Functional Servicing Report

Pinnacle International (Ontario) Limited

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

Prepared for Pinnacle International
by IBI Group Professional Services (Canada) Inc.
March 22, 2019

Table of Contents

Table of Contents (continued)

TABLES

FIGURES

APPENDICES

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/ZBLA. The FSR will also be used to support any site alterations that may be required for the proposed site plan applications.

1.2 Subdivision Design

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

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

Proposed intensification is concentrated fully on Blocks 16 and 17 and on portions of Blocks 1 and 2. Figure 1 displays the proposed condominium towers. One proposed modification from the original submission includes an additional building in Block 16, part of Phase IV Part 2. The other proposed modification includes an additional building in Block 17, part of Phase V. Phase V also includes a small portion of Block 1. In Block 2, both buildings have increased in Gross Floor Area.

The proposed subdivision concept contemplated development of 3 Development Blocks as shown in Appendix “A”. The original staging plan is still in effect and the proposed 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 plan in Figure 1.

2. PROPERTY DESCRIPTION

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

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

3. EXISTING DATA COLLECTION/BACKGROUND REPORTS

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

- IBI Group Phase II approved engineering drawings, approved July 2013.
- IBI Group Functional Servicing Report for Phase II OPA / Rezoning / Draft Plan of Subdivision, dated January 2011
- Approved storm and sanitary sewer design sheets, approved July 2013. The approved storm sewer design sheet may be found in Appendix “D”.
- Suite summary and commercial areas, prepared by Richmond Architects in March 2019, used as parameters to update the sanitary sewer design sheet. This may be in found in Appendix “B”.
- Aecom Technical Memorandum, Jan. 14, 2011 prepared to update their previous assessment of off site water and sewer capacity and availability now or in the future, of the water and sanitary infrastructure to meet this project’s demands. This includes updates to water modeling as requested by the Region of Peel in comments on the previous submission. This memorandum is attached as Appendix “G”. When referring to this memorandum, note that it was prepared for the original subdivision plan.
- Hydrant Flow Test prepared by GTA Waterworks on June 2017. Accompanied by Detailed Hydrant Flow calculation in Table F1 prepared by Masongsong Engineering Associates Engineering Limited on December 2017. This may be in found in Appendix “H”.

4. SITE SERVICING

4.1 Proposed Site Servicing

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

4.1.1 PHASE IV PART 2

This phase of the development is comprised of two Towers (45 stories - Building 3 and 32 stories Building 4). The Towers will be linked at the ground level with a 5-storey podium. The two towers will have a total contemplated number of residential units of 883 units, with 0.30 ha of commercial/retail space area. This equivalent population for this phase is:

Site Specific Population = 2.7 ppu x 883
= 2,384 persons

Commercial Population = 3011.5 m^2 x (50 persons/10,000m^2)
= 15 persons

Therefore, the total population for this Phase IV Part development is **2,399 persons.**

4.1.2 PHASE V

The total number of residential units contemplated for this phase 883 units, with 0.30 ha area of commercial/retail space. This results in an equivalent population:

Site Specific Population = 2.7 ppu x 1,288
= 3,477 persons

Commercial Population = 8623.8 m^2 x (50 persons/10,000m^2)
= 43 persons

Therefore, the total population for this Phase IV Part development is **3,520 persons.**

4.2 Sanitary Drainage System

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

4.2.1 WEST TRUNK

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

The current capacity of the west trunk is 212.2 L/s less the 76L/s which have been allowed for the ultimate design build out. The current design flows provide total development flow of 78.6L/s for the ultimate build out the subdivision. This is approximately 3.5% increase in flows and comprises approximately 38% of the west trunk capacity. See Appendix “C” for details.

4.2.2 EAST TRUNK

As per the approved master sewerage plan part of the Phase V development is serviced via 250mm diameter sanitary sewer along Littlecreek Road. The sewer ultimately discharges into an existing 300 mm diameter sanitary sewer located in an easement on the east side of the creek on the south side of Eglinton Avenue West. The sewer allowance for the Pinnacle development is 37.5 l/sec. The additional flows by the intensification will increase from the 31 L/s to 37.4 l/s, as shown on the Sanitary Sewer Design Sheet (Appendix “C”). This will enable the servicing of the development including the intensification (Phase I a portion of Phase II) as shown on Figure 2.

4.3 Storm Water Management

4.3.1 SITE CHARACTERISTICS

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

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

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

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

4.3.2 STORMWATER MANAGEMENT DESIGN CRITERIA

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

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

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

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

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

4.3.3 STORMWATER MANAGEMENT PLAN

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

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

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

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

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

   Under the current development plan, the landscape area occupies approximately 33.5% of the 0.9280 ha Phase IV Part 2 development (including the northeast corner of Phase IV Part 1), and 18.9% of the 0.9767 ha Phase V development. These percentages are relatively high, considering the nature of the proposed mixed use development.

b) Under the current development plan, a significant portion of the Phase IV Part 2 and Phase V developments is occupied by the proposed buildings (approximately 40.1% for the Phase IV Part 2 development and approximately 61.2% for the Phase V development). As currently proposed, the multi-level underground parking structures are proposed within the Phase IV Part 2 and Phase V developments. Since the parking structures are shallow in depth from the ground surface and their footprints occupy the entire Phase IV Part 2 (also into the northeast corner of Phase IV Part 1) and Phase V development blocks, it significantly limits types of LID measures that are physically feasible or suitable to be used within the developments.

   Considering high imperviousness and constraints of the proposed development, green roofs, landscaped terraces and outdoor recreation with landscape features are proposed within the Phase IV Part 2 and Phase V developments (in addition to the relatively large landscape area proposed) to increase evapotranspiration and runoff retention on site. As currently proposed, the building area that is covered by proposed green roofs, landscaped terraces and outdoor recreational landscape features is approximately 24.3% of total building area for the Phase IV Part 2 development and 15.8% for the Phase V development, subject to the detailed design.

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

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

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

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

4.3.4 ON-SITE RETENTION AND DETENTION STORAGE REQUIREMENTS

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

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

Table 1. Results of Water Balance Analysis for Phase IV Part 2 Development

Type of Land Use w ithin Expansion Area	Surface Area of Landuse (m^2)	Landuse Area as Percentage of Entire Area (-)	Initial Abstraction for Landuse (mm)	Initial Abstraction over Entire Area (mm)
	Part I: Water Balance Analysis w ith Proposed Green Roofs and Landscape Terraces/Features			
Roofs and Other Hard Surface Areas	5,263	57%	0.0	0.0
Green Roofs , Landscape Terraces/Features and Landscape Areas	4,017	43%	5.0	2.2
Total =	9,280	100%	NOT OK	2.2 < 5.0
Type of Landuse (Stormw ater Retention Measures)	Retention Storage Required for Landuse (mm)	Retention Storage Required for Landuse (m^3)	Retention Storage Required over Entire Sarea (mm)	
	Part II: Additional Retention Storage Required to Meet Runoff Volume Reduction Design Criteria			
Roofs and Other Hard Surface Areas	5.0	26.3	2.8	
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	0.0	
Total =		26.3	2.8	
Type of Landuse (Stormw ater Retention Measures)	Retention Storage Provided for Landuse (mm)	Retention Storage Provided for Landuse (m^3)	Initial Abstraction & Retention Storage Provided for Landuse (mm)	Initial Abstraction & Retention Provided over Entire Area (mm)
	Part III: Water Balance Analysis under Stormw ater Management Plan			
Roofs and Other Hard Surface Areas to Rainw ater Harvesting Tank 1	5.1	27.0	5.1	2.9
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	5.0	2.2
Total =		27.0	OK	5.1 > 5.0

The results of the water balance analysis indicate that, even with the proposed landscape areas, green roofs, landscaped terraces and outdoor recreational landscape features, the runoff volume reduction design criteria cannot be satisfied for the Phase IV Part 2 development without additional runoff retention measures (see Part I of Table 1), with retention storage shortage of approximately 26.3 m^3 (see Part II of Table 1).

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

Under the current stormwater management plan, total design retention storage of 27.0 m^3 (see Part III of Table 1) is proposed for Rainwater Harvesting Tank 1, and it is higher than additional retention storage of 26.3 m^3 required for retaining the first 5.0 mm of runoff for the development. In another word, the desired runoff volume reduction control design criteria can be satisfied for the Phase IV Part 2 development with the proposed stormwater management plan in place.

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

With the proposed green roofs, landscaped terraces and other landscape measures, water balance analysis was conducted for the purpose of retaining the first 5 mm of runoff on site within the Phase V development, and the results are summarized in Table 2.

Table 2. Results of Water Balance Analysis for Phase V Development

Type of Land Use within Expansion Area	Surface Area of Landuse (m^2)	Landuse Area as Percentage of Entire Area (-)	Initial Abstraction for Landuse (mm)	Initial Abstraction over Entire Area (mm)
	Part I: Water Balance Analysis with Proposed Green Roofs and Landscape Terraces/Features			
Roofs and Other Hard Surface Areas	6,982	71%	0.0	0.0
Green Roofs , Landscape Terraces/Features and Landscape Areas	2,785	29%	5.0	1.4
Total =	9,767	100%	NOT OK	1.4 < 5.0
Type of Landuse (Stormwater Retention Measures)	Retention Storage Required for Landuse (mm)	Retention Storage Required for Landuse (m^3)	Retention Storage Required over Entire Sarea (mm)	
	Part II: Additional Retention Storage Required to Meet Runoff Volume Reduction Design Criteria			
Roofs and Other Hard Surface Areas	5.0	34.9	3.6	
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	0.0	
Total =		34.9	3.6	
Type of Landuse (Stormwater Retention Measures)	Retention Storage Provided for Landuse (mm)	Retention Storage Provided for Landuse (m^3)	Initial Abstraction & Retention Storage Provided for Landuse (mm)	Initial Abstraction & Retention Provided over Entire Area (mm)
	Part III: Water Balance Analysis under Stormwater Management Plan			
Roofs and Other Hard Surface Areas to Rainwater Harvesting Tank 2	5.1	35.5	5.1	3.6
Green Roofs , Landscape Terraces/Features and Landscape Areas	0.0	0.0	5.0	1.4
Total =		35.5	OK	5.1 > 5.0

Even with the proposed green roof, landscaped terraces and other landscaped features, the results of the water balance analysis indicate that the runoff volume reduction control design criteria cannot be satisfied for the Phase V development without additional runoff retention measures (see Part I of Table 2), with retention storage shortage of approximately 34.9 m^3 (see Part II of Table 2).

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

Under the current development and stormwater management plan, the design stormwater retention storage of 35.5 m^3 (see Part III of Table 2) is proposed for Rainwater Harvesting Tank 2, and it is slightly higher than the additional retention storage requirement of 34.9 m^3 to retain the first 5 mm of runoff within the development. Therefore, the storm runoff volume reduction control design criteria can be satisfied for the Phase V development with the proposed stormwater management plan.

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

The results of the detailed hydrologic analysis indicate that the pre-development unit runoff peak flows are 30.1, 83.1 and 151.5 l/s/ha under the 1:2, 1:10 and 1:100-year design storms respectively. Considering the development areas of 0.9280 ha and 0.9767 ha for the Phase IV Part 2 (Drainage Area 3B and 3A2 as shown in **Figure 3**, including the northeast corner of Phase IV Part 1) and Phase V (Drainage Area 2B) developments respectively, their corresponding pre-development runoff peak flows are calculated and summarized in Table 3 for the 1:2, 1:10 and 1:100-year design storms. To satisfy the desired water quantity control design criteria, it is required to control the 1:100 year post-development runoff peak discharges from the Phase IV Part 2 and Phase V developments to their corresponding 2-year pre-development levels of 0.028 m^3/s and 0.029 m^3/s respectively (see Table 3).

Table 3. Pre-Development Design Peak Flows for Phase IV Part 2 and Phase V Developments

Drainage Area Identification Number (-)	Drainage Area (ha)	Total Imperviousness (-)	Pre-Development Design Peak Flows (m^3/s)		
			2-Year Design Storm	10-Year Design Storm	100-Year Design Storm
Area 2B (Phase V)	0.9767	71.5%	0.029	0.081	0.148
Area 3B and 3A2 (Phase IV Part 2 and Northeast Corner of Phase IV Part 1)	0.9280	56.7%	0.028	0.077	0.141

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

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

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

The main post-development hydrologic model parameters and the estimated design stormwater detention storage required for Phase IV Part 2 and Phase V developments are summarized and presented in Table 4. A copy of the post-development OTTHYMO model output is included in **Appendix “F”**.

Table 4. Estimates of On-Site Detention Storage

Requirements for Phase IV Part 2 and Phase V Developments

Drainage Area Identification Number	Drainage Area (ha)	Total Imperviousness	2-Year Design Storm	10-Year Design Storm	100-Year Design Storm
Area 2B (Phase V)	0.9767	71.5%	Storage Requirement for Stormwater Detention Facility 2 (m^3)		
			190	330	493
			Uncontrolled Post-Development Peak Discharges (m^3/s)		
			0.208	0.359	0.546
			Controlled Post-Development Peak Discharges (m^3/s)		
			0.011	0.020	0.029
			Pre-Development Peak Flows (m^3/s)		
			0.029	0.081	0.148
Area 3B and 3A2 (Phase IV Part 2 and Northeast Corner of Phase IV Part 1)	0.9280	56.7%	Storage Requirement for Stormwater Detention Facility 1 (m^3)		
			150	270	420
			Uncontrolled Post-Development Peak Discharges (m^3/s)		
			0.157	0.274	0.417
			Controlled Post-Development Peak Discharges (m^3/s)		
			0.010	0.018	0.028
			Pre-Development Peak Flows (m^3/s)		
			0.028	0.077	0.141

Considering the development intensification area of 0.9280 ha for the Phase IV Part 2 development (Drainage Areas 3B and 3A2) and 0.9767 ha for the Phase V development (Drainage Area 2B), the results of the post-development hydrologic analysis (see **Appendix "F"** for details) indicate that, under the current development plan, the design detention storage requirement is approximately 420 m^3 (see Table 4) for Stormwater Detention Facility 1 for the Phase IV Part 2 development (including the northeast corner of Phase IV Part 1) and 493 m^3 for Stormwater Detention Facility 2 for the Phase V development to control the 1:100-year post-development runoff peak discharges to their corresponding 2-year pre-development levels of 0.028 and 0.029 m^3/s respectively.

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

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

4.3.5 EROSION AND SEDIMENT CONTROLS

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

- Install silt fence at the downslope side of disturbed areas and snow fence (if necessary) along the perimeter of the development site, prior to the start of construction.

- Install stone mud mats at all construction entrances.

- Stockpile topsoil at designated locations and at least 15 m away from the top bank of the watercourse. Stockpiles will be contained by silt fences on the downslope side.

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

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

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

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

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

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

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

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

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

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

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

- The contractor is responsible to locate and protect existing all existing utilities and municipal services and make arrangements with utility companies prior to construction.
- All excavations shall be in accordance with the Ontario "Occupational Health and Safety Act", and other federal and provincial regulations related to construction projects.

4.4 Water Distribution System (see Figure 4)

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

The detailed demand calculations in the AECOM Hydraulic Modelling Analysis (dated January 14, 2011, see Appendix “G”) was used as a basis to which the additional proposed intensification would contribute to the subdivision water demand. Using the Peel Region Water Demand Criteria, the estimated water demand is summarized in the Table 5 below and detailed in Appendix H.

Table 5. Water Demand Table for Phase IV Part 2 and Phase V Developments

	Typical Water Demand		
	Avg. Day (L/s)	Max Day (L/S)	Peak Hour (L/S)
Phase IV Part II	6.63	13.23	19.90
Phase V	24.58	49.07	73.74
Total Water Demand	**31.21**	**62.30**	**93.64**

The Maximum day demand is based on the subdivision’s residential water demand and commercial average day demand for both development. The total Maximum day demand of 62.30 L/s will be used for domestic flows.

The required fire flow of 83.3 L/s and 100 L/s for Phase IV-Part II and Phase V respectively have ben obtained from the Fire Underwriter’s Survey (FUS).

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.

Further to the above a hydrant flow test, enclosed in Appendix H, was performed in June 2017 to obtain the available municipal flow on Watergarden Drive. The test confirms that the existing 300mm diameter watermain on Watergarden Drive is capable of delivering a fire flow of 14,358 L/min (239.3L/s) at a minimum pressure of 140 KPa, which satisfies both FUS and ISO fire flows superimposed on the max-day domestic consumption rate 7,944 L/min (132.4 L/s). The value is the maximum day from Phase V (being the larger of the two phases) with the fire flow form Phase V (49.07 + 83.33 = 149.07L/s).

As per the Ontario Building Code, structures greater than 84m are to be provided with two independent fire supply lines which can be isolated on separate system. As both development exceed this height (with the smallest being 30 storeys) a second connection to both phases is required. Therefore, we have proposed a new fire connection from Salishan Circle for Phase IV – Part II and a connection from Little Creek for Phase V (see Figure 4 for proposed location). These connections are to be detailed as part of Site Plan Application.

4.5 Site Grading

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

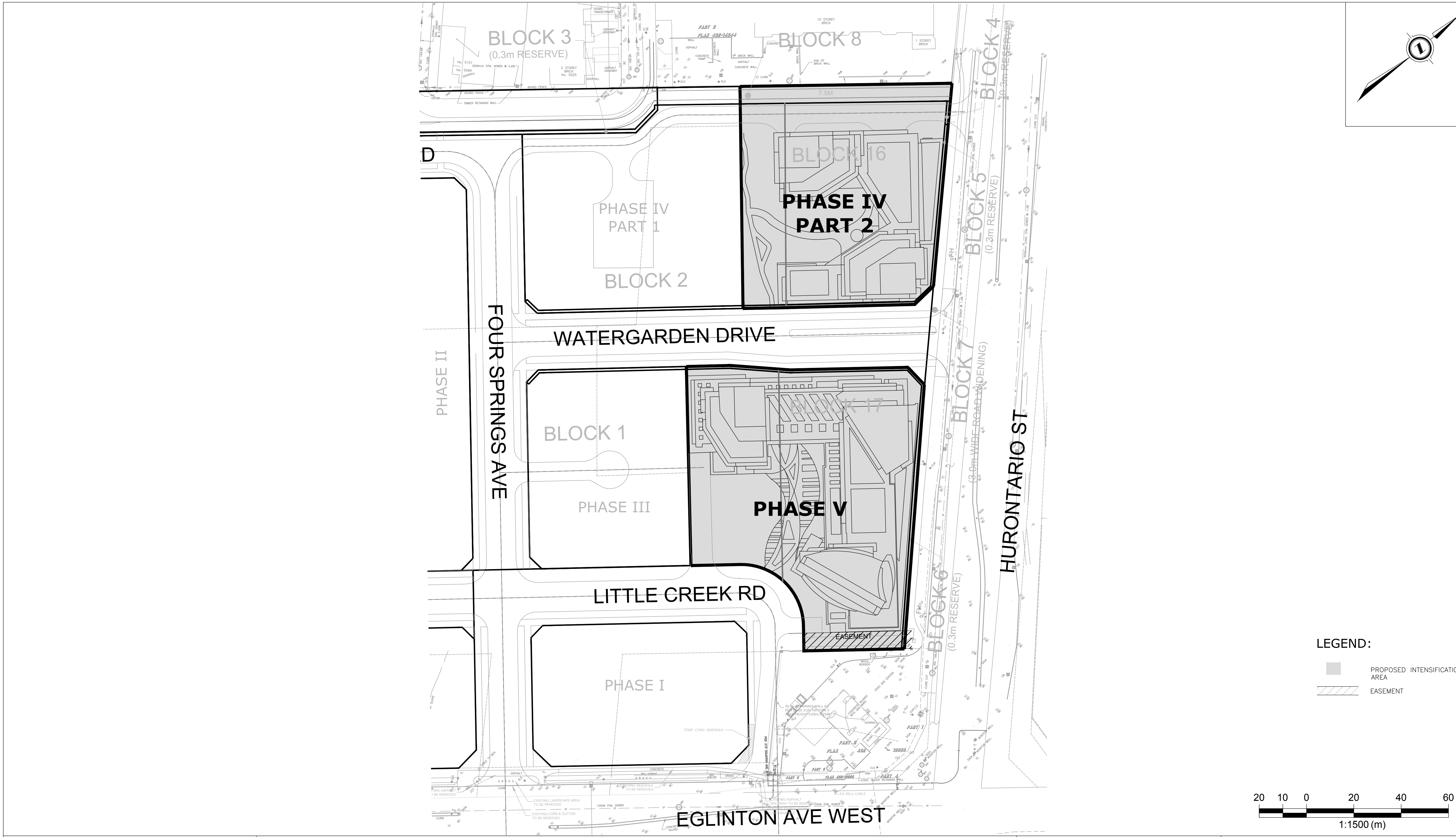
5. CONCLUSION

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

Respectfully submitted by:

IBI Group Professional Services (Canada) Inc.

Angelo Covello, P. Eng.	Nicky Chang, Ph. D.
Associate Director - Practice Lead, Civil Engineering	Water Resources Group



T:\108686_Pinn_Uptown\10.0 Reports\10.11 FSR\Working Files – Appendices and Figures\14604-FSR-PHASE II-FIG-1.dwg 2019-03-25 – 10:19am

PINNACLE
INTERNATIONAL

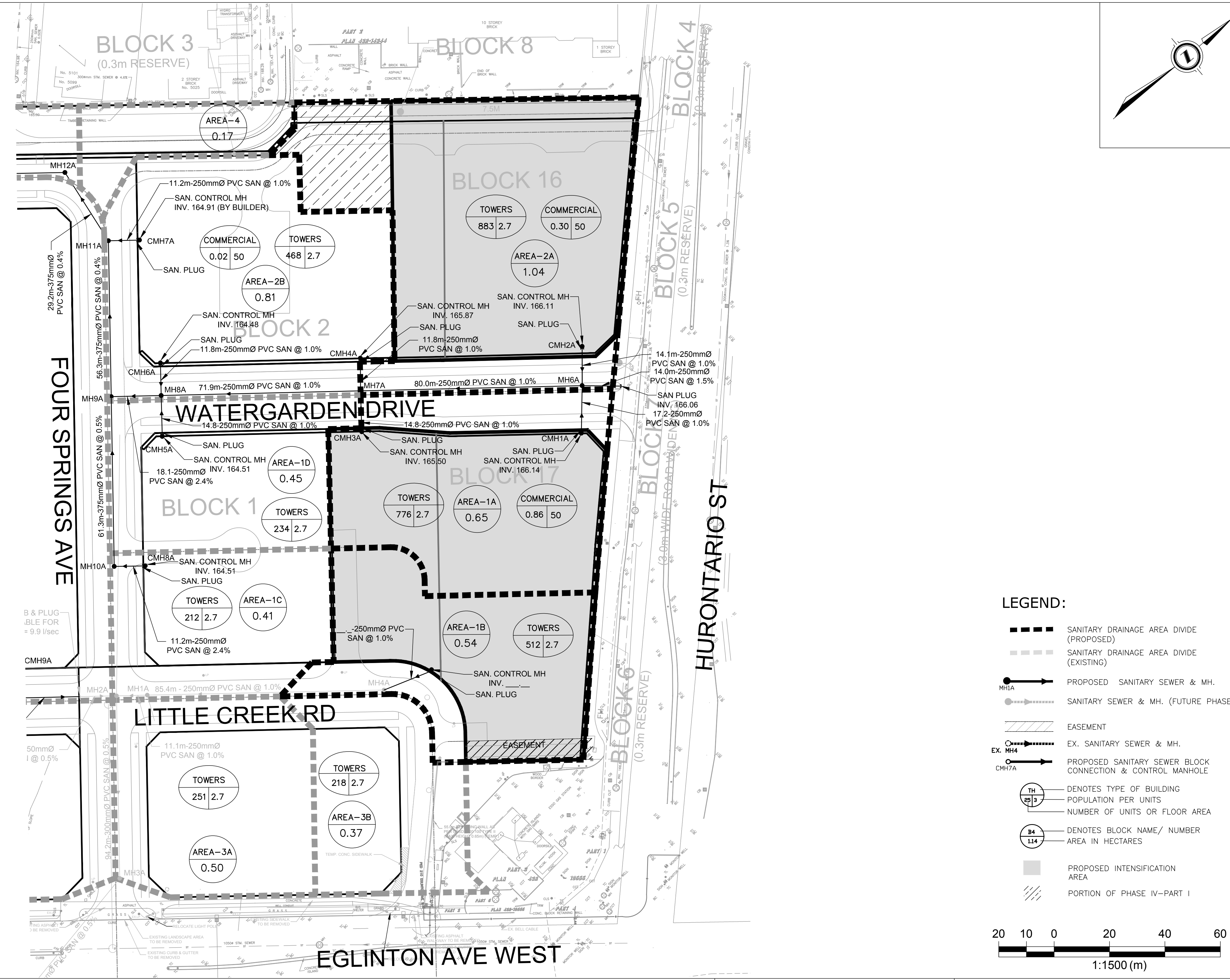
TITLE:

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

JOB No:	108686
SCALE:	1:1500
DATE:	MARCH, 2019
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\Working Files – Appendices and Figures\14604-FSR-PHASE II-FIG-2.dwg 2019-03-25 – 10:32am

PINNACLE
INTERNATIONAL

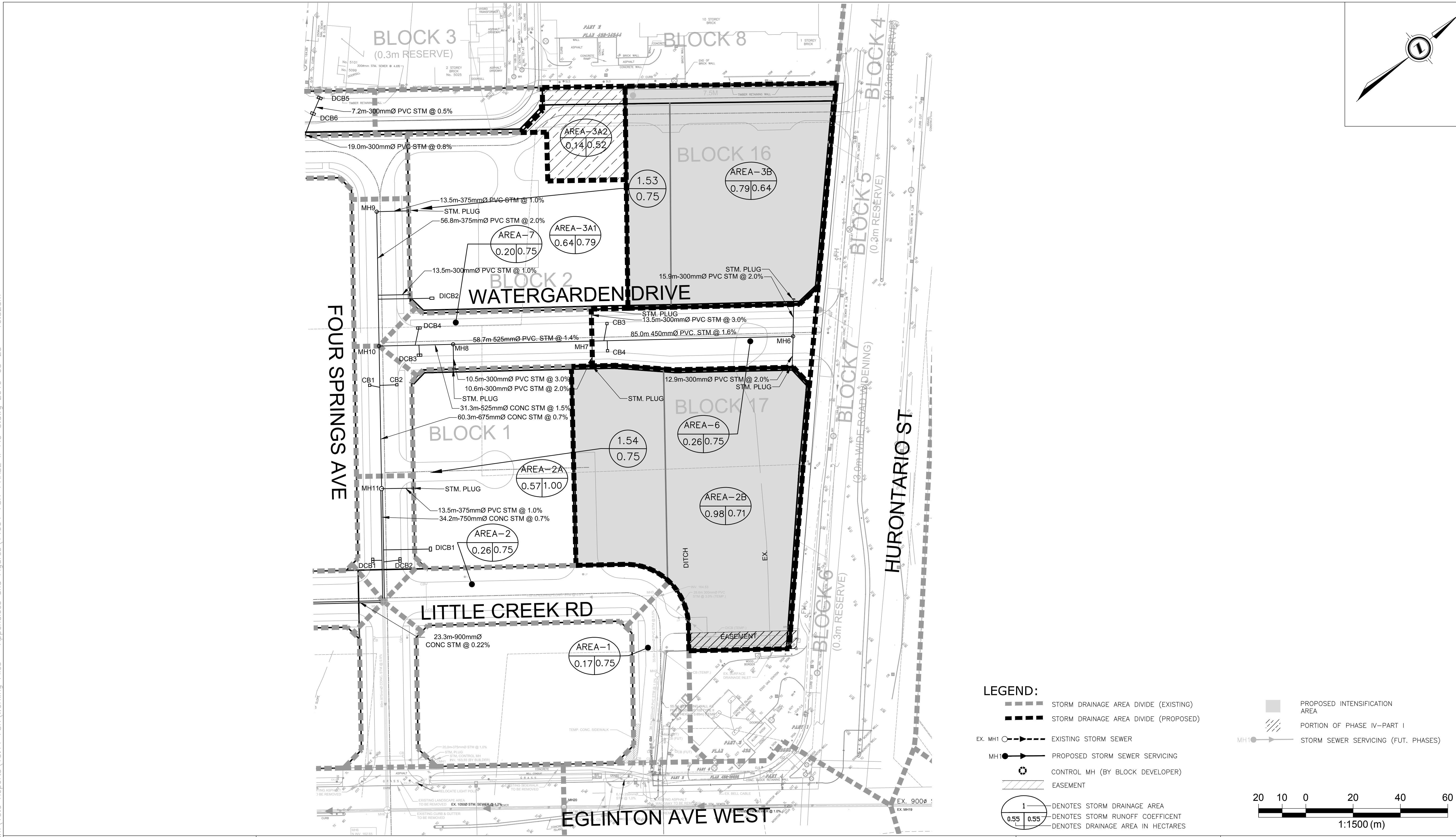
TITLE:

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

JOB No:	108686
SCALE:	1:1500
DATE:	MARCH, 2019
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 II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	MARCH, 2019
FIGURE No:	FIGURE 3

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

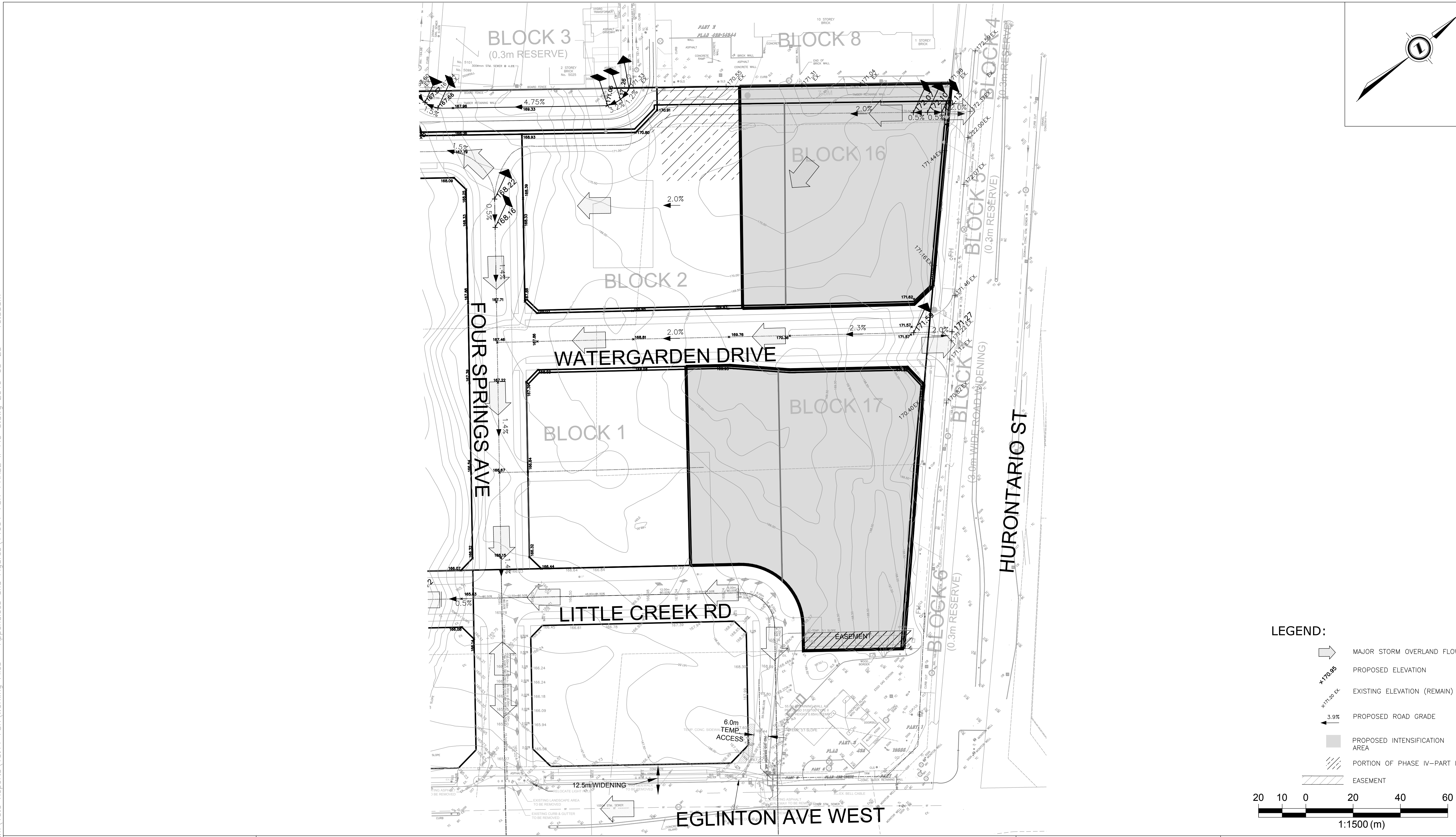
PINNACLE
INTERNATIONAL

TITLE:

WATER DISTRIBUTION SYSTEM
PHASE IV-PART II AND PHASE V

JOB No:	108686
SCALE:	1:1500
DATE:	MARCH, 2019
FIGURE No:	FIGURE 4

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

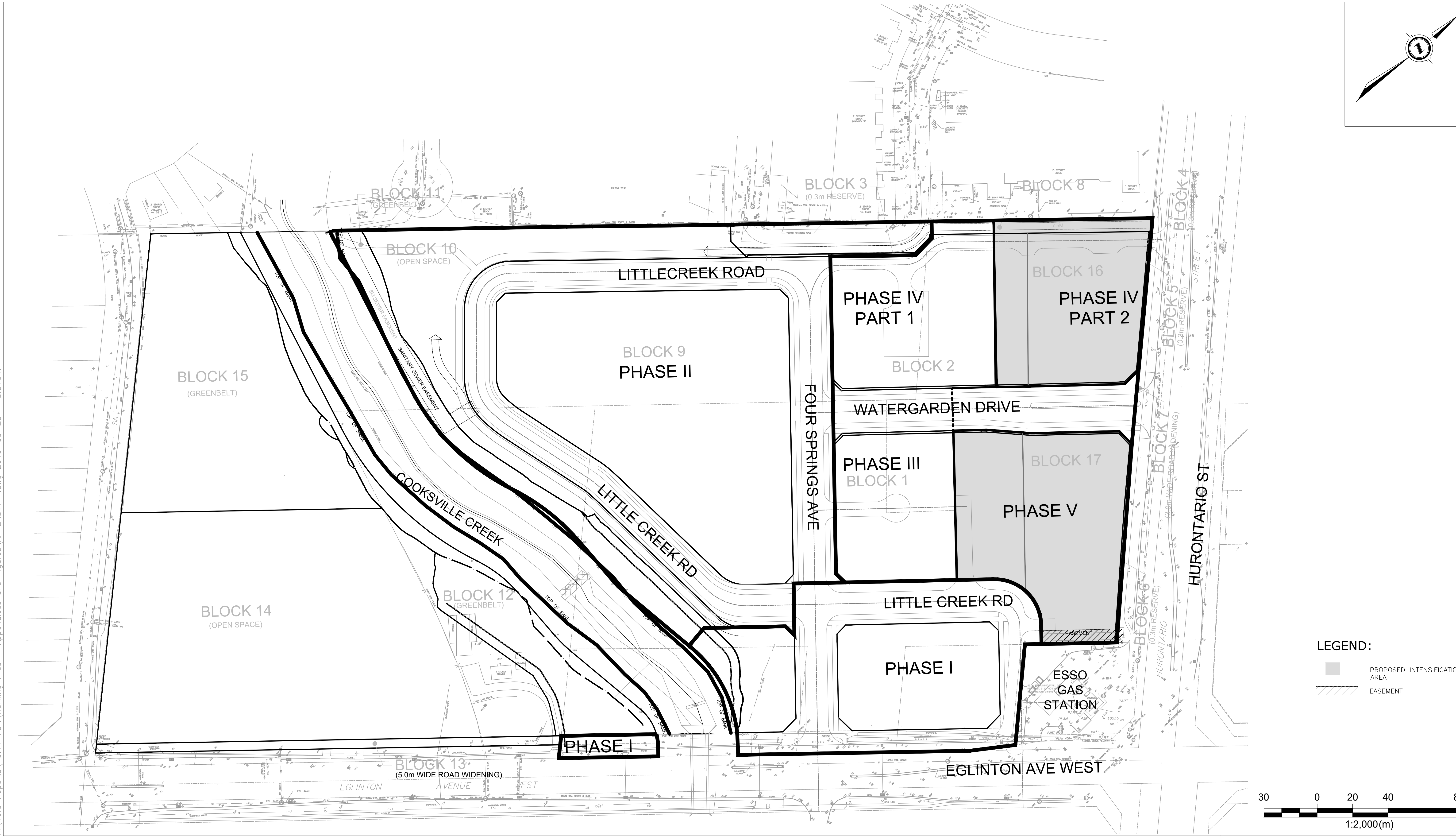


PINNACLE INTERNATIONAL	TITLE: ROAD GRADES AND OVERLAND FLOW ROUTING PHASE IV-PART II AND PHASE V	JOB No:	108686	IBI
		SCALE:	1:1500	9133 Leslie Street Suite 200 Richmond Hill, Ontario Canada L4B 4N1 Tel (905)763-2322 FAX(905)763-9983
		DATE:	MARCH, 2019	
		FIGURE No:	FIGURE 5	

T:\108686_Pinn_Uptown\10.0 Reports\10.11 FSR\Working Files — Appendices and Figures\14604-FSR-PHASE II-FIG-5.dwg 2019-03-25 — 10:33am

APPENDIX A

SUBDIVISION – STAGING PLAN



PINNACLE
INTERNATIONAL

TITLE:

STAGING PLAN
SUBDIVISION

JOB No:	108686
SCALE:	1:2000
DATE:	MARCH,2019
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 4 , PART 1 AND 2 SITE DATA (BY LAW# 0275-2012)

LOT AREA/ COVERAGE

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

	PART 1		PART 2	TOTAL FSI - PART (1 + 2)
LOT AREA	7327.2 SM (100%)	LOT AREA	7229.0 SM (100%)	14,556.2 SM (100%)
COVERED AREA/ FLOOR AREA	3494.2 SM (46.8%)	COVERED AREA/ FLOOR AREA	3661.1 SM (50.6%)	7155.3 SM (49.1%)
PAVED AREA	2005.8 SM (28.1%)	PAVED AREA	833.7 SM (11.5%)	2839.5 SM (19.5%)
LANDSCAPED AREA	1827.5 SM (25.1%)	LANDSCAPED AREA	2734.2 SM (37.8%)	4561.7 SM (31.3%)

MINIMUM LANDSCAPED AREA — 25% OF LOT AREA

G.F.A AND F.S.I

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

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

PROPOSED FLOOR SPACE INDEX (FSI)

	FSI - PART 1		FSI - PART 2	TOTAL FSI - PART (1 + 2)
FLOOR SPACE INDEX (FSI)	5.12X	FLOOR SPACE INDEX (FSI)	8.04X	6.58X

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)	55,134.15 SM	92,483.25 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)	3,011.5 SM	3,244.5 SM
TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	37,582.1 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	58,145.65 SM	95,727.75 SM

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

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

BUILDING HEIGHT

BUILDING HEIGHT ALLOWED AS PER OZ 16/10

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

BUILDING HEIGHT PROPOSED

	PART 1 (ZONE A AND B)			PART 2 (ZONE C)	
	BUILDING 1	BUILDING 2		BUILDING 3	BUILDING 4
BUILDING HEIGHT	15 STOREYS	34 STOREYS	BUILDING HEIGHT	45 STOREYS	32 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	50	53	103	248
1 BEDROOM + DEN	47	44	91	1 BEDROOM + DEN	210	145	355	446
2 BEDROOM	5	2	7	2 BEDROOM	88	61	149	156
2 BEDROOM + DEN	64	152	216	2 BEDROOM + DEN	85	58	143	359
3 BEDROOM	–	–	–	3 BEDROOM	2	0	2	2
2 STOREY	5	4	9	2 STOREY	–	–	–	9
TOTAL SUITES PROPOSED	178	290	468	TOTAL SUITES PROPOSED	435	317	752	1220

AMENITY

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

	PART 1	PART 2	TOTAL - PART (1 + 2)
MINIMUM AMENITY AREA ALLOWED (5.6SM PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)	2620.8 SM	4211.2 SM	6832.0 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	680.9	944.0	1,624.9 SM	2,565.9 SM
OUTDOOR AMENITY AREA PROVIDED	633.3	1090.1	1723.4 SM	OUTDOOR AMENITY AREA PROVIDED	1,315.0	2,517.1	3,832.1 SM	5,555.5 SM
TOTAL AMENITY AREA PROVIDED	973.4	1691.0	2664.4 SM	TOTAL AMENITY AREA PROVIDED	1,995.9	3,461.1	5,457.0 SM	8,121.4 SM
TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	N/A	N/A	2620.8 SM	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	N/A	N/A	4,211.2 SM	6,832.0 SM

AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT — 5.74 SM

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	998 SM	1730.6	2728.6 SM	4038.6 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	1611.3	1567.3	3178.6 SM	3921.6 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	1202.2	1333.3	2535.5 SM	3571.5 SM

PARKING

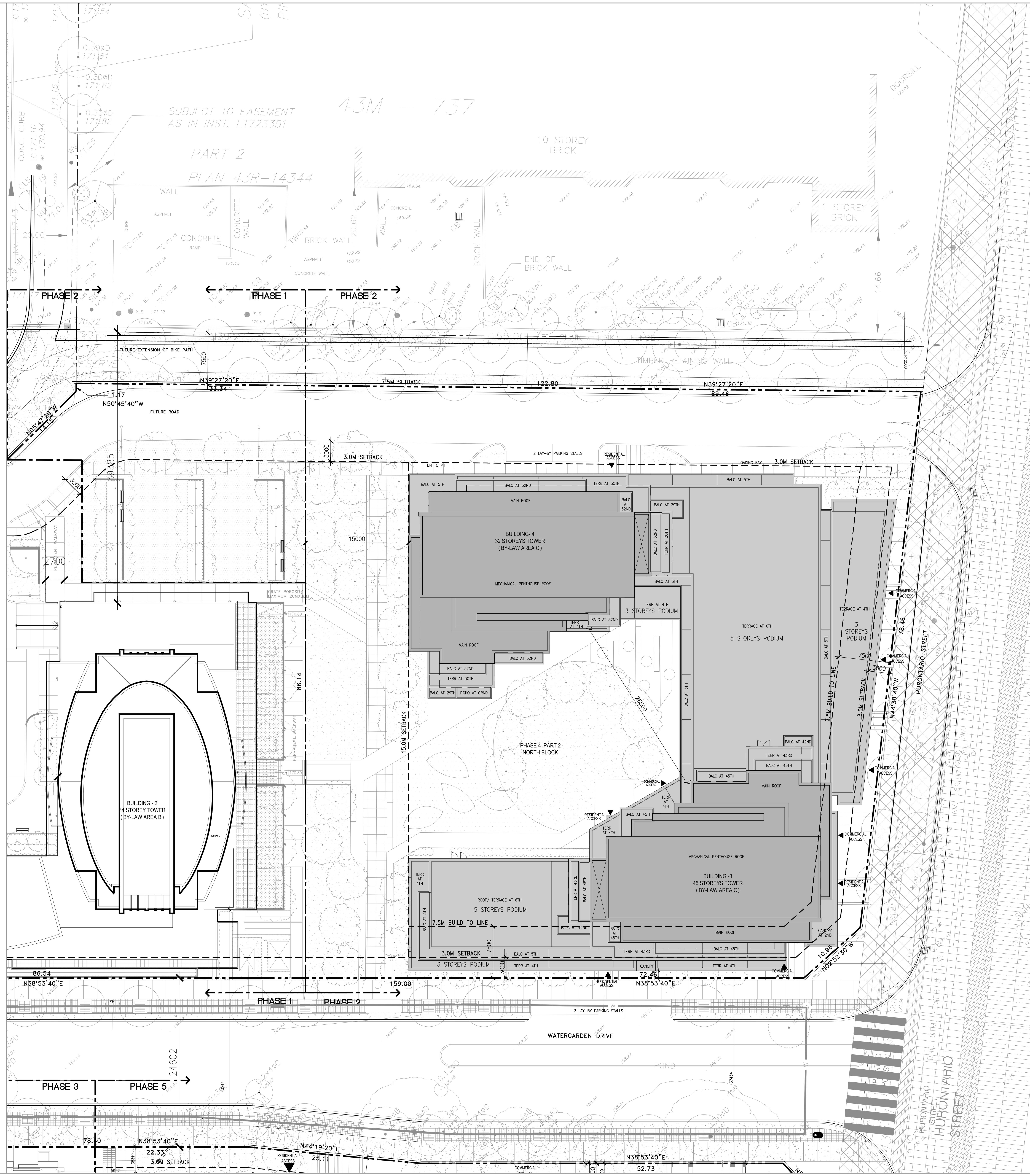
PARKING REQUIRED:(BY-LAW # 0225-2007)

	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	477	349	826	1331
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	3	–	3	14
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	66*	48	66*	137
COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	13*	–		COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	53	78*	78*	78
TOTAL PARKING REQUIRED:	220	367	587	TOTAL PARKING REQUIRED:	546	427	973	1560
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)	3	4	7	10

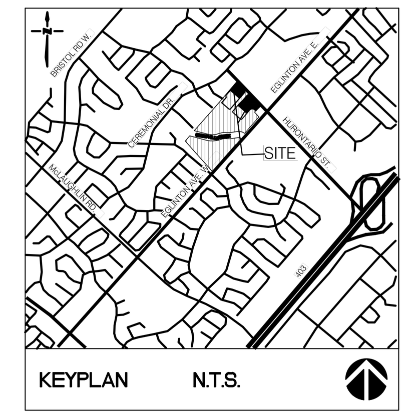
PARKING PROVIDED:

	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	66	32	78	32	208	312
PARKING LEVEL 2	0	72	0	93	165	PARKING LEVEL 2	0	124	0	91	215	380
PARKING LEVEL 3	0	72	0	102	174	PARKING LEVEL 3	0	124	0	92	216	390
PARKING LEVEL 4	0	72	0	102	174	PARKING LEVEL 4	0	124	0	92	216	390
PARKING LEVEL 5		20			20	PARKING LEVEL 5	0	124	0	91	215	235
TOTAL PARKING PROPOSED	27	252	44	314	637	TOTAL PARKING PROPOSED	66	528	78	398	1070	1707

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



GENERAL NOTES



KEYPLAN N.T.S.

LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

PLAN OF SURVEY OF
BLOCKS 1, 2 ,16 AND 17
REGISTERED PLAN 43M-1957

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

R. AVIS SURVEYING INC.
235 YORKLAND BOULEVARD ,SUITE 203
TORONTO, ONTARIO
M2J 4Y8
TEL.: (416) 490-8352 FAX: (416) 491-6206
www.ravissurveying.com
DATE :
FEBRUARY 23, 2018

NO	DATE	REMARKS
1	31 MAY 2018	RE-ZONING SUBMISSION

REVISIONS

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

PRINTS ARE NOT TO BE SCALED.

RICHMOND | Architects Ltd.

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

PROPOSED CONDOMINIUM DEVELOPMENT
'WATERGARDEN' (PHASE 4, PART 2)(H-RA5-43) & (PHASE 5)(H-RA5-42)
WATERGARDEN DRIVE
MISSISSAUGA, ONTARIO
Pinnacle International (Ontario) Limited
Suite 300- 911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

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

North | Seal

Date Plotted: 05 MAR 2019	Date : 25 MAY 2018
Drawn By: AA	Scale : 1:200
Checked By :	Revision Number:
Job No : 2017-002-N	Sheet No: SP A102-N

PHASE 3+5 BLOCK 2

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

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

BLOCK-2 PHASE 3 EXISTING RESIDENTIAL RA5-42	BLOCK-2 PHASE 5 H- RA5-42

LOT AREA/ COVERAGE

	PHASE 3		PHASE 5	TOTAL - PHASE (3+5)
LOT AREA (INCLUDE RESERVE AREA)				15,466.1 SM
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	5861.1 SM (56.9%)	8875.1 SM (57.4%)
PAVED AREA	390.6 SM (7.90%)	PAVED AREA	543.2 SM (5.3%)	933.8 SM (6.3%)
LANDSCAPED AREA	1727.4 SM (33.4%)	LANDSCAPED AREA	3895.8 SM (37.8%)	5623.2 SM (36.3%)
MINIMUM LANDSCAPED AREA				25% OF LOT AREA

G.F.A 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)	9.50X	8.65X
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)	89,272.75 SM	125,137.75 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)	8,623.8 SM	-
TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	35,865.0 SM	TOTAL PROPOSED RESIDENTIAL+NON-RESIDENTIAL GROSS FLOOR AREA (GFA)	97,896.55 SM	133,761.55 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	40 STOREYS	50 STOREYS

UNITS

NUMBER OF UNITS ALLOWED										1121
NUMBER OF UNITS PROPOSED										1,734
	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	61	161	71	293		-
1 BEDROOM + DEN	-	-	-	1 BEDROOM + DEN	161	139	280	580		-
2 BEDROOM	-	-	-	2 BEDROOM	125	42	71	238		-
2 BEDROOM + DEN	-	-	-	2 BEDROOM + DEN	12	26	63	101		-
3 BEDROOM	-	-	-	3 BEDROOM	4	45	27	76		-
2 STOREY	-	-	-	2 STOREY	-	-	-	-		-
TOTAL SUITES PROPOSED	234	212	446	TOTAL SUITES PROPOSED	363	413	512	1,288		1,734

AMENITY

AMENITY REQUIRED:(GENERAL BY-LAW-CITY OF MISSISSAUGA) MINIMUM AMENITY AREA ALLOWED (5.6SM PER DWELLING UNIT-COMBINED INDOOR AND OUTDOOR)			2497.6 SM					7,212.8 SM	9,710.4 SM
MINIMUM AMENITY AREA ALLOWED TO BE PROVIDED OUTSIDE AT GRADE									55.0 SM
AMENITY PROVIDED:									
	PHASE 3				PHASE 5				TOTAL - PHASE (3+5)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 1	BLDG. 2	BLDG. 3	TOTAL	
INDOOR AMENITY AREA PROVIDED	831.0	177.0	1,008.0 SM	INDOOR AMENITY AREA PROVIDED	565.7	968.9	1,415.2	2,949.8	3,957.8 SM
OUTDOOR AMENITY AREA PROVIDED	1,060.0	612.0	1,672.0 SM	OUTDOOR AMENITY AREA PROVIDED	1,500.0	2,098.6	2,598.9	6,197.5	7,869.5 SM
TOTAL AMENITY AREA PROVIDED	1,891.0	789.0	2,680.0 SM	TOTAL AMENITY AREA PROVIDED	2,065.7	3,067.5	4,014.1	9,147.3	11,827.3 SM
TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)	-	-	2,497.6 SM	TOTAL AMENITY AREA REQUIRED (GENERAL BY-LAW-CITY OF MISSISSAUGA)				7,212.8	9,757.85 SM
AVERAGE AMENITY AREA PROVIDED PER DWELLING UNIT									5.60 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	655.0 SM	593.6 SM	1,248.6 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA REQUIRED IN ONE CONTIGUOUS AREA	1032.8	1533.7	2007.0	4573.6	5913.6 SM
50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	945.5 SM	394.5 SM	1,340.0 SM	50% MINIMUM PERCENTAGE OF TOTAL REQUIRED AMENITY AREA PROVIDED IN ONE CONTIGUOUS AREA	1032.8	1533.7	2007.0	4573.6	5913.6 SM
MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE			55.0 SM	MINIMUM AMENITY AREA PROVIDED OUTSIDE AT GRADE				55.0	110.0 SM
AMENITY AREA PROVIDED OUTSIDE AT GRADE	402.0	321.0	723.0 SM	AMENITY AREA PROVIDED OUTSIDE AT GRADE	1,500.2	715.3	881.4	3,096.9	3819.9 SM

PARKING

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

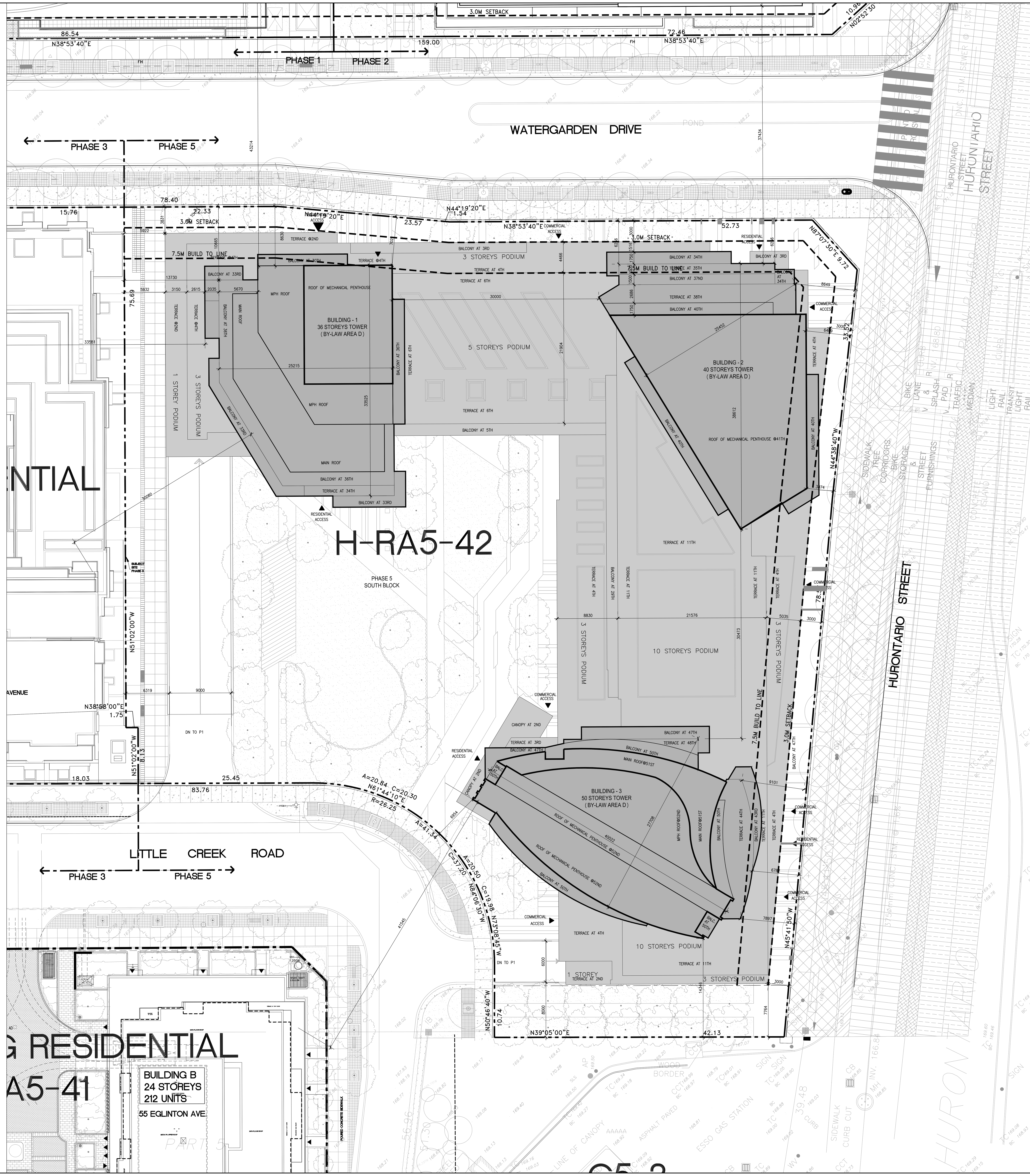
	PHASE 3				PHASE 5				TOTAL - PHASE (3+5)
	BUILDING 1	BUILDING 2	TOTAL (1+2)		BLDG. 1	BLDG. 2	BLDG. 3	TOTAL	
RESIDENTIAL PARKING REQUIRED PER 1 BEDROOM AND 2 BEDROOM: 1.1 SPACE/DWELLING UNIT	-	-	-	RESIDENTIAL PARKING REQUIRED: 0.68 STALLS/UNIT	247	281	349	877	-
RESIDENTIAL PARKING REQUIRED PER 3 BEDROOM: 1.2 SPACE/DWELLING UNIT	-	-	-	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	40*	45*	53*	138*	-
COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	-	-	-	COMMERCIAL RETAIL PARKING REQUIRED: 4.3 SPACES/100 SM	-	143*	227*	370*	-
TOTAL PARKING REQUIRED:	-	-	-	TOTAL PARKING REQUIRED:	247	424	576	1,247	-
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	3	4	9	-

PARKING PROVIDED:

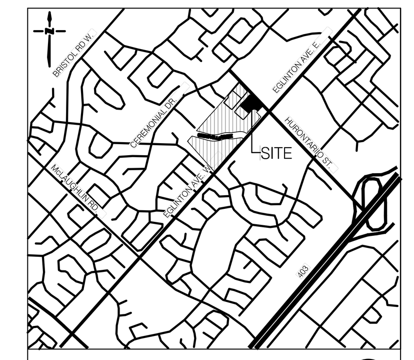
	PHASE 3				
	BUILDING 1		BUILDING 2		TOTAL (1+2)
	# OF CARS		# OF CARS		
	VISITORS	TENANTS	VISITORS	TENANTS	TOTAL
PARKING LEVEL 1	-	-	-	-	-
PARKING LEVEL 2	-	-	-	-	-
PARKING LEVEL 3	-	-	-	-	-
PARKING LEVEL 4	-	-	-	-	-
TOTAL PARKING PROPOSED	-	-	-	-	-

	PHASE 5							TOTAL - PHASE (3+5)
	BLDG. 1		BLDG. 2		BLDG. 3		TOTAL	
	# OF CARS		# OF CARS		# OF CARS			
	VIS./COMM	TENANTS	VIS./COMM	TENANTS	VIS./COMM	TENANTS		
PARKING LEVEL 1	-	-	-	-	-	-	199	-
PARKING LEVEL 2	-	-	-	-	-	-	189	-
PARKING LEVEL 3	-	-	-	-	-	-	287	-
PARKING LEVEL 4	-	-	-	-	-	-	287	-
PARKING LEVEL 5	-	-	-	-	-	-	287	-
PARKING LEVEL 6	-	-	-	-	-	-	177	-
TOTAL PARKING PROPOSED	0	312	0	347	370	397	1426	-

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



GENERAL NOTES



KEYPLAN N.T.S.

LEGAL DESCRIPTION AND SURVEY INFORMATION

SURVEY INFORMATION TAKEN FROM:

PLAN OF SURVEY OF BLOCKS 1, 2 ,16 AND 17 REGISTERED PLAN 43M-1957

CITY OF MISSISSAUGA REGIONAL MUNICIPALITY OF PEEL

SURVEY INFORMATION PROVIDED BY:

R. AVIS SURVEYING INC.
235 YORKLAND BOULEVARD ,SUITE 203
TORONTO, ONTARIO
M2J 4Y8
TEL.: (416) 490-8352 FAX: (416) 491-6206
www.ravissurveying.com
DATE :
FEBRUARY 23, 2018

NO	DATE	REMARKS
1	31 MAY 2018	RE-ZONING SUBMISSION

REVISIONS

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

PRINTS ARE NOT TO BE SCALED.

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

PROPOSED CONDOMINIUM DEVELOPMENT 'WATERGARDEN' (PHASE 4, PART 2)(H-RA5-43) & (PHASE 5)(H-RA5-42)
WATERGARDEN DRIVE
MISSISSAUGA, ONTARIO
Pinnacle International (Ontario) Limited
Suite 300- 911 Homer Street
Vancouver, BC, V6B 2W6
PH. (604) 602 7747 Fax (604) 680 7749

Sheet Title: SITE PLAN&STATS PHASE 5; (BLDG. 1,2&3)

North | Seal

Date Plotted: 05 MAR 2019	Date : 25 MAY 2018
Drawn By: -	Scale : 1: 200
Checked By :	Revision Number:
Job No : 2017-002	Sheet No: SP A101-S

APPENDIX C

SANITARY SEWER DESIGN SHEET AND SITE STATISTICS

DEVELOPMENT ___14604__PINNACLE INTERNATIONAL (ONTARIO) LTD._

REGIONAL MUNICIPALITY OF PEEL

Last Updated : March 22, 2019

CONSULTANT _____IBI GROUP__________________________________

SANITARY SEWER DESIGN CHART

Designed By: _____AC___

MAJOR DRAINAGE AREA __________________________

Checked By: _____AC___

LOCATION			DRAINAGE AREA			RUNOFF						PIPE SELECTION						
STREET	FROM MH. No.	TO MH. No.	A ha	Density Persons Per (ha) or Unit	Number of Units or Floor Area in ha	Cummul. Area ha	Cummul. Population	1 Sewage Flow m3/sec	2 Infiltration Flow m3/sec	3 Foundation Drains m3/sec	Total Flow 1+2+3 m3/s	Pipe Length L m	Cumm. Pipe Length m	Pipe Dia> mm	Pipe Gradient % m/m	Actual Capacity (full) m3/s	Velocity (full) V m/s	Time Of Flow min.
TO EX. MH T4																		
AREA 2A (Phase IV - Part II)	STUB	MH6A																
TOWER				2.7	883.0		2384											
OFFICE/COMMERCIAL				50.0	0.30		15											
INFILTRATION			1.04			1.04	2399	0.0296	0.0002		0.0298	10.0		250	1.00%	0.059	1.211	0.138
AREA 1A (Phase V - BLDG 1 & 2)	STUB	MH6A																
TOWER				2.7	776.0		2095											
OFFICE/COMMERCIAL				50.0	0.86		43											
INFILTRATION			0.65			0.49	2138	0.0267	0.0001		0.0268	15.5		250	1.00%	0.059	1.211	0.213
EXTERNAL AREA	STUB	MH6A																
Plaza East HWY #10				50.0	2.24	2.24	112	0.0017	0.0004		0.0021	18.4		250	1.50%	0.073	1.484	0.206
WATERGARDEN DRIVE	MH6A	MH7A				3.77	4649	0.0533	0.0008		0.0541	80.0		250	1.00%	0.059	1.211	1.101
WATERGARDEN DRIVE	MH7A	MH8A				3.77	4649	0.0533	0.0008		0.0541	72.0		250	1.00%	0.059	1.211	0.990
AREA 2B	STUB	MH8A																
TOWER				2.7	468.0		1264											
OFFICE/COMMERCIAL				50.0	0.02		1											
INFILTRATION			0.81			0.81	1265	0.0165	0.0002		0.0167	11.8		250	1.00%	0.059	1.211	0.162
AREA 1D	STUB	MH8A																
TOWER				2.7	234.0		632											
INFILTRATION			0.45			0.45	632	0.0087	0.0001		0.0088	14.8		250	1.00%	0.059	1.211	0.204
WATERGARDEN DRIVE	MH8A	MH9A				5.03	6546	0.0719	0.0010		0.0729	18.1		250	2.38%	0.092	1.869	0.161
AREA 1C	STUB	MH10A																
HIGH RISE				2.7	212.0		572											
INFILTRATION			0.41			0.41	572	0.0079	0.0001		0.0080	11.2		250	1.00%	0.059	1.211	0.154
FOURSPRINGS AVE.	MH10A	MH9A				0.41	572	0.0079	0.0001		0.0080	61.3		250	0.50%	0.042	0.857	1.192
	MH9A	MH11A				5.44	7118	0.0773	0.0011		0.0784	56.3		375	0.40%	0.111	1.004	0.934
LITTLE CREEK RD																		
AREA 7A	MH11A	MH12A	0.17			5.61	7118	0.0773	0.0011		0.0784	29.2		375	0.40%	0.111	1.004	0.485
AREA 5A	MH12A	MH13A	0.23			5.84	7118	0.0773	0.0012		0.0785	85.9		375	0.40%	0.111	1.004	1.426
AREA 5B	MH13A	MH14A	0.20			6.04	7118	0.0773	0.0012		0.0785	77.4		375	0.59%	0.135	1.219	1.058
EASEMENT																		
AREA 5C	MH14A	MH15A	0.08			6.12	7118	0.0773	0.0012		0.0785	28.3		375	1.50%	0.215	1.944	0.243
AREA 5D	MH15A	MH16A	0.23			6.35	7118	0.0773	0.0013		0.0786	69.2		375	2.00%	0.248	2.245	0.514
CREEK CROSSING																		
AREA 5E	MH16A	EX. MHT4	0.05			6.40	7118	0.0773	0.0013		0.0786	48.9		375	0.30%	0.096	0.870	0.937
TO EX. MH 9A																		
AREA 1B (Phase V - BLDG 3)																		
TOWER				2.7	512.0		1382											
OFFICE/COMMERCIAL				50.0	0.00		0											
INFILTRATION	STUB	MH4A	0.54			0.54	1382	0.0179	0.0001		0.018	13.5		200	4.00%	0.066	2.088	0.108
AREA 3B (PHASE I)	STUB	MH4A																
HIGH RISE				2.7	218.0		589											
INFILTRATION			0.37			0.37	589	0.0081	0.0001		0.008	14.5		200	2.00%	0.046	1.476	0.164
LITTLE CREEK RD	MH4A	MH1A				0.91	1971	0.0248	0.0002		0.025	85.4		250	1.00%	0.059	1.211	1.175
AREA 3A (PHASE I)	STUB	MH1A																
HIGH RISE				2.7	251.0		678											
INFILTRATION			0.50			0.50	678	0.0093	0.0001		0.009	15.5		200	4.00%	0.066	2.088	0.124
LITTLE CREEK RD	MH1A	MH2A				1.41	2649	0.0324	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				4.49	3029	0.0365	0.0009		0.0374	65.9		300	0.50%	0.068	0.967	1.135
Foursprings Avenue	MH3A	EX.MH9A				**4.49**	**3029**	0.0365	0.0009		**0.0374**	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

LOCATION					DRAINAGE AREA				RUNOFF					PIPE SELECTION								PROFILE		
STREET (DRAINAGE AREA LABEL)	FROM MH. No.	FROM Sta.	TO MH. No.	TO Sta.	A ha	Coeff. C	A*C	ACCUM. A*C	Cummul. Tc (Ti-__) min.	i_{10} mm/h	i_{100} mm/h	Q_{10} m^3/sec	Q_{100} m^3/sec	Pipe Length L m	Pipe Slope S %	Pipe Dia> mm	Actual Capacity (full) m^3/s	Velocity (full) V m/s	Time Of Flow min.	Overland Flow along Road Q_{Rd} m^3/sec	**Minor Losses m**	Fall In Sewer m	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

Distributed by the INTERHYMO Centre. Copyright (c), 1989.  Paul Wisner & Assoc.
  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 hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN 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-313261313110

Distributed by the INTERHYMO Centre. Copyright (c), 1989.  Paul Wisner & Assoc.
  LICENSED TO:  Cumming Cockburn Limited, Ottawa

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

DATE: 02-14-2019                      TIME: 12:21:00

COMMENTS: ____________________________________________________________

-------------------------------------------------------------------------------
***************************************************************************
*                                                                         *
* POST-DEVELOPMENT CONDITIONS                                             *
* PRELIMINARY ESTIMATE OF STORMWATER DETENTION STORAGE REQUIREMENT        *
* PINNACLE PHASE IV PART 2 AND PHASE V (OZ/OPA 18 11)                     *
* NORTHWEST OF EGLINTON AVENUE WEST AND HURONTARIO STREET                 *
* CITY OF MISSISSAUGA                                                     *
*                                                                         *
* IBI GROUP, FEB. 2019                                                    *
* PROJECT: 108686                                                         *
*                                                                         *
***************************************************************************
*
***************************************************************************
*                                                                         *
* DESIGN STORMS: CITY OF MISSISSAUGA'S STANDARD DRAWING NO. 2111.010      *
*                                                                         *
***************************************************************************
*
************************* 1:2 YEAR DESIGN STORM **************************
--------------------
| CHICAGO STORM    |     IDF curve parameters: A= 610.000
| Ptotal= 36.75 mm |                           B=   4.600
--------------------                           C=    .780
                         used in:   INTENSITY =  A / (t + B)^C

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

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

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

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN            |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73          .24
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n           =       .013         .250

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

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     104.51        21.91
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.95 (ii)    6.80 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .31          .14
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .20          .01             .21 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33            2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16           29.71
     TOTAL RAINFALL   (mm)=      36.63        36.63           36.63
     RUNOFF COEFFICIENT   =        .98          .30             .81

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .049

                              AREA     QPEAK     TPEAK       R.V.
                              (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)     .98       .21      2.25      29.71
     OUTFLOW: ID= 8 (2025)     .98       .01      3.17      29.28

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.512
                  TIME SHIFT OF PEAK FLOW        (min)= 55.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .019

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 74 |   QPEAK     (cms)=    .21 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  29.71
  Filename: C:H2Bi_2.HYD
  Comments:
```

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=270 |   QPEAK      (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK      (hrs)=   3.17
--------------------   VOLUME     (mm)=  29.28
  Filename: C:H2Bo_2.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
|ID= 3 DT= 5.0 min |   Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     104.51        14.68
                over (min)        5.00        20.00
     Storage Coeff.  (min)=       1.92 (ii)   18.50 (ii)
     Unit Hyd. Tpeak (min)=       5.00        20.00
     Unit Hyd. peak  (cms)=        .31          .06
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .15          .01            .16 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.58           2.25
     RUNOFF VOLUME    (mm)=      35.83        11.16          26.08
     TOTAL RAINFALL   (mm)=      36.63        36.63          36.63
     RUNOFF COEFFICIENT   =        .98          .30            .71

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .028       .042

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)      .93       .16      2.25      26.08
     OUTFLOW: ID=10 (2035)      .93       .01      3.25      25.67

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.471
                  TIME SHIFT OF PEAK FLOW         (min)= 60.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .015

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
```

```
| ID= 3   PCYC= 81 |   QPEAK     (cms)=    .16 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  26.08
  Filename: C:H3Bi_2.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=243 |   QPEAK     (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.25
--------------------   VOLUME     (mm)=  25.67
  Filename: C:H3Bo_2.HYD
  Comments:

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

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

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

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

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73          .24
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n          =        .013         .250

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

```
                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     140.49        37.11
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.74 (ii)    6.05 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .32          .15
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .28          .02            .29 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33           2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50          41.01
     TOTAL RAINFALL   (mm)=      49.24        49.24          49.24
     RUNOFF COEFFICIENT   =        .98          .38            .83

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .029       .049

                              AREA     QPEAK     TPEAK       R.V.
                              (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)     .98       .29      2.25      41.01
     OUTFLOW: ID= 8 (2025)     .98       .02      3.17      40.58

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.511
                  TIME SHIFT OF PEAK FLOW        (min)= 55.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .027

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .29 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  41.01
  Filename: C:H2Bi_5.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=288 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  40.58
  Filename: C:H2Bo_5.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
|ID= 3 DT= 5.0 min |   Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
```

```
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     140.49        29.49
                over (min)       5.00        15.00
     Storage Coeff.  (min)=       1.71 (ii)   14.25 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=        .32          .08
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .21          .02             .22 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.42            2.25
     RUNOFF VOLUME    (mm)=      48.44        18.50           36.61
     TOTAL RAINFALL   (mm)=      49.24        49.24           49.24
     RUNOFF COEFFICIENT   =        .98          .38             .74

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .028       .042

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)       .93       .22      2.25      36.61
     OUTFLOW: ID=10 (2035)       .93       .01      3.25      36.20

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.587
                   TIME SHIFT OF PEAK FLOW         (min)= 60.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .022

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
| ID= 3   PCYC= 80 |   QPEAK     (cms)=    .22 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  36.61
  Filename: C:H3Bi_5.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=260 |   QPEAK     (cms)=    .01 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.25
--------------------   VOLUME     (mm)=  36.20
  Filename: C:H3Bo_5.HYD
  Comments:

     (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

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73          .24
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n          =       .013         .250

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

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     173.04        52.90
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.60 (ii)    5.56 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .32          .16
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .34          .03            .36 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33           2.25
     RUNOFF VOLUME    (mm)=      59.85        26.00          51.45
     TOTAL RAINFALL   (mm)=      60.65        60.65          60.65
     RUNOFF COEFFICIENT   =        .99          .43            .85

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
```

```
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .049

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)       .98       .36      2.25      51.45
     OUTFLOW: ID= 8 (2025)       .98       .02      3.17      51.02

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.515
                  TIME SHIFT OF PEAK FLOW         (min)= 55.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .033

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .36 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  51.45
  Filename: C:H2Bi_10.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=301 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  51.02
  Filename: C:H2Bo_10.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
|ID= 3 DT= 5.0 min |   Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n          =        .013         .250

     Max.eff.Inten.(mm/hr)=     173.04        52.90
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       1.57 (ii)   11.50 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=        .33          .09
                                                           *TOTALS*
     PEAK FLOW       (cms)=        .26          .03            .27 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.42           2.25
     RUNOFF VOLUME    (mm)=      59.85        26.00          46.47
     TOTAL RAINFALL   (mm)=      60.65        60.65          60.65
     RUNOFF COEFFICIENT   =        .99          .43            .77

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

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .042

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)       .93       .27      2.25      46.47
     OUTFLOW: ID=10 (2035)       .93       .02      3.17      46.07

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.682
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .027

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
| ID= 3   PCYC= 79 |   QPEAK     (cms)=    .27 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  46.47
  Filename: C:H3Bi_10.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=272 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  46.07
  Filename: C:H3Bo_10.HYD
  Comments:

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

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

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

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

```
                 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

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN            |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73          .24
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n          =        .013         .250

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

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     198.74        66.45
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.51 (ii)    5.26 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .16
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .40          .04            .42 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33           2.25
     RUNOFF VOLUME    (mm)=      68.86        32.35          59.80
     TOTAL RAINFALL   (mm)=      69.66        69.66          69.66
     RUNOFF COEFFICIENT   =        .99          .46            .86

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .029       .049

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)      .98       .42      2.25      59.80
     OUTFLOW: ID= 8 (2025)      .98       .02      3.08      59.37

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.520
                   TIME SHIFT OF PEAK FLOW         (min)= 50.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .039

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .42 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=   59.80
  Filename: C:H2Bi_25.HYD
```

```
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=309 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  59.37
  Filename: C:H2Bo_25.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
|ID= 3 DT= 5.0 min |   Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     198.74        66.45
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.49 (ii)    6.42 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .14
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .30          .05             .33 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33            2.25
     RUNOFF VOLUME    (mm)=      68.86        32.35           54.44
     TOTAL RAINFALL   (mm)=      69.66        69.66           69.66
     RUNOFF COEFFICIENT   =        .99          .46             .78

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)      (ha.m.)  |   (cms)     (ha.m.)
                           .000       .000    |     .028       .042

                             AREA     QPEAK     TPEAK       R.V.
                             (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)    .93       .33      2.25      54.44
     OUTFLOW: ID=10 (2035)    .93       .02      3.08      54.03

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.485
                  TIME SHIFT OF PEAK FLOW         (min)= 50.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .032

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

```
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
| ID= 3   PCYC= 76 |   QPEAK     (cms)=    .33 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  54.44
  Filename: C:H3Bi_25.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=279 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  54.03
  Filename: C:H3Bo_25.HYD
  Comments:

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

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

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

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

--------------------------------------------------------------------------------
*
*---------------------
* PHASE V (AREA 2B) |
*---------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73          .24
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      80.69        40.00
     Mannings n          =        .013         .250
```

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

                              IMPERVIOUS    PERVIOUS (i)
    Max.eff.Inten.(mm/hr)=     220.93        79.34
               over (min)        5.00        10.00
    Storage Coeff.  (min)=       1.45 (ii)    5.04 (ii)
    Unit Hyd. Tpeak (min)=       5.00        10.00
    Unit Hyd. peak  (cms)=        .33          .16
                                                       *TOTALS*
    PEAK FLOW       (cms)=        .44          .05           .47 (iii)
    TIME TO PEAK    (hrs)=       2.25         2.33          2.25
    RUNOFF VOLUME    (mm)=      77.25        38.55         67.65
    TOTAL RAINFALL   (mm)=      78.05        78.05         78.05
    RUNOFF COEFFICIENT   =        .99          .49           .87

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)      (ha.m.)  |   (cms)     (ha.m.)
                           .000       .000    |     .029       .049

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
    INFLOW : ID= 2 (2020)       .98       .47      2.25      67.65
    OUTFLOW: ID= 8 (2025)       .98       .03      3.17      67.22

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.566
                   TIME SHIFT OF PEAK FLOW         (min)= 55.000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .044

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 75 |   QPEAK     (cms)=    .47 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  67.65
  Filename: C:H2Bi_50.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=316 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.17
--------------------   VOLUME     (mm)=  67.22
  Filename: C:H2Bo_50.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
```

```
|ID= 3 DT= 5.0 min |    Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     220.93        79.34
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.43 (ii)    6.15 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .15
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .34          .07            .37 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33           2.25
     RUNOFF VOLUME    (mm)=      77.25        38.55          61.96
     TOTAL RAINFALL   (mm)=      78.05        78.05          78.05
     RUNOFF COEFFICIENT   =        .99          .49            .79

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

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)      (ha.m.)   |   (cms)     (ha.m.)
                             .000        .000   |     .028       .042

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)        .93       .37      2.25      61.96
     OUTFLOW: ID=10 (2035)        .93       .02      3.08      61.56

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.555
                  TIME SHIFT OF PEAK FLOW         (min)= 50.000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .037

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
| ID= 3   PCYC= 76 |   QPEAK     (cms)=    .37 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  61.96
  Filename: C:H3Bi_50.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=286 |   QPEAK     (cms)=    .02 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  61.56
  Filename: C:H3Bo_50.HYD
  Comments:

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

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

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

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

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

--------------------------------------------------------------------------------
*
*--------------------
* PHASE V (AREA 2B) |
*--------------------
--------------------
| DESIGN           |
| STANDHYD  (2020) |   Area    (ha)=    .98
|ID= 2 DT= 5.0 min |   Total Imp(%)=  75.20   Dir. Conn.(%)=  75.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .73           .24
     Dep. Storage     (mm)=        .80          1.50
     Average Slope     (%)=       1.50          1.50
     Length            (m)=      80.69         40.00
     Mannings n          =        .013          .250

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

                              IMPERVIOUS    PERVIOUS (i)
     Max.eff.Inten.(mm/hr)=     242.53         93.14
                over (min)        5.00          5.00
     Storage Coeff.  (min)=       1.40 (ii)     4.86 (ii)
     Unit Hyd. Tpeak (min)=       5.00          5.00
     Unit Hyd. peak  (cms)=        .33           .22
                                                               *TOTALS*
     PEAK FLOW       (cms)=        .49           .06              .55 (iii)
     TIME TO PEAK    (hrs)=       2.25          2.25             2.25
     RUNOFF VOLUME    (mm)=      86.22         45.43            76.10
     TOTAL RAINFALL   (mm)=      87.02         87.02            87.02
     RUNOFF COEFFICIENT   =        .99           .52              .87

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

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

```
* STORMWATER DETENTION FACILITY #2
--------------------
| RESERVOIR (2025) |
| IN= 2---> OUT= 8 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .000       .000    |     .029       .049

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 2 (2020)      .98      .546      2.25      76.10
     OUTFLOW: ID= 8 (2025)      .98      .029      3.08      75.67

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  5.3710
                  TIME SHIFT OF PEAK FLOW         (min)= 50.0000
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0493

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2020) |   AREA       (ha)=    .98
| ID= 2   PCYC= 74 |   QPEAK     (cms)=    .55 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  76.10
  Filename: C:H2Bi_100.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2025) |   AREA       (ha)=    .98
| ID= 8   PCYC=323 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  75.67
  Filename: C:H2Bo_100.HYD
  Comments:

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

--------------------------------------------------------------------------------
*
*----------------------------------------------------------------------------
* PHASE IV PART 2 (AREA 3B) & PHASE IV PART 1 NORTHEAST CORNER (AREA 3A2) |
*----------------------------------------------------------------------------
--------------------
| DESIGN           |
| STANDHYD  (2030) |   Area    (ha)=    .93
|ID= 3 DT= 5.0 min |   Total Imp(%)=  60.50   Dir. Conn.(%)=  60.50
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .56          .37
     Dep. Storage     (mm)=        .80         1.50
     Average Slope     (%)=       1.50         1.50
     Length            (m)=      78.66        40.00
     Mannings n           =       .013         .250

     Max.eff.Inten.(mm/hr)=     242.53        93.14
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.37 (ii)    5.93 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .15
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .37          .08             .42 (iii)
     TIME TO PEAK    (hrs)=       2.25         2.33            2.25
     RUNOFF VOLUME    (mm)=      86.22        45.44           70.11
     TOTAL RAINFALL   (mm)=      87.02        87.02           87.02
     RUNOFF COEFFICIENT   =        .99          .52             .81

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  77.1    Ia = Dep. Storage  (Above)
```

```
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* STORMWATER DETENTION FACILITY #1
--------------------
| RESERVOIR (2035) |
| IN= 3---> OUT=10 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .000       .000   |     .028       .042

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
     INFLOW : ID= 3 (2030)       .93      .417      2.25      70.11
     OUTFLOW: ID=10 (2035)       .93      .028      3.08      69.70

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  6.6700
                   TIME SHIFT OF PEAK FLOW         (min)= 50.0000
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0420

--------------------------------------------------------------------------------
*
--------------------
| SAVE HYD  (2030) |   AREA       (ha)=    .93
| ID= 3   PCYC= 76 |   QPEAK     (cms)=    .42 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   2.25
--------------------   VOLUME     (mm)=  70.11
  Filename: C:H3Bi_100.HYD
  Comments:

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

--------------------------------------------------------------------------------
--------------------
| SAVE HYD  (2035) |   AREA       (ha)=    .93
| ID=10   PCYC=292 |   QPEAK     (cms)=    .03 (i)
| DT= 5.0 min      |   TPEAK     (hrs)=   3.08
--------------------   VOLUME     (mm)=  69.70
  Filename: C:H3Bo_100.HYD
  Comments:

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

--------------------------------------------------------------------------------

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

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.

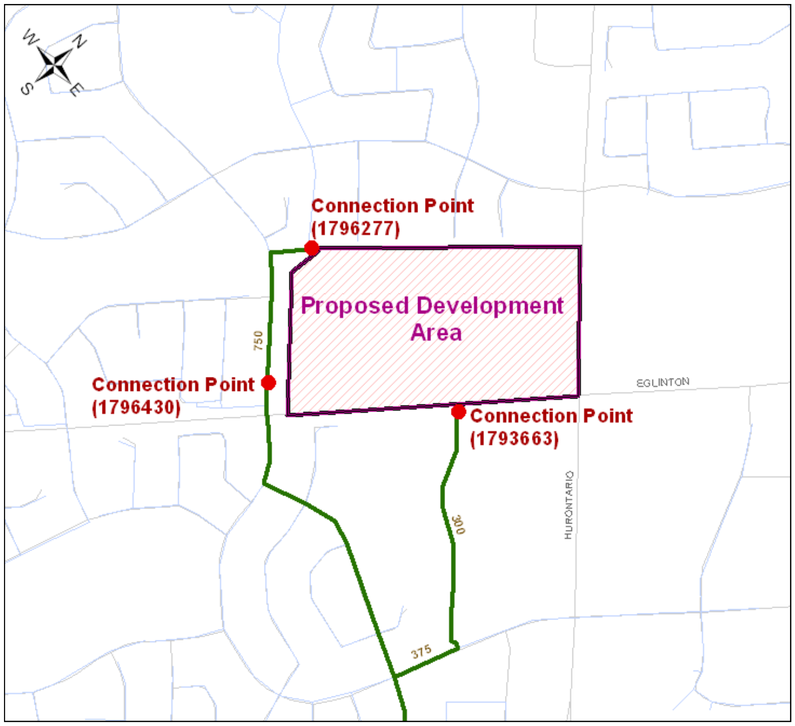


Figure 2 - Sanitary System Overview & Possible Connection Points

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

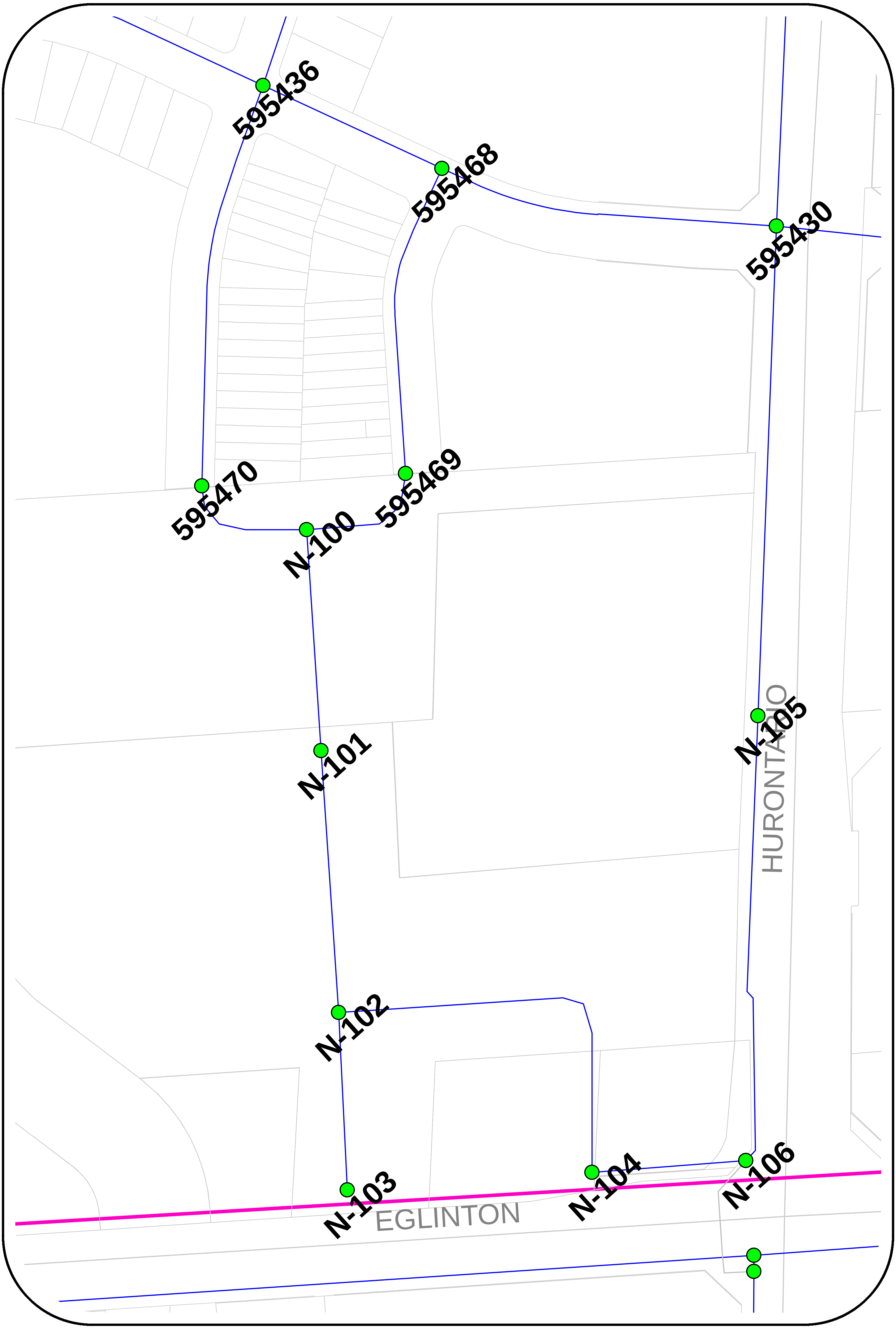
Appendix 1: Detailed Demand Calculations

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

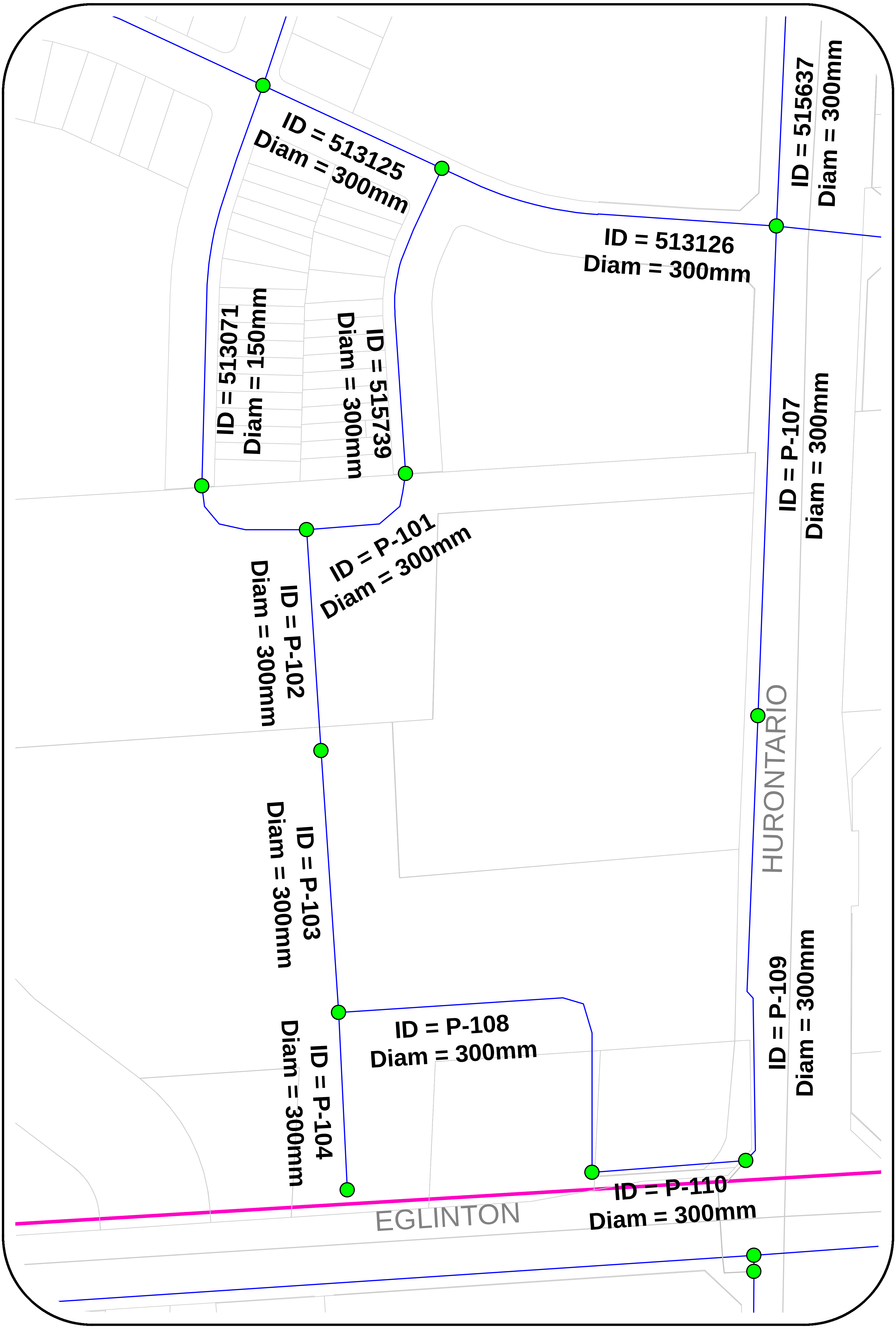
West Parcel	Block	Area (ha)
West Parcel	Block 8 - Hazard Land	0.58
	Block 9 -Open Space	4.14
	Road	0.2
West Parcel Total		4.92

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

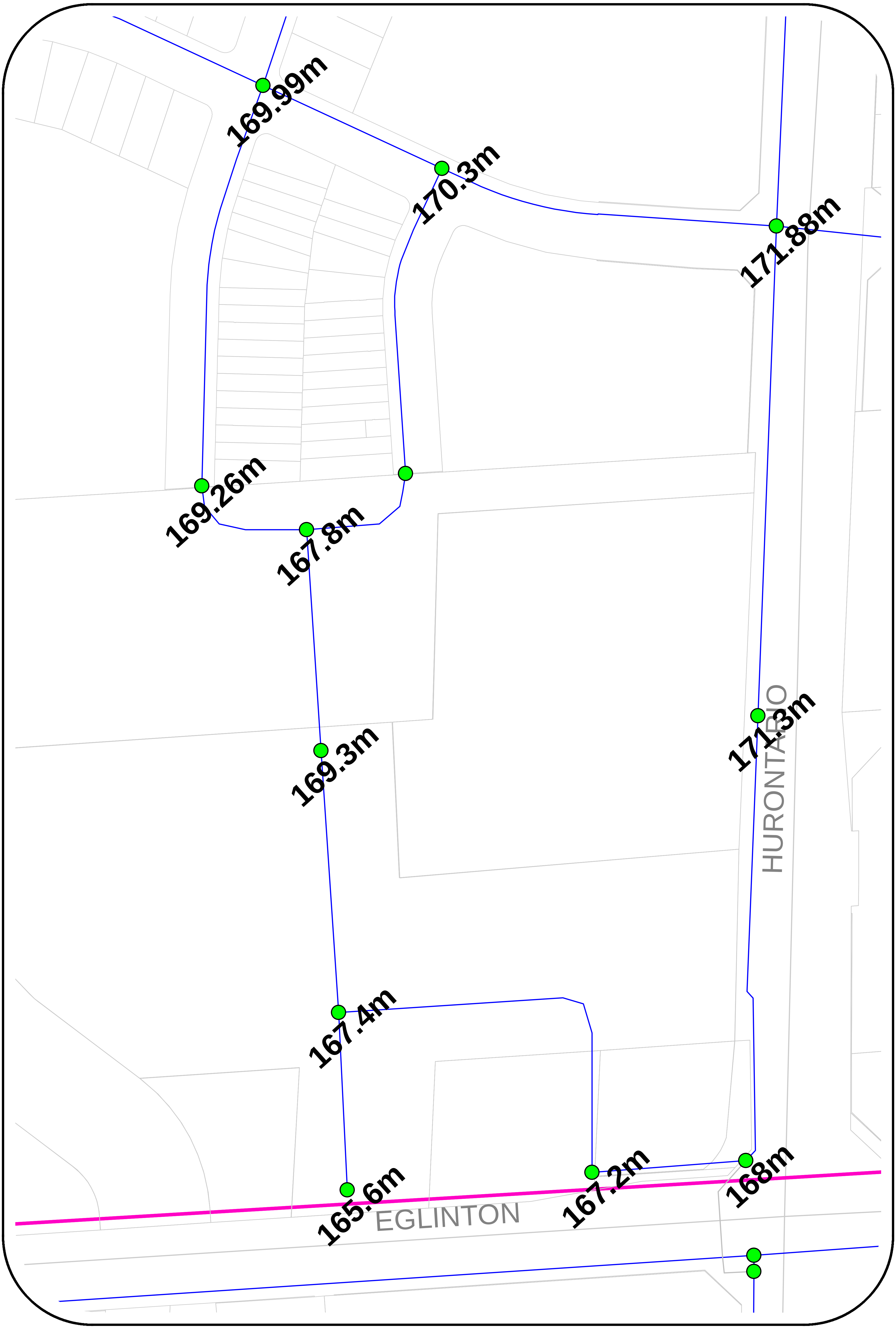
2016	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.00	240.57	233.04	231.93	227.27	224.83
595470	241.00	240.57	233.03	231.91	227.24	224.79
N-100	241.00	240.57	233.04	231.91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

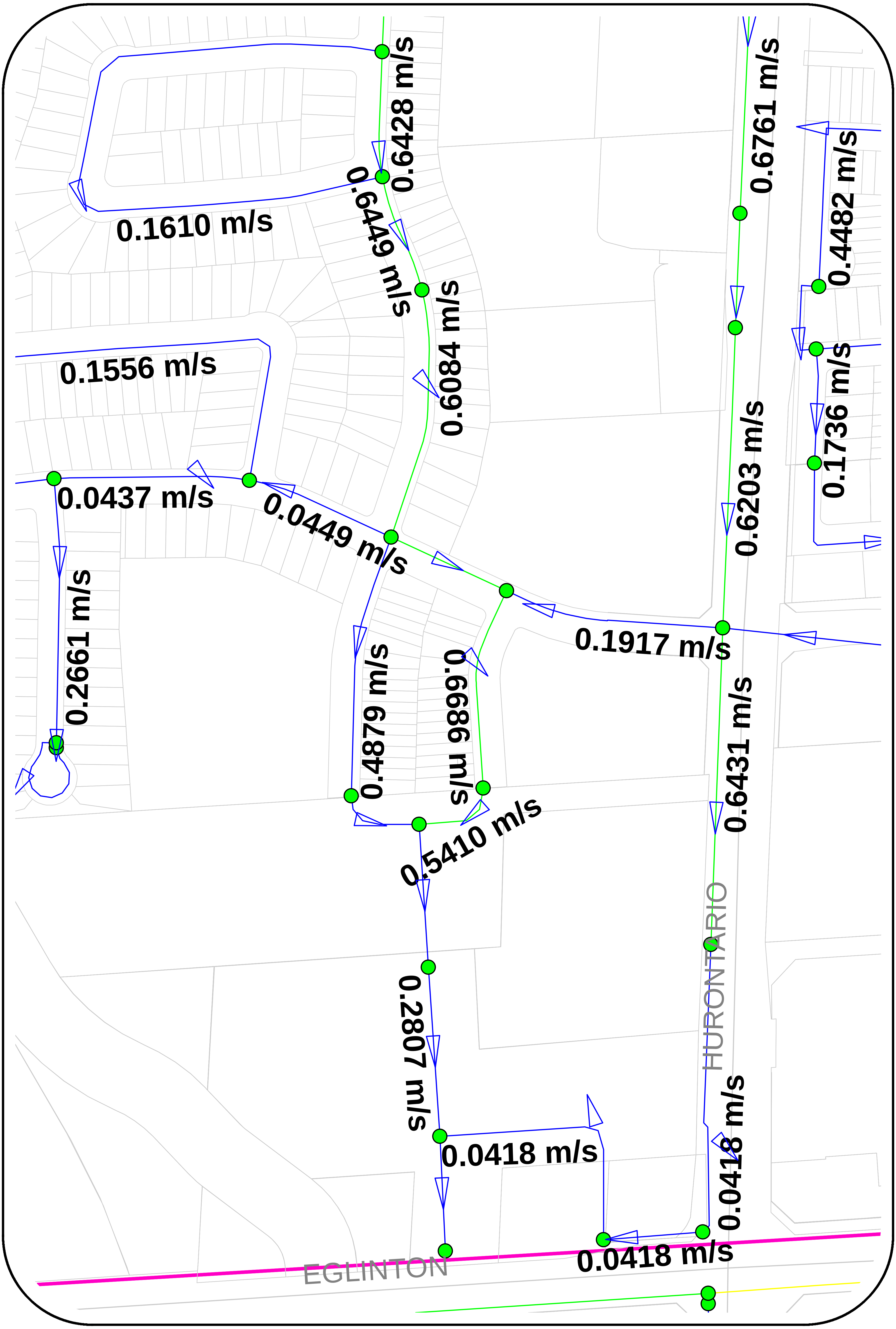
2031	HGL (m)					
Model Node ID	Ave. Day		Max. Day		Peak Hour	
	Without Development	With Development	Without Development	With Development	Without Development	With Development
595469	241.36	240.91	244.83	243.73	227.16	224.55
595470	241.36	240.91	244.82	243.71	227.13	224.50
N-100	241.36	240.91	244.83	243.71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

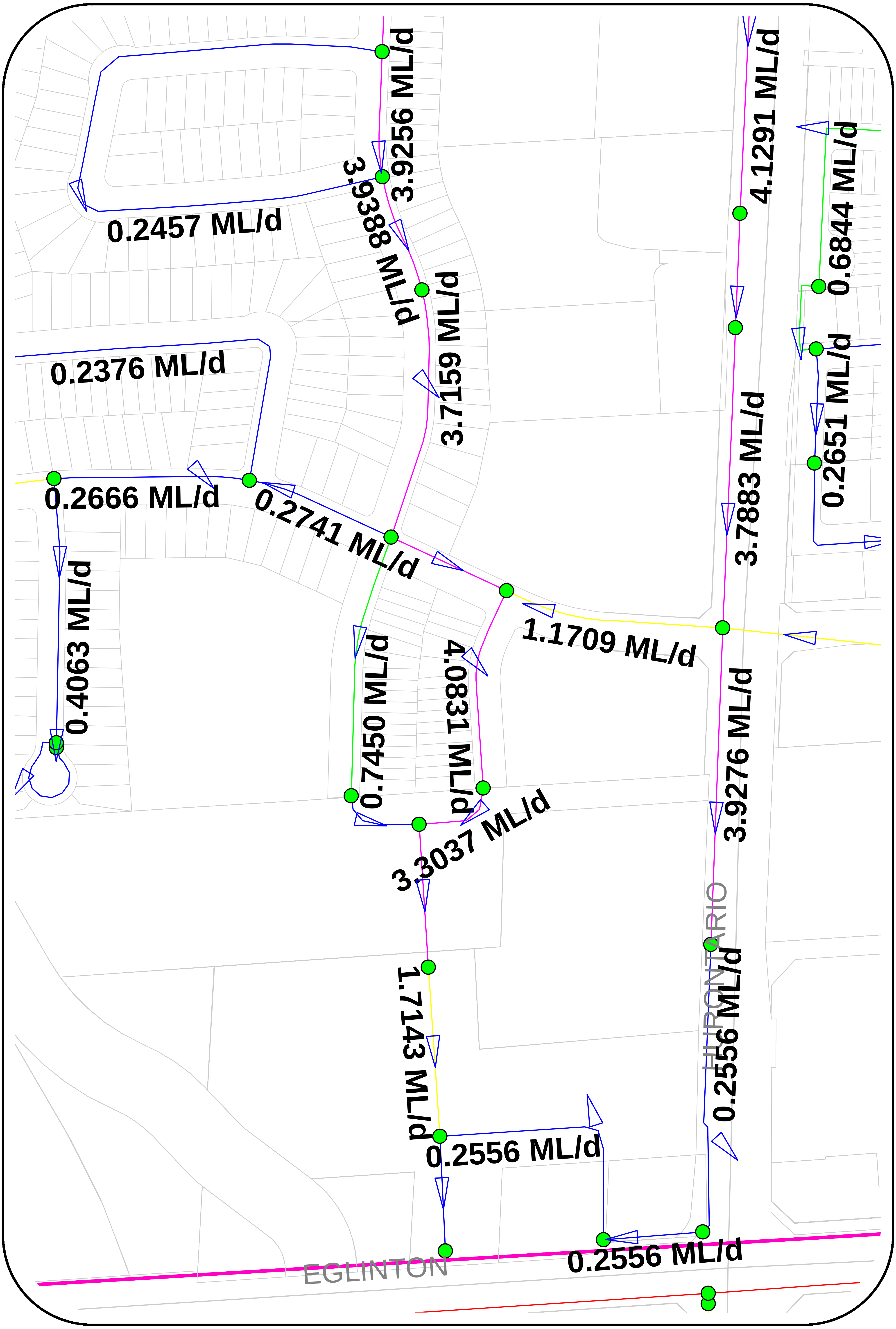
Pinnacle Uptown Mississauga Development
Appendix 5: Node Elevations



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

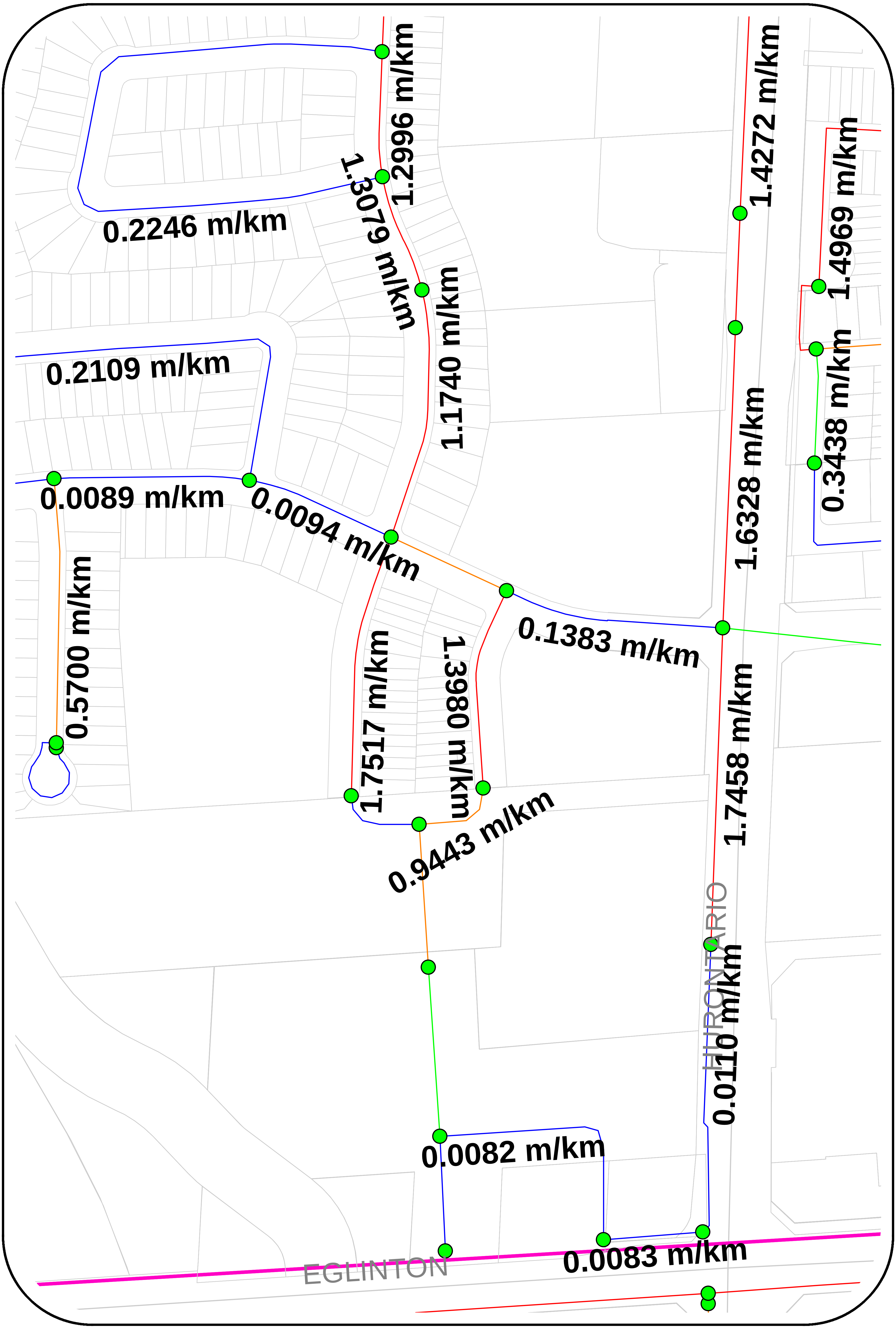


Pinnacle Uptown Mississauga Development
Appendix 7: 2031 Peak Hour Demand - Flow



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 DEV AGREEMENT	Easements will be required for Water and Sanitary services in order to service phase 1 of the development. **Created :** 2009-06-22 15:05:29 **Last Modified : -**
8	NOTE:	For clarifications regarding the comments above please contact Orest Jacyla at 905-791-7800 extension 7809. **Created :** 2009-06-22 15:05:30 **Last Modified : -**

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

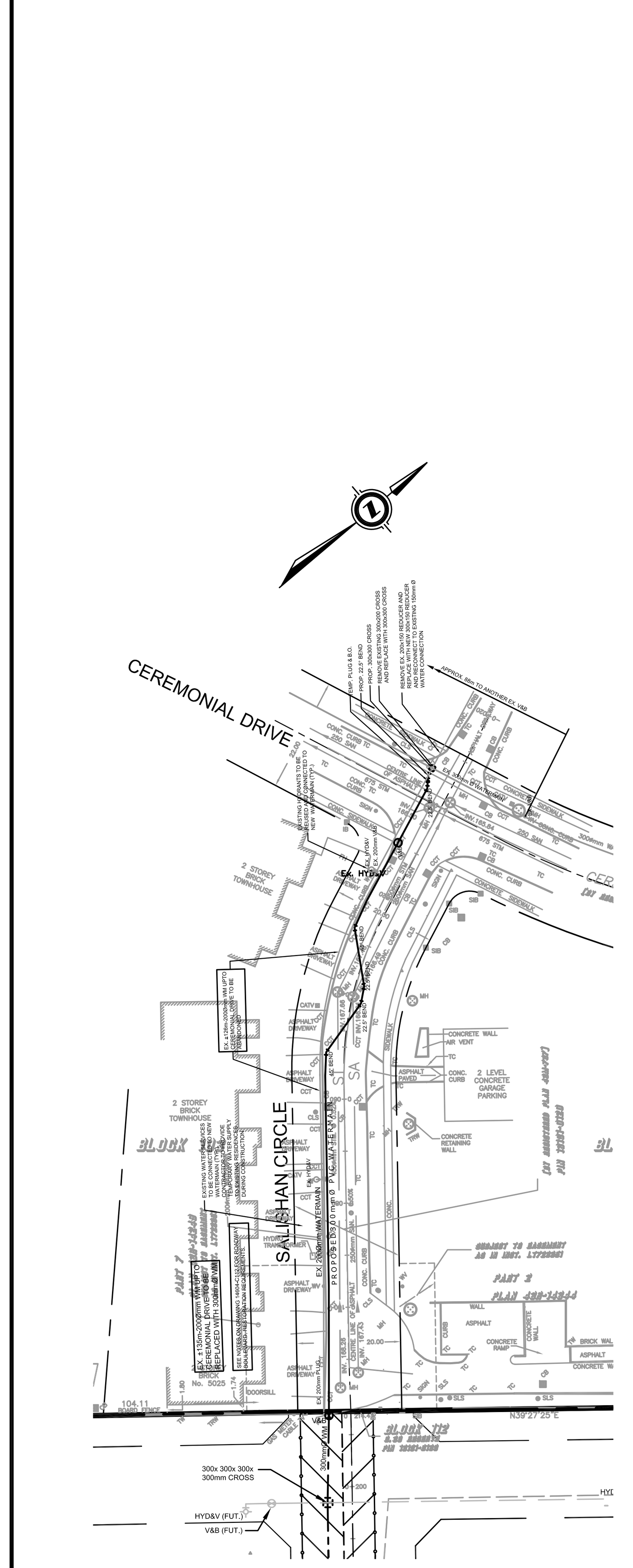
230 Richmond Street West
5th Floor
Toronto, Ontario
Canada M5V 1V6
Tel (416)596-1930
FAX(416)596-0644

Pinnacle International (Ontario) Limited

PINNACLE UPTOWN MISSISSAUGA

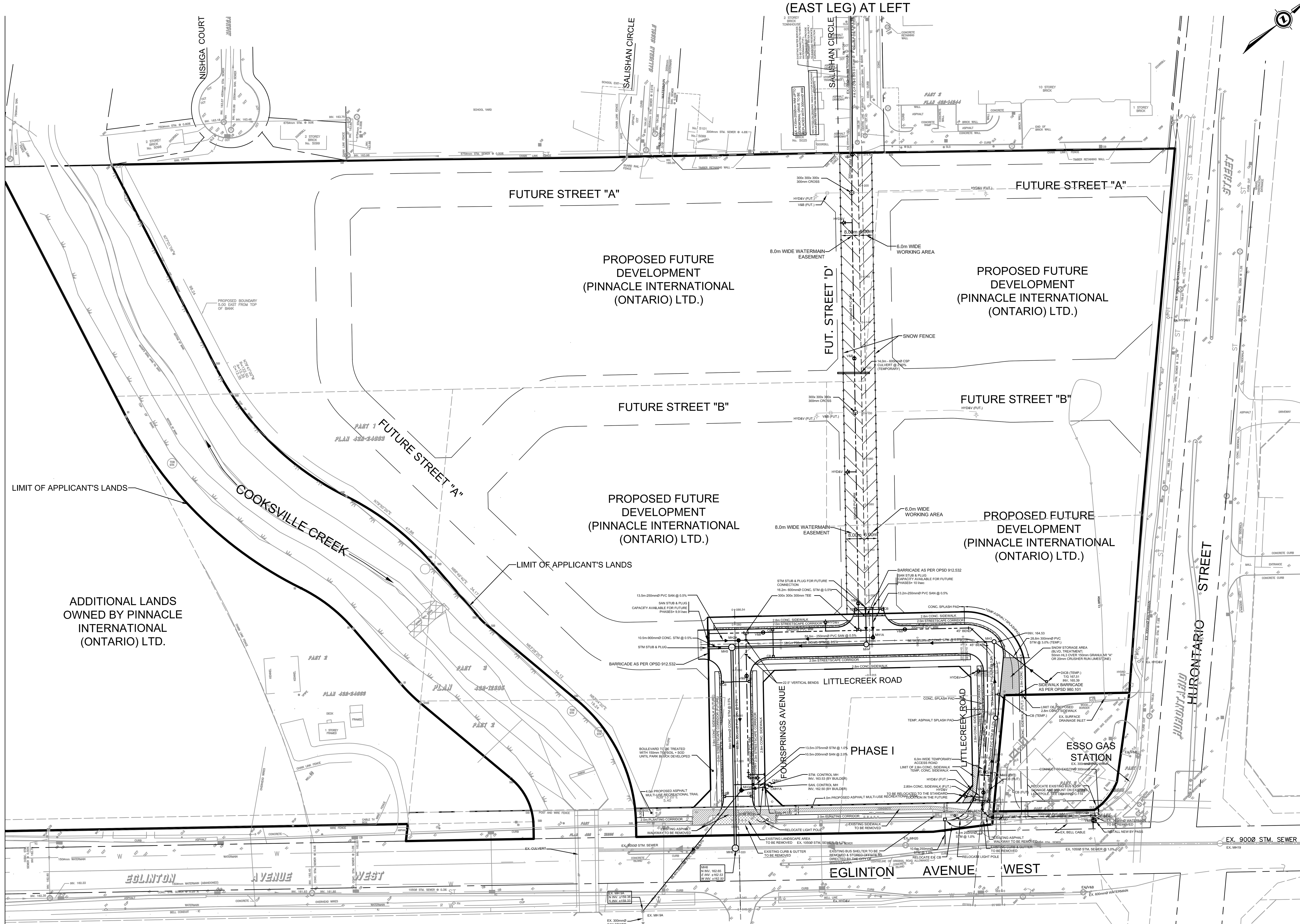
PRELIMINARY MASTER PLAN

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

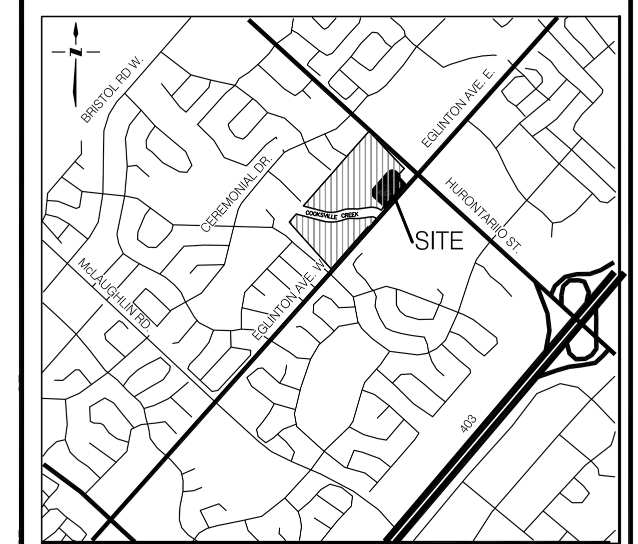
NO	ITEM	QUANTITY
1	FIRE HYDRANT	6
2	WATER SERVICE	1
3	300mmØ VALVE & BOX	9
4	300mmØ WATERMAIN	656.4m
5	SANITARY MANHOLE 1200mmØ	3
6	200mmØ SANITARY SEWER	10.5m
7	250mmØ SANITARY SEWER	85.7m
8	300mmØ SANITARY SEWER	135.6m

LEGEND:
- PROPERTY BOUNDARY
- PHASE I BOUNDARY
- EX. MH — EXISTING STORM SEWER
- MH1O — PROPOSED STORM SEWER SERVICING
- CONTROL MH (BY BLOCK DEVELOPER)
- EASEMENT
- MH1A — PROPOSED SANITARY SEWER & MH.
- EX. MH — 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

SUBMISSION | 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

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 1:1000	Date AUGUST 2009
Drawn UP/WS	Checked GM/DB
Project No. TO-14604	Drawing No. 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	Area (ha.)
	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**					

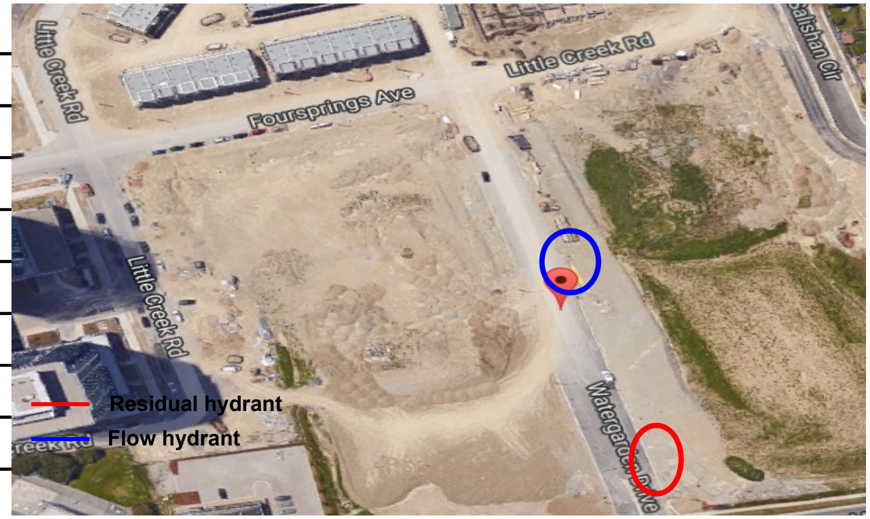
Open Space Breakdown	Parkland Area	**Easement Area** (outside Hazard Area)	Hazard Area	Total
Block 5	0.81	0.04		**0.85**
Block 6	0.19	0.00	0.07	**0.26**
Block7			0.42	**0.42**
East Parcel Sub-Total	**1.00**	**0.04**	**0.49**	**1.53**
West Parcel Block 8			**0.58**	**0.58**
West Parcel Block 9	4.14		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

GTA Waterworks

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



Flow Test Measurements

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

Test Results

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

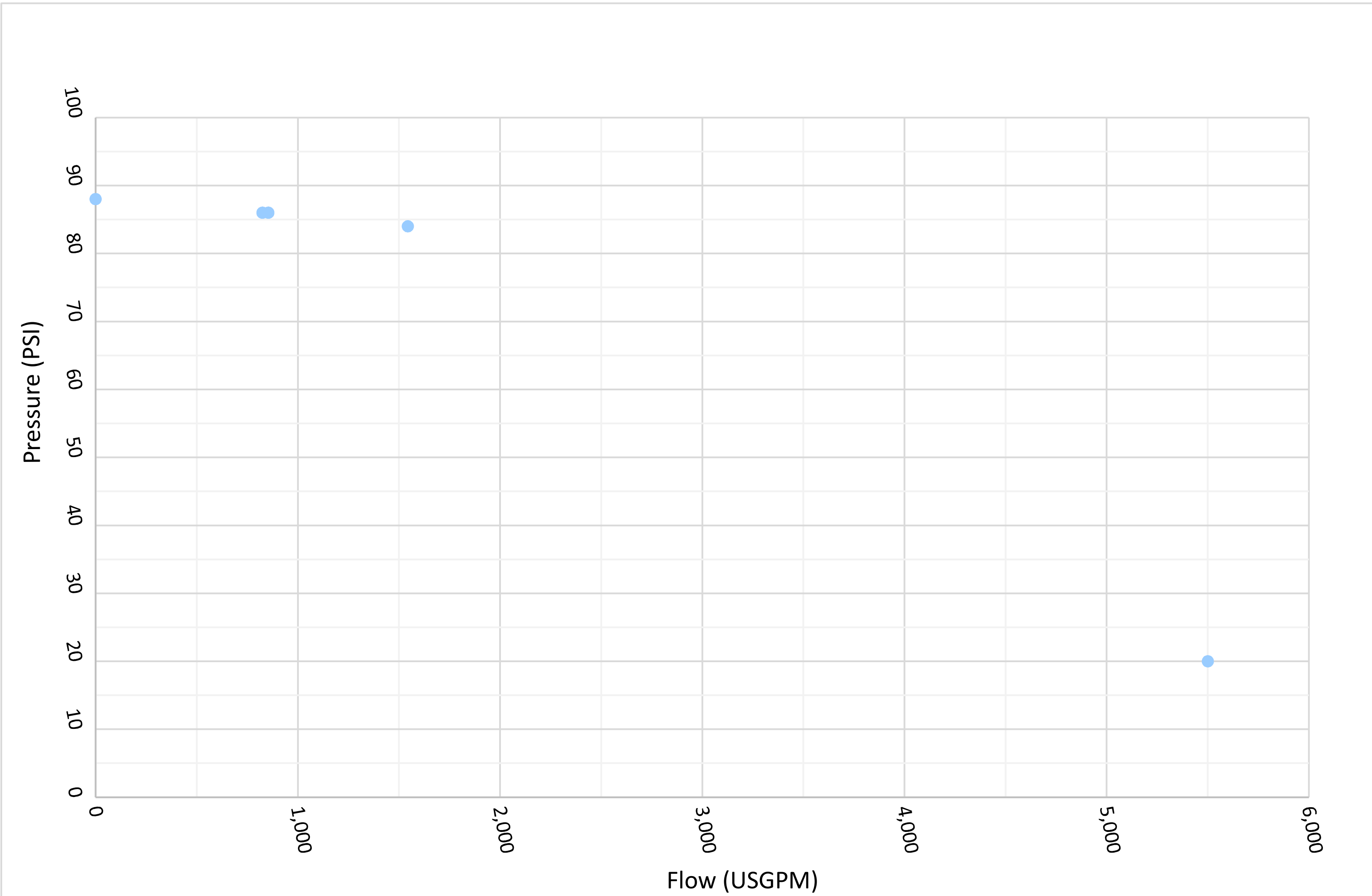
$$Q_r = Q_t \left(\frac{P_s - P_r}{P_s - P_t} \right)^{0.54}$$

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

Hydrant Classification as per NFPA 291	
Class	Colour
AA	Blue

Hydrant Flow Curve

35 Watergarden Dr.



GTA Waterworks

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

Flow Test Measurements

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

Test Results

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

Hydrant Classification as per NFPA 291	
Class	Colour
AA	Blue

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

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

Hydrant Flow Curve

65 Watergarden Dr.

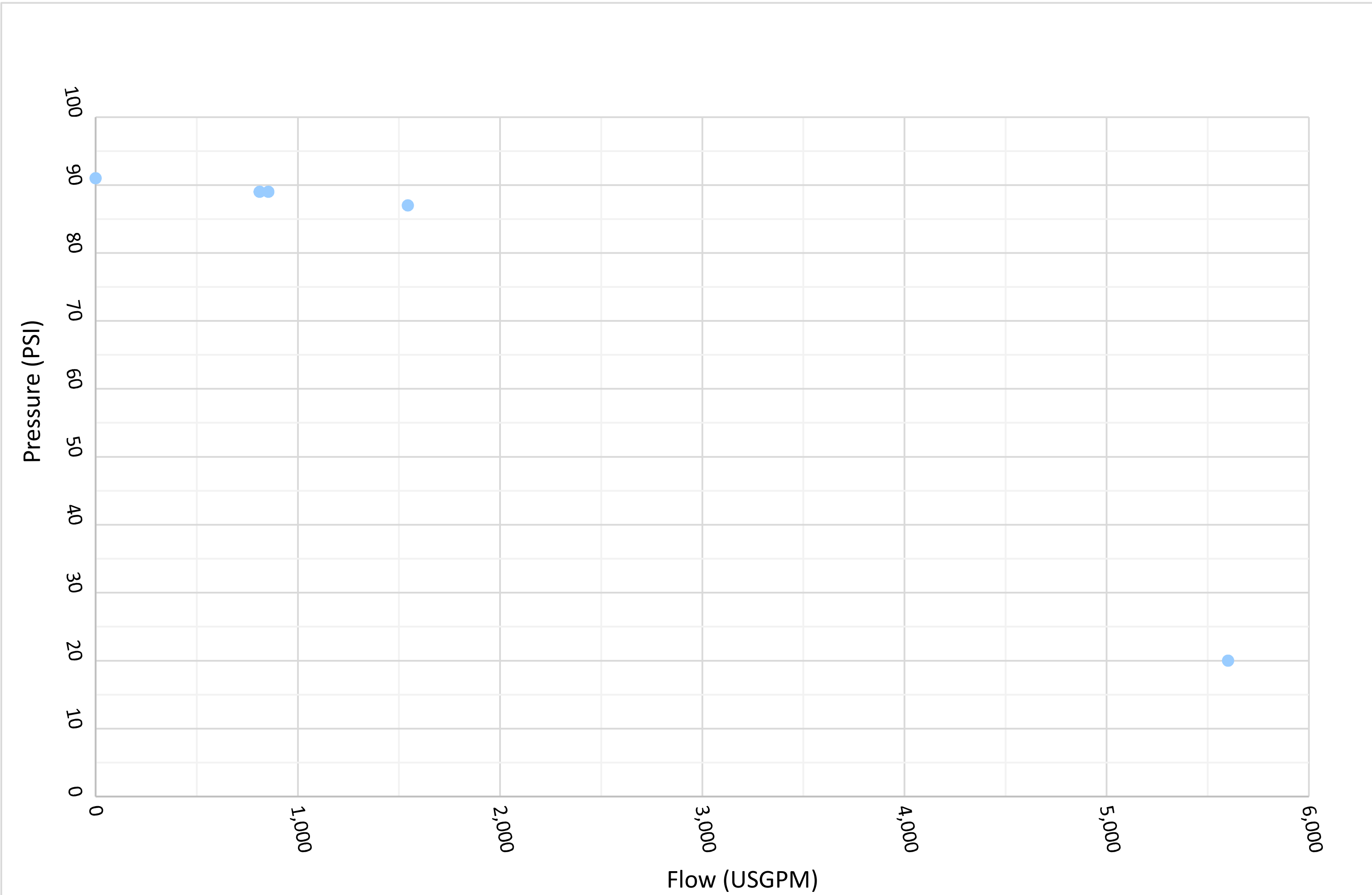


Table F1 Available Fire Flow Calculations

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

Outlet diameter:	**2**	in, one port	Location:	**35 Watergarden Drive, Mississauga**
Static pressure:	**88**	psi	Date of Test:	**20-Jun-17**
Resid. pressure:	**86**	psi, one port	Operator:	**GTA Waterworks**

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

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

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

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

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

Preliminary Fire Flow Estimate

Based on Fire Underwriters Survey Method, Insurance Advisory Organization

Item **Pinnacle - Phase IV Part II**

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

INPUT

Where:
F = the required fire flow in Litres per minute (L/min)
C = Coefficient related to the type of construction — C= 0.6
1) 1.5 For wood fram construction
2) 1.0 for ordinary construction
3) 0.8 for noncombustible constuction

4) 0.6 for fire-resistive construction (two largest successive floors, plus 50% of each of any floors immediately above them up to 8, if vertical openings are inadequately protected

5) 0.6 for fire-resistive construction (largest floor, plus 25% each of 2 adjoining floors, if vertical openings are adequately protected (1-hour rating).

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

	INPUT	
Mixed USE		
Floor Area	2,216.53	m2

Total building floor area (m2)	2216.5 m2	A=	2,217	m2
		F =	**6,215**	**Litres/minute**
Round to nearest 1000 Litres/min		**F =**	**6,000**	**Litres/minute**

		INPUT		
	Increases or Reductions			
2	Low/high fire hazard occupancy (open garage credit 25%)		-	
3a	Automatic sprinkler protection credit (conforming to NFPA standards)	-15%	(900)	
3b	If sprinkler system, an additional 10% credit if water supply is standard for both system and fire department hose lines.	-30%	(1,800)	
3c	If sprinkler system, an additional 10% credit if fully supervised (water flow and control valve alarm system).	0%	-	
4	Exposure (% sum of all sides of building)	20%	1,200	
			-	
	Note: Fire flow shall not exceed 45,000L/min nor be less than 2,000L/min.	**F =**	**4,500**	**Litres/minute**
	Round to nearest 1000 Litres/min	**FINAL F =**	**5000**	**Litres/minute**

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

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

Preliminary Fire Flow Estimate

Based on Fire Underwriters Survey Method, Insurance Advisory Organization

Item **Pinnacle - Phase V**

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

INPUT

Where:
F = the required fire flow in Litres per minute (L/min)
C = Coefficient related to the type of construction — C= 0.6
1) 1.5 For wood fram construction
2) 1.0 for ordinary construction
3) 0.8 for noncombustible constuction

4) 0.6 for fire-resistive construction (two largest successive floors, plus 50% of each of any floors immediately above them up to 8, if vertical openings are inadequately protected

5) 0.6 for fire-resistive construction (largest floor, plus 25% each of 2 adjoining floors, if vertical openings are adequately protected (1-hour rating).

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

	INPUT				
Mixed USE					
Floor Area	2,820.10	m2			
Total building floor area (m2)	2820.1	m2	A=	2,820	m2
			F =	**7,010**	**Litres/minute**
Round to nearest 1000 Litres/min			**F =**	**7,000**	**Litres/minute**

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

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

PHASE IV - PART II GFA BREAKDOWN

BUILDING 3

Level	GFA (SF)	GFA (SM)
P5	752.0	69.9
P4	956.0	88.9
P3	956.0	88.9
P2	956.0	88.9
P1	1087.0	101.0
1	8274.0	768.7
2	12784.0	1,187.7
3	13070.0	1,214.2
4	9470.0	879.8
5	11818.0	1,097.9
6	6749.0	627.0
7 to 41	266907.0	24,796.4
42	7626.0	708.5
43	6595.0	612.7
44	6597.0	612.9
45	6597.0	612.9
MPH	0.0	0.0
ROOF	0.0	0.0
TOTAL GFA	**361196.0**	**33,556.0**

Maximum Floor GFA =	13,070.0	1,214.2
25% of each of the adjoing floors =	5,563.5	516.9
Total Floor Area =	18,633.5	1,731.1

PHASE IV - PART II GFA BREAKDOWN

BUILDING 4

Level	GFA (SF)	GFA (SM)
1	7,764	721.30
2	12,234	1,136.60
3	18,727	1,739.80
4	8,291	770.30
5	14,832	1,377.95
6	7,053	655.20
7 to 28	168,273	15,633.10
29	7,649	710.60
30	6,618	614.80
31	6,620	615.00
32	6,620	615.00
MPH	0	0.00
ROOF	0	0.00
TOTAL GFA	**361196.0**	**33,556.0**

Maximum Floor GFA =	18,727.0	1,739.8
25% of each of the adjoing floors =	5,131.3	476.7
Total Floor Area =	23,858.3	2,216.5

PHASE V - GFA BREAKDOWN

BUILDING 1

Level	GFA (SF)	GFA (SM)
P6	841	67.4
P3 to P5	2,593	240.9
P2	1,401	130.1
P1	1,307	121.4
1	3,929	365.0
2	7,284	676.7
3	8,094	752.0
4	6,899	640.9
5	6,899	640.9
6	6,270	582.5
7 to 32	196,053	18,213.8
33	7,540	700.5
34	6,013	558.6
35	6,013	558.6
36	6,013	558.6
TOTAL GFA	**267,149.00**	**24,807.9**

Maximum Floor GFA =	8,094.0	752.0
25% of each of the adjoing floors =	3,545.8	329.4
Total Floor Area =	**11,639.8**	**1,081.4**

PHASE V - GFA BREAKDOWN

BUILDING 2

Level	GFA (SF)	GFA (SM)
1	16,310	1,515
2	15,304	1,421.8
3	20,886	1,940.4
4	11,203	1,040.8
5	15,956	1,482.3
6	8,517	791.3
7	9,966	925.9
8 to 9	19,796	1,839.1
10	9,966	925.9
11	5,176	480.9
12 to 33	165,020	15,330.7
34	7,489	695.7
35	6,756	627.7
36 to 37	13,512	1,255.3
38	5,468	508.0
39	5,468	508.0
40	5,468	508.0
TOTAL GFA	**342,261.00**	**31,796.8**

Maximum Floor GFA =	20,886.0	1,940.4
25% of each of the adjoing floors =	6,626.8	615.7
Total Floor Area =	**27,512.8**	**2,556.1**

PHASE V - GFA BREAKDOWN

BUILDING 3

Level	GFA (SF)	GFA (SM)
1	19,467	1,821.1
2	20,567	1,910.7
3	19,553	1,816.5
4	2,392	222.2
5	14,065	1,306.7
6	14,065	1,306.7
7	14,065	1,306.7
8	14,065	1,306.7
9	14,065	1,306.7
10	14,065	1,306.7
11	5,153	453.4
12 to 42	239,407	22,241.5
43	7,818	726.3
44	6,753	627.4
45 to 47	20,259	1,882.2
48	6,280	583.4
49	6,280	583.4
50	6,280	583.4
TOTAL GFA	**444,599.00**	**41,291.7**

Maximum Floor GFA =	20,567.0	1,910.7
25% of each of the adjoing floors =	9,755.0	909.4
Total Floor Area =	**30,322.0**	**2,820.1**

Detailed Water Demand Calculations

East Parcel	Block	Area (ha)	BLDG No.	Bldg Description	Commercial		# of Units			Population		Residential ADD (L/s)	
					Retail/Office SQM	Comm. ADD	Condo units	Town	BLK Total	Residential	ICI	Short Term	Long Term
Phase IV - Part II	Block 2	0.72	4	West building	0.0	0	317	0		856	0	4.1	2.8
	Block 16		3	East building	3011.5	0	435	0		1175	15	5.6	3.8
Sub - Total Phase IV Part II					**3,011.50**	**0**	**752**	**0**	**752**	**2030**	**15.06**	**9.61**	**6.58**
Phase V	Block 1	1.03	1	North building	2946.9	0	363	0	363	980	15	4.6	3.2
	Block 17		2	North building	2895.6	0	413	0	925	1115	14	5.3	3.6
			3	South building	2781.3	0	512	0		1382	14	6.5	4.5
Sub - Total Phase V		1.03			**8,623.80**	**0**	**1,288.00**	0	**1,288.00**	**7,538.40**	**43.12**	**16.5**	**11.3**

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 Maximum Day Short Term Demand Factor =	2	x Ave. Day Demand
Residential Peak Hour Demand Factor =	3	x Ave. Day Demand
ICI Max. Day Demand Factor =	1.4	x Ave. Day Demand
ICI Max. Day Short Term Demand Factor =	2.0	x Ave. Day Demand
ICI Peak Hour Demand Factor =	3	x Ave. Day Demand

	Typical Water Demand			Short Term Water Demand		
	Avg. Day (L/s)	Max Day (L/S)	Peak Hour (L/S)	Avg. Day	Max Day	Peak Hour
Phase IV Part II	6.63	13.23	19.90	9.66	13.26	19.90
Phase V	24.58	49.07	73.74	35.83	49.16	73.74
Total Water Demand	**31.21**	**62.30**	**93.64**	**45.50**	**62.42**	**93.64**

Connection Demand Table - Phase IV - Part II

WATER CONNECTION

Connection point [3]			
Watergarden Drive			
Pressure zone of connection point			
Total equivalent population to be serviced [1]			*2,399*
Total lands to be serviced			*1.04 ha*
Hydrant flow test			
Hydrant flow test location	*35 Watergraden Drive*		
(Northeast of Watergraden and Foursprings Ave.)			
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	*165*	*48.2*	*10:00 AM*
Maximum water pressure	*607*	*0*	*10:00 AM*

No.	Water demands			
		Demand (in l/s)		
	Demand type	Use 1 [5]	Use 2 [5]	Total
1	Average day flow	*6.58*	*0.05*	*6.63*
2	Maximum day flow	*13.16*	*0.07*	*13.23*
3	Peak hour flow	*19.74*	*0.15*	*19.89*
4	Fire flow [2]			*83.33*
Analysis				
5	Maximum day plus fire flow			*96.56*

LICENSED PROFESSIONAL ENGINEER
A. COVELLO
MARCH 22, 2019
PROVINCE OF ONTARIO

WASTEWATER CONNECTION

			Total
Connection point [4]			
Total equivalent population to be serviced [1]			*2,399*
Total lands to be serviced			*1.04 ha*
6 Wastewater sewer effluent (in l/s)			

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

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

Connection Demand Table - Phase V

WATER CONNECTION

Connection point [3)]			
Watergarden Drive			
Pressure zone of connection point			
Total equivalent population to be serviced [1)]			*3,520*
Total lands to be serviced			*1.19 ha*
Hydrant flow test			
Hydrant flow test location	*35 Watergraden Drive*		
(Northeast of Watergraden and Foursprings Ave.)			
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	*165*	*48.2*	*10:00 AM*
Maximum water pressure	*607*	*0*	*10:00 AM*

No.	Water demands			
		Demand (in l/s)		
	Demand type	Use 1 [5)]	Use 2 [5)]	Total
1	Average day flow	*11.27*	*13.31*	*24.58*
2	Maximum day flow	*22.54*	*18.634*	*41.174*
3	Peak hour flow	*33.81*	*39.93*	*73.74*
4	Fire flow [2)]			*83.33*
Analysis				
5	Maximum day plus fire flow			*124.504*

WASTEWATER CONNECTION

	Population	Area (ha)	Total
Connection point [4)]			
Watergraden Drive	2,138.00	0.65	
Little Creek Road	1,382.00	0.54	
Total equivalent population to be serviced [1)]			*3,520*
Total lands to be serviced			*1.19 ha*
6 Wastewater sewer effluent (in l/s)			

1) Please refer to design criteria for population equivelncies
2) Please reference the Fire Underwriters Survey Document
3) Please Specify the connectionpoint ID

4) Please specifiy the connection point (wastewater line or manhole ID). Also, the "total equivalent population to be serviced" and the "total lands to be serviced" should reference the connection point.

5) Please complete as many uses are nescessary for the development.

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