# Marianneville Developments Ltd.



Functional Servicing Report
Section 7: Stormwater Management
Without Prejudice

## Estates of Glenway

## Town of Newmarket

**Project No. L09-301**

July 2013

COLE ENGINEERING
*Experience Enhancing Excellence*

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## APPENDICES

# 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 has been designed in consultation with the drainage and SWM requirements of the Town, the Lake Simcoe Region Conservation Authority (LSRCA) and the MOE standards.

The following guidelines were referenced for SWM design criteria:

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

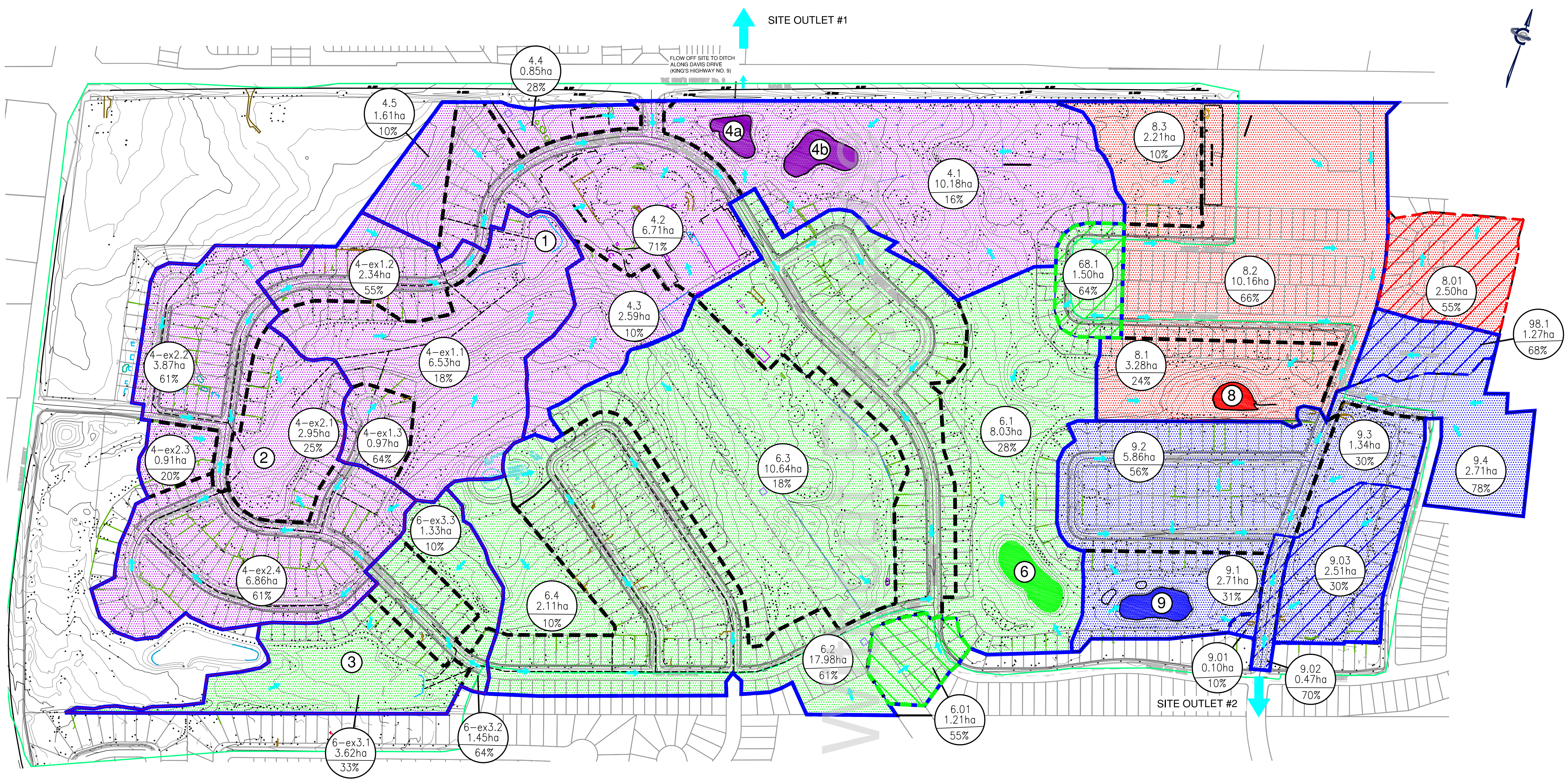


The following criteria were used to size the wet ponds:

- Quality Control – MOE Enhanced (Level 1) Protection;
- Quantity Control – Post-development peak flow control to the existing pond two (2) to 100-year peak outflows for greater of the 24-hour SCS, 12-hour SCS and four (4)-hour Chicago design storms;
- Erosion Control – 24-hour detention of the 25 mm, 4-hour Chicago storm; and,
- All Pond design characteristics to meet Town Criteria, with the exception of minimum side slopes of Ponds six (6) and nine (9).

## 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**.

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## LEGEND

- A2pre 5.60ha 61% — DRAINAGE AREA ID / AREA / RUNOFF COEFFICIENT (#) OR % IMPERVIOUS (#%)
- EXISTING POND PERMANENT WATER LEVEL
- EXISTING 100 YEAR EVENT POND WATER LEVEL
- EXISTING POND DRAINAGE AREA
- EXISTING POND SUB-CATCHMENT DRAINAGE AREA
- 6 POND ID
- DIRECTION OF OVERLAND FLOW
- EXISTING STORM SEWER

# PRE-DEVELOPMENT STORM DRAINAGE AREA PLAN

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:6000 | FIGURE No.: | 7-1 |

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's requirements, and the four (4)-hour Chicago distribution, as per the Town and LSRCA requirements. The intensity-duration-frequency (IDF) data used for the four (4)-hour Chicago storms was taken from the Town's design standards. The four (4)-hour Chicago IDF curve parameters for all storm events from the two (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 four (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 | 3036 | 0.697 | 4344 | 0.996 | 5302 | 1.215 | 6363 | 1.771 | 7117 | 2.232 | 7924 | 2.610 |
| 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 | 4487 | 0.553 | 5521 | 0.602 | 6881 | 0.667 | 7948 | 0.699 | 9052 | 0.725 |

**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 | 3582 | 0.821 | 4399 | 1.008 | 6256 | 1.709 | 7482 | 2.415 | 8522 | 2.890 | 9055 | 3.121 |
| 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 | 4354 | 0.547 | 6566 | 0.652 | 8115 | 0.703 | 9702 | 0.740 | 10156 | 0.750 |



**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 | 2736 | 0.627 | 4131 | 0.946 | 5119 | 1.173 | 6067 | 1.584 | 7294 | 2.327 | 8074 | 2.684 |
| 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 | 5678 | 0.610 | 6886 | 0.667 | 8543 | 0.713 | 9556 | 0.736 |

As observed in **Table 7.3**, **Table 7.4** and **Table 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's 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 lands to be retained, 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 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 using the ponds original design for storage and discharge. 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 original design Storage-Discharge rating for each pond has been taken from Glenway Estates SWM Study (The Lathem Group Inc., 1983).

The design standards for SWM ponds have changed since the existing ponds were designed and constructed. The original design was based on a one (1)-hour AES design storm. A combination of the current Town and LSRCA criteria require post to pre-development peak flow control and pond design for the greater of the two (2) to 100-year four (4)-hour Chicago, 12-hour SCS and 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's design storm was chosen to determine the target flows for each of the ponds. The analysis completed for each pond is described in the following **Sections 7.3.1** to **7.3.4.**



### 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 conditions drainage areas are 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 (0.4440) |
| 35.0 (0.991) | 8.1 (1.000) |
| 46.0 (1.303) | 11.3 (1.3940) |
| 53.0 (1.500) | 14.6 (1.8008) |
| 62.0 (1.756) | 19.4 (2.3930) |

* - Discharge / Area from Glenway Estates SWM Study, The Lathem Group Inc. (1983)
** - Discharge / Area converted to $m^3/s$ and ha-m for use in VO2 hydrologic model. (10000 $m^3$ = 1 ha-m)

When the existing site conditions were modelled and routed through the original design Pond 4 Storage-Discharge rating curve, the pond outlet flow rates were produced for the 24-hour storm event. These existing conditions pond release rates will become the peak flow target release rates for the proposed SWM pond 4 controls. 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 existing Pond 3, which is located on the east 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 1800 mm 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) |

* - Discharge / Area from Glenway Estates Stormwater Management Study, The Lathem Group Inc. (1983)
** - Discharge / Area converted to $m^3/s$ and ha-m for use in VO2 hydrologic model. (10000 $m^3$ = 1 ha-m)

When the existing site conditions were modelled and routed through the original design Pond 6 Storage-Discharge rating curve, the pond outlet flow rates were produced for the 24-hour storm event. These existing conditions pond release rates will become the peak flow target release rates for the proposed SWM pond 6 controls. 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) |



* Discharge / Area from Glenway Estates Stormwater Management Study, The Lathem Group Inc. (1983)
**Discharge / Area converted to $m^3/s$ and ha-m for use in VO2 hydrologic model. (10000 $m^3$ = 1 ha-m)

When the existing site conditions were modelled and routed through the original design Pond 8 Storage-Discharge rating curve, the pond 24-hour storm peak outlet flow rates were produced. These existing conditions pond outflow rates will become the peak flow target release rates for the proposed SWM pond 8 controls. 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 7.1** and shown on **Figure 7-1.**

Pond 9 has one (1) inlet and one (1) outlet and discharges to an existing 1050 mm 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) |

* Discharge / Area from Glenway Estates Stormwater Management Study, The Lathem Group Inc. (1983)
** Discharge / Area converted to $m^3/s$ and ha-m for use in VO2 hydrologic model. (10000 $m^3$ = 1 ha-m)



When the existing site conditions were modelled and routed through the original design Pond 9 Storage-Discharge rating curve, the pond 24-hour storm peak outlet flow rates were produced. These existing conditions pond outflow rates will become the peak flow target release rates for the proposed SWM Pond 9 controls. 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

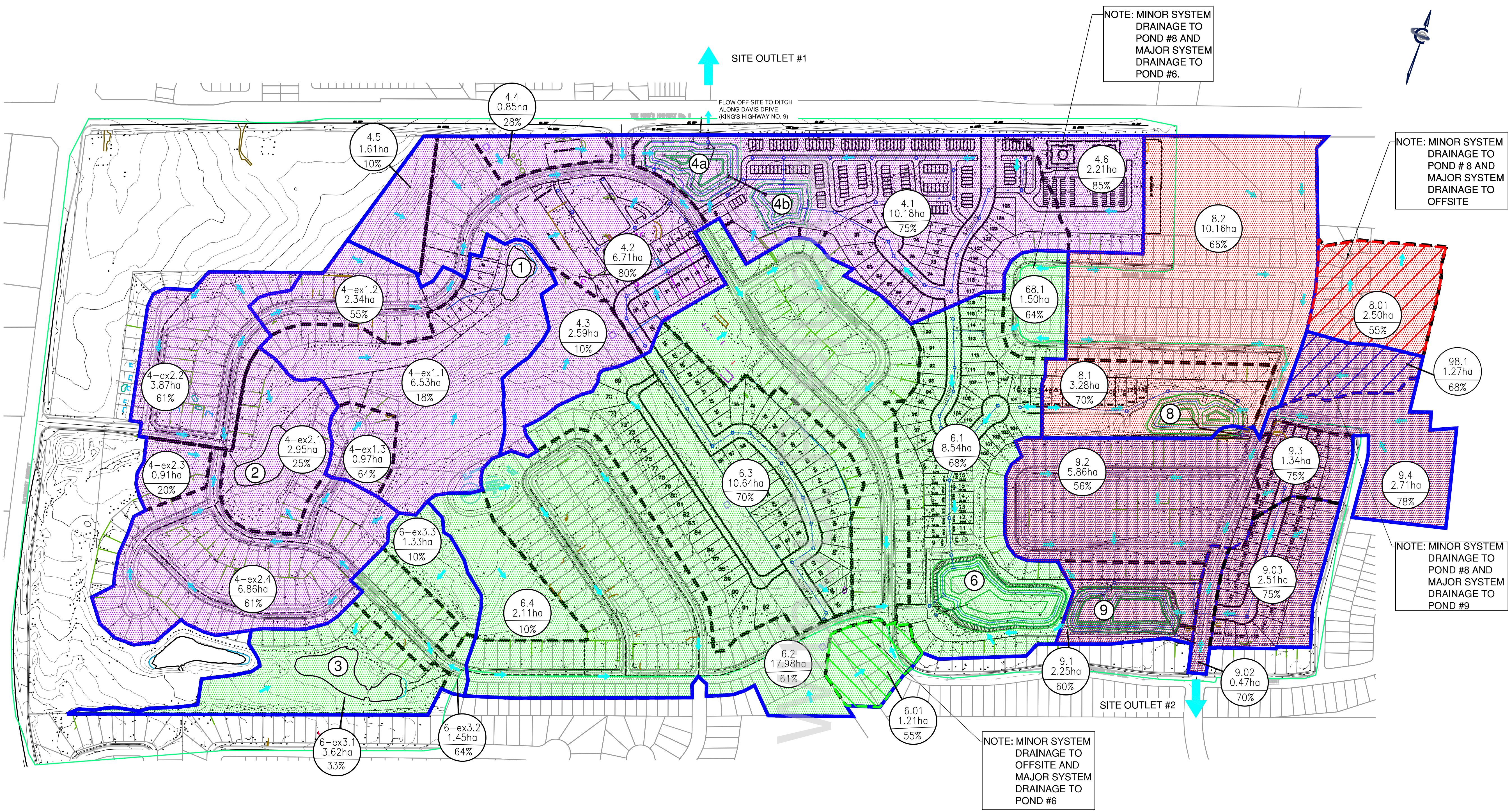
Under post-development conditions, it is expected that changes to site drainage patterns and land cover will affect the hydrologic behaviour of the site. The post-development drainage conditions for the major and minor system are shown in **Figure 7-2**. To mitigate these hydrologic changes, it is proposed to direct storm drainage from the development to 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 with dedicated parkland and a trail system. The proposed development will increase the total impervious cover of the site to approximately 55% 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 February 2013. 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 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.**

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## LEGEND

| | |
|---|---|
| A2pre 5.60ha 61% | DRAINAGE AREA ID; AREA; RUNOFF COEFFICIENT (#) OR % IMPERVIOUS (#%) |
| | PROPOSED POND PERMANENT WATER LEVEL |
| | PROPOSED 100 YEAR EVENT POND WATER LEVEL |
| | PROPOSED POND DRAINAGE AREA |
| | PROPOSED POND SUB-CATCHMENT DRAINAGE AREA |
| 6 | POND ID |
| | DIRECTION OF OVERLAND FLOW |
| | EXISTING STORM SEWER |

# POST-DEVELOPMENT STORM DRAINAGE AREA PLAN

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:6000 | FIGURE No.: | 7-2 |

**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 | 8.53 | | | 0.65 | 0.65 |
| | 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 | 2.25 | | | 0.60 | 0.60 |
| | 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 **7.5.4**.

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's standards was used for the storage analysis.

As discussed in **Section477.4**, 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 Town's standards, LSRCA criteria and MOE SWM guidelines. The existing ponds currently provide some attenuation, but were not designed to meet a specific level of protection, however many of the ponds cause flooding on private property during major events (100-year storm) as modelled using the one (1)-hour AES storm by The Lathem Group (1983).

### 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 (2) 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.

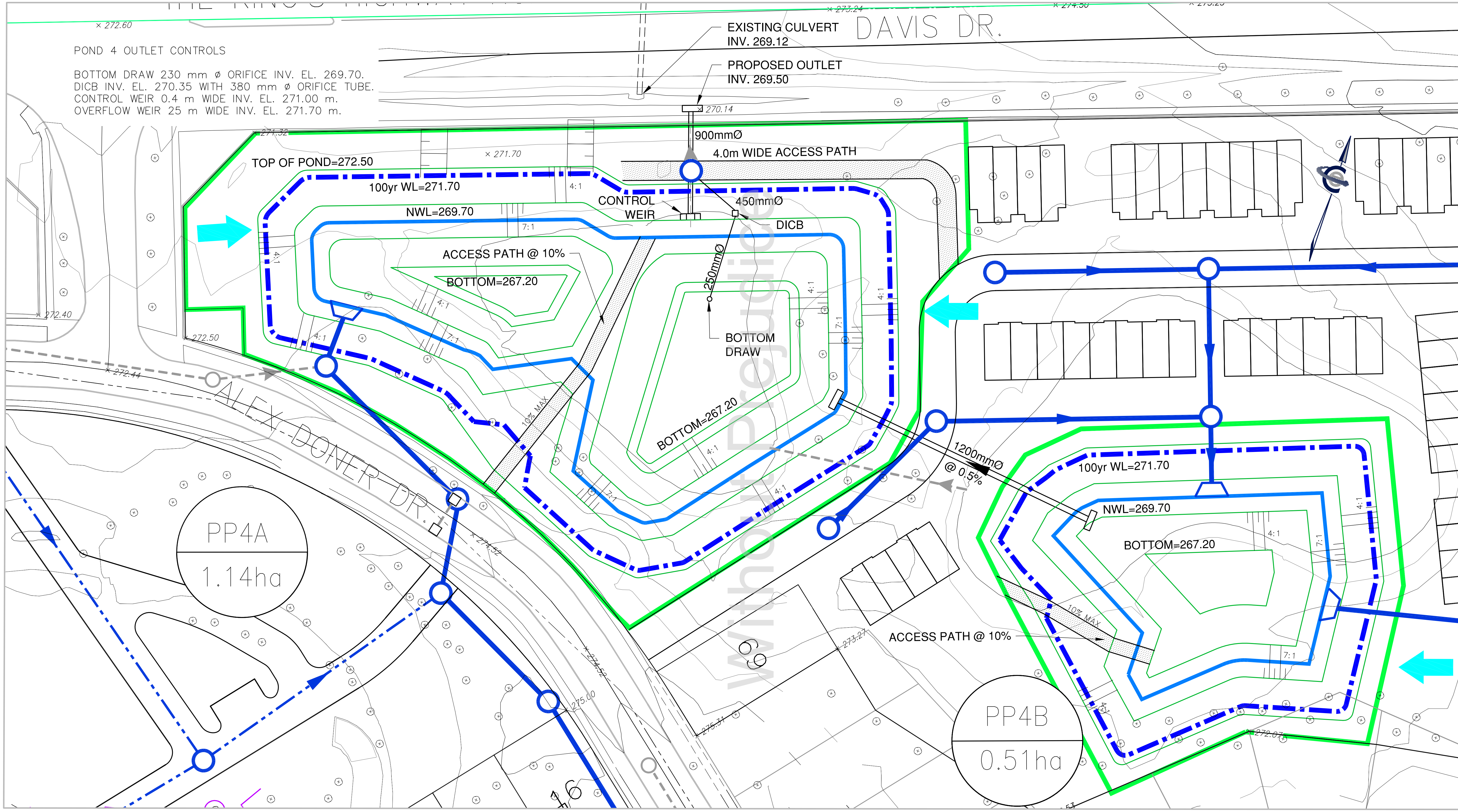


The outlet location for the retrofitted Pond 4 is proposed to remain the same as the existing pond; however the outlet controls will require improvements. The 900 mm diameter outlet pipe discharges to the ditch that runs along Davis Drive and ultimately flows through a culvert under Davis Drive. The pond outlet controls will be revised to include a bottom draw to a 230 mm diameter orifice plate, a ditch inlet catch-basin with a 430 mm diameter orifice plate and a 0.4 m wide control weir for 2-100 year 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 | 7,342 | 0.366 |
| 5-year | 0.453 | 3.703 | 8,947 | 0.407 |
| 10-year | 0.636 | 5.198 | 12,087 | 0.595 |
| 25-year | 0.759 | 6.144 | 13,834 | 0.727 |
| 50-year | 0.879 | 6.650 | 15,379 | 0.871 |
| 100-year | 0.915 | 7.352 | 15,833 | 0.914 |
| Provided Active Storage (2.0 m) | -- | -- | 16,432 | 0.970 |

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LEGEND

- PROPOSED POND PERMANENT WATER LEVEL
- PROPOSED 100 YEAR EVENT POND WATER LEVEL
- PROPOSED POND BLOCK BOUNDARY
- PP1 4.17ha — PROPOSED POND BLOCK (ha)
- DIRECTION OF OVERLAND FLOW
- EXISTING STORM SEWER
- PROPOSED STORM SEWER

# PROPOSED POND BLOCKS 4A-B

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:750 | FIGURE No.: | 7-3 |

As shown in **Table 7.15**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 15,833 $m^3$. The proposed retrofitted SWM Pond 4 provides 16,432 $m^3$ of active storage at an elevation of 271.70 m, and therefore meets the quantity control requirements for MOE and the Town. The conceptual retrofitted Pond 4 layout is shown in **Figure 7-3**.

The overflow spillway location is near the main outlet structure consisting of a weir, sized to pass the uncontrolled 100 year storm. The overflow begins at 2.0 m above the permanent pool and will also discharge to Davis Drive to the north, as it currently during existing conditions. The emergency spillway will have a 25 m wide bottom width and a height of 0.5 m. This has been modelled with Bentley Flowmaster using the Town's standard of 0.1 $m^3/s/ha$ to pass 4.57 $m^3/s$ and is provided in **Appendix H**.

### 7.5.2. Pond 6

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 100 year storm. The pond is also being expanded in order to limit the maximum water level, during storage of the 100 year storm runoff, to less than or equal to 2.0 m.

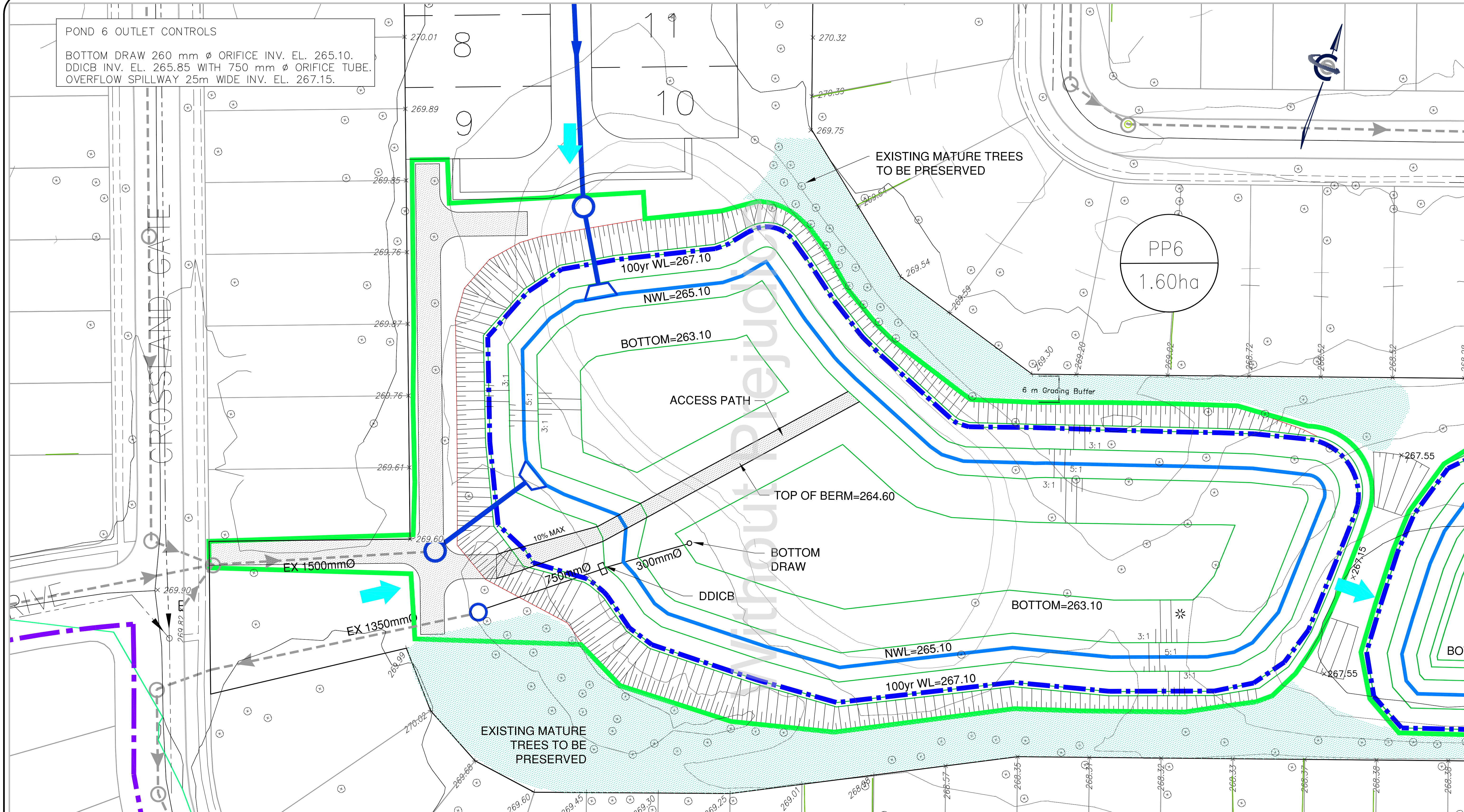
The proposed Pond 6 outlet location will remain the same as the existing conditions; however the outlet controls will change from the existing. The 1350 mm diameter outlet pipe connects to the 1800 mm storm sewer system which flows east on Crossland Gate and south along Eagle Street to Western Creek. The proposed outlet controls include a bottom draw pipe to a 260 mm diameter orifice plate and a ditch inlet catch-basin with a 750 mm diameter orifice tube and a 25 m wide emergency spillway. 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 | 0.821 | 4.840 | 8,902 | 0.542 |
| 5-year | 1.008 | 5.721 | 10,072 | 0.993 |
| 10-year | 1.709 | 8.004 | 13,303 | 1.628 |
| 25-year | 2.415 | 9.603 | 15,671 | 1.776 |
| 50-year | 2.890 | 10.452 | 17,806 | 1.907 |
| 100-year | 3.121 | 11.693 | 18,768 | 1.968 |
| Provided Active Storage (2.0 m) | -- | -- | 19,119 | 2.026 |




LEGEND

| | |
|---|---|
| | PROPOSED POND PERMANENT WATER LEVEL |
| | PROPOSED 100 YEAR EVENT POND WATER LEVEL |
| | EXISTING MATURE GROWTH TREES TO BE PRESERVED |
| | PROPOSED POND BLOCK BOUNDARY |
| PP1 4.17ha | PROPOSED POND BLOCK (ha) |
| | DIRECTION OF OVERLAND FLOW |
| | EXISTING STORM SEWER |
| | PROPOSED STORM PIPES |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# PROPOSED POND BLOCK 6

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:750 | FIGURE No.: | 7-4 |

As shown in **Table 7.16**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 18,768 $m^3$. The proposed retrofitted SWM Pond 6 provides 19,119 $m^3$ of active storage at an elevation of 267.10 m, and therefore; meets the quantity control requirements for MOE and the Town. The conceptual retrofitted Pond 6 layout is shown in **Figure 7-4**.

The overflow weir from Pond 6 is located on the southeast end of the pond and flows directly east to Pond 9. In order to reach the overflow location, water would need to fill up 0.05 m above the 100 year water level, which is the remaining freeboard. The emergency spillway will have a 25 m wide bottom width and a minimum height of 0.5 m. The proposed trail passes along the overflow spillway; therefore the side slopes of the spillway will not exceed 10%. This has been modelled with Bentley Flowmaster using the Town's standard of 0.1 $m^3$/s/ha to pass 4.57 $m^3$/s and is provided in **Appendix H**.

### 7.5.3. Pond 8

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 year storm will be 2 m or less.

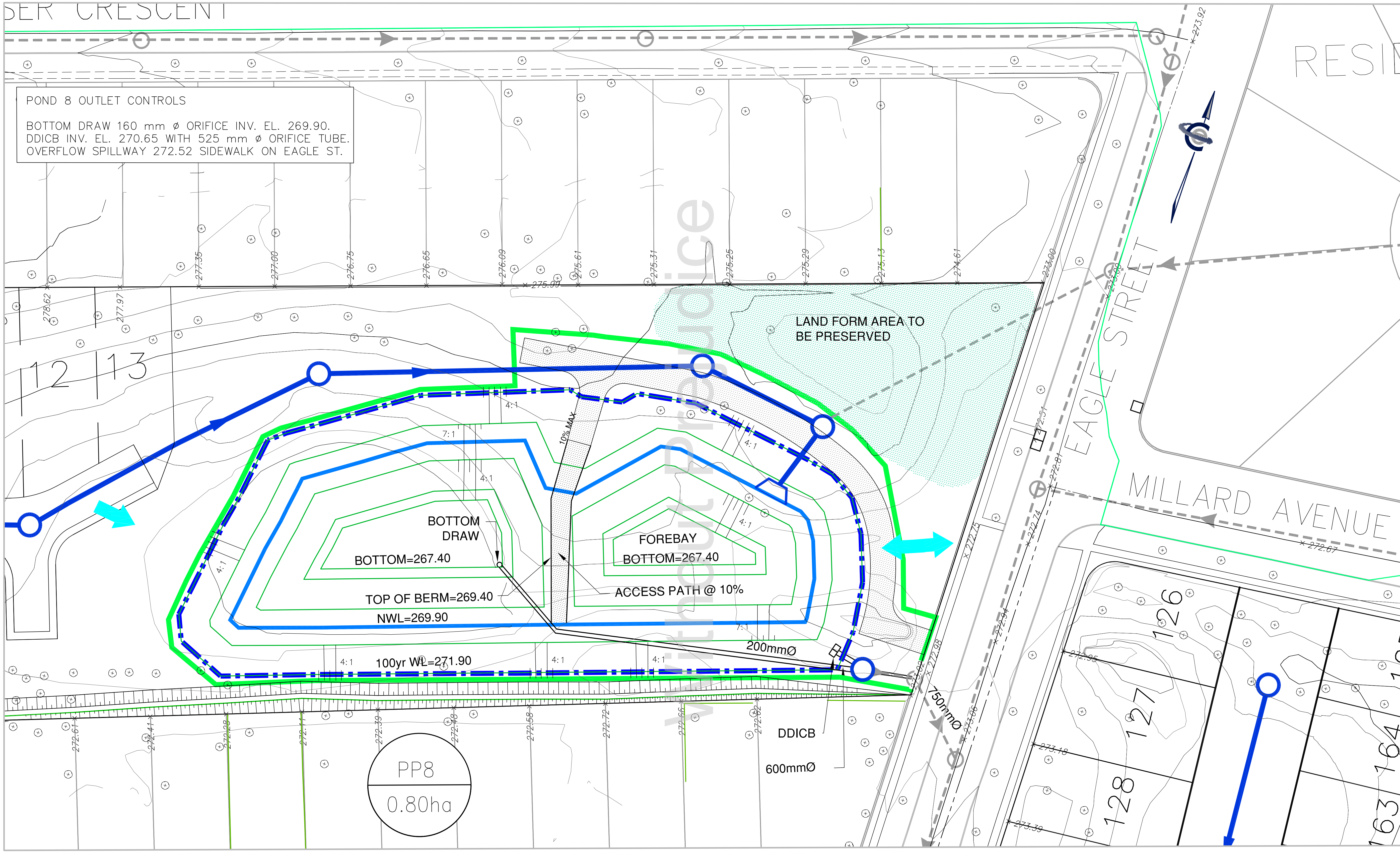
The proposed Pond 8 outlet location is proposed to remain the same as the existing pond; however the outlet controls and sizing will change from the existing. The existing 750 mm diameter outlet pipe connects to the 975 mm diameter storm sewer and flows south along Eagle Street and west along Glenway Circle from which it discharges into Pond 9. Quantity controls for Pond 8 will include a bottom draw pipe to a 160 mm diameter orifice plate, a ditch inlet catch-basin and a 525 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 | 4,067 | 0.479 |
| 5-year | 0.779 | 2.840 | 4,668 | 0.711 |
| 10-year | 0.937 | 3.922 | 6,303 | 0.813 |
| 25-year | 1.031 | 4.548 | 7,306 | 0.875 |
| 50-year | 1.103 | 4.818 | 8,162 | 0.929 |
| 100-year | 1.131 | 5.232 | 8,481 | 0.948 |
| Provided Active Storage (2.0 m) | -- | -- | 9,482 | 1.011 |





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LEGEND

- PROPOSED POND PERMANENT WATER LEVEL
- PROPOSED 100 YEAR EVENT POND WATER LEVEL
- LAND FORM AREA TO BE PRESERVED
- PROPOSED POND BLOCK BOUNDARY
- PP1 4.17ha PROPOSED POND BLOCK (ha)
- DIRECTION OF OVERLAND FLOW
- EXISTING STORM SEWER
- PROPOSED STORM PIPES

# PROPOSED POND BLOCK 8

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:750 | FIGURE No.: | 7-5 |

As shown in **Table 7.17**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 8,481 $m^3$. The proposed retrofitted SWM Pond 8 provides 9,482 $m^3$ of active storage at an elevation of 271.90 m, and therefore meets the quantity control requirements for MOE and Town. The conceptual retrofitted Pond 8 layout is shown in **Figure 7-5**.

The overflow path for Pond 8 will not remain in the same location as the existing, which currently passes south through existing residential lots. Emergency overflow from Pond 8 is proposed to flow back out to Eagle Street, back up along Millard Ave to pass down proposed Street D to Pond 9. This will be made possible through creating a berm along the south side of Pond 8 to a height of 273.0 m, whereas the lowest elevation along the sidewalk on Eagle Street is 272.52 m. There is approximately an additional 3,300 $m^3$ of emergency storage prior to reaching the overflow location onto Eagle Street.

### 7.5.4. Pond 9

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 2 m or less for all storms up to the 100 year.

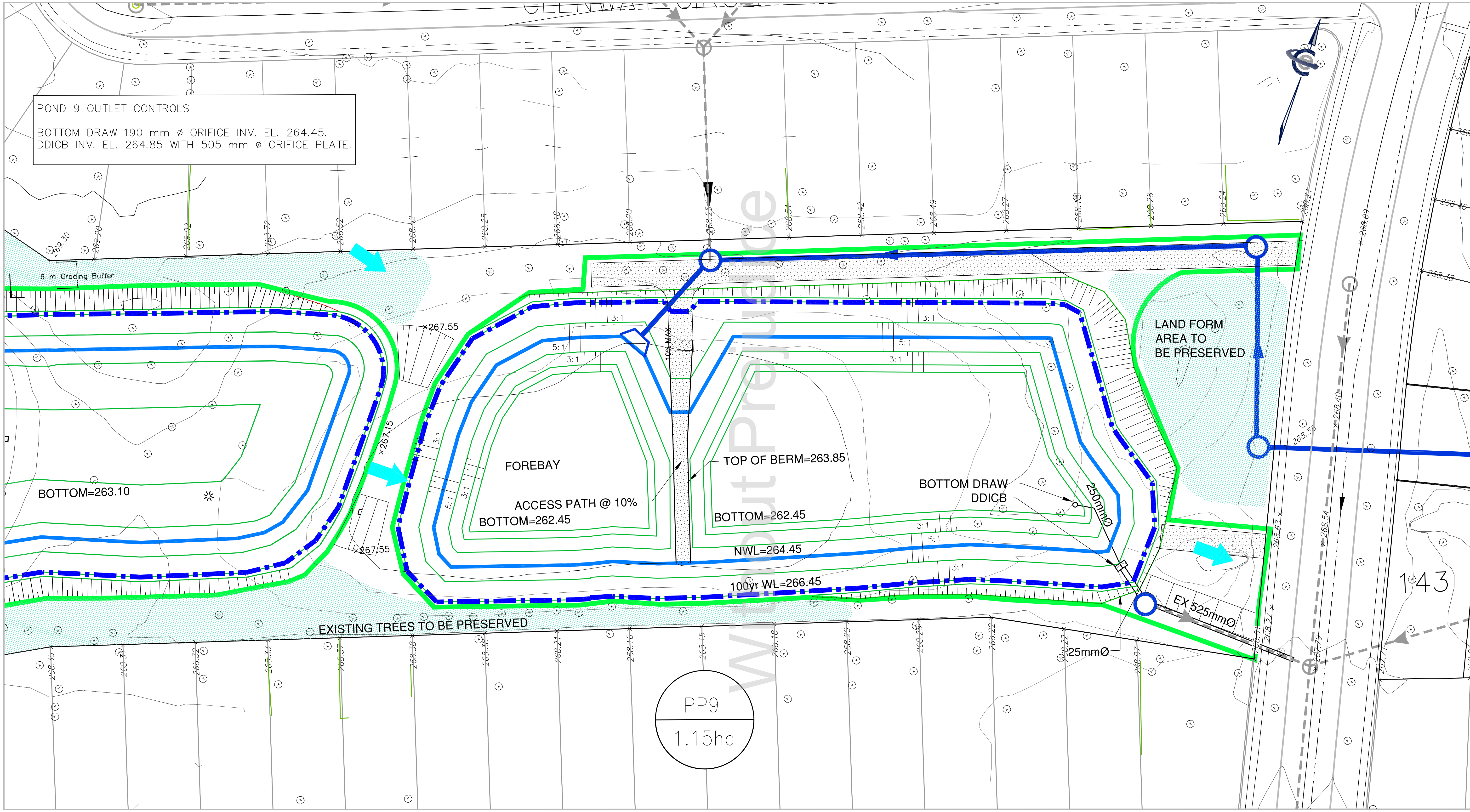
The proposed Pond 9 outlet location is proposed to remain the same as the existing pond; however the outlet controls will change to meet peak flow requirements. The 525 mm diameter outlet pipe connects to the 1050 mm diameter storm sewer system and flows south along Eagle Street to Western Creek. Proposed quantity controls for Pond 9 will include a bottom draw pipe to a 200 mm diameter orifice plate, a ditch inlet catch-basin 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 | 1.881 | 4494 | 0.366 |
| 5-year | 0.546 | 2.333 | 5808 | 0.415 |
| 10-year | 0.651 | 3.636 | 8977 | 0.533 |
| 25-year | 0.702 | 4.375 | 10932 | 0.605 |
| 50-year | 0.739 | 4.743 | 12690 | 0.671 |
| 100-year | 0.749 | 5.239 | 13209 | 0.690 |
| Provided Active Storage (2.0 m) | -- | -- | 14,514 | 0.739 |




LEGEND

PROPOSED POND PERMANENT WATER LEVEL

PROPOSED 100 YEAR EVENT POND WATER LEVEL

EXISTING FEATURES TO BE PRESERVED

PROPOSED POND BLOCK BOUNDARY

PP1 4.17ha PROPOSED POND BLOCK (ha)

DIRECTION OF OVERLAND FLOW

EXISTING STORM SEWER

PROPOSED STORM PIPES

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# PROPOSED POND BLOCK 9

MARIANNEVILLE DEVELOPMENTS LTD.
ESTATES OF GLENWAY NEWMARKET
TOWN OF NEWMARKET

| | | | |
|---|---|---|---|
| DATE: | JULY, 2013 | PROJECT No.: | L09-301 |
| SCALE: | 1:750 | FIGURE No.: | 7-6 |

As shown in **Table 7.18**, the maximum required active pond storage to control the post-development peak flows to pre-development conditions is 13,726 $m^3$. The proposed retrofitted SWM Pond 9 provides 15,459 $m^3$ 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 emergency overflow path from Pond 9 will remain as existing. During extreme events, Pond 9 receives overflow from Pond 6. The overflow from Pond 9 will flow towards the east and spill on to the Eagle Street R.O.W. and flow south. The existing lots along the south end of Pond 9 have been surveyed at an approximate minimum elevation of 268.00 m, which is located at Eagle Street.

## 7.6. Pond Physical Design Characteristics

The proposed SWM ponds have been designed to meet the Town's standards where possible, and the MOE SWMP Design Guidelines where further constraints have been imposed and met.

### 7.6.1. Constraints

There were a number of physical features and buffers requested by the Town to be retained during the design of the proposed ponds.

There are a number of landforms, tree plantings and mature growth around Ponds 6, 8 and 9 that were requested to be retained. Grading buffers were also provided for the implementation of a trail system through the park system and to keep the ponds back from the roads. The lists of the additional constraints are listed below:



- Retain tree cluster at southwest corner of Pond 6;
- Retain tree plantings at northeast edge of Pond 6;
- Retain landform at northeast corner of Pond 8;
- Retain landform at northeast corner of Pond 9;
- Provide a 10 m buffer from pond grading between ponds and existing residential properties for provision of trail system;
- Provide 20 m buffer between Eagle St. ROW and Pond 9 grading for park access, etc.;
- Provide 10 m buffer between Eagle St. ROW and Pond 8 grading for park access, etc.; and
- Retain any other existing trees where possible.

These constraint features have all been included on the proposed SWM pond **Figures 7-3 to 7-6** for reference.

### 7.6.2. Design Criteria

The Town's engineering design criteria have been met for all ponds where constraints did not interfere with the layout of pond. The Town's design criteria are listed in **Table 7.19** and each pond is listed with how it was designed.

**Table 7.19 – Town of Newmarket SWM Pond Design Characteristics**

| Design Characteristic | Town Minimum Standard | Pond 4 | Pond 6 | Pond 8 | Pond 9 |
|---|---|---|---|---|---|
| Sideslopes | 4:1 | 4:1 | 3:1 | 4:1 | 3:1 |
| Safety Shelf | 7:1 for 3.5 m either side of NWL | 7:1 for 3.5 m either side of NWL | 5:1 for 0.5 m above and below NWL | 7:1 for 3.5 m either side of NWL | 5:1 for 0.5 m above and below NWL |
| P. Pool Depth | 2.5 m | 2.5 m | 2.0 m | 2.5 m | 2.0 m |
| Water Flucuation NWL to HWL | 2.0 m | 2.0 m | 2.0 m | 2.0 m | 2.0 m |
| Min. Freeboard | 0.25 m | 0.5 m | 0.5 m | 0.8 m | 1.55 m |
| Emergency Overflow | 0.1 $m^3$/s/ha | 0.1 $m^3$/s/ha | 0.1 $m^3$/s/ha | Dependant on Eagle St. grades | Dependant on Eagle St. grades |
| Maintenance Access | 3.0 m wide and 10% max grade | 4.0 m wide @ 10% | 4.0 m wide @ 10% | 4.0 m wide @ 10% | 4.0 m wide @ 10% |

It can be seen that the design features of each pond meet or exceed the Town's minimum standard, with the exception of the side slopes and safety shelf of Ponds 6 and 9. The physical constraints placed around these two (2) ponds limit the useable surface area for grading of these ponds. However, in order to meet he volume requirements to achieve quantity control, the MOE minimum grading recommendations were used.



### 7.6.3. Grading

The grading of the proposed SWM ponds was designed to incorporate the Town's minimum standard side slopes of 4:1 and the 7:1 safety shelf everywhere possible, as physical constraints and storage requirements would allow. Ponds 4 and 8 have been designed to meet all Town's standard grading requirements. Ponds 6 and 9 present the challenge of retaining existing tree and landform constraints, while also providing the required storage

Due to the large storage volumes required and the physical constraints implemented, it became clear that to generate the storage volume needed was to use an alternative method of grading. Using 3:1 side slopes and 5:1 safety shelf provides adequate storage volume, without increasing the pond upper grading limits. This alternative grading option, using MOE guidelines, was preferred to the removal existing tree cover and loss of possible parkland and trails.

## 7.7. Water Quality

Stormwater treatment must meet the Town's criteria of Enhanced (Level 1) Protection quality treatment 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.

### 7.7.1. Permanent Pool

The permanent pool depth of the existing ponds are unknown, thus the current quality control capabilities of the ponds cannot be confirmed. The permanent pool storage volumes for the proposed retrofitted SWM ponds required to meet the MOE Enhanced Protection 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 | 4,200 | 966 |
| Pond 6 | 39.27 | 65.0 | 6,900 | 1600 |
| Pond 8 | 18.71 | 65.0 | 3,300 | 750 |
| Pond 9 | 15.14 | 70.0 | 2,800 | 600 |

**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 | 6,598 | 269.70 |
| Pond 6 | 7000 | 2.0 | 9,650 | 265.10 |
| Pond 8 | 3500 | 2.5 | 3,769 | 269.90 |
| Pond 9 | 2900 | 2.0 | 7,928 | 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.

### 7.7.2. 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's 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-40.6<br>B-35.1 | A-50<br>B-56 | A-18.5<br>B-6.2 | A-50<br>B-56 | A-2.3<br>B-0.8 | A-10<br>B-15 |
| 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 |

### 7.7.3. 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.

## 7.8. Extended Detention

For outlet erosion control, the 24 hour detention of the 25 mm four (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.

$$t = \frac{0.66C_2h^{1.5} + 2C_3h^{0.5}}{2.75A_O}$$

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 (2245)
$C_3$ = Intercept from the area-depth linear regression (5795)
$h$ = Maximum water elevation above center-line of orifice (0.65 m)
*t = 24.06 hr*



With the calculated extended detention time of 24.06 hours, the proposed orifice plate meets the 24 hour minimum detention time requirements. **Table 7.24** summarizes the 24 drawdown capabilities of the proposed ponds and controls.

**Table 7.24 – 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 | 2245 | 5795 | 0.65 | 24.06 |
| Pond 6 | 260 | 1415 | 7868 | 0.70 | 26.09 |
| Pond 8 | 160 | 1229 | 3387 | 0.75 | 32.12 |
| Pond 9 | 190 | 1387 | 5356 | 0.40 | 24.96 |

It can be seen from **Table 7.24** that all ponds have been upgraded to meet the MOE recommended drawdown time of 24 hours for the 25 mm storm event.