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
May 28, 2018

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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) intensification (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 intensification of the Pinnacle lands in support of processing of OPA/Rezoning. 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 that the proposed intensification will function under the current approved servicing.

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

Proposed intensification is concentrated fully on Blocks 16 and 17 and on portions of Blocks 1 and 2. Figure 1 displays the proposed condo towers and their configuration. One proposed modification includes one condo turned into two condo towers in Block 16, which is Phase IV part 2. The other proposed modification includes two condos turned into three condo towers in Block 17, which is Phase V. Phase V also includes a small portion of Block 1. In Block 2, the two condo towers proposed are larger in size.

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 intensification affects Stage 4 and a portion of Stage 3.

With the proposed intensification, 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 two proposed condo towers in Block 16. The road can be seen in the site map 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.

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 April 2018, 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.

4. SITE SERVICING

4.1 Sanitary Drainage System

IBI has generated sanitary flows from the subject subdivision (Phase IV Part 2 and Phase V), based on the site statistics identified in Appendix B of this report. This FSR is in support of proposed intensification of the site. The approved subdivision sanitary design was used as a basis to which the additional proposed intensification flows were incorporated to ensure the sanitary system has sufficient capacity.

4.1.1 WEST TRUNK

The site is currently being serviced by the west trunk which consists of a 750 mm sewer located at the north property limit in an easement on the west side of the creek. The subdivision is serviced by a network of sanitary sewers, via a sanitary crossing at the creek, which connects to the main trunk sewer at the most west road of the development site. The capacity of the west trunk is 212.2L/s. Currently, the system allows for 39.0L/s for the ultimate build out of the subdivision. It has the capability of accepting an additional 36.1L/s due to the proposed intensification, thus supporting a capacity of 75.1L/s for the ultimate build out subdivision. This is approximately a 93% increase from the current capacity of the subdivision and comprises approximately 35% of the west trunk capacity. See Appendix “C” for details.

4.2 Storm Water Management

4.2.1 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, Jan. 2011). The Phase IV Part 2 and Phase V development are located north of the Phase I development within the Cooksville Creek Watershed and referred to in this report as Drainage Areas 2B and 3B respectively.

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.

4.2.2 STORMWATER MANAGEMENT DESIGN CRITERIA

The approved Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision (IBI Group, Jan. 2011) previously had 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 OPA / Rezoning / Draft Plan of Subdivision (IBI Group, Jan. 2011) and confirmed with Mr. Ghazwan Yousif, P. Eng., M. Sc. at the City of Mississauga's Transportation and Work Department on Aug. 17, 2017, the stormwater management design criteria are identified as follows for the development intensification within Phase V (i.e. Drainage Area 2B legally described as Block 17 and part of Block 1, formerly known as the eastern portion of Development Block 2 in Phase II FSR dated January 2011) and Phase IV Part 2 (i.e. Drainage Area 3B legally described as Block 16 and part of Blocks 2 and 8, formerly known as the eastern portion of Development Block 3 in Phase II FSR dated January 2011) as illustrated in **Figure 3**:

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 post-development design runoff peak discharges from the development Blocks below their corresponding pre-development levels for the 1:2 to 1:5 year design storms.

4.2.3 STORMWATER MANAGEMENT PLAN

In accordance with the approved Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision (IBI Group, Jan. 2011), the proposed stormwater management plan for Phase IV Part 2 and Phase V development intensification is as follows:

a) Dual storm drainage systems are proposed to drain the minor and major runoff flows within the Phase IV Part 2 and Phase V (intensification area) development site.

b) The storm sewer systems are proposed to convey the 1:10 year post-development design runoff peak flows into on-site stormwater detention facilities for water quantity control before discharging into the Phase IV Part 2 and Phase V (intensification area) storm sewer systems (see **Figure 3** and **Appendix "D"**).

c) 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 (see **Figure 5**).

d) It is noted that the storm sewer system on Eglinton Avenue West just has sufficient capacity to accommodate the uncontrolled 10-year design runoff peak flows from the Phases 1 and 2 (including intensification) development sites with minor surcharge at the very last leg of the storm sewer system (see **Appendix "D"**). To eliminate this minor storm sewer system surcharge, on-site stormwater detention facilities equipped with flow control devices are proposed in order to limit the post-development design runoff peak discharges from the development Blocks to/below their corresponding pre-development levels for the 1:2 to 1:10 year design storms.

4.2.4 ON-SITE DETENTION STORAGE REQUIREMENTS

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 pre-development peak flows for the development intensification blocks under the design storms. A hard copy of the pre-development OTTHYMO model output is included in **Appendix "E"**.

The results of the hydrologic analysis indicate that the pre-development unit runoff peak flows are 30.1, 56.1, 83.1 and 151.5 litres/s/ha under the 2-, 5-, 10- and 100-year design storms respectively. According to the proposed stormwater management plan, the post-development design runoff peak discharges from the development intensification blocks to storm sewer systems must be controlled to/below their corresponding pre-development levels for the 2, 5 and 10-year storms. Considering the pre-development unit runoff peak flows and the development intensification area of 0.98 and 0.79 ha for Drainage Areas 2B and 3B respectively (see **Figure 3**), their pre-development design runoff peak flows are computed and summarized in Table 1 for the 2, 5, 10 and 100-year design storms.

Table 1. Pre-Development Design Peak Flows for Phase IV Part 2 and Phase V Development Blocks

Drainage Area Identification Number	Drainage Area (ha)	Pre-Development Design Peak Flows (m^3/s)			
		2-year storm	5-year storm	10-year storm	100-year storm
Area 2B (Ph V)	0.98	0.029	0.055	0.081	0.148
Area 3B (Ph IV Part 2)	0.79	0.024	0.044	0.065	0.119

On-Site Detention Requirements: According to the proposed stormwater management plan, the on-site stormwater detention facilities equipped with flow control devices will be required for the development intensification Blocks to satisfy the stormwater quantity control design criteria. The storm sewer systems within each development intensification Block will be designed in accordance with the City of Mississauga's 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 on-site detention facilities may consist of the interconnected surface (such as parking lots) and underground (such as super pipes/storage tanks) storage, and must be equipped with flow control devices at the outlet of the detention facilities in order to discharge the controlled post-development peak discharges from the development intensification Blocks into the storm sewer systems below their corresponding pre-development design runoff peak flows for the 2, 5 and 10-year storms.

The OTTHYMO computer program has been used to conduct the hydrologic analysis, determine the design peak flows and estimate detention storage requirement 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.

Because the development intensification plans for Drainage Areas 2B and 3B have not been finalized at this time, detention storage requirements for on-site stormwater detention facilities within the Drainage Areas 2B and 3B were preliminarily estimated, using the OTTHYMO program, to satisfy the water quantity control design criteria by limiting the post-development design runoff peak discharges from these drainage areas into the storm sewer systems below their corresponding pre-development levels under the 2, 5 and 10-year design storms.

Considering the development intensification area of 0.98 ha for Drainage Area 2B with the total imperviousness of 100% (to be conservative), for example, the results of the hydrologic analysis under the post-development conditions (see **Appendix "F"**) indicate that the storage requirement for the on-site stormwater detention facility is approximately 299 m^3 for the Drainage Area 2B, in order to limit the 2, 5 and 10-year post-development runoff peak discharges from the Block into the storm sewer systems below its corresponding pre-development levels of 0.029, 0.055 and 0.081 m^3/s (see Table 1) respectively. The assumed design parameters and estimated on-site stormwater detention storage requirements for the development intensification Blocks are summarized in Table 2. A hard copy of the post-development OTTHYMO model output is included in **Appendix "F"**.

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

Drainage area Identification Number	Drainage Area (ha)	Total Imperviousness (%)	Detention Storage Requirement (m^3)		
			2-year Storm	5-year Storm	10-year Storm
Area 2B (Ph V)	0.98	100	196	251	299
Area 3B (Ph IV Part 2)	0.79	100	157	202	241

It should be noted that due to the lack of final development intensification plan, the above estimates of on-site stormwater detention storage requirements are preliminary in nature and subject to the detailed design. When Drainage Areas 2B and 3B are to be developed, it must proceed with the site plan application and approval process, including preparation of a stormwater management design brief in accordance with the overall stormwater management plan/design criteria as outlined in this Report.

4.2.5 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.

4.3 Water Distribution System (see Figure 4)

There is an existing 300 mm diameter watermain located on the west side of Hurontario Street, which has been recently upgraded to a 400mm diameter watermain. The east and west legs of Salishan Circle have a 300mm and 200mm diameter watermain respectively.

The proposed subdivision (Phase IV Part 2 and Phase V) development will be comprised of Blocks 1, 2, 16, and 17 as well as open space Blocks 10 and 11 and will include a mix of townhouse (Block 9), and high rise residential buildings located on the lands just east of the Cooksville Creek corridor. The total number of residential units contemplated at this time for Phase IV Part 2 and Phase V is 2095, generating a total population of 5657 and a commercial population of 53 persons.

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.

The average day demand is based on the subdivision’s short term residential water demand and commercial average day demand of 26.8 L/s and 0.19 L/s respectively, giving a total average day demand of 26.99 L/s used for domestic flows. The required fire flow of 250.00 L/s, obtained from the Fire Underwriter’s Survey (FUS), is used. Therefore, the maximum day demand (MDD) and fire flow for the entire site is 276.99 L/s

Based on AECOM’s Hydraulic Analysis (for the full build out in 2031), the available flows in the watermain were found to be the lowest at Node 102. The flow at this location is modelled as 724.16 L/s. The available flow from Node 102 subtracting the MDD and the fire flow provides 447.17 L/s of additional available capacity. A table summarizing the calculations can be found in Appendix H.

The AECOM analysis concluded that the existing trunk watermain system will be able to maintain “minimum operating pressure under average day demand conditions, maximum day demand conditions and maximum day demand plus fire flow demand conditions, all as recommended by MOE Design Guidelines”. The analysis also concluded that the existing treatment plant has sufficient capacity under maximum day demand scenarios to accommodate the additional water demand associated with the proposed development. The AECOM Hydraulic Modelling was based on a 300mm diameter watermain on Hurontario, which has been since upgraded to a 400mm diameter watermain.

4.4 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 Blocks 16, 17, and portions of Block 1, 2, and 8 of the Phase IV Part 2 and Phase V 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. 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

A. Covello, P.Eng.
Associate

Nicky Chang, Ph.D.
Water Resources Specialist

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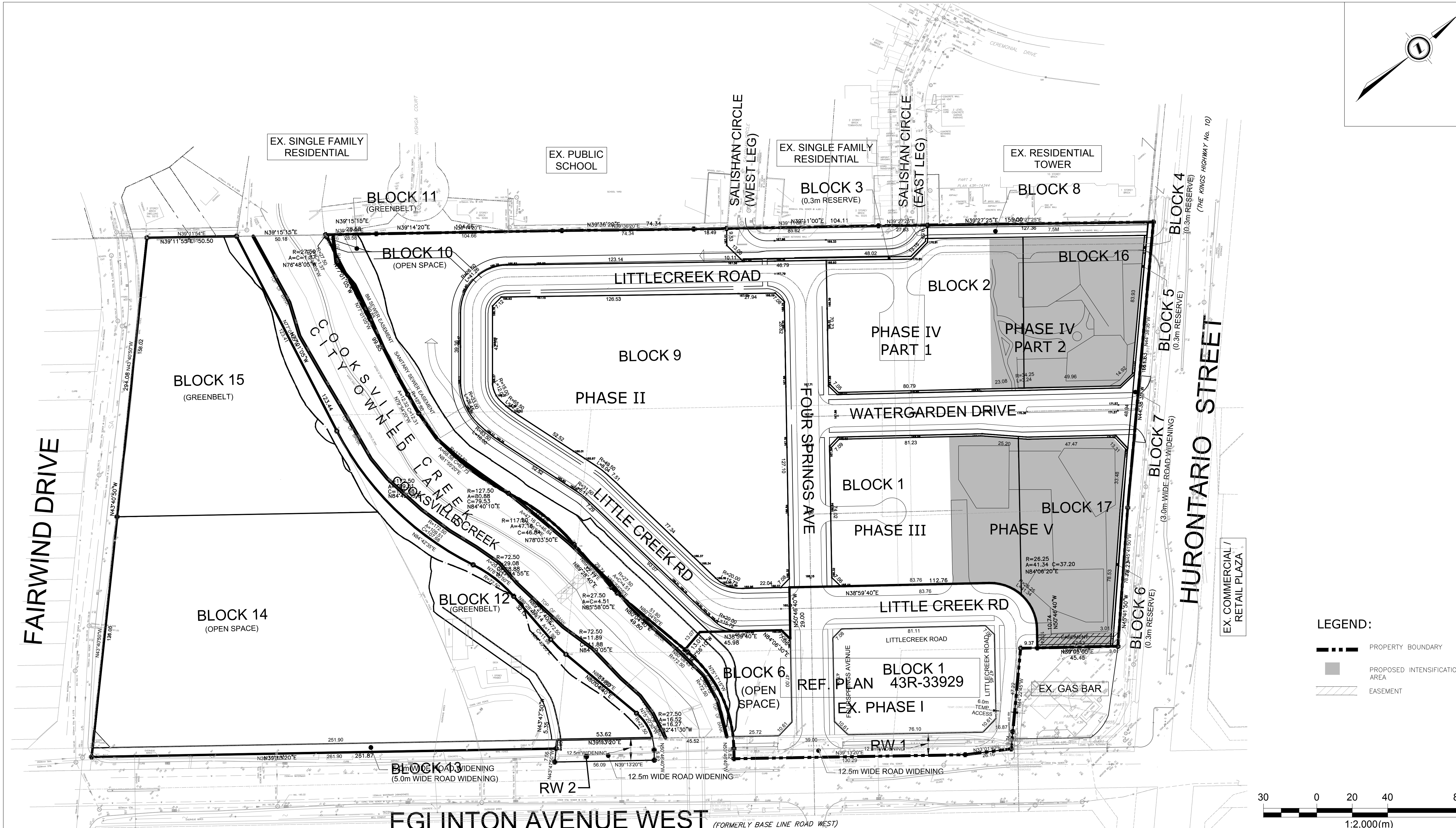


PINNACLE
INTERNATIONAL

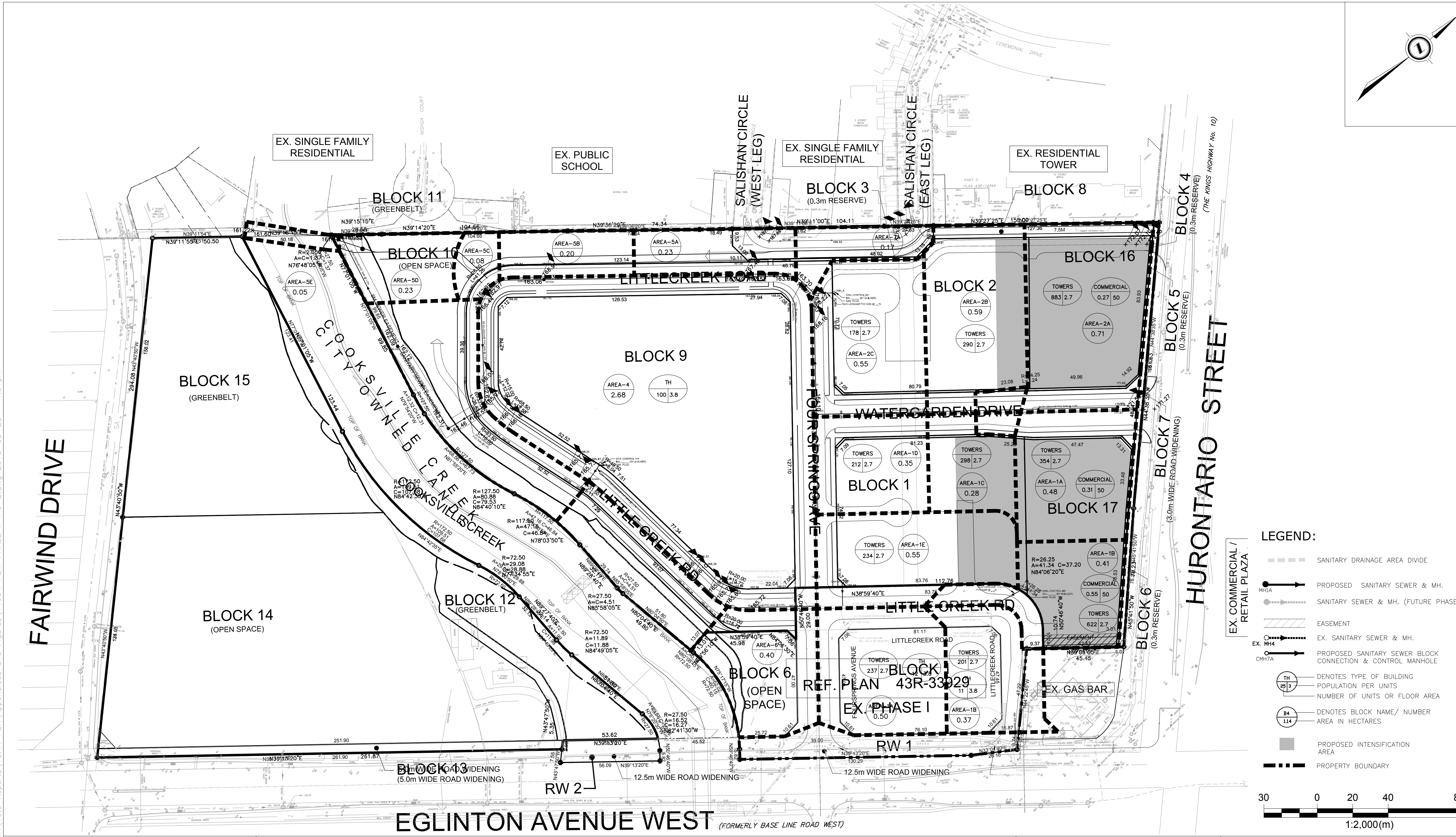
TITLE:

SITE PLAN AREA / PROPOSED ROADS PLAN
PHASE IV PART 2 AND PHASE V

JOB No:	108686
SCALE:	1:2000
DATE:	MAY, 2018
FIGURE No:	FIGURE 1

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

T:\108686_Pinn_Uptown\10.0 Reports\10.11 FSR\Appendices and Figures\14604-FSR-PHASE II-FIG-1.dwg 2018-05-28 - 4:01pm



PINNACLE INTERNATIONAL

TITLE:

SANITARY DRAINAGE AREAS AND SEWER SYSTEM
PHASE IV PART 2 AND PHASE V

JOB No:	108686
SCALE:	1:2000
DATE:	MAY, 2018
FIGURE No:	FIGURE 2

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

PINNACLE INTERNATIONAL

TITLE:

MINOR STORM DRAINAGE AREA AND SEWER SYSTEM PHASE IV PART 2 AND PHASE V

JOB No:	108686
SCALE:	1:2500
DATE:	MAY, 2018
FIGURE No:	FIGURE 3

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

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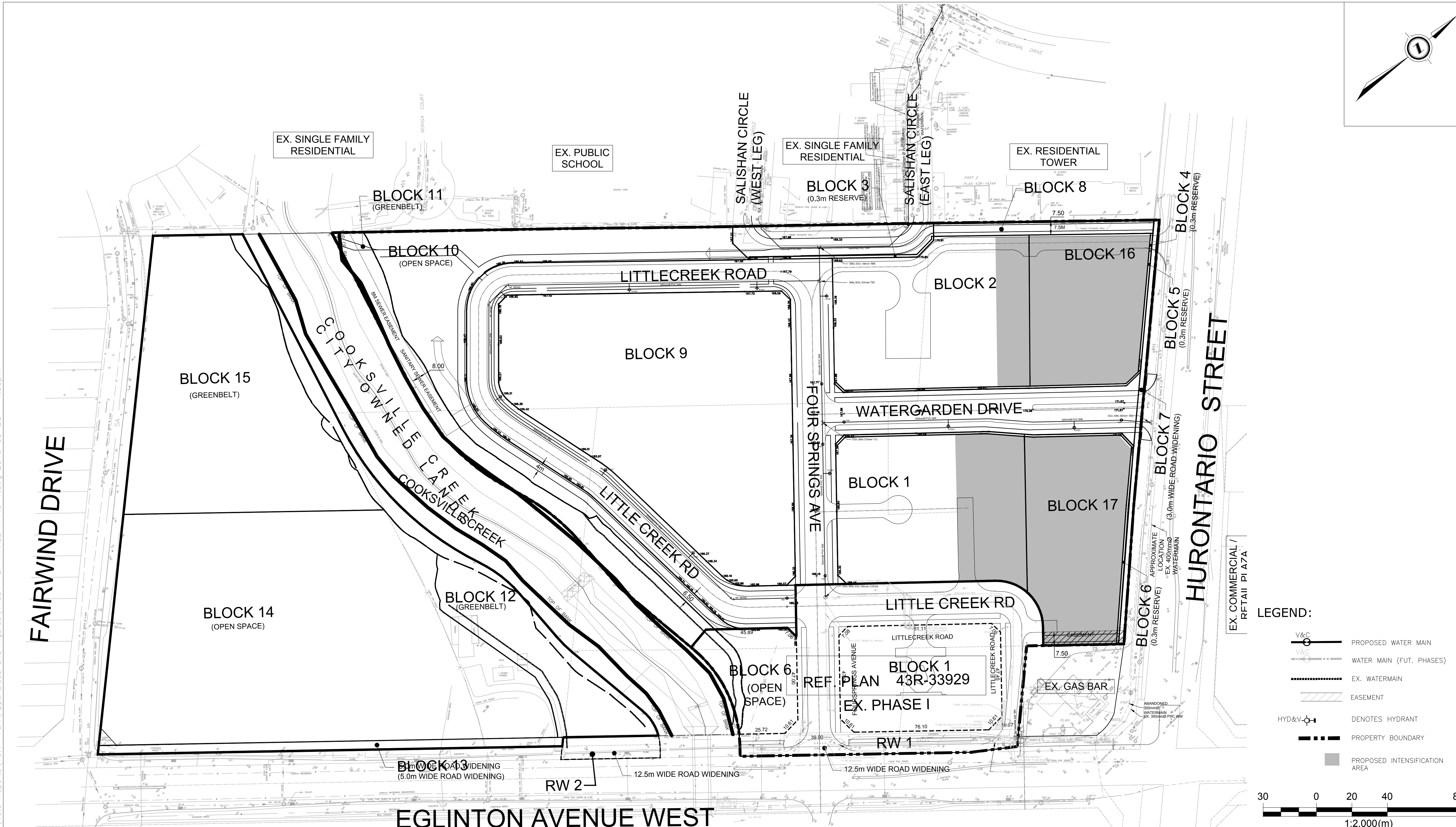


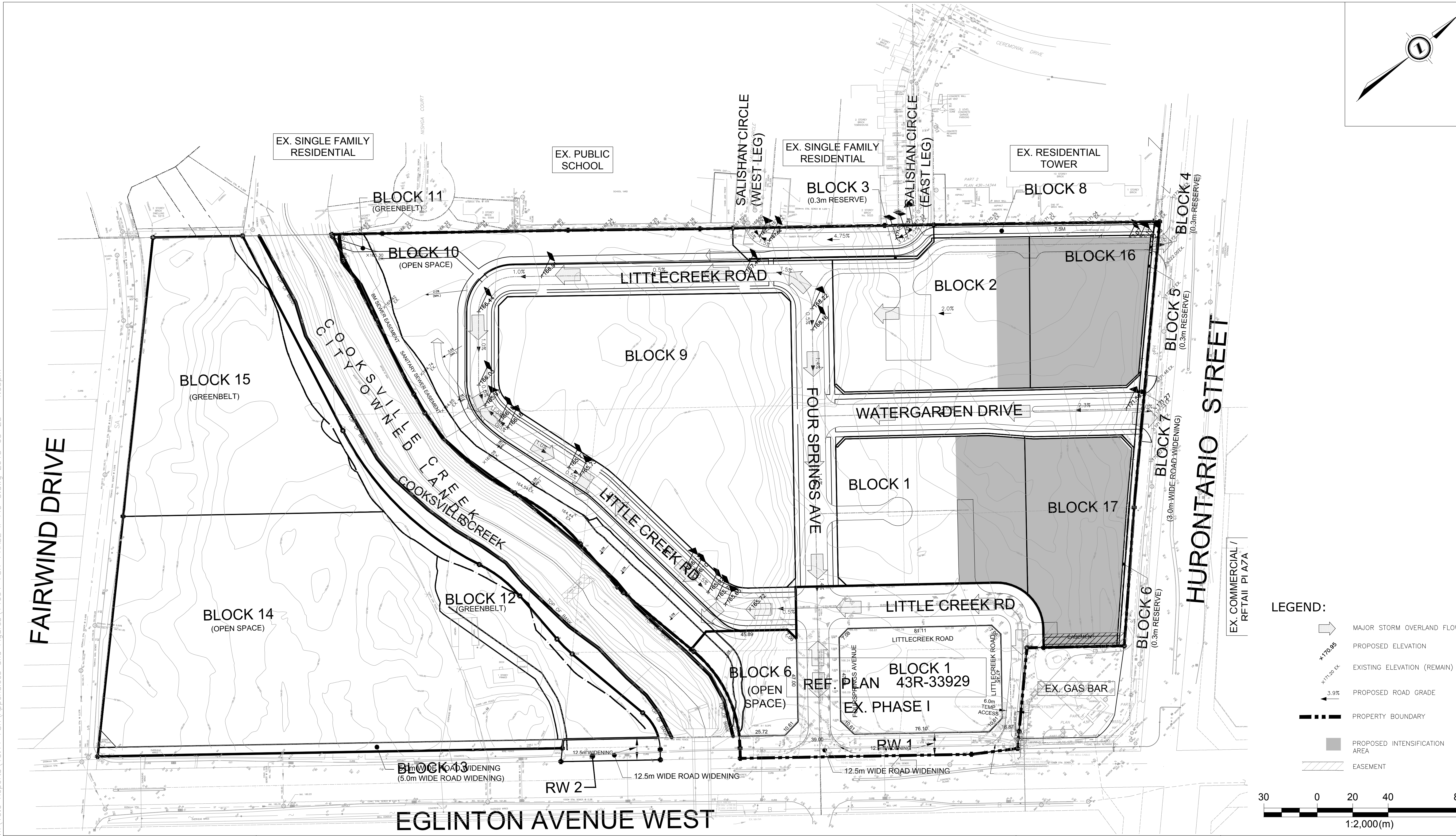
PINNACLE INTERNATIONAL

TITLE:

WATER DISTRIBUTION SYSTEM
PHASE IV PART 2 AND PHASE V

JOB No:	108686
SCALE:	1:2000
DATE:	MAY, 2018
FIGURE No:	FIGURE 4

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



PINNACLE
INTERNATIONAL

TITLE:

ROAD GRADES AND OVERLAND FLOW ROUTING
PHASE IV PART 2 AND PHASE V

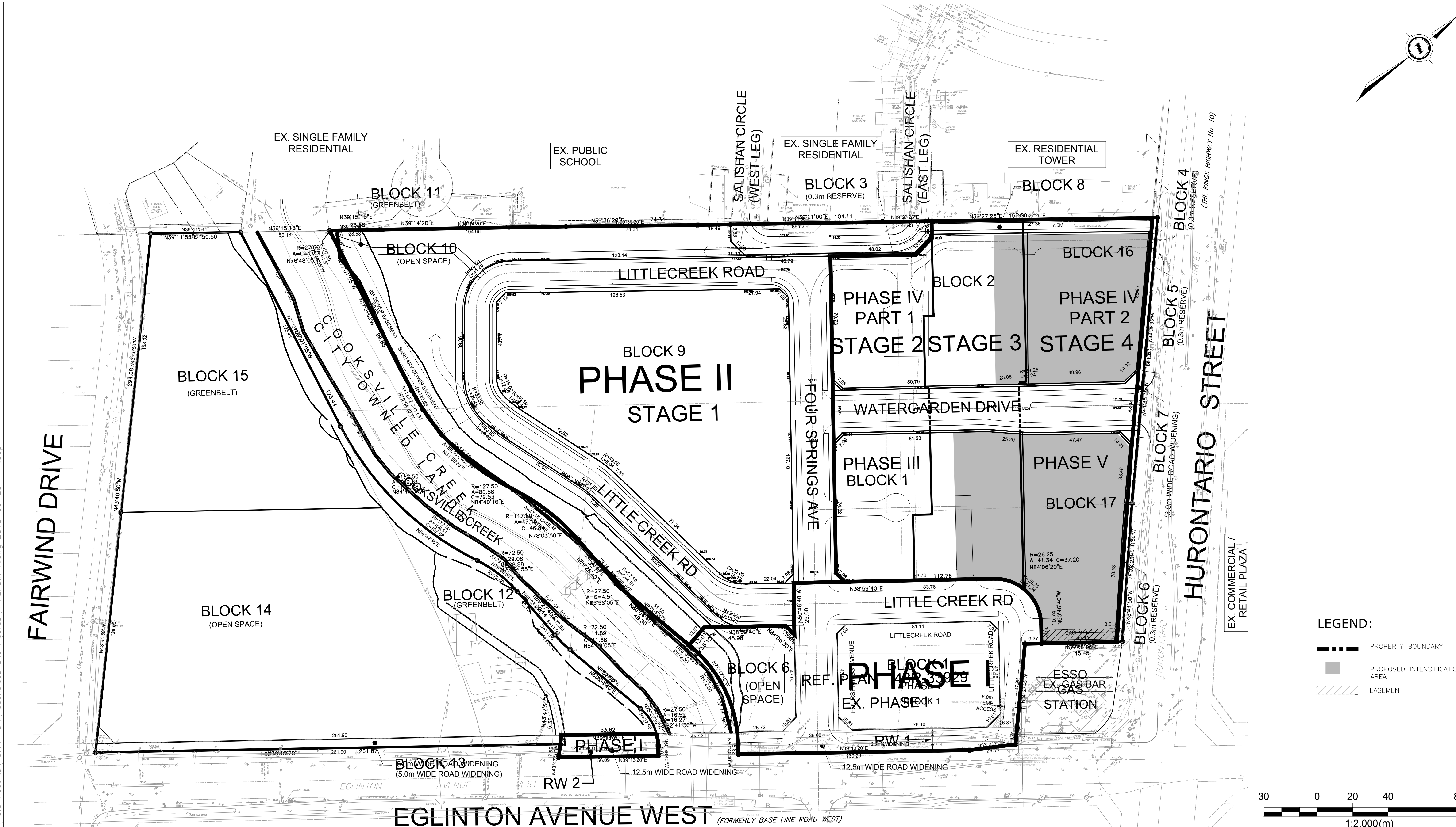
JOB No:	108686
SCALE:	1:2000
DATE:	MAY, 2018
FIGURE No:	FIGURE 5

IBI

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

APPENDIX A

SUBDIVISION – STAGING PLAN



PINNACLE
INTERNATIONAL

TITLE:

STAGING PLAN
SUBDIVISION

JOB No:	108686
SCALE:	1:2000
DATE:	NOVEMBER, 2017
FIGURE No:	APPENDIX 'A'

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

APPENDIX B

SUITE SUMMARY

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)

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%)	15,466.1 SM (100%)
COVERED AREA/ FLOOR AREA	3014.0 SM (58.7%)	COVERED AREA/ FLOOR AREA	5920.0 SM (57.4%)	8934.0 SM (57.7%)
PAVED AREA	390.6 SM (7.90%)	PAVED AREA	568.0 SM (5.5%)	958.6 SM (6.1%)
LANDSCAPED AREA	1727.4 SM (33.4%)	LANDSCAPED AREA	3865.1 SM (37.0%)	5592.5 SM (36.1%)

MINIMUM LANDSCAPED AREA 25% OF LOT AREA

GFA AND F.S.I

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)	8.88X	8.23X

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)	82,843.2 SM	118,708.2 SM

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)	8632.5 SM	–
TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	35,865.0 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	91,475.7 SM	127,340.7 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	30 STOREYS	35 STOREYS	50 STOREYS

UNITS

NUMBER OF UNITS ALLOWED 1121

NUMBER OF UNITS PROPOSED 1606

	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	48	143	81	272	–
1 BEDROOM + DEN	–	–	–	1 BEDROOM + DEN	134	84	232	450	–
2 BEDROOM	–	–	–	2 BEDROOM	94	67	108	269	–
2 BEDROOM + DEN	–	–	–	2 BEDROOM + DEN	15	19	67	101	–
3 BEDROOM	–	–	–	3 BEDROOM	3	41	24	68	–
2 STOREY	–	–	–	2 STOREY	–	–	–	–	–
TOTAL SUITES PROPOSED	234	212	446	TOTAL SUITES PROPOSED	294	354	512	1160	1606

AMENITY

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

	PHASE 3	PHASE 5	TOTAL - PHASE (3+5)
MINIMUM AMENITY AREA ALLOWED (5.6SM PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)	2497.6 SM	6496.0 SM	8993.5 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	1008.0 SM	INDOOR AMENITY AREA PROVIDED	624.1	1,014.9	1,522.2	3,161.2	4169.2 SM
OUTDOOR AMENITY AREA PROVIDED	1060.0	612.0	1672.0 SM	OUTDOOR AMENITY AREA PROVIDED	1,479.1	1,984.9	2,519.6	5,983.6	7655.6 SM
TOTAL AMENITY AREA PROVIDED	1891.0	789.0	2680.0 SM	TOTAL AMENITY AREA PROVIDED	2103.0	2999.8	4041.8	9144.6	11824.6 SM
TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	–	–	2497.6 SM	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	–	–	–	6496.0	8993.6 SM

AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT 5.60 SM

	PHASE 3 BUILDING 1	PHASE 3 BUILDING 2	PHASE 3 TOTAL (1+2)		PHASE 5 BLDG. 1	PHASE 5 BLDG. 2	PHASE 5 BLDG. 3	PHASE 5 TOTAL	TOTAL - PHASE (3+5)
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	655.0 SM	593.6 SM	1248.6 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	823.2	991.2	1433.6	3248 SM	4496.8 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	945.5 SM	394.5 SM	1340.0 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	1051.5	1499.9	2020.9	4572.3 SM	5550.3 SM
MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE			55.0 SM	MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE				55.0 SM	110.0 SM
AMENITY AREA PROVIDED OUTSIDE AT GRADE	402.0	321.0	723.0 SM	AMENITY AREA PROVIDED OUTSIDE AT GRADE	1479.1	698.6	882	3059.0 SM	3782.0 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	320	345	537	1202	–
RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	–	–	–	RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	4	50	29	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	44*	53*	77*	174*	–
COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	–	–	–	COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	–	143*	229*	372*	–
TOTAL PARKING REQUIRED:	–	–	–	TOTAL PARKING REQUIRED:	324	538	795	1657	–
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 REQUIRED(4% OF TOTAL NON-RESIDENTIAL PARKING SPACES- AS PER TABLE 3.1.3.1-ZONING BY LAW 0225-2007)	2	2	4	8	–

PARKING PROVIDED:

PHASE 3	BUILDING 1		BUILDING 2		TOTAL (1+2)
	# OF CARS		# OF CARS		
	VISITORS	TENANTS	VISITORS	TENANTS	TOTAL
PARKING LEVEL 1	–	–	–	–	–
PARKING LEVEL 2	–	–	–	–	–
PARKING LEVEL 3	–	–	–	–	–
PARKING LEVEL 4	–	–	–	–	–
TOTAL PARKING PROPOSED	–	–	–	–	–

PHASE 5	BUILDING 1		BUILDING 2		BUILDING 3		TOTAL	TOTAL - PHASE (3+5)
	# OF CARS		# OF CARS		# OF CARS			
	VIS./COMM	TENANTS	VIS./COMM	TENANTS	VIS./COMM	TENANTS		
PARKING LEVEL 1	–	–	–	–	–	–	189	–
PARKING LEVEL 2	–	–	–	–	–	–	178	–
PARKING LEVEL 3	–	–	–	–	–	–	276	–
PARKING LEVEL 4	–	–	–	–	–	–	276	–
PARKING LEVEL 5	–	–	–	–	–	–	276	–
PARKING LEVEL 6	–	–	–	–	–	–	276	–
PARKING LEVEL 7	–	–	–	–	–	–	199	–
TOTAL PARKING PROPOSED							1670	–

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.

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	7276.9 SM (100%)	14,604.1 SM (100%)
COVERED AREA/ FLOOR AREA	3494.2 SM (46.8%)	COVERED AREA/ FLOOR AREA	3607.0 SM (49.0%)	7066.2 SM (48.3%)
PAVED AREA	2005.8 SM (28.1%)	PAVED AREA	865.0 SM (11.8%)	2870.8 SM (19.6%)
LANDSCAPED AREA	1827.5 SM (25.1%)	LANDSCAPED AREA	2804.2 SM (38.5%)	4631.7 SM (31.7%)

MINIMUM LANDSCAPED AREA 25% OF LOT AREA

G.F.A AND F.S.I

MINIMUM ALLOWED FLOOR SPACE INDEX (2.9 X LOT AREA) 42351.9 SM

MAXIMUM ALLOWED FLOOR SPACE INDEX (5.19X LOT AREA) 75795.3 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)	8.87X	6.99X

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)	62,535.5 SM	99,884.6 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)	2031.0 SM	2264.0 SM
TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	37,582.1 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	64,566.5 SM	102,148.6 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	MINIMUM BUILDING HEIGHT	MAXIMUM BUILDING 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 4
BUILDING HEIGHT	15 STOREYS	34 STOREYS	BUILDING HEIGHT	50 STOREYS	38 STOREYS

UNITS

NUMBER OF UNITS ALLOWED 748

NUMBER OF UNITS PROPOSED 1403

	PART 1				PART 2			TOTAL - PART (1 + 2)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BUILDING 3	BUILDING 4	TOTAL(3+4)	
1 BEDROOM	57	88	145	1 BEDROOM	101	144	245	390
1 BEDROOM + DEN	47	44	91	1 BEDROOM + DEN	316	118	434	525
2 BEDROOM	5	2	7	2 BEDROOM	96	136	232	239
2 BEDROOM + DEN	64	152	216	2 BEDROOM + DEN	10	8	18	234
3 BEDROOM	–	–	–	3 BEDROOM	4	2	6	6
2 STOREY	5	4	9	2 STOREY	–	–	–	9
TOTAL SUITES PROPOSED	178	290	468	TOTAL SUITES PROPOSED	527	408	935	1403

AMENITY

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

	PART 1	PART 2	TOTAL
MINIMUM AMENITY AREA ALLOWED (5.6SM PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)	2620.8 SM	5236.0 SM	7856.8 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	BLDG. 4	TOTAL(3+4)	
INDOOR AMENITY AREA PROVIDED	340.1	600.9	941.0 SM	INDOOR AMENITY AREA PROVIDED	605.5	1,078.3	1683.9 SM	2624.9 SM
OUTDOOR AMENITY AREA PROVIDED	633.3	1090.1	1723.4 SM	OUTDOOR AMENITY AREA PROVIDED	1,123.5	2,582.9	3,706.4 SM	5,429.8 SM
TOTAL AMENITY AREA PROVIDED	973.4	1691.0	2664.4 SM	TOTAL AMENITY AREA PROVIDED	1,729.0	3,661.2	5390.2 SM	8054.6 SM
TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	N/A	N/A	2620.0 SM	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	N/A	N/A	5236.0 SM	7856.0 SM

AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT 5.74 SM

	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 3	BLDG. 4	TOTAL(3+4)	TOTAL - PART (1 + 2)
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	490.0 SM	820.0 SM	1310.0 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	1475.6 SM	1142.4 SM	2618.4 SM	3928.0 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	371.5 SM	371.5 SM	743.0 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	864.5	1830.6	2695.1 SM	3438.1 SM
MINIMUM AMENITY AREA REQUIRED OUTSIDE AT GRADE			55.0 SM	MINIMUM AMENITY AREA REQUIRED OUTSIDE AT GRADE			55.0 SM	110.0 SM
AMENITY AREA PROVIDED OUTSIDE AT GRADE			1036.0 SM	AMENITY AREA PROVIDED OUTSIDE AT GRADE	1039.3	1306.2	2,345.5 SM	3381.5 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	BLDG. 4	TOTAL(3+4)	
RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM: 1.1 SPACE/DWELLING UNIT	187	318	505	RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM: 1.1 SPACE/DWELLING UNIT	576	447	1023	1528
RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	6	5	11	RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	5	3	8	19
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	27*	44	71	VISITOR PARKING REQUIRED: 0.15 SPACES/DWELLING UNIT	79*	61*	140*	211
COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	13*	–		COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	30*	58*		–
TOTAL PARKING REQUIRED:	220	367	587	TOTAL PARKING REQUIRED:	660	511	1171	1758
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	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)	4	3	7	10

PARKING PROVIDED:

	PART 1						PART 2					TOTAL - PART (1 + 2)
	BUILDING 1		BUILDING 2		TOTAL (1+2)		BLDG. 3		BLDG. 4		TOTAL(3+4)	
	# OF CARS		# OF CARS				# OF CARS		# OF CARS			
	VISITORS	TENANTS	VISITORS	TENANTS	TOTAL		VIS./COMM	TENANTS	VIS./COMM	TENANTS		
PARKING LEVEL 1	27	16	44	17	104	PARKING LEVEL 1					210	314
PARKING LEVEL 2	0	72	0	93	165	PARKING LEVEL 2					215`	380
PARKING LEVEL 3	0	72	0	102	174	PARKING LEVEL 3					215	389
PARKING LEVEL 4	0	72	0	102	174	PARKING LEVEL 4					215	389
PARKING LEVEL 5		20			20	PARKING LEVEL 5					215	235
						PARKING LEVEL 6					148	148
TOTAL PARKING PROPOSED	27	252	44	314	637	TOTAL PARKING PROPOSED	0	287	134	694	1218	1855

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.

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 : May 28, 2018

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 1A	STUB	MH6A																
TOWER				2.7	354.0		956											
OFFICE/COMMERCIAL				50.0	0.33		17											
INFILTRATION			0.48			0.48	972	0.0130	0.0001		0.013	16.6		250	1.00%	0.059	1.211	0.228
AREA 2A	STUB	MH6A																
TOWER				2.7	935.0		2525											
OFFICE/COMMERCIAL				50.0	0.20		10											
INFILTRATION			0.71			0.71	2535	0.0311	0.0001		0.031	13.5		250	1.00%	0.059	1.211	0.186
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	0.206
WATERGARDEN DRIVE	MH6A	MH7A				3.43	3619	0.0428	0.0007		0.043	80.0		250	1.00%	0.059	1.211	1.101
AREA 2B	STUB	MH7A																
TOWER				2.7	290.0		783											
INFILTRATION			0.59			0.59	783	0.0106	0.0001		0.011	11.8		250	1.00%	0.059	1.211	0.163
AREA 1C	STUB	MH7A																
TOWER				2.7	294.0		794											
INFILTRATION			0.28			0.28	794	0.0107	0.0001		0.011	14.1		250	1.00%	0.059	1.211	0.195
WATERGARDEN DRIVE	MH7A	MH8A				4.30	5196	0.0588	0.0009		0.060	72.0		250	1.00%	0.059	1.211	0.990
AREA 2C	STUB	MH8A																
TOWER				2.7	178.0		481											
INFILTRATION			0.55			0.55	481	0.0067	0.0001		0.007	11.2		250	1.00%	0.059	1.211	0.154
AREA 1D	STUB	MH8A																
TOWER				2.7	212.0		572											
INFILTRATION			0.35			0.35	572	0.0079	0.0001		0.008	14.2		250	1.00%	0.059	1.211	0.195
WATERGARDEN DRIVE	MH8A	MH9A				5.20	6249	0.0691	0.0010		0.070	18.1		250	2.38%	0.092	1.869	0.161
AREA 1E	STUB	MH10A																
HIGH RISE				2.7	234.0		632											
INFILTRATION			0.55			0.55	632	0.0087	0.0001		0.009	11.2		250	1.00%	0.059	1.211	0.154
FOURSPRINGS AVE.	MH10A	MH9A				0.55	632	0.0087	0.0001		0.009	61.3		250	0.50%	0.042	0.857	1.192
	MH9A	MH11A				5.75	6881	0.0751	0.0012		0.076	56.3		375	0.40%	0.111	1.004	0.934
LITTLE CREEK RD																		
AREA 7A	MH11A	MH12A	0.17			5.92	6881	0.0751	0.0012		0.076	29.2		375	0.40%	0.111	1.004	0.485
AREA 5A	MH12A	MH13A	0.23			6.15	6881	0.0751	0.0012		0.076	85.9		375	0.40%	0.111	1.004	1.426
AREA 5B	MH13A	MH14A	0.20			6.35	6881	0.0751	0.0013		0.076	77.4		375	0.59%	0.135	1.219	1.058
EASEMENT																		
AREA 5C	MH14A	MH15A	0.08			6.43	6881	0.0751	0.0013		0.076	28.3		375	1.50%	0.215	1.944	0.243
AREA 5D	MH15A	MH16A	0.23			6.66	6881	0.0751	0.0013		0.076	69.2		375	2.00%	0.248	2.245	0.514
CREEK CROSSING																		
AREA 5E	MH16A	EX. MHT4	0.05			6.71	6881	0.0751	0.0013		0.076	48.9		375	0.30%	0.096	0.870	0.937
TO EX. MH 9A																		
AREA 1B																		
TOWER				2.7	512.0		1382											
OFFICE/COMMERCIAL				50.0	0.53		27											
INFILTRATION	STUB	MH4A	0.41			0.41	1409	0.0183	0.0001		0.018	13.6		200	4.00%	0.066	2.088	0.109
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	0.116
LITTLE CREEK RD	MH4A	MH1A				0.78	1994	0.0251	0.0002		0.025	85.4		250	1.00%	0.059	1.211	1.175
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	0.124
LITTLE CREEK RD	MH1A	MH2A				1.28	2679	0.0327	0.0003		0.033	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				4.36	3059	0.0368	0.0009		0.038	65.9		300	0.50%	0.068	0.967	1.135
Foursprings Avenue	MH3A	EX.MH9A				**4.36**	**3059**	0.0368	0.0009		**0.038**	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. 2011

CONSULTANT IBI GROUP.

STORM DRAINAGE DESIGN CHART
FOR CIRCULAR DRAINS FLOWING FULL
FINAL DESIGN

Designed By: AC

MAJOR DRAINAGE AREA PHASE II

Checked By: NC

STREET (DRAINAGE AREA LABEL)	LOCATION FROM MH. No.	FROM Sta.	TO MH. No.	TO Sta.	DRAINAGE AREA A ha	Coeff. C	A*C	ACCUM. A*C	RUNOFF Cummul. Tc (Ti-__) min.	i_{10} mm/h	i_{100} mm/h	Q_{10} m^3/sec	Q_{100} m^3/sec	PIPE SELECTION 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**	PROFILE Fall In Sewer m	UPSTREAM Surface Elev. m	UPSTREAM 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																								
STREET 'B'	MH 6		MH 7		0.26	0.75	0.195	0.195	15.000	99.166	140.690	0.054	0.076	85.0	1.00%	**300**	0.101	1.383	1.025					
	MH 7		MH 9		0.20	0.75	0.150	0.345	16.025	95.302	135.287	0.091	0.130	90.0	0.50%	**375**	0.129	1.134	1.322					
LITTLE CREEK	MH 8		MH 9		1.67	0.75	1.253	1.598	17.347	90.793	128.973	0.403	0.572	56.0	0.70%	**600**	0.536	1.836	0.508					
CONVERGENCE	MH 9		MH 10		0.12	0.75	0.090	2.033	17.855	89.186	126.720	0.504	0.715	55.0	0.50%	**675**	0.620	1.679	0.546					
	MH 10		MH 5		1.65	0.75	1.238	3.270	18.401	87.530	124.398	0.795	1.130	53.0	1.00%	**750**	1.161	2.547	0.347					
SALISHAN CIRCLE	MH 11		MH 13		0.13	0.75	0.098	0.098	15.000	99.166	140.690	0.027	0.038	55.0	1.00%	**300**	0.101	1.383	0.663					
	MH 12		MH 13		0.07	0.75	0.053	0.053	15.000	99.166	140.690	0.014	0.021	25.0	1.00%	**300**	0.101	1.383	0.301					
	MH 13		MH 14		0.00	0.00	0.000	0.150	15.66301	96.626	137.139	0.040	0.057	15.4	0.50%	**300**	0.071	0.978	0.263					
	MH 14		MH 15		0.11	0.75	0.083	0.285	15.96437	95.519	135.591	0.076	0.107	37.3	0.50%	**375**	0.129	1.134	0.548					
STREET 'A'	MH 15		MH 16		0.21	0.75	0.158	0.158	16.51236	93.580	132.877	0.041	0.058	115.0	0.50%	**375**	0.129	1.134	1.690					
	MH 16		MH 17		0.04	0.75	0.030	0.188	18.202	88.126	125.234	0.046	0.065	23.0	0.50%	**375**	0.129	1.134	0.338					
	MH 17		MH 18		0.02	0.75	0.015	0.203	18.540	87.121	123.824	0.049	0.070	17.5	0.50%	**375**	0.129	1.134	0.257					
	MH 18		MH 19		0.10	0.75	0.075	0.278	18.797	86.373	122.775	0.067	0.095	57.3	0.30%	**450**	0.163	0.992	0.962					
	MH 19		MH 20		0.02	0.75	0.015	0.293	19.759	83.700	119.021	0.068	0.097	13.0	0.30%	**450**	0.163	0.992	0.218					
	MH 20		MH 21		0.11	0.75	0.083	0.375	19.978	83.119	118.205	0.087	0.123	63.8	0.30%	**450**	0.163	0.992	1.072					
	MH 21		MH 22		2.16	0.75	1.620	1.995	21.049	80.398	114.380	0.446	0.634	109.9	0.30%	**750**	0.636	1.395	1.313					
	MH 22		MH 5		0.11	0.75	0.083	2.078	22.362	77.327	110.060	0.446	0.635	50.3	0.30%	**750**	0.636	1.395	0.601					
					6.98																			
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	25.0	1.00%	**300**	0.101	1.383	0.301					
LITTLECREEK ROAD	MH2		MH3		0.00	0.00	0.000	0.128	15.301	97.992	139.050	0.035	0.049	33.5	0.50%	**300**	0.071	0.978	0.571					
LITTLECREEK ROAD	MH3		MH4		0.00	0.75	0.000	0.128	15.872	95.854	136.059	0.034	0.048	56.1	0.50%	**450**	0.210	1.281	0.730					
LITTLECREEK ROAD	MH4		MH5		0.26	0.75	0.195	0.323	16.602	93.270	132.443	0.084	0.119	59.0	0.50%	**525**	0.317	1.420	0.693	0.035				
					0.43																			
CONVERGENCE	MH5		MH6		0.70	0.75	0.525	6.195	22.963	76.009	108.204	1.308	1.862	89.0	0.50%	**975**	1.653	2.145	0.692					
					8.11																			
TO EXISTING CULVERT																								
EGLINTON AVE. WEST	MH6		EX.CULVERT		0.00	0.00	0.000	15.817	23.655	74.554	106.154	3.275	4.664	60.0	1.20%	**1050**	3.121	3.491	0.286					
																	3.121							

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    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


--------------------------------------------------------------------------------
*
* 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-313261310589

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  LICENSED TO:  Cumming Cockburn Limited, Ottawa

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

DATE: 08-29-2017                      TIME: 16:27:32

COMMENTS: ____________________________________________________________

--------------------------------------------------------------------------------
****************************************************************************
*                                                                          *
* POST-DEVELOPMENT CONDITIONS                                              *
* PINNACLE PHASE II REVISED FOR DEVELOPMENT INTENSIFICATION                *
* PART OF LOT 1, CONCESSION 1, WEST OF HURONTARIO STREET                   *
* NORTHWEST OF EGLINTON AVENUE WEST AND HURONTARIO STREET                  *
* CITY OF MISSISSAUGA                                                      *
*                                                                          *
* IBI GROUP, AUGUST 2017                                                   *
* 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

--------------------------------------------------------------------------------
*
*-----------------------------------------
* DRAINAGE AREA 2B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .98          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     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         5.00
     Storage Coeff.  (min)=       2.21 (ii)    2.24 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .30          .31
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .26          .00            .26 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16          35.83
     TOTAL RAINFALL   (mm)=      36.63        36.63          36.63
     RUNOFF COEFFICIENT   =        .98          .30            .98

       (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.

--------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 7 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .055       .025
                           .029       .020    |     .081       .030

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)      .98       .26      2.25      35.83
     OUTFLOW: ID= 7 (2025)      .98       .03      2.58      35.66

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 10.996
                   TIME SHIFT OF PEAK FLOW         (min)= 20.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .020

--------------------------------------------------------------------------------
*
*-----------------------------------------
* DRAINAGE AREA 3B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN           |
```

```
| STANDHYD  (2040) |   Area    (ha)=    .79
|ID= 4 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .79          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     Length            (m)=      72.46        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     104.51        21.91
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       2.07 (ii)    2.10 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .31          .31
                                                           *TOTALS*
     PEAK FLOW       (cms)=        .21          .00            .21 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16          35.83
     TOTAL RAINFALL   (mm)=      36.63        36.63          36.63
     RUNOFF COEFFICIENT   =        .98          .30            .98

       (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.

--------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2045) |
| IN= 4---> OUT= 9 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .044       .020
                           .024       .016    |     .065       .024

                             AREA     QPEAK     TPEAK       R.V.
                             (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 4 (2040)    .79       .21      2.25      35.83
     OUTFLOW: ID= 9 (2045)    .79       .02      2.58      35.62

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 11.159
                   TIME SHIFT OF PEAK FLOW        (min)= 20.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .016

--------------------------------------------------------------------------------
*
************************ 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

--------------------------------------------------------------------------------
*
*-----------------------------------------
* DRAINAGE AREA 2B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .98          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     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         5.00
     Storage Coeff.  (min)=       1.96 (ii)    1.99 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .31          .31
                                                           *TOTALS*
     PEAK FLOW       (cms)=        .36          .00            .36 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50          48.44
     TOTAL RAINFALL   (mm)=      49.24        49.24          49.24
     RUNOFF COEFFICIENT   =        .98          .38            .98

       (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.

--------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 7 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)      (ha.m.)  |   (cms)     (ha.m.)
                            .000       .000   |     .055       .025
                            .029       .020   |     .081       .030

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)       .98       .36      2.25      48.44
     OUTFLOW: ID= 7 (2025)       .98       .05      2.50      48.27

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 15.249
                   TIME SHIFT OF PEAK FLOW         (min)= 15.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .025

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

```
*-----------------------------------------
* DRAINAGE AREA 3B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2040) |   Area    (ha)=    .79
|ID= 4 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .79          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     Length            (m)=      72.46        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     140.49        37.11
                over (min)       5.00         5.00
     Storage Coeff.  (min)=       1.84 (ii)    1.87 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .32          .32
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .29          .00            .29 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50          48.44
     TOTAL RAINFALL   (mm)=      49.24        49.24          49.24
     RUNOFF COEFFICIENT   =        .98          .38            .98

       (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.

-------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2045) |
| IN= 4---> OUT= 9 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .044       .020
                           .024       .016    |     .065       .024

                             AREA     QPEAK     TPEAK       R.V.
                             (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 4 (2040)    .79       .29      2.25      48.44
     OUTFLOW: ID= 9 (2045)    .79       .04      2.50      48.24

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 14.976
                  TIME SHIFT OF PEAK FLOW         (min)= 15.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .020

-------------------------------------------------------------------------------
*
*********************** 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

--------------------------------------------------------------------------------
*
*-----------------------------------------
* DRAINAGE AREA 2B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN            |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .98          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     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)=     173.04        52.90
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.80 (ii)    1.83 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .32          .32
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .45          .00            .45 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      59.85        26.00          59.85
     TOTAL RAINFALL   (mm)=      60.65        60.65          60.65
     RUNOFF COEFFICIENT   =        .99          .43            .99

       (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.

--------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 7 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .055       .025
                           .029       .020    |     .081       .030

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)       .98       .45      2.25      59.85
     OUTFLOW: ID= 7 (2025)       .98       .08      2.50      59.68

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 17.999
```

```
                  TIME SHIFT OF PEAK FLOW          (min)= 15.000
                  MAXIMUM  STORAGE   USED        (ha.m.)=   .030

-------------------------------------------------------------------------------
*
*-----------------------------------------
* DRAINAGE AREA 3B - DEVELOPMENT INTENSIFICATION |
*-----------------------------------------
--------------------
| DESIGN             |
| STANDHYD  (2040) |   Area    (ha)=    .79
|ID= 4 DT= 5.0 min |   Total Imp(%)= 100.00   Dir. Conn.(%)= 100.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .79          .00
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.00         1.00
     Length            (m)=      72.46        40.00
     Mannings n          =       .013         .250

     Max.eff.Inten.(mm/hr)=     173.04        52.90
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.69 (ii)    1.72 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .32          .32
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .37          .00            .37 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.25           2.25
     RUNOFF VOLUME    (mm)=      59.85        26.00          59.85
     TOTAL RAINFALL   (mm)=      60.65        60.65          60.65
     RUNOFF COEFFICIENT   =        .99          .43            .99

       (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.

-------------------------------------------------------------------------------
*
--------------------
| RESERVOIR (2045) |
| IN= 4---> OUT= 9 |
| DT=  5.0 min     |      OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000        .000   |     .044       .020
                            .024        .016   |     .065       .024

                              AREA     QPEAK     TPEAK       R.V.
                              (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 4 (2040)     .79       .37      2.25      59.85
     OUTFLOW: ID= 9 (2045)     .79       .06      2.50      59.65

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 17.730
                  TIME SHIFT OF PEAK FLOW          (min)= 15.000
                  MAXIMUM  STORAGE   USED        (ha.m.)=   .024

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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.

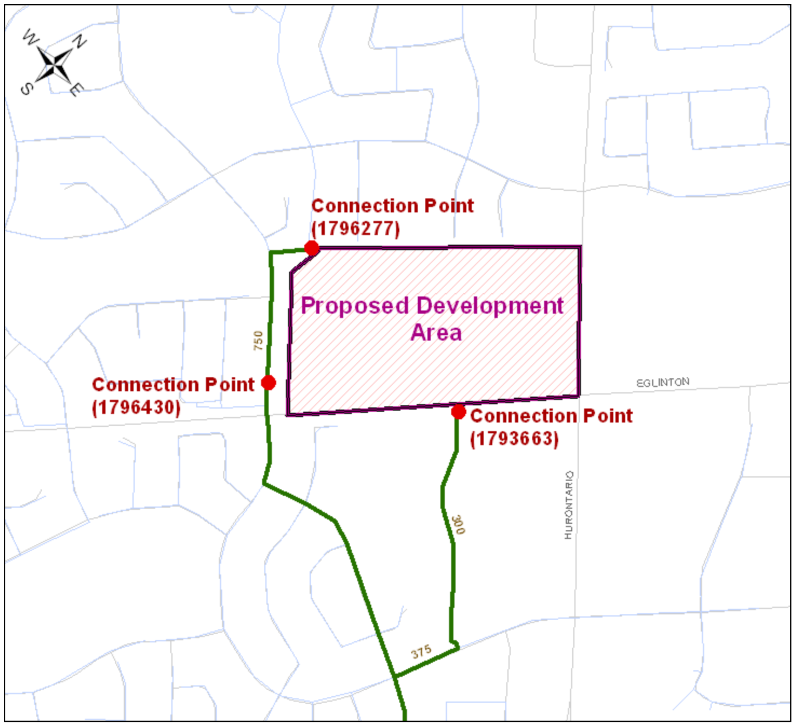


Figure 2 - Sanitary System Overview & Possible Connection Points

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. <u>Hydraulic Impact Assessment Basis</u>

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

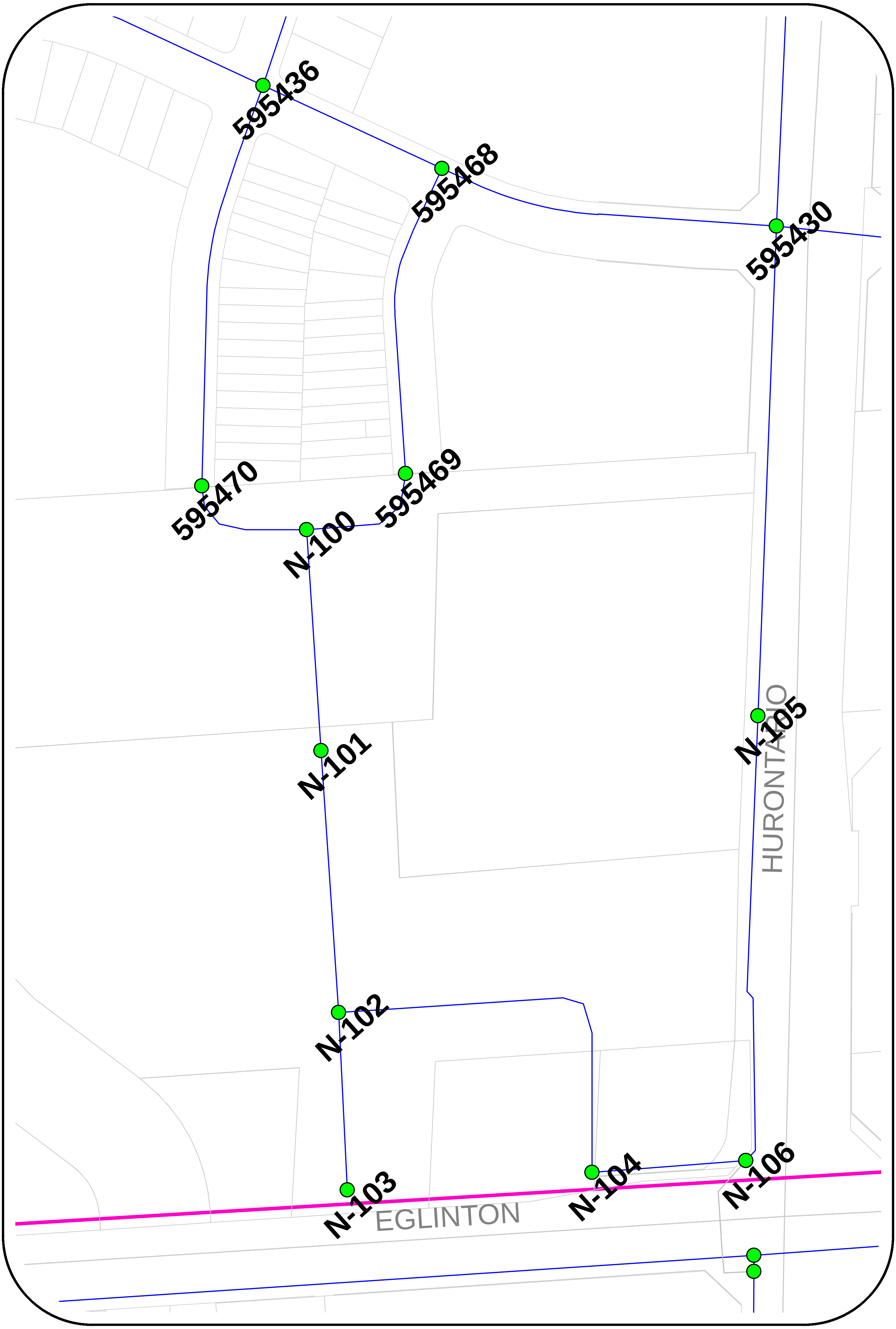
Appendix 1: Detailed Demand Calculations

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

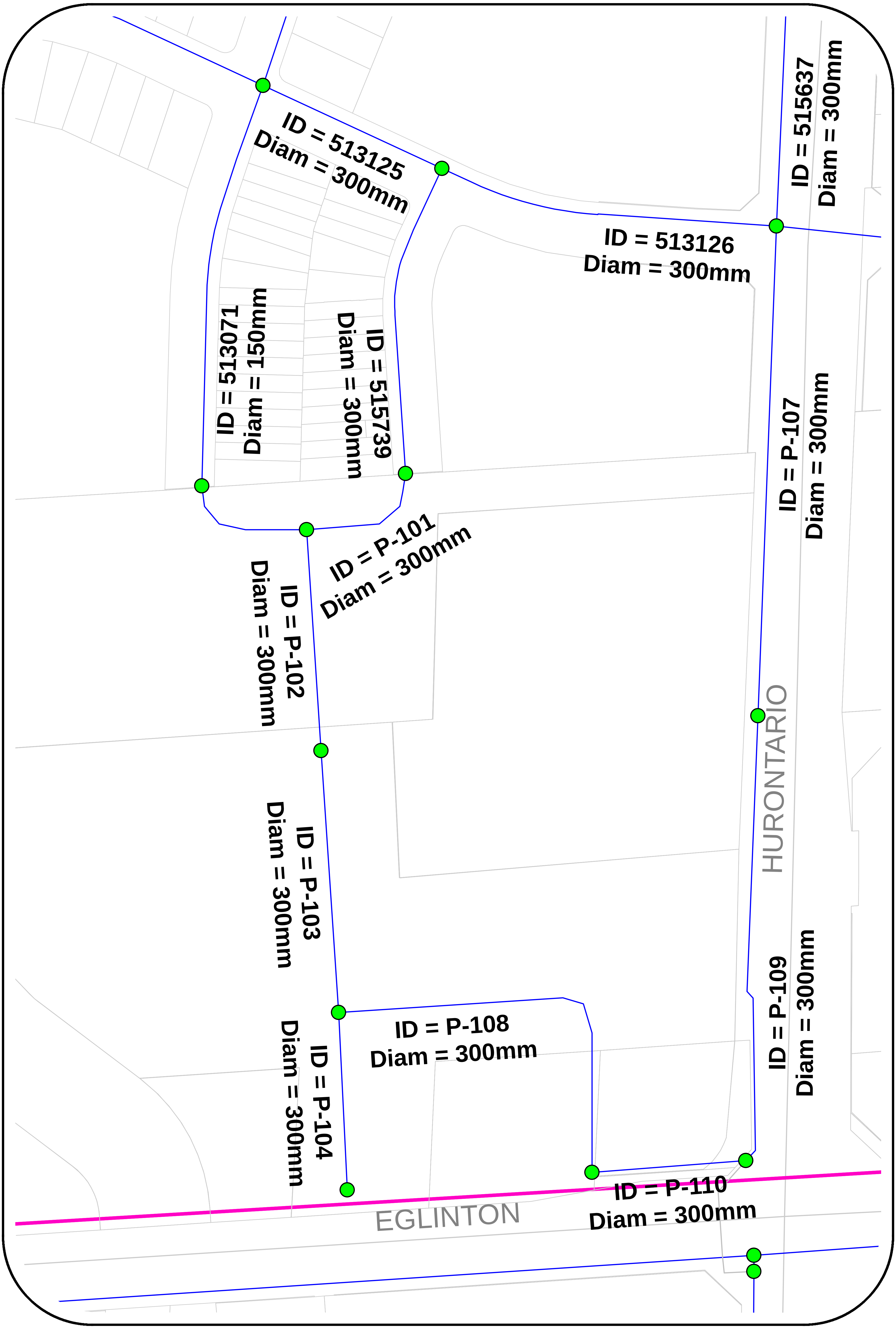
West Parcel	Block	Area (ha)
West Parcel	Block 8 - Hazard Land	0.58
	Block 9 -Open Space	4.14
	Road	0.2
West Parcel Total		4.92

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

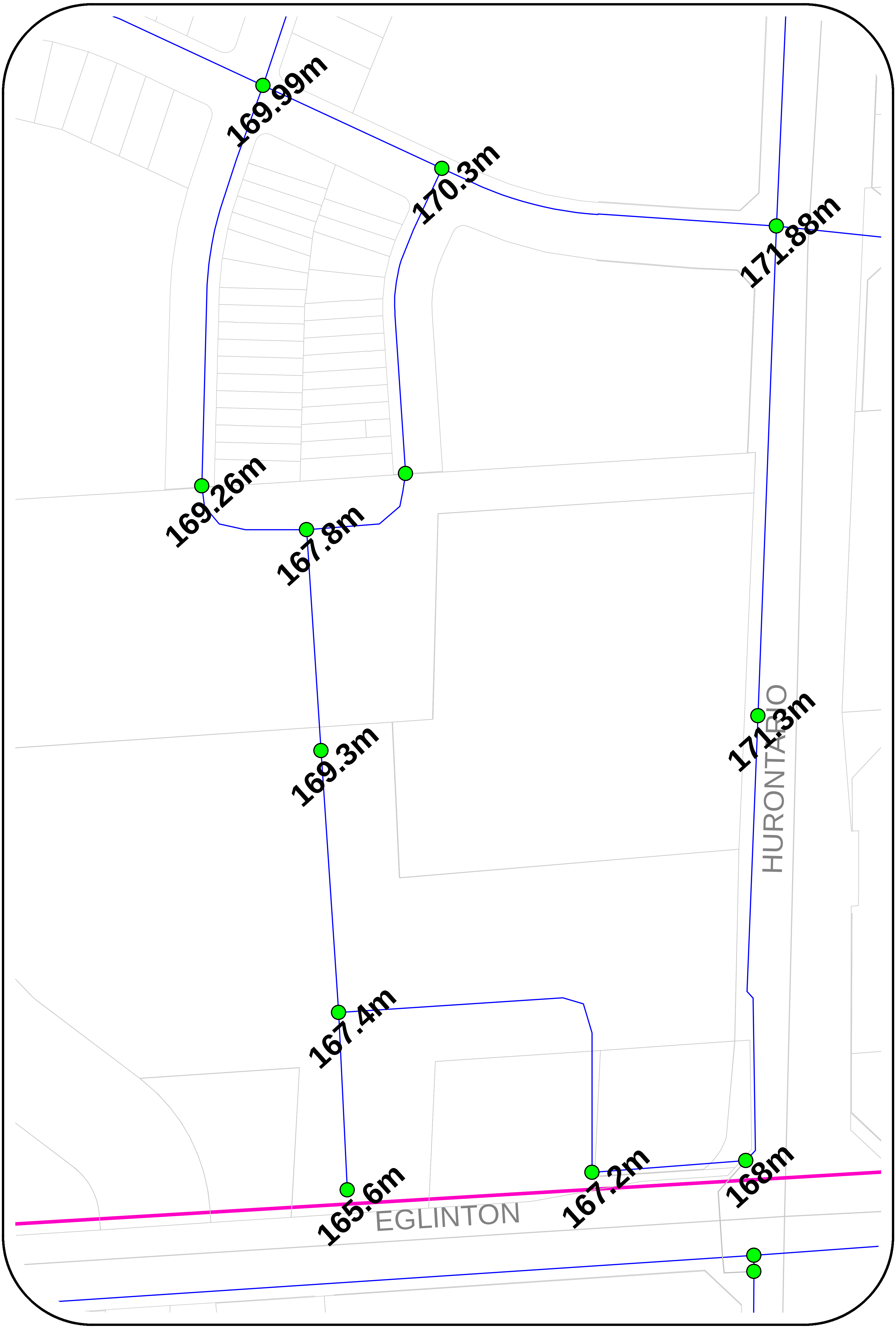
2016	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.00	240.57	233.04	231.93	227.27	224.83
595470	241.00	240.57	233.03	231.91	227.24	224.79
N-100	241.00	240.57	233.04	231.91	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.

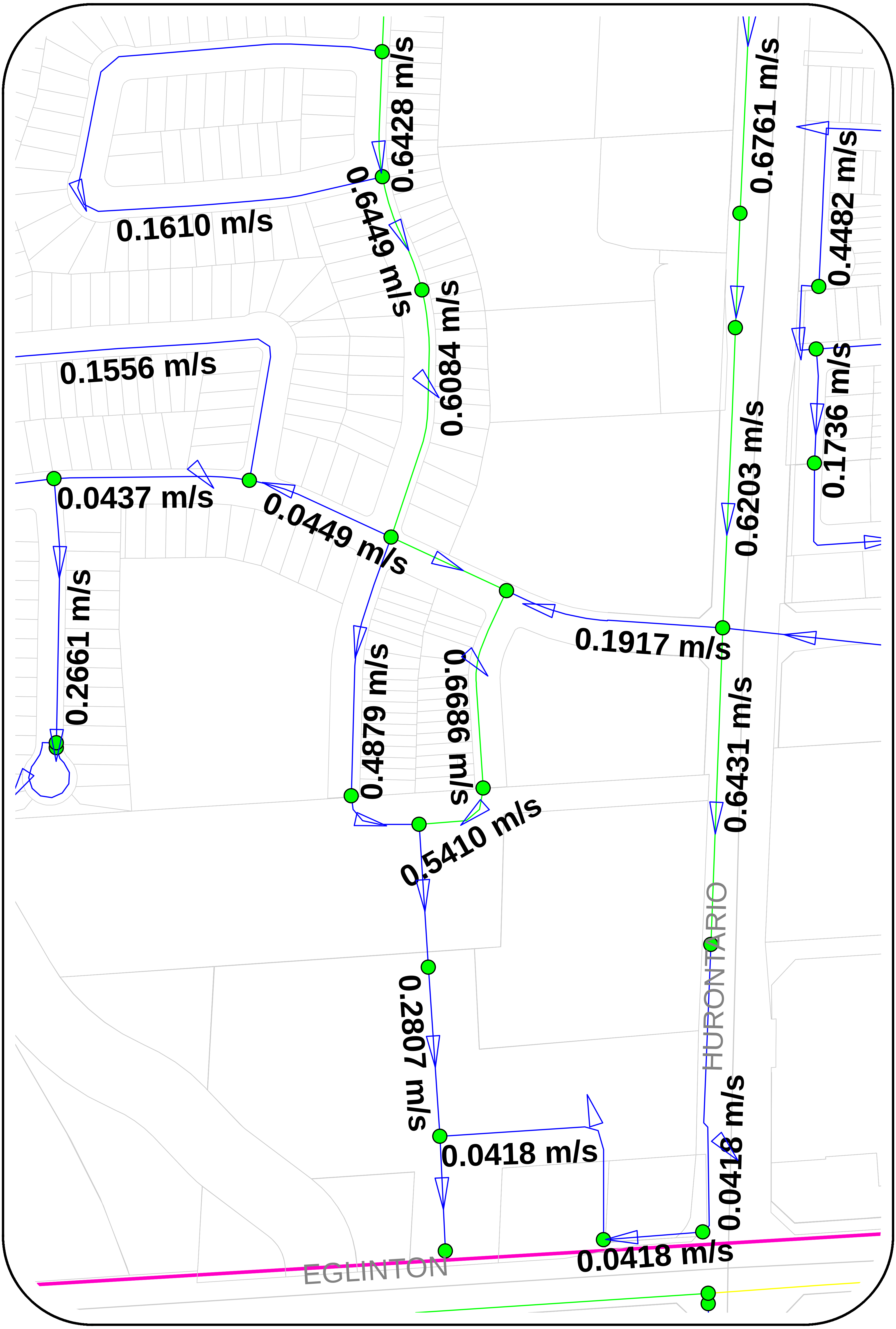
2031	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.36	240.91	244.83	243.73	227.16	224.55
595470	241.36	240.91	244.82	243.71	227.13	224.50
N-100	241.36	240.91	244.83	243.71	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.

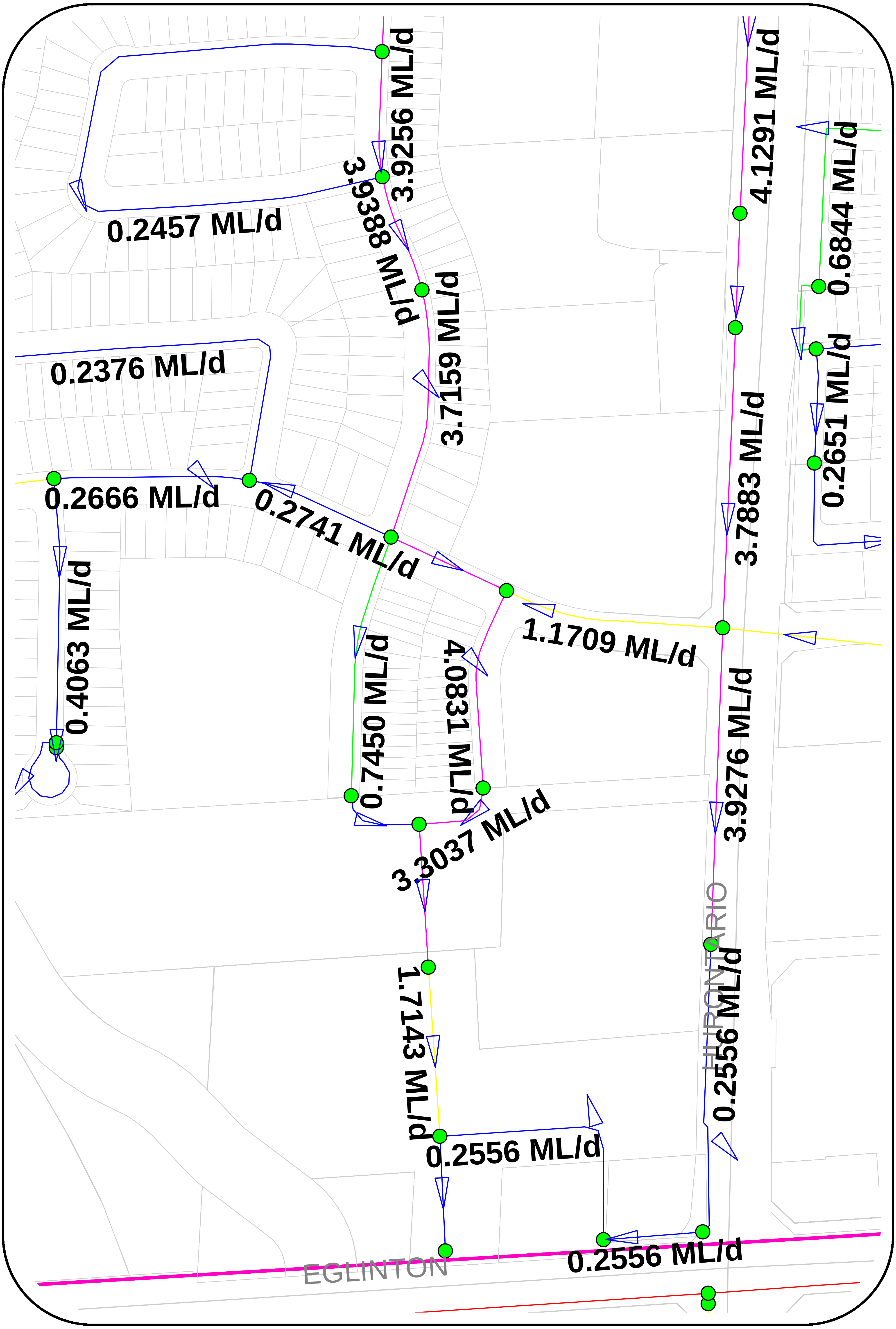
Pinnacle Uptown Mississauga Development
Appendix 5: Node Elevations



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

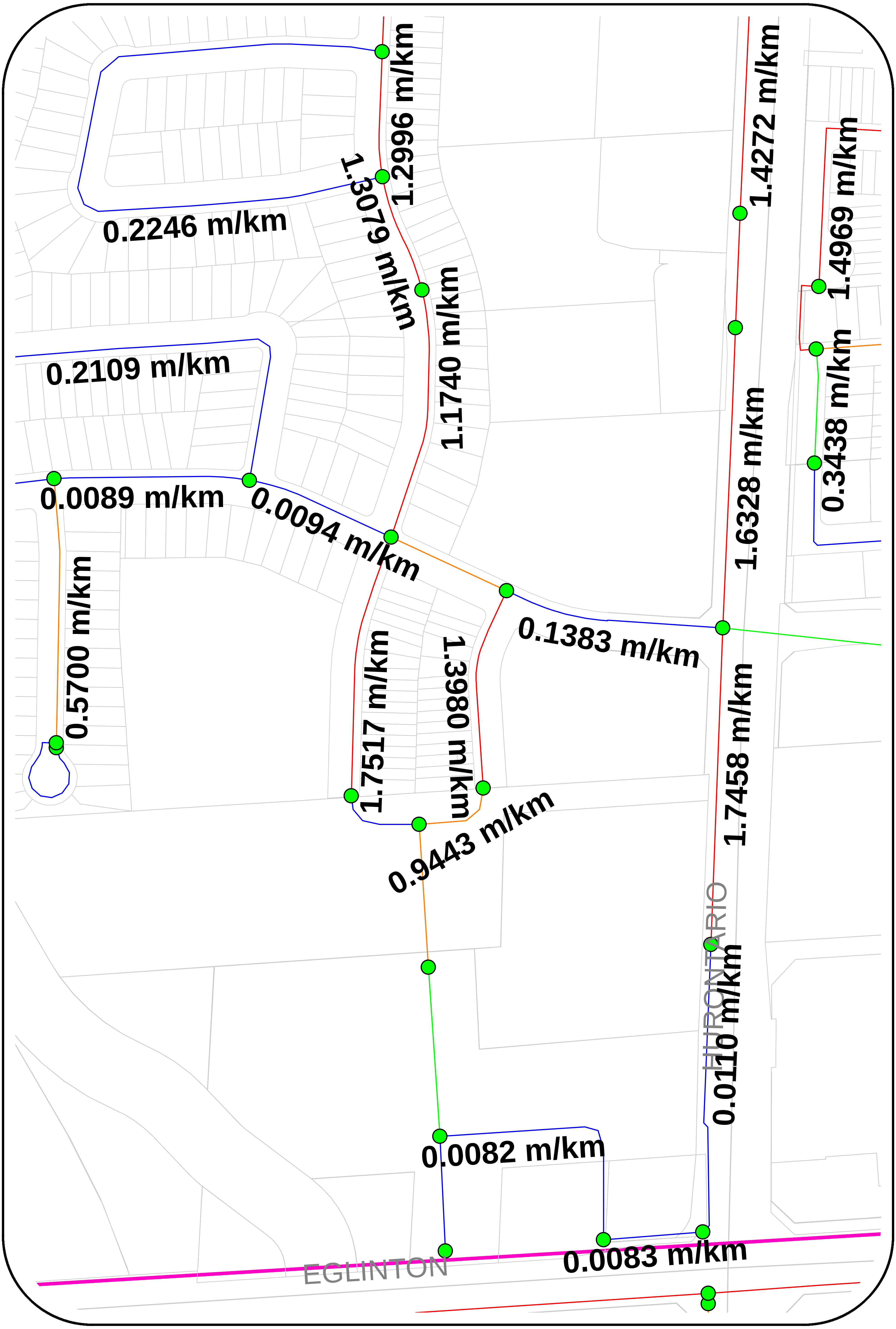


Pinnacle Uptown Mississauga Development
Appendix 7: 2031 Peak Hour Demand - Flow



Pinnacle Uptown Mississauga Development

Appendix 8: 2031 Peak Hour Demand - Headloss Gradient



Appendix 9: Detailed Fire Flow Results

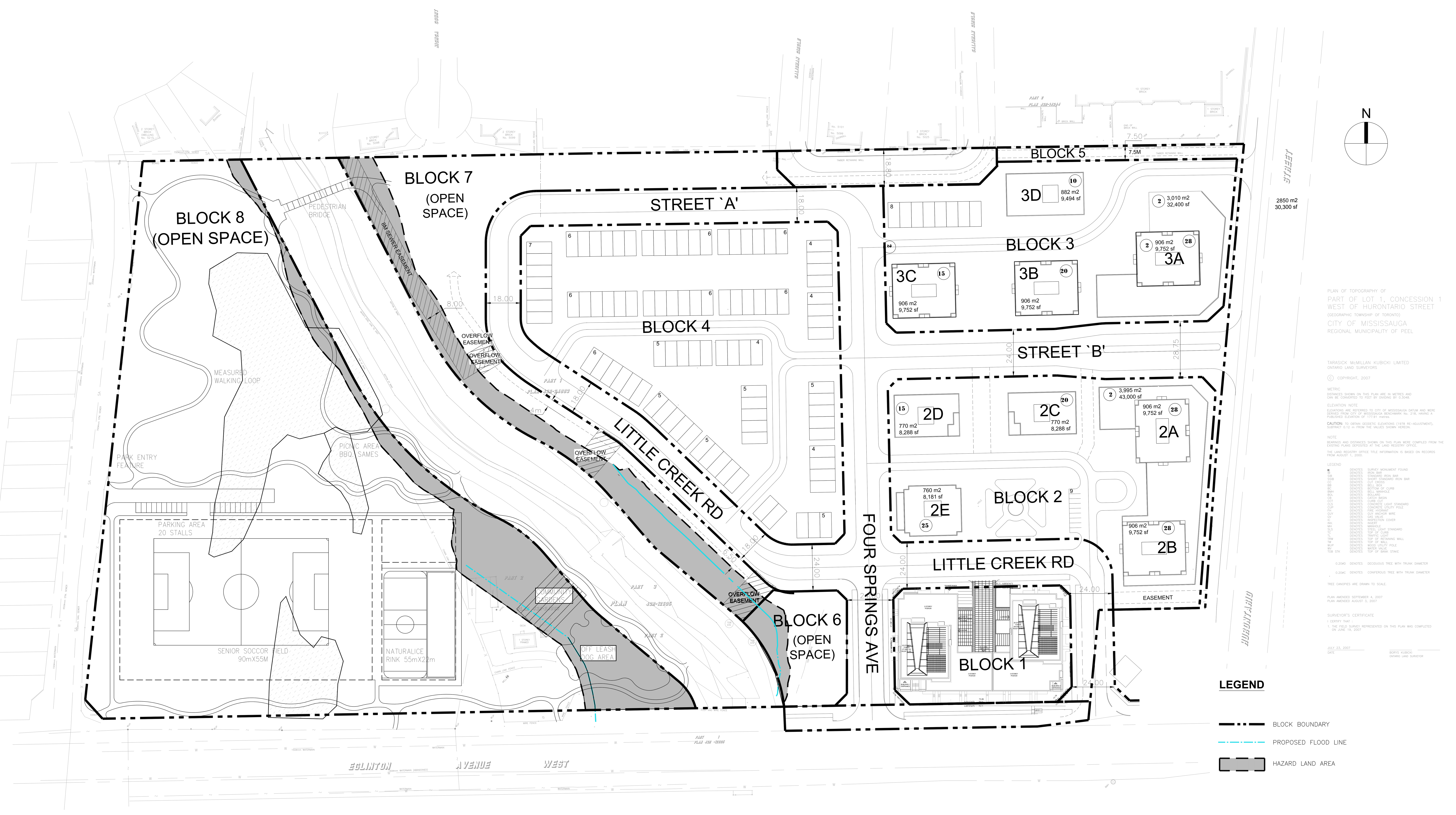
Analyses With All Pipes Active								
2016 -MDD + Fire Flow								
ID	Static Demand (ML/d)	Static Pressure (psi)	Static Head (m)	Fire-Flow Demand (ML/d)	Residual Pressure (psi)	Available Flow at Hydrant (ML/d)	Available Flow Pressure (psi)	Available Flow at Hydrant (L/s)
N-100	0.29	91.14	231.91	21.60	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 DEVE 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 :** -



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

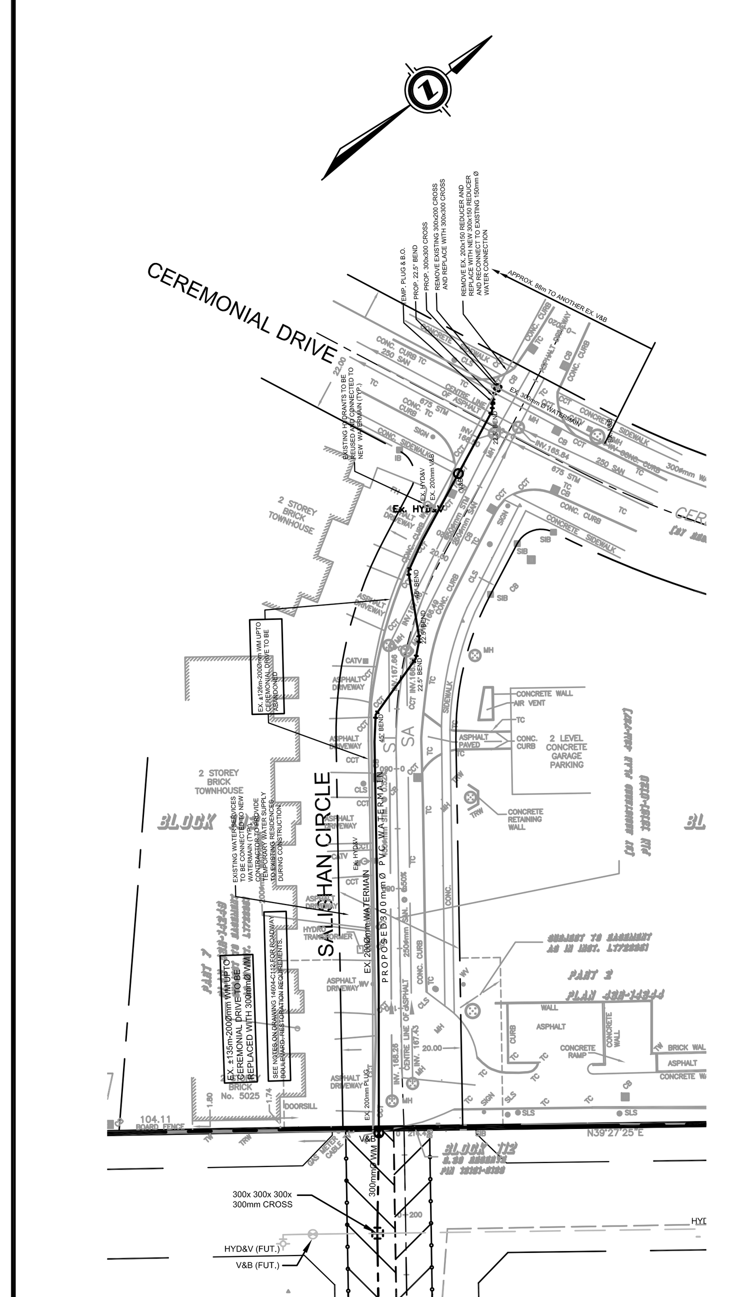
230 Richmond Street West
5th Floor
Toronto, Ontario
Canada M5V 1V6
Tel (416)596-1930
FAX(416)596-0644

Pinnacle International (Ontario) Limited

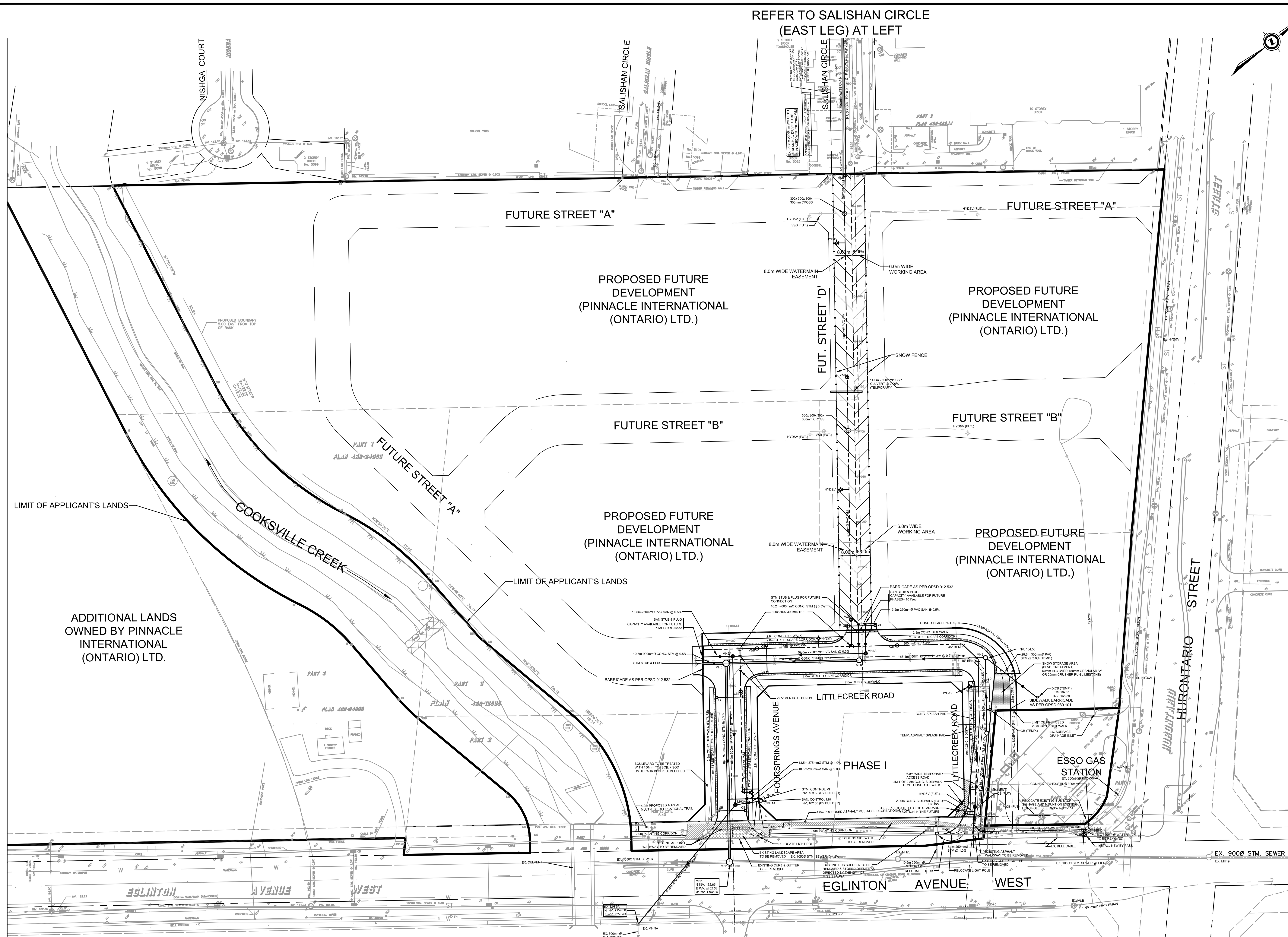
PINNACLE UPTOWN MISSISSAUGA

PRELIMINARY MASTER PLAN

Design:		Drawn:	GG	Checked:	PL
					TO-9368 (14604)
Scales:	HOR: 1:1000 (m)			Date: SEPT 2010	Drawing Number SK-01
					Sheet Set No.



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

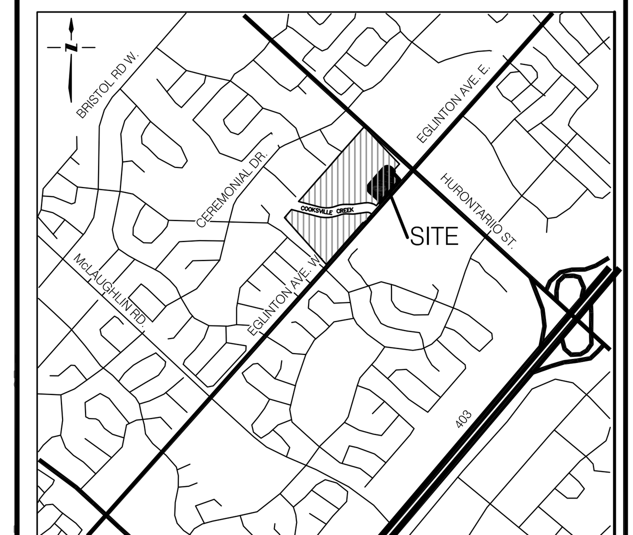
LEGEND:

- PROPERTY BOUNDARY
- PHASE I BOUNDARY
- EX. MH1 — EXISTING STORM SEWER
- MH1 — PROPOSED STORM SEWER SERVICING
- CONTROL MH (BY BLOCK DEVELOPER)
- EASEMENT
- MH1A — PROPOSED SANITARY SEWER & MH.
- EX. MH4 — EX. SANITARY SEWER & MH.
- CMH7A — PROPOSED SANITARY SEWER BLOCK CONNECTION & CONTROL MANHOLE
- V&B — PROPOSED WATER MAIN
- EX. WATERMAIN
- HYD&V — 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)

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

SUBBMISSION	DWG No

DESIGN BY — CHKD — APPROVED BY

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

MISSISSAUGA
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
J:\14604_PinnacleDev\5.0 Design Phase\5.9 Drawings\59civil\layouts\14604-C100B.dwg

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

J:\14604_PinnacleDev\5.0 Design Phase\5.9 Drawings\59civil\layouts\14604-C100B.dwg Plot Date: 2010/11/30
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

Detailed Demand Calculations

East Parcel	Block	Area (ha)	BLDG No.	Bldg Description	Commercial		# of Units			Population	
					Retail/Office SQM	Comm. ADD	Condo units	Town	BLK Total	Residential	ICI
Phase 2	Block 2	0.55	A	West building	0.0	0	178	0	468	481	0
		0.59	B	East building	0.0	0	290	0		783	0
	Block 16	0.71	C	South building	755.8	0	527	0	935	1423	4
			D	North building	1275.0	0	408	0		1102	6
	Block 1	0.35	2	North building	0.0	0	212	0	740	572	0
		0.28	E		0.0	0	294	0		794	0
		0.54	1	South building	0.0	0	234	0		632	0
	Block 17	0.48	F	North building	3311.0	0	354	0	866	956	17
		0.41	G	South building	5321.0	0	512	0		1382	27
East Parcel Total		3.91			10663	0	3009	0	3009	8124	53

As per Aecom's report:

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 =	2.7	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

For Phase 4 Part 2 and Phase 5

Short term residential water demand	26.8 L/s
Commercial average day demand	0.19 L/s