Marianneville Developments Ltd.

Functional Servicing Report

Estates of Glenway

Town of Newmarket

Project No. L09-301

March 2012

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Executive Summary

Cole Engineering Group was retained by Marianneville Developments Limited to undertake a Functional Servicing Report in support of the proposed Estates of Glenway Newmarket development. This report examines the existing sanitary and storm sewer conveyance network, water distribution network and stormwater management strategy, and recommends a servicing and road grading scheme to accommodate the proposed development's requirements in accordance with the Town of Newmarket and Ministry of the Environment standards.

The proposed development is situated within the eastern half of the existing Glenway Community, south of Davis Drive, generally between Bathurst Street and Eagle Street and consists of a combination of low, medium and high density residential units (total 730 units) and a small commercial block. The proposed development spans an area of 36.3 ha and is situated within several of the golf course holes of the former Glenway County Club. The proposed development plan consists of a combination of new public right-of-ways (Streets A, B, C and D) and private roads within the Medium Density and Condo Blocks.

A preliminary road grading design for proposed public and private streets has been achieved with road gradients conforming to municipal standards and largely maintaining the current stormwater runoff drainage patterns. Major system storm overland flow will be directed along the roads towards the existing stormwater ponds located throughout the site. The use of retaining walls will be required in certain areas to accommodate significant differences in elevation adjacent to existing properties.

New watermains will be required along proposed right-of-ways and private roads and shall connect to the existing water distribution network surrounding the site. Two Regional pressure districts are located within the proposed development, specifically the North Central District and North West District. Based on the elevation range serviced by each pressure district, the majority of proposed development will connect to the higher pressure district (North West) with the remaining, lower elevation development in the southeast corner of the site connected to the North Central district.

Sanitary flow generated from the proposed development will be conveyed via new sewers and connected to the existing surrounding sanitary sewer network at various locations. A sanitary flow monitoring program was completed from June 2010 to December 2010 to measure actual sanitary flow and calibrated to the Chicago 24hr Storm based on the recording of several rain events. The intent of the program was to determine a realistic peak sanitary flow rate from the existing Glenway Community compared to the original theoretical design flow for the existing subdivision. Based on monitored data just downstream of the sanitary outlet for the Community (MH110A, SE of Peevers Crescent), the additional sanitary flows generated from proposed development can be accommodated within the existing local sewer and sub-trunk system.

The stormwater management strategy to accommodate proposed development involves upgrading the existing ponds within the eastern half of the former golf course to meet the stormwater quality and quantity control requirements. The existing ponds are inline with the existing storm sewer system for the Glenway Community and provide limited stormwater runoff controls or water quality treatment. The ponds will be expanded in area and volume to meet current standards with outlet controls and quality treatment for existing and proposed development within the contributing drainage areas. Quantity control targets are existing pond outflows for the 2-year to 100-year 24 hour SCS storm, by Town of Newmarket Standards. The pond bottoms will be deepened and reshaped to provide Enhanced (Level 1) Quality Control as outlined by the MOE.

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1.0 Introduction

1.1. Scope of Functional Servicing Report

Cole Engineering Group Ltd. (Cole Engineering) has been retained by Marianneville Developments Ltd. to prepare a Functional Servicing Report (FSR) in support of Official Plan Amendment, Re-Zoning and Draft Plan of Subdivision applications for a proposed residential development located within the existing Glenway Estates and Country Club Community in the Town of Newmarket, Regional Municipality of York.

This report has been prepared to review the existing sanitary servicing, water distribution network, storm sewer systems and stormwater management features and provide recommendations for their potential improvements required to accommodate proposed development based on the proposed Draft Plan prepared by Zelinka Priamo Limited, dated March 2012. This FSR also includes the preliminary road grading design for the proposed development areas covered by the proposed Draft Plan.

1.2. Background Review

The following background studies and information were referenced while preparing this Report:

- As Constructed Engineering Servicing Drawings, Glenway Estates & Country Club, prepared by G.M. Sernas and Associates Limited (file #8202), 1989;
- Stormwater Management Study, Glenway Estates & Country Club, prepared by The Lathem Group Inc., dated December 19, 1983; and,
- Environmental Assessment, Glenway Reservoir Expansion, prepared by GHD Inc., dated July 4, 2011.

1.3. Site Location

The Glenway Estates and Country Club Community are bordered by Davis Drive (formerly Highway 9) to the north, Bathurst Street to the west and west of Yonge Street to the east. The Community's southern boundary is generally defined by the existing Summerhill Subdivision (Binns Ave.) and the Ray Twinney Recreation Complex.

Situated within the Glenway Community, the subject site is generally located within the eastern half of the Community, bordered by Davis Drive to the north, Eagle Street to the east, Crossland Gate to the south and the existing Hydro One corridor to the west.

Refer to **Figure 1-1** for a depiction of the Glenway Community boundaries.

1.4. Existing Conditions

The Glenway Community consists primarily of low density residential land uses surrounded by an 18-hole golf course. Medium density residential and commercial land uses exist within the northeast quadrant of the Community. The subject site is situated within the eastern half of the 18-hole golf course. The Community is bisected by an existing Hydro One corridor (approximately 38m in width) centrally aligned through the Community from north to south. Four (4) existing hydro towers are aligned within the corridor through the Community.

Spread across the Community, the existing golf course consists of landscaped open space with several stands of trees primarily aligned adjacent to the existing residences. A total of nine (9) existing stormwater ponds are located within the golf course lands, which services the surrounding residential units and the golf course itself. These ponds take the form of water hazards throughout the course and serve an aesthetic as well as functional purpose. Four (4) of the stormwater ponds are located on the eastern half of the golf course with the remaining five (5) ponds located in the western half of the golf course.

Also internal to the Community is the Glenway Reservoir site located at the northwest corner Kirby Crescent. Currently the site has a pump house and chlorination building in addition to an above ground storage reservoir positioned to the west of the pump house. The Region of York (the “Region”) has recently completed an Environmental Assessment to locate a second water storage reservoir on the Kirby Crescent site.

In addition to the golf course and low density residential, the existing Glenway Community includes additional land uses within the northeastern quadrant such as retail complexes, restaurants and smaller strip-mall type commercial areas. The southeast corner of the intersection of Eagle Street and Davis Drive contains a private townhouse complex (Newmarket Cooperative), while the Newmarket GO Transit Bus Terminal is located at the southwest corner.

Figure 1-1 Location Plan

2.0 Proposed Development

The proposed re-development of the eastern half of the existing 18-hole golf course consists primarily of a combination of low, medium and high density residential land uses ranging between the existing hydro corridor to just east of Eagle Street. In addition, a small commercial block is proposed at the southwest corner of Crossland Gate and Davis Drive. The proposed road network and lot layout is based on the Draft Plan of Subdivision prepared by Zelinka Priamo Ltd., dated March 2012.

Table 2.1 summarizes the proposed land uses and corresponding development areas.

Table 2.1 – Proposed Land Uses and Areas

Land Use	Units	PPU	Population	Areas (hectares)
Residential (Lots 1 – 165)	165	3.38	558	11.81
Residential, Medium Density (Blocks 166-168)	219	2.88	631	7.60
Residential, Condos (Blocks 169 – 170)	54	3.38	183	7.85
Residential, High Density (Block 171)	292	1.95	569	2.34
Commercial (Block 172)				0.65
Parkland (Block 173)				2.34
Proposed Roadways (Public)				3.71
Total	**730**		**1941**	**36.30**

New municipal right-of-ways are proposed for 159 proposed residential lots along Streets 'A', 'B', 'C' and 'D'. The remaining six (6) proposed residential lots shall front onto existing Alex Doner Drive. The Medium Density, Condo and High Density Blocks will utilize private internal roads, however in certain instances, municipal servicing extensions will be required through the private development blocks within an appropriate easement in favour of the municipality. Servicing requirements are discussed in the following chapters. Refer to **Figure 2-1** which indicates the public and private development areas.

The existing stormwater ponds within the development area to the east of the hydro corridor will continue to service the surrounding lands and will be enhanced to suit the requirements of the proposed development. The ponds are currently located within the private golf course property and will continue to be privately owned and function within the developed private residential blocks.

The existing land use to the west of the hydro corridor will be subject to minimal disturbance. Six (6) single family dwellings will be constructed along existing Alex Doner Drive. Parkland (Block 173) will be developed between the hydro corridor and the existing lots on Kirby Crescent to create community lands which are publicly accessible. The golf course property on the west side of the hydro corridor is intended to be re-designed and re-opened as an executive 9-hole golf course, which is subject to a separate municipal application process.

Figure 2-1 Re-Development Boundaries

3.0 Area Grading

3.1. Existing Topography

The existing grading conditions within the Glenway Community are reflective of their current use, i.e. as an 18-hole golf course adjacent to an existing residential community.

The existing roads within the residential component of the Community are generally graded with slopes ranging between 1 - 2%. A couple of existing roads were constructed with grades ranging from 4% (Alex Doner Drive) to 6% (Kirby Crescent). The existing roads convey major storm drainage towards the existing stormwater ponds located throughout the golf course lands. The 18-hole golf course lands were graded to suit the operational conditions required of a golf course combined with the original topography. The area proposed for re-development is generally situated within the golf course holes to the east of the hydro corridor.

The existing topography within the golf course holes east of the hydro corridor generally ranges from a high elevation of 282.50m in the northeast corner, adjacent to Davis Drive, to a low elevation of 268.0m in the south east corner of the site at Eagle Street and Crossland Gate. Within this elevation range, the existing topography in certain locations exhibits significant gradient differential reflective of the rolling nature of golf courses.

Overland drainage from the golf course holes located to the east of the hydro corridor is currently divided into four (4) separate drainage areas directing runoff to four (4) separate stormwater ponds (Ponds #4a/4b, #6, #8 and #9) which outlet to two (2) separate drainage outlets. The first outlet is located at the southeast corner of the site at Eagle Street, just north of Crossland Gate, directing flows to Western Creek. The second outlet from the site is located at Davis Drive, just east of Crossland Gate and directs flows in a northerly direction across Davis Drive.

3.2. Proposed Grading

A preliminary grading plan has been prepared for the proposed roads and lots within the subject lands. Perimeter grades along the existing residential lots and along the existing abutting municipal right-of-ways will be maintained. The proposed grades along the Davis Drive south streetline are set at 0.30m above the existing centre line of the road, based on typical Regional requirements. Grading encroachments within the existing Hydro One corridor are proposed to accommodate vertical grade differentials between the existing ground and proposed window roads adjacent the corridor.

The preliminary grading scheme is developed based on the current Town of Newmarket Engineering Design Standards and Criteria and defines the major system drainage divides to conform to the proposed stormwater management strategy described within **Section 7.0** of this report.

The proposed public roads are generally graded in the range of 1% to 2.5% with only limited sections graded in 3.5% - 3.7% range. The road grading along the private roads ranges from 0.7% to 3.7%.

The proposed development will utilize conventional lot drainage patterns such as Front and Split drainage, where possible. In areas where grading is constricted due to significant grade differences with existing perimeter grades, the use of Walk-Out and Walk-Up lot types utilizing 3:1 sloping will be specified, requiring additional attention at the detailed design stage. As the proposed development can be defined as being of "infill type", the use of rear lot catchbasins will be necessary to contain minor storm runoff within the proposed lots. In certain areas where significant grade differences occur along the site perimeter, retaining walls will be necessary to be constructed within the proposed development area. Specific retaining wall type and material will be confirmed during the detailed design stage, in consultation with the Town of Newmarket staff and the project landscape architect. In accordance with accepted best practices, including use of slopes (maximum of 3:1) and surface or swale gradients ranging from 2%-5%, the use of retaining walls will be minimized wherever possible.

Following the recommendations of the Tree Inventory report prepared by York Urbanist, special attention has been used when designing the grading of the proposed lots 109 & 110 located at the southern tip of Street B. A significant tree specimen (90cm dia Ash – Figure L2 -tree 2E-9) was identified for preservation and is located along the common lot line between lots 109/110. Under the current grading concept, the existing grades within an 8m radius surrounding the tree will be preserved. Specific protection measures to be applied during construction will be confirmed during the detail design stage.

The grading designs for Block 168 and Block 171 located in the northeast corner of the subject lands (medium and high density residential) are developed at the conceptual level only with full details to be provided during the Site Plan submission stage. A schematic internal road grading design is prepared to depict the general drainage direction and compatibility with surrounding perimeter grades.

The preliminary road and lot grading design for Blocks 167, 169 and 170 have been advanced with additional detail since these lands will serve as the corridors linking the existing and proposed municipal roads where the full servicing and overland flow routes are proposed.

The grading design for Medium Density Block 166, previously the location of the Glenway Country Club Clubhouse, will be compatible with the existing streetline grades of Crossland Gate, Alex Doner Drive and maintaining the existing elevations of all other perimeter conditions.

The preliminary road and lot grading design is illustrated on **DWG GR-1**.

3.3. Erosion and Sediment Control

Prior to any construction within the site, a comprehensive Erosion and Sediment Control Plan acceptable to the Town of Newmarket and Regional Municipality of York would be implemented.

The Erosion and Sediment Control Plan will detail all necessary measures and will be designed in accordance with current Town guidelines and the Erosion and Sediment Control Guidelines for Urban Construction. In addition, Town and / or Regional approval will be secured for the location of the temporary construction entrance.

4.0 Water Supply and Distribution System

4.1. Existing Water Supply and Distribution Network

4.1.1. Existing Pressure Districts

The subject site covers approximately 36 ha and is situated to the east of the existing Glenway Reservoir and Kirby Pumping Station, located at 335 Kirby Crescent, Newmarket. Based on Pressure District mapping prepared by The Regional Municipality of York, there are three (3) distinct Pressure Districts within the Town of Newmarket, specifically:

1. Newmarket West District (NW);
2. Newmarket Central District (NC); and,
3. Newmarket East District (NE).

The Region's pressure district mapping indicates a pressure zone divide aligned through the Glenway Community, specifically the divide between the Newmarket Central District and Newmarket West District. The Pressure District boundary between the NC and NW pressure zones generally follows the existing Hydro One corridor, centrally aligned through the Glenway Community in a northwest to southeast direction.

The Newmarket Central District is the largest pressure district within Newmarket and generally extends from Yonge St. / Glenway Community to west of Leslie Street. Municipal water for the NC District is supplied via a series of wells along Yonge Street and from the Newmarket East District via a pressure reducing valve on Davis Drive. Storage for the NC District is provided from the Glenway Reservoir, London Road Elevated Tank and Magna Elevated Tank.

The Newmarket West District is supplied with municipal water from the Newmarket Central District via the existing Kirby Pumping Station. Storage for the NC District is provided by the Newmarket West Reservoir, located at Bathurst Street, between Mulock Drive and St. John's Sideroad.

Existing residences within the Glenway Community situated to the east of the Hydro One corridor are serviced by municipal water connected to the NC pressure district, while existing residences to the west of the corridor are connected to the NW pressure district.

The existing ground elevation within the site ranges from approximately 282m in the northern portion to 269m in the southern portion of the site.

Based on the Region's Pressure District data, the Glenway Reservoir exhibits a low water level of 300.8m and a high water level of 308.4 m. thus, the NC pressure district can generally service development areas with elevations lower than 273.5m. Development areas with ground elevations higher than 273.5m may be serviced by the NW pressure district, for which the system pressure is controlled by the Newmarket West elevated tank (water level range of 328m to 340 m). Refer to **Figure 4-1** for a depiction of the pressure district zones within the Glenway Community boundaries.

Figure 4-1 Water Pressure Districts

4.1.2. Existing Water Distribution Network

The site is surrounded by several existing watermains serving both the Newmarket Central and West pressure districts. The following lists presents the existing watermains located generally east of the existing Glenway Reservoir, separated based on their respective pressure zone:

- Newmarket Central District (suction supply pipelines to the Glenway Reservoir)
- Eagle Street: 200mm dia. – 300mm dia. watermains from Davis Drive to Peevers Crescent.
- Millard Avenue W.: 200mm dia. watermain from Eagle Street easterly.
- Bowser Crescent (south leg): 300mm dia. watermain.
- Crossland Gate: 300mm dia. – 200mm dia. watermains from Eagle Street to the Hydro One corridor.
- Fairway Garden: 300mm dia. – 200mm dia. watermains.
- Easement (Bowser Crescent to Fairway Garden): 300mm dia. watermain.
- Easement (Crossland Gate to Kirby Crescent): 300mm dia. watermain.
- Alex Doner Drive: 300mm dia. watermain from Crossland Gate to Kirby Crescent.
- Kirby Crescent (north and east legs): 400mm dia. watermain.
- Newmarket West District (discharge pipelines from the Kirby Pumping Station)
- Easements (Kirby Pumping Station to Alex Doner Drive): 200mm dia. and 400mm dia. watermains within separate easements.
- Alex Doner Drive: 200mm dia. watermain from Hydro One corridor westerly.
- Kirby Crescent: 150mm dia. – 200mm dia. watermains.

Refer to **Drawing WAT-2** provided at the end of the report for the location of the existing watermains.

4.1.3. Existing System Pressure

In order to investigate the capabilities of the existing water distribution system in the vicinity of the proposed development, Cole Engineering Group retained Applied Fire Technology Inc. to conduct hydrant flow / pressure tests. Two (2) hydrant flow / pressure tests (one (1) at NC pressure district and another at NW pressure district) were performed along the existing watermain in the vicinity of the proposed development.

1st test at NC pressure district: The first hydrant flow / pressure test was conducted along Alex Doner Drive in the NC pressure district on October 6, 2009. The static pressures are approximately 55 psi (system head = 308m), which is approximately equal to the high water level (water depth = 8m) at the Glenway reservoir. The pressure drops by approximately 4 psi (3m with a corresponding system head of 305m) when it is flowing at 107 L/s. The system head at this flow test location might have been lower (approximately equal to 298 m = 301m - 3 m) if the Glenway Reservoir was near its low water level at 301m (7 m lower than the reservoir water level during the test day).

2nd test at NW pressure district: The second hydrant flow / pressure test was conducted along Alex Doner Drive in the NC pressure district on October 15, 2009. The static pressures are approximately 65 psi (system head = 332m), which is approximately equal to the 33% full water level (water depth = 4m) at the Newmarket West elevated tank level (if there is no significant head loss in the system). The pressure drops 15 psi (10m, corresponding system head of 321m) when it is flowing at 87 L/s. The pressure at this flow test location might have been lower if the system head is 4 m lower than 332m during the test day.

Refer to **Appendix A.1** for the results of the hydrant flow / pressure tests completed by Applied Fire Technology Inc.

4.2. Design Guidelines

For the purposes of this report, the 2008 Ministry of Environment (MOE) Guidelines for the Design of Water Distribution Systems and the Town of Newmarket's design standards for the municipal water distribution system layout were used to estimate the system design pressure and demand requirements for the subject development.

The following design guidelines were used to estimate the water demand for the subject site:

4.2.1. Domestic Water Demand

The average day water demand of 300 L/cap/day was adopted from the Town's 2009 standards.

4.2.2. Peaking Factor

The peaking factors were taken from the Town's 2009 standards. The peaking factors for the Maximum Day and Peak Hour demand scenarios are 2.0 and 3.0 respectively.

4.2.3. Population Density in Residential Development

As per the Town's 2009 standards, the following densities were used to determine the expected populations in the residential developments:

- Single Detached Dwellings: 3.378 ppu
- Semi-Detached Dwellings: 3.378 ppu
- Townhouses: 2.88 ppu
- Apartments: 1.95 ppu

4.2.4. Water Demand for the Commercial Development

The Town's 2009 guidelines provide the following water demand requirements for commercial area:

- Retail & Office: 4 L/d/m2
- Restaurant: 60 L/d/m2

The 2008 MOE Guidelines stipulates a water demand rate of 28 m^3/ha for the commercial areas. Due to the unknown floor areas for the type of retail / office and restaurant in the proposed commercial area, the MOE guideline was used to estimate the domestic water demand for the commercial area.

4.2.5. Fire Flow

As per the Town's requirement, the minimum fire flow requirement is as follows:

- Detached and semi-detached dwellings: 7,000 L/min (117 L/s)
- Townhouses: 10,000 L/min (167 L/s)
- Apartment: 15,000 L/min (250 L/s)
- Industrial / commercial: 15,900 L/min (265 L/s)

The fire flow for the commercial development was determined using FUS, 1999.

4.2.6. System Pressure

The Town of Newmarket's 2009 standards provides the following system pressure requirements:

- Minimum pressure during peak hourly demand: 350 kPa (50 psi)
- Maximum pressure under any flow scenario: 550 kPa (80 psi)
- Minimum pressure during maximum day + fire flow: 140 kPa (20 psi)

The 2008 MOE Guidelines provide the following system pressure requirements:

- Minimum pressure during peak hourly demand: 275 kPa
- Maximum pressure under any flow scenario: 700 kPa
- Minimum pressure during maximum day + fire flow: 140 kPa

4.2.7. Selection of Watermain Sizes

The suggested Hazen-Williams C factors are to be used to size pipes within the subject site as per the Town's 2009 design standards:

- 150 mm: C = 100
- 200 – 250 mm: C = 110
- 300 mm or larger: C = 120

4.3. Proposed Development

Based on the Draft Plan of Subdivision prepared by Zelinka Priamo Ltd., dated March 2012, the proposed land uses for the Glenway Country Club Re-development (dated March 2012) consists of a combination of low, medium and high density residential uses and a small commercial parcel. A total of 730 residential units are proposed, generally east of the existing Hydro One corridor.

The proposed residential development shall be connected to existing watermains within either the Newmarket Central or Newmarket West pressure districts, based on the proposed ground elevation surrounding the new units. Development areas exhibiting proposed ground elevations lower than 273.5m shall be connected to the Newmarket Central District while development areas with ground elevations higher than 273.5m will be connected to the Newmarket West District.

4.3.1. Estimated Water Demand

Based on the Town's standards for the proposed residential development area and the MOE's guidelines for proposed commercial area, the estimated water demands for the subject site are summarized within **Table 4.1** below. The domestic water demand for the development requires flows of 8 L/s, 14 L/s and 21 L/s for the average day, maximum day and peak hour conditions, respectively. The Town's required fire flow of 117, 167, 250 and 267 L/s is for the low density residential, medium density residential, high density residential and commercial, respectively. The fire flow for the commercial area is calculated based on criteria from Fire Underwriters Survey 1999, while the remaining development areas rely on the Town's suggested fire flow.

Table 4.1 – Water Demand Estimation

Land Use	Residential (units)	Pop.	Water Demand (L/s)			
			Average Day	Max day AM	Peak Hour	Fire Flow
Low Density Residential	219	741	2	4	7	117
Medium Density Residential	219	631	3	5	8	167
High Density Residential	292	569	2	4	6	250
Commercial	-	-	1	1	1	167*
Sub-total	**730**	**1941**	**8**	**14**	**21**	**-**

*Fire Flow was estimated from the FUS, 1999 guideline.

Based of the proposed commercial block area (0.65 ha), the required fire flow is 167 L/s using the Fire Underwriters Survey criteria, 1999. It was assumed that the commercial buildings will consist of fire-resistive construction (e.g. fully protected frame, floors and roof, etc.) and limited-combustible occupancies, with a minimum of 3m spacing separation from the other buildings. The building is to be provided with an adequately designed water system conforming to NFPA sprinkler standards. Refer to detailed calculations located in **Appendix A.1**.

4.3.2. Newmarket Central District Connections

Two proposed development areas are situated below an elevation of 273.5m and will be connected to the Newmarket Central District water distribution network:

- Street 'D' (Single Family Residential): located to the east of Eagle Street; this area exhibits a proposed ground elevation range of approximately 272.80m to 268.80m and shall connect to the existing NC District 200mm dia. watermain on Millard Avenue West and to the existing NC District 200mm dia. watermain on Eagle Street. The proposed municipal watermain shall be aligned along the proposed municipal right-of-way.

- Block 170, south leg (Condo Residential): exhibits a proposed ground elevation range of approximately 268.0m to 269.0m. The primary water connection for this development area shall be to the existing NC District 200mm dia. watermain on Eagle Street with a secondary connection to a proposed public / private NW District watermain extended along Alex Doner Drive and aligned between the existing gap in residential units on Crossland Gate. Since the proposed watermain extension into Block 170 from Alex Doner Drive is serviced from the Newmarket West District, the private NC watermain within Block 170 (south leg) will be separated from the NW watermain by a proposed valve chamber with a small diameter by-pass line for water quality circulation purposes only.

The proposed development is situated near the Glenway Reservoir. With the existing 300 mm (and 400 mm) pipeline and current looping system surrounding the proposed development in the NC district, no significant head loss between the Glenway Reservoir and the proposed development is anticipated. The system head for the area connected to NC system is approximately equal to the water level (300.8 m to 308.4 m) of the Glenway Reservoir under normal conditions. The proposed ground elevations are between 268 m and 273 m. The estimated maximum pressure and minimum pressure system for the area to be connected to NC District are summarized in **Table 4.2** and detailed as follows:

4.3.2.1 System Pressure under Normal Operation

The maximum pressure likely occurs at the relatively low ground location (elevation equal = 268 near Block 13 residential at Crossland Gate & Eagle Street. The estimated maximum system head is approximately equal to 308.4 m (equal to the high water level at Glenway Reservoir) and the maximum pressure is approximately equal to 40 m (395 kPa). The minimum pressure likely occurs at the relatively high ground location (elevation = 273 m) at the middle of Street D (north Crossland Gate & Eagle Street) and the minimum pressure is approximately to 28 m (275 kPa) when the Glenway Reservoir is near its low water level.

4.3.2.2 Minimum Pressure under Fire Flow Condition

The maximum pressure likely occurs at the relatively high ground location at the middle of Street D (north Crossland Gate & Eagle Street). The estimated system head is approximately equal to 301 m (additional 6 m head loss across the proposed 250 mm of 200 m pipeline along Street D with assuming 50 % (60 L/s) of the design flow rate via one side of connection at Crossland Gate and Eagle Street).

Table 4.2 – Proposed System Pressures for the Development Area Connected to NC District

Design Conditions	Maximum*		Minimum**	
	Head (m)	Pressure (m)	Head (m)	Pressure (m)
Normal Operation	308m	40 (390 kPa)	301m	28 (275 kPa)
Maximum Day + Fire Demand	-	-	298m	25 (245 kPa)

*Maximum pressure likely occurs near the relatively low ground elevation (=268 m) near Block 19 at Crossland Gate & Eagle Street and near high system head of 308 m.

**Minimum pressure likely occurs near the relatively high ground elevation (=273 m) at the middle of Street D when the system is near its low system head of 301 m.

4.3.3. Newmarket West District Connections

The remaining development areas within the site will be connected to the Newmarket West District since the proposed ground elevations in these areas are above 273.5m. The proposed development areas within the Newmarket West District can be divided into eight distinct areas and their proposed water connections are described as follows:

- Block 166 (Medium Density Residential) and Block 172 (Commercial): both parcels are located in the northwestern portion of the site and exhibit a proposed ground elevation range of approximately 272.40m to 275.0m and shall connect to the NW District via connections to an existing 200mm dia. watermain along Alex Doner Drive. Currently the existing watermain on Alex Doner is serviced by the NC District and will be switched to the NW District through the installation of a new check valve on the Alex Doner Dr. watermain in front of proposed lot no. 69. Coupled with the re-arrangement of valving within the existing check valve chamber on Alex Doner Drive near the Hydro One corridor, this will effectively transition a section of the Alex Doner Drive watermain from the NC District to the NW District. Private watermains shall be extended within Blocks 166 and 172 to service the proposed townhouse units and commercial development, respectively.
- Block 167 (Medium Density Residential): exhibits a proposed ground elevation range of approximately 273.60m to 281.0m and is situated adjacent to Davis Drive. A proposed municipal watermain will be extended through the private townhouse development under appropriate municipal easement. Connections to the adjacent municipal water distribution network will occur at Alex Doner Drive (to the NW District 200mm dia. main) and along proposed Street ‘B’ (to the NW District main). The municipal designation of the watermain through the private development is required to provide a second water feed to Streets ‘B’ and ‘C’.
- Block 168 (Medium Density Residential) and Block 171 (High Density Residential): The proposed ground elevation is approximately 282.89 m at Block 171 (High Density Residential). The proposed water connections for the townhouse and high rise complex shall be provided from the proposed NW District watermain aligned along Street ‘B’.
- Lots 1 - 6 (Single Family Residential): fronting along Alex Doner Drive, these six proposed lots located to the west of the existing Hydro One corridor will be individually connected to the existing 200mm dia. watermain aligned within Alex Doner Drive.
- Street ‘A’ (Single Family Residential): situated adjacent to the existing Hydro One corridor, this area exhibits a proposed ground elevation range of approximately 272.0m to 280.0m. A proposed municipal watermain shall be aligned along the proposed right-of-way connecting at the southern end to a proposed NW District watermain extension aligned easterly along Alex Doner Drive from its current terminus at the existing Hydro One corridor. At the northern end of Street ‘A’, the proposed watermain shall connect to a proposed NW District watermain extended along Alex Doner Drive from the existing Hydro Corridor.
- Streets ‘B’ and ‘C’ (Single Family Residential): situated between Fairway Garden and Bowser Crescent, this area exhibits a proposed ground elevation range of approximately 273.50m to 281.0m. Proposed municipal watermains shall be aligned along the proposed right-of-ways and connected to a proposed municipal watermain extended through Block 167 at the northern end of Street ‘B’ and a proposed municipal watermain extended from Block 170 at the southern end.

- Block 169 (Condo Residential): this area exhibits a proposed ground elevation range of approximately 272.50m to 277.50m. A private NW District watermain shall be extended internally through Block 169, with a primary water connection to the proposed NW District municipal watermain on Street 'B' and a secondary water connection to the existing NC District 200mm dia. watermain on Eagle Street. The private NC watermain connection to Eagle Street will be separated from the NW watermain by a proposed valve chamber with a small diameter by-pass line for water quality circulation purposes only.

- Block 170, west leg (Condo Residential): this area exhibits a proposed ground elevation range of approximately 270.75m to 273.25m. A proposed municipal watermain will be extended through the private townhouse development under appropriate municipal easement. Connections to the adjacent municipal water distribution network will occur at Alex Doner Drive/Crossland Gate (to the proposed NW District 200mm dia. main extension) and at proposed Street 'B' (to the proposed NW District main). The municipal designation of the watermain through the private development is required to provide a second water feed to Streets 'B' and 'C'. As a result of extending the NW District municipal watermain through the western section of Block 170, proposed condo units no.'s 1 – 7 and 18 – 24 may require individual pressure reducing fixtures on their unit water service connections since the proposed elevation for this area is below the threshold of 273.50m for the NW District and these units may otherwise be subject to higher than acceptable water pressure.

The proposed connection information above is based on the Region's water system information and assuming the system head is approximately equal to the water level (327.5 to 340 m) at the Newmarket West Elevated Tank under normal operating conditions. The estimated maximum pressure and minimum pressure system for the area to be connected to NW are summarized in **Table 4.3** and detailed as follows:

4.3.3.1 System Pressure under Normal Operation

The maximum pressure likely occurs at the relatively low ground location (elevation equal = 271 m) near Block 170 (Condos residential) at Crossland Gate & Eagle Street. The estimated system head is approximately equal to 340 m (equal to the high water level at Newmarket west elevated tank) and the maximum pressure is approximately equal to 69 m (680 kPa). The minimum pressure likely occurs at the relatively high ground location (elevation = 283 m) near Block 171 (High Density apartment residential). It is approximately equal to 45 m (440 kPa) under normal operation.

4.3.3.2 Minimum System Pressure under Fire Flow

The minimum pressure likely occurs at the relatively high ground location and large required fire flow near Block 171 (High Density apartment residential). The required fire flow is 250 L/s as per Town's guideline, the estimated system head is approximately equal to 304 m when the system head near it's low level (= 328 m, the lowest water level at Newmarket West elevated tank). Approximately 75% (200 L/s) of the design fire flow 250 L/s is from the proposed 300 mm along the Blocks 167 and 168 Medium density residential and the other 25 % (50 L/s) of the fire flow is supplied from the proposed 250 mm pipeline along Street B.

Table 4.3 – Proposed System Pressures for the Area connection to NW

Design Conditions	Maximum *		Minimum**	
	Head (m)	Pressure (kPa)	Head (m)	Pressure (kPa)
Normal Operation	340m	69m (680kPa)	328m	45m (441 kPa)
Maximum Day + Fire Demand	-	-	304m	21 (200 kPa)

*Maximum pressure likely occurs near the relatively low ground elevation (=271 m) near Block 170 (Condos Residential) at Crossland Gate & Eagle Street and near high system head of 340m.

**Minimum pressure likely occurs near the relatively high ground elevation (=283 m) at the Block 171 High Density Residential when the system is near its low system head of 328m.

5.0 Storm Drainage

5.1. Minor Storm Drainage System

The minor storm drainage system for the overall plan area will be designed in accordance with the Town of Newmarket and MOE criteria, including the following criteria:

- Storm sewers to be sized to accommodate runoff from a 5-year storm event;
- Minimum flow velocity – 0.8 m/s;
- Maximum flow velocity – 4.0 m/s;
- Minimum pipe size – 300 mm; and,
- Minimum pipe depth – 2.7 m measured to obvert.

As defined by the above standards, the minor storm flows will be captured by the underground sewers. Where the size of the post development drainage basin is below 2.0 ha, quality and quantity control can be provided by the installation of oil / grit separator units and the creation of temporary stormwater storage within the underground (and oversized) storm sewers. The sewers will be constructed along the municipal and private roads closely following typical road cross-section configurations. The sewers will outlet to the existing Stormwater Management Ponds 4A, 4B, 6, 8 and 9 all being positioned east of the existing Hydro Corridor.

The proposed configuration of the storm sewer system is shown schematically on **Dwg. STM-1**.

5.2. Major Storm Drainage System

Storm drainage flows exceeding the design capacity of the underground sewers, which are sized to convey the minor storm flows, will be directed overland along the road surfaces. The use of inlet control devices (ICDs) placed in catchbasins will be implemented, where necessary, to control the rate of stormwater entering the storm sewers. Specific positions for the ICD's will be established at the detailed design stage. The conveyance capacity of the proposed roads will also be analyzed during final design stage, taking into consideration width of pavement, type of curb and road gradient. Any overland flows directed along the municipal roads will be fully contained within the street right-of-way, while for the private roads the analysis will take into consideration the minimum horizontal and vertical distances to any structure (garage, home). If required, any major flows conveyed on the municipal road surface will be captured into the underground mains before entering the condominium areas (Blocks 169 & 170). The need for easements and their extent will be confirmed during detailed design stage when final configuration of the development plan is established.

As described above, all major storm runoff will be directed to the existing stormwater ponds 4A 4B, 6, 8 and 9. **Section 7.0** of this FSR provides functional design details for the improvements to the existing stormwater ponds to accommodate post-development drainage conditions.

6.0 Sanitary Sewers

6.1. Existing Conditions

The existing municipal sanitary sewer network servicing the Glenway Community is composed of two (2) main branches ranging in size from 250mm dia. to 450mm dia. Generally, sanitary flows are conveyed from the northwest to southeast direction, towards an existing 450mm dia. sanitary sub-trunk sewer located along Peevers Crescent.

The main branch of the sanitary sewer network within the Community is aligned along Peevers Crescent and Crossland Gate and conveys sanitary flows in a southeasterly direction. The main branch services the majority of the existing Glenway residential community, west of Eagle Street. The sewer size ranges between 300mm dia. to 450mm dia. for a significant section of the downstream sewer. The second branch of the sanitary sewer network is aligned along Eagle Street and directs sanitary flows southerly towards the Peevers Crescent sanitary sub-trunk. The Eagle Street sanitary sewer services the easterly portion of the Glenway Community, including the existing Go Bus Station at Eagle Street and Davis Drive and commercial lands located in the northwest corner of Millard Avenue W. and Yonge Street. The sewer branches combine and convey sanitary flow through an existing 450mm dia. sanitary sub-trunk outleting from the southeast corner of Peevers Crescent, just south of the Regional Municipality of York's Administrative Centre. The 450mm dia. sub-trunk directs sewage towards the intersection of Yonge Street and Eagle Street, through York Region's open space and parking area to the south of the Administrative Centre.

Based on the "As Constructed" Sanitary Sewer Design Sheets for the Glenway Community, prepared by G.M. Sernas & Associates, revision dated January 3, 1995, the theoretical design peak flow rate (including an allowance for infiltration) from the entire Glenway Community is calculated at 177 L/s. This theoretical sanitary sewer flow rate was designed between Ex. MH120A to Ex. MH104A / Ex. MH 110A. Refer to **Appendix B** for the As Constructed Sanitary Sewer design sheet by G.M. Sernas & Associates. Based on the municipal standards available when the original Glenway subdivision was designed, the following sanitary flow rates were used to develop design peak flows:

- Single Family (15m): 0.0013 cms/ha
- Single Family (9.75m): 0.0016 cms/ha
- Commercial / Industrial: 0.0017 cms/ha
- School / Multi Family: 0.0025 cms/ha

Compared to present day municipal standards to calculate sanitary flow generation, the above noted flow rates are conservative and produce higher design flows.

Downstream of Ex. MH 110A, additional sewage is directed easterly towards the Glenway Community's sanitary outlet sewer, specifically towards Ex.MH 112A and directed easterly across Yonge Street. Downstream of Ex. MH 112A, the existing sanitary trunk system is referred to as the *Western Sub-Trunk Sanitary Sewer.* The Western Sub-Trunk Sanitary Sewer conveys flows northeasterly towards the Bayview Sewage Pumping Station.

6.2. Existing Sanitary Flow Analysis

In order to determine available sanitary sewer capacities within the existing system, Cole Engineering undertook a 6-month sanitary sewer flow monitoring program in the later half of 2010. The intent of the monitoring program was to correlate actual sanitary flows measured within the 450mm dia. sewer outletting from the Glenway Community at Peevers Ave. against the original theoretical sanitary design flows within the As Constructed Sanitary Sewer design sheet. The location selected for sanitary flow monitoring was within Ex. MH 110A, located to the northwest of Eagle Street and Yonge Street.

The findings of the monitoring program are described below.

6.2.1. Flow and Precipitation Monitoring

Sanitary flow and precipitation monitoring was completed from June 1st, 2010 to December 7, 2010 and all data collected is presented in **Appendix A.2**. Throughout the monitoring period, several large storms were captured including a 48 mm event on July 23, 2010 as well as several storms greater than 20 mm. The monitoring equipment used included redundant depth sensors and a velocity sensor that provided 100% data coverage for the duration of the monitoring period. Periodic maintenance visits were performed to confirm all sensors were working within normal parameters; no debris was built-up on the sensor and good sewer hydraulics was maintained.

6.2.2. Modeling and Data Analysis

Using the monitored precipitation and flow data in combination with existing and proposed land use conditions, the InfoWorks hydrodynamic model was prepared to assess flows within the existing sewer.

6.2.2.1 Rainfall and Flow Data Screening

Several high intensity rain events were selected for model calibration. **Table 6.1** summarizes the rainfall intensities and depths during the four (4) largest events which have wet weather flow response. **Figure 6-1** shows the Intensity-Duration-Frequency curves calculated for each of these events.

Table 6.1 – Rainfall Intensities and Volumes

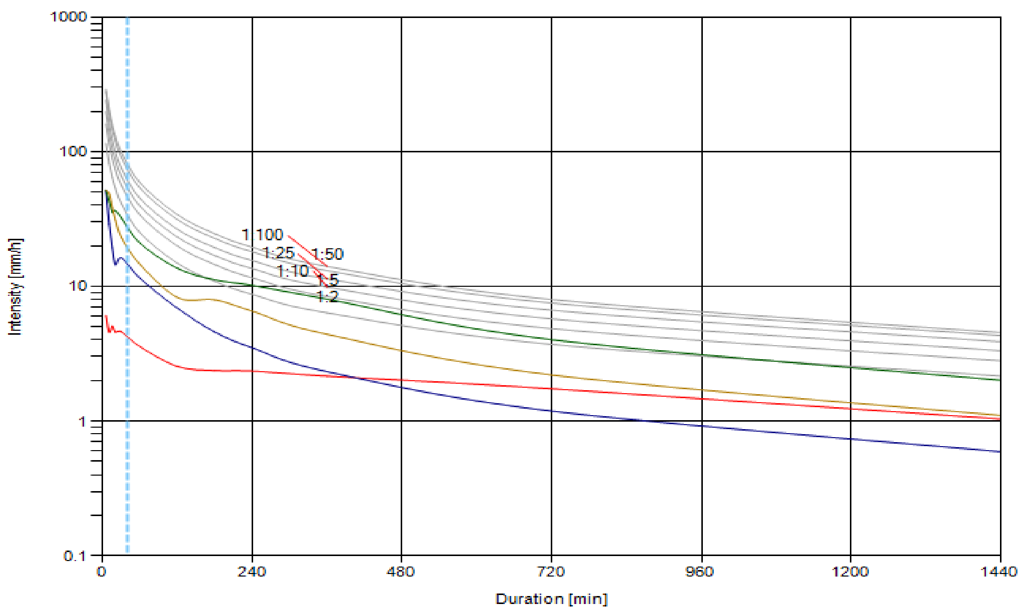
Event	Event Date	Rainfall	
		Volume (mm)	Peak 5-min Intensity (mm/hr)
1	July 23, 2010	48	45.7
2	June 24, 2010	26	45.7
3	November 30, 2010	23	6.1
4	September 21, 2010	14	30.5

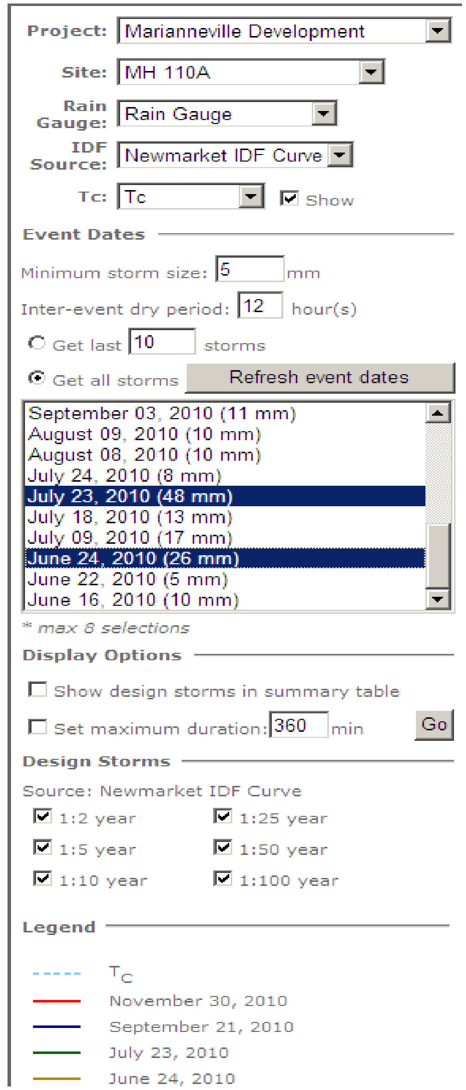
Table 6.2 summarizes the flows and wet-weather volumes after separating the dry-weather portion from the total measured hydrograph. The wet and dry weather flows were separated to quantify the extraneous flows entering the sewer during each storm event. The wet weather flow hydrograph separation for these flow gauges are illustrated in **Figure 6-2 to Figure 6-5.**

Table 6.2 – Wet Weather Flows and Volumes

Event	Date	Rainfall Volume (mm)	(L/s) (MH 110A)	Wet Weather Flow Volume (m3) (MH 110A)
1	July 23, 2010	48	13.3	248
2	June 24, 2010	26	13.9	205
3	November 30, 2010	23	9.5	151
4	September 21, 2010	14	4.4	31

IDF Return Period Analysis
MH 110A





Storm Return Period Over Time Of Concentration

Storm Date	Time of Concentration T_c (min)	Return Period over T_c
Nov 30, 2010	40	< 2 yr
Sep 21, 2010	40	< 2 yr
Jul 23, 2010	40	< 2 yr
Jun 24, 2010	40	< 2 yr

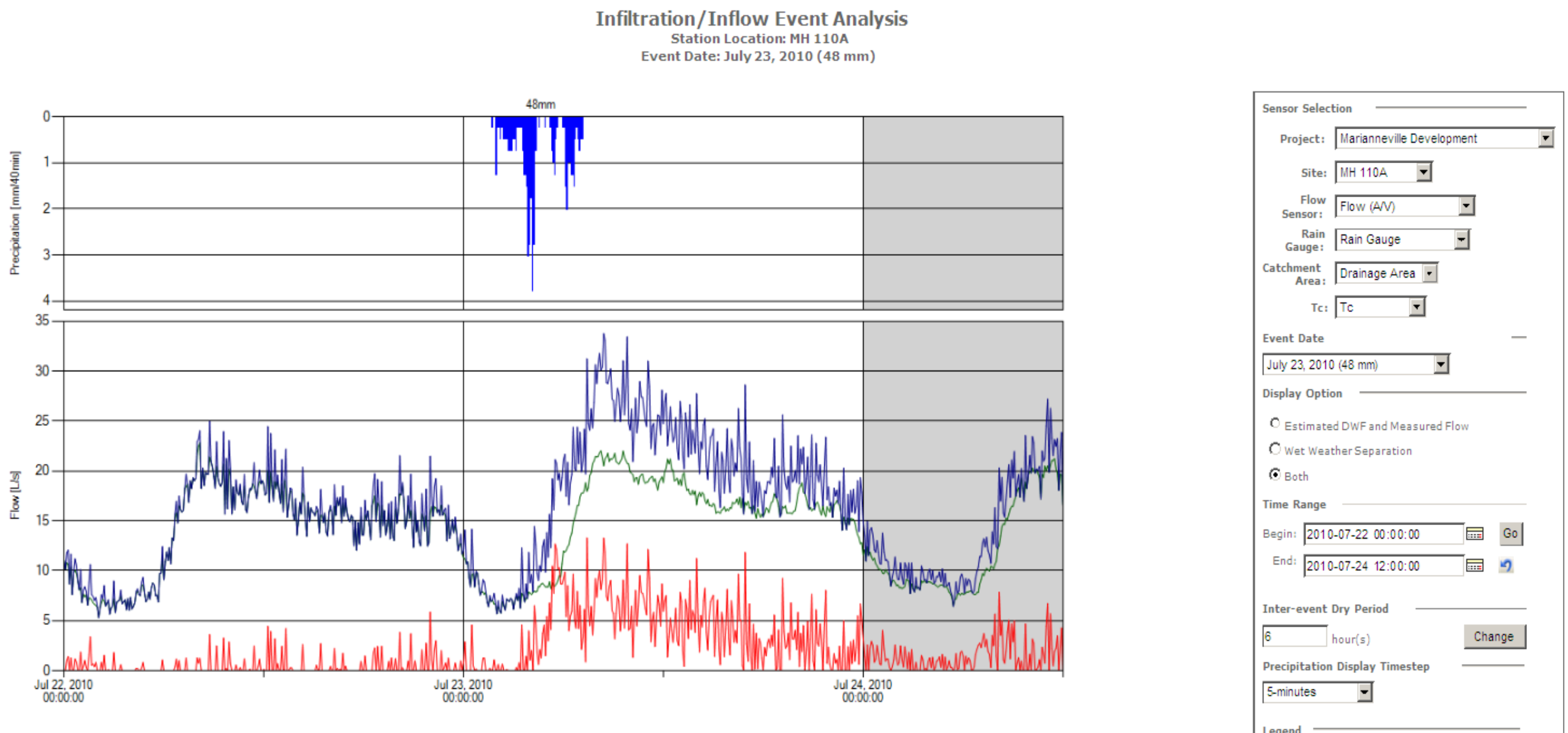
Site Information

Rain Gauge Location:	RG Glenway Golf Course
Latitude:	44.050011
Longitute:	-79.496216

Storm Date	Total Volume (mm)	Peak Intensity over Minute Timestep (mm/h)											
		5	10	15	20	30	60	120	180	240	360	720	1440
Nov 30, 2010	26	6.1	4.6	5.1	4.6	4.6	3.6	2.5	2.4	2.3	2.2	1.7	1
Sep 21, 2010	14	51.8	29	19.3	14.5	16.3	11.4	6.9	4.6	3.5	2.3	1.2	0.6
Jul 23, 2010	48	51.8	42.7	35.6	36.6	32	20.8	13.6	11.2	10.2	8	4	2
Jun 24, 2010	26	51.8	48.8	38.6	31.2	22.9	14.5	8.3	8	6.5	4.4	2.2	1.1

Export Peak Intensity Table

Figure 6-1 IDF Analysis for Largest Events Measured in Marianneville MH 110A during Monitoring Period



Statistics

Catchment Area :	105 ha	Total I/I Volume:	401 m^3
Time of Concentration T_c:	40 min	Total Inflow Volume:	28 m^3
Total Precipitation:	48 mm (50400 m^3)	Total Est. Infiltration:	374 m^3
Peak Precip. Intensity:	249.9 mm/h	Volumetric Runoff Coefficient:	0.00797
Peak Precip. Intensity Over T_c:	27.8 mm/h	Time of Peak I/I Flow (T_D):	Jul 23, 2010 07:25
Time of Peak WWF:	Jul 23, 2010 08:25	Est. DWF at T_D:	18 L/s
Measured Peak Flow:	33.8 L/s	Peak I/I Flow:	13.3 L/s
Total Volume:	3483 m^3	Peak I/I Rate:	0.126 L/s/ha
Total Dry Weather Volume:	3039 m^3	Peak I/I Coefficient:	0.00163

Figure 6-2 I/I Analysis of July 23 2010 Event

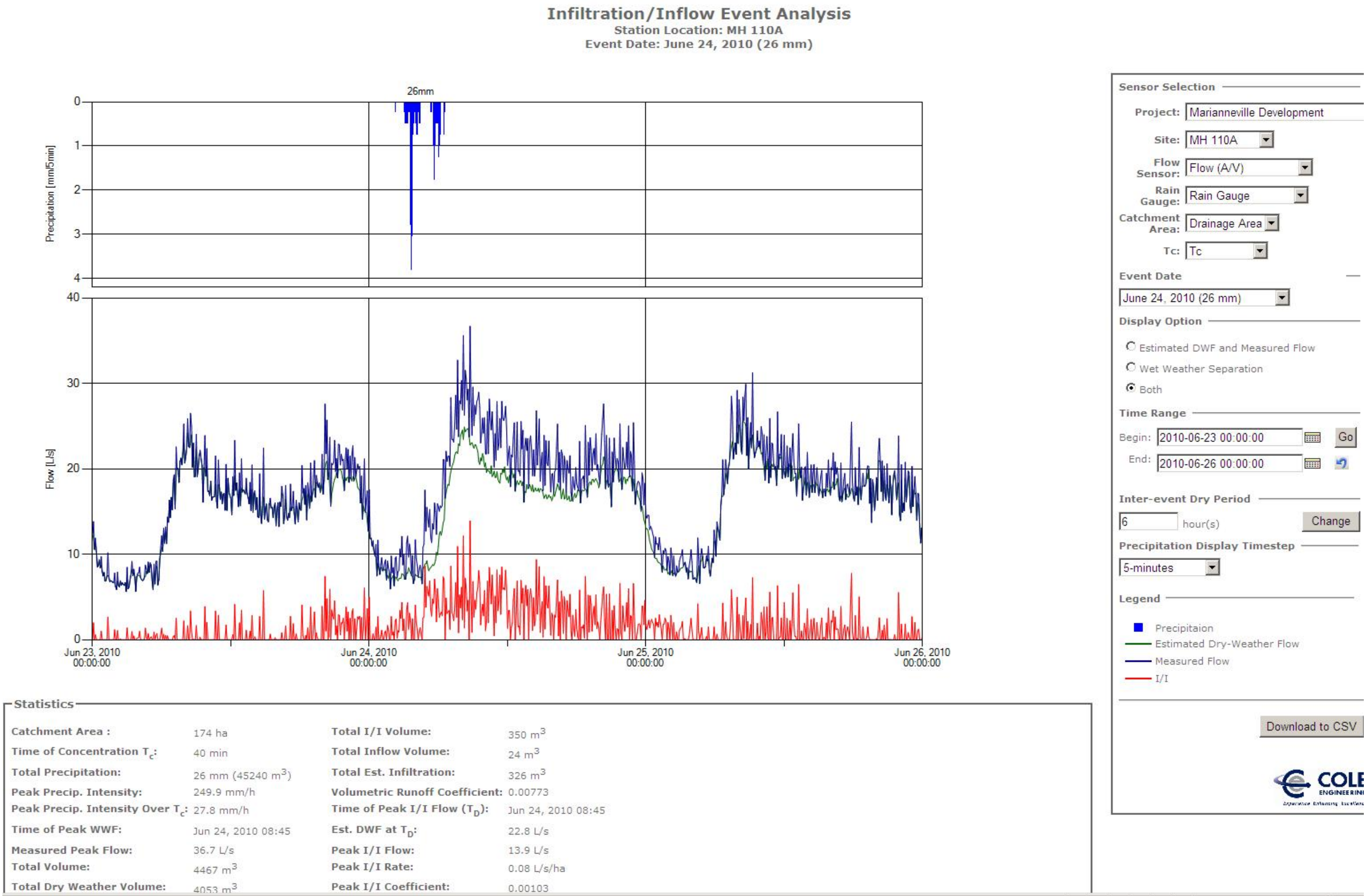


Figure 6-3 I/I Analysis of June 24 2010 Event

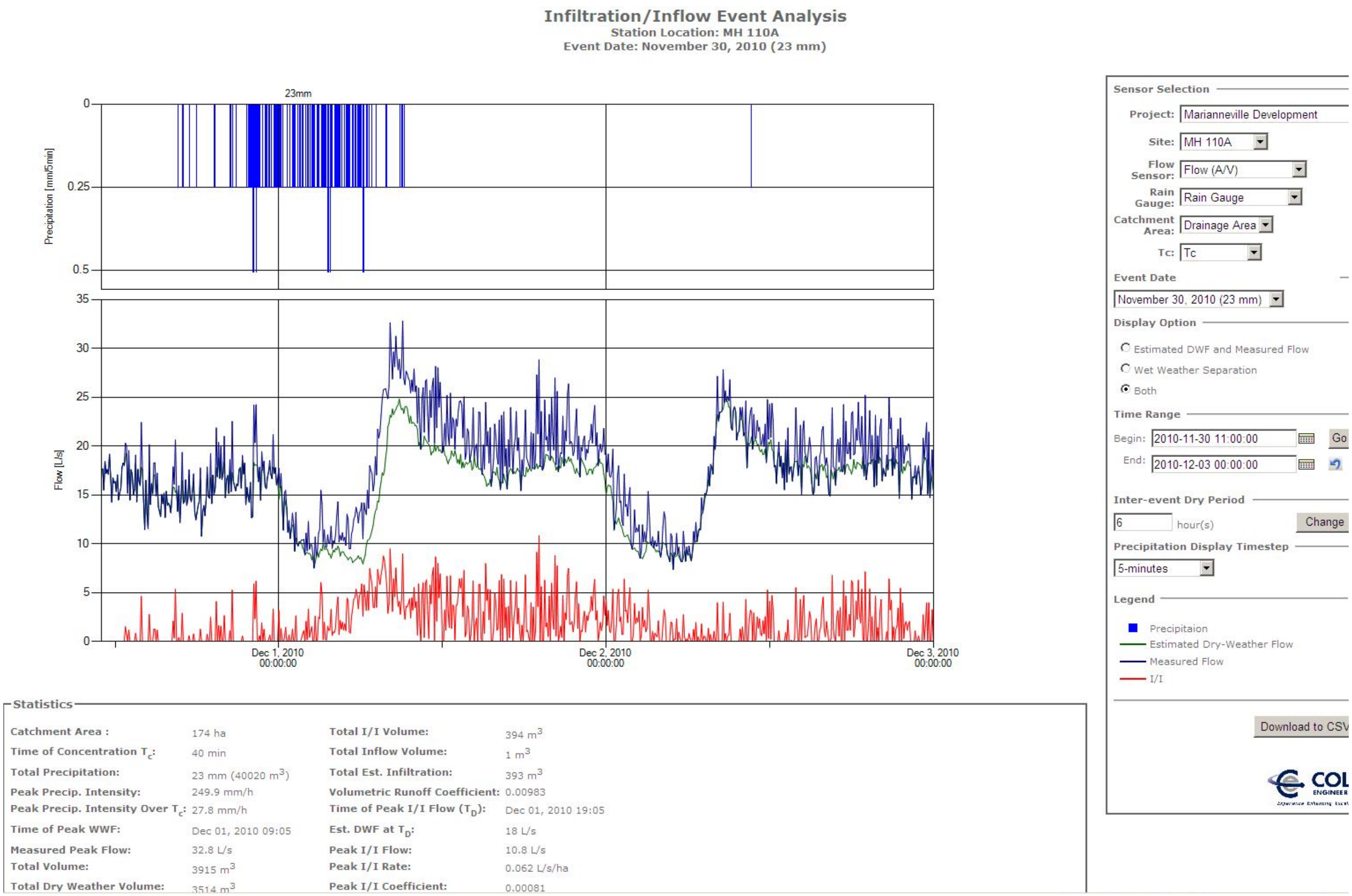


Figure 6-4 I/I Analysis of November 30 2010 Event

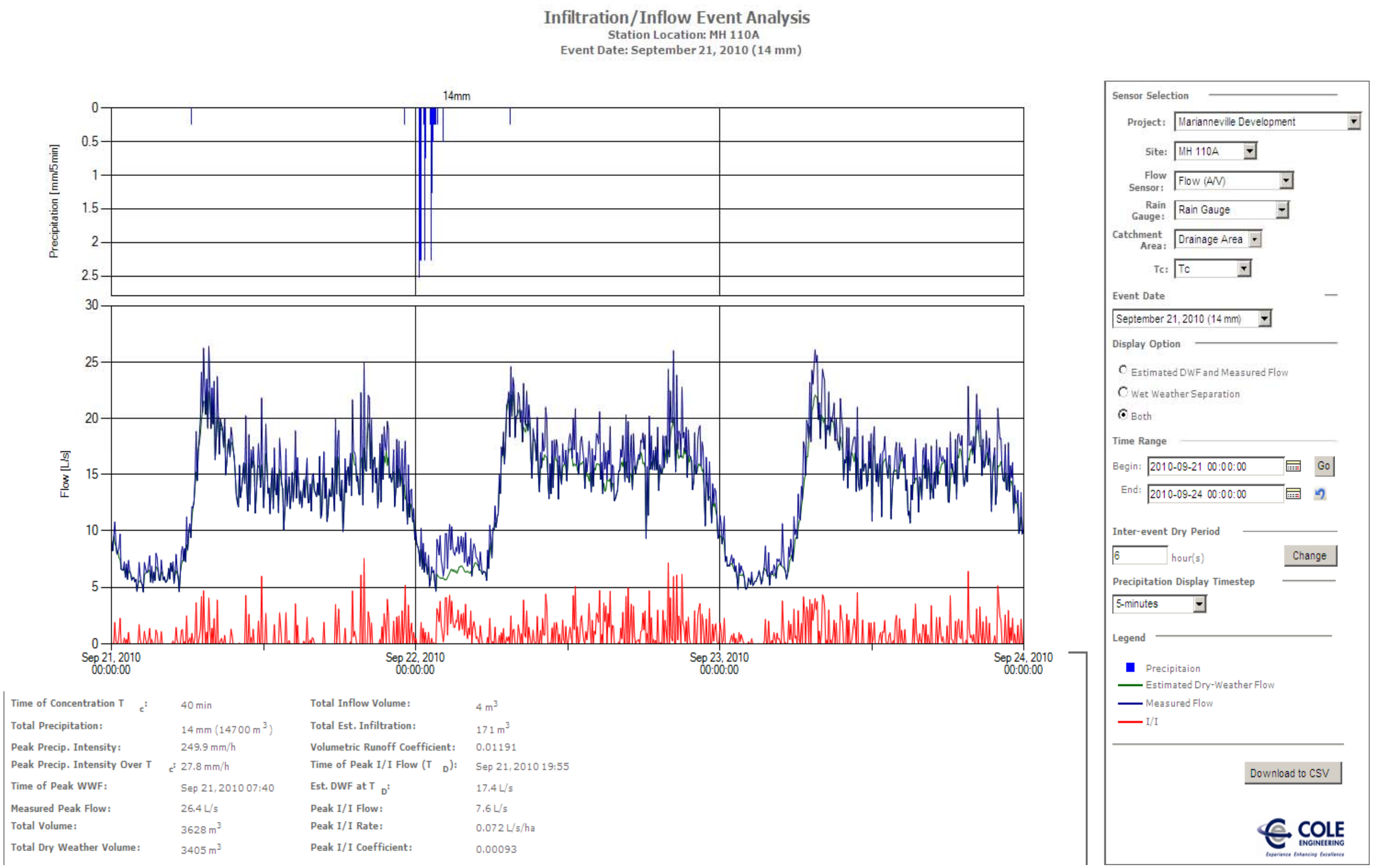


Figure 6-5 I/I Analysis of September 21 2010 Event

6.2.3. Existing Conditions Model Calibration

InfoWorks provides different methods to predict extraneous inflow or Rainfall-Derived-Infiltration and Inflow (RDII) into the system. The method selected for this study was the Ground Infiltration Model (GIM) since it is the standard currently used by the Region.

The GIM methodology uses numerous parameters to calibrate the model. **Table 6.3** summarizes the runoff surface parameters.

Table 6.4 summarizes the groundwater infiltration model parameters. **Table 6.5** describes the various GIM parameters.

Table 6.3 – Runoff Surface Parameters

Runoff Surface ID	Description	Runoff Routing Type	Runoff Routing Value	Initial Loss Type	Initial Loss Value (m)	Routing Model	Fixed Runoff Coefficient
1	Area 110A	Abs	0.13	Abs	0.005	SWMM	0.043

Table 6.4 – Groundwater Infiltration Model (GIM) Parameters

Ground Infiltration ID	Soil Depth (m)	Percolation Coefficient (day)	Baseflow Coefficient (day)	Infiltration Coefficient (day)	Percolation Threshold (%)	Percolation Percentage Infiltrating (%)	Porosity of Soil (%)	Porosity of Ground (%)	Initial Soil Depth (%)
MH110A	3	0.05	0.01	0.2	40	2	40	40	35

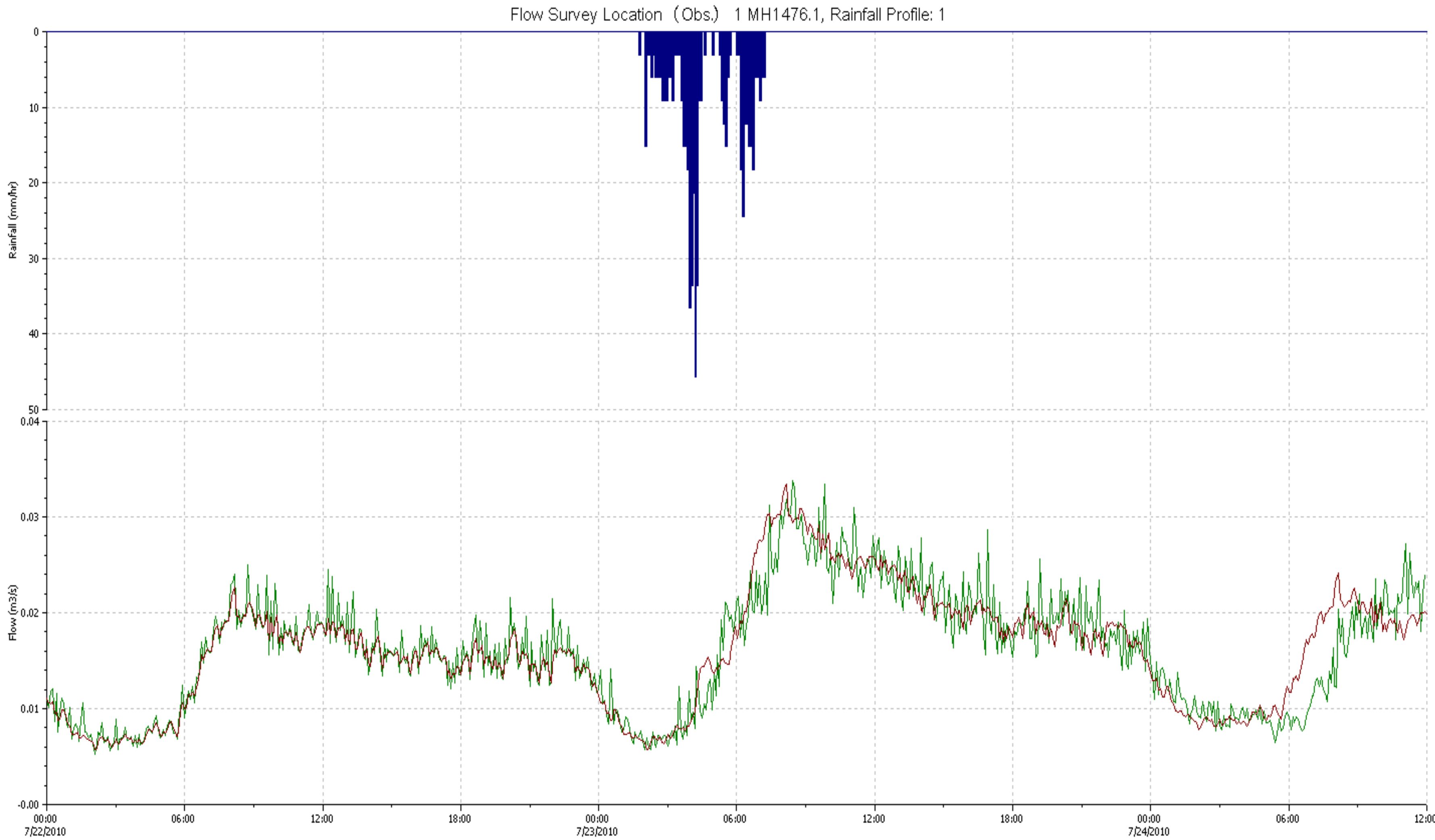
Table 6.5 – Description of GIM parameters

Parameter	Definition
Soil depth	Cover depth of pipe in meters
Percolation coefficient	Speed of contribution from soil storage reservoir in days
Baseflow coefficient	Speed of contribution to “Lost to groundwater” in days
Infiltration coefficient	Contribution from groundwater store to the sewer in days
Percolation threshold	% of water in soil depth at which there is a contribution from soil storage
Percolation percent infiltrating	% of flow goes into the sewer
Porosity of Soil	% of void spaces in unit volume of soil
Porosity of ground	% of void spaces
Initial soil depth	% of initial soil saturation

The results of the calibration are shown **Figure 6-6 to Figure 6-9** which highlight the measured and modelled hydrographs and measured hyetographs during the four (4) selected storms. **Table 6.6** summarizes the measured and modelled peak flows and volumes at the monitoring location at MH 110A.

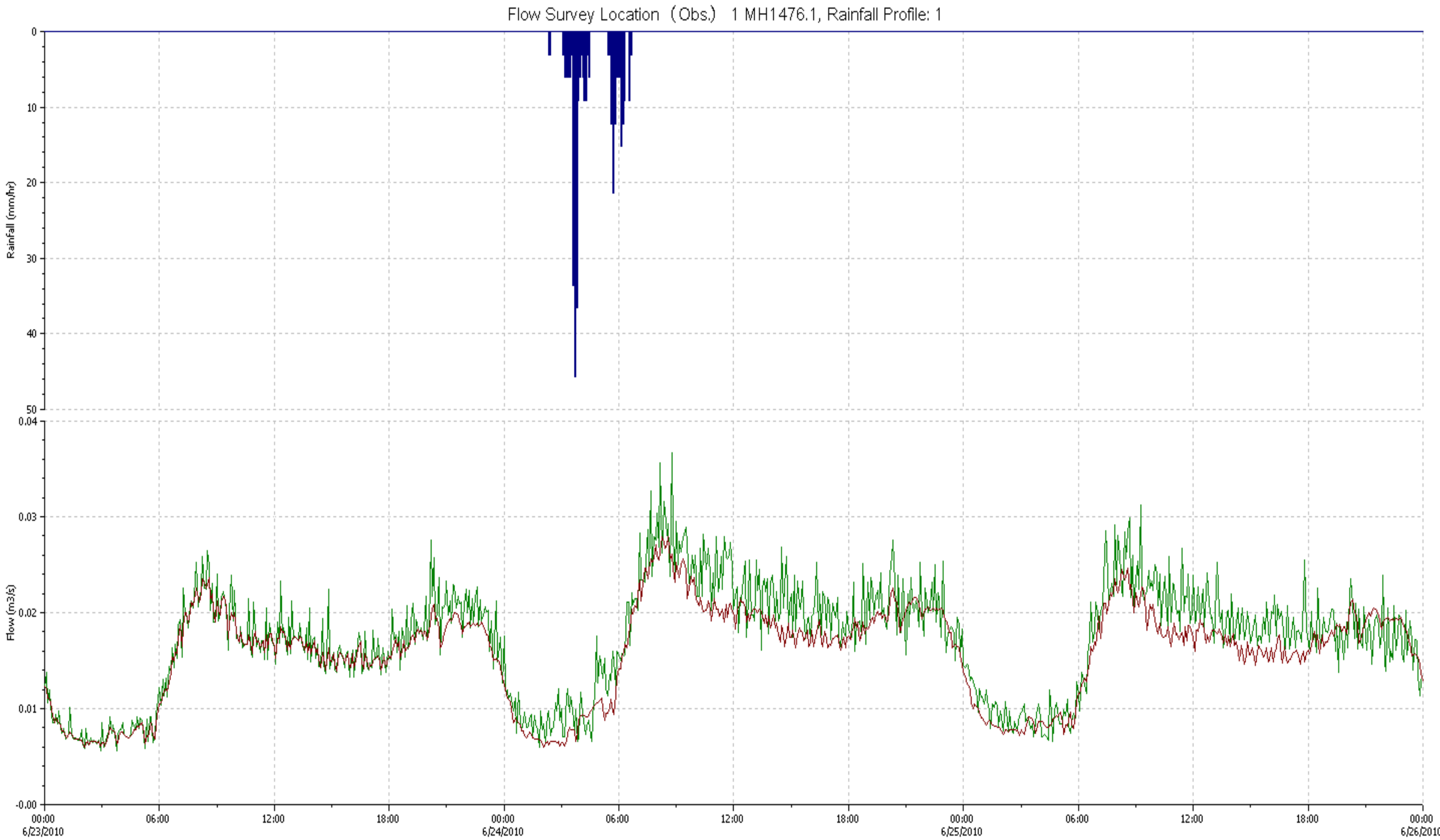
Table 6.6 – Measured versus Modelled Peak Flows and Volumes – Location MH 110A

Event	Event Date	Rainfall		Q-peak			Volume		
		Volume (mm)	Intensity (mm/hr)	Measured (m3/s)	Modeled (m3/s)	Difference (%)	Measured (m3)	Modeled (m3)	Difference (%)
1	July 23, 2010	48	45.7	0.034	0.031	0.0	3464	3457	-0.2
2	June 24, 2010	26	45.7	0.037	0.028	-9.7	4452	410.	-7.8
3	November 30, 2010	23	6.1	0.033	0.027	-6.9	3895	3826	-1.8
4	September 21, 2010	14	30.5	0.025	0.023	0.0	3615	3433	-4.9



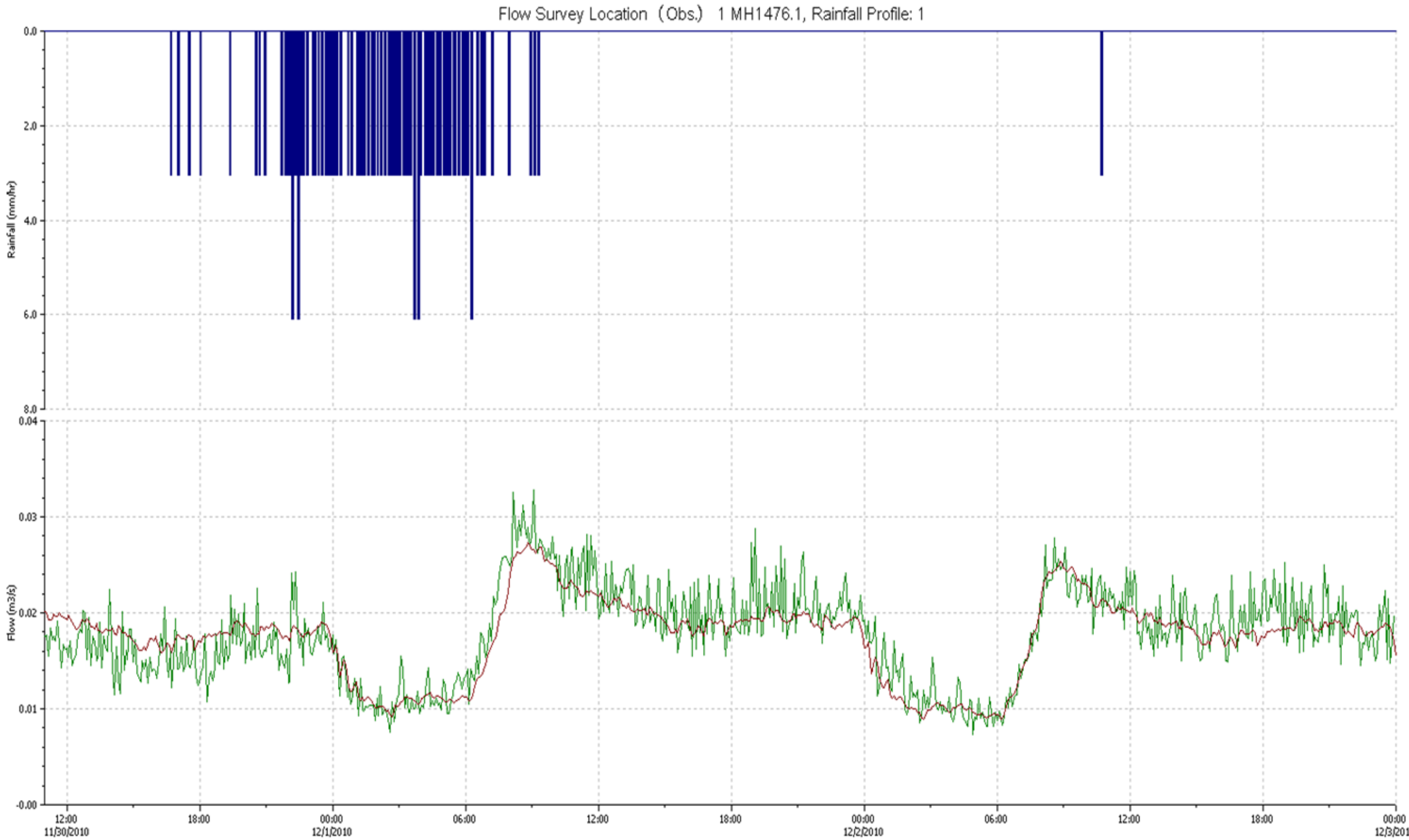
	Rainfall			Flow (m3/s)		
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min	Max	Volume (m3)
Rain	48.260	45.720	0.803			
Obs.				0.005	0.034	3472.849
...ment Group>Run Group>WestSystemv4-July23>Rain-July23				0.006	0.033	3482.050

Figure 6-6 Measured and Modeled Hydrograph Comparison, July 23, 2010 Event



	Rainfall			Flow (m3/s)		
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min	Max	Volume (m3)
Rain	26.416	45.720	0.366			
Obs.				0.006	0.037	4459.657
...ent Group>Run Group>WestSystemv4-June24>June24_Rain				0.006	0.028	4106.725

Figure 6-7 Measured and Modeled Hydrograph Comparison, June 24 2010 Event



	Rainfall			Flow (m3/s)		
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min	Max	Volume (m3)
Rain	23.114	6.096	0.378			
Obs.				0.007	0.033	3904.813
...ent Group>Run Group>WestSystemv4-Nov30>Nov30_Rain				0.009	0.027	3839.845

Figure 6-8 Measured and Modeled Hydrograph Comparison, November 30 2010 Event

	Rainfall			Flow (m3/s)		
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min	Max	Volume (m3)
Rain	14.478	30.480	0.201			
Obs.				0.005	0.026	3621.103
...ent Group>Run Group>WestSystemv4-Sept21>Sep21_Rain				0.005	0.023	3437.039

Figure 6-9 Measured and Modeled Hydrograph Comparison, September 21 2010 Event

6.2.4. DWF and I/I Rates Comparison

Once the model calibration was complete, the dry and wet weather flows were compared to existing standards. Population information was provided by the Town's H2OMap model and drainage areas were determined based on the drainage area plans and aerial photography. The existing population was estimated at 3,216 persons within the estimated drainage area shown in **Figure 6-10** along with the monitoring location (MH 110A). The infiltration rate identified in the monitoring program will be used for all future scenario modelling of existing background conditions.

The estimated wet weather flow was compared with previous reports and the existing standards and is summarized in **Table 6.7**. The current Town of Newmarket Design Standards provide for 0.30 L/s/ha of an extraneous flow rate that is not linked to a specific storm event. **Figure 6-11** shows a statistical analysis of the monitored I/I as compared to the Town's design events forecast for the 1:2 to 1:100 year design storms.

Table 6.7 – Comparison I/I with Previous Reports

Newmarket Design Storms	Flow Station MH 110A RDA Forecast I/I Rate (L/s/ha)	Calibrated Model Run Chicago - 24 hr Storms I/I Rate at MH 110A (L/s/ha)	Town of Newmarket - Assessment of Sanitary Sewer Design Flow Criteria (Giffels, 1995) I/I Rate (L/s/ha)	Town of Newmarket - Master Sanitary Sewer Hydraulic Study (R.V.Anderson, 2008) I/I Rate (L/s/ha)	YDSS Master Plan Update (2002) I/I Rate (L/s/ha) ***
2 Year Storm	0.15	0.08		0.22	
5 Year Storm	0.17	0.12		0.30	0.56
10 Year Storm	0.19	0.16		0.38	
25 Year Storm	0.20	0.25		0.46	0.72
50 Year Storm	0.23	0.35		0.51	
100 Year Storm	0.25	0.42	1.55	0.57	

Note: *** YDSS Master Plan Update (2002) recommended an allowance of 0.50 L/s/ha for peak I & I for all of Newmarket.

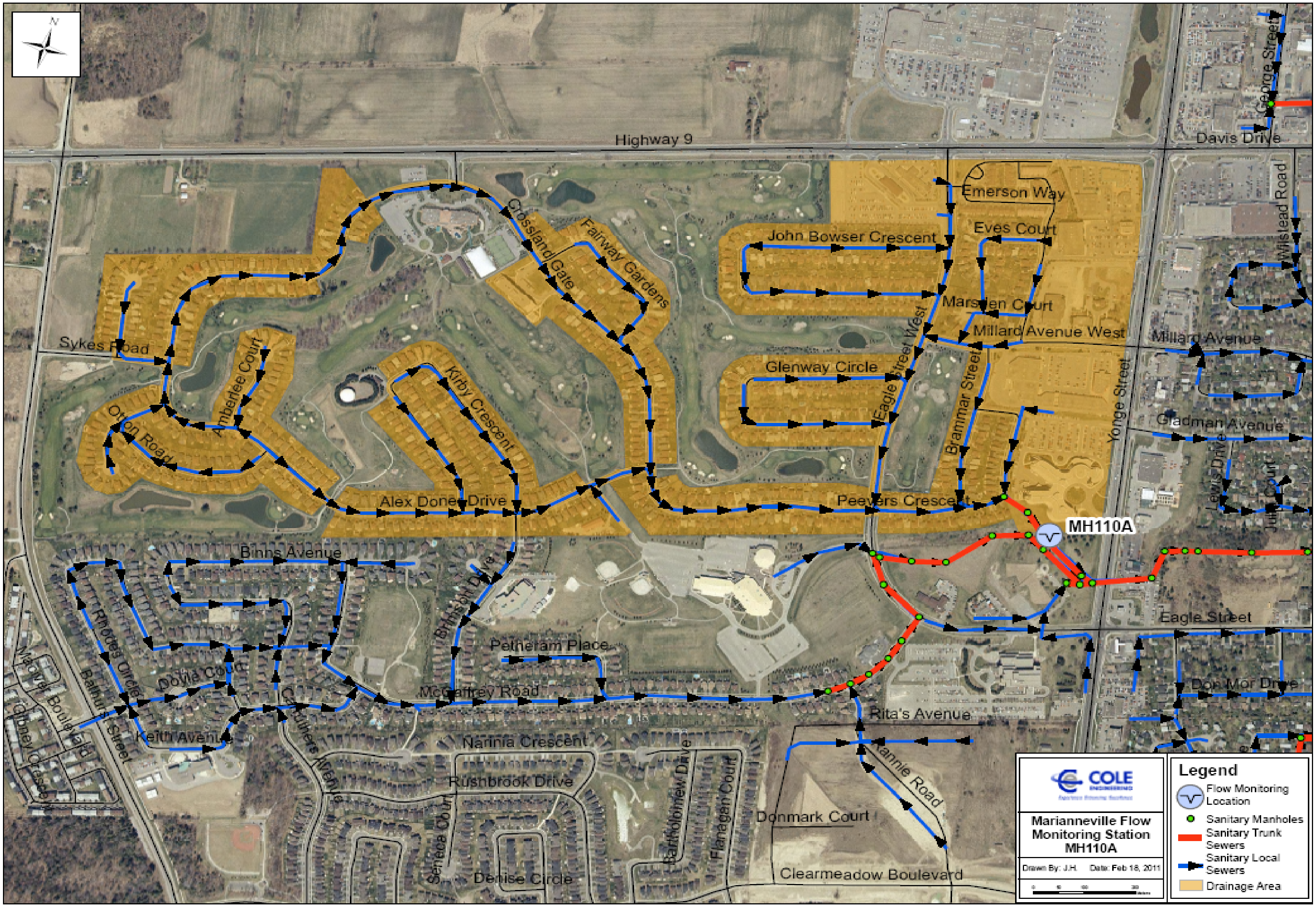


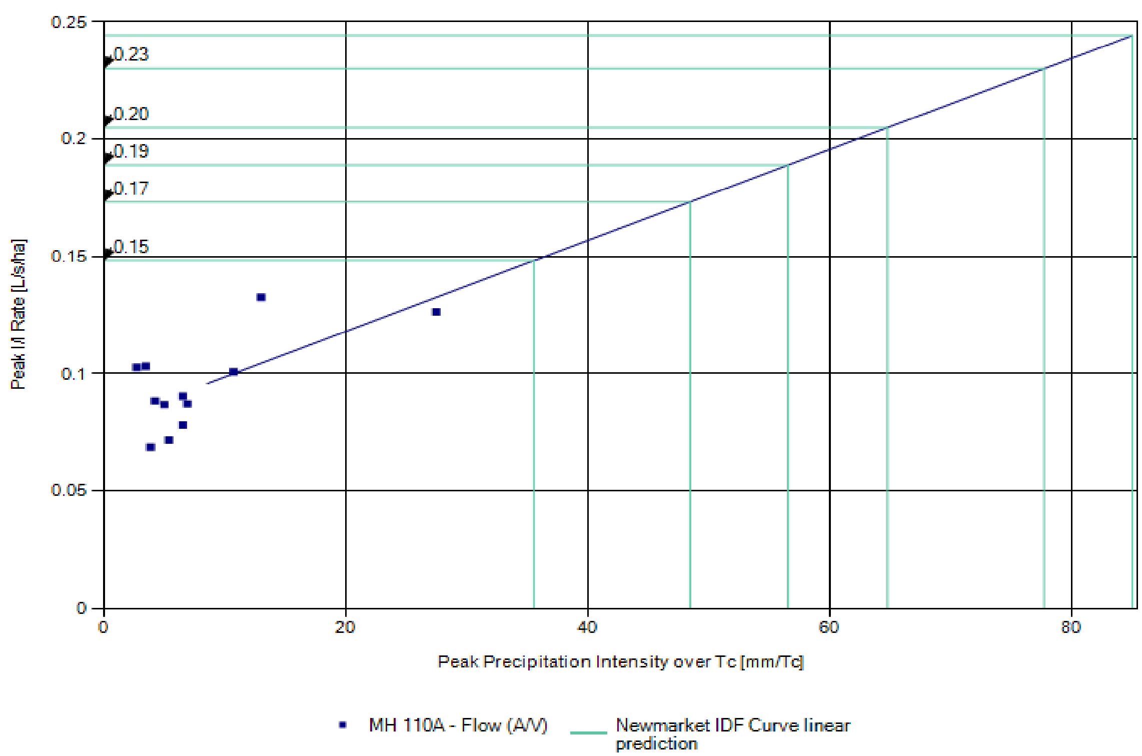
Figure 6-10 Flow Monitoring Station, Location and Drainage Area

Sanitary I/I Comparison Report

Cole Engineering Automated Analysis System

Project: Marianneville Development
Site: MH 110A
Start: 2010-Jun-01 12:30:00
End: 2010-Dec-07 16:20:00

Measured Flow and Precipitation



Fitted Equation

Flow Sensor	Equation
MH 110A Flow (A/V)	y = 0.002x + 0.079

Design Storms
Newmarket IDF Curve

- ☑ 1:2 year
- ☑ 1:5 year
- ☑ 1:10 year
- ☑ 1:25 year
- ☑ 1:50 year
- ☑ 1:100 year

Update

Figure 6-11 RDA Forecast I/I Rate for 2 to 100 Year Design Storm in Newmarket

6.2.5. Existing Sanitary Flow Monitoring and Model Results

Based on the data collected through the sanitary flow monitoring program, the existing peak sanitary flow rate is calibrated for Chicago 24 hr Storms (1:2 to 1:100 year events) and summarized in **Table 6.8.**

Table 6.8 – Existing Peak Sanitary Flows Generated During 2 to 100 Year Design Storms

Newmarket Design Storms	Existing Peak Sanitary Flow Condition Chicago 24 hr Storms (L/s)
2 Year Storm	28.9
5 Year Storm	33.2
10 Year Storm	36.6
25 Year Storm	46.0
50 Year Storm	55.5
100 Year Storm	61.9

For future sanitary flow analysis purposes, the most conservative storm event (100-year) and corresponding peak sanitary flow of 61.9 L/s shall be selected for comparison purposes under proposed development conditions.

6.3. Proposed Sanitary Sewers

The proposed development within the former Glenway Country Club will utilize connections to the existing surrounding sanitary sewer network. New sanitary sewers will be constructed along the proposed municipal Streets A, B, C, and D, or the private roads as required to service all new lots and medium or high density development blocks. All sanitary flows from the proposed development will outlet to the existing 450mm diameter *Western Sub-Trunk Sanitary Sewer* located at southeast corner of Peevers Crescent.

The additional sanitary sewer flows generated within the proposed development will be distributed to the existing surrounding sewers as follows:

a) Lots 1-6 fronting onto Alex Doner Drive, west of Hydro Easement shall connect to the existing 250mm dia. sewer along Alex Doner Dr.;
b) Blocks 166, 167, 172 and north leg of Street A will outlet to Crossland Gate system upstream of Fairway Garden;
c) South leg of Street A will direct flows to MH 71A ;
d) Street B, Street C, Blocks 168, 171 and a condo laneway south and east of Street B cul-de-sac will outlet to MH 69A; and,
e) Street D (located east of Eagle Street) and two small portions of condo lands directly abutting Eagle Street will outlet to Eagle Street sewers at or upstream of MH 48A.

Refer to **Drawing SAN-1** (within map pocket at the end of Report) for a depiction of the proposed sanitary sewer alignments and connection locations to the existing system.

6.4. Proposed Sanitary Flow Analysis

The proposed peak sanitary flow calculations completed for this report are based on a unit flow rate of 360 L/cap/day and the "population per unit" counts defined in the current design standards as shown below:

- Single family homes (and Condo Bungalows): 3.38 ppu
- Townhomes (Medium Density Blocks): 2.88 ppu
- High Density / Apartment units: 1.95 ppu

The additional population from the proposed development calculated based on Town of Newmarket standards is 1,940 people with an additional sanitary drainage area of 30.43 ha of residential and commercial area. Based on these parameters, a total peak sanitary flow rate of 41.60 L/s is calculated by considering each proposed development parcel individually.

To consider the total additional sanitary flow generated from the proposed development population of 1940 combined with the existing sanitary flows from the current population of 3216, the new total population of 5156 exhibits a peaking factor of 3.23 at the existing sanitary outlet for the entire Glenway Community at Peevers Crescent (existing MH110).

Table 6.9 provides a breakdown of the proposed development unit and area statistics and their corresponding peak sanitary flow generation rates.

Table 6.9 – Proposed Sanitary Flow Generation

Land Use	Units	Area[1] (ha)	PPU	Pop.	Av. Daily Sanitary Flow (L/s)[2]	Harmon Peaking Factor	Peak San. Flow (L/s)	I & I (L/s)[4]	Total Peak Sanitary Flow (L/s)
Residential (Lots 1 – 165)	165	11.81	3.38	558	2.32	4.0	9.30	3.54	12.84
Residential, Medium Density (Blocks 166-168)	219	6.00	2.88	631	2.63	4.0	10.51	1.80	12.31
Residential, Condos (Blocks 169 – 170)	54	5.93	3.38	183	0.76	4.0	3.04	1.78	4.82
Residential, High Density (Block 171)	292	2.34	1.95	569	2.37	3.94	9.35	0.70	10.05
Commercial (Block 172)	--	0.65	--	--	0.3[3]	--	0.30	0.20	0.50
Parkland (Block 173)	--	--	--	--	--	--	--	--	--
Proposed Roadways (Public)	--	3.71	--	--	--	--	--	1.11	1.11
Total	**730**	**30.44**		**1941**				**9.13**	**41.63**

[1] Area does not include ponds
[2] Based on a residential flow rate of 360 L/cap/day
[3] Based on a commercial flow rate of 0.46 L/s/ha
[4] Inflow and Infiltration based on a rate of 0.3 L/s/ha

While it is appropriate to utilize each development parcel's individual peak sanitary flow rate based on the higher peaking factor for examination of local sewer capacity at proposed connection points, the combined population peaking factor should be used to examine the total expected flow exiting from the Glenway Community. Based on the combined population peaking factor of 3.23, the proposed development generates a peak sanitary flow of 35.5 L/s at existing MH 110.

As discussed under **Section 6.1**, the original theoretical peak sanitary flow from the Glenway Community was calculated at 177 L/s just downstream of existing manhole 110A, based on the Sewer Design Sheets dated January 1995 and prepared by G.M. Sernas. To analyze the anticipated future sanitary flow conditions, the proposed peak sanitary flow rate of 35.5 L/s was added to each of the following two (2) scenarios for comparison to the original theoretical peak sanitary flow design:

<u>Proposed Scenario 1:</u>

- Existing Peak Sanitary Flow Rate at ex. MH110A under 100-Year Storm based on the Monitoring Program and Chicago 24 hr Storms = 61.9 L/s.
- Proposed Development Peak Theoretical Sanitary Flow Rate at ex. MH110A = 35.5 L/s.
- Total Peak Sanitary Flow Rate at ex. MH110A = **97.4 L/s.**

<u>Proposed Scenario 2:</u>

(Refer to **Appendix B** for a Sanitary Sewer Design Sheet modeling the existing Glenway Community utilizing present day municipal sanitary design flow generation standards).

- Existing Peak Sanitary Flow Rate at ex. MH110A based on Theoretical Design Flows utilizing present day municipal standards = 76.6 L/s.
- Proposed Development Peak Theoretical Sanitary Flow Rate at ex. MH110A = 35.5 L/s.
- Total Peak Sanitary Flow Rate at ex. MH110A = **112.1 L/s.**

Under both scenarios, the total peak sanitary flow rate including the proposed development is less than the original theoretical peak sanitary design flow of 177 L/s.

The limiting existing sanitary sewer outleting from the Glenway Community downstream of Peevers Crescent is a 450mm diameter sanitary sub-trunk at 0.34% grade, exhibiting a full flow capacity of 166.2 L/s, therefore the total peak sanitary flow rate including the proposed development under both scenarios can be adequately conveyed through the existing sanitary sub-trunk.

7.0 Stormwater Management

The proposed Glenway re-development will consist of a combination of single family residential lots, medium density townhouses, a high density residential apartment building complex and a commercial block all connected and serviced by an internal network of municipal and private roads and four (4) private stormwater management (SWM) ponds. The proposed change in land use will increase the volume and rate of stormwater runoff from the site. Therefore, a SWM plan is required to reduce peak runoff rates and provide quality treatment of runoff for the proposed re-development.

7.1. Design Criteria

The proposed development within the Town of Newmarket has been designed in consultation with the drainage and SWM requirements of the Town of Newmarket, the Lake Simcoe Region Conservation Authority (LSRCA) and the Ministry of the Environment (MOE) standards.

The following guidelines were referenced for SWM design criteria:

- Ministry of Environment (MOE) – Stormwater Management Planning and Design Manual (2003);
- LSRCA – Technical Guidelines for Stormwater Management Submissions (November 2010), (Technical Guidelines); and,
- Town of Newmarket – Engineering Design Standards and Criteria (January 2009).

The following criteria were used to size the wet ponds:

- Quality Control – MOE Enhanced (Level 1) Protection;
- Quantity Control – Post-development peak flow control to the existing pond peak outflows for the 2 to 100-year 24-hour SCS design storms, as per Town of Newmarket Standard;
- Erosion Control – 24-hour detention of the 25mm, 4-hour Chicago storm; and,
- Physical pond characteristics and dimensions – MOE guidelines for Pond 6, 8 and 9 and Town standards for Pond 4.

7.2. Existing Hydrologic Conditions

The existing Glenway Community includes an 18-hole golf course surrounded by residential and commercial development. Pre-development drainage areas were delineated based on review of the as-built storm drainage area plans of the existing Glenway Community subdivision completed by The Lathem Group Inc. (1983) and aerial topography information received in October, 2009 from First Base Solutions and a detailed survey conducted by J.D. Barnes in January, 2012. The area proposed for re-development is generally situated east of the existing Hydro One corridor. The existing site is currently divided into four (4) separate drainage areas discharging to four (4) separate ponds located within the eastern half of the 18-hole golf course. There are two (2) drainage outlets from the site, one (1) south along Eagle Street and one (1) north to Davis Drive. The pre-development drainage area plan is illustrated on **Figure 7-1**.

Figure 7-1 Pre-Development Storm Drainage Area Plan

The existing soil conditions were determined to be silty clay till based on the soil investigation done by Soil Engineers Ltd. on December 17, 2011. The local soil is classified under soil group C in the Ministry of Transportation (MTO) Design Chart 1.08. In applying a land use type of pasture and a good hydrologic condition, a soil conservation service (SCS) curve number (CN) of 74 was determined using MTO Design Chart 1.09. The CN* conversion was performed as recommended by the VO2 manual; however there was no change from the initially derived CN value of 74. The CN* conversion calculation and MTO Design Charts 1.08, 1.09 and 1.10 are included in **Appendix C.**

- The imperviousness of the existing land uses was assumed using the Town's design standards. Where it was observed that the existing development has a higher imperiousness than the Town standards, the impervious value used was increased to reflect the actual conditions. The excerpt from the Town of Newmarket design standards providing assumed % imperviousness and runoff coefficients for various land uses is provided in **Appendix J**.

Visual OTTHYMO 2.4 (VO2) was used to model pre-development hydrologic conditions in order to determine the pre-development flows from each of the four (4) ponds that will be affected by the proposed development. A mix of NashHyd and StandHyd objects were used in the model to represent the existing conditions. The input for NashHyds include a runoff coefficient (C) and a time to peak (Tp), the input for StandHyds include a directly connected impervious value (XIMP) and a total impervious value (TIMP). The detailed input parameter calculations for the pre-development hydrologic model are provided in **Appendix D** and summarized below in **Table 7.1.**

Table 7.1 – Pre-Development Input Parameters

Receiving Pond	Catchment	Drainage Area (ha)	CN value	Tp (hr)	XIMP (%)	TIMP (%)
4	4-ex1.1	6.53	74	0.19		
	4-ex1.2	2.34			0.55	0.55
	4-ex1.3	0.97			0.64	0.64
	4-ex2.1	2.95			0.25	0.25
	4-ex2.2	3.87			0.61	0.61
	4-ex2.3	0.91	74	0.17		
	4-ex2.4	6.86			0.61	0.61
	4.1	10.18	74	0.27		
	4.2	6.71			0.71	0.71
	4.3	2.59	74	0.22		
	4.4	0.85			0.28	0.28
	4.5	1.61	74	0.13		
6	6-ex3.1	3.62			0.28	0.28
	6-ex3.2	1.45			0.64	0.64
	6-ex3.3	1.33	74	0.13		
	6.1	8.03	74	0.22		
	6.2	17.98			0.61	0.61
	6.3	10.64	74	0.24		
	6.4	2.11	74	0.26		
	6.01 (major system only)	1.21			0.55	0.55
	68.1 (major system only)	1.5			0.64	0.64

Table 7.1 – Pre-Development Input Parameters (cont'd)

Receiving Pond	Catchment	Drainage Area (ha)	CN value	Tp (hr)	XIMP (%)	TIMP (%)
8	8.1	3.28	74	0.10		
	8.2	10.16			0.66	0.66
	8.3	2.21	74	0.23		
	8.01 (minor system only)	2.5			0.55	0.55
	68.1 (minor system only)	1.5			0.64	0.64
	98.1 (minor system only)	1.27			0.68	0.68
9	9.1	2.71			0.25	0.25
	9.2	5.86			0.56	0.56
	9.3	1.34	74	0.22		
	9.4	2.71			0.25	0.25
	98.1 (major system only)	1.27			0.68	0.68
	9.01 (major system only)	0.10	74	0.05		
	9.02 (major system only)	0.47			0.70	0.70
	9.03 (major system only)	2.51	74	0.27		

The storm distributions used to model pre-development conditions include the 12-hour SCS Type II distribution, as per LSRCA requirements, the 24-hour SCS distribution, as per Town of Newmarket requirements, and the 4-hour Chicago distribution, as per the Town and LSRCA requirements. The intensity-duration-frequency (IDF) data used for the 4-hour Chicago storms was taken from the Town of Newmarket design standards. The 4-hour Chicago IDF curve parameters for all storm events from the 2-year to the 100-year storm are summarized in **Table 7.2**.

Table 7.2 – Town of Newmarket IDF Curve Parameters

Storm Event	A	B	C
2-year	648	4	0.784
5-year	930	4	0.798
10-year	1021	3	0.787
25-year	1100	2	0.776
50-year	1488	3	0.803
100-year	1770	4	0.820

The pre-development peak flows for the 12-hour SCS, 24-hour SCS and 4-hour Chicago storm distributions are summarized below in **Table 7.3, Table 7.4 and Table 7.5** respectively, and the detailed pre-development model output is provided in **Appendix D**.

Table 7.3 – Pre-development Peak Flows – 12-hour SCS Type II Distribution

	2 year		5 year		10 year		25 year		50 year		100 year	
Catchments	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)
Pond 4	3105	0.306	4529	0.447	5348	0.528	6404	0.633	7213	0.714	8045	0.796
Pond 6	3177	0.729	4533	1.040	5507	1.262	6533	1.881	7321	2.331	8150	2.706
Pond 8	1817	0.650	2559	0.788	3103	0.861	3822	0.958	4365	1.020	4845	1.074
Pond 9	3034	0.476	4478	0.553	5510	0.602	6863	0.666	7924	0.698	9026	0.724

Table 7.4 – Pre-development Peak Flows – 24-hour SCS Distribution

	2 year		5 year		10 year		25 year		50 year		100 year	
Catchments	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)
Pond 4	3759	0.371	4592	0.453	6428	0.636	7666	0.759	8878	0.879	9240	0.915
Pond 6	3738	0.858	4584	1.051	6432	1.815	7684	2.506	8749	2.992	9291	3.168
Pond 8	2047	0.700	2476	0.779	3637	0.937	4449	1.031	5107	1.103	5367	1.131
Pond 9	3497	0.505	4343	0.546	6543	0.651	8091	0.702	9658	0.739	10108	0.749

Table 7.5 – Pre-development Peak Flows – 4-hour Chicago Distribution

	2 year		5 year		10 year		25 year		50 year		100 year	
Catchments	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)	V (m^3)	Peak Flow (m^3/s)
Pond 4	2758	0.272	4283	0.422	5154	0.509	6055	0.598	7212	0.713	7889	0.781
Pond 6	2871	0.658	4311	0.988	5317	1.218	6226	1.678	7483	2.413	8284	2.779
Pond 8	1724	0.640	2601	0.798	3221	0.882	3873	0.968	4779	1.065	5321	1.126
Pond 9	2869	0.465	4502	0.554	5668	0.609	6867	0.666	8514	0.712	9502	0.736

As observed in **Tables 7-3**, **7-4** and **7-5**, the results of the pre-development hydrologic analysis indicate that the 24-hour SCS storm distribution provided the largest peak flows and requires the greatest amount of storage volume. Therefore, the pre-development flow targets are to be based on the 24-hour SCS storm distribution, which matches the Town standard design storm to be used for SWM pond design.

7.3. Adjacent Development Constraints

The proposed development is bound by existing residential lots, golf course land, Davis Drive and a commercial site (Go Station). The majority of the development is occurring within the eastern half of the Glenway Country Club golf course lands. A small portion of the golf course on the east side of Eagle St. is also proposed for re-development.

There are four (4) existing ponds that accept drainage from land that will be affected by the proposed development as shown on **Figure 7-1** and described in **Section 7.2** of this report. Three (3) of the ponds outlet to the existing Glenway Estates and Country Club storm sewer system, flowing south via Eagle St. One (1) of the ponds outlets off-site to the roadside ditch along Davis Drive.

In order to mitigate negative impacts to the existing storm infrastructure, the peak discharge rate from each pond under the proposed conditions will be controlled to match the peak discharge rate from each of the ponds under the existing condition. This assumes that the existing storm infrastructure is adequate to accommodate the existing development conditions. It is proposed that the existing storm sewer remain unchanged. The existing conditions Storage-Discharge rating for each pond has been taken from Glenway Estates Stormwater Management Study (The Lathem Group Inc., 1983).

The design standards for stormwater management ponds have changed since the existing ponds were designed and built. The original design was based on a 1-hour AES design storm. The current Town of Newmarket standards require post- to pre-peak flow control and pond design for the 2 to 100-year event 24-hour SCS design storms. The existing conditions were analyzed using the hydrologic modeling software, Visual Otthymo 2.4 (VO2), and the 24-hour SCS Town design storms to determine the target flows for each of the pond outlets. The analysis completed for each pond is described in the following **Sections 7.3.1** to **7.3.5.**

7.3.1. Pond 4

Pond 4 currently receives flow from both Pond 1 and Pond 2, which are located on the west half of the golf course, via the Glenway Estates and Country Club storm sewer system as well as drainage from the surrounding golf course and residential lots. The existing Pond 4 drainage area is the same for both the minor and major system and includes drainage areas described in **Table 7.1** and shown on **Figure 7-1.** Pond 4 is divided into two (2) cells (4a and 4b) that are hydraulically connected by a 1200 mm diameter culvert between the two(2) cells whereby cell 4b drains into cell 4a. Pond cell 4a has three (3) inlets, one (1) from pond cell 4b and two (2) from the storm sewer system, and outlets offsite to the ditch along Davis Drive via a 900 mm diameter pipe.

The existing Storage-Discharge rating curve for Pond 4 is presented in **Table 7.6** below.

Table 7.6 – Pond 4 Storage-Discharge Rating

Discharge cfs (m^3/s)	Storage ac.ft (ha-m)
0	0
15.5 (0.438)	3.6 (0.444)
35.0 (0.991)	8.1 (1.000)
46.0 (1.303)	11.3 (1.394)
53.0 (1.500)	14.6 (1.8008)
62.0 (1.756)	19.4 (2.3930)

The target flows for Pond 4 are summarized in **Table 7.7**, for which the detailed VO2 model output is provided in **Appendix D**.

Table 7.7 – Target Flows: Pond 4

Storm Event	Peak Flows: 24-hour SCS (m^3/s)
2-year	0.371
5-year	0.453
10-year	0.636
25-year	0.759
50-year	0.879
100-year	0.915

7.3.2. Pond 6

Pond 6 currently receives flow from Pond 3, which is located on the west half of the golf course, via the Glenway Estates and Country Club storm sewer system as well as drainage from the surrounding golf course and residential lots. The existing drainage areas are described in **Table 7.1** and shown on **Figure 7-1.** Pond 6 has one (1) inlet and one (1) outlet and discharges to the storm sewer system through a 1350 mm diameter pipe and connected to an existing 1800mm dia. storm sewer on Crossland Gate. The 1800 mm diameter storm sewer flows east along Crossland Gate and south at Eagle Street to Western Creek.

The existing Storage-Discharge rating curve for Pond 6 is presented in **Table 7.8** below.

Table 7.8 – Pond 6 Storage-Discharge Rating

Discharge cfs (m^3/s)	Storage ac.ft (ha-m)
0	0
45.0 (1.274)	4.5 (0.555)
80.0 (2.265)	5.8 (0.7154)
110.0 (3.115)	7.3 (0.9004)
128.0 (3.625)	9.4 (1.160)
140.0 (3.964)	11.0 (1.357)

The target flows for Pond 6 are summarized in **Table 7.9**, for which the detailed VO2 model output is provided in **Appendix D**.

Table 7.9 – Target Flows: Pond 6

Storm Event	Peak Flows: 24-hour SCS (m^3/s)
2-year	0.858
5-year	1.051
10-year	1.815
25-year	2.506
50-year	2.992
100-year	3.168

7.3.3. Pond 8

Pond 8 currently receives runoff from the surrounding golf course, residential lots and nearby commercial lots at Davis Drive and Yonge Street. The onsite stormwater controls of the commercial lots are unknown, therefore it was assumed that runoff from these lots is uncontrolled. The existing drainage areas are described in **Table 7.1** and shown on **Figure 7-1.** Pond 8 has one (1) inlet and one (1) outlet and discharges to the storm sewer system through a 750 mm diameter pipe. The storm sewer flows south along Eagle Street and west under Glenway Circle from which it discharges into Pond 9.

The existing Storage-Discharge rating curve for Pond 8 is presented in **Table 7.10** below.

Table 7.10 – Pond 8 Storage-Discharge Rating

Discharge cfs (m^3/s)	Storage ac.ft (ha-m)
0	0
16.0 (0.543)	1.0 (0.1233)
27.0 (0.765)	1.9 (0.2343)
34.0 (0.963)	3.1 (0.3823)
46.0 (1.303)	5.6 (0.6907)
56.0 (1.586)	8.9 (1.0977)

The target flows for Pond 8 are summarized in **Table 7.11**, for which the detailed VO2 model output is provided in **Appendix D**.

Table 7.11 – Target Flows: Pond 8

Storm Event	Peak Flows: 24-hour SCS (m^3/s)
2-year	0.700
5-year	0.779
10-year	0.937
25-year	1.031
50-year	1.103
100-year	1.131

7.3.4. Pond 9

Pond 9 currently receives flow from Pond 8, via the Glenway Estates and Country Club storm sewer system as well as drainage from the surrounding golf course and residential lots. The existing drainage areas are described in **Table 7.12** and shown on **Figure 7-1.** Pond 9 has one (1) inlet and one (1) outlet and discharges to an existing 1050mm dia. storm sewer on Eagle Street through a 525 mm diameter outlet pipe. The 1050 mm diameter storm sewer flows south along Eagle Street to Western Creek.

The existing Storage-Discharge rating curve for Pond 9 is presented in **Table 7.12** below.

Table 7.12 – Pond 9 Storage-Discharge Rating

Discharge cfs (m^3/s)	Storage ac.ft (ha-m)
0	0
10.5 (0.297)	1.0 (0.1233)
15.0 (0.425)	2.9 (0.222)
18.0 (0.51)	2.9 (0.3577)
24.0 (0.68)	5.8 (0.7154)
28.0 (0.793)	9.7 (1.1964)

The target flows for Pond 9 are summarized in **Table 7.13**, for which the detailed VO2 model output is provided in **Appendix D**.

Table 7.13 – Target Flows: Pond 9

Storm Event	Peak Flows: 24-hour SCS (m^3/s)
2-year	0.505
5-year	0.546
10-year	0.651
25-year	0.702
50-year	0.739
100-year	0.749

7.4. Proposed Conditions

Under post-development conditions, it is expected that changes to site drainage patterns and land cover will affect the hydrologic behaviour of the site. The post-development drainage conditions for the major and minor system are shown in **Figure 7-2**. To mitigate these hydrologic changes, it is proposed to direct storm drainage from the development to one (1) of four (4) proposed retrofitted on-site SWM ponds, as shown on **Figure 7-2**.

The proposed development involves converting existing golf course land into single detached units, condo units, townhouses, an apartment building and a commercial block. The proposed development will increase the total impervious cover of the site to approximately 57% from the existing golf course condition. The imperviousness of proposed land uses was assumed using the Town's design standards. Where it was observed that the proposed development plan would have a higher imperiousness than the Town standards, the impervious value used was increased to reflect the actual proposed conditions shown in the Draft Plan of Subdivision prepared by Zelinka Priamo Ltd., dated March 2012. The excerpt from the Town of Newmarket design standards, providing assumed percent imperviousness and runoff coefficients for various land uses, is provided in **Appendix J**. The following typical imperviousness was assigned to the following land uses based on Town standards and proposed conditions based on the development plan:

- 0% impervious or a runoff coefficient of 0.20 for existing and proposed golf course and open grassed areas;
- 55% impervious for proposed single detached units and proposed condo blocks;

- 55% to 65% impervious or a runoff coefficient of 0.59 to 0.66 for existing single detached units based on conditions observed in satellite images of the existing development;
- 75% impervious or a runoff coefficient of 0.73 for proposed townhouse blocks;
- 85% impervious for the proposed apartment block;
- 100% impervious or a runoff coefficient of 0.90 for existing and proposed ponds;
- 90% impervious or a 0.83 runoff coefficient for existing and proposed commercial blocks; and,
- 70% impervious or a 0.69 runoff coefficient for existing and proposed roads and right-of-ways;

Visual OTTHYMO 2.4 (VO2) was used to model post-development hydrologic conditions in order to determine the required pond sizes to match post-development peak flows to pre-development peak flows from each of the four (4) ponds that will be affected by the proposed development. A mix of NashHyd and StandHyd objects were used in the model to represent the existing conditions. The input for NashHyds include a runoff coefficient (C) and a time to peak (Tp), the input for StandHyds include a directly connected impervious value (XIMP) and a total impervious value (TIMP). The detailed input parameter calculations for the post-development hydrologic model are provided in **Appendix E** and summarized below in **Table 7.14.**

Figure 7-2 Post-Development Storm Drainage Area Plan

Table 7.14 – Post-Development Input Parameters

Receiving Pond	Catchment	Drainage Area (ha)	CN value	Tp (hr)	XIMP (%)	TIMP (%)
4	4-ex1.1	6.53	74	0.19		
	4-ex1.2	2.34			0.55	0.55
	4-ex1.3	0.97			0.64	0.64
	4-ex2.1	2.95			0.25	0.25
	4-ex2.2	3.87			0.61	0.61
	4-ex2.3	0.91	74	0.17		
	4-ex2.4	6.86			0.61	0.61
	4.1	10.18			0.75	0.75
	4.2	6.71			0.80	0.80
	4.3	2.59	74	0.22		
	4.4	0.85			0.28	0.28
	4.5	1.61	74	0.13		
	4.6	2.21			0.85	0.85
6	6-ex3.1	3.62			0.28	0.28
	6-ex3.2	1.45			0.64	0.64
	6-ex3.3	1.33	74	0.13		
	6.1	8.03			0.75	0.75
	6.2	17.98			0.61	0.61
	6.3	10.64			0.70	0.70
	6.4	2.11	74	0.26		
	6.01 (major system only)	1.21			0.55	0.55
	68.1 (major system only)	1.50			0.64	0.64
8	8.1	3.28			0.70	0.70
	8.2	10.16			0.66	0.66
	8.01 (minor system only)	2.50			0.55	0.55
	68.1 (minor system only)	1.50			0.64	0.64
	98.1 (minor system only)	1.27			0.68	0.68
9	9.1	2.81			0.70	0.70
	9.2	5.86			0.56	0.56
	9.3	1.34			0.75	0.75
	9.4	2.71			0.25	0.25
	98.1 (major system only)	1.27			0.68	0.68
	9.02	0.47			0.70	0.70
	9.03	2.51			0.75	0.75

The proposed SWM plan, which includes four (4) retrofitted SWM pond facilities, will satisfy water quality and quantity control requirements. The proposed ponds are to provide quality, quantity and erosion control, as discussed in **Sections 7.5** and **7.6**.

7.5. Stormwater Quantity Control

A hydrologic model was prepared to simulate the hydrologic conditions of the site under post-development conditions at all four (4) ponds. The post-development conditions for each pond are described in **Sections 7.5.1** to **7.5.5**.

A hydrologic VO2 model was used to determine the required storage of the proposed pond to control peak flows to target flow rates. The 24-hour SCS storm distribution provided in the Town of Newmarket standards was used for the storage analysis.

As discussed in **Section 7.4**, the post-development flows discharging from each pond are to be controlled to pre-development flow rates. The discharge from the developments that drains to each pond is proposed to be controlled by retrofitting the existing ponds to accommodate the additional runoff and meet current MOE SWM guidelines, which require greater controls than when the existing ponds were originally built. The existing ponds do not meet MOE quantity control requirements for proposed conditions and the permanent pool depth of the existing ponds are unknown, thus the current quality control capabilities of the ponds cannot be confirmed.

7.5.1. Pond 4

The proposed pond is designed to provide adequate control and storage volume required in order to control the post-development peak flows to pre-development flow rates from Pond 4. Physically, the pond will remain as two hydraulically connected cells, but will be resized and repositioned. The 4A cell will be increased in size, while cell 4B will be moved further south, but remain roughly the same size.

The outlet location for the retrofitted Pond 4 is proposed to remain the same as the existing pond; however the outlet controls will require improvements. The 900 mm diameter outlet pipe discharges to the ditch that runs along Davis Drive and ultimately flows through a culvert under Davis Drive. The pond outlet controls will be revised to include a 230 mm diameter bottom draw orifice for detention control and a ditch inlet 380 mm diameter orifice tube and a 0.4 m wide control weir for 2-100 yr quantity controls. The pond stage-storage-discharge design sheet is included in **Appendix F**.

The post-development quantity control analysis of Pond 4 is summarized in **Table 7.15**, for which the detailed hydrologic model output is provided in **Appendix G**.

Table 7.15 – Quantity Control Analysis: Pond 4

Storm Event	Target Flow at Pond Outlet (m^3/s)	Inflow To Pond (m^3/s)	Pond Active Storage (m^3)	Outflow From Pond (m^3/s)
2-year	0.371	3.104	7408	0.366
5-year	0.453	3.703	9004	0.405
10-year	0.636	5.198	12,289	0.582
25-year	0.759	6.144	13,960	0.724
50-year	0.879	6.650	15,506	0.864
100-year	0.915	7.352	15,969	0.906
Provided Active Storage (2.0 m)	--	--	16,652	0.969

Figure 7-3 Proposed Pond Blocks 4A-B

As shown in **Table 7-15**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 15,969 m^3. The proposed retrofitted SWM Pond 4 provides 16,652 m^3 of active storage at an elevation of 271.70 m, and therefore meets the quantity control requirements for MOE and the town of Newmarket. The conceptual retrofitted Pond 4 layout is shown in **Figure 7-3**.

The overflow weir location is near the main outlet structure consisting of a weir, sized to pass the uncontrolled 100yr storm. The overflow begins at 2.0 m above the permanent pool and will also discharge to Davis Drive to the north.

7.5.2. Pond 6

As discussed in **Section 7.4**, the post-development flows discharging from Pond 6 are to be controlled to pre-development flow rates. The discharge from the development that drains to Pond 6 is proposed to be controlled by retrofitting the existing Pond 6 to accommodate the additional runoff and meet current MOE SWM guidelines which are more stringent than when the existing pond was built. The existing pond does not meet MOE quantity control requirements and the active storage depth is greater than the maximum depth allowed by the MOE. The permanent pool depth of the existing pond is unknown thus the existing quality control capabilities of the pond cannot be confirmed.

The proposed pond is designed to provide adequate control and storage volume required in order to control the post-development peak flows to the existing conditions target flow rates from Pond 6. The existing pond will be expanded to provide more storage to control runoff from the proposed and existing developments to the existing conditions peak flow rates up to the 100yr storm. The pond is also being expanded in order to limit the maximum water level, during storage of the 100yr storm runoff, to less than or equal to 2.0m.

The proposed Pond 6 outlet location will remain the same as the existing conditions; however the outlet controls will change from the existing. The 1350 mm diameter outlet pipe connects to the 1800 mm storm sewer system which flows east on Crossland Gate and south along Eagle Street to Western Creek. The proposed outlet controls include a bottom draw 265mm diameter orifice, a ditch inlet with a 575 mm diameter orifice tube and a 1.7m wide control weir. The pond stage-storage-discharge design sheet is included in **Appendix F**.

The post-development quantity control analysis of Pond 6 is summarized in **Table 7.16**, for which the detailed hydrologic model output is provided in **Appendix G**.

Table 7.16 – Quantity Control Analysis: Pond 6

Storm Event	Target Flow at Pond Outlet (m3/s)	Inflow To Pond (m3/s)	Pond Active Storage (m3)	Outflow From Pond (m3/s)
2-year	0.858	5.007	8763	0.762
5-year	1.051	5.908	10,288	0.878
10-year	1.815	8.173	13,680	1.602
25-year	2.506	9.757	15,698	2.300
50-year	2.992	10.593	17,414	2.880
100-year	3.168	11.827	18,240	3.150
Provided Active Storage (2.0 m)	--	--	18,366	3.225

Figure 7-4 Proposed Pond Block 6

As shown in **Table 7.16**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 18,240 m^3. The proposed retrofitted SWM Pond 6 provides 18,366 m^3 of active storage at an elevation of 266.75 m, and therefore meets the quantity control requirements for MOE and the town of Newmarket. The conceptual retrofitted Pond 6 layout is shown in **Figure 7-4**.

The overflow weir from Pond 6 is located on the south end of the pond and outlets as surface flow along the private drive to discharge to Pond 9. In order to breach the overflow location, water would need to fill up 0.5m above the 100 yr water level, which is the remaining freeboard. The remaining freeboard consists of an extra 5500 m^3 of emergency storage.

7.5.3. Pond 8

As discussed in **Section 7.4**, the post-development flows discharging from Pond 8 are to be controlled to existing conditions flow rates. The discharge from the development that drains to Pond 9 is proposed to be controlled by retrofitting the existing Pond 8 to accommodate the additional runoff and meet current MOE and town of Newmarket SWM guidelines. The existing pond does not meet MOE quantity control requirements for proposed conditions and the active storage depth is greater than the maximum allowable depth (2.0m). The permanent pool depth of the existing pond is unknown, thus the existing quality control capabilities of the pond cannot be confirmed.

The proposed pond is designed to provide adequate control and storage volume required in order to control the post-development peak flows to existing conditions target flow rates from Pond 8. The existing pond is proposed to be expanded to provide storage required to match proposed development peak flows to existing conditions. The maximum storage depth during a 100 yr storm will be 2m or less.

The proposed Pond 8 outlet location is proposed to remain the same as the existing pond; however the outlet controls and sizing will change from the existing. The existing 750 mm diameter outlet pipe connects to the 975 mm diameter storm sewer and flows south along Eagle Street and west along Glenway Circle from which it discharges into Pond 9. Quantity controls for Pond 8 will be a 170 mm diameter bottom draw orifice for extended detention, a ditch inlet and a 530 mm diameter orifice tube. The pond stage-storage-discharge design sheet is included in **Appendix F**.

The post-development quantity control analysis of Pond 8 is summarized in **Table 7.17**, for which the detailed hydrologic model output is provided in **Appendix G**.

Table 7.17 – Quantity Control Analysis: Pond 8

Storm Event	Target Flow at Pond Outlet (m3/s)	Inflow To Pond (m3/s)	Pond Active Storage (m3)	Outflow From Pond (m3/s)
2-year	0.700	2.409	3866	0.651
5-year	0.779	2.840	4455	0.753
10-year	0.937	3.922	5990	0.872
25-year	1.031	4.548	6957	0.947
50-year	1.103	4.818	7750	1.007
100-year	1.131	5.232	8048	1.030
Provided Active Storage (2.0 m)	--	--	8,027	1.030

Figure 7-5 Proposed Pond Block 8

As shown in **Table 7.17**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 8048 m^3. The proposed retrofitted SWM Pond 8 provides 8027 m^3 of active storage at an elevation of 272.40 m, and therefore meets the quantity control requirements for MOE and Town of Newmarket. The conceptual retrofitted Pond 8 layout is shown in **Figure 7-5**.

The overflow path for Pond 8 will remain in the same location and at the same elevation as the existing.

7.5.4. Pond 9

As discussed in **Section 7.4**, the post-development flows discharging from Pond 9 are to be controlled to existing conditions flow rates. The discharge from the development that drains to Pond 9 is proposed to be controlled by retrofitting the existing Pond 9 to accommodate the additional runoff and meet current MOE and Town of Newmarket SWM guidelines. The existing pond does not meet MOE quantity control requirements for proposed conditions and the active storage depth is greater than the maximum depth allowed by the MOE. The permanent pool depth of the existing pond is unknown thus the existing quality control capabilities of the pond cannot be confirmed.

The proposed pond is designed to provide the adequate control and storage volume required in order to control the post-development peak flows to existing conditions flow rates from Pond 9. The existing pond is proposed to be expanded to provide the storage required to match proposed development peak flow rates to existing conditions. The maximum active storage will be controlled to 2m or less for all storms up to the 100 yr.

The proposed Pond 9 outlet location is proposed to remain the same as the existing pond; however the outlet controls will change to meet peak flow requirements. The 525 mm diameter outlet pipe connects to the 1050 mm diameter storm sewer system and flows south along Eagle Street to Western Creek. Proposed quantity controls for Pond 9 will include a 200 mm diameter bottom draw orifice, a ditch inlet and a 505 mm diameter orifice plate. The pond stage-storage-discharge design sheet is included in **Appendix F**.

The post-development quantity control analysis of Pond 9 is summarized in **Table 7.18**, for which the detailed hydrologic model output is provided in **Appendix G**.

Table 7.18 – Quantity Control Analysis: Pond 9

Storm Event	Target Flow at Pond Outlet (m^3/s)	Inflow To Pond (m^3/s)	Pond Active Storage (m^3)	Outflow From Pond (m^3/s)
2-year	0.505	2.033	4954	0.399
5-year	0.546	2.587	6320	0.450
10-year	0.651	3.917	9546	0.571
25-year	0.702	4.592	11,569	0.647
50-year	0.739	4.979	13,365	0.714
100-year	0.749	5.496	13,890	0.733
Provided Active Storage (2.0 m)	--	--	14,033	0.739

Figure 7-6 Proposed Pond Block 9

As shown in **Table 7.18**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 13,890 m^3. The proposed retrofitted SWM Pond 9 provides 14,033 m^3 of active storage at an elevation of 266.45 m, and therefore meets the quantity control requirements. The conceptual retrofitted Pond 9 layout is shown in **Figure 7-6**.

The overflow path from Pond 9 will remain as existing. During extreme events, Pond 9 receives overflow from Ponds 6 and 8. The overflow from Pond 9 will flow through the rear-yard of the proposed lots to the east and spill on to the Eagle Street R.O.W. to flow south. The pond would need to fill up by another 1.0m above the 100 yr level before beginning to spilling onto Eagle Street. The existing lots along the south end of Pond 9 have been surveyed at an elevation of 268.00m.

7.6. Water Quality

Stormwater treatment must meet Enhanced (Level 1) Protection criteria as defined by the MOE SWMPD Manual (2003). The existing ponds were originally designed to provide quantity control but not quality control. It is proposed that the existing ponds remain as wet pond facilities and be retrofitted to meet current MOE SWM pond guidelines for both quantity and quality control. Minor storm drainage to Ponds 4, 6, 8 and 9 is to be treated by the proposed retrofitted wet pond facilities.

7.6.1. Detention Storage

For outlet erosion control, the 24 hour detention of the 25 mm 4 hour Chicago Storm is targeted for additional quality control measure as required by MOE SWM guidelines. A bottom draw orifice plate system is proposed to control the extended detention portion of each pond's active storage.

The existing ponds do not account for any 24 hour detention storage as a quality control feature. The 25 mm Chicago Storm rainfall event is used to determine the runoff volumes required for detention storage, which dictates the height of the water above the orifice. The 25 mm VO2 output can be found in **Appendix G**.

Pond 4 and the proposed controls for that pond will be used for the example calculation of the detention time met for each pond. Water stored in the extended detention portion of the pond is to be controlled by a 230 mm diameter orifice plate at an invert elevation of 269.70 m. Calculations were undertaken to confirm that extended detention would occur for a minimum of 24 hours using *equation 4.11* of the MOE SWM Planning and Design Manual.

$$t = \frac{0.66 C_2 h^{1.5} + 2 C_3 h^{0.5}}{2.75 A_O}$$

Where:

A_o = Cross-sectional area of orifice ([Pi * (0.23m/2)^2], m^2)
C_2 = Slope co-efficient from the area-depth linear regression (2081.3)
C_3 = Intercept from the area-depth linear regression (6069.4)
h = Maximum water elevation above center-line of orifice (0.65 m)
t = 25.54 hr

With the calculated extended detention time of 25.54 hours, the proposed orifice plate meets the 24 hour minimum detention time requirements. **Table 7.19** summarizes the 24 drawdown capabilities of the proposed ponds and controls.

Table 7.19 – Drawdown Time: SWM Ponds

SWM Pond	Bottom Draw Orifice Size (mm)	Slope Coeff. (C2)	Y-Intercept (C3)	Maximum Depth of Detention Storage (m)	Drawdown Time (hr)
Pond 4	230	2081.3	6069.4	0.65	25.54
Pond 6	265	1607.1	7478.5	0.70	24.05
Pond 8	170	986.29	3003	0.75	25.02
Pond 9	200	1516.4	5540.0	0.45	24.87

It can be seen from **Table 7.19** that all ponds have been upgraded to meet the MOE recommended drawdown time of 24 hours for the 25 mm storm event.

7.6.2. Permanent Pool

The permanent pool storage volumes for the proposed retrofitted SWM ponds required to meet the quality control criteria are shown in **Table 7.20**. It has been assumed that quality control is being provided only for the areas draining directly into each pond. External catchments that pass through other existing ponds with no proposed development, i.e. ponds west of the hydro corridor, are assumed to be treated by those existing ponds west of the corridor. Detailed permanent pool calculations are provided in **Appendix H.**

Table 7.20 – Water Quality Requirements: SWM Ponds

SWM Pond	Total Drainage Area to SWM Pond (ha)	% Impervious	Required Permanent Pool Volume (m^3)	Minimum Required Extended Detention Volume (m^3)
Pond 4	24.15	65.0	4200	966
Pond 6	39.97	65.0	7000	1600
Pond 8	18.71	70.0	3500	748
Pond 9	15.70	70.0	2900	628

Table 7.21 – Permanent Pool Summary

SWM Pond	Permanent Pool Required (m^3)	Max. Depth of Permanent Pool (m)	Permanent Pool Volume Provided (m^3)	Permanent Pool Elevation (m)
Pond 4	4200	2.5	7062	269.70
Pond 6	7000	2.5	10784	264.75
Pond 8	3500	3.0	3554	270.40
Pond 9	2900	2.5	7158	264.45

The proposed retrofitted ponds have been reshaped to account for permanent pool storage as well as active storage. The permanent pool portion of each pond has been designed to MOE standards and includes a berm separating the forebays from the rest of the permanent pool. The required and provided permanent pool for the ponds is shown in **Table 7.21**. Sufficient permanent pool has been provided to exceed the required volume for each pond, which therefore meets quality control requirements, as per MOE Level 1 protection criteria.

7.6.3. Forebay Sizing

Forebay sizing calculations were undertaken to confirm the forebay dimensions required to conform to the quality control criteria. A minimum required length to width ratio of 2:1 was applied in order to comply with MOE and Town of Newmarket design criteria. A maximum permanent pool depth of 2.5 m was applied for the retrofitted SWM ponds where space was not limited. The forebay sizing requirements for all SWM ponds are summarized in **Table 7.22**, for which the detailed sizing calculations are provided in **Appendix H**.

Table 7.22 – Forebay Sizing Requirements

SWM Pond	Minimum Forebay Length for Settling - V_S = 0.0003 m/s (m)		Minimum Dispersion Length (m)		Minimum Bottom Width (m)	
	Required	Provided	Required	Provided	Required	Provided
Pond 4	A-21.4 B-17.2	A-25 B-22.0	A-14.8 B-11.8	A-25.0 B-22.0	A-1.9 B-1.9	A-8 B-10
Pond 6	29	40	37.8	40	4.7	20
Pond 8	29.7	36	15.1	36	1.9	8
Pond 9	22.4	25	16.4	25	2	10

7.6.4. Phosphorus Loading

The proposed development will change the runoff characteristics of the site and will result in an increase in phosphorus loading to the watershed. A portion of the subject site (Pond 4) is situated in the West Holland subwatershed and a portion of the site is in the East Holland subwatershed (Ponds 6, 8 and 9).

LSRCA's recent study on phosphorus loading to Lake Simcoe (Estimation of the Phosphorus Loadings to Lake Simcoe, September 2010) indicates that in the East Holland Creek watershed the annual phosphorus loading rates in a growth scenario (for conservative calculation) are as summarized in **Table 7.23**.

Table 7.23 – Phosphorus Loading

Land Use	Pre-Development Area (ha)	Pre-Development Phosphorus Load (kg/year)	Post-Development Area (ha)	Post-Development Phosphorus Load (kg/year)	SWM Reduction (%)	Post-Development Phosphorus Load After SWM (kg/yr)
Grass/Pasture	2.0	0.24	1.5	0.18	63	0.07
Commercial/Industrial	9.8	17.87	9.7	17.62	63	6.52
High-Density Residential	47.7	63.04	73.7	97.32	63	36.01
Open Water	1.5	0.38	4.5	1.17	63	0.43
Golf Course	37.0	8.87	8.6	2.06	63	0.76
TOTAL	**98.0**	**90.40**	**98.0**	**118.35**	**63**	**43.79**

The wet ponds will be accounted to remove 63% of phosphorus on the site. Previously, wet ponds could be assumed to remove 80% phosphorus (LSRCA SWM Technical Guidelines, 2010), however this has been changed since the Lake Simcoe Protection Plan (October, 2011) has been introduced. New guidelines have been set for phosphorus removal targets, removal efficiencies and loading rates. A phosphorus loading and removal tool has been developed by the LSRCA and MOE and was used for the purposes of this development. The phosphorus removal calculation sheet is provided in **Appendix I**. Phosphorus loading for the development must meet Post to Pre-development conditions and are summarized in **Table 7.23**.

Further removal of phosphorus may be achieved through infiltration techniques, such as low impact development (LID) practices, which may be located throughout the Site. For example, the following measures could be used to achieve the further reduction:

- Bioswales;
- Infiltration trenches;
- Tree pits and/or extended curbs; and/or,
- Vegetated filter strips.

It is noted that phosphorus loading reduction through the use of traditional oil / grit separators are generally not accepted without supporting studies. Phosphorus loading calculations are to be confirmed based on LID practices proposed at detailed design.

8.0 Conclusions and Recommendations

Based on our review and analysis, we conclude the site is readily serviceable and provide the following summary and recommendations:

Grading

The proposed road and lot grading scheme follows Town of Newmarket Engineering Design Standards and respects the perimeter grades of the surrounding properties. The use of retaining walls will be minimized. The grading design provides for preservation of an existing Ash tree located on proposed lots 109 & 110.

The proposed grading respects the existing and proposed drainage patterns as defined in the stormwater management section of this report. Conceptual grading designs have been presented for all medium density and high density residential blocks.

Water Supply

The proposed system pressures are between 441 kPa and 680 kPa for the areas to be connected to NW district under the normal operation. They are within the system operational pressures as suggested by MOE 2009 but higher than the Town's suggested operational pressure.

The proposed system pressures are between 275 kPa and 390 kPa for the areas to be connected to NC district under the normal operation. They are within the system operational pressures as suggested by MOE 2009 but lower than the Town's suggested operational pressure. Sufficient system pressure (higher than 14 m or 200 kPa) can be maintained within the proposed development under the fire condition.

Due to the piping layout within the proposed development, one valve chamber (at Eagle Street / Millard Ave.) will be required along the pressure boundary between the NC and NW pressure districts. A recirculation line valve in each chamber is recommended to promote water quality and looping at this location.

The system demand, system storage facility and pump capacity need to be investigated further to ensure there is sufficient storage volume and system head to support the proposed development. Additional flow tests may be required to check to the distribution system. A detailed hydraulic analysis of the water supply system would be performed during the design stage.

Storm Drainage

Storm water conveyance will be accomplished by constructing pipes through new areas of development. The proposed development will be designed to capture all existing flows draining towards the property and flows that will result from the increased density.

Existing conditions for the adjacent areas will be enhanced as the proposed development will capture storm drainage, preventing minor storm overland runoff from entering neighbouring properties.

Sanitary Sewers

The proposed development will generate a peak sanitary flow rate of 35.5 L/s at the existing sanitary outlet just downstream of Peevers Crescent, based on the combined total population peaking factor for the entire Glenway Community. A sanitary flow monitoring program was completed from June 2010 to December 2010 to measure actual sanitary flow within the existing sewers downstream of the Glenway Community within existing MH110A. The monitoring program revealed that calibrated peak sanitary flows from the Glenway Community (61.9 L/s, 100-year storm) are significantly lower than the original theoretical sewage generation rate of 177 L/s based on the original subdivision design sheets prepared by G.M. Sernas, dated January 1995. Under post development conditions, the expected peak sanitary flow rate at ex. MH110A is 97.4 L/s (35.5 L/s + 61.9 L/s) which is less than the original theoretical design flow rate. In addition, the existing 450mm diameter sanitary sub-trunk at 0.34% grade downstream of Peevers Crescent exhibits a full flow capacity of 166.2 L/s, therefore the additional sanitary flow can be accommodated by the local downstream sanitary sub-trunk system.

New sewers will be required to service the proposed development areas and shall be designed in compliance with current Town standards.

Stormwater Management

A SWM plan is proposed to reduce the increase in runoff volumes and peak flows as a result of change in land use for the proposed development. In order to meet the design criteria set forth by the Town of Newmarket, LSRCA and the MOE, quantity and quality control measures are proposed.

As part of LSRCA requirements and the Lake Simcoe Protection Plan, measures have been taken to reduce the phosphorus loading from the new development through the use of wet SWM ponds, which provide 63% removal efficiency. This alone is enough to not only meet pre development loading rates, but also reduce them by 50%.

Four (4) SWM pond facilities are proposed to meet quantity and quality requirements of the development by upgrading and expanding the four (4) existing SWM ponds onsite. There are two (2) main outlets from the site. The first is located adjacent to Pond 4 (north end of the site), discharging directly to the Davis Drive road side ditch. The second is the Eagle Street storm sewer at Crossland Gate which directs stormwater southerly from the site, received flows from Ponds 6, 8 and 9. Quantity control targets were set to meet pond outflow rates under existing conditions by using the Town Standard 24-hour SCS Design Storms. Quality control targets were based on MOE Level 1 protection and assumed the existing ponds had no quality treatment as part of the original design. The proposed SWM pond upgrades include providing sufficient treatment capacity to account for both proposed and existing residential development. Storm drainage from the proposed development area is directed to the proposed upgraded SWM ponds, including areas that currently flow uncontrolled offsite under existing conditions.

Yours truly,

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