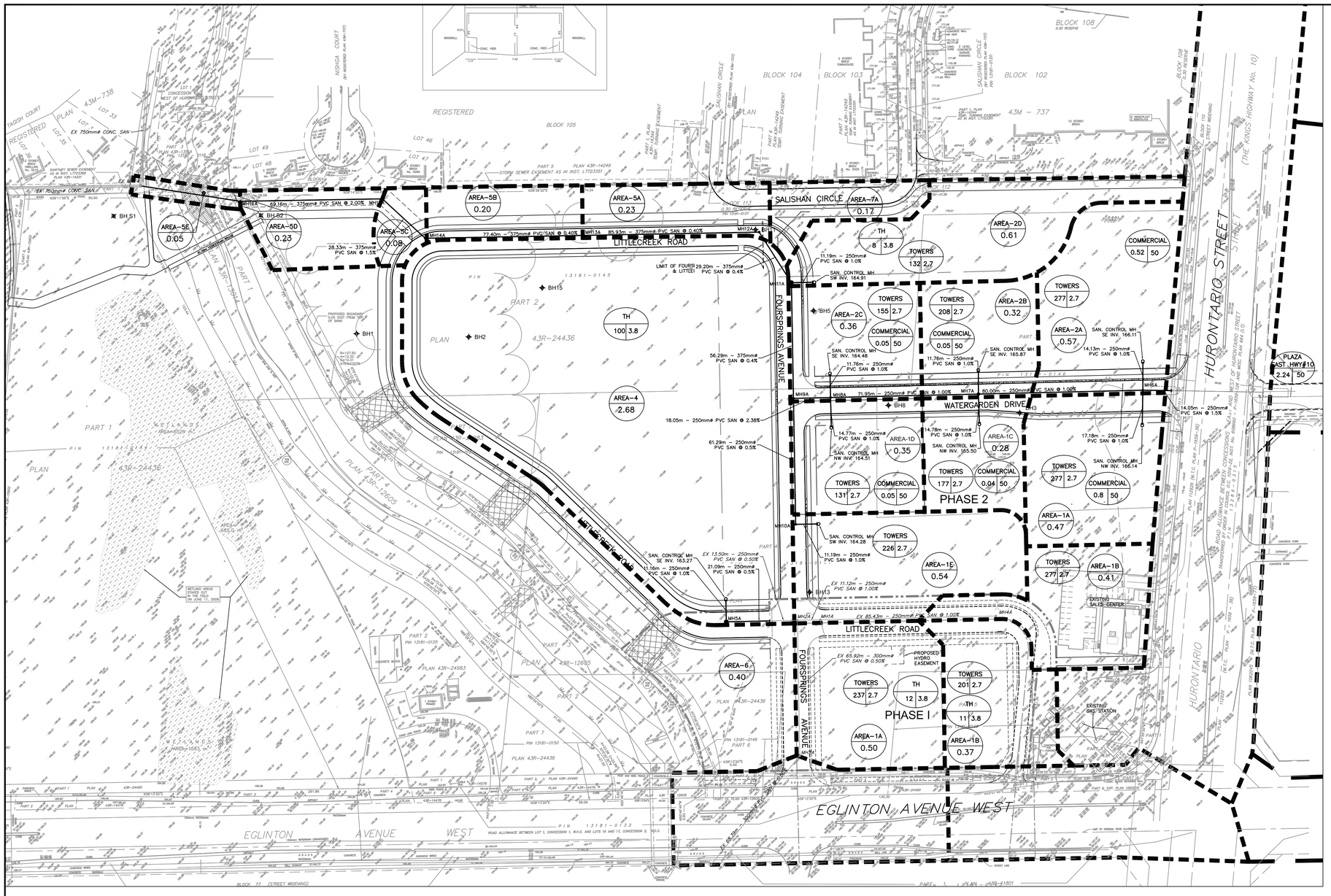
Checked By: ND

LOCATION			DRAINAGE AREA			RUNOFF						PIPE SELECTION					
STREET	FROM MH. No.	TO MH. No.	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population	(1) Sewage Flow m3/sec	(2) Infiltration Flow m3/sec	(3) Foundation Drains m3/sec	Total Flow (1)+(2)+(3) m3/s	Pipe Length L m	Cumm. Pipe Length m	Pipe Dia> mm	Pipe Gradient % m/m	Actual Capacity (full) m3/s	Velocity (full) V m/s

For infiltration flow 0.0002 m^3/sec/ha was used as Min. designed flow

Population for Town Houses is considered as 3.8 persons /unit

Population for Commercial is considered as 50 persons/ Hectare of land


AREA–5B 0.20
AREA–5A 0.23
AREA–7A 0.17
AREA–5E 0.05
AREA–5D 0.23
AREA–5C 0.09
AREA–2D 0.61
AREA–2C 0.36
AREA–2B 0.32
AREA–2A 0.57
AREA–4 2.68
AREA–1D 0.35
AREA–1C 0.28
AREA–1A 0.47
AREA–1E 0.54
AREA–1B 0.41
AREA–6 0.40
AREA–1A 0.50
AREA–1B 0.37
TH 8 3.8
TOWERS 132 2.7
COMMERCIAL 0.52 50
TOWERS 277 2.7
TOWERS 155 2.7
COMMERCIAL 0.05 50
TOWERS 208 2.7
COMMERCIAL 0.05 50
TH 100 3.8
TOWERS 131 2.7
COMMERCIAL 0.05 50
TOWERS 177 2.7
COMMERCIAL 0.04 50
TOWERS 277 2.7
COMMERCIAL 0.8 50
TOWERS 226 2.7
TOWERS 277 2.7
TOWERS 237 2.7
TH 12 3.8
TOWERS 201 2.7
TH 11 3.8
PLAZA EAST HWY#10 2.24 50
LITTLECREEK ROAD
SALISHAN CIRCLE
FOURSPRINGS AVENUE
WATERGARDEN DRIVE
HURONTARIO STREET
EGLINTON AVENUE WEST
PHASE 2
PHASE I
EXISTING SALES CENTER
EXISTING GAS STATION
WETLAND AREA=5529 m²
WETLAND AREA=1663 m²
BLOCK 108 0.30 RESERVE
BLOCK 104
BLOCK 103
BLOCK 102
BLOCK 105
REGISTERED PLAN 43M–738
43M – 737
43R–24436
43R–12605
43M–1265
43R–11801
69.16m – 375mmø PVC SAN @ 2.00%
77.40m – 375mmø PVC SAN @ 0.40%
49.93m – 375mmø PVC SAN @ 0.40%
28.33m – 375mmø PVC SAN @ 1.5%
29.20m – 375mmø PVC SAN @ 0.4%
56.29m – 375mmø PVC SAN @ 0.4%
18.05m – 250mmø PVC SAN @ 2.38%
61.29m – 250mmø PVC SAN @ 0.5%
73.95m – 250mmø PVC SAN @ 1.00%
80.00m – 250mmø PVC SAN @ 1.00%
14.05m – 250mmø PVC SAN @ 1.5%
14.77m – 250mmø PVC SAN @ 1.0%
14.78m – 250mmø PVC SAN @ 1.0%
17.18m – 250mmø PVC SAN @ 1.0%
11.19m – 250mmø PVC SAN @ 1.0%
11.76m – 250mmø PVC SAN @ 1.0%
14.13m – 250mmø PVC SAN @ 1.0%
EX 85.43m – 250mmø PVC SAN @ 1.00%
EX 65.92m – 300mmø PVC SAN @ 0.50%
EX 68.02m – 300mmø PVC SAN @ 0.50%
EX 11.12m – 250mmø PVC SAN @ 1.00%
EX 13.50m – 250mmø PVC SAN @ 0.50%
21.09m – 250mmø PVC SAN @ 0.5%
11.16m – 250mmø PVC SAN @ 1.0%
SAN. CONTROL MH SW INV. 164.91
SAN. CONTROL MH SE INV. 164.48
SAN. CONTROL MH SE INV. 165.87
SAN. CONTROL MH SE INV. 166.11
SAN. CONTROL MH NW INV. 164.51
SAN. CONTROL MH NW INV. 165.50
SAN. CONTROL MH NW INV. 166.14
SAN. CONTROL MH SW INV. 164.28
SAN. CONTROL MH SE INV. 163.27
BH1
BH2
BH3
BH4
BH5
BH6
BH7
BH8
BH15
BH16A
BH17
BH18
BH19
MH1A
MH2A
MH3A
MH4A
MH5A
MH6A
MH7A
MH8A
MH9A
MH10A
MH11A
MH12A
MH13A
MH14A
MH15A
MH16A
MH17A
MH18A
MH19A
MH20A
MH21A
MH22A
MH23A
MH24A
MH25A
MH26A
MH27A
MH28A
MH29A
MH30A
MH31A
MH32A
MH33A
MH34A
MH35A
MH36A
MH37A
MH38A
MH39A
MH40A

APPENDIX J

FUNCTIONAL SERVICING AND STORM WATER MANAGEMENT REPORT PREPARED BY MASONGSONG ASSOCIATES ENGINEERING LTD – DEC. 2017

Pinnacle Uptown Phase 4, “Perla” Residential Condominium
Watergarden Drive, Mississauga
Mondiale Development Ltd.

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

December 2017

MAEL Reference 16-803

MASONGSONG ASSOCIATES ENGINEERING LIMITED
ENGINEERING SUSTAINABLE FUTURES

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

PROPOSED PINNACLE UPTOWN PHASE 4, "PERLA" RESIDENTIAL CONDOMINIUM
WATERGARDEN DRIVE AND HURONTARIO STREET

FOR

MONDIALE DEVELOPMENTS LTD.

CITY OF MISSISSAUGA

December 2017

Prepared by:

MASONGSONG ASSOCIATES ENGINEERING LIMITED
7800 Kennedy Road, Unit 201
Markham, Ontario • L3R 2C7
T (905) 944-0162 • F (905) 944-0165

MAEL Project No. 16-803

1.0 INTRODUCTION

Masongsong Associates Engineering Limited has been retained by Mondiale Developments Ltd. to prepare a *Functional Servicing and Stormwater Management Report* in support of a Site Plan Application for a proposed Plan of Condominium. The property is proposed to be developed into a residential condominium site plan located at the northwest corner of Hurontario Street and Watergarden Drive in the City of Mississauga.

This study provides an overview of the proposed development and examines servicing feasibility within the framework of existing infrastructure. Specifically, this report will address the Regional servicing jurisdiction of water distribution and sanitary sewerage, as well as the City of Mississauga criteria for storm drainage and grading.

For clarity, this project is referenced as Uptown Phase 4, "Perla".

1.1 Background

The rectangular-shaped site of approximately 0.61 ha (1.51 ac) is the west-half of the northeasterly high-density block within the Pinnacle subdivision, located northwest of Eglinton Avenue and Hurontario in Mississauga (Refer to the Site Location Plan below in Figure 1). The development block is framed by existing municipal roads, namely Hurontario to the east and Salishan Circle to the north; the west and south boundaries of the block are new subdivision roads developed by Pinnacle, being Foursprings Avenue and Watergarden Drive, respectively.

Figure 1 *Site Location Key Plan*

A larger scale architectural Site Plan is enclosed in Appendix D.

The development site is known as "Perla Phase 4" in a marketing context, but is in fact the first phase of development within this block. The proposed development will comprise two residential condominium towers registered as **one** Condominium Corporation with **468 total units**:

- Building 1 15 stories, 178 units
- Building 2 34 stories, 290 units

The towers will be linked at ground-level with a 3-storey podium, and share a 4-level underground parking garage.

It is noted that the east-half of the block will comprise of a future residential tower (known as Building 3), but will be developed as another condominium corporation. This distinction is important to the overall servicing strategy, as each of the east- and west-halves will need separate services.

1.2 Planning Status and Previous Studies

It is understood that the underlying subdivision is OPA and Draft-Approved, and the appropriate zoning is in-place for the subject block (Related City Files: 0Z 07/024 and 0Z 07/025, and Subdivision Files T-M07005 and T-M07006). The present application and the scope of this proposal is to support a Site Plan Application for OPA and ReZoning amendment.

The underlying subdivision was supported by a number of studies, notably including "master" Functional Servicing and Stormwater Management Reports:

- *Functional Servicing Report, Phase II OPA/Rezoning/Draft Plan Of Subdivision, Pinnacle International (Ontario) Limited*, January 2011, IBI Group, Reference No: 14604.

- *Technical Memorandum: Hydraulic Modelling Analysis for Proposed Development – Uptown Mississauga Phase 2*, January 14, 2011, AECOM Canada Limited, Reference No: 60190634

This current report for the subject block relies on the acceptance of the underlying subdivision master reports as the basis for receiving system capacity and stormwater control targets. It is noted that the subdivision roads and infrastructure (namely Little Creek Road, Watergarden Drive and Foursprings Avenue) are constructed and operational at the time of this writing.

2.0 EXISTING INFRASTRUCTURE

The site is primarily serviced by existing infrastructure on Watergarden Drive and Foursprings Avenue, both of which are newly constructed municipal roadways as part of the underlying Pinnacle Subdivision. More specifically:

A **300 mm diameter watermain** loops runs within all abutting rights-of-ways, including Hurontario Street and Salishan Circle (pre-existing municipal roadways), and Watergarden Drive and Foursprings Avenue (new municipal roadways). A general plan of the surrounding watermain network is excerpted from the IBI FSR, enclosed in Appendix A as Figure 4. As part of the subdivision, AECOM was engaged to evaluate the water distribution network and concluded that:

"• There is no significant hydraulic impact to the system when adding the proposed development. Some pressures were observed to exceed the maximum pressure based on MOE guidelines in the average and maximum day demand scenarios. However, as described in Section 4a) this is not caused by the addition of the proposed new development.

- *Servicing to the development can be supplied via the 300mm diameter watermain along Salishan Circle and the 300mm diameter watermain along Hurontario Street. These watermains also provide sufficient water service security to the system because of the looping that would exist within the development.*

- *The velocity and head loss gradients within the pipes in the development would be within the recommended range under average day, maximum day and peak hour conditions. Therefore, the sizing and routing of the pipes is considered acceptable.*

- *Allowable fire flow that maintains the minimum required pressure of 20psi was calculated…The required 250L/s fire flow could be supplied within the recommended pressures. The maximum available fire flows were calculated as follows:*
  - *2016: 433.2 L/s*
  - *2031: 492.9 L/s "*

A **250 mm diameter local sanitary sewer** runs within Watergarden Drive, westerly to Foursprings Avenue, and then further northwest to a servicing easement which ultimately connects to the 750mm subtrunk on Fairwind Drive. A general plan of the sanitary sewer system is excerpted from the IBI FSR, enclosed in Appendix A as Figure 2.

As described in the IBI FSR, the Pinnacle Subdivision has been split into two sewersheds which drain to the:

- West subtrunk (Fairwind Drive, 200.9 L/s residual capacity)
- East subtrunk (Eglinton Avenue, 38.1 L/s residual capacity)

The East subtrunk has limited residual capacity, whereas the West subtrunk is less constrained. As presently designed, the subject site is serviced by a local sanitary sewer that is tributary to the West subtrunk which has sufficient capacity to accommodate the proposed development without any downstream systemic upgrades.

A **525 mm diameter and 375 mm storm sewer** runs within Watergarden Drive and Foursprings Avenue, respectively, which collectively continues south on Foursprings Avenue to the Eglinton storm trunk system; ultimately, the Eglinton storm trunk discharges to Cooksville Creek, which also traverses the underlying subdivision. A general plan of the minor storm sewer system is excerpted from the IBI FSR, enclosed in Appendix A as Figure 3. Note that at the time of the IBI FSR report, the subject site was denoted as "Block 3".

Minor storm from the subject site is tributary to the Foursprings system. A detailed OTTHYMO hydrologic anlaysis in the IBI report resulted in control targets for each of the blocks. As excerpted from the IBI Report:

"Section 4.2.5…Pre-Development Peak Flows: As part of the current regulatory floodplain mapping study for the Cooksville Creek, a detailed hydrologic analysis has previously been conducted for the watershed by R. V. Anderson Associates Limited in Feb. 1996 in the Study entitled "Cooksville Creek Floodline Mapping Study", using the OTTHYMO computer program. The same hydrologic model established in the Cooksville Creek Floodline Mapping Study was applied in this report to estimate the predevelopment peak flows for the development Blocks (Blocks 2, 3, and 4) within the Phase II site under the design storms.

…According to the proposed stormwater management plan, the post-development design peak discharges from all of the development blocks (Blocks 2, 3 and 4) into the Phase II storm sewer systems must be controlled to or below their corresponding pre-development levels for the 2, 5 and 10-year design storms.

Table 1. Pre-Development Design Peak Flows for Phase II Development Blocks (m^3/s)
Block 3, 1.53 ha 0.046 (2 yr) 0.086 (5 yr) 0.127 (10 yr) 0.232 (100 yr)

… on-site stormwater detention facilities equipped with flow control devices, will be required for all development blocks (Blocks 2, 3 and 4) within the Phase II development site in order to satisfy the desired water quantity control requirements. As a result, the storm sewer systems within each development block will be designed in accordance with the City of Mississauga's current storm sewer design criteria to collect and convey the 10-year design peak flows into the on-site stormwater detention facilities for peak flow attenuation.

The **major storm (100 year) overland flow routes**, being Watergarden Drive, Foursprings Avenue, and Little Creek Road are all recently constructed municipal roads, and they are designed to convey the 100-year overland flow directly to Cooksville Creek. An overland spillway at Little Creek Drive provides the safe discharge into Cooksville Creek. As noted in the IBI Report:

> "*Section 4.2.4…The major storm runoff flows (up to the 100-year design storm), in excess of the storm sewer conveyance capacities, will drain overland along the road network and discharge through two overland flow routes directly into the Cooksville Creek."*

3.0 PROPOSED SERVICING

The equivalent population basis (for sanitary sewerage and water servicing demands) is partially derived from the Peel Region Public Works Design, Specification & Procedures manual. The Manual provides for generic population densities of various building forms, and more specifically 475 pph (persons per hectare) for high-rise developments, or based on 2.7 ppu (persons per unit) for higher density forms.

The subject site comprises 468 residential units on 0.61 ha, which far exceeds the generic rate of 475 pph. Using the 2.7 ppu criteria, the resulting equivalent population is therefore:

Site Specific Population = 2.7 ppu x 468
= 1,264 persons

Population Density = 1,264 persons / 0.61 ha
= 2,072 pph

In addition, a commercial portion of 234 m^2 is also proposed within Phase 1. For commercial areas, an equivalent population of 50 persons per hectare will be applied as per Region standards; the resulting equivalent population is therefore:

Commercial Population = 234 m^2 x (50 persons/10,000 m^2)
= 1.2 persons

Therefore, the total population for this Phase 1 develoment is **1,266 persons.** These population statistics are carried forward in the following sections on water demands and sanitary sewerage.

3.1 Water Distribution

In the Region of Peel, the overall water network model is maintained by the Region, and therefore a detailed distribution model is not included with this report. The local subdivision water distribution network was also already analyzed by AECOM in their 2011 *Modelling Analysis*, which accommodated the subject site as a high-rise high-density development. For comparative purposes, Table 2.3 of the AECOM 2011 Modelling Analysis included an equivalent population of 1,914 persons (but also included 6,179 sq.m. of commercial development).

The present phase of development is estimated population of 1,266 persons is well within the original projections. Using the Peel Region water demand criteria, the estimated water demand is summarized in Table 3.1 below.

Table 3.1 *Development Statistics*

Site Description	No. of Units	Equivalent Population (2.7 ppu) (50 persons/1ha)	Avg Day Demand (280 L/c/d) Res (300 L/c/d) Comm	Max Day Demand (2.0 Factor) Res. (1.4 Factor) Comm.	Peak Hour Demand (3.0 Factor) Res. (3.0 Factor) Comm.
Building 1	178	481	1.56 L/s	3.12 L/s	4.68 L/s
Building 2	290	783	2.54 L/s	5.08 L/s	7.62 L/s
Commercial	-	2	0.007 L/s	0.01 L/s	0.021 L/s
Totals	468	1,266	4.11 L/s	**8.21 L/s**	**12.33 L/s**

The subject site is presently serviced with three existing 200 mm diameter water connections located at:

- the northwest corner (serviced off Four Springs Avenue),
- the southwest corner (serviced off Watergarden Drive),
- the southeast corner (serviced off Watergarden Drive).

It is proposed to utilize the southwest Watergarden Drive connection for domestic water supply to the residential development. The southeast Watergarden water connection will be removed.

A Site Servicing Plan is included in the Drawings Appendix as SS-1, for reference.

3.1.1 Fire Protection

Three (3) existing municipal hydrants are within close proximity (less than 20m from the site boundaries), located on Salishan Circle, Four Springs Avenue and Watergarden Drive. These are also shown on the Site Servicing Plan. Therefore municipal hydrant coverage is more than sufficient for complete and overlapping coverage.

The Building 2 tower will rise 34 storeys, which is greater than 84m. In accordance with the Ontario Building Code, structures greater than 84m are to be provided with two independent fire supply lines which can be isolated on separate systems. To this end, both existing 200 mm connections (southwest Watergarden Drive and northwest Four Springs Avenue) will be retained for supply to the building's fire fighting system.

3.1.2 Water Demand

The max-day domestic consumption rate of 8.21 L/s or 492.6 L/min is a fraction of the 300 mm diameter watermain that is well looped; therefore domestic water demand can be easily met.

However, the fire flow is much more critical. Based on the Fire Underwriters Survey (FUS), the required fire flow is 6,000 L/min. See Tables F2-F3 (Appendix B).

A hydrant flow test, enclosed in Appendix B, was performed in June 2017 to ascertain the available municipal supply on Watergarden Drive.
Detailed hydrant flows are calculated in Table F1 in Appendix B, confirming that the existing Watergarden Drive system is capable of delivering a fire flow of **14,358 L/min at a minimum pressure of 140 kPa**, which satisfies both FUS and ISO fire flows superimposed on the max-day domestic consumption rate of 492.6 L/min.

3.2 Sanitary Sewerage

As noted above, the subject site is accommodated in the master sewerage analysis of the underlying subdivision. The equivalent population of 1,266 persons in the present block development is less than the original projection of 1,914 persons and therefore deemed to have sufficient receiving capacity.

The as-constructed plan profile drawings from IBI Group provides for three (3) control manholes to this block, perhaps in anticipation of multiple condominium corporations. As presently proposed, the development is proceeding as a single condominium corporation, and therefore, only one connection point will be utilized (southwest corner of the block, as indicated on the Site Servicing Plan, Drawing SS1).

The other two control manholes will be removed, and the leads grouted in place.

3.3 Storm Drainage

The subject site is part of the Cooksville Creek subwatershed, and drainage within this catchment is controlled as part of the underlying Pinnacle Subdivision Master Functional Servicing Plan. In accordance with the City and Subdivision stormwater management guidelines, this site is required to control the 10-year post-development flows to pre-development levels; storm drainage above the 10-year peak will continue overland.

Locally, the subject site is tributary to the storm sewer system within Four Springs Avenue and Watergarden Drive. The approved storm drainage plan of the underlying subdivision contemplates the subject block to be split-drained between these two municipal roadway systems.

The available existing storm sewer connections include:

- A 375 mm diameter storm lead off Four Springs Avenue (northwest Block corner),
- A 300 mm diameter storm lead off Four Springs Avenue (southwest Block corner),
- A 300 mm diameter storm lead off Watergarden Drive (southeast Block corner).

3.3.1 Allowable Discharge

Post development flows up to the 10-yr post will be controlled to the allowable release rate of 30.1 L/s/ha as indicated page 6-7 of the *Functional Servicing Report, Phase II OPA/Rezoning/Draft Plan Of Subdivision, Pinnacle International (Ontario) Limited*, January 2011, IBI Group, Reference No: 14604.

Therefore, total allowable release rate from this development phase is **19.2 L/s** [30.1 L/s/ha*0.6390 ha.]

3.3.2 Proposed SWM Scheme – Minor System Control

To meet the stormwater quantity objectives, the subject site is proposed to provide on-site water quantity control up to the maximum allowable release rate **19.2 L/s**. The following table outlines the composite site runoff coefficients.

Table 3.2 *Post-Development Composite Runoff Coefficient*

Component Area	Total Area (Ha)	Runoff Coefficient "R"	Average Runoff Coefficient
Rooftop Area	0.3505	0.90	0.3079
Landscaping Area	0.1052	0.25	0.0263
Asphalt/Conc. Area	0.1833	0.90	0.1650
Total	**0.6390**		**0.79**

Note: Refer to Fig. C1, Appendix C for Post-Development Drainage Plan

The mass Rational Method was used to calculate the 10-year storage requirement for the site. Computation tables for the volumetric sizing are included in Appendix C. Given the limited space available on-site due to the building footprint occupying most of the site area, a below grade cistern within the parking structures is proposed to provide the volumetric attenuations.

A summary of post development flows is presented in the following Table 3.3.

Table 3.3 *Summary of Post Development Flows*

Control Method	Total Area (Ha)	Average Runoff Coefficient	Maximum Release Rate (Ls)	Required Storage (m^3)	Provided Storage (m^3)
75mm Orifice Tube	**0.6390**	**0.79**	**18.0**	**133.6**	**150.0**

The discharge rate is proposed to be controlled by a 75mm diameter orifice control tube located upstream of the proposed control maintenance hole #1.

The proposed storm connection can be seen on the Proposed Site Servicing Plan (SS1) attached in the drawing appendix.

3.3.3 Major Overland flow

The proposed grade within the block have been designed such that for storms greater than the 10-yr events or in the case of emergency overflow due to clogging in the storm system, safe overland flow route exists are established to convey flow away from the site and into the Four Springs Avenue and Watergarden Avenue where they are designed to convey the 100-year overland flow directly to Cooksville Creek

The overland flow routes will have no depth of ponding greater than 0.25m and will not result in flood damage to proposed and adjacent public and private properties.

3.3.4 Stormwater Quality

The City policy on infill developments allows for cash-in-lieu payments for stormwater quality controls towards downstream improvements. The nature of the proposed development, being residential units with no industrial/commercial use and a low risk of spills-contaminations, will likely be best served with cash-in-lieu stormwater quality and is recommended in the IBI master functional servicing report.

3.3.5 Mississauga Green Development Standards

The current plan comprises a structural footprint occupying virtually the entire site, therefore the site is not suitable for infiltration-type Best Management Practice devices (BMPs) such permeable pavement, bioretention area and soakaway pit. However, the addition of green roof and the first 5 mm water balance will be implemented for this development block.

The volume of on-site water retention for the first 5 mm water balance is estimated in the following Table 3.4.

Table 3.4 5 mm Water Balance Volumes

Surface Area Component	Area	Initial Abstraction		Target Water Retention	Deficit Storage Required to meet Water Balance Target	
	(m^2)	(mm)	(m^3)	(mm)	(mm)	(m^3)
Rooftops	3,505	1	3.50	5	4	14.02
Conc./Paved	1,833	1	1.83	5	4	7.33
Sod/Landscape	1,052	5	5.26	5	0	0
Total	**6,390**		**10.59**			**21.35**

A total of **21.35 m^3** of additional on-site storage is required to meet the first 5 mm site retention targets. The required volume will be storage in a proposed cistern for irrigation and water re-use. Detailed design of storage tank and irrigation system will be completed by the building mechanical consultant at the time of detailed building design.

3.3.6 Erosion and Sediment controls

An erosion and sediment control strategy should be implemented during the construction to mitigate the transportation of silt from the site.

To prevent construction generated sediments from entering the storm sewer or leaving the site by overland flow, the following measures should be implemented:

- Temporary silt fencing
- Temporary catch basin sediment control
- Temporary rock mud mats
- Seeding and mulching of disturbed undeveloped areas
- Erosion monitoring and sediment removal program throughout the construction period

An Erosion and Sediment Control Plan showing all of the measures are showing on the Erosion and Sediment Control Plan (ESC-1).

4.0 FUTURE DEVELOPMENT TO THE EAST (PHASE 2)

It is noted that the east-half of the block will comprise of a future residential tower (known as Building 3), but will be developed as another condominium corporation. The future Phase 2 will be provided with separate services and stormwater management techniques. Based on the current layout, a 200m^3 storage volume is required within Phase 2 (see Fig. B1). Therefore, total onsite storage volume provided within this development block should equal to 350 m^3 (150m^3 Phase 1 & 200m^3 Phase 2) which is in line with a letter dated October 24, 2014 singed by Mark G. Bales of Pinnacle International Ltd.

Based on a current topographic survey of existing conditions, Phase 2 site is generally drains from north to south at an average slope to 2%. Based on field observation, the area is covered entirely by overgrown grass. Therefore, based on the existing conditions, Phase 2 land can remain as it as drainage does not affect Phase 1 development. See SGR-1 plan for drainage pattern. It should be noted that when Phase 2 is developed, the overland flow route will continue to drain into Watergarden Drive as per the original subdivision design.

5.0 SUMMARY AND RECOMMENDATIONS

Existing infrastructure on Watergarden Drive and Four Springs Avenue have sufficient capacity to accommodate servicing of the subject development. More specifically:

Water Supply will be accommodated by the existing 200 mm and 100 mm diameter watermain connections at the southwest corner of the block off Watergarden Avenue. The 200 mm diameter waterline will serve as the primary fire service and the 100 mm diameter waterline will serve as the domestic water line. The existing 200 mm diameter water connection at the northwest of the block off Four Springs Avenue will serve as the secondary fire line as Building 2 is over 84.0m tall. Hydrant flow tests and analysis confirms there is adequate supply and pressure fire fighting purposes.

Sanitary Sewerage will be accommodated by the existing 250 mm diameter lateral at the southwest corner of the block off Watergarden Drive. An equivalent population of 1,266 persons is calculated for this block development which is less than the original projection of 1,914 persons and therefore deemed to have sufficient receiving capacity.

Storm Drainage will be accommodated by the existing 300 mm diameter storm lateral at the southwest corner of the block off Four Springs Avenue. The entire block will be captured and attenuated from the 10-year post peak event down to the allowable pre-development discharge rate of 30.1 L/s/ha. Quantity attenuation storage will be provided by an underground storage tank, with controlled discharge through a 75 mm diameter orifice tube. Stormwater quality is recommended as a cash-in-lieu

payment as per the City policy on infill developments.

Respectfully Submitted,

MASONGSONG ASSOCIATES ENGINEERING LIMITED

Professional Engineers Ontario
Limited Licensee
Name: K. K. LO
Number: 100209166
Category: CIVIL
Limitations:
This licence is subject to the limitations as detailed on the certificate.
Association of Professional Engineers of Ontario

LICENSED PROFESSIONAL ENGINEER
W. C. IP
PROVINCE OF ONTARIO

Ken Lo, LEL, C.E.T.
Senior Project Designer

Andrew Ip, P.Eng
Principal

H:\PROJECTS\16\803\Design\16803_FSR_SWM_r0.docx

EX. SINGLE FAMILY RESIDENTIAL

EX. PUBLIC SCHOOL

SALISHAN CIRCLE (WEST LEG)

SALISHAN CIRCLE (EAST LEG)

EX. SINGLE FAMILY RESIDENTIAL

EX. RESIDENTIAL TOWER

CEREMONIAL DRIVE

BLOCK 5 (OPEN SPACE)

STREET `A'

BLOCK 3

BLOCK 4

STREET `B'

BLOCK 2

STREET

FOUR SPRINGS AVE

LITTLE CREEK RD

CITY OWNED LANDS

COOKSVILLE CREEK

BLOCK 9 (OPEN SPACE)

FAIRWIND DRIVE

BLOCK 7

BLOCK 8

PROPOSED FLOOD LINE (UPDATED BY IBI GROUP)

BLOCK 6 (OPEN SPACE)

BLOCK 1

PHASE I BLOCK 1

FOURSPRINGS AVENUE

LITTLECREEK ROAD

ESSO GAS STATION

HURONTARIO STREET

3.0m WIDE ROAD WIDENING

EX. COMMERCIAL / RETAIL PLAZA

RW 1

RW 2

RW 3

RW 4

12.5m WIDE ROAD WIDENING

5.0m WIDE ROAD WIDENING

EGLINTON AVENUE WEST

LEGEND:

- PROPOSED WATER MAIN
- WATER MAIN (FUT. PHASES)
- EX. WATERMAIN
- EASEMENT
- DENOTES HYDRANT
- PROPERTY BOUNDARY
- PHASE I BOUNDARY

30 0 20 40 80

1:2,000(m)

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PINNACLE INTERNATIONAL

TITLE:

WATER DISTRIBUTION SYSTEM PHASE II

JOB No:	14604
SCALE:	1:2000
DATE:	JANUARY, 2011
FIGURE No:	FIGURE 4

IBI GROUP
9133 Leslie Street
Suite 200
Richmond Hill, Ontario
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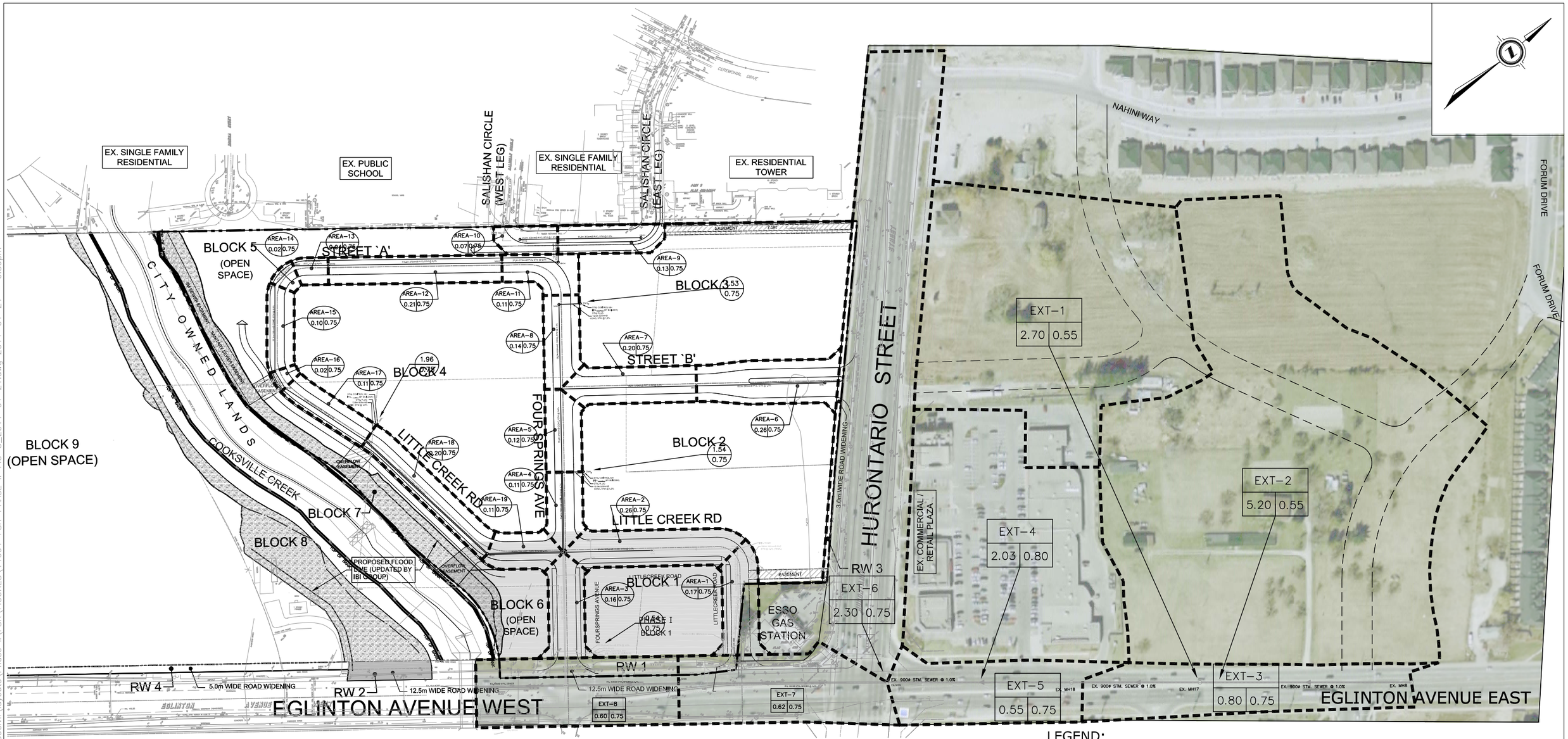

LEGEND:

- PROPERTY BOUNDARY
- STORM DRAINAGE AREA DIVIDE (PROPOSED)
- EX. MH1 — EXISTING STORM SEWER
- MH1 — PROPOSED STORM SEWER SERVICING
- CONTROL MH (BY BLOCK DEVELOPER)
- EASEMENT
- 0.61 / 0.55 — DENOTES DRAINAGE AREA IN HECTARES / DENOTES STORM RUNOFF COEFFICENT
- PHASE I BOUNDARY
- STORM DRAINAGE AREA DIVIDE (FUT. PHASES)
- MH1 — STORM SEWER SERVICING (FUT. PHASES)
- 0.61 / 0.55 — DENOTES DRAINAGE AREA (FUT. PHASES) / DENOTES STORM RUNOFF COEFFICENT

30 15 0 25 50 100

1:2,500(m)

PINNACLE INTERNATIONAL

TITLE:

MINOR STORM DRAINAGE AREA AND SEWER SYSTEM PHASE II

JOB No:	14604
SCALE:	1:2500
DATE:	JANUARY, 2011
FIGURE No:	FIGURE 3

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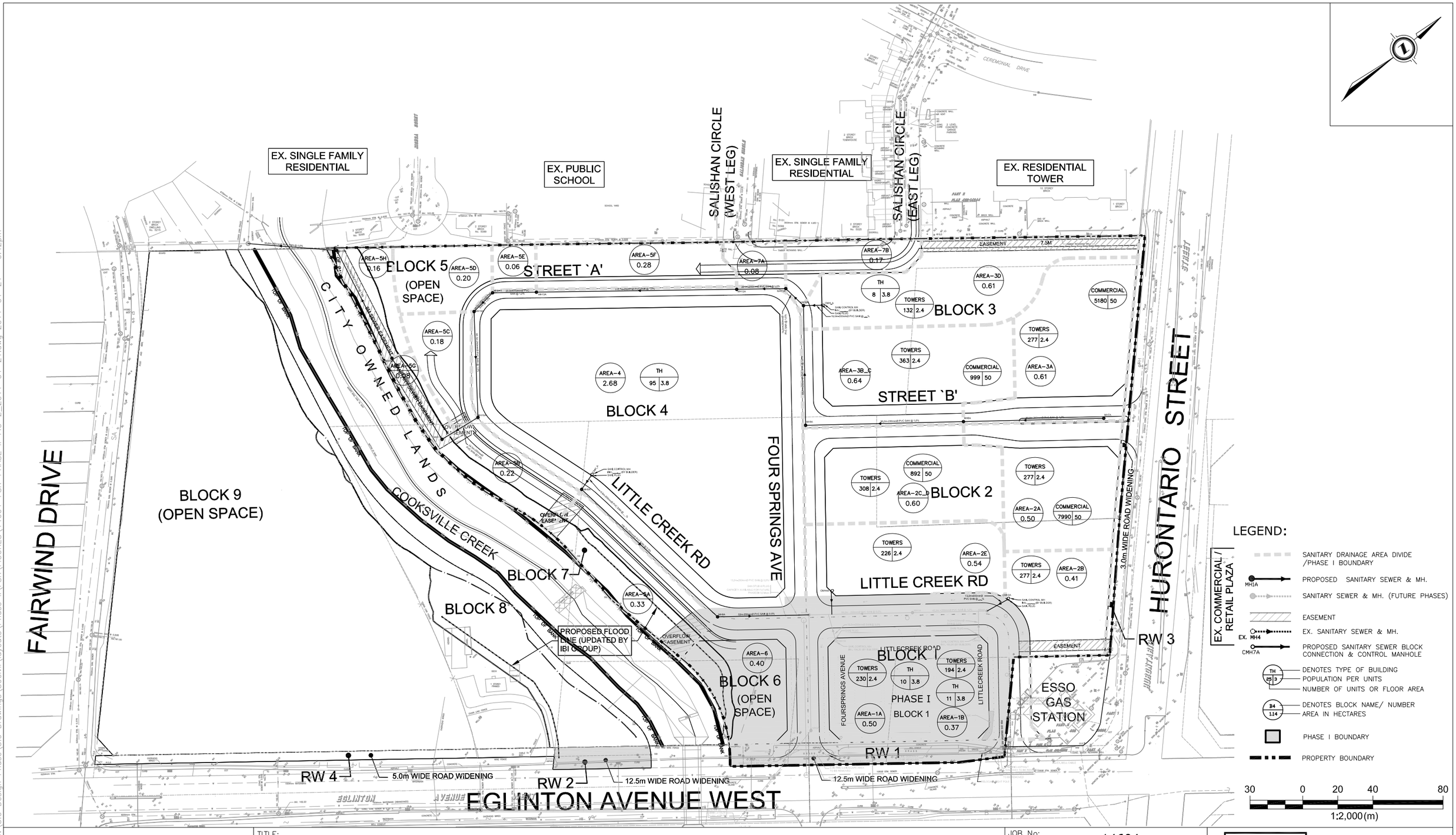
J:\14604_PinnacleDev\5.0 Design Phase\5.9 Drawings\59civil\layouts\Phase II\FSR\FIGURES\14604-FSR-PHASE II-FIG-3_2011-01-21.dwg 2011-01-21 - 6:59pm



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PINNACLE INTERNATIONAL

TITLE:

SANITARY DRAINAGE AREAS AND SEWER SYSTEM PHASE II

JOB No:	14604
SCALE:	1:2000
DATE:	JANUARY, 2011
FIGURE No:	FIGURE 2

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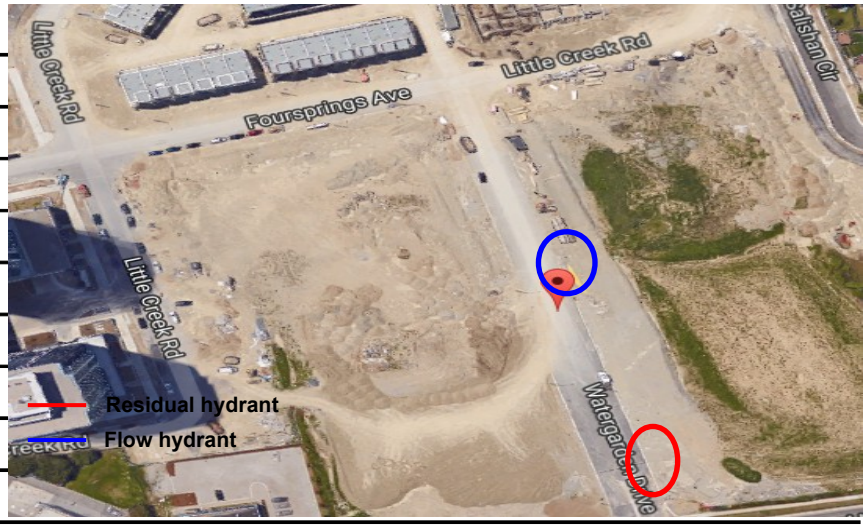
Appendix B

Watermain Analysis:

- Hydrant Flow Test
- FUS Fire Demand Calculation

GTA Waterworks

Main Size	300 mm
Static Pressure (psi)	88
Elevation Difference (m)	0 m
Location	35 Watergarden
Municipality	Mississauga
Operator	Jim and Gary
Tested By	M.Larocca
Date	20-Jun-17
Time	10:00 AM
Remarks	



Flow Test Measurements

# of Ports	Nozzle Size (in)	Pressure Flow Gauge (psi)	Flow (U.S.G.P.M)	Flow (L/s)	Residual Pressure (psi)
1	2	30	854	53.9	86
2	2	24	764	48.2	84
	2	25	780	49.2	84
1	2	28	825	52.0	86

Test Results

Flow at 20 psi (140 kPa) **5,500** U.S.G.P.M

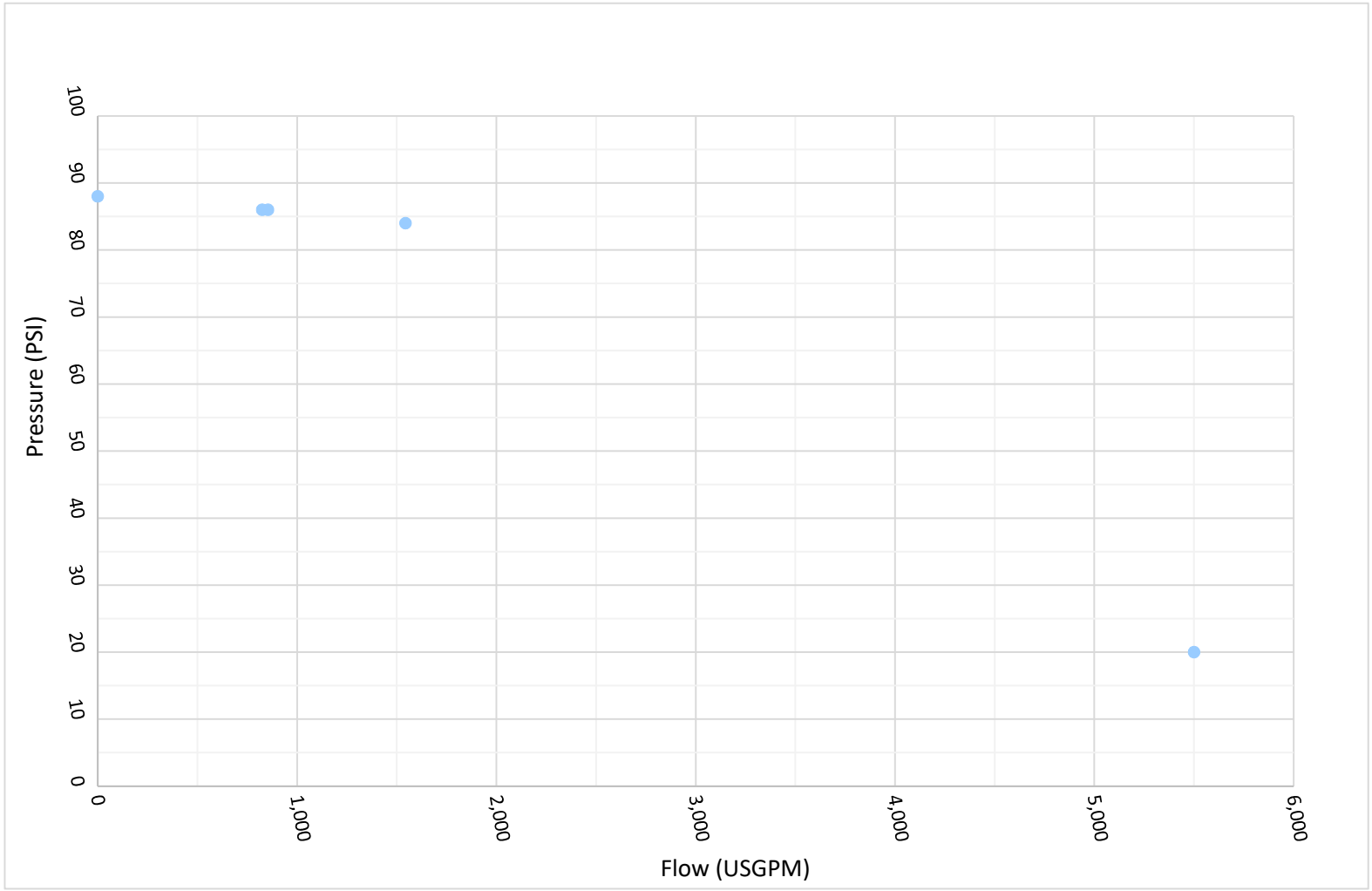
Hydrant Classification as per NFPA 291	
Class	Colour
AA	Blue

$$Q_r = Q_t \left(\frac{P_s - P_r}{P_s - P_t} \right)^{0.54}$$

Qr = fire flow at residual pressure P (gpm, l/s)
Qt = hydrant discharge during test (gpm, l/s)
Ps = static pressure (psi, kPa)
Pr = desired residual pressure (psi, kPa)
Pt = residual pressure during test (psi, kPa)

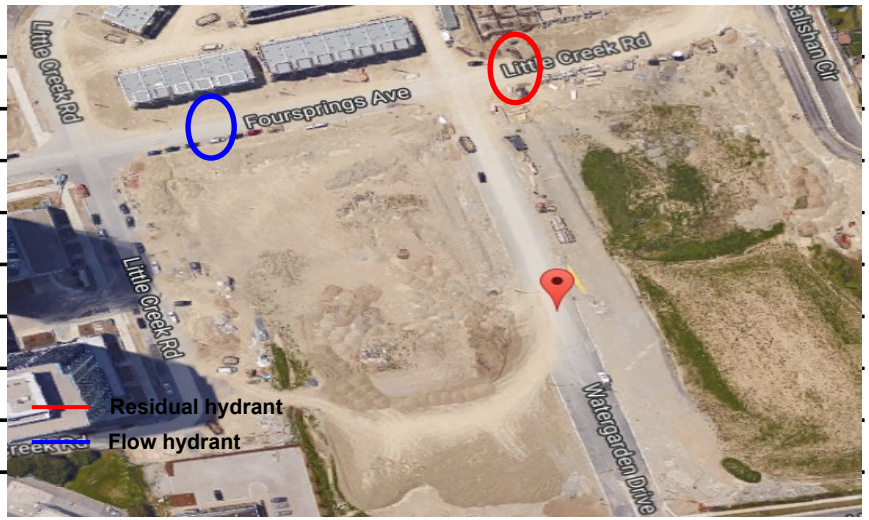
Hydrant Flow Curve

35 Watergarden Dr.



GTA Waterworks

Main Size	300 mm
Static Pressure (psi)	91
Elevation Difference (m)	0 m
Location	65 Watergarden
Municipality	Mississauga
Operator	Jim and Gary
Tested By	M.Larocca
Date	20-Jun-17
Time	10:30 AM
Remarks	



Flow Test Measurements

# of Ports	Nozzle Size (in)	Pressure Flow Gauge (psi)	Flow (U.S.G.P.M)	Flow (L/s)	Residual Pressure (psi)
1	2	30	854	53.9	89
2	2	24	764	48.2	87
	2	25	780	49.2	87
1	2	27	811	51.2	89

Test Results

Flow at 20 psi (140 kPa) **5,600** U.S.G.P.M

Hydrant Classification as per NFPA 291	
Class	Colour
AA	Blue

$$Q_r = Q_t \left(\frac{P_s - P_r}{P_s - P_t} \right)^{0.54}$$

Qr = fire flow at residual pressure P (gpm, l/s)
Qt = hydrant discharge during test (gpm, l/s)
Ps = static pressure (psi, kPa)
Pr = desired residual pressure (psi, kPa)
Pt = residual pressure during test (psi, kPa)

Hydrant Flow Curve

65 Watergarden Dr.

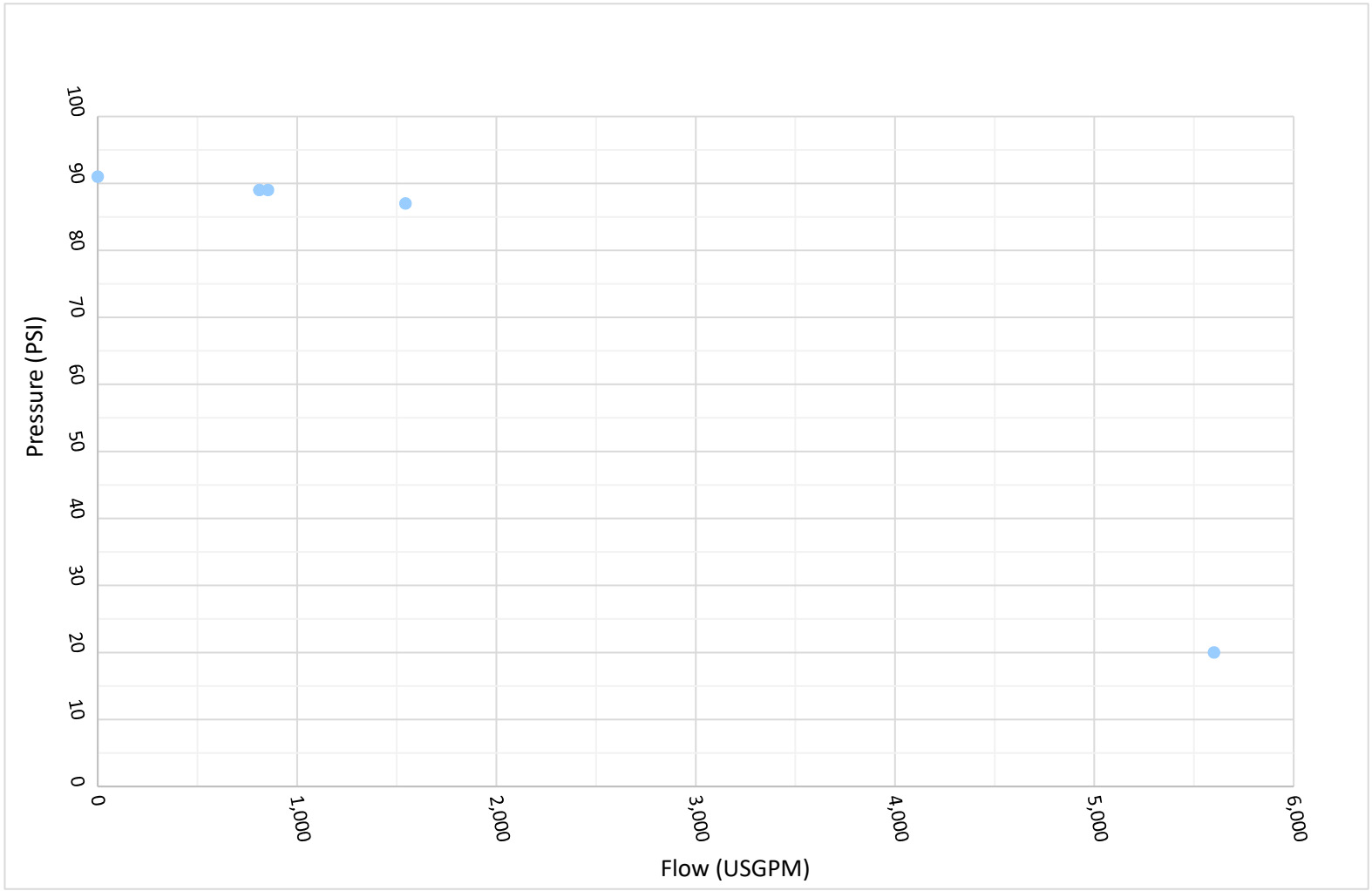


Table F1 Available Fire Flow Calculations

Project:	**Perla Residential Condominium Developments**		
Client:	**Mondiale Development Ltd.**		
Outlet diameter:	**2** in, one port	Location:	**35 Watergarden Drive, Mississauga**
Static pressure:	**88** psi	Date of Test:	**20-Jun-17**
Resid. pressure:	**86** psi, one port	Operator:	**GTA Waterworks**

• **Observed Flow** $Q_F = 29.83 \times C \times (d^2) \times (p^{0.5})$

where	C =	**0.88**	Coefficient
	d =	**2.00**	in, Outlet diameter
	p =	**30.00**	psi, Pitot Pressure
⇨	Q_F =	**565**	USGPM
		2,138	L/min

• **Available Flow** $Q_R = Q_F \times (h_R^{0.54}) / (h_F^{0.54})$

where	h_F =	2.00	psi, Pressure difference, static to measured residual
	h_R =	68.00	psi, Pressure difference, static to required residual
	Required =	**20.00**	psi
⇨	Q_F =	**3,793**	USGPM
		14,358	L/min

Table F2 Required Fire Flow Calculations

Project: **Perla Residential Condominium Developments**
Client: **Mondiale Development Ltd.**

- **Base Flow**

$F_B = 220 \times C_C \times A^{0.5}$

where C_C = 0.60 — from Table F3
A = 2876 m^2 — from Table F3
⇨ F_B = 7,079 L/min
8,000 L/min — rounded to nearest 1,000 L/min

- **Occupancy Factor**

C_O = **-15%** — from Table F3
$F_O = F_B + (F_B \times C_O)$
= 6,800 L/min

- **Sprinkler Factor**

C_S = **-30%** — from Table F3
$f_S = F_O \times C_S$
= -2,040 L/min

- **Exposure Factor**

C_E = **20%** — from Table F3
$f_E = F_O \times C_E$
= 1,360 L/min

- **Total Required Flow**

$F = F_O + f_S + f_E$
= 6,120 L/min
= **6,000** L/min — rounded to nearest 1,000 L/min

Table F3 Building Area and Coefficients

Project: **Perla Residential Condominium Developments**
Client: **Mondiale Development Ltd.**

- **Area of Building** **2,876** m^2

The total floor area in square metres (including all storeys, but excluding basements at least 50 percent below grade) in the building being considered.

For fire-resistive buildings, consider the two largest adjoining floors plus 50 percent of each of any floors immediately above them up to eight, when the vertical openings are inadequately protected.

✓ *If the vertical openings and exterior vertical communications are property protected (one hour rating), consider only the area of the largest floor plus 25 percent of each of the two immediately adjoining floors.*

- **Construction Coefficient** floors. **0.60** ⇔

1.50	Wood Frame
1.00	Ordinary Construction
0.80	Non-Combustible
0.70	Fire Resistive (<2 hrs)
0.60	Fire Resistive (>2 hrs)

- **Occupancy Coefficient** C_O = **-15%** ⇔

-25%	Non-Combustible
-15%	Limited Combustible
0%	Combustible
15%	Free Burning
25%	Rapid Burning

- **Sprinkler Coefficient** C_S = **-30%** ⇔

-30%	NFPA 13 standard
-40%	+ fully supervised
-50%	+ std water supply

- **Exposure Coefficient** C_E = **20%** ⇔

N	> 30m	5%
S	> 30m	5%
E	> 30m	5%
W	> 30m	5%

25%	0 - 3m separation
20%	3.1- 10m separation
15%	10.1- 20m separation
10%	20.1- 30m separation
5%	> 30m separation
	percentages counted per side, max 75%

2017-05-08

Perla GFA Breakdown				
	Building 1		Building 2	
Level	GFA (sf)	GFA (sm)	GFA (sf)	GFA (sm)
P5	834	77.5		
P4	2,296	213.3		
P3	2,296	213.3		
P2	2,297	213.4		
P1/Lower Grnd	12,564	1,167.2		
1	22,532	2,093.3		
2	19,695	1,829.7		
3	14,010	1,301.6		
4	7,827	727.1	7,806	725.2
5 to 15	86,090	7,998.0		
5 to 17			101,466	9,426.5
18 to 33			124,863	11,600.1
34			7,874	731.6
Mech	0	0.0	587	54.5
Roof	0	0.0	0	0.0
Σ=	170,441	**15,832.9**	242,596	**22,535.7**

Total of GFA		cover page site stats	
38,368.6	≈	38,372.3	Okay

note: break out GFA outside wall to outside wall for fee calculation

Perla Underground Areas Breakdown				
	Parking Below Grade		Storage Below Grade	
Level	GFA (sf)	GFA (sm)	GFA (sf)	GFA (sm)
P5	5,672	527.0		
P4	58,335	5,419.5	5,750	534.2
P3	58,335	5,419.5	5,750	534.2
P2	56,171	5,218.4	5,380	499.9
P1/Lower Grnd	38,916	3,615.4	6,140	570.5
Σ=	217,429	**20,199.8**	23,020	**2,138.8**

Connection Demand Table

WATER CONNECTION

Connection point [3]	
35 WATERGARDEN DRIVE, MISSISSAUGA	
Pressure zone of connection point	
Total equivalent population to be serviced [1]	1,266 PEOPLE
Total lands to be serviced	0.639 Ha.
Hydrant flow test	
Hydrant flow test location @ 35 WATERGARDEN DRIVE, (NORTHEAST OF @ WATERGARDEN AND FOUR SPRINGS AVE.)	

	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	165	48.2	10:00 AM
Maximum water pressure	607	∅	10:00 AM

No.	Water demands			
	Demand type	Demand (in l/s)		
		Use 1 [5]	Use 2 [5]	Total
1	Average day flow	4.10	0.007	4.11
2	Maximum day flow	8.20	0.01	8.21
3	Peak hour flow	12.3	0.021	12.33
4	Fire flow [2]			100.0
Analysis				
5	Maximum day plus fire flow			108.2

WASTEWATER CONNECTION

				Total
Connection point [4]				
Total equivalent population to be serviced [1]				1,266
Total lands to be serviced				0.639 Ha.
6	Wastewater sewer effluent (in l/s)			

[1] Please refer to design criteria for population equivencies
[2] Please reference the Fire Underwriters Survey Document
[3] Please specify the connection point ID
[4] Please specify the connection point (wastewater line or manhole ID)
Also, the "total equivalent popopulation to be serviced" and the "total lands to be serviced" should reference the connection point. (The FSR should contain one copy of Site Servicing Plan)
[5] Please complete as many uses are necessary for the development.
(Please specify the use)

Please include the graphs associated with the hydrant flow test information table
Please provide Professional Engineer's signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

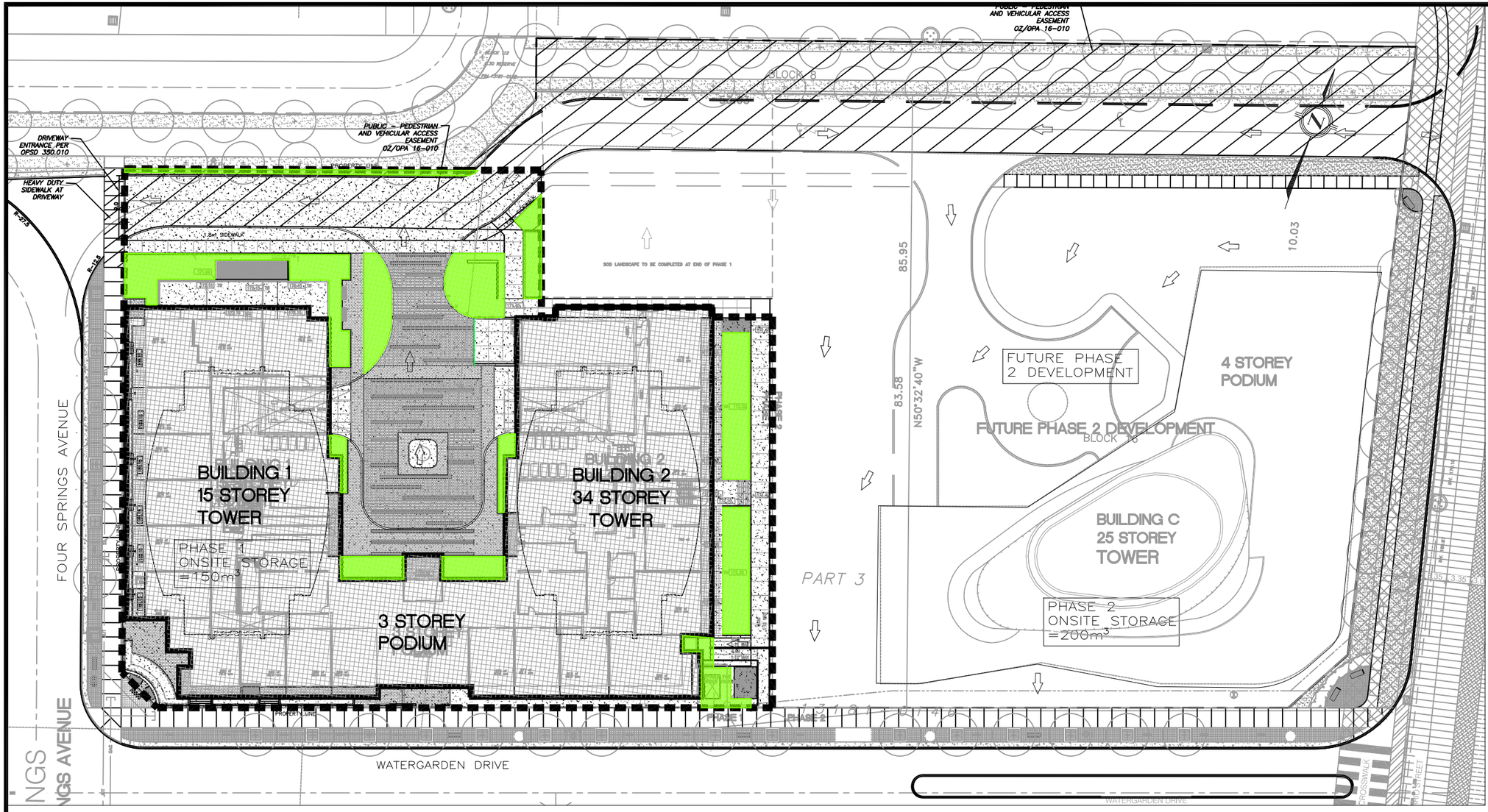
Professional Engineers Ontario
Limited Licensee
Name: K. K. LO
Number: 100209166
Category: CIVIL
Limitations:
This licence is subject to the limitations as detailed on the certificate.
Association of Professional Engineers of Ontario

JUNE 26, 2017

Appendix C

SWM Calculations:

- Fig. C1 – Post Development Drainage Plan
- Table C.1 - Orifice Flow Calculations
- Table C.2 - On-site Storage Calculator



PHASE 1

	COMPONENT		Total	RUNOFF COEF.	AVG. RUNOFF COEF.
	ROOF	=	0.3505	0.90	0.3079
	LANDSCAPE	=	0.1052	0.25	0.0263
	HARD SURFACE	=	0.1833	0.90	0.1650
	TOTAL	=	0.6390 ha		0.79

PHASE 1 DRAINAGE BOUNDARY

PROPERTY LINE

OVERLAND FLOW

MUNICIPALITY:
MISSISSAUGA
Transportation and Works

CONSULTANT
MASONGSONG ASSOCIATES
7800 KENNEDY ROAD
SUITE 201
MARKHAM, ONTARIO
L3R 2C7
T: (905) 944-0162
www.maeng.ca

PROJECT
STORM DRAINAGE COMPONENTS
"PERLA" - BLOCK 2, PHASE 1

Scale	N.T.S.
Project No.	2016-803
Date	APRIL 2016
Figure No.	Fig. C1

Table C.1 - Orifice Flow Calculations

$$Q = CA(2 \times g \times h)^{0.5}$$

d=	0.075	m	Restrictor Diameter
h=	1.2625	m	Water Surface Height
a=	0.0044	m2	Pipe Cross-section Area
q=	0.0180	18.03	Total Flow
	m3/s	L/S	
Invert Elevation	166.40	m	Pipe Outlet - Control
HWL Elevation	167.70	m	
Top of Tank	167.90	m	

where C = 0.82
A = π d2 /4
g = 9.81 m/s2

and h = Ponding Elevation-(invert elevation orifice+d/2)

Centre of orifice = d/2

Table C.2

On-Site Storage Calculator

MISSISSAUGA 10 -Year

Project: Perla Condominium
Project No.: 16-803
By: K.L.
Date: 23-Jun-17

Location: **Phase 1 Site**

A = 0.6390 ha
Composite C = 0.790
Q_{ACTUAL} = 0.0180 m^3/s

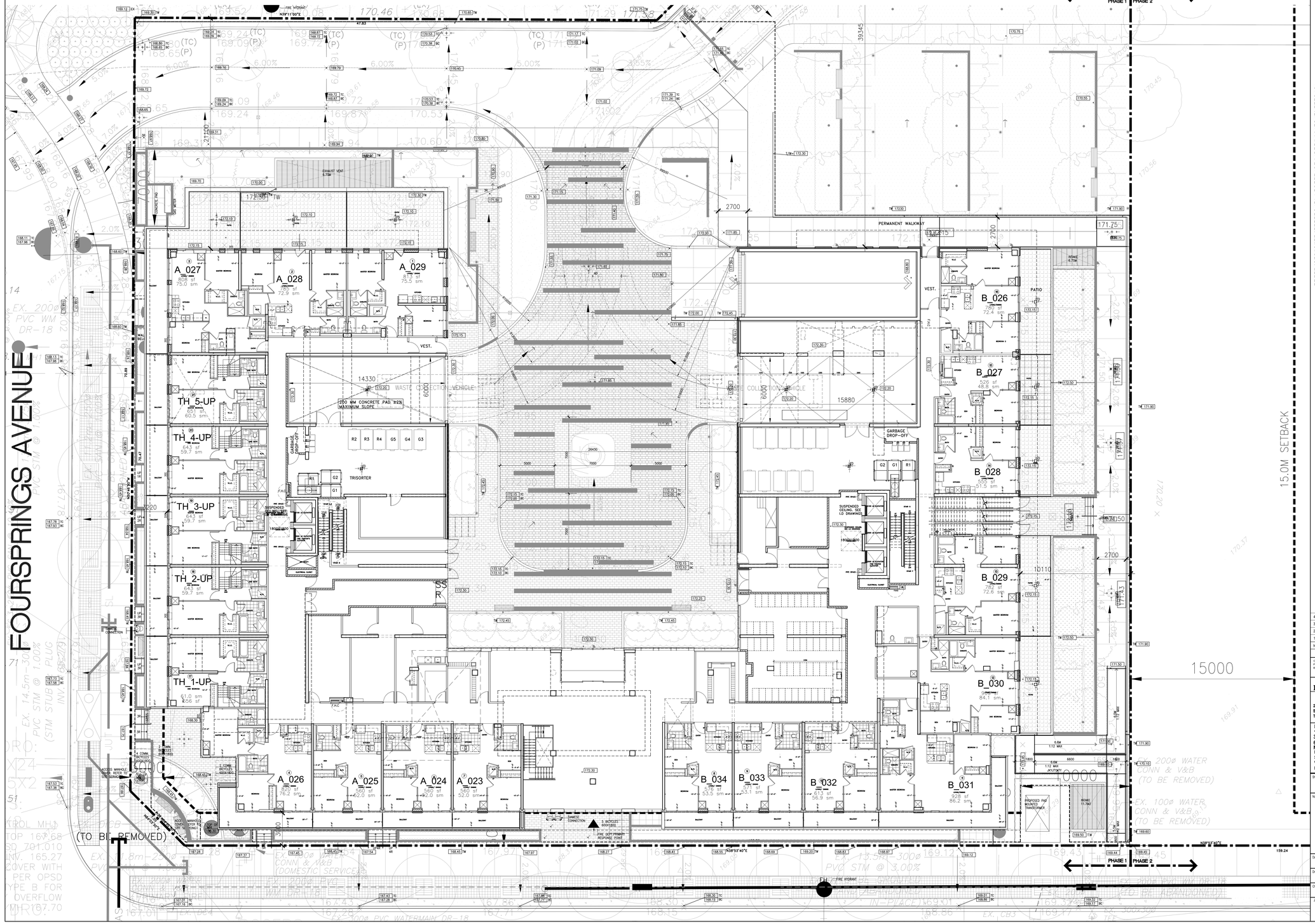
$$i_{10} = \frac{1010}{(t_c + 4.6)^{0.78}}$$

t_c (min)	i_{10} (mm/hr)	Q_{10} (m^3/s)	Q_{stored} (m^3/s)	Peak Volume (m^3)
15	99.166	0.139	0.121	108.950
16	95.390	0.134	0.116	111.131
17	91.928	0.129	0.111	113.124
18	88.739	0.124	0.106	114.950
19	85.793	0.120	0.102	116.625
20	83.060	0.116	0.098	118.165
21	80.518	0.113	0.095	119.582
22	78.147	0.110	0.092	120.888
23	75.930	0.106	0.088	122.092
24	73.851	0.104	0.086	123.203
25	71.898	0.101	0.083	124.227
26	70.058	0.098	0.080	125.173
27	68.323	0.096	0.078	126.045
28	66.682	0.094	0.076	126.849
29	65.129	0.091	0.073	127.590
30	63.656	0.089	0.071	128.272
31	62.257	0.087	0.069	128.899
32	60.926	0.085	0.067	129.474
33	59.659	0.084	0.066	130.000
34	58.450	0.082	0.064	130.481
35	57.295	0.080	0.062	130.919
36	56.192	0.079	0.061	131.316
37	55.135	0.077	0.059	131.675
38	54.123	0.076	0.058	131.998
39	53.152	0.075	0.057	132.287
40	52.220	0.073	0.055	132.543
41	51.325	0.072	0.054	132.767
42	50.464	0.071	0.053	132.963
43	49.635	0.070	0.052	133.130
44	48.837	0.068	0.050	133.270
45	48.067	0.067	0.049	133.385
46	47.324	0.066	0.048	133.475
47	46.607	0.065	0.047	133.542
48	45.915	0.064	0.046	133.586
49	45.245	0.063	0.045	133.609
50	**44.598**	**0.063**	**0.045**	**133.611**
51	43.971	0.062	0.044	133.593
52	43.364	0.061	0.043	133.556

Appendix D

Appendix Drawings:

- A206 – Architectural Site Plan (Phase 1)
- SS-1 – Site Servicing Plan (Phase 1)
- GR-1 – Site Grading Plan (Phase 1)
- ESC-1 – Erosion & Sediment Control Plan (Phase 1)



GENERAL NOTES

SITE PLAN APPLICATION NUMBER(SP 17135)

NOTES:

THE REGION OF PEEL WILL PROVIDE FRONT-END COLLECTION OF GARBAGE AND RECYCLABLE MATERIALS SUBJECT TO THE FOLLOWING CONDITIONS: WASTE COLLECTION VEHICLE ACCESS ROUTE COMMENTS: THE WASTE COLLECTION VEHICLE ACCESS ROUTE, INCLUDING ALL ROADWAYS, DRIVEWAYS, TURNING AREAS AND TURNING MOVEMENTS IS TO BE CLEARLY LABELLED ON THE DRAWING, INCLUDING ENTRANCE TO THE DEVELOPMENT. THE TURNING RADIUS FROM THE CENTRE LINE MUST BE A MINIMUM OF 13 METRES ON ALL TURNS. A MINIMUM 18 METRE STRAIGHT APPROACH IS REQUIRED TO THE FIRST COLLECTION POINT. THIS APPROACH IS TO BE LEVEL (+/- 2%[2]) AND THE SAME WIDTH AS THE COLLECTION POINT. IF THE WASTE COLLECTION VEHICLE IS REQUIRED TO DRIVE OVER OR OVER A SUPPORTED STRUCTURE (SUCH AS AN AIR GRATE, TRANSFORMER COVER, OR UNDERGROUND PARKING GARAGE) THE REGION MUST BE PROVIDED WITH A LETTER FROM A PROFESSIONAL ENGINEER CONFIRMING THAT THE STRUCTURE CAN SUPPORT A COLLECTION VEHICLE WASTE COLLECTION VEHICLE WEIGHING 35 TONNES. GARBAGE AND RECYCLING RECEPTACLE REQUIREMENTS: THE LOCATION OF THE COMPACTOR (IF PRESENT) MUST BE SHOWN.THE FOLLOWING TABLE CAN BE USED AS A GUIDELINE WHEN CALCULATING THE NUMBER OF FRONT-END GARBAGE AND RECYCLING BINS REQUIRED. THE NUMBER OF BINS PROVIDED ON THE PLANS MUST BE CLEARLY LABELLED AND SHOWN ON THE SITE PLAN. COMPACTED GARBAGE BIN: 3YD3 (54 UNITS/BIN) 4YD3 (72 UNITS/BIN) UNCOMPACTED GARBAGE BIN: 3YD3 (18 UNITS/BIN) 4YD3 (24 UNITS/BIN) 8YD3 (36 UNITS/BIN). RECYCLING BIN: 3YD3 (45 UNITS/BIN) 4YD3 (60 UNITS/BIN) 8YD3 (90 UNITS/BIN). PLEASE NOTE THAT RECYCLABLE MATERIALS ARE NOT TO BE COMPACTED. OUTDOOR WASTE COLLECTION POINT COMMENTS: FOR WASTE COLLECTED OUTDOORS, A COLLECTION POINT IS REQUIRED FOR EACH BUILDING. ONE COLLECTION POINT PAD IS REQUIRED. SEE APPENDIX 4 OF THE WASTE COLLECTION DESIGN STANDARDS MANUAL FOR FRONT-END BIN DIMENSIONS. FOR 1 CUBIC YARD FRONT-END BIN, THE MINIMUM WIDTH REQUIRED IS 3 METRES FOR EVERY FRONT-END BIN PRESENT, PLUS 0.6 METRES FOR EACH SIDE WALL. FOR A 1.5 CUBIC YARD FRONT-END BINS, A MINIMUM DEPTH OF 3 METRES IS REQUIRED. FOR A 6 METRE WIDE COLLECTION POINT, FRONT-END BINS SHALL BE PLACED IN SINGLE FILE ON ONE SIDE OF THE COLLECTION POINT AND THE LENGTH OF THE COLLECTION POINT SHALL BE ADEQUATE TO ACCOMMODATE THE REQUIRED NUMBER OF FRONT-END BINS. THE CONCRETE PAD SHOULD EXTEND A MINIMUM OF 1.5 METRES BEYOND THE FRONT OF THE COLLECTION POINT TO ACCOMMODATE THE FRONT WHEELS OF THE WASTE COLLECTION VEHICLE. SEE APPENDIX 4 FOR AN ILLUSTRATION OF THESE REQUIREMENTS. FOR MORE INFORMATION, PLEASE CONSULT THE WASTE COLLECTION DESIGN STANDARDS MANUAL, AVAILABLE HTTP://WWW.PEELREGION.CA/PW/STANDARDS/DESIGN/WASTE-COLLECTION-DESIGN-MANUAL-2016.PDF

NO	DATE	REMARKS
4	28 JUN 2017	RE-SUBMISSION FOR SPA
3	8 FEB 2017	ISSUED FOR SPA
2	13 SEP 2016	ISSUED FOR OPA/ REZONING
1	7 JLY 2016	ISSUED FOR DARC

REVISIONS

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME.REPORTING ANY DISCREPANCIES TO THE ARCHITECT BEFORE COMMENCING THE WORK.

PRINTS ARE NOT TO BE SCALED.

RICHMOND | Architects Ltd.

243 COLLEGE STREET
SECOND FLOOR
TORONTO, ONTARIO M5T 1R5

t. 416 961 1567
f. 416 961 1321
RICHMONDARCH.COM

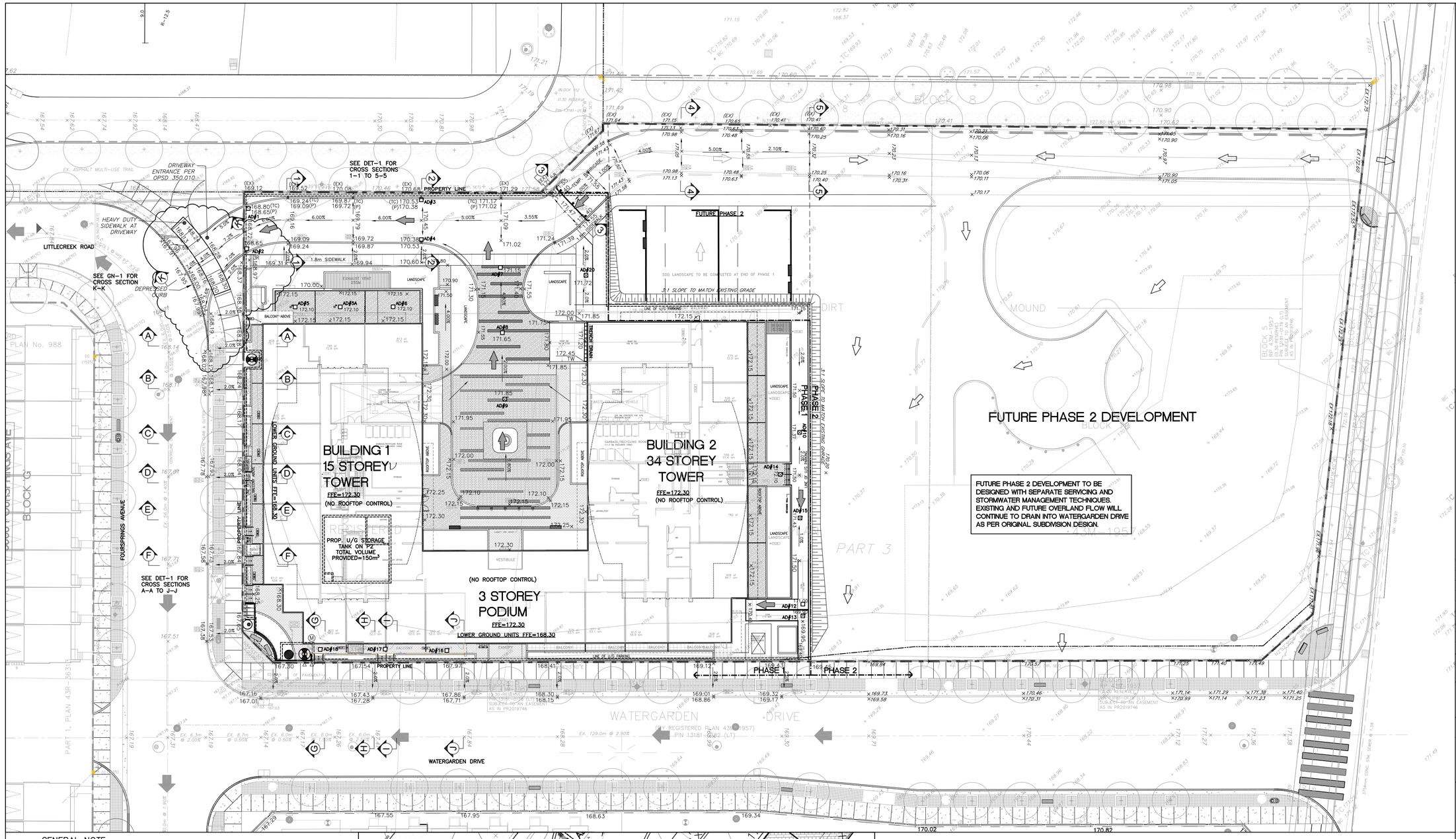
PROPOSED CONDOMINIUM DEVELOPMENT
'PERLA'
35 WATERGARDEN DRIVE
65 WATERGARDEN DRIVE
MISSISSAUGA, ONTARIO
for Pinnacle International (Ontario) Limited
Suite 300- 911 Homer Street
Vancouver, BC. V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

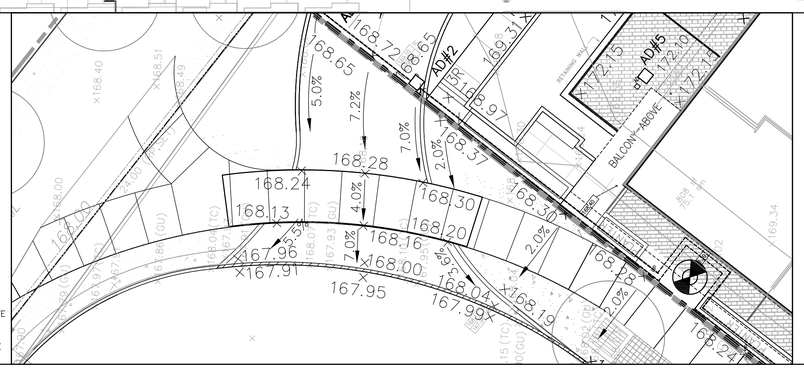
Sheet No.
GROUND FLOOR PLAN

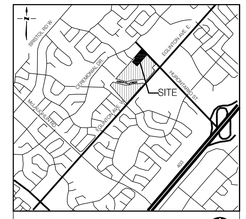
North

Issd

Date Plotted:	11 JAN 2018
Date:	29 SEP 2015
Drawn By:	SD
Scale:	1:100
Checked By:	
Revision Number:	4
Job No.:	2015-020
Sheet No.:	SP A301







KEYPLAN N.T.S.

LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

DRAFT PLAN OF SUBDIVISION
SEPTEMBER 25, 2013
&
PLAN OF TOPOGRAPHY
OCTOBER 29, 2007

PART OF LOT 1, CONCESSION 1
WEST OF HURONTARIO STREET
(ORIGINALLY IN TOWNSHIP OF TORONTO)

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

TARASICK McMILLAN KUBICKI LIMITED
ONTARIO LAND SURVEYORS
4181 SLADEVIEW CRESCENT, UNIT 42, MISSISSAUGA, ONTARIO L5L 5R2
TEL: (905) 569-8649 FAX: (905) 569-3160
E-MAIL: office@tmksurveyors.com

ELEVATIONS ARE REFERRED TO CANADIAN GEODETIC VERTICAL DATUM-1928, AND WERE DERIVED FROM CITY OF MISSISSAUGA BENCHMARK No. 218, ELEVATION 177.91 metres.

NO	DATE	REMARKS
7	JAN 04/18	PER CITY COMMENTS
6	NOV 15/17	PER REGION COMMENTS
5	SEPT 05/17	ISSUED FOR BUILDING PERMIT
4	JUN 26/17	ISSUED FOR OPA/RE-ZONING
3	MAY 19/17	ISSUED FOR BUILDING PERMIT
2	FEB. 08/17	ISSUED FOR OPA/RE-ZONING
1	SEPT. 15/16	ISSUED FOR OPA/RE-ZONING

REVISIONS

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME REPORTING ANY DISCREPANCIES TO THE ARCHITECT BEFORE COMMENCING THE WORK.

PRINTS ARE NOT TO BE SCALED.

Consultant:
MASONGSONG ASSOCIATES
7501 KENNEDY ROAD SUITE 300 MARKHAM, ONTARIO L3R 2C7
T: (905) 944-0162
www.msaeng.ca

RICHMOND | Architects Ltd.
243 COLLEGE STREET
SECOND FLOOR
TORONTO, ONTARIO M5T 1R5
T: 416 961 1567
F: 416 961 1321
RICHMONDARCH.COM

PROPOSED CONDOMINIUM DEVELOPMENT
'PERLA' (BLOCK 2, PHASE 1)
35 & 65 WATERGARDEN DRIVE, MISSISSAUGA

Mondiale Development Ltd.
Suite 300- 911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

Sheet Title
SITE GRADING PLAN-PHASE 1

North

Seal: Professional Engineers Ontario — Limited Licensee — Name: R. K. LO — License No.: 100166 — Category: CONL — Limitations: This licence is subject to the terms and conditions set forth on the certificate. — Association of Professional Engineers of Ontario — JAN. 04/18

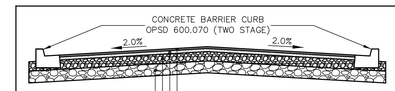
Date Plotted:
Date: 16 APRIL 2016
Drawn By:
Scale: 1: 300
Checked By:
Revision Number:
Job No: 16803
Sheet No: GR-1

C-600206

GENERAL NOTE:
REFER TO DRAWING GN-1 FOR ALL NOTES.

LEGEND :

- EXISTING HYDRANT
- EXISTING VALVE AND BOX
- EXISTING DETECTOR CHECK VALVE IN CHAMBER
- EXISTING CATCHBASIN
- EXISTING DOUBLE CATCHBASIN
- EXISTING SANITARY MANHOLE
- EXISTING STORM MANHOLE
- PROPOSED SANITARY MANHOLE
- PROPOSED STORM MANHOLE
- PROPOSED AREA DRAIN
- PROPOSED HYDRANT & VALVE
- PROPOSED VALVE AND BOX
- WATERMETER
- PROPERTY LIMIT
- DC — DEPRESSED CURB
- 199.17 — EXISTING ELEVATION
- x178.10 — PROPOSED ELEVATION
- 170.63× — PROPOSED FUTURE GRADE
- 172.50× — SUBDIVISION DESIGN GRADE TO MATCH
- FLOW DIRECTION
- OVERLAND FLOW ROUTE
- EXISTING OVERLAND FLOW ROUTE
- FUTURE OVERLAND FLOW ROUTE



NOTE: PARKING LOTS SHALL BE STRUCTURALLY DESIGNED TO THE EQUIVALENT OF THE INTERNAL ROAD DESIGN STANDARDS.

MINIMUM PAVEMENT STRUCTURE FOR INTERNAL PRIVATE ROADWAYS

NT.S.

CITY OF MISSISSAUGA
SITE GRADING CERTIFICATION

I have reviewed the plans for the construction of a PROPOSED CONDOMINIUM DEVELOPMENT (PERLA-PHASE 1) located at FOUR SPRING AVENUE AND WATERGARDEN DRIVE, and have prepared this plan to indicate the compatibility of the proposal to existing properties and municipal services. It is my belief that adherence to the proposed grades as shown will produce adequate surface drainage and proper facility of the municipal services without any detrimental effect to the existing drainage patterns or adjacent properties.

MASONGSONG ASSOCIATES ENGINEERING LIMITED
Consulting Engineers • Planners • Project Managers

REGION OF PEEL NOTES (WATERMAIN)

a. All materials and construction methods must correspond to the current Peel Public Works standards and specifications.

b. Watermain and/or water service materials 100 mm (4") and larger must be PVC DR-18 AWWA C900-16. Size 50 mm (2") and and smaller must be copper.

c. Watermains and/or water services are to have a minimum cover of 1.7 m (5'6") with a minimum horizontal spacing of 1.2 m (4') from themselves and all other utilities.

d. Provisions for flushing water line prior to testing, etc., must be provided with a least a 50 mm (2") outlet on 100 mm (4") and larger lines. Copper lines are to have flushing points at the end, the same same as the line. They must also be hosed or piped to allow the water to drain onto a parking lot or down a drain. On fire lines, flashing outlet to be 100 mm (4") diameter minimum on a hydrant.

e. All curb stops to be 3.0 m (10') off the face of the building unless otherwise noted.

f. Hydrant and valve set to Region standard 1-6-1 Dimension A and B, 0.7 m (2') and 0.90 m (3') and to have pumper nozzle.

g. Watermains to be installed to grades as shown on approved site plan. Copy of grade sheet must be supplied to inspector prior to commencement of work, where requested by inspector.

h. Watermains must have a minimum vertical clearance of 0.30 m (12") over / 0.5 m (20") under sewers and all other utilities when crossing.

i. All proposed water piping must be isolated from existing lines in order to allow independent pressure testing and chlorinating from existing systems.

j. All live tapping and operation of Region water valves shall be arranged through the Regional inspector assigned or by contacting the Operations and Maintenance Division.

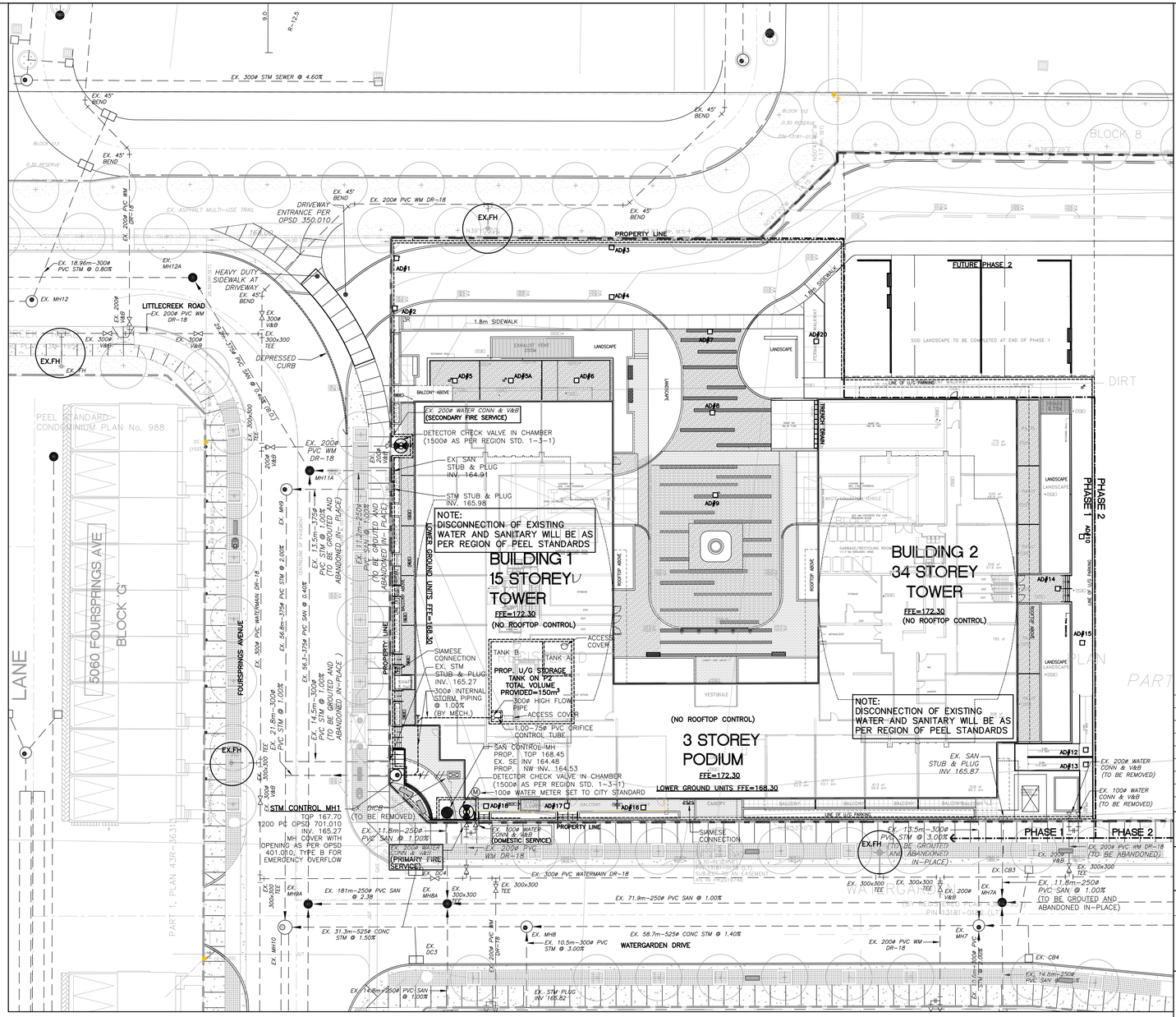
ADDITION NOTES:

1. LOCATION OF ALL EXISTING UTILITIES IN THE FIELD TO BE ESTABLISHED BY THE CONTRACTOR.
2. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE FOR LOCATES, EXPOSING, SUPPORTING AND PROTECTING OF ALL UNDERGROUND AND OVERHEAD UTILITIES AND STRUCTURES EXISTING AT THE TIME OF CONSTRUCTION IN THE AREA OF HIS WORK. WHETHER SHOWN ON THE PLANS OR NOT, AND FOR ALL REPAIRS AND CONSEQUENCES RESULTING FROM DAMAGE TO SAME.
3. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE TO GIVE 72 HOURS WRITTEN NOTICE TO UTILITIES PRIOR TO CROSSING SUCH UTILITIES, FOR THE PURPOSE OF INSPECTION BY THE CONCERNED UTILITY. THIS INSPECTION WILL BE FOR THE DURATION OF THE CONSTRUCTION, WITH THE CONTRACTOR RESPONSIBLE FOR ALL COSTS ARISING FROM SUCH INSPECTION.
4. ALL PROPOSED WATER PIPING MUST BE ISOLATED THROUGH A TEMPORARY CONNECTION THAT SHALL INCLUDE AN APPROPRIATE CROSS-SECTION CONTROL DEVICE, CONSISTENT WITH THE DEGREE OF HAZARD, FOR BACKFLOW PREVENTER OF THE ACTIVE DISTRIBUTION SYSTEM, CONFORMING TO REGION OF PEEL STANDARDS 1-7-7 OR 1-7-8.

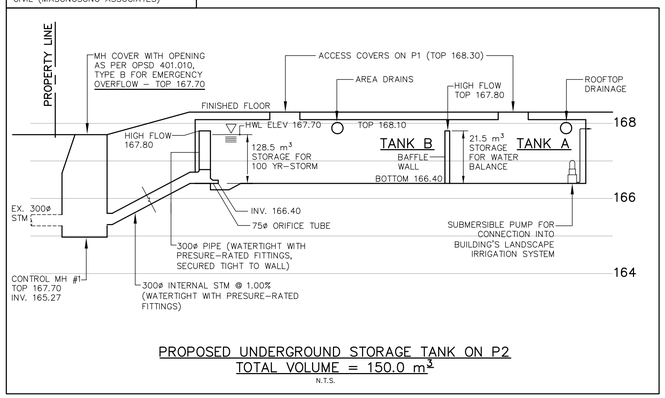
GENERAL NOTE:
REFER TO DRAWING GN-1 FOR ALL NOTES.

LEGEND :

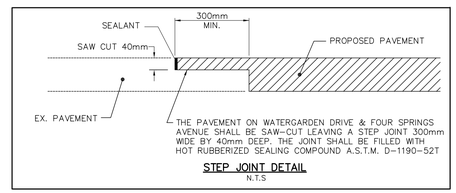
- EXISTING HYDRANT
- EXISTING VALVE AND BOX
- EXISTING CATCHBASIN
- EXISTING DOUBLE CATCHBASIN
- EXISTING SANITARY MANHOLE
- EXISTING STORM MANHOLE
- PROPOSED SANITARY MANHOLE
- PROPOSED STORM MANHOLE
- PROPOSED AREA DRAIN
- PROPOSED HYDRANT & VALVE
- PROPOSED VALVE AND BOX
- WATERMETER LOCATION
- PROPERTY LIMIT
- DC DEPRESSED CURB
- DETECTOR CHECK VALVE IN CHAMBER



NOTE:
PROPOSED U/G STORAGE TANK TO BE DESIGNED BY MECHANICAL AND STRUCTURAL AT BUILDING DESIGN STAGE AND COORDINATED WITH ARCHITECT AND CIVIL (MASONGSONG ASSOCIATES)



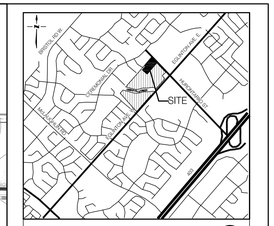
PROPOSED UNDERGROUND STORAGE TANK ON P2
TOTAL VOLUME = 150.0 m³
N.T.S.



STEP JOINT DETAIL
N.T.S

ROAD RESTORATION NOTES:

1. CONTRACTOR TO MAINTAIN MINIMUM 1-LANE OF TRAFFIC OPEN AT ALL TIMES.
2. ALL TRENCHES WITHIN MUNICIPAL RIGHT OF WAY, TO BE RESTORED WITH NON-SHRINK BACKFILL.
3. EXISTING ASPHALT TO BE MATCHED WITH A 40mm DEPTH STEP JOINT, AND ALL CRACKS TO BE SEALED WITH HOT RUBBERIZED SEALANT.
4. ALL MUNICIPAL BOULEVARDS TO BE RESTORED WITH 100mm TOPSOIL AND No.1 GRADE NURSERY SOD.



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CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

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Sheet Title
SITE SERVICING PLAN

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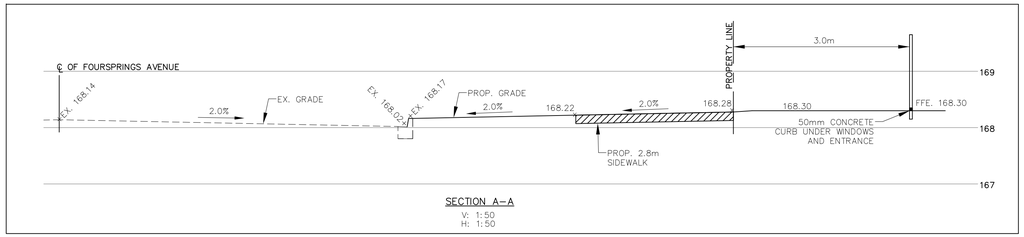
Drawn By:	Scale: 1: 300
Checked By:	Revision Number:
Job No.: 16803	Sheet No: SS-1

C-600206



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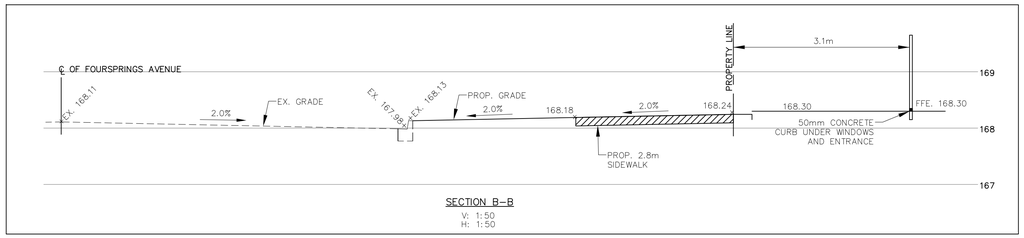

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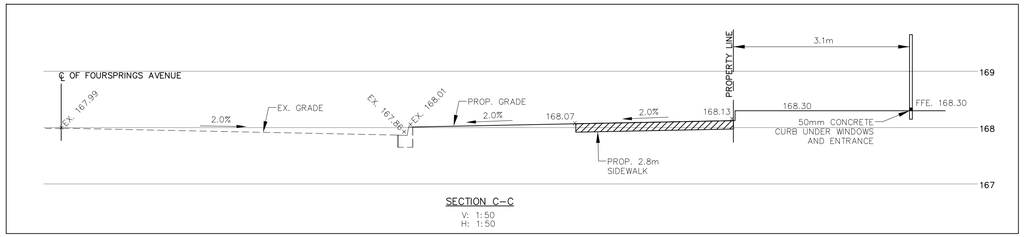

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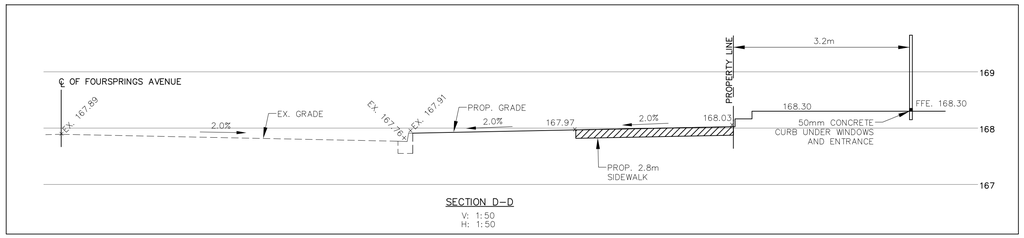

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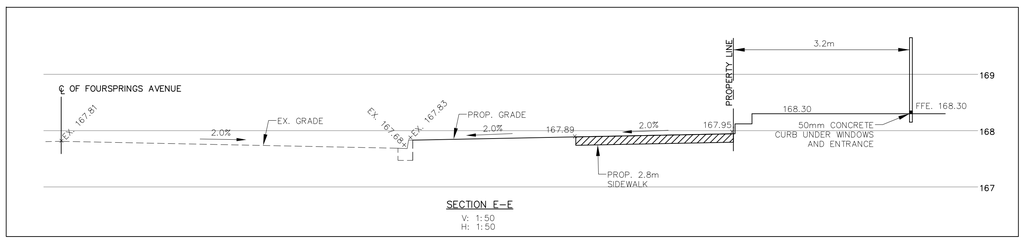

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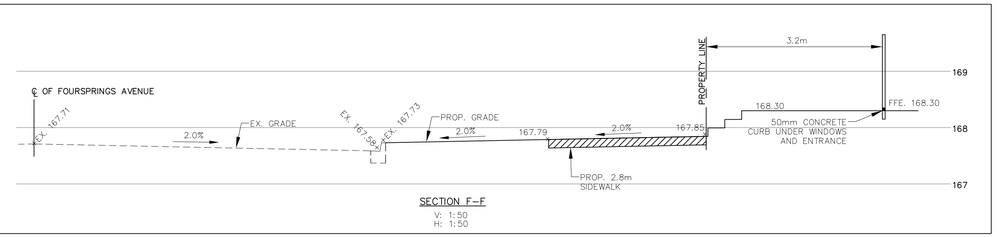

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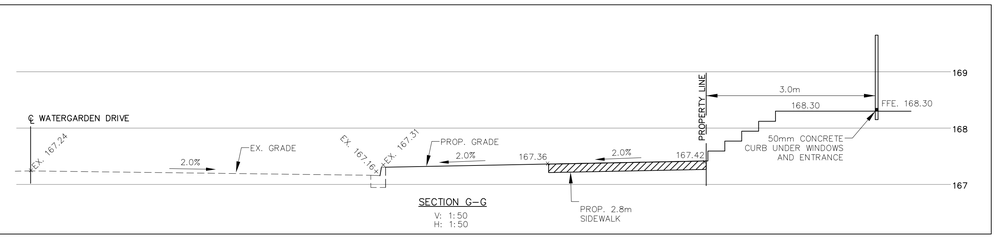

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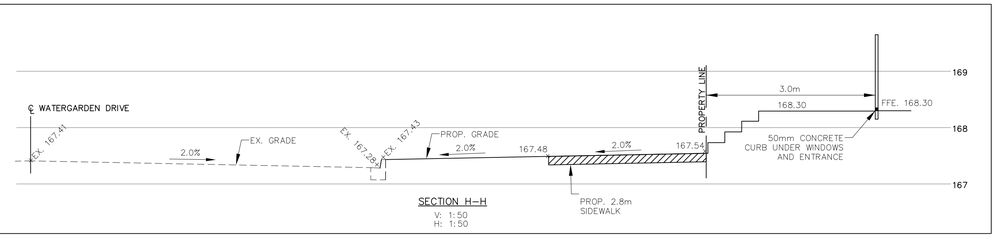

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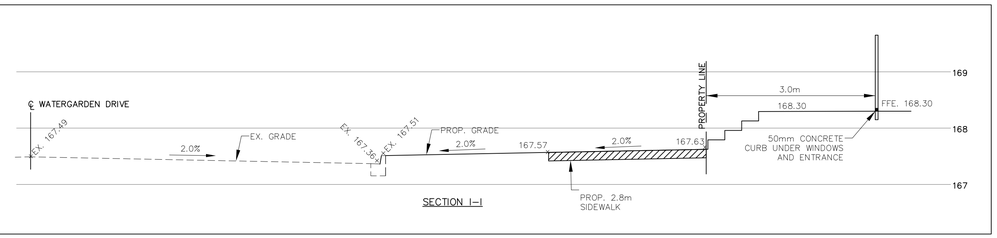

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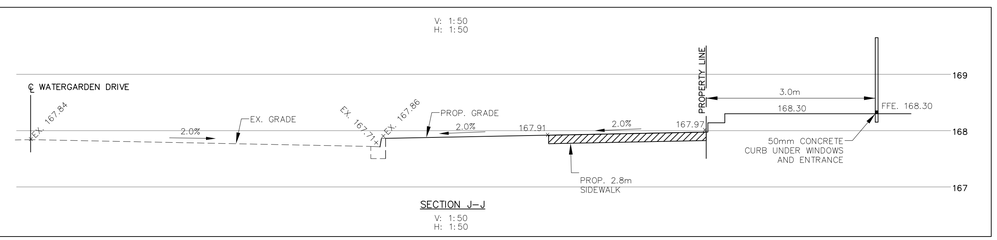

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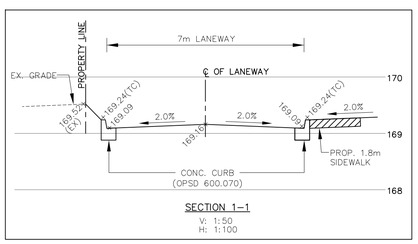

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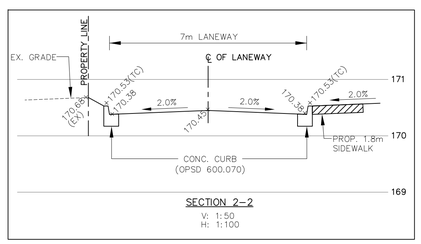

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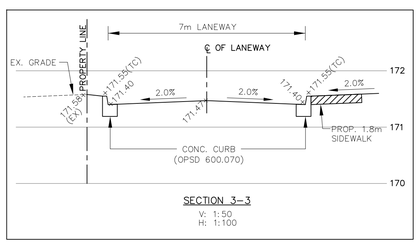

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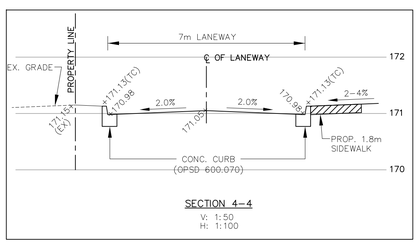

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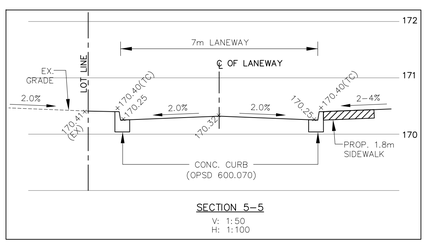

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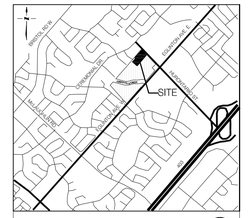

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CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

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ONTARIO LAND SURVEYORS
4181 SLADEVIEW CRESCENT, UNIT 42, MISSISSAUGA, ONTARIO L5L 5R2
TEL: (905) 569–8649 FAX: (905) 569–3160
E–MAIL: office@tmksurveyors.com

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MASONGSONG ASSOCIATES
7800 KENNEDY ROAD
SUITE 201
MARKHAM, ONTARIO
L3R 2C7
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243 COLLEGE STREET
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Sheet Title
DETAILS

North

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Limited Licensee
Name: K. S. LO
Number: 100207166
Category: CIVIL
Limitations: N/A
This licence is authorized by the practice of engineering on the corporation
Association of Professional Engineers of Ontario
JAN. 04/18

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Checked By :	Revision Number:
Job No : 16803	Sheet No: DET-1

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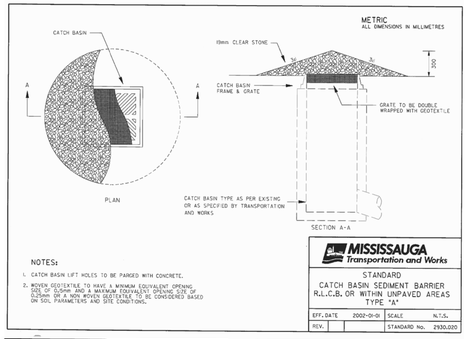

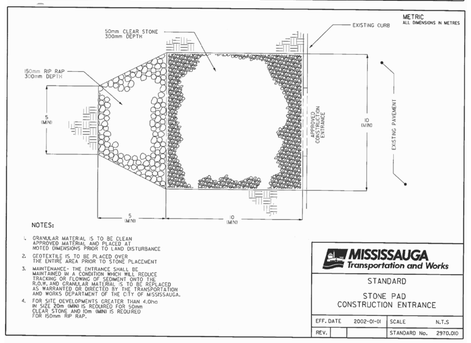

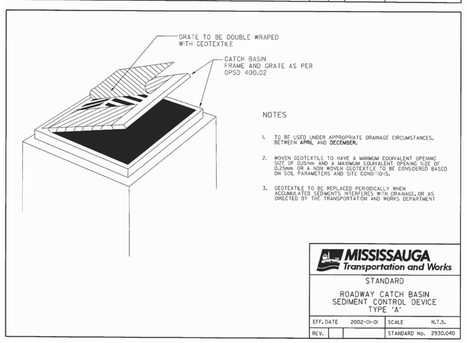

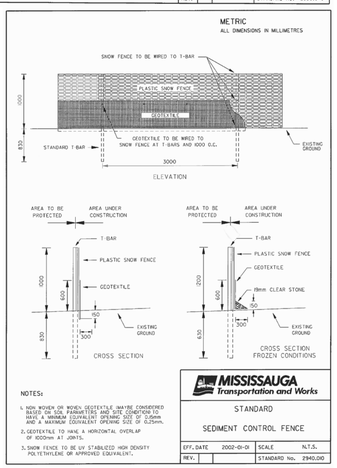

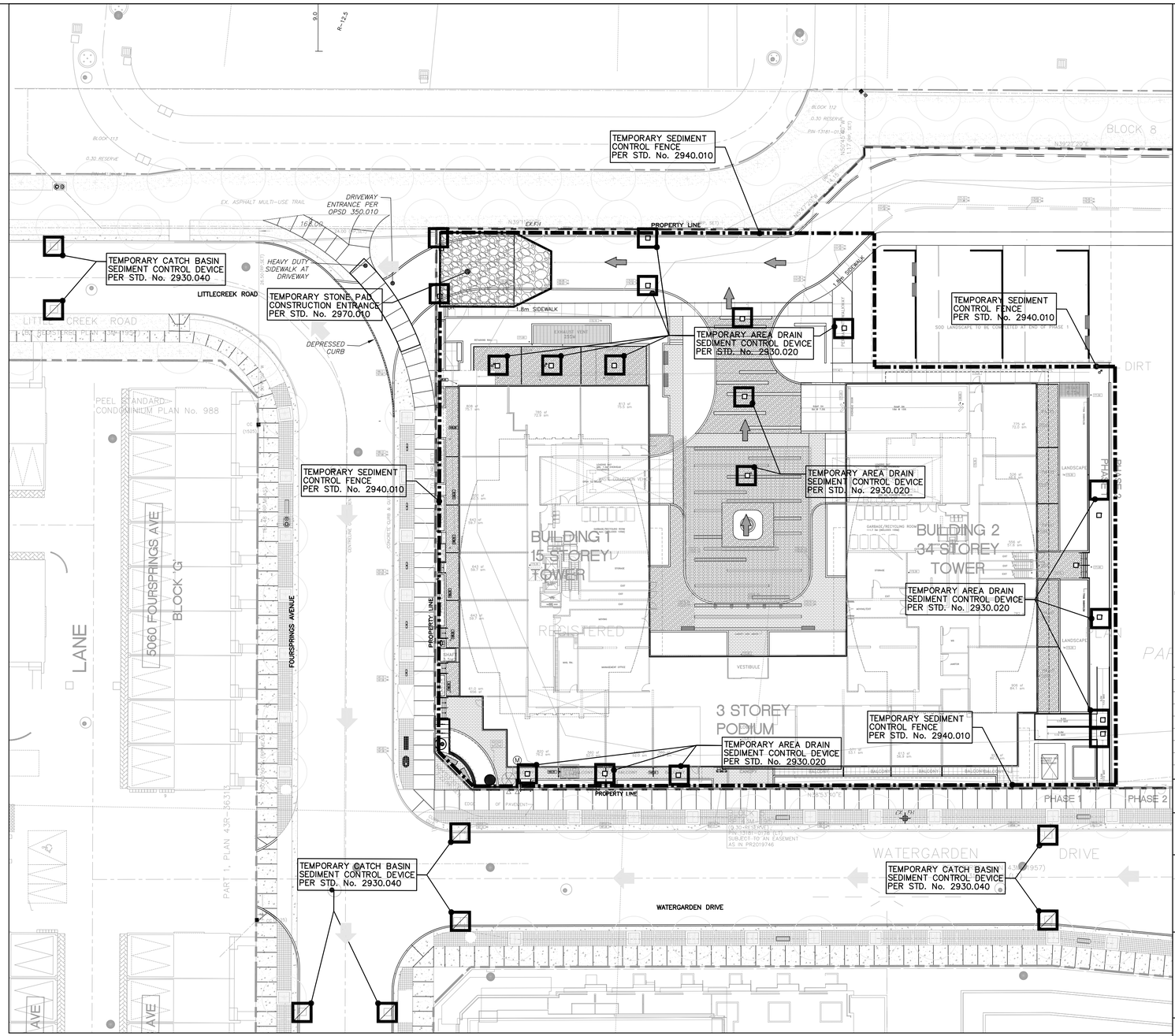

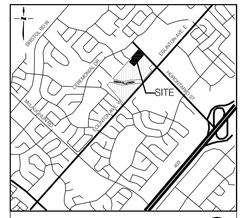

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LEGEND :

- EXISTING CATCHBASIN
- EXISTING DOUBLE CATCHBASIN
- PROPOSED AREA DRAIN
- PROPOSED OVERLAND FLOW ROUTE
- EXISTING OVERLAND FLOW ROUTE
- MUD MAT (PER STD. No. 2970.010)
- SEDIMENT CONTROL FENCE (PER STD. No. 2940.010)
- PROPERTY LIMIT
- AREA DRAIN SEDIMENT BARRIER (PER STD. No. 2930.020) (TYP. ON ALL PROPOSED ON-SITE AREA DRAIN)
- CATCH BASIN SEDIMENT CONTROL DEVICE (STD. No. 2930.040) (TYP. ON ALL EX. ROAD CB AND DCB)

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Sheet Title: EROSION & SEDIMENT CONTROL PLAN-PHASE 1

North

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Date Plotted	Date: 16 APRIL 2016
Drawn By:	Scale: 1: 300
Checked By:	Revision Number:
Job No: 16803	Sheet No: ESC-1

SEDIMENT CONTROL NOTES

a) ALL SEDIMENT CONTROL FENCING IS TO BE ERECTED PRIOR TO THE COMMENCEMENT OF ANY SITE GRADING OPERATIONS, AS PER CITY OF MISSISSAUGA STANDARD 2940.010

b) ALL AREA DRAINS TO HAVE SEDIMENT BARRIER (CITY OF MISSISSAUGA STANDARD 2930.020) ERECTED IMMEDIATELY AFTER AREA DRAIN INSTALLATION. SEDIMENT PROTECTION BARRIER TO BE MAINTAINED ON A REGULAR BASIS OR TO THE SATISFACTION OF THE CITY OF MISSISSAUGA.

c) ALL ROADSIDE EXISTING CATCHBASINS WITHIN PROXIMITY OF SITE TO HAVE SEDIMENT PROTECTION AS PER CITY OF MISSISSAUGA STANDARD 2930.040 INSTALLED PRIOR TO START OF SITE ALTERATION. SEDIMENT PROTECTION BARRIER TO BE MAINTAINED ON A REGULAR BASIS OR TO THE SATISFACTION OF THE CITY MISSISSAUGA.

d) CONSTRUCTION SEQUENCE:
INITIAL SEDIMENT CONTROL INSTALLATION
SITE GRADING OPERATIONS
UNDERGROUND SERVICING OPERATIONS
BUILDING CONSTRUCTION
FINAL GRADING OPERATIONS

e) IF SITE CONSTRUCTION ACTIVITIES ARE INTERRUPTED AND/OR INACTIVITY EXCEEDS 30 DAYS, ALL STRIPPED AND/OR BARE SOIL AREAS ARE TO BE STABILIZED BY SODDING/SEEDING/MULCHING OR OTHER APPROVED METHOD, TO THE SATISFACTION OF THE CITY OF MISSISSAUGA.

f) THIS CONTROL PLAN IS PREPARED FOR SUBMISSION TO THE CITY OF MISSISSAUGA IN CONJUNCTION WITH AN APPLICATION FOR EROSION & SEDIMENT CONTROL PERMIT NO._____ UNDER THE EROSION & SEDIMENT CONTROL BY-LAW NO 512-91, AS AMENDED.

g) ALL EROSION AND SEDIMENT CONTROL MEASURE ARE TO BE INSPECTED WEEKLY AND AFTER EVERY RAINFALL AND MAINTAINED IMMEDIATELY, AS REQUIRED, TO THE SATISFACTION OF THE CITY OF MISSISSAUGA.

h) DURING ALL CONSTRUCTION PHASES, MUD TRACKING CONTROL, CONSISTING OF FLUSHING AND SWEEPING ROADS, IS TO BE PROVIDED FOR ALL ROADS, AS WARRANTED, IN ACCORDANCE WITH THE CITY OF MISSISSAUGA MUD TRACKING CONTROL POLICY.

C-600206

GENERAL NOTES

LEGAL DESCRIPTION OF LANDS

Part of Lot 1, Concession 1, West of Horuntario Street.
Registered Plan 43M-1523, City of Mississauga, Regional Municipality of Peel.

FIRE DEPARTMENT

a. Fire routes shall be designated as per By-Law 1036-81 as amended, prior to occupancy of the building.

b. All fire access routes shall be constructed of hard surface materials such as asphalt, concrete or lockstone, and designed to support a load of not less than 11,363 kg per axle and have a change in gradient of not more than 1 in 12.5 (8%) over a minimum distance of 15m.

PLANNING AND BUILDING DEPARTMENT, DESIGN DIVISION

a. Any retaining wall over 0.60m to be designed and certified by a Structural Engineer, and shown on all grading and servicing drawings.

b. Grades at the property line are to be met with a maximum 3:1 (33%) slope within the site.

c. Continuous 150mm high (min) concrete barrier type curbs to be provided between all new asphalt and landscaped areas throughout the site, with depressed curbs at the ends of all pedestrian walkways.

d. All roof-top mechanical units to be screened from view.

TRANSPORTATION & WORKS DEPARTMENT, DEVELOPMENT ENGINEERING

a. All surface drainage will be self contained, collected and discharged at a location to be approved prior to issuance of a building permit.

b. The portions of the driveway within the municipal boulevard will be paved by the applicant

c. At the entrances to the site, the municipal curb will be continuous through the driveway and a curb depression will be provided for each entrance.

d. The tops of any curbs bordering the driveways within the municipal boulevard will be flush with the municipal road curb and sidewalk.

1. GENERAL

a. All site layout information, including building dimensions, setbacks, curbs, sidewalks and landscape features must be referenced from the Architect's plans.

b. Prior to the start of construction, the contractor must verify all dimension and layout information. Any discrepancies must be reported to the Consultant before resuming construction operations.

c. All services shall be installed to the current City of Mississauga Standards (City Std.), Region of Peel (Peel Std.), Ontario Provincial Standard Drawings (OPSD), Ontario Provincial Standard Specifications (OPSS), and Ontario Building Code (OBC) unless otherwise specified, to the satisfaction of the City and the Consultant.

d. The position of existing pole lines, conduits, watermains, sewers and other underground and aboveground utilities, structures and appurtenances is not necessarily shown on the contract drawing, and where shown, the accuracy of the position of such utilities and structures is not guaranteed. Prior to construction, the Contractor shall satisfy himself of the exact location of all such utilities and structures, and shall assume all liability for damage to them during the course of construction.

e. All trenching to be in accordance with the latest revisions of the Occupational Health and Safety Act and Regulations for Construction Projects.

f. All trenches shall be backfilled to the City of Mississauga Standards (Section 4.02.06) and in accordance with on-site recommendations of the geotechnical consultant.

g. All disturbed areas outside proposed grading limits to be restored to original elevations and conditions unless otherwise specified. All restoration shall be completed with the geotechnical requirements for backfill, compaction and approved engineering drawings.

h. All dimensions and elevations in metres, pipe sizes in millimetres.

2. GRADING

a. Grade all areas to provide positive drainage to catchbasins or overland outlets. All flows to be directed away from any building or other structure.

b. All grading adjacent to buildings or structures to be graded with a 0.60m (min.) apron draining away from the structure.

c. Where drainage direction arrows are not shown, grade area to provide positive sheet drainage (no swale) from higher elevations to lower elevations.

d. Minimum swale grades to be 2.0% (desirable), 1.5% absolute minimum. Maximum slope within site is 3:1 (hor:vert).

e. Where the grade difference cannot be met with a 3:1 slope, a retaining wall must be used. For retaining walls greater than 0.60m in height, shop drawings and details of the wall designed and certified by a Professional Engineer must first be submitted to the City and the Grading Consultant for approval.

3. ROADWORKS AND PAVEMENT

a. Native subgrade shall be approved by the Geotechnical Consultant prior to any roadworks.

b. The zone of subgrade 1.0m below the underside of granular road base shall be compacted with the moisture content on the dry side of optimum. The top 1000mm of the sub-grade shall be compacted to a minimum 98% Standard Proctor Density at optimum moisture content and proof-rolled. Unstable areas may require sub-excavation and re-compaction or increased thickness of granular sub-base, as directed by the geotechnical consultant. All fill within roads and parking areas to be compacted to a minimum 95% Standard Proctor Density and the suitability and compaction of all fill material are to be confirmed by a soils consultant, to the city prior to the installation of any road base material. (City of Mississauga Std. 4.02.06, OPSS 514.07.08)

c. The suitability and compaction of all fill materials shall be confirmed by a geotechnical consultant prior to placement of road base.

d. Pavement structure as noted in Architect's plans, and as recommended by the geotechnical consultant.

e. Concrete curb shall be single-stage barrier curb as per Architect's plans.

4. WATERMAINS

a. All watermain, appurtenances and construction methods shall conform to current Region of Peel standards.

b. All watermain 300 mm diameter and smaller to be Polyvinyl Chloride (PVC) Class 150 DR18 meeting AWWA specification C-900.

c. All watermain to be installed with continuous 14 gauge solid copper light-coloured tracer wire along the top of pipe and brought to the surface at every valve box.

d. Fittings to be Cast Iron or Ductile Iron cement-lined in conformance with AWWA C110 and ANSI A21.10, or PVC in conformance with AWWA C900 Class 150.

e. Where watermains cross over other utilities, a minimum 0.15 m clearance shall be maintained; where watermains cross under other utilities, a minimum 0.30 m clearance shall be maintained, while still maintaining a minimum depth of cover at all times.

f. Watermains shall be installed with a min. 1.7m cover and a min. 1.2m horizontal clearance from themselves and all other utilities.

g. Watermain bedding shall conform to Peel Std. 1-5-1. Watermain support shall conform to Peel Std. 1-5-2.

h. Concrete thrust blocks are to be installed at all tees, bends, hydrants, ends of mains and connections 100 mm and larger in accordance with Peel Std. 1-5-3, 4, 5, 6 and 7.

i. Watermain shall be restrained where bedding consists of disturbed native material or fill, and as shown on the drawings. Mechanical restraint shall be obtained by using Uni-Flange bell clamps and retaining glands with threaded rod, or approved equal in conformance with UNI-B-13-92.

j. All curb stops to be 3.0m off the face of building unless otherwise noted.

k. Provisions for flushing water line prior to testing, etc., must be provided with at least a 50mm outlet on 100mm and larger lines. and be hosed or piped to allow the water to drain onto a parking lot or down a drain. On fire lines, flushing outlet to be 100mm diameter minimum on a hydrant.

l. All proposed water piping to be isolated from existing lines in order to allow independent pressure testing and chlorinating from existing systems.

m. Fire hydrants and valves to have Storz pumper nozzle, installed in accordance with Peel Std. 1-6-1 and 1-6-2.

n. Hydrant flange elevations shall be set at a grade that will give a flange elevation of 50 mm to 100 mm above the final grade.

o. Hydrants shall be located a minimum of 1.0 metre from the edge of driveways, roadways, utilities, light-poles or other above-ground obstacles.

p. 150 mm to 300 mm diameter valves shall be resilient seat gate valves installed in round cast iron valve box with inside screw non-rising spindle, complete with mechnanical joint ends.

q. All direct buried flange-to-flange connections to have stainless steel (Passive 304) nuts and bolts.

r. All direct buried valves and fittings to have large size Protecto Caps in accordance with ASTM 418.1, to be installed on alternating bolts. Provide sufficient bolt length to accomodate Protecto Caps.

5. SANITARY SEWERS AND MANHOLES

a. All sanitary sewer and sanitary sewer appurtenances shall conform to the current Region of Peel standards.

b. Sanitary sewer pipe size 250 mm in diameter to be PVC SDR-35 (unless specified otherwise on drawings) with rubber gasket type joints in conformance with CSA B-182.2,3,4.

c. Sewer bedding material to be OPSS HL-6 or 6mm washed chip limestone installed in accordance with Peel Std. 2-3-1 Class 'B'.

d. All manholes to be 1200 mm minimum diameter unless specified otherwise. Precast manholes shall be in accordance with Peel Std. 2-1-1. Frame and cover shall be in accordance with Peel Std. 2-2-2.

e. Precast manholes greater than 5.0 m deep shall be constructed with a safety platform(s) in accordance with Peel Std. 2-2-1.

f. Benching per Peel Std. 2-1-4, unless otherwise specified.

g. Top of manhole cover to be set to base course asphalt grade and then adjusted to final grade with Moduloc rings when top lift of asphalt is placed.

6. STORM SEWERS AND MANHOLES

a. Concrete sewers up to 250 mm in diameter shall conform to CSA A-257.1, ES. Concrete sewers 300 mm in diameter or larger shall conform to CSA 257.2, class as specified on drawings, with rubber gaskets per CSA A-257.3.

b. Sewer bedding material to be OPSS HL-6 or 6mm washed chip limestone installed in accordance with City of Mississauga Std. 2112.08 Class 'B' or as specified by the Geotechnical Consultant. In all cases, the minimum depth of bedding shall be 150 mm. All sewer bedding and cover material shall conform to City Stds. 2112.09, 2112.10 and 2112.11.

c. Where trench widths are over-excavated, Contractor to increase structural capacity of sewer bedding as directed by the Consultant in accordance with the requirements of the Geotechnical Consultant.

d. All structures within paved surfaces shall have 4:1 frost tapers from frost line to subgrade.

e. Approved native material or granular backfill as specified by the Geotechnical consultant shall be installed on all concrete pipes per OPSD 803.03 (concrete pipe).

f. Storm manholes per OPSD 701.01 (1200 mm diameter), 701.02 (1500 mm and 1800 mm diameter), and 701.06 (2400 mm diameter). Manhole components in accordance with OPSD 701.03, 701.04, 701.05 and 701.06. Manholes deeper than 5.0 m shall be constructed with a safety platform(s) in accordance with OPSD 404.020.

g. Benching per OPSD 1004.01, unless otherwise specified.

h. Top of manhole cover to be set to base course asphalt grade and then adjusted to final grade with Moduloc rings when top lift of asphalt is placed.

7. CATCHBASINS

a. Single catchbasins per OPSD 705.02 Type 'A', frame and grate per OPSD 400.02. Double catchbasins per OPSD 705.01. 150mm diameter subdrains to be constructed where shown.

b. Catchbasin connection to main line sewer in accordance with OPSD 708.03. Risers for CB leads only greater than 1.0m are reported on the drawing.

c. All single catchbasin leads to be 250ø PVC SDR-35, all double catchbasin leads to be 300ø concrete CL-ES, unless otherwise noted on drawings.

8. SITE PLAN NOTES

a. All surface drainage will be self contained, collected and discharged at a location to be approved prior to the issuance of a building permit.

b. The portion of the driveway within the municipal boulevard will be paved by the applicant.

c. At the entrances to the site the municipal curb and sidewalk will be continuous through the driveway and a curb depression will be provided for each entrance.

d. All proposed curbing within the municipal boulevard area for the site is to suit as follows:

1.) For all single family residential properties including on street townhouses, all curbing is to stop at the property limit or the back of the municipal sidewalk, whichever is applicable, or,

2.) for all other proposals including industrial, commercial and condominium developments, all entrances to the site are to be in accordance with O.P.S.D. 350.010.

e. All excess excavated material will be removed from the site.

f. The existing drainage pattern will be maintained except where noted.

g. The applicant will be required to contact all utility companies to obtain all required locates prior to the installation of hoading within the municipal right of way.

h. The applicant will be responsible for the cost of any utility relocations necessitated by the site plan.

i. Prior to construction taking place, all required hoarding in accordance with the Ontario occupational health & safety act and regulations for construction projects must be erected and then maintained throughout all phases of construction.

j. A utility installation coordination meeting is required prior to the stake out installation of a any utilities within the municipal boulevard of Hurontario Street and Matthews Gate.

k. All internal curbs are to be standard 2 stage curb and gutter as per O.P.S.D. 600.070.

l. Due to the limited snow storage provided onsite, the Condominium Disclosure Statement and the Condominium Declaration will include a clause stating that: The City of Mississauga does not require off site snow removal, however, in the case of heavy snow falls, the limited snow storage space available on the property may make it necessary to truck snow off the site and the cost of same will be included in the common expense fees.

REGION OF PEEL NOTES (WATERMAIN)

a. All materials and construction methods must correspond to the current Peel Public Works standards and specifications.

b. Watermain and/or water service materials 100 mm (4") and larger must be PVC. Size 50 mm (2") and smaller must be copper.

c. Watermains and/or water services are to have a minimum cover of 1.7 m (5'6") with a minimum horizontal spacing of 1.2 m (4') from themselves and all other utilities.

d. Provisions for flushing water line prior to testing, etc., must be provided with a least a 50 mm (2") outlet on 100 mm (4") and larger lines. Copper lines are to have flushing points at the end, the same same as the line. They must also be hosed or piped to allow the water to drain onto a parking lot or down a drain. On fire lines, flashing outlet to be 100 mm (4") diameter minimum on a hydrant.

e. All curb stops to be 3.0 m (10') off the face of the building unless otherwise noted.

f. Hydrant and valve set to Region standard 1-6-1 Dimension A and B, 0.7 m (2') and 0.90 m (3') and to have pumper nozzle.

g. Watermains to be installed to grades as shown on approved site plan. Copy of grade sheet must be supplied to inspector prior to commencement of work, where requested by inspector.

h. Watermains must have a minimum vertical clearance of 0.30 m (12") over / 0.5 m (20") under sewers and all other utilities when crossing.

i. All proposed water piping must be isolated from existing lines in order to allow independent pressure testing and chlorinating from existing systems.

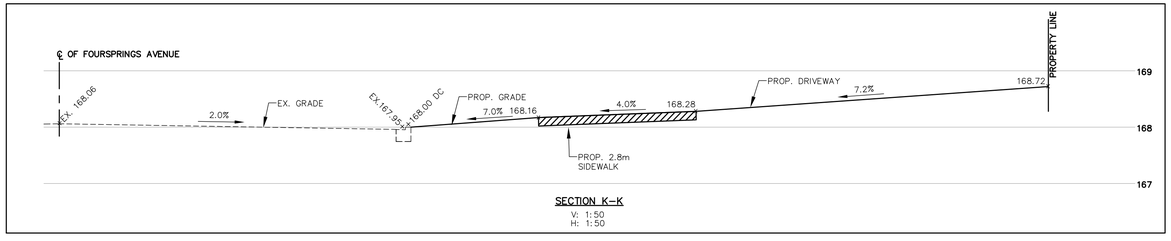
j. All live tapping and operation of Region water valves shall be arranged through the Regional inspector assigned or by contacting the Operations and Maintenance Division.

ADDITION NOTES:

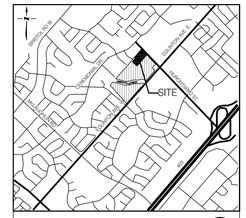
1. LOCATION OF ALL EXISTING UTILITIES IN THE FIELD TO BE ESTABLISHED BY THE CONTRACTOR.

2. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE FOR LOCATES, EXPOSING, SUPPORTING AND PROTECTING OF ALL UNDERGROUND AND OVERHEAD UTILITIES AND STRUCTURES EXISTING AT THE TIME OF CONSTRUCTION IN THE AREA OF HIS WORK. WHETHER SHOWN ON THE PLANS OR NOT, AND FOR ALL REPAIRS AND CONSEQUENCES RESULTING FROM DAMAGE TO SAME.

3. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE TO GIVE 72 HOURS WRITTEN NOTICE TO UTILITIES PRIOR TO CROSSING SUCH UTILITIES, FOR THE PURPOSE OF INSPECTION BY THE CONCERNED UTILITY. THIS INSPECTION WILL BE FOR THE DURATION OF THE CONSTRUCTION, WITH THE CONTRACTOR RESPONSIBLE FOR ALL COSTS ARISING FROM SUCH INSPECTION.



SECTION K-K
V: 1:50
H: 1:50



KEYPLAN N.T.S.

LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

DRAFT PLAN OF SUBDIVISION
SEPTEMBER 25, 2013
&
PLAN OF TOPOGRAPHY
OCTOBER 29, 2007

PART OF LOT 1, CONCESSION 1
WEST OF HURONTARIO STREET
(ORIGINALLY IN TOWNSHIP OF TORONTO)

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

TARASICK McMILLAN KUBICKI LIMITED
ONTARIO LAND SURVEYORS
4181 SLADEVIEW CRESCENT, UNIT 42, MISSISSAUGA, ONTARIO L5L 5R2
TEL: (905) 569-8649 FAX: (905) 569-3160
E-MAIL: office@tmksurveyors.com

ELEVATIONS ARE REFERRED TO CANADIAN GEODETIC VERTICAL DATUM-1928, AND WERE DERIVED FROM CITY OF MISSISSAUGA BENCHMARK No. 218, ELEVATION 177.91 metres.

NO	DATE	REMARKS
7	JAN 04/18	PER CITY COMMENTS
6	NOV 15/17	PER REGION COMMENTS
5	SEPT 05/17	ISSUED FOR BUILDING PERMIT
4	JUN 26/17	ISSUED FOR OPA/RE-ZONING
3	MAY 19/17	ISSUED FOR BUILDING PERMIT
2	FEB. 08/17	ISSUED FOR OPA/RE-ZONING
1	SEPT. 15/16	ISSUED FOR OPA/RE-ZONING

REVISIONS

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME REPORTING ANY DISCREPANCIES TO THE ARCHITECT BEFORE COMMENCING THE WORK.

PRINTS ARE NOT TO BE SCALED.

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PROPOSED CONDOMINIUM DEVELOPMENT
'PERLA' (BLOCK 2, PHASE 1)
35 & 65 WATERGARDEN DRIVE, MISSISSAUGA

Mondiale Development Ltd.
Suite 300- 911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

Sheet Title
GENERAL NOTES-PHASE 1

North

Seal
Professional Engineers Ontario
Limited Licensee
Name: K. S. LO
Number: 100379166
Category: CIVIL
Limitations:
This licence is subject to the terms as stated on the certificate.
Association of Professional Engineers of Ontario
JAN. 04/18

Date Plotted	Date: 16 APRIL 2016
Drawn By:	Scale: 1: 300
Checked By:	Revision Number:
Job No.: 16803	Sheet No.: GN-1

C-600206

Municipal and Development Engineering

Water Resources Engineering

Planning

Project Manag

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