Victoria Square Boulevard Class Environmental Assessment

Woodbine Avenue (north connection) to Woodbine Avenue (south connection)

Environmental Study Report

Appendix

F

Drainage and Stormwater Management

Drainage and Stormwater Management Report

Class Environmental Assessment for Victoria Square Boulevard from Woodbine Avenue (north connection) to Woodbine Avenue (south connection)

City of Markham
May 8, 2018

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1 Introduction

The City of Markham initiated a Municipal Class Environmental Assessment (Class EA) to study potential improvements to Victoria Square Boulevard from Woodbine Avenue (north connection) to Woodbine Avenue (south connection).

This Drainage and Stormwater Management Report has been prepared in support of the Class EA Study and complies with the Ministry of the Environment and Climate Change (MOECC), Toronto Region Conservation Authority (TRCA), and City of Markham's Policies and Standards. The Victoria Square Boulevard Class EA Study limits are illustrated in **Figure** 1.

Figure 1 Victoria Square Boulevard EA Study Area and Area Segments

Victoria Square Boulevard, within the study limits, is a north-south rural road that is approximately 3 km in length as illustrated in **Figure** 1. Elgin Mills Road intersects the corridor approximately mid-way between the north and south connections to Woodbine Avenue. The lands adjacent to the Victoria Square Boulevard corridor consist of a mixture of residential properties, public parks and some commercial and institutional land use. The study corridor has been divided into six (6) segments along the Victoria Square

Boulevard study corridor based on land use and characteristics. **Table 1-1** provides details of each road segment area.

Table 1-1: Victoria Square Boulevard Study Corridor Road Segment Area

Area	Area Description
Area 1: North Gateway	From the north connection with Woodbine Avenue to approximately 40m north of Vetmar Road
Area 2: Residential Main Street	From approximately 40m north of Vetmar Road to Prince of Wales Drive
Area 3: Hamlet of Victoria Square	From Prince of Wales Drive to approximately 90 m north of Reflection Road/Rinas Avenue
Area 4: Cathedral Residential	From approximately 90 m north of Reflection Road/Rinas Avenue to Church View Avenue
Area 5: Cathedral Precinct	From Church View Avenue to approximately 30 m south of Murison Drive
Area 6: Cathedral Community Gateway	From approximately 30 m south of Murison Drive to Woodbine Avenue south connection

As per the design concepts of the preferred road improvement solution, the objective of this Drainage and Stormwater Management Report is to develop a strategic approach to the level of development of the proposed project that will:

- Identify and evaluate existing drainage patterns and performance of existing transverse culverts which convey external drainage across Victoria Square Boulevard;
- Identify potential stormwater runoff quality and quantity impacts to the receiving watercourses as a result of potential pavement area increases; and
- Propose an appropriate drainage system, transverse culvert upgrades and stormwater management system in conjunction with the proposed road widening.

1.1 Background Information

In preparation of the Victoria Square Boulevard Class Environmental Assessment Drainage and Stormwater Management Report, essential documents were obtained and reviewed as follows:

1. MOE Stormwater Management Planning and Design Manual, March 2003;
2. TRCA Stormwater Management Criteria, August 2012;
3. York Region Road Design Guidelines, June 2014;
4. City of Markham Stormwater Management Guidelines, October 2016;
5. City of Markham Design Criteria Section E - Storm Drainage & Stormwater Management, June 2016 (Rev. 3);
6. Stormwater Management Design Brief, Woodbine Mackenzie Developments Limited – East Cathedral Community prepared by Stantec Consulting Ltd., revised September 2002;

7. Stormwater Management Report, West Cathedral Community, Cathedral Town Phase 3 Development prepared by URS Corporation, March 2006;

8. Stormwater Management Report, East Cathedral Community prepared by Stantec Consulting Ltd., revised May 2002;

9. Environmental and Stormwater Master Servicing Plan for West Cathedral prepared by URS Cole Sherman, October 2002;

10. Stormwater Management Pond Design Report - Interim Pond B3 prepared by Masongsong Associates Engineering Limited, August 2008;

11. Stormwater Management and Pond Design Report for Monarch Corporation Traditions at Victoria Square, July 2007.

2 Existing Drainage Conditions

Victoria Square Boulevard is currently a two-lane roadway with a predominately rural cross-section and an existing right-of-way ranging between 34 to 36 metres. The study area is located within the of Carlton Creek subwatershed, which lies within the Rouge River watershed. Two tributaries to Carlton Creek, namely the east and west branches, are located within TRCA regulated areas. The east branch of Carlton Creek conveys permanent flow across the Victoria Square Boulevard right-of-way, approximately 700 m north of Woodbine Avenue south connection. The west branch of Carlton Creek conveys intermittent flow and crosses Woodbine Avenue 250 m south of the Victoria Square Boulevard and Woodbine Avenue south connection.

As per the existing roadway's vertical profile and grading information from the adjacent developments, roadway runoff south of Elgin Mills Road is conveyed southerly via roadside ditches and culverts to the two tributaries of Carlton Creek in the southern portions of the study area. North of Elgin Mills Road, roadway runoff is conveyed northerly via roadside ditches, storm sewers and stormwater management pond to Berczy Creek.

Within the study limits, there are no significant wetlands or woodlots. A **Project Key Map** is included in **Appendix A** for location reference. The **Existing Drainage Area Plan** in **Appendix B** and **Table 2-1** below provide details of the existing drainage area information along the Victoria Square Boulevard study limits.

Carlton Creek

Carlton Creek traverses the study area. The east branch of Carlton Creek is a permanent flowing stream that crosses Victoria Square Boulevard just south of Betty Roman Boulevard via a 1.25m x 1.25m concrete box culvert, then traverses southward along the east side of Victoria Square Boulevard to Major Mackenzie Drive via two online stormwater management ponds (Pond OL-E1 and Pond OL-E2), as illustrated on the Existing Drainage Area Plan (Appendix B).. There are four offline stormwater management ponds that discharge flow into the east branch of Carlton Creek; namely Pond E-1, Pond E-2, Pond E-3 and Pond E-4. Pond E-1, Pond E-2 and Pond E-3 are located in the central and southern portions of the study area. Pond E-4 is located north of the Elgin Mills Road and discharges flow southerly to the east branch of Carlton Creek via an intermittent swale and an existing culvert underneath Elgin Mills Road.

The west branch of Carlton Creek is an intermittent stream, which is located on the west side of Woodbine Avenue at the south boundary of the study limits. An offline pond (W-1) located at the south end of the study area currently discharges into the west branch of Carlton Creek. Further downstream, the two branches converge into one tributary and Carlton Creek confluences with Berczy Creek approximately 3 km south and east of the study area.

Berczy Creek

The existing roadway's vertical profile indicates a crest located approximately 480 m south of Woodbine Avenue north connection that divides the roadway drainage northerly with eventual discharge to Berczy Creek via two stormwater management ponds (Pond B-3 and B-4) that collect runoff from Victoria Square Boulevard and ultimately discharges to Berczy Creek.

Table 2-1: Drainage Area Summary

Drainage Area ID	From Station	To Station	Right-of-way	Existing Drainage Area (ha)	Receiving Drainage System
A	10+000	10+240	E & W	0.58	Roadside Ditch, Culvert C-1, SWM Pond OL-E1
B	10+240	10+700	W	1.38	Storm Sewer, SWM Pond W-1
C	10+240	10+700	E	0.79	Roadside Ditch, Pond OL-E1
D	10+700	10+900	E & W	0.70	Roadside Ditch, Carlton Creek
E	10+900	11+300	W	0.99	Roadside Ditch, SWM Pond E-3
F	11+300	11+740	W	0.55	Roadside Ditch, SWM Pond E-3
G	10+900	11+740	E	1.05	Roadside Ditch, Storm Sewer
H	11+740	12+020	E & W	2.06	Storm Sewers at Elgin Mills Road, SWM Pond E-3
I	11+930	12+355	E & W	2.46	Storm Sewers, SWM Pond E-4
J	12+020	12+560	E	0.35	Roadside Ditch/ Sewer, SWM Pond B-4
K	12+355	12+750	E & W	1.23	Storm Sewer, Woodbine Avenue, SWM Pond B-3

2.1 Transverse Water Crossings

There are a total of four (4) transverse water crossings located within the study area, which includes two transverse crossing culverts and two storm sewer crossings. For culvert crossing locations and corresponding identification numbers, refer to the **Existing Drainage Plan** provided in **Appendix B**. **Table 2-2** summarizes the existing water crossing and the physical characteristics associated with each crossing.

Table 2-2: Summary of Transverse Culvert Existing Condition

Culvert/ Crossing ID	Crossing Type	Material Type	Size (mm)	Length (m)	Flow Direction
C-1	Transverse Culvert	CSP	800	37.2	E
C-2	Transverse Culvert	Concrete	1250x1250	19.4	E
C-3	Transverse Culvert/Storm Sewer Crossing, discharges to Carlton Creek via existing storm sewer	Concrete	1200x900 and 1200 sewer	15.0 100	W S
C-4	Transverse Culvert/ Storm Sewer Crossing, the east headwall is to be removed and connected to the storm sewer trunk proposed by the future residential subdivision	Concrete	825	16.0	E

2.2 Assessment Criteria

In view of the proposed improvements, hydraulic assessments of the culverts within the Victoria Square Boulevard Class EA study area were undertaken in accordance with the Ontario Ministry of Transportation's "Highway Drainage Design Standards (2008)" and the "York Region Road Design Guidelines (2013)."

Design Flows

Based on MTO Drainage Standard WC-1, the design return period for structures crossing Freeway & Urban Arterial roadways with a span less than 6.0 metres is the 50-year return period event.

Freeboard and Clearance for Culverts on a Watercourse

The minimum required freeboard for culverts is specified to be a minimum of 1.0 metre between the design high water level to the edge of the travelled lane for an arterial road classification. The ratio of the flood depth at the upstream face of the culvert to the diameter or rise of the culvert (HW/D) is specified to be equal or less than a ratio of 1.5 for culverts with a diameter or rise less than 3.0 metres as per MTO Drainage Standard WC-7: Culvert Crossings on a Watercourse.

The minimum clearance for open-footing culverts shall be measured 0.3 metres between the design high water level and the effective rise of the culvert as per MTO Drainage Standard WC-7: Culvert Crossings on a Watercourse.

Freeboard for Culverts not on a Watercourse

There is no minimum freeboard for culverts conveying major system design flows as per MTO Drainage Standard SD-13: Design Flows and Freeboards for Culverts not on a Watercourse.

Minimum Culvert Sizes

As per the MTO Drainage Design Standards WC-8, the minimum culvert size for urban arterial road crossing culverts is 800 mm, and the minimum culvert size for rural arterial road crossing culverts is 600 mm. For any existing culverts that do not meet the minimum size requirement, culvert upgrades are required.

Storm Sewers

The conveyance capacity of the storm sewers is to be designed as per City of Markham Design Guidelines to accommodate a 5 year storm event.

2.3 Hydrologic and Hydraulic Assessment

The external drainage area information including the existing drainage pattern, storm sewers and crossing information were obtained from a review of the West Cathedral Community Stormwater Management Plan and the adjacent subdivisions drainage information. A major water crossing (C-2) that aligns with the east branch of Carlton Creek has been identified within the study area. The design peak flow was obtained from the hydraulic model provided by the TRCA.

Based on a review of the drainage information of the subdivision adjacent to Victoria Square Boulevard, existing water crossings (C-1, C-3 and C-4) were designed to accommodate the major system flow. The transverse Culvert/Storm Sewer Crossing C-3, currently conveys drainage from the east half of the Victoria Square Boulevard right-of-way, discharging to Carlton Creek via a temporary 1200mm storm sewer (Drainage Area G on Existing Drainage Area Plan). The design flow for crossing C-3 was determined using the Rational Method and the Intensity-Duration Frequency curves obtained from the City of Markham Storm Water Management Guidelines.

Excerpts of the adjacent subdivision stormwater management reports are provided in **Appendix E** to reference the peak flow to each crossing.

It is recommended that during detail design, the assessment results be reviewed and verified to confirm any changes to the land-use and associated hydrologic information that may affect the peak flow presented in this Class EA study. A summary table of the storm design peak flows of the various transverse crossings is presented in **Table 2-3.**

Table 2-3: Design Peak Flow for the Transverse Crossings.

Culvert/ Crossing ID	General Description	Catchment Area (ha)	Peak Flow (m^3/s)		
			50 Year Storm	100 Year Storm	Regional
C-1	Transverse culvert located 100m north of the Woodbine Avenue and Victoria Square Boulevard south connection	2.90[1]	0.42 (100 Year external overland flow as per adjacent subdivision drainage design)		
C-2	Transverse culvert located approximately 200m south of Betty Roman Boulevard that aligns with the east branch of Carlton Creek	100.0[2]	4.8	5.52	10.39
C-3	Storm sewer crossing located south of Church View Avenue discharging to Carlton Creek	1.05	0.23 (Major System design flow 100 year minus 5 year of the Victoria Square Boulevard East half of the Right-Of-Way)		
C-4	Transverse culvert /storm sewer crossing located approximately 50m south of Duke of Cornwall Drive	5.55[3]	1.26 (Major System design flow 100 year minus 5 year as per adjacent subdivision drainage design)		

Note 1: Drainage information obtained from As Constructed drawings of the Heritage at Cathedraltown Subdivision Design, dated August 2010.
Note 2: Drainage information obtained from Environmental & Stormwater Management Plan for West Cathedral Community, Carlton Creek West Branch, dated October 2002.
Note 3: Drainage information obtained from Stormwater Management and Pond Design Report for Monarah Corporation Transitions at Victoria Square, dated July 2007.

The hydraulic performance of culvert crossing (C-1) was assessed using the CulvertMaster software to determine the freeboard and clearance of the culvert. The storm sewer conveyance capacities of the storm sewer crossings (C-3 and C-4) were assessed using the FlowMaster model.

A hydraulic assessment of the existing water crossing (C-2) was completed using the HEC-RAS model obtained from the TRCA. Based on a review of the HEC-RAS model, input parameters and the detailed surveyed data, minor modifications to the input data file was applied in the vicinity of Culvert C-2 to reflect updated topographical information at this location. As per the MTO Highway Drainage Design Standards, existing culvert C-2 was assessed based on the 50-year design storm event to determine the freeboard at this location. In addition, the flood depth to the culvert diameter ratio is required to be less than 1.5 for culverts with a diameter less than 3.0 metres.

Table 2-4 and **Table 2-5** summarize the hydraulic assessment results of the transverse crossing culverts and the storm sewer crossings. All hydraulic assessment output files are provided in **Appendix D**.

Table 2-4: Hydraulic Analysis Results for the Transverse Culverts (Existing Condition)

Crossing ID	Size / Material (mm)	U/S Invert (m)	D/S Invert (m)	Length (m)	Road Elev. (m)	Water Surface Elevation (m)			Free-board (m)	
						50 Yr	100 Yr	Reg.	50 yr	100 yr
C-1	800 CSP	208.44	208.17	37.2	209.80	N/A	209.09	N/A	N/A	0.71
C-2	1250x1250 Concrete	213.58	213.29	19.4	216.35	216.33	216.39	216.50	0.02	-0.04

Table 2-5: Hydraulic Analysis Results for the Storm Sewer Crossings (Existing Condition)

Crossing ID	Size / Material (mm)	U/S Invert (m)	D/S Invert (m)	Length (m)	Slope (%)	Road Elev. (m)	Design Flow (m^3/s)	Available Pipe Conveyance Capacity (m^3/s)
C-3	1200x900 Box Conc. & 1200 sewer	218.48	218.08	15.0	1.7	221.2	0.23	4.38
		218.21	217.55	100.0	0.66			3.17
C-4	825 Conc.	237.09	236.93	16.0	1.0	239.3	1.26	1.54

The results presented in **Table 2-4** and **Table 2-5** indicate that Crossing C-1 does not provide the vertical freeboard criterion of a minimum 1.0 m from the high water level during the 100-Year storm event. Note that the design flow for this culvert should be the 50-year storm event; however, the design peak flow for the 50-year storm event is not available. Therefore, the vertical freeboard clearance was conservatively calculated based on the 100-year water surface elevation.

Crossing C-2 does not meet the vertical freeboard criterion of minimum 1.0 m for the design high water level (50-Year storm event), and results in flow overtopping the road. The ratio of the flood depth at the upstream face of the culvert to the diameter or rise of the culvert is 2.33, which exceeds the design standard of 1.5. Crossings C-3 and C-4 provide adequate pipe conveyance capacity to convey the major system design flows.

3 Proposed Conditions

The entire study corridor is divided into six (6) segments along Victoria Square Boulevard based on existing land use and corridor characteristics. Based on the roadway and land use characteristics of each segment, the preferred alternative design concept along Victoria Square Boulevard varies. **Table 3-1** lists the preferred roadway improvements for each segment between Woodbine Avenue (south connection) and Woodbine Avenue (north connection).

Table 3-1: Preferred Roadway Improvements

Area	Preferred Roadway Improvement
Area 1: North Gateway *(Station No: 12+600 to 12+750)*	Two travel lanes, one left turn lane, multi-use path on both west and east sides
Area 2: Residential Main Street *(Station No: 11+900 to 12+600)*	Two travel lanes, continuous centre left turn lane, multi-se path on both west and east sides, on-street parking lane on east side
Area 3: Hamlet of Victoria Square Area *(Station No: 11+230 to 11+900)*	Two travel lanes, multi-use path on both west and east sides
Area 4: Cathedral Residential Area *(Station No: 10+900 to 11+230)*	Two travel lanes, continuous centre left turn lane, multi-use path on both west and east sides, on-street parking on the west side
Area 5: Cathedral Precinct Area *(Station No: 10+180 to 10+900)*	Two travel lanes, continuous centre left turn lane, multi-use path on both west and east sides, on-street parking on the west side
Area 6: Cathedral Community Gateway Area *(Station No: 10+000 to 10+180)*	Two travel lanes, one left turn lane, multi-use path on both west and east sides, on-street parking lane on the west side

3.1 Roadway Drainage System

The rural portions of the roadway corridor will be converted to an urban cross-section that will include concrete curb and gutter and a sub-surface drainage system. Overall, the existing drainage pattern will not be altered based on the proposed roadway improvements. It is expected that the quantity of runoff from the paved section of the roadway will result only in a very minor increase in runoff, and as such, specific techniques to reduce the quantity and rate of runoff may be required depending on further analysis.

3.1.1 Minor Drainage System

There are a number of existing storm sewer systems that currently convey runoff within the right-of-way, which outlet to existing storm sewer systems. It is proposed to maintain the existing storm sewer systems within the right-of-way to the extent possible, and to relocate catchbasins and lead extensions to accommodate the road widening. The existing storm sewer information is provided on the subdivision design drawings included in **Appendix E**.

The proposed storm sewer system will provide additional drainage conveyance within Victoria Square Boulevard right-of-way between Woodbine Avenue (south connection) and Woodbine Avenue (north connection). The storm sewer system draining the pavement for the ultimate roadway configuration is to be designed for a 5-year storm event as per City of Markham Design Guidelines. Proposed roadway drainage will be collected by a series of catchbasins and conveyed by storm sewers to existing outlets.

Storm sewer configuration and calculations are to be completed during detail design as per the City's design criteria.

3.1.2 Major Drainage System

The roadway design should ensure that the major system runoff up to the 100-year storm event can be safely conveyed to the watercourse locations and should allow one lane in each direction to be clear of any flooding. Major system relief will occur at major watercourse crossings. At these locations, major system inlets will be provided to capture the 100-year flow and direct it to the outfalls.

The general direction of roadway overland flow is from north to south between Edward Roberts Drive and the south study limit boundary. Several major drainage inlet locations are provided within the right-of-way to accommodate the 100-year flow minus the 5 year flow as per the current drainage design of the residential developments adjacent to Victoria Square Boulevard. Ultimately, major system relief will discharge to the west branch of Carlton Creek.

From Edward Roberts Drive to the north study limit boundary, major system relief will occur at crossing culvert C-4 and at the northeast quadrant of Woodbine Avenue and Victoria Square Boulevard (north connection). The proposed roadway improvements will maintain the major system inlets at crossing culvert C-4, which will be connected to the storm sewer system designed by the residential developments adjacent to Victoria Square Boulevard, accommodating 100-year flow and directing the flow to Pond B-4. Overland flow will also be directed northerly to the stormwater management Pond B-3. Ultimately, the stormwater management ponds discharge flow to Berczy Creek. The major drainage systems associated with the adjacent subdivisions are provided on the subdivision design drawings included in **Appendix E**.

3.2 Transverse Water Crossings

As per **Section 2.2**, there are a total of four (4) drainage crossings located within the study limits. Upgrades to the existing water crossings are required to accommodate the proposed roadway improvements.

The proposed size, structure and location of each crossing was determined based on the existing culvert condition assessments, proposed roadway geometry, grading impacts and hydraulic performance, with the objective of improving the drainage condition at each crossing and addressing any existing deficiencies. **Table 3-2** summarizes the recommended approach at each watercourse crossing.

Table 3-2: Watercourse Crossing Recommendations

Culvert ID	Roadway Area	Recommendations for Culvert Upgrades
C-1	Area 6	**A minor culvert extension** is required to accommodate the additional grading. A headwall structure, however, will eliminate the need for a culvert extension. It is recommended to clean out the roadside ditch and the existing culvert to ensure positive drainage flow.
C-2	Area 5	**Culvert replacement** is proposed to accommodate the roadway improvements. The existing culvert is recommended to be replaced by an 8.5m span by 1.5m high con-span structure, consistent with existing water crossings that are aligned with Carlton Creek.
C-3	Area 4	**Storm sewer extension** is proposed to accommodate the roadway improvements. The temporary 1200mm sewer to be re-directed to the storm sewer on Betty Roman Boulevard. The remaining 1200mm storm sewer and headwall at the outfall to be removed
C-4	Area 2	**Culvert/storm sewer extension** is proposed to accommodate the roadway improvements and the proposed residential development. Existing 825mm diameter is proposed to be connected to a storm sewer system east of Victoria Square Boulevard, discharging to Pond B-4.

3.2.1 Hydrologic and Hydraulic Assessment for Transverse Culverts

The drainage boundary and design peak flow for the crossing culverts would remain unchanged compared to the existing conditions, since the increased pavement area within the corridor improvements is negligible in comparison to the larger external drainage areas. Refer to **Table 2-3** for the design peak flow details.

The hydraulic assessment for the proposed culvert crossings is based on the preliminary proposed horizontal road design and vertical centerline profile design. Note that the proposed inverts of the culvert crossings are to be confirmed during detail design to accommodate the road design and the roadside ditch grading.

Crossing C-1

Based on the proposed roadway improvement where Crossing C-1 is located, it is recommended to maintain the existing culvert with the addition of a headwall. As a result, a hydraulic assessment for the proposed condition is not required as the hydraulic performance is anticipated to be consistent with the existing condition. During detail design, it is recommended to review the roadway improvement alternative and grading impact to confirm the hydraulic performance.

Crossing C-2

As per the MTO Highway Drainage Design Standards, the design storm for culverts that have a span greater than 6.0 m should be the 100-year event. The minimum required freeboard is 1 metre between the road surface and the design water elevation.

The existing 1250 mm x 1250 mm box culvert has a narrow hydraulic opening that restricts the flow at the crossing location, which results in excessive backwater and higher water surface elevations upstream of the crossing. The results of the existing condition hydraulic assessment (refer to Section 2.3), found that the existing culvert provides inadequate freeboard and results in flow overtopping the roadway under Regional storm conditions. Consequently, a new replacement culvert is recommended at this location.

Based on the Carlton Creek Drainage Improvement for the West Cathedral Community, the culvert crossings that align within this reach of Carlton Creek are 8.5 m by 1.5 m open bottom concrete structures with wingwalls to reduce the grading footprint.

It is therefore recommended to replace the existing culvert at the Victoria Square Boulevard crossing C-2 with a similar 8.5m x 1.5m open footing structure, consistent with the size of the hydraulic opening of the four culvert crossings located upstream and downstream of Crossing C-2. The recommended culvert replacement will provide a larger hydraulic opening; which increases the flow width, lowers the water surface elevation and decreases the flow velocity at the crossing C-2 location. The wider culvert opening will enhance the riparian habitat conditions at the crossing location, providing a continuous naturalized corridor along the east branch of Carlton Creek. In addition, the proposed culvert replacement will eliminate the risk of flow overtopping Victoria Square Boulevard under Regional storm conditions. To account for the changes to the hydraulic regime, the existing condition HEC-RAS model was modified to account for the proposed culvert improvements at the Victoria Square Boulevard crossing. **Table 3-3** provides a summary of the hydraulic assessment. Refer to **Appendix C** for the HEC-RAS model output files. Refer to **Appendix F** for the Carlton Creek water crossing detail.

Table 3-3: Hydraulic Analysis Results - Transverse Culverts (Proposed Condition)

Crossing ID	Size / Material (m)	U/S Invert (m)	D/S Invert (m)	Length (m)	Road Elev. (m)	Water Surface Elevation (m)			Free-board (100 yr) (m)	Clearance[1] (m)
						50 Yr	100 Yr	Reg.		
C-2	8.5 x 1.5 Open footing	213.68	213.60	20.2	216.3	214.13	214.18	214.43	2.12	1.32

Note 1: The soffit elevation of the proposed open footing con-span culvert is to be confirmed during the detail design stage, it is assumed the thickness of the soffit is 0.80m.

Due to the proposed culvert replacement, there will be minor grading impacts to the upstream and downstream segments of the Carlton Creek Crossing at Victoria Square Boulevard. Opportunities for channel realignment to improve the existing aquatic habitat conditions and incorporate natural channel features within the impacted area are recommended. For the existing floodplain information provided by TRCA and the proposed floodline modifications as a result of the culvert upgrade, refer to the floodplain mapping provided in **Appendix F**. A summary table comparing the water surface elevations upstream and downstream of crossing C-2 is provided in **Table 3-4**.

Table 3-4: Floodplain Hydraulic Analysis Water Surface Elevation Summary

Crossing-Section	Existing Regional WSEL (m)	Proposed Regional WSEL (m)	Difference in WSEL (m)
7205.34	216.62	216.62	-0.21
7205.33	216.55	215.61	-1.15
7205.32	216.54	215.41	-1.13
7205.31	216.53	214.83	-1.70
7205.30	216.53	214.43	-2.10
Crossing C-2 at Victoria Square Boulevard			
7205.29	214.92	214.08	-0.84
7205.28	213.73	213.73	-
7205.27	213.61	213.61	-
7205.26	213.61	213.61	-
7205.255	213.61	213.61	-
7205.254	213.55	213.55	-
Culvert (On-line Pond E2)			
7205.252	212.87	212.87	-
7205.251	212.94	212.94	-
7205.25	212.82	212.82	-
Crossing at Vine Cliff Boulevard			
7205.24	212.19	212.19	-

It is anticipated that the amount of fill within the floodplain will be minimal. An incremental cut fill analysis should be undertaken during detail design to ensure the grading modification does not cause a negative impact to the floodplain. The construction of the culvert replacement shall respect the appropriate timing window to minimize impacts to in-stream function and fish habitat.

Crossing C-3

Crossing C-3 is a 1200mm x 900mm concrete box sewer that capture the drainage from the east half of Victoria Square Boulevard right-of-way, and connects to the existing temporary 1200mm storm sewer system that discharges to Carlton Creek approximately 200m south of Betty Roman Boulevard.

As part of the Cathedral Town Subdivision development, the storm sewer on Betty Roman Boulevard was designed to accommodate the drainage from the Victoria Square Boulevard right-of-way. Therefore, it is recommended to redirect the existing 1200mm storm sewer to the storm sewer on Betty Roman Boulevard. The remaining portion of the 1200mm storm sewer and outlet headwall will be removed.

The hydraulic performance of the downstream storm sewer is anticipated to be consistent with existing conditions upon completion of the roadway improvements since the sewers were designed to account for the drainage from Victoria Square Boulevard. For the hydraulic assessment and storm sewer crossing C-3 details, refer to **Table 2-5**.

Crossing C-4

Crossing C-4 is currently a culvert that conveys external drainage west of Victoria Square Boulevard. As part of the future subdivision development, the storm sewer system has been designed to accommodate the future roadway widening. The existing headwall of the culvert crossing will be removed. A manhole will be placed within the right-of-way to connect the crossing to a storm sewer system which conveys drainage to a proposed stormwater management pond (B-4) that ultimately discharges to Berczy Creek. For the hydraulic assessment of the storm sewer system, refer to the Eaton Square Development Residential subdivision drainage design.

4 Stormwater Management Strategy

4.1 Stormwater Management Criteria

The stormwater management plan will be designed to comply with the MOECC Stormwater Management Planning and Design Manual and the TRCA Stormwater Management Criteria as the following:

- Water quality control: Enhanced protection (level 1) for the increased pavement area project-wide;
- Water quantity control: Control post- development peak flows to pre-development levels for all storms up to and including the 100 year storm (i.e., 2, 5, 10, 25, 50, 100 year storms);
- Erosion Control: 25mm – 48 hour detention for drainage areas with SWM ponds; 5mm retention in other areas
- Water Balance: 5mm retention on site

4.2 Pavement Area Analysis

A pavement area analysis was performed to determine the increase in the impervious surface, including the road widening pavement area, multi-use paths within the right-of-way, as a result of the roadway improvements. It was determined that the proposed roadway improvements will result in an increase of 1.55 ha for the entire roadway corridor within the study area.

Table 4-1: Pavement Area Analysis

Roadway Condition	Area (ha)
Existing Pavement Area within ROW	4.08
Proposed Pavement Area	5.63
Pavement Area Increase	1.55

4.3 Stormwater Best Management Practice Options

The proposed stormwater management plan for the study area has been developed by examining the existing on-site stormwater management as well as opportunities and constraints within the entire project area. Runoff from the paved roadway areas will be conveyed to the existing and proposed roadway storm sewer systems and discharge into the storm sewer system that ultimately drains to stormwater management facilities or directly to natural watercourses.

Due to limited space within the roadway right-of-way of this linear transportation corridor and the presence of the existing stormwater management ponds, it is recommended to maintain the existing drainage pattern and to continue to utilize the existing stormwater management ponds for water quality and quantity controls.

4.3.1 Existing Stormwater Management Facilities (Wet Ponds)

A stormwater management pond operates on the basis of temporary storage (detention) of runoff to promote the removal of pollutants through sedimentation. They are generally effective in removing particulate constituents, such as sediments and metals, but ineffective at removing dissolved constituents such as salt. Extended detention wet ponds are considered to be effective at achieving an enhanced level of treatment for roadway runoff. A stormwater management pond can also provide quantity control to the discharge flow at a regulated rate to match the existing flow rate to ensure that the existing hydraulic condition will be maintained.

Due to the proximity of Victoria Square Boulevard right-of-way to the existing stormwater management ponds, a total existing drainage area of 5.87 ha from the Victoria Square Boulevard right-of-way (2.90 ha Pavement area) currently drains to existing stormwater management ponds, including Pond W-1, Pond E-3, Pond E-4, Pond B-4 and Pond B-3. These ponds currently serve as combined water quality and quantity facilities that provide stormwater treatment to the residential developments adjacent to the study corridor as well as portions of the Victoria Square right-of-way. Stormwater Management Pond OL-E1 is an on-line stormwater management pond that do not provide water quality control.

Based on the proposed roadway widening improvements, a minimum of 4.05 ha of pavement area (sum of additional pavement area of 1.55 ha and the existing treated pavement area of 2.50 ha) will require stormwater quality control. **Table 4-2** summarizes the existing drainage and pavement area that currently contribute to the existing stormwater management ponds. For locations of the SWM facilities, refer to the **Project Key Map** included in **Appendix A**. Drainage Plans corresponding to the adjacent subdivisions are included in **Appendix E**.

Table 4-2: Summary of Existing Drainage Areas from the Victoria Square Boulevard Right-of-Way Contributing to Stormwater Management Ponds

Stormwater Management Pond	Location	Drainage Area ID	Existing ROW Drainage Area to Pond (ha)	Existing ROW Pavement Area to Pond (ha)
W-1	Southwest quadrant of Victoria Square Boulevard and Woodbine Avenue (South Connection)	B	0.79	0.38
E-3	Southwest quadrant of Victoria Square Boulevard and Betty Roman Boulevard	E, F, H	1.66	0.77
E-4	Southwest quadrant of Woodbine Avenue and Elgin Mills Road E	I	0.51	0.26
B-4	200m east of the intersection of Victoria Square Boulevard and Bruce Thomson Drive	J	1.10	0.60
B-3	Northwest corner of Victoria Square Boulevard and Woodbine Avenue (North Connection)	K	1.23	0.48
Totals			5.29	2.50

Stormwater Management Pond W-1

Pond W-1 is located southwest of the intersection of Victoria Square Boulevard and Woodbine Avenue (south connection) and currently provides stormwater quality and quantity control to a total drainage area of approximately 100.10 ha, including a drainage area of 0.79 ha (0.38 ha pavement area) from the Victoria Square Boulevard corridor. As part of the Majorwood Developments, Pond W-1 has been sized to accommodate the drainage from the Victoria Square Boulevard right-of-way between Donald Buttress Boulevard East and Murison Drive. Refer to the Majorwood Developments Inc. Lakeview Homes Inc. External Drainage Plan in **Appendix E** for details.

The proposed stormwater management strategy recommends to convey a total of 1.58 ha (1.05 ha pavement area) of Victoria Square Boulevard ROW area (Drainage Area B & C) to Pond W-1 via the existing storm sewer system, consistent with the Pond W-1 drainage area plan. Overall, the increased pavement area will result in a net increase in drainage area of 0.54 ha (0.38 ha pavement area), or approximately a 0.5 % drainage area increase to the overall contributing area to Pond W-1. To assess the applicability of the proposed SWM strategy, Pond W-1 and the associated storm sewer network should be further investigated during detail design to ensure that the stormwater objectives are effectively addressed.

Stormwater Management Pond E-3

Pond E-3 is located at the southwest corner of Victoria Square Boulevard and Betty Roman Boulevard and currently provides stormwater quality and quantity control to a total drainage area of approximately 80.50 ha.

Under existing conditions, a drainage area (Drainage Area E, F & H) of 1.66 ha (0.77 ha pavement area) from the Victoria Square Boulevard corridor contributes runoff to Pond E-3 via the existing storm sewer system. The proposed stormwater management strategy recommends to convey Victoria Square Boulevard ROW (Drainage Area E, F, G, H) totalling 2.71 ha (1.88 ha pavement area) to Pond E-3. The net increase in pavement area of 1.11 ha (1.88 ha-0.77 ha) will result in approximately a 0.5% increase in the overall imperviousness ratio to Pond E-3. Further investigations and analysis should be undertaken during detail design to confirm the hydraulic performance of Pond E-3 based on the proposed drainage strategy.

Stormwater Management Pond E-4

Pond E-4 is located at the northwest corner of Woodbine Avenue and Elgin Mills Road. Currently, Pond E-4 provides stormwater quality and quantity control to a drainage area of 54.34 ha, which includes a drainage area of 0.51 ha (0.56 ha pavement area) from the Victoria Square Boulevard right-of-way corridor.

The west side of the Victoria Square Boulevard roadway corridor will continue to be conveyed to Pond E-4. The proposed stormwater management strategy recommends to convey Victoria Square Boulevard ROW (Drainage Area I), totalling 0.51 ha (0.38 ha pavement area) to Pond E-4, consistent with the current drainage pattern. The proposed stormwater strategy is consistent with the drainage plan of the future residential development (Eaton Square) located east of Victoria Square Boulevard and North of Elgin Mills Road. For the proposed drainage area information, refer to the **Proposed Drainage Area Plan** in **Appendix C**.

Stormwater Management Pond B-4

Pond B-4 is proposed to be located approximately 200 m east of the intersection of Victoria Square Boulevard and Bruce Thomson Drive, providing stormwater quality and quantity control to a drainage area of approximately 21.0 ha. As part of the Eaton Square subdivision drainage design, the Victoria Square Boulevard future road widening was taken into consideration; therefore, the storm sewer system and Pond B-4 have been designed to accommodate a drainage area of approximately 1.10 ha (0.54 ha pavement area) from the east side of the roadway corridor. The proposed stormwater management strategy recommends to convey a total of 1.10 ha pavement area (0.71 ha pavement area) to Pond B-4. The net increase in pavement area of 0.11 ha (0.71 ha - 0.60 ha) will result in approximately a 0.5% increase in the overall imperviousness ratio to Pond B-4. Ultimately, Pond B-4 will discharge to the west tributary of Berczy Creek. For details of the Pond B-4 design, refer to the stormwater management report prepared for the Eaton Square Development. For the proposed drainage area information, refer to the **Proposed Drainage Area Plan** in **Appendix C**.

Stormwater Management Pond B-3

Pond B-3 is located at the northwest corner of Victoria Square Boulevard and Woodbine Avenue (North connection), providing stormwater quality and quantity control to a drainage area of 66.0 ha including a drainage area of 1.23 ha (0.48 ha pavement area) from the Victoria Square Boulevard right-of-way. The proposed stormwater strategy recommends maintaining the existing drainage pattern. The future roadway widening will result in a net increase of 0.14 ha (0.62 ha - 0.48 ha) in the pavement area, which is approximately 0.2 % increase of the overall imperviousness contributing to Pond B-3.

Ultimately, Pond B-3 discharges to the west tributary of Berczy Creek. Further investigations and analysis should be undertaken during detail design to confirm the performance of Pond B-3 based on the proposed drainage strategy.

Table 4-3 summarizes the drainage and pavement areas that are proposed to contribute to the existing stormwater management ponds.

Table 4-3: Summary of Proposed Drainage Areas from the Victoria Square Boulevard Right-of-Way Contributing to Stormwater Management Ponds

Stormwater Management Pond	Location	Drainage Area ID	Proposed ROW Drainage Area to Pond (ha)	Proposed Pavement Area to Pond (ha)
W-1	Southwest quadrant of Victoria Square Boulevard and Woodbine Avenue (South Connection)	B, C	1.58	1.05
E-3	Southwest quadrant of Victoria Square Boulevard and Betty Roman Boulevard	E, F, G, H	2.71	1.88
E-4	Southwest quadrant of Woodbine Avenue and Elgin Mills Road E	I	0.51	0.38
B-4	200m east of the intersection of Victoria Square Boulevard and Bruce Thomson Drive	J	1.10	0.71
B-3	northwest corner of Victoria Square Boulevard and Woodbine Avenue (North Connection)	K	1.23	0.62
Totals			**7.13**	**4.64**

All information relating to the design of the existing SWM facilities was obtained from the stormwater management reports supplied by the City of Markham. Overall, the existing drainage pattern will not be altered based on the proposed roadway improvements. It is expected that the quantity of runoff from the additional paved section of the roadway will result only in a very minor increase in the runoff and that sufficient storage capacity is available in each SWM Pond to accommodate the minor increase in runoff volume.

A summary table of the permanent pool and extended detention volume requirement is presented in **Table 4-4**. The analysis carried out below indicates that the stormwater management ponds have sufficient capacity to provide water quality control to the additional pavement area from the Right-Of-Way. Note that during the detailed design phase of the project, the stormwater management pond requirements shall be reviewed and to confirmed.

Table 4-4: Stormwater Management Pond Permanent Pool and Extended Detention Volume Summary

Stormwater Management Pond	Available Permanent Pool Storage Volume (m^3)	Available Extended Detention Volume (m^3)	Permanent Pool Volume Requirement (m^3)			Extended Detention Volume Requirement (m^3)		
			Original Req'd Volume	ROW Req'd Volume (Note 2)	Total Req'd Volume	Original Req'd Volume	ROW Req'd Volume (Note 2)	Total Req'd Volume
W-1	22,183	22,679	18,872	141	19,013	22,000	27	22,027
E-3	17,076	23,464	6,750	233	6,983	8,975	44	9,019
E-4	9,185	9,010	8,528	25	8,553	8,368	5	8,373
B-4 (Note 1)	n/a	n/a	n/a	36	n/a	n/a	7	n/a
B-3	12,800	10,667	12,275	29	12,304	7,451	6	7,457

Notes:

1. No information is available at the time when this report was prepared. Note that as part of the Subdivision design, the stormwater management pond was designed to accommodate the ROW drainage from Victoria Square Boulevard.

2. Volume obtained from Table 3.2 from MOE Stormwater Management Planning and Design Manual (2003) based on 85% Imperviousness level. Unit volume determined to be 210 m^3/ha for permanent pool and 40 m^3/ha for extended detention.

4.4 Low Impact Development Opportunities

Low impact development stormwater practices shall be explored based on physical site conditions and further geotechnical and hydrogeological investigations during the next phase of design. Efforts are to be made to examine opportunities to provide at-source and conveyance controls that would encourage groundwater recharge, thereby reducing stormwater runoff volumes and pollutants prior to discharging to receiving systems.

There exists an opportunity to implement an LID feature at a proposed storm outfall located at the Carlton Creek (Crossing C-2) location, where drainage area D (a pavement area of 0.45 ha) is proposed to discharge directly to Carlton Creek. This is a suitable location for LID implementation as there is ample space available at the outfall location to implement stormwater management features. As per the geotechnical investigation, silty clay and sand deposit was encountered within the borehole investigation in the area adjacent to Carlton Creek. As such, the soil appears to provide suitable conditions for LID feature implementation. The groundwater table is approximately at a depth of 5.79m (Elevation 210.70 m) within the study area, which also would allow for the implementation of LID features.

The proposed storm system can be implemented with an oil grit separator and an infiltration gallery at the outfall location to provide overall water quality control and runoff volume reduction. The geometry layout of the infiltration gallery shall be designed as per the Low Impact Development Stormwater Management Planning Design Guide. Details of the storm and LID system should be explored further during detail design. Refer to the **Proposed Drainage Plan** in **Appendix C** for conceptual location.

4.5 Stormwater Management Plan Summary

The recommended stormwater management strategy will provide water quality treatment to a total pavement area of 5.09 ha, which exceeds the minimum required pavement area to be treated of 4.05 ha. **Table 4-5** provides a summary of the stormwater management plan and overall pavement treatment.

Table 4-5: Stormwater Management Summary

Roadway Condition	Area (ha)
Pavement Area Increase	1.55
Pavement Area Treated by SWM Ponds (Existing Condition)	2.50
Minimum Required Pavement Area to be treated	4.05
Pavement Area To Be Treated by SWM Ponds (Proposed Condition)	4.64
Pavement Area To Be Treated by LID (Proposed Condition)	0.45
Total Pavement Area Provided with Water Quality Control	5.09

5 Conclusions

The Victoria Square Boulevard corridor from Woodbine Avenue (south connection) to Woodbine Avenue (north connection) is proposed to be widened to accommodate the roadway improvements consisting of an urban roadway cross-section. Storm sewers shall be sized to accommodate a 5-year storm event as per City of Markham Design Guidelines. A series of catchbasins and storm sewers are proposed to collect and convey storm runoff and discharge to existing storm drainage systems and outfall locations that maintain the existing drainage patterns.

The proposed improvements to the Victoria Square corridor will result in replacing one culvert at the Carlton Creek crossing (Crossing C-2) that is located approximately 200 m south of Betty Roman Boulevard. Crossing C-1 is recommended to be maintained and cleaned out to ensure positive drainage flow. Crossing C-3 and C-4, which are form part of the existing storm sewer network, are to be extended to accommodate the roadway improvements. Crossing C-3 is to be connected to the subdivision storm sewer system to accommodate the roadway improvements, and the existing outfall to Carlton Creek is to be removed. Crossing C-4 is to be connected to the subdivision storm sewer system to accommodate the roadway improvements.

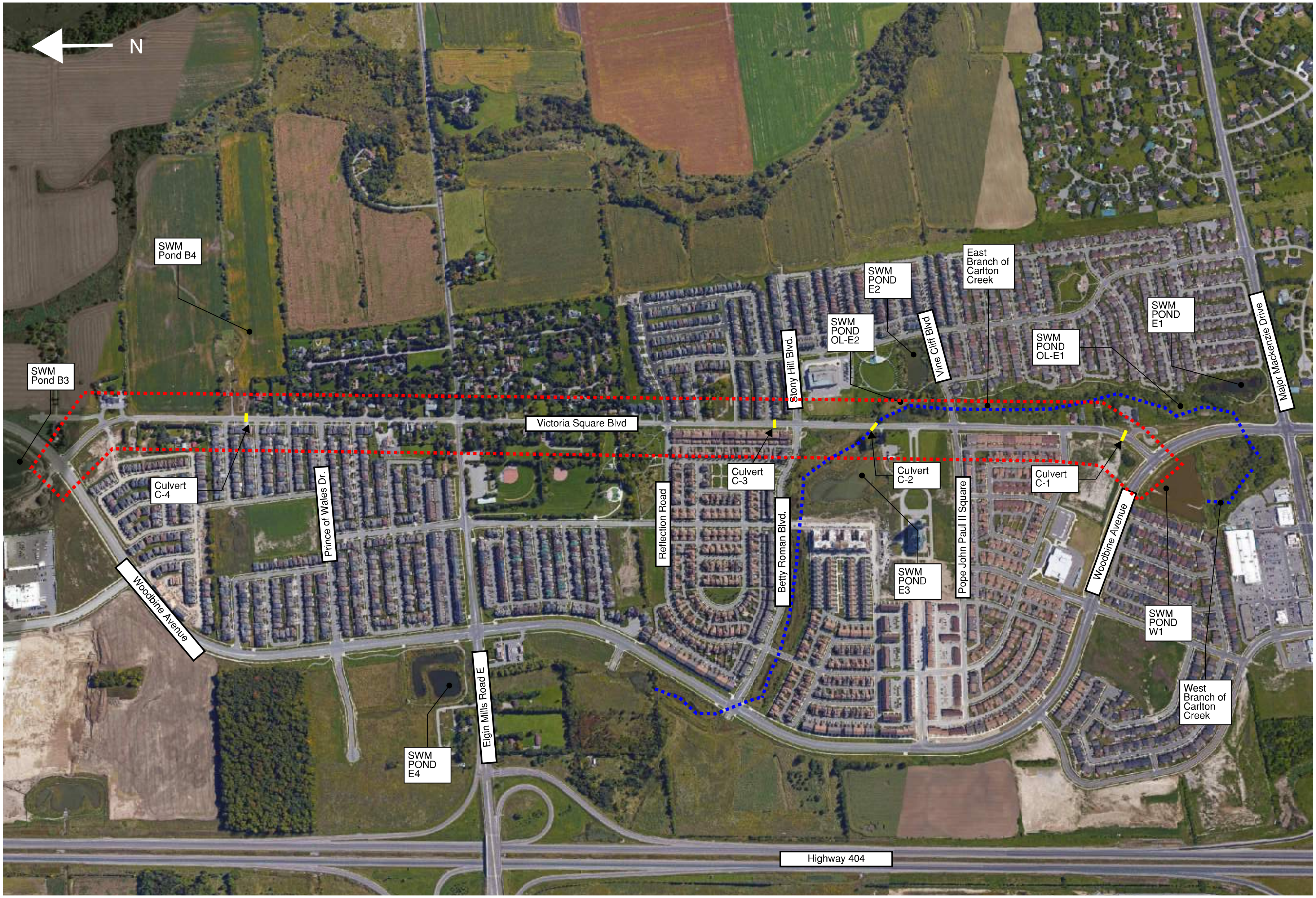
The proposed stormwater management strategy will provide quality and quantity control in accordance with the design criteria outlined in the MOECC Stormwater Management Planning and Design Manual, City of Markham Design Guidelines and TRCA Stormwater Management Criteria.

The proposed road widening will result in an additional pavement area of 1.55 hectares. Presently, stormwater management ponds (Pond W-1, E-3, E-4, B-4 and B-3) located in the vicinity of the roadway corridor within existing residential subdivisions, currently provides quality treatment and peak flow control to the Victoria Square Boulevard pavement area of 2.50 hectares. Consequently, with the proposed roadway improvements, stormwater quality treatment will be required for a minimum 4.05 ha pavement area. The proposed stormwater management strategy will maintain the existing drainage pattern and provide quality and quantity control for 4.64 hectares of pavement area via the existing five stormwater management ponds, including a proposed storm sewer system integrated with an oil grit separator and an infiltration gallery with discharge to Carlton Creek south of Betty Roman Boulevard.

To assess the applicability of the proposed SWM strategy, the existing stormwater management ponds are to be further investigated during detail design to ensure that the stormwater objectives are effectively addressed. Additional low impact development stormwater best management practices are to be further explored during detail design, based on physical site applicability and soil/ground water conditions.

A

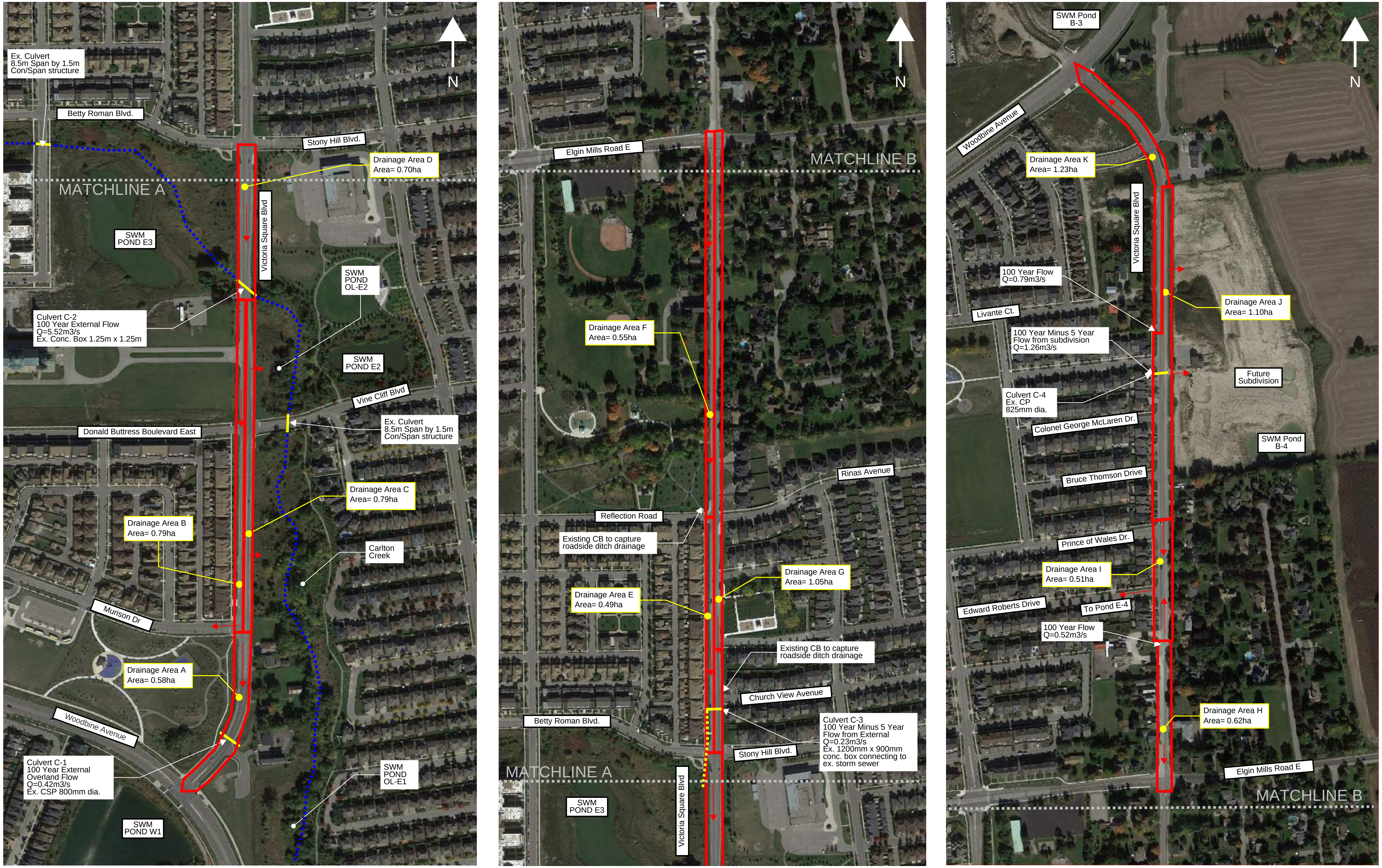
Project Key Map



Project Key Map
Victoria Square Boulevard Class EA
Woodbine Avenue (North Intersection) to Woodbine Avenue (South Intersection)

B

Existing Drainage Area Plan



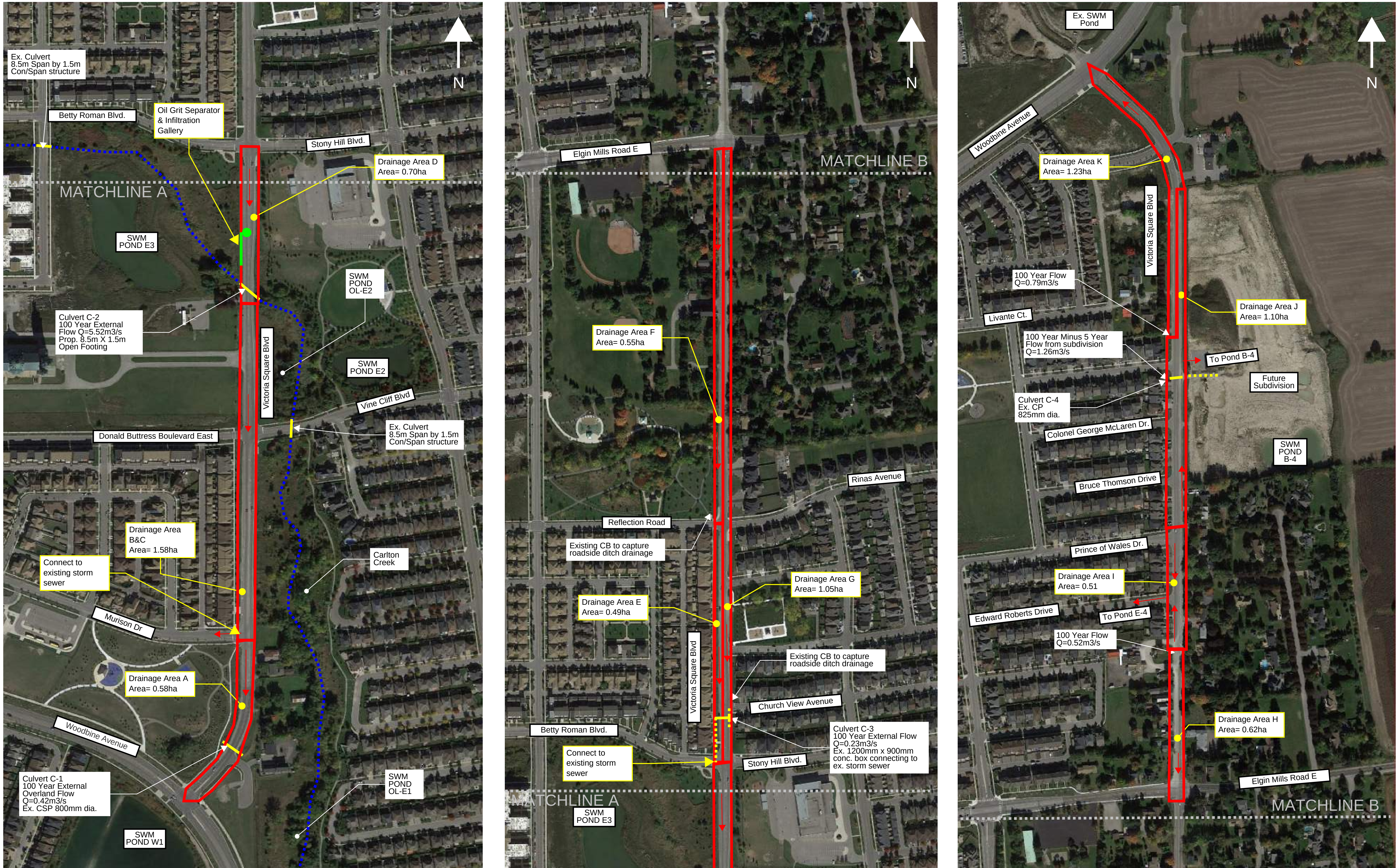
Existing Drainage Area Plan
Victoria Square Boulevard Class EA
Woodbine Avenue (North Intersection) to Woodbine Avenue (South Intersection)

Carlton Creek
Existing External Conveyance Storm Sewers
Drainage Boundary

HDR

C

Proposed Drainage Area Plan



Carlton Creek

Existing/ Future External Conveyance Storm Sewers

Drainage Boundary

Proposed Drainage Area Plan
Victoria Square Boulevard Class EA
Woodbine Avenue (North Intersection) to Woodbine Avenue (South Intersection)

HDR

D

Hydraulic Assessment Output

Culvert Calculator Report
Culvert C1

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	209.80	m	Headwater Depth/Height	0.82	
Computed Headwater Eleva	209.09	m	Discharge	0.4200	m³/s
Inlet Control HW Elev.	209.04	m	Tailwater Elevation	208.56	m
Outlet Control HW Elev.	209.09	m	Control Type	Outlet Control	

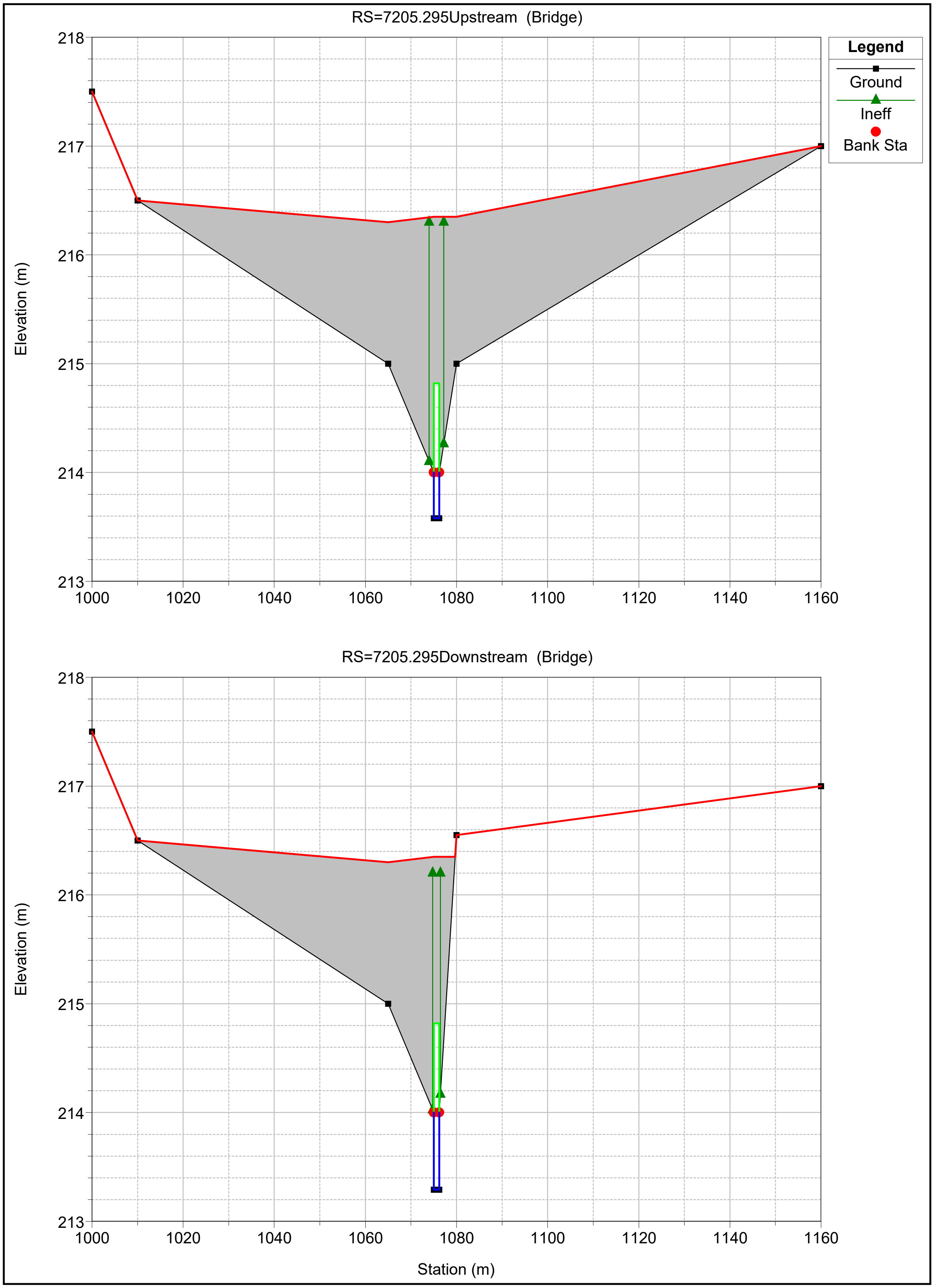
Grades					
Upstream Invert	208.44	m	Downstream Invert	208.17	m
Length	37.20	m	Constructed Slope	0.007258	m/m

Hydraulic Profile					
Profile	M2		Depth, Downstream	0.39	m
Slope Type	Mild		Normal Depth	0.49	m
Flow Regime	Subcritical		Critical Depth	0.39	m
Velocity Downstream	1.73	m/s	Critical Slope	0.015014	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	0.80	m
Section Size	800mm		Rise	0.80	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	209.09	m	Upstream Velocity Head	0.09	m
Ke	0.90		Entrance Loss	0.08	m

Inlet Control Properties					
Inlet Control HW Elev.	209.04	m	Flow Control	N/A	
Inlet Type	Projecting		Area Full	0.5	m²
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				


RS=7205.295Upstream (Bridge)
Legend
Ground
Ineff
Bank Sta
Elevation (m)
218
217
216
215
214
213
1000
1020
1040
1060
1080
1100
1120
1140
1160
RS=7205.295Downstream (Bridge)
Elevation (m)
218
217
216
215
214
213
1000
1020
1040
1060
1080
1100
1120
1140
1160
Station (m)

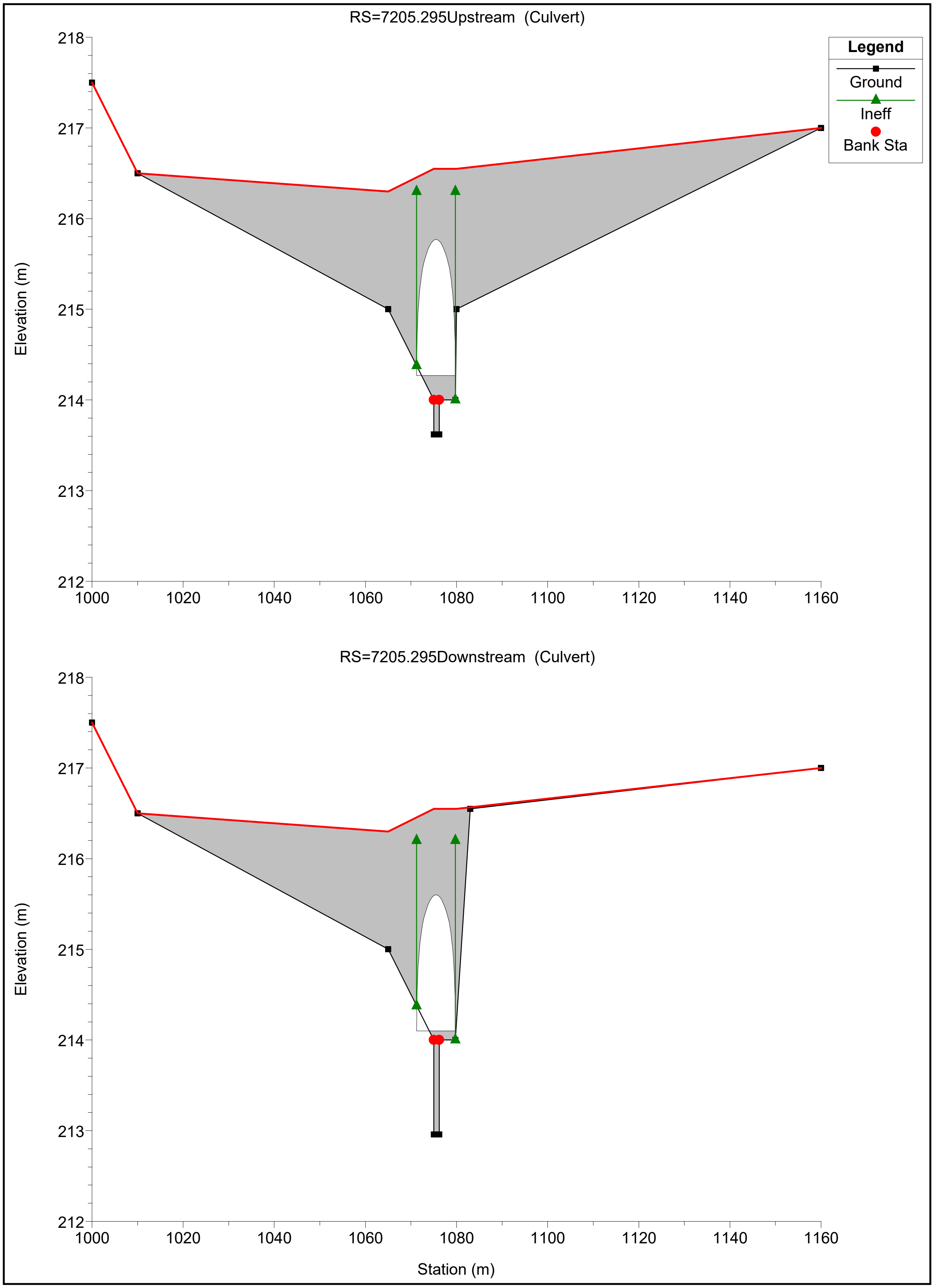
HEC-RAS Plan: Plan 01 River: Carleton Creek Reach: Reach 2

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 2	7205.38	Regional	8.59	218.50	219.675	219.67	219.91	0.010551	2.56	7.15	18.98	0.76
Reach 2	7205.38	100 YR	4.56	218.50	219.383	219.38	219.61	0.012230	2.27	3.18	9.66	0.78
Reach 2	7205.38	50 YR	3.97	218.50	219.322	219.32	219.55	0.012906	2.22	2.63	8.44	0.79
Reach 2	7205.38	25 YR	3.40	218.50	219.251	219.25	219.48	0.014133	2.18	2.08	7.02	0.82
Reach 2	7205.38	10 YR	2.65	218.50	219.174	219.12	219.37	0.013436	1.97	1.60	5.49	0.78
Reach 2	7205.38	5 YR	2.09	218.50	219.121	219.00	219.27	0.011559	1.73	1.34	4.41	0.72
Reach 2	7205.38	2 YR	1.31	218.50	218.964		219.08	0.012500	1.49	0.88	1.99	0.72
Reach 2	7205.37	Regional	8.59	218.00	219.389		219.49	0.004145	1.80	10.63	19.78	0.49
Reach 2	7205.37	100 YR	4.56	218.00	219.065		219.17	0.004829	1.62	5.27	13.30	0.51
Reach 2	7205.37	50 YR	3.97	218.00	219.004		219.10	0.004952	1.58	4.49	12.07	0.51
Reach 2	7205.37	25 YR	3.40	218.00	218.939		219.04	0.005046	1.52	3.76	10.78	0.51
Reach 2	7205.37	10 YR	2.65	218.00	218.826		218.92	0.005630	1.47	2.66	8.51	0.52
Reach 2	7205.37	5 YR	2.09	218.00	218.697	218.50	218.81	0.007300	1.49	1.73	5.94	0.58
Reach 2	7205.37	2 YR	1.31	218.00	218.576		218.65	0.006010	1.18	1.16	3.51	0.51
Reach 2	7205.36	Regional	8.59	217.50	218.771	218.77	219.09	0.011185	2.78	5.22	10.76	0.80
Reach 2	7205.36	100 YR	4.56	217.50	218.373	218.37	218.68	0.015427	2.53	2.25	4.98	0.88
Reach 2	7205.36	50 YR	3.97	217.50	218.298	218.30	218.60	0.016626	2.47	1.90	4.38	0.90
Reach 2	7205.36	25 YR	3.40	217.50	218.218	218.22	218.51	0.018224	2.40	1.58	3.74	0.92
Reach 2	7205.36	10 YR	2.65	217.50	218.139	218.10	218.37	0.017072	2.14	1.30	3.11	0.87
Reach 2	7205.36	5 YR	2.09	217.50	218.132		218.28	0.011018	1.71	1.28	3.06	0.70
Reach 2	7205.36	2 YR	1.31	217.50	217.907	217.87	218.06	0.018630	1.71	0.77	1.96	0.88
Reach 2	7205.35	Regional	9.45	216.50	217.577	217.58	217.76	0.011211	2.49	9.43	27.49	0.77
Reach 2	7205.35	100 YR	5.02	216.50	217.389	217.39	217.55	0.010279	2.09	5.05	19.10	0.72
Reach 2	7205.35	50 YR	4.37	216.50	217.350	217.35	217.51	0.010114	2.01	4.34	17.38	0.71
Reach 2	7205.35	25 YR	3.74	216.50	217.295	217.30	217.46	0.010748	1.98	3.46	14.98	0.72
Reach 2	7205.35	10 YR	2.92	216.50	217.211	217.21	217.38	0.011686	1.91	2.36	11.30	0.74
Reach 2	7205.35	5 YR	2.30	216.50	217.076	217.08	217.29	0.018290	2.07	1.23	5.35	0.89
Reach 2	7205.35	2 YR	1.44	216.50	217.028	216.89	217.13	0.009918	1.43	1.02	3.23	0.64
Reach 2	7205.34	Regional	9.45	215.50	216.623	216.62	216.83	0.011262	2.57	8.47	22.14	0.78
Reach 2	7205.34	100 YR	5.02	215.50	216.406	216.41	216.59	0.010829	2.17	4.42	15.12	0.74
Reach 2	7205.34	50 YR	4.37	215.50	216.360	216.36	216.54	0.010862	2.10	3.76	13.63	0.73
Reach 2	7205.34	25 YR	3.74	215.50	216.303	216.30	216.49	0.011306	2.04	3.04	11.80	0.74
Reach 2	7205.34	10 YR	2.92	215.50	216.199	216.20	216.39	0.013375	2.02	1.99	8.43	0.79
Reach 2	7205.34	5 YR	2.30	215.50	216.170	216.06	216.31	0.010017	1.70	1.75	7.49	0.67
Reach 2	7205.34	2 YR	1.44	215.50	215.897	215.90	216.09	0.024222	1.93	0.75	1.96	1.00
Reach 2	7205.33	Regional	9.45	214.50	216.517		216.53	0.000374	0.70	34.60	39.42	0.16
Reach 2	7205.33	100 YR	5.02	214.50	216.396		216.40	0.000149	0.42	30.02	36.39	0.10
Reach 2	7205.33	50 YR	4.37	214.50	216.334		216.34	0.000135	0.39	27.83	34.86	0.09
Reach 2	7205.33	25 YR	3.74	214.50	215.803		215.82	0.000673	0.69	12.65	23.55	0.20
Reach 2	7205.33	10 YR	2.92	214.50	215.289	215.20	215.41	0.007441	1.64	2.94	11.81	0.60
Reach 2	7205.33	5 YR	2.30	214.50	215.093	215.06	215.29	0.016453	2.00	1.28	5.15	0.85
Reach 2	7205.33	2 YR	1.44	214.50	215.107		215.18	0.005890	1.22	1.36	5.62	0.51
Reach 2	7205.32	Regional	10.39	214.00	216.513		216.51	0.000064	0.33	83.02	67.89	0.07
Reach 2	7205.32	100 YR	5.52	214.00	216.394		216.39	0.000023	0.20	75.13	65.23	0.04
Reach 2	7205.32	50 YR	4.80	214.00	216.333		216.33	0.000020	0.18	71.18	63.86	0.04
Reach 2	7205.32	25 YR	4.11	214.00	215.800		215.80	0.000068	0.28	40.31	51.92	0.07
Reach 2	7205.32	10 YR	3.21	214.00	215.297		215.30	0.000341	0.49	17.36	37.40	0.14
Reach 2	7205.32	5 YR	2.53	214.00	215.086		215.10	0.000690	0.62	10.31	29.28	0.19
Reach 2	7205.32	2 YR	1.59	214.00	214.632	214.42	214.71	0.005934	1.26	1.63	8.35	0.51
Reach 2	7205.31	Regional	10.39	213.50	216.501		216.51	0.000168	0.61	57.19	74.95	0.11
Reach 2	7205.31	100 YR	5.52	213.50	216.390		216.39	0.000063	0.36	49.37	65.56	0.07
Reach 2	7205.31	50 YR	4.80	213.50	216.329		216.33	0.000055	0.33	45.54	60.42	0.06
Reach 2	7205.31	25 YR	4.11	213.50	215.791		215.80	0.000117	0.42	23.88	28.28	0.09
Reach 2	7205.31	10 YR	3.21	213.50	215.274		215.28	0.000293	0.56	12.01	17.64	0.14
Reach 2	7205.31	5 YR	2.53	213.50	215.059		215.07	0.000343	0.56	8.69	13.20	0.14
Reach 2	7205.31	2 YR	1.59	213.50	214.578		214.60	0.000737	0.64	3.78	7.78	0.20
Reach 2	7205.3	Regional	10.39	213.58	216.503	215.08	216.50	0.000050	0.29	117.69	130.14	0.05
Reach 2	7205.3	100 YR	5.52	213.58	216.391	214.68	216.39	0.000020	0.18	103.57	121.61	0.03
Reach 2	7205.3	50 YR	4.80	213.58	216.330	214.62	216.33	0.000018	0.17	96.31	116.94	0.03
Reach 2	7205.3	25 YR	4.11	213.58	215.749	214.55	215.78	0.000801	0.96	5.91	72.42	0.21
Reach 2	7205.3	10 YR	3.21	213.58	215.222	214.45	215.26	0.001409	1.05	4.23	32.01	0.26
Reach 2	7205.3	5 YR	2.53	213.58	215.011	214.36	215.05	0.001498	0.99	3.55	15.83	0.26
Reach 2	7205.3	2 YR	1.59	213.58	214.498	214.19	214.55	0.003620	1.14	1.91	8.07	0.38
Reach 2	7205.295		Bridge									
Reach 2	7205.29	Regional	10.39	213.29	215.160	215.16	215.96	0.025652	4.14	2.79	18.81	0.97
Reach 2	7205.29	100 YR	5.52	213.29	214.584	214.58	215.12	0.027462	3.35	1.81	7.91	0.94
Reach 2	7205.29	50 YR	4.80	213.29	214.483	214.48	214.98	0.027890	3.19	1.64	6.75	0.94

HEC-RAS Plan: Plan 01 River: Carleton Creek Reach: Reach 2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 2	7205.29	25 YR	4.11	213.29	214.384	214.38	214.83	0.028030	3.02	1.47	5.61	0.92
Reach 2	7205.29	10 YR	3.21	213.29	214.224	214.22	214.62	0.030366	2.83	1.20	3.77	0.94
Reach 2	7205.29	5 YR	2.53	213.29	214.081	214.08	214.45	0.034006	2.68	0.96	2.13	0.96
Reach 2	7205.29	2 YR	1.59	213.29	213.859	213.86	214.14	0.035035	2.35	0.68	1.20	1.00
Reach 2	7205.28	Regional	11.43	213.00	213.727		213.75	0.001045	0.72	20.62	48.77	0.28
Reach 2	7205.28	100 YR	6.07	213.00	213.521		213.54	0.001094	0.58	11.99	34.93	0.27
Reach 2	7205.28	50 YR	5.28	213.00	213.493		213.51	0.001023	0.53	11.04	33.18	0.26
Reach 2	7205.28	25 YR	4.53	213.00	213.461		213.47	0.000977	0.50	10.00	31.64	0.25
Reach 2	7205.28	10 YR	3.53	213.00	213.417		213.43	0.000879	0.44	8.64	29.51	0.23
Reach 2	7205.28	5 YR	2.78	213.00	213.373		213.38	0.000835	0.39	7.41	27.43	0.22
Reach 2	7205.28	2 YR	1.74	213.00	213.305		213.31	0.000725	0.31	5.64	24.13	0.20
Reach 2	7205.27	Regional	11.43	213.00	213.611		213.65	0.002069	0.89	15.61	41.72	0.38
Reach 2	7205.27	100 YR	6.07	213.00	213.323		213.38	0.006978	1.01	6.11	25.32	0.61
Reach 2	7205.27	50 YR	5.28	213.00	213.307	213.23	213.35	0.006515	0.94	5.69	24.45	0.59
Reach 2	7205.27	25 YR	4.53	213.00	213.295	213.20	213.33	0.005568	0.84	5.41	23.86	0.54
Reach 2	7205.27	10 YR	3.53	213.00	213.267	213.18	213.29	0.005091	0.74	4.75	22.37	0.51
Reach 2	7205.27	5 YR	2.78	213.00	213.242	213.17	213.26	0.004625	0.66	4.21	21.24	0.47
Reach 2	7205.27	2 YR	1.74	213.00	213.194	213.11	213.21	0.003935	0.54	3.24	19.74	0.42
Reach 2	7205.26	Regional	11.43	213.00	213.611		213.63	0.000931	0.60	22.51	53.12	0.25
Reach 2	7205.26	100 YR	6.07	213.00	213.270		213.31	0.006154	0.84	7.26	33.11	0.56
Reach 2	7205.26	50 YR	5.28	213.00	213.192		213.25	0.015857	1.09	4.86	29.30	0.85
Reach 2	7205.26	25 YR	4.53	213.00	213.158	213.16	213.23	0.023175	1.17	3.88	27.87	1.00
Reach 2	7205.26	10 YR	3.53	213.00	213.135	213.13	213.19	0.024244	1.09	3.25	26.91	1.00
Reach 2	7205.26	5 YR	2.78	213.00	213.115	213.12	213.17	0.025913	1.02	2.73	26.09	1.01
Reach 2	7205.26	2 YR	1.74	213.00	213.094	213.09	213.13	0.020009	0.79	2.19	25.22	0.86
Reach 2	7205.255	Regional	11.43	212.00	213.608		213.61	0.000083	0.33	67.50	81.27	0.09
Reach 2	7205.255	100 YR	6.07	212.00	213.276		213.28	0.000071	0.26	42.35	70.16	0.08
Reach 2	7205.255	50 YR	5.28	212.00	213.213		213.22	0.000069	0.25	37.94	67.97	0.08
Reach 2	7205.255	25 YR	4.53	212.00	213.154		213.16	0.000065	0.23	34.02	65.96	0.07
Reach 2	7205.255	10 YR	3.53	212.00	213.079		213.08	0.000056	0.20	29.13	63.36	0.07
Reach 2	7205.255	5 YR	2.78	212.00	213.004		213.01	0.000050	0.18	24.52	60.81	0.06
Reach 2	7205.255	2 YR	1.74	212.00	212.911		212.91	0.000031	0.13	19.26	52.45	0.05
Reach 2	7205.254	Regional	11.43	212.00	213.554	212.64	213.60	0.000756	1.05	14.47	79.57	0.27
Reach 2	7205.254	100 YR	6.07	212.00	213.254	212.44	213.27	0.000442	0.70	11.58	69.37	0.20
Reach 2	7205.254	50 YR	5.28	212.00	213.194	212.40	213.21	0.000396	0.64	11.00	67.32	0.19
Reach 2	7205.254	25 YR	4.53	212.00	213.139	212.37	213.15	0.000342	0.58	10.48	65.44	0.17
Reach 2	7205.254	10 YR	3.53	212.00	213.068	212.32	213.08	0.000258	0.48	9.80	63.01	0.15
Reach 2	7205.254	5 YR	2.78	212.00	212.997	212.27	213.00	0.000203	0.41	9.11	60.39	0.13
Reach 2	7205.254	2 YR	1.74	212.00	212.908	212.20	212.91	0.000109	0.28	8.26	52.15	0.09
Reach 2	7205.253		Culvert									
Reach 2	7205.252	Regional	11.43	212.00	212.873	212.64	213.04	0.005805	1.99	7.23	48.87	0.68
Reach 2	7205.252	100 YR	6.07	212.00	212.573	212.42	212.69	0.006734	1.62	4.71	21.68	0.68
Reach 2	7205.252	50 YR	5.28	212.00	212.525	212.39	212.63	0.006846	1.54	4.31	17.39	0.68
Reach 2	7205.252	25 YR	4.53	212.00	212.476	212.35	212.57	0.006979	1.46	3.91	14.84	0.67
Reach 2	7205.252	10 YR	3.53	212.00	212.411	212.30	212.49	0.006949	1.32	3.36	13.93	0.66
Reach 2	7205.252	5 YR	2.78	212.00	212.356	212.26	212.42	0.007040	1.20	2.90	13.15	0.64
Reach 2	7205.252	2 YR	1.74	212.00	212.274	212.19	212.32	0.006662	0.98	2.22	12.01	0.60
Reach 2	7205.251	Regional	11.43	212.00	212.941		212.97	0.001047	0.81	21.25	55.16	0.28
Reach 2	7205.251	100 YR	6.07	212.00	212.609		212.64	0.002041	0.80	8.03	24.94	0.37
Reach 2	7205.251	50 YR	5.28	212.00	212.556		212.59	0.002283	0.79	6.83	20.19	0.38
Reach 2	7205.251	25 YR	4.53	212.00	212.503		212.53	0.002590	0.77	5.88	15.44	0.40
Reach 2	7205.251	10 YR	3.53	212.00	212.430		212.46	0.002824	0.74	4.80	14.19	0.40
Reach 2	7205.251	5 YR	2.78	212.00	212.369		212.39	0.003038	0.70	3.97	13.34	0.41
Reach 2	7205.251	2 YR	1.74	212.00	212.280		212.30	0.003203	0.61	2.84	12.09	0.40
Reach 2	7205.25	Regional	11.43	212.00	212.820	212.55	212.94	0.003924	1.57	8.18	43.93	0.55
Reach 2	7205.25	100 YR	6.07	212.00	212.537	212.37	212.62	0.004584	1.28	5.31	18.51	0.56
Reach 2	7205.25	50 YR	5.28	212.00	212.491	212.33	212.56	0.004719	1.22	4.84	15.04	0.56
Reach 2	7205.25	25 YR	4.53	212.00	212.444	212.30	212.51	0.004856	1.16	4.37	14.39	0.56
Reach 2	7205.25	10 YR	3.53	212.00	212.380	212.26	212.43	0.004997	1.06	3.71	13.49	0.55
Reach 2	7205.25	5 YR	2.78	212.00	212.324	212.22	212.37	0.005343	0.98	3.14	12.70	0.55
Reach 2	7205.25	2 YR	1.74	212.00	212.242	212.16	212.28	0.005652	0.83	2.31	11.56	0.54
Reach 2	7205.245		Bridge									
Reach 2	7205.24	Regional	12.58	211.50	212.192	212.11	212.43	0.009649	2.20	5.85	29.84	0.84
Reach 2	7205.24	100 YR	6.68	211.50	211.979	211.91	212.12	0.009304	1.69	4.04	25.96	0.78
Reach 2	7205.24	50 YR	5.81	211.50	211.940	211.87	212.07	0.009343	1.60	3.71	24.99	0.77
Reach 2	7205.24	25 YR	4.98	211.50	211.890	211.83	212.01	0.010257	1.54	3.29	23.74	0.79

Carlton Creek, Crossing at Victoria Square Boulevard (Proposed Condition)



HEC-RAS Plan: Plan 02 River: Carleton Creek Reach: Reach 2

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 2	7205.38	Regional	8.59	218.50	219.67	219.67	219.91	0.010551	2.56	7.15	18.98	0.76
Reach 2	7205.38	100 YR	4.56	218.50	219.38	219.38	219.61	0.012230	2.27	3.18	9.66	0.78
Reach 2	7205.38	50 YR	3.97	218.50	219.32	219.32	219.55	0.012906	2.22	2.63	8.44	0.79
Reach 2	7205.38	25 YR	3.40	218.50	219.25	219.25	219.48	0.014133	2.18	2.08	7.02	0.82
Reach 2	7205.38	10 YR	2.65	218.50	219.17	219.12	219.37	0.013436	1.97	1.60	5.49	0.78
Reach 2	7205.38	5 YR	2.09	218.50	219.10	219.00	219.27	0.012825	1.79	1.27	4.08	0.75
Reach 2	7205.38	2 YR	1.31	218.50	218.96		219.08	0.012501	1.49	0.88	1.99	0.72
Reach 2	7205.37	Regional	8.59	218.00	219.39		219.49	0.004145	1.80	10.63	19.78	0.49
Reach 2	7205.37	100 YR	4.56	218.00	219.07		219.17	0.004829	1.62	5.27	13.30	0.51
Reach 2	7205.37	50 YR	3.97	218.00	219.00		219.10	0.004952	1.58	4.49	12.07	0.51
Reach 2	7205.37	25 YR	3.40	218.00	218.94		219.04	0.005046	1.52	3.76	10.78	0.51
Reach 2	7205.37	10 YR	2.65	218.00	218.83		218.92	0.005630	1.47	2.66	8.51	0.52
Reach 2	7205.37	5 YR	2.09	218.00	218.73		218.83	0.005877	1.39	1.97	6.68	0.53
Reach 2	7205.37	2 YR	1.31	218.00	218.58		218.65	0.006008	1.18	1.16	3.51	0.51
Reach 2	7205.36	Regional	8.59	217.50	218.77	218.77	219.09	0.011185	2.78	5.22	10.76	0.80
Reach 2	7205.36	100 YR	4.56	217.50	218.37	218.37	218.68	0.015427	2.53	2.25	4.98	0.88
Reach 2	7205.36	50 YR	3.97	217.50	218.30	218.30	218.60	0.016626	2.47	1.90	4.38	0.90
Reach 2	7205.36	25 YR	3.40	217.50	218.22	218.22	218.51	0.018224	2.40	1.58	3.74	0.92
Reach 2	7205.36	10 YR	2.65	217.50	218.14	218.10	218.37	0.017072	2.14	1.30	3.11	0.87
Reach 2	7205.36	5 YR	2.09	217.50	218.05	218.00	218.25	0.017933	1.98	1.06	2.41	0.87
Reach 2	7205.36	2 YR	1.31	217.50	217.91	217.87	218.06	0.018647	1.71	0.76	1.96	0.88
Reach 2	7205.35	Regional	9.45	216.50	217.58	217.58	217.76	0.011211	2.49	9.43	27.49	0.77
Reach 2	7205.35	100 YR	5.02	216.50	217.39	217.39	217.55	0.010279	2.09	5.05	19.10	0.72
Reach 2	7205.35	50 YR	4.37	216.50	217.35	217.35	217.51	0.010114	2.01	4.34	17.38	0.71
Reach 2	7205.35	25 YR	3.74	216.50	217.30	217.30	217.46	0.010748	1.98	3.46	14.98	0.72
Reach 2	7205.35	10 YR	2.92	216.50	217.21	217.21	217.38	0.011686	1.91	2.36	11.30	0.74
Reach 2	7205.35	5 YR	2.30	216.50	217.16	217.08	217.30	0.010343	1.71	1.83	9.04	0.68
Reach 2	7205.35	2 YR	1.44	216.50	217.03	216.89	217.13	0.009915	1.43	1.02	3.23	0.64
Reach 2	7205.34	Regional	9.45	215.50	216.62	216.62	216.83	0.011262	2.57	8.47	22.14	0.78
Reach 2	7205.34	100 YR	5.02	215.50	216.41	216.41	216.59	0.010819	2.17	4.43	15.12	0.74
Reach 2	7205.34	50 YR	4.37	215.50	216.36	216.36	216.54	0.011009	2.11	3.73	13.56	0.74
Reach 2	7205.34	25 YR	3.74	215.50	216.30	216.30	216.49	0.011306	2.04	3.04	11.80	0.74
Reach 2	7205.34	10 YR	2.92	215.50	216.20	216.20	216.39	0.013375	2.02	1.99	8.43	0.79
Reach 2	7205.34	5 YR	2.30	215.50	216.06	216.06	216.29	0.020801	2.15	1.12	3.85	0.94
Reach 2	7205.34	2 YR	1.44	215.50	215.90	215.90	216.09	0.024220	1.93	0.75	1.96	1.00
Reach 2	7205.33	Regional	9.45	214.50	215.61	215.61	215.81	0.011268	2.55	8.36	20.64	0.78
Reach 2	7205.33	100 YR	5.02	214.50	215.40	215.40	215.59	0.010736	2.16	4.52	15.71	0.74
Reach 2	7205.33	50 YR	4.37	214.50	215.38	215.36	215.54	0.009253	1.97	4.20	15.00	0.68
Reach 2	7205.33	25 YR	3.74	214.50	215.36	215.30	215.49	0.008024	1.80	3.80	14.07	0.63
Reach 2	7205.33	10 YR	2.92	214.50	215.30	215.20	215.41	0.006728	1.58	3.14	12.35	0.57
Reach 2	7205.33	5 YR	2.30	214.50	215.26	215.06	215.34	0.005617	1.38	2.60	10.77	0.52
Reach 2	7205.33	2 YR	1.44	214.50	215.14		215.20	0.004674	1.13	1.58	6.83	0.46
Reach 2	7205.32	Regional	10.39	214.00	215.41		215.44	0.002066	1.28	21.88	41.78	0.35
Reach 2	7205.32	100 YR	5.52	214.00	215.08		215.14	0.003319	1.36	10.27	29.22	0.42
Reach 2	7205.32	50 YR	4.80	214.00	215.02		215.08	0.003886	1.41	8.34	26.57	0.45
Reach 2	7205.32	25 YR	4.11	214.00	214.94		215.01	0.004515	1.44	6.57	23.31	0.48
Reach 2	7205.32	10 YR	3.21	214.00	214.84	214.75	214.92	0.005736	1.50	4.33	18.13	0.53
Reach 2	7205.32	5 YR	2.53	214.00	214.74	214.65	214.84	0.007197	1.54	2.77	13.36	0.58
Reach 2	7205.32	2 YR	1.59	214.00	214.56	214.42	214.67	0.009786	1.48	1.15	4.84	0.65
Reach 2	7205.31	Regional	10.39	213.50	214.83	214.83	215.18	0.012107	2.99	6.05	10.30	0.83
Reach 2	7205.31	100 YR	5.52	213.50	214.48	214.48	214.78	0.013639	2.57	3.04	6.77	0.84
Reach 2	7205.31	50 YR	4.80	213.50	214.44	214.41	214.70	0.012320	2.37	2.78	6.37	0.79
Reach 2	7205.31	25 YR	4.11	213.50	214.39	214.32	214.61	0.011111	2.18	2.50	5.92	0.75
Reach 2	7205.31	10 YR	3.21	213.50	214.33	214.20	214.50	0.009272	1.89	2.13	5.26	0.67
Reach 2	7205.31	5 YR	2.53	213.50	214.27	214.08	214.40	0.007696	1.63	1.84	4.68	0.61
Reach 2	7205.31	2 YR	1.59	213.50	214.17	213.92	214.24	0.005233	1.22	1.42	3.66	0.49
Reach 2	7205.3	Regional	10.39	213.62	214.43	214.43	214.73	0.011984	2.48	4.80	9.59	0.93
Reach 2	7205.3	100 YR	5.52	213.62	214.18	214.18	214.40	0.015009	2.09	2.76	7.23	0.97
Reach 2	7205.3	50 YR	4.80	213.62	214.13	214.13	214.34	0.016157	2.02	2.44	6.79	0.99
Reach 2	7205.3	25 YR	4.11	213.62	214.09	214.09	214.28	0.016688	1.92	2.17	6.40	0.99
Reach 2	7205.3	10 YR	3.21	213.62	214.03	214.03	214.19	0.018391	1.79	1.79	5.81	1.01
Reach 2	7205.3	5 YR	2.53	213.62	213.98	213.98	214.12	0.018964	1.66	1.52	5.42	1.00
Reach 2	7205.3	2 YR	1.59	213.62	213.89	213.89	214.00	0.021648	1.50	1.06	4.85	1.03
Reach 2	7205.295		Culvert									
Reach 2	7205.29	Regional	10.39	212.96	214.08	214.08	214.43	0.013244	2.62	4.25	7.56	0.95
Reach 2	7205.29	100 YR	5.52	212.96	213.78	213.78	214.05	0.016882	2.27	2.43	4.74	1.01
Reach 2	7205.29	50 YR	4.80	212.96	213.73	213.73	213.98	0.016871	2.19	2.19	4.50	1.00

HEC-RAS Plan: Plan 02 River: Carleton Creek Reach: Reach 2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 2	7205.29	25 YR	4.11	212.96	213.68	213.68	213.90	0.017175	2.12	1.94	4.25	1.00
Reach 2	7205.29	10 YR	3.21	212.96	213.59	213.59	213.80	0.017479	2.00	1.61	3.90	0.99
Reach 2	7205.29	5 YR	2.53	212.96	213.51	213.51	213.70	0.018762	1.92	1.31	3.56	1.01
Reach 2	7205.29	2 YR	1.59	212.96	213.40	213.40	213.55	0.018971	1.71	0.93	3.07	0.99
Reach 2	7205.28	Regional	11.43	213.00	213.73		213.75	0.001045	0.72	20.62	48.77	0.28
Reach 2	7205.28	100 YR	6.07	213.00	213.52		213.54	0.001094	0.58	11.99	34.93	0.27
Reach 2	7205.28	50 YR	5.28	213.00	213.49		213.51	0.001023	0.53	11.04	33.18	0.26
Reach 2	7205.28	25 YR	4.53	213.00	213.46		213.47	0.000977	0.50	10.00	31.64	0.25
Reach 2	7205.28	10 YR	3.53	213.00	213.42		213.43	0.000879	0.44	8.64	29.51	0.23
Reach 2	7205.28	5 YR	2.78	213.00	213.37		213.38	0.000835	0.39	7.41	27.43	0.22
Reach 2	7205.28	2 YR	1.74	213.00	213.30		213.31	0.000725	0.31	5.64	24.13	0.20
Reach 2	7205.27	Regional	11.43	213.00	213.61		213.65	0.002069	0.89	15.61	41.72	0.38
Reach 2	7205.27	100 YR	6.07	213.00	213.32		213.38	0.006978	1.01	6.11	25.32	0.61
Reach 2	7205.27	50 YR	5.28	213.00	213.31	213.23	213.35	0.006515	0.94	5.69	24.45	0.59
Reach 2	7205.27	25 YR	4.53	213.00	213.30	213.20	213.33	0.005568	0.84	5.41	23.86	0.54
Reach 2	7205.27	10 YR	3.53	213.00	213.27	213.18	213.29	0.005091	0.74	4.75	22.37	0.51
Reach 2	7205.27	5 YR	2.78	213.00	213.24	213.17	213.26	0.004625	0.66	4.21	21.24	0.47
Reach 2	7205.27	2 YR	1.74	213.00	213.19	213.11	213.21	0.003935	0.54	3.24	19.74	0.42
Reach 2	7205.26	Regional	11.43	213.00	213.61		213.63	0.000931	0.60	22.51	53.12	0.25
Reach 2	7205.26	100 YR	6.07	213.00	213.27		213.31	0.006154	0.84	7.26	33.11	0.56
Reach 2	7205.26	50 YR	5.28	213.00	213.19		213.25	0.015857	1.09	4.86	29.30	0.85
Reach 2	7205.26	25 YR	4.53	213.00	213.16	213.16	213.23	0.023175	1.17	3.88	27.87	1.00
Reach 2	7205.26	10 YR	3.53	213.00	213.13	213.13	213.19	0.024244	1.09	3.25	26.91	1.00
Reach 2	7205.26	5 YR	2.78	213.00	213.12	213.12	213.17	0.025913	1.02	2.73	26.09	1.01
Reach 2	7205.26	2 YR	1.74	213.00	213.09	213.09	213.13	0.020009	0.79	2.19	25.22	0.86
Reach 2	7205.255	Regional	11.43	212.00	213.61		213.61	0.000083	0.33	67.50	81.27	0.09
Reach 2	7205.255	100 YR	6.07	212.00	213.28		213.28	0.000071	0.26	42.35	70.16	0.08
Reach 2	7205.255	50 YR	5.28	212.00	213.21		213.22	0.000069	0.25	37.94	67.97	0.08
Reach 2	7205.255	25 YR	4.53	212.00	213.15		213.16	0.000065	0.23	34.02	65.96	0.07
Reach 2	7205.255	10 YR	3.53	212.00	213.08		213.08	0.000056	0.20	29.13	63.36	0.07
Reach 2	7205.255	5 YR	2.78	212.00	213.00		213.01	0.000050	0.18	24.52	60.81	0.06
Reach 2	7205.255	2 YR	1.74	212.00	212.91		212.91	0.000031	0.13	19.26	52.45	0.05
Reach 2	7205.254	Regional	11.43	212.00	213.55	212.64	213.60	0.000756	1.05	14.47	79.57	0.27
Reach 2	7205.254	100 YR	6.07	212.00	213.25	212.44	213.27	0.000442	0.70	11.58	69.37	0.20
Reach 2	7205.254	50 YR	5.28	212.00	213.19	212.40	213.21	0.000396	0.64	11.00	67.32	0.19
Reach 2	7205.254	25 YR	4.53	212.00	213.14	212.37	213.15	0.000342	0.58	10.48	65.44	0.17
Reach 2	7205.254	10 YR	3.53	212.00	213.07	212.32	213.08	0.000258	0.48	9.80	63.01	0.15
Reach 2	7205.254	5 YR	2.78	212.00	213.00	212.27	213.00	0.000203	0.41	9.11	60.39	0.13
Reach 2	7205.254	2 YR	1.74	212.00	212.91	212.20	212.91	0.000109	0.28	8.26	52.15	0.09
Reach 2	7205.253		Culvert									
Reach 2	7205.252	Regional	11.43	212.00	212.87	212.64	213.04	0.005805	1.99	7.23	48.87	0.68
Reach 2	7205.252	100 YR	6.07	212.00	212.57	212.42	212.69	0.006734	1.62	4.71	21.68	0.68
Reach 2	7205.252	50 YR	5.28	212.00	212.52	212.39	212.63	0.006846	1.54	4.31	17.39	0.68
Reach 2	7205.252	25 YR	4.53	212.00	212.48	212.35	212.57	0.006979	1.46	3.91	14.84	0.67
Reach 2	7205.252	10 YR	3.53	212.00	212.41	212.30	212.49	0.006949	1.32	3.36	13.93	0.66
Reach 2	7205.252	5 YR	2.78	212.00	212.36	212.26	212.42	0.007040	1.20	2.90	13.15	0.64
Reach 2	7205.252	2 YR	1.74	212.00	212.27	212.19	212.32	0.006662	0.98	2.22	12.01	0.60
Reach 2	7205.251	Regional	11.43	212.00	212.94		212.97	0.001047	0.81	21.25	55.16	0.28
Reach 2	7205.251	100 YR	6.07	212.00	212.61		212.64	0.002041	0.80	8.03	24.94	0.37
Reach 2	7205.251	50 YR	5.28	212.00	212.56		212.59	0.002283	0.79	6.83	20.19	0.38
Reach 2	7205.251	25 YR	4.53	212.00	212.50		212.53	0.002590	0.77	5.88	15.44	0.40
Reach 2	7205.251	10 YR	3.53	212.00	212.43		212.46	0.002824	0.74	4.80	14.19	0.40
Reach 2	7205.251	5 YR	2.78	212.00	212.37		212.39	0.003038	0.70	3.97	13.34	0.41
Reach 2	7205.251	2 YR	1.74	212.00	212.28		212.30	0.003203	0.61	2.84	12.09	0.40
Reach 2	7205.25	Regional	11.43	212.00	212.82	212.55	212.94	0.003924	1.57	8.18	43.93	0.55
Reach 2	7205.25	100 YR	6.07	212.00	212.54	212.37	212.62	0.004584	1.28	5.31	18.51	0.56
Reach 2	7205.25	50 YR	5.28	212.00	212.49	212.33	212.56	0.004719	1.22	4.84	15.04	0.56
Reach 2	7205.25	25 YR	4.53	212.00	212.44	212.30	212.51	0.004856	1.16	4.37	14.39	0.56
Reach 2	7205.25	10 YR	3.53	212.00	212.38	212.26	212.43	0.004997	1.06	3.71	13.49	0.55
Reach 2	7205.25	5 YR	2.78	212.00	212.32	212.22	212.37	0.005343	0.98	3.14	12.70	0.55
Reach 2	7205.25	2 YR	1.74	212.00	212.24	212.16	212.28	0.005652	0.83	2.31	11.56	0.54

Crossing C-4

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	m/m
Diameter	0.83	m
Discharge	1.26	m³/s

Results

Normal Depth	0.60	m
Flow Area	0.42	m²
Wetted Perimeter	1.68	m
Hydraulic Radius	0.25	m
Top Width	0.74	m
Critical Depth	0.68	m
Percent Full	72.7	%
Critical Slope	0.00772	m/m
Velocity	3.03	m/s
Velocity Head	0.47	m
Specific Energy	1.07	m
Froude Number	1.29	
Maximum Discharge	1.54	m³/s
Discharge Full	1.44	m³/s
Slope Full	0.00771	m/m
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	72.65	%
Downstream Velocity	Infinity	m/s

Crossing C-4

GVF Output Data

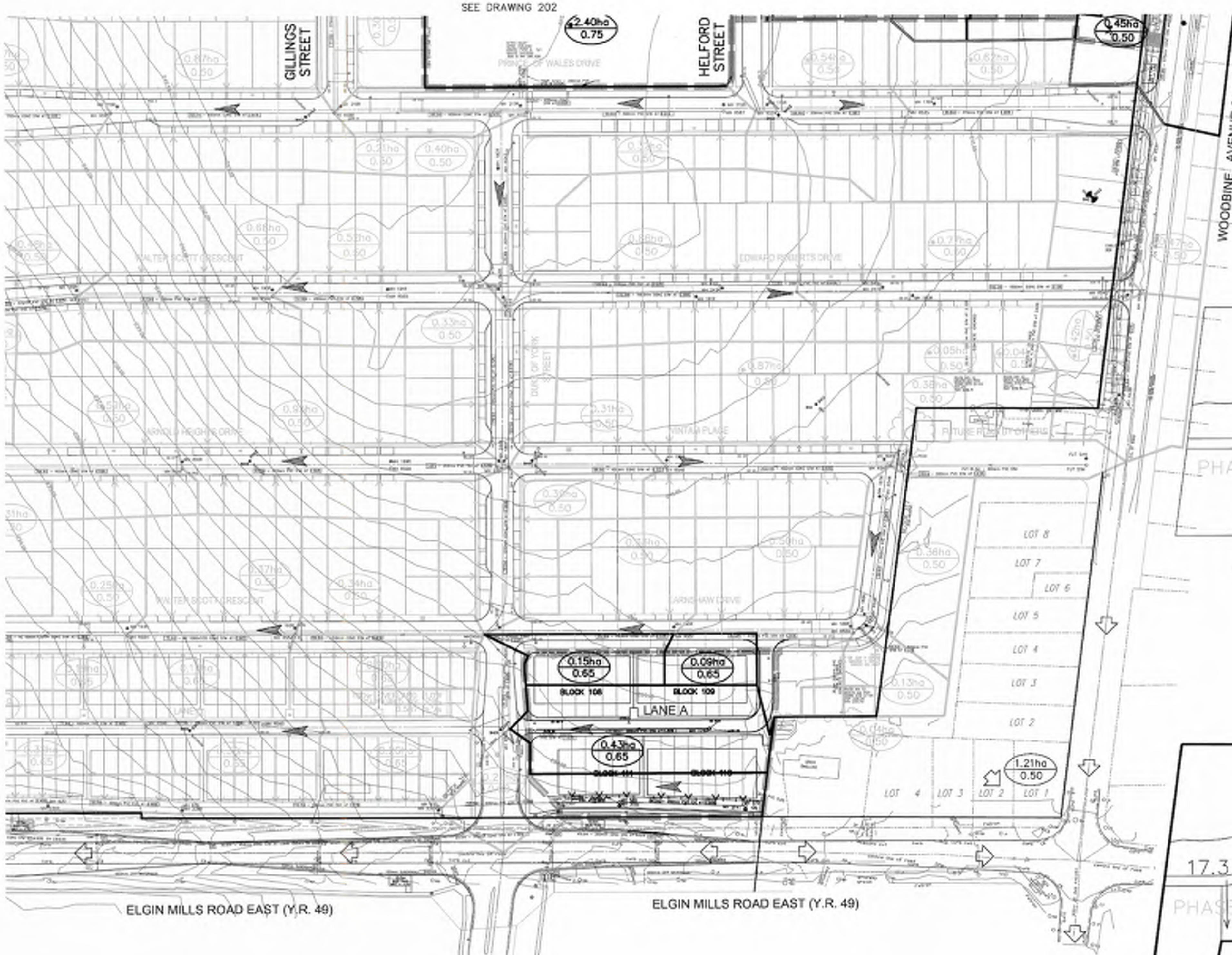
Upstream Velocity	Infinity	m/s
Normal Depth	0.60	m
Critical Depth	0.68	m
Channel Slope	0.01000	m/m
Critical Slope	0.00772	m/m

E

Neighbouring Development SWM Reports - Excerpts

AS CONSTRUCTED

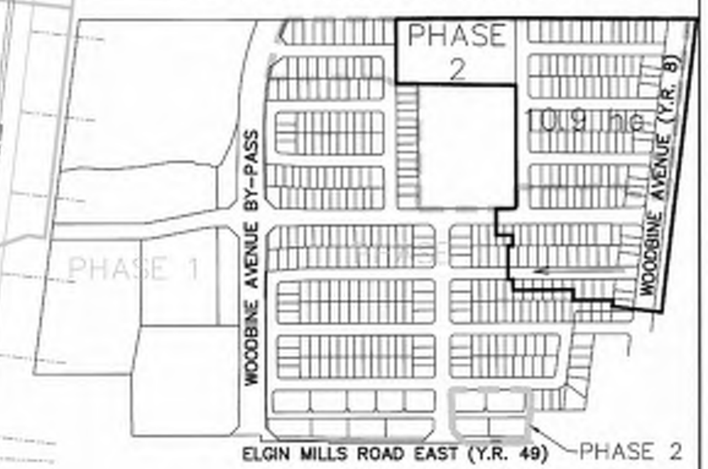
SEE DRAWING 202



Minor system for the northeast portion of subdivision designed to capture 10 year storm event to achieve predevelopment drainage targets for major storm events as per section 4.2 of the approved SWM Report dated July 2007.

KEY PLAN

N.T.S.



10yr DRAINAGE AREA

KEY PLAN

N.T.S.

100yr DRAINAGE AREA

KEY PLAN

N.T.S.

LEGEND

- HYDRANT
- WATERMAIN VALVE
- WATER BOX
- DUAL SEWER CONNECTION
- SINGLE SEWER CONNECTION
- STORM MANHOLE
- SANITARY MANHOLE
- CATCHBASIN – NO ICD
- CATCHBASIN – TYPE "B" ICD = 28.3 L/S
- CATCHBASIN – TYPE "D" ICD = 54.0 L/S
- HYDRO TRANSFORMER
- COMMUNITY MAIL BOX
- PHASE 2 LIMIT

REFER TO DRAWING 1 FOR GENERAL NOTES

- 0.52ha / 0.50 — 10 YEAR DRAINAGE AREA IN HECTARES RUN-OFF COEFFICIENT
- 0.52ha / 0.50 — AREA IN HECTARES RUN-OFF COEFFICIENT
- OVERLAND FLOW ROUTE
- EXTERNAL OVERLAND FLOW ROUTE

LEGEND (AS-CONSTRUCTED)

- 10.5m — PHASE 2 AS-CONSTRUCTED STORM & SANITARY SEWER

BENCHMARK

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO TOWN OF MARKHAM MONUMENT No. M-11-003
ELEVATION 238.786m
TWO STOREY BRICK HOUSE ON WEST SIDE OF WOODBINE AVE 39.6m S OF JUNCTION OF WOODBINE AVE AND ELGIN MILLS ROAD IN THE HAMLET OF VICTORIA SQUARE AND 27.9m WEST OF CENTRELINE OF WOODBINE AVE. TABLET SET HORIZONTALLY IN S FACE OF STONE FOUNDATION, 27cm WEST OF SOUTHEAST CORNER AND 30cm BELOW BRICKWORK.

No.	DESCRIPTION	DATE	BY
5	AS-CONSTRUCTED SUBMISSION TO TOWN – (TOWN COMMENTS ADDRESSED)	JULY 2013	NK
4	AS-CONSTRUCTED SUBMISSION TO TOWN	JANUARY 2013	NK
3	SUBMISSION FOR APPROVAL SIGNATURE	MAY 28, 2010	NK
2	SECOND SUBMISSION	FEB 17, 2010	NK
1	FIRST SUBMISSION	NOV 9, 2009	NK

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER N. KARANIS 100037074 Ap 22/14 PROVINCE OF ONTARIO

HERITAGE AT VICTORIA SQUARE (PHASE 2)
MONARCH CORPORATION
STORM DRAINAGE PLAN
SHEET 1 OF 2

COLE ENGINEERING

DESIGN BY: DH/NK	DRAWN BY: JM/PT	PROJECT No. M03-115
SCALE: 1:1000	DATE: MAY/2010	DRAWING No. 201

Approved as to form in reliance upon the professional skill and ability of Cole Engineering Group Limited, Consulting Engineers, as to design and specification.

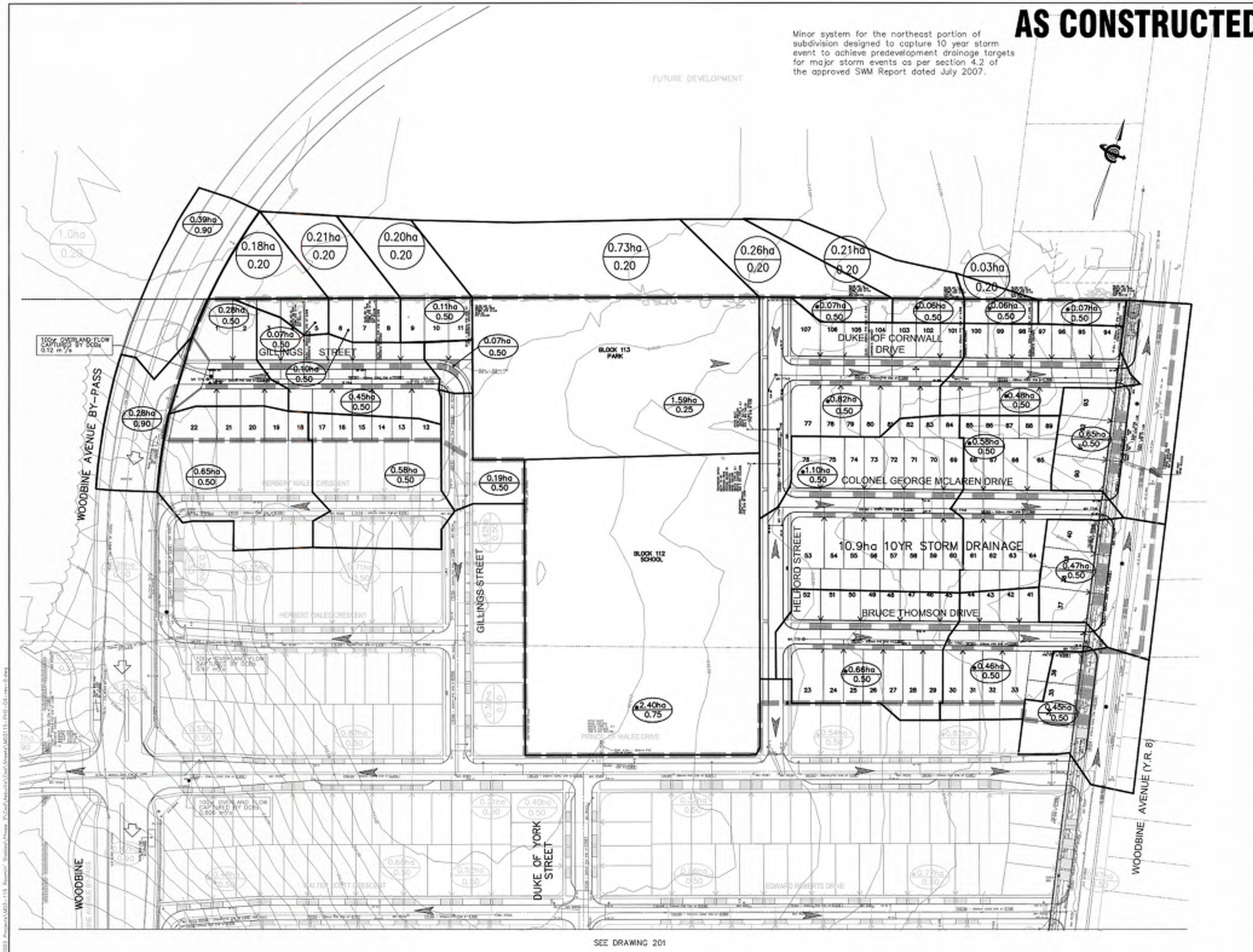
Brian Lee, P.Eng. — Date
Manager of Development Engineering & Transportation

TOWN FILE No.: 1025

TECH 11 112016

AS CONSTRUCTED

Minor system for the northeast portion of subdivision designed to capture 10 year storm event to achieve predevelopment drainage targets for major storm events as per section 4.2 of the approved SWM Report dated July 2007.



KEY PLAN

N.T.S.

LEGEND

- HYDRANT
- WATERMAIN VALVE
- WATER BOX
- DUAL SEWER CONNECTION
- SINGLE SEWER CONNECTION
- STORM MANHOLE
- SANITARY MANHOLE
- CATCHBASIN – NO ICD
- CATCHBASIN – TYPE "B" ICD = 28.3 L/S
- CATCHBASIN – TYPE "D" ICD = 54.0 L/S
- HYDRO TRANSFORMER
- COMMUNITY MAIL BOX
- PHASE 2 LIMIT

REFER TO DRAWING 1 FOR GENERAL NOTES

- 0.52ha / 0.50 — 10 YEAR DRAINAGE AREA IN HECTARES RUN-OFF COEFFICIENT
- 0.52ha / 0.50 — AREA IN HECTARES RUN-OFF COEFFICIENT
- OVERLAND FLOW ROUTE
- EXTERNAL OVERLAND FLOW ROUTE

LEGEND (AS-CONSTRUCTED)

- 10.5m — PHASE 2 AS-CONSTRUCTED STORM & SANITARY SEWER

BENCHMARK

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO TOWN OF MARKHAM MONUMENT No. W-11-003

ELEVATION 236.786m

TWO STOREY BRICK HOUSE ON WEST SIDE OF WOODBINE AVE 39.6m S OF JUNCTION OF WOODBINE AVE AND ELGIN MILLS ROAD IN THE HAMLET OF VICTORIA SQUARE AND 27.9m WEST OF CENTRELINE OF WOODBINE AVE. TABLET SET HORIZONTALLY IN S FACE OF STONE FOUNDATION, 27cm WEST OF SOUTHEAST CORNER AND 30cm BELOW BRICKWORK.

No.	DESCRIPTION	DATE	BY
5	AS-CONSTRUCTED SUBMISSION TO TOWN – (TOWN COMMENTS ADDRESSED)	JULY 2013	NK
4	AS-CONSTRUCTED SUBMISSION TO TOWN	JANUARY 2013	NK
3	SUBMISSION FOR APPROVAL SIGNATURE	MAY 28, 2010	NK
2	SECOND SUBMISSION	FEB 17, 2010	NK
1	FIRST SUBMISSION	NOV 6, 2009	NK

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
N. KARAKIS
100037374
PROVINCE OF ONTARIO

HERITAGE AT VICTORIA SQUARE (PHASE 2)
MONARCH CORPORATION
STORM DRAINAGE PLAN

COLE ENGINEERING

DESIGN BY: DH/NK	DRAWN BY: JM/PT	PROJECT No. M03-115
SCALE: 1:1000	DATE: MAY/2010	

Approved as to form in reliance upon the professional skill and ability of Cole Engineering Group Limited, Consulting Engineers, as to design and specification.

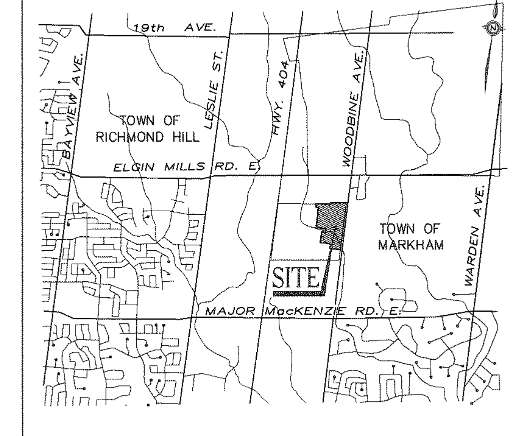
Brian Lee, P.Eng. — Manager of Development Engineering & Transportation

Date

DRAWING No. 202

TOWN FILE No.: 1025

TECH 11 112016



KEYPLAN
N.T.S.

LEGEND

0.61 / 0.60 — AREA IN HECTARES / RUN-OFF COEFFICIENT

0.72 / 0.90 — EXTERNAL AREA IN HECTARES / RUN-OFF COEFFICIENT

DRAINAGE BOUNDARY

EXTERNAL TRIBUTARY DRAINAGE BOUNDARY

OVERLAND FLOW DRAINAGE ROUTE

212.5 — EXISTING CONTOUR

65M-3830

No.	DESCRIPTION	DATE	BY	TOWN APP.
4.	ISSUED FOR ABOVE GROUND APPROVAL	15 OCT 04	M.A.E.	S.A.
3.	THIRD SUBMISSION	7 JULY 04	T.R.	
2.	SECOND SUBMISSION	13 APR 04	M.A.E.	
1.	FIRST SUBMISSION	8 JAN 04	M.A.E.	

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
B.T. BURKE
PROVINCE OF ONTARIO

Cathedral Town Ltd. and
404 Developments Inc.
Cathedraltown - Phase 1

EXTERNAL STORM DRAINAGE PLAN

Marshall Macklin Monaghan
PROJECT MANAGERS · ENGINEERS · SURVEYORS · PLANNERS

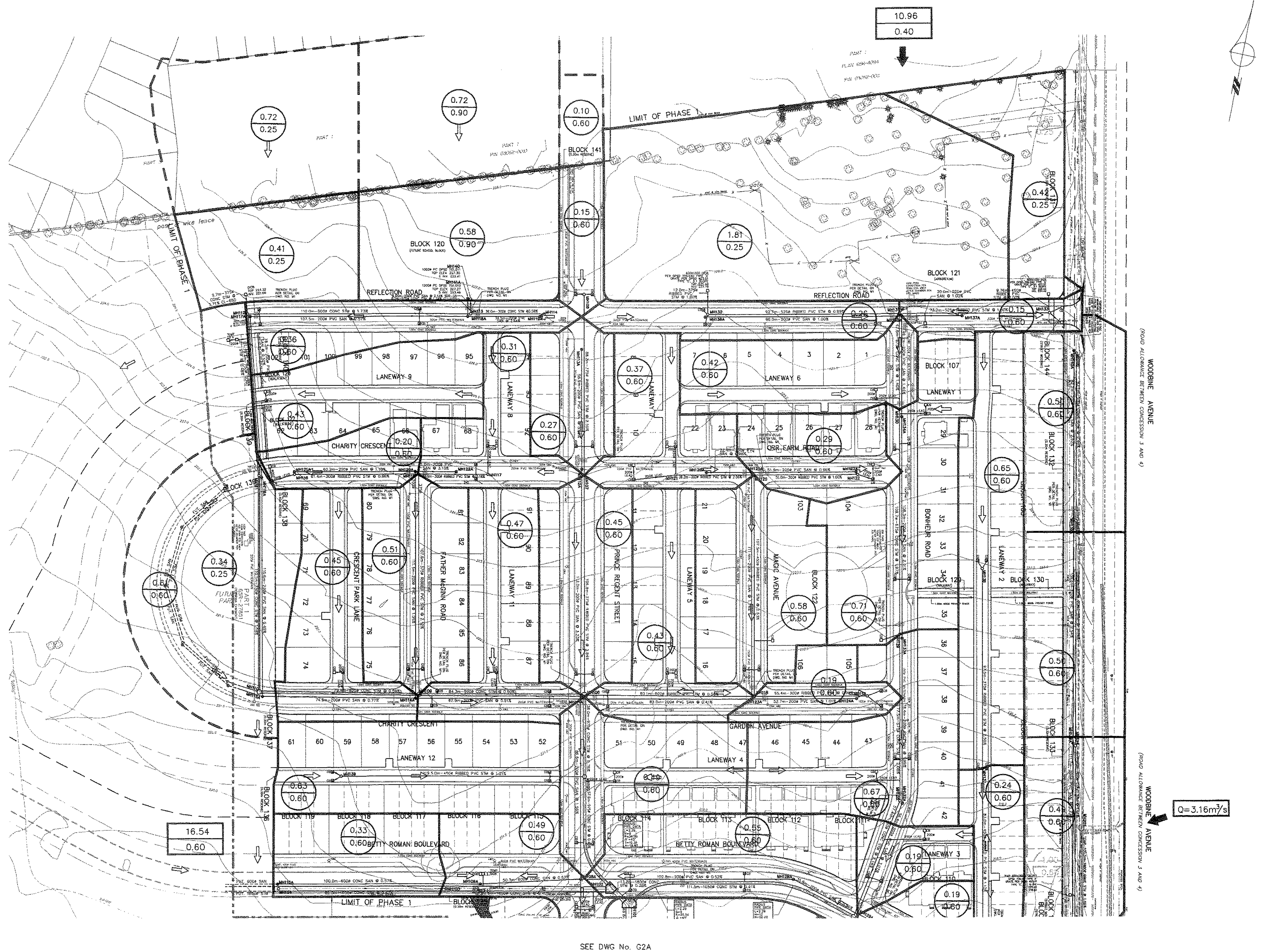
DESIGN BY: T.Q.	CHECKED BY: M.A.E.	PROJECT No. 10-03057
DRAWN BY: 10/12 CAD	CHECKED BY: T.R.	
SCALES: 1:4000	DATE: OCTOBER 2003	

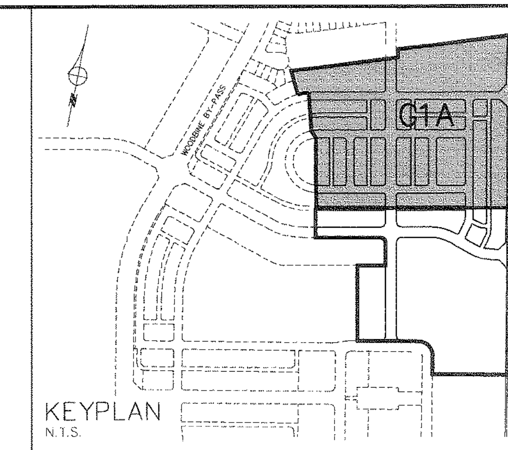
Approved as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.
STEVE ANDREWS — 30 NOV/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

DRAWING No. G3

Approved underground services as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.
STEVE ANDREWS — AUG. 20/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

TOWN FILE No. XXXX PHASE 1
D19-03.010.M





KEYPLAN
N.T.S.

LEGEND

0.61 / 0.60	AREA IN HECTARES / RUN-OFF COEFFICIENT
0.72 / 0.90	EXTERNAL AREA IN HECTARES / RUN-OFF COEFFICIENT
0.61 / 0.60	AREA IN HECTARES / RUN-OFF COEFFICIENT
	DRAINAGE BOUNDARY
	EXTERNAL TRIBUTARY DRAINAGE BOUNDARY
	INTERIM DRAINAGE BOUNDARY
	OVERLAND FLOW DRAINAGE ROUTE
212.5	EXISTING CONTOUR

65M-3830

No.	DESCRIPTION	DATE	BY	TOWN APP.
6.	AS CONSTRUCTED UNDERGROUNDS	DEC. 08	B.T.B.	
5.	ISSUED FOR ABOVE GROUND APPROVAL	15 OCT 04	M.A.E.	S.A.
4.	REVISED REFLECTION RD SEWERS PER YCSB COMMENTS	3 SEPT 04	M.A.E.	
3.	THIRD SUBMISSION	7 JULY 04	T.R.	
2.	SECOND SUBMISSION	13 APR 04	M.A.E.	
1.	FIRST SUBMISSION	8 JAN 04	M.A.E.	

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
B.T. BURKE
PROVINCE OF ONTARIO

Cathedral Town Ltd. and
404 Developments Inc.
Cathedraltown - Phase 1
GENERAL PLAN
AND STORM DRAINAGE PLAN

Marshall Macklin Monaghan
PROJECT MANAGERS · ENGINEERS · SURVEYORS · PLANNERS

DESIGN BY:	T.Q.	CHECKED BY:	M.A.E.	PROJECT No.
DRAWN BY:	10/12 CAD	CHECKED BY:	T.R.	10-03057
SCALES:	1:1000	DATE:	OCTOBER 2003	

Approved as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.

STEVE ANDREWS — 30 NOV/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

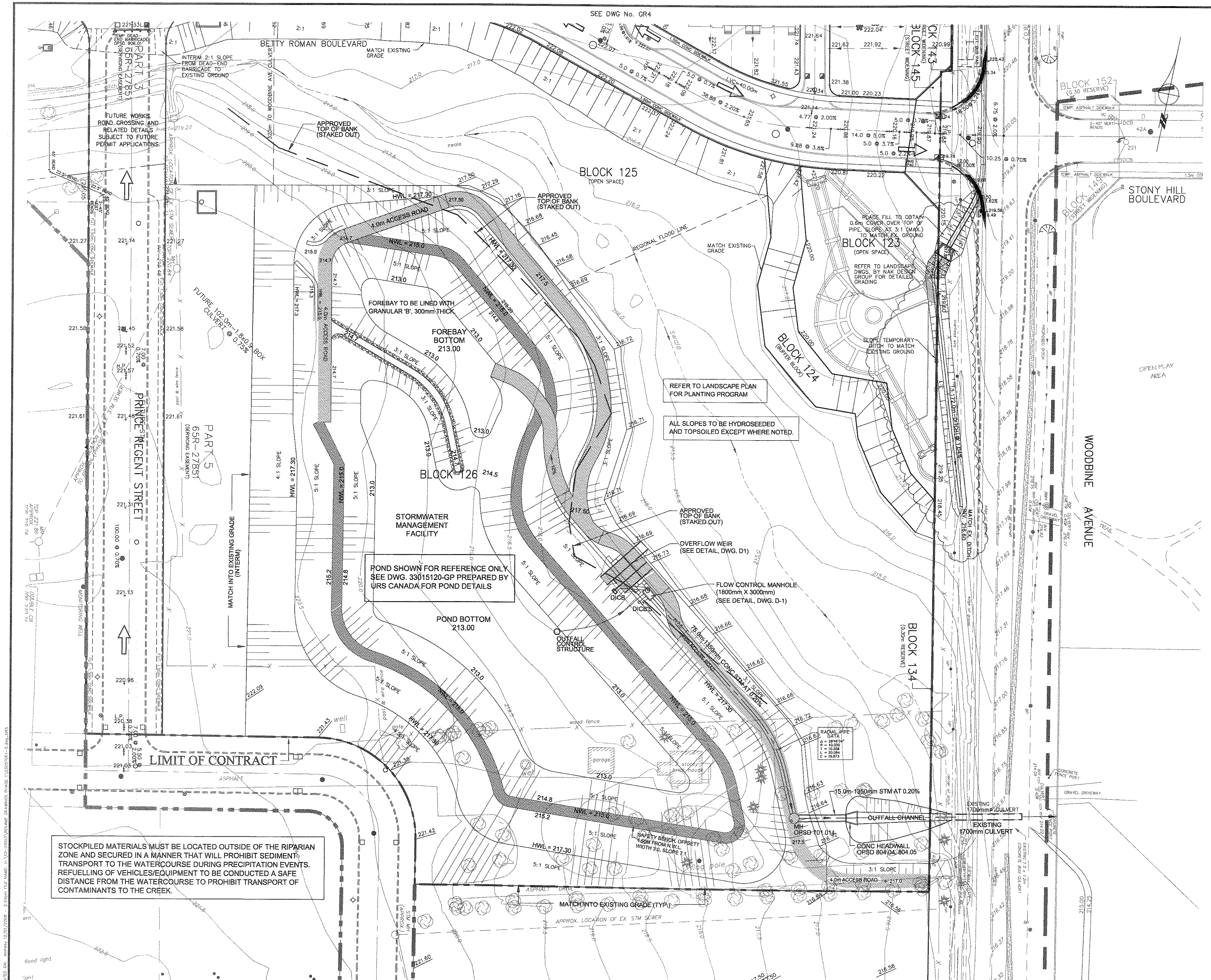
DRAWING No. G1A

Approved underground services as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.

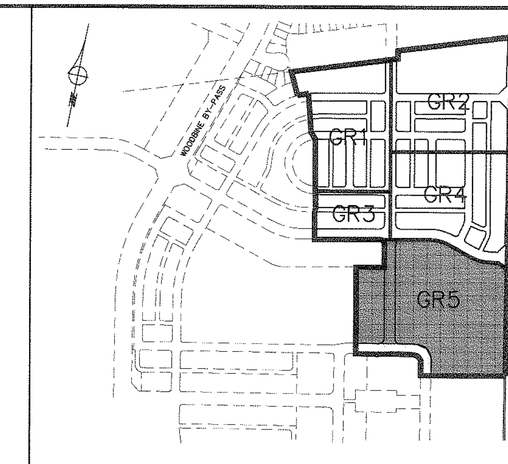
STEVE ANDREWS — AUG. 20/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

TOWN FILE No. XXXX PHASE 1
D19-03.010.M

SEE DWG No. GR4



STOCKPILED MATERIALS MUST BE LOCATED OUTSIDE OF THE RIPARIAN ZONE AND SECURED IN A MANNER THAT WILL PROHIBIT SEDIMENT TRANSPORT TO THE WATERCOURSE DURING PRECIPITATION EVENTS. REFUELLING OF VEHICLES/EQUIPMENT TO BE CONDUCTED A SAFE DISTANCE FROM THE WATERCOURSE TO PROHIBIT TRANSPORT OF CONTAMINANTS TO THE CREEK.



KEYPLAN
N.T.S.

NOTES:

1. FOR LOT GRADING DETAILS SEE DRAWING M1.
2. FOR BENCHMARKS AND GENERAL NOTES SEE INDEX DRAWING.
3. TOPOGRAPHIC BASE INFORMATION PREPARED BY YATES & PURCELL LIMITED AND IS BASED ON TOPOGRAPHIC SURVEY COMPLETED DECEMBER 24, 2001.
4. ALL SLOPING TO BE 3:1 UNLESS OTHERWISE SPECIFIED.

LEGEND

F	FRONT DRAINING LOT
B	SPLIT DRAINING LOT
*	ENGINEERED FILL LOTS
200.0	EXISTING GROUND CONTOUR
209.74	EXISTING ELEVATION
(207.04)	TEMPORARY ELEVATION
	EXISTING FENCE
	EXISTING DITCH
	EXISTING TREES
	EXISTING SURFACE FEATURE
	TREES TO BE PRESERVED
207.04	PROPOSED ELEVATION
	PROP. SWALE
	PROP. MANHOLES
	PROP. CB
	PROP. DCB
	PROP. H&V
	PROP. V&C
206	PROP. CONTOUR
	HGL
△	RISER PLUG TO BE INSTALLED PER DETAIL M1

No.	DESCRIPTION	DATE	BY	TOWN APP.
13.	DELETED LOOKOUT OPPOSITE BONHEUR RD PER TRCA	3 FEB 05	M.A.E.	S.A.
12.	REV SLOPING ALONG BETTY ROMAN BLVD PER TRCA	29 JAN 05	M.A.E.	
11.	REVISED PER TRCA COMMENTS	10 JAN 05	M.A.E.	
10.	REVISED PER REGION OF YORK COMMENTS	25 OCT 04	M.A.E.	S.A.
9.	ISSUED FOR ABOVE GROUND APPROVAL	15 OCT 04	M.A.E.	
8.	REVISED NOTE IN LEGEND RE: TOPOGRAPHIC SURVEY	16 SEPT 04	M.A.E.	
7.	3rd SUBMISSION	7 JULY 04	T.R.	
6.	SECOND SUBMISSION	13 APR 04	M.A.E.	
5.	GRADING REVISIONS PER TOWN COMMENTS	2 APR 04	M.A.E.	
4.	ISSUED FOR AREA GRADING	4 FEB 04	M.A.E.	
3.	FIRST SUBMISSION	8 JAN 04	M.A.E.	
2.	PRELIMINARY – ISSUED FOR ROUGH GRADING	17 DEC 03	T.R.	
1	PRELIMINARY GRADING FOR EARTHWORKS	NOV. 11/03	TR	

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
B.T. BURKE
PROVINCE OF ONTARIO

Cathedral Town Ltd. and
404 Developments Inc.
Cathedraltown - Phase 1

GRADING PLAN

Marshall Macklin Monaghan
PROJECT MANAGERS · ENGINEERS · SURVEYORS · PLANNERS

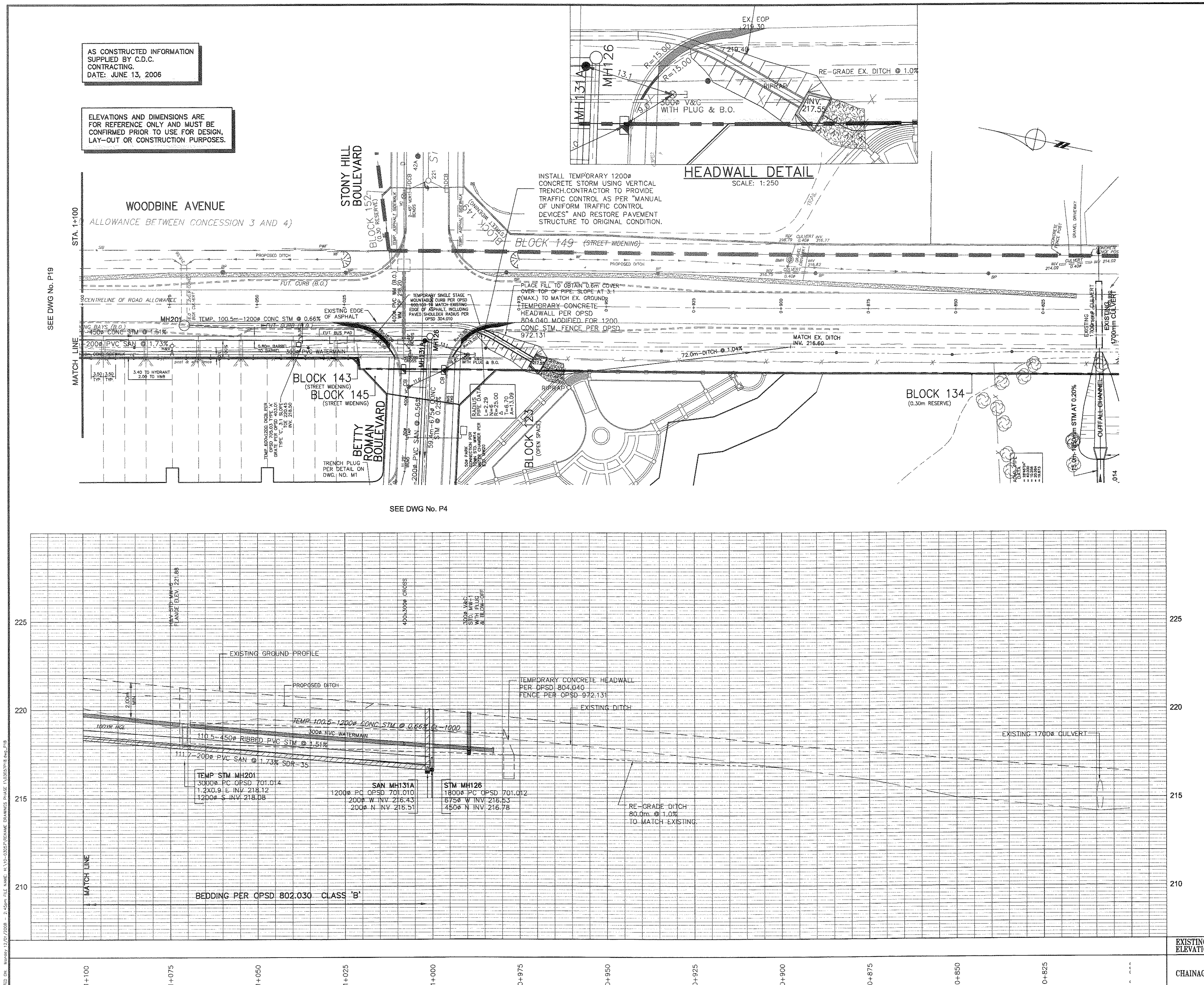
DESIGN BY:	T.Q.	CHECKED BY:	M.A.E.	PROJECT No. 10-03057
DRAWN BY:	10/12 CAD	CHECKED BY:	T.R.	
SCALES:	1:500	DATE:	OCTOBER 2003	

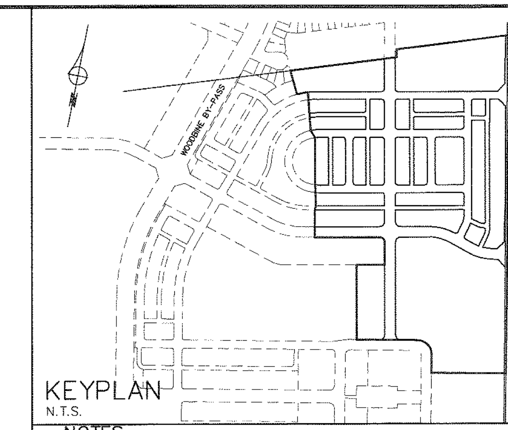
Approved as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.
STEVE ANDREWS — 30 NOV/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

DRAWING No. GR5

Approved underground services as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification.
STEVE ANDREWS — AUG. 20/04
Steve Andrews, P.Eng., Manager Of Engineering — Date

TOWN FILE No. XXXX PHASE 1
D19-03.010.M





NOTES

1. FOR ALL CONSTRUCTION DETAILS NOT SHOWN ON PLANS, REFERENCE SHALL BE MADE TO THE DESIGN STANDARDS OF THE TOWN OF MARKHAM, AND REGION OF YORK WITHIN WOODBINE AVE R.O.W.
2. SEE INDEX PAGE FOR ADDITIONAL NOTES.
3. STORM SEWER PIPES UP TO AND INCLUDING 600 mm ø SHALL BE RIBBED PVC (SDR 35) EQUAL TO A.S.T.M. SPECIFICATIONS D-3034 OR LATEST AMENDMENT UNLESS OTHERWISE NOTED.
4. ALL CONCRETE SEWER PIPES 675mmø AND OVER SHALL BE EQUAL TO C.S.A.SPECIFICATIONS A257-2 CLASS 100D OR LATEST AMENDMENT UNLESS OTHERWISE NOTED.
5. WATER SERVICES TO BE COPPER WITH SADDLE AND ANODE PROTECTION PER STD W-8A - 19mm ø FOR SINGLE AND 25MM ø FOR DOUBLE CONNECTION.
6. ALL POLYVINYL CHLORIDE (PVC) SANITARY SEWER PIPES SHALL MEET CURRENT M.O.E.E. SPECIFICATIONS.
ALL PVC LATERAL SEWER PIPES (SDR 28) SHALL BE EQUAL TO A.S.T.M. SPECIFICATIONS D-3034 OR LATEST AMENDMENT UNLESS OTHERWISE NOTED.
ALL PVC GRAVITY SEWER PIPES (SDR 35) SHALL BE EQUAL TO A.S.T.M. SPECIFICATIONS D-3034 OR LATEST AMENDMENT UNLESS OTHERWISE NOTED. ALL SEWER SERVICE CONNECTIONS SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE MODIFIED SEWER SERVICE CONNECTION DETAIL. SEE DETAIL DWG. M-1
7. ALL MANHOLES SHALL BE PROVIDED WITH MINIMUM 250 mm BENCHING THROUGHOUT
8. ALL PIPES CONNECTED TO MANHOLES TO HAVE TYPE 3 BEDDING (S-42) FROM MANHOLE TO THE FIRST JOINT.
9. STORM AND SANITARY SEWER BEDDING TO BE IN ACCORDANCE WITH STD. S-41, CLASS 'B' UNLESS OTHERWISE NOTED.
10. ALL CATCHBASIN LEADS TO BE ENCASED WITH 15 MPa. CONCRETE PER TOWN STD. S-39.
11. ALL MANHOLE AND CATCHBASIN EXCAVATIONS TO BE BACKFILLED WITH GRANULAR 'B' COMPACTED TO 95% PROCTOR DENSITY.
12. 'MODULOC' OR APPROVED MANHOLE AND CATCHBASIN ADJUSTERS TO BE USED IN LIEU OF BRICKING.
13. P.V.C. WATERMAIN TO BE A.W.W.A. CLASS 150 (DR - 18) WITH TRACER WIRE.
14. ALL CURBS TO BE DEPRESSED AT SIDEWALKS WITH THE EXCEPTION OF LANE ENTRANCES. AT LANE ENTRANCES CURBS SHALL BE TERMINATED AT SIDEWALKS AND SIDEWALKS SHALL BE CONTINUOUS.
15. ALL RESTORATIONS AND RELOCATIONS TO BE COMPLETED TO THE SATISFACTION OF THE DIRECTOR OF ENGINEERING.
16. C.B. WITH INLET CONTROL (28 l/s UNLESS OTHERWISE NOTED) PER TOWN STD MS-21.
17. ALL HYDRANTS TO BE PROVIDED WITH STORZ TYPE PUMPER NOZZLES.
18. ALL STREET CATCHBASIN LEADS TO BE 200ø. DOUBLE CATCHBASIN LEADS TO BE 250ø UNLESS OTHERWISE NOTED.
19. ALL STREET FURNITURE TO CLEAR DRIVEWAYS BY 1.20m MIN.
20. FOR BENCHMARKS SEE INDEX SHEET.
21. FOR LEGEND SEE INDEX SHEET.
22. WATERMAINS IN FILL AREAS TO BE INSTALLED WITH RESTRAINING JOINTS. IN THE FILL AREAS ALL WATERMAINS TO BE BEDDED WITH 150mm OF SAND BACKFILL BELOW INVERT AND 300mm ABOVE OBVERT. IN NON-FILL AREAS WATERMAIN BEDDING PER STD. W-12.
23. WATERMAIN MATERIAL TO MEET TOWN OF MARKHAM MATERIAL SPECIFICATION DATED MAY 1997 OR LATEST EDITION.
24. ALL WATERMAIN FITTINGS TO BE COVERED WITH DENSO PASTE AND WRAPPED IN DENSO TAPE.
25. SHOP DRAWINGS SHALL BE PREPARED BY THE CONTRACTOR AND SUBMITTED TO THE CONSULTANT FOR REVIEW PRIOR TO CONSTRUCTION FOR ALL 3000ø AND 2400x3000 PRECAST MANHOLES AND FOR ALL CAST IN PLACE MANHOLES.
26. TOP OF CONCRETE ON ALL VALVE CHAMBER FLAT CAPS TO BE SET 350mm BELOW GUTTER ELEVATION
27. REFER TO DWG. No. M-3 FOR LANE ENTRANCE CURBING DETAILS.
28. VALVE CHAMBER DRAINS TO BE CONNECTED TO STORM SEWER SYSTEM WHEREVER POSSIBLE. IN AREAS WHERE A STORM SEWER DOES NOT EXIST, THE CONTRACTOR SHALL CONSTRUCT A DRYWELL AS DETAILED ON DWG. No. M1.
29. DRIVEWAYS WITHIN LANE NOS. 1, 4, 5, 19, 20 AND 21 TO BE CONSTRUCTED IN ACCORDANCE WITH THE PAVEMENT STRUCTURE NOTED IN THE GENERAL NOTES ON THE INDEX DRAWING.
30. ALL WATER BOXES SHALL BE INSTALLED IN SUCH A WAY THAT THEY DO NOT CONFLICT WITH FUTURE DRIVEWAYS. IN CASE OF CONFLICT, THE BUILDER SHALL BE RESPONSIBLE FOR RELOCATING THE WATERBOX OUT OF THE DRIVEWAY.
31. HYDRANTS TO BE CONSTRUCTED AS PER TOWN STD. MW-6.
32. WATERMAIN BEDDING AS PER STD. MW-12.
33. THRUST BLOCKING FOR WATERMAIN TO BE AS PER STDS MW-16.
34. ALL WATERMAIN TO HAVE MINIMUM 2.0m COVER.
35. PROVIDE 0.30m CLEARANCE BETWEEN INVERT OF WATERMAIN & CROWN OF SEWER WHEN WATERMAIN CROSSES OVER SEWER AND 0.50m CLEARANCE BETWEEN THE INVERT OF THE SEWER & THE CROWN OF THE WATERMAIN WHEN THE WATERMAIN CROSSES UNDER THE SEWER.
36. 100 YEAR HYDRAULIC GRADE LINE IS WITHIN THE PIPE UNLESS SHOWN OTHERWISE BY 100yr HGL
37. MINIMUM BASEMENT ELEVATION TO BE 0.50m ABOVE OBVERT OF STORM SEWER OR 100 YEAR HGL, WHICHEVER IS GREATER.
38. INSULATE WATERMAIN AT ALL CATCHBASINS IN ACCORDANCE WITH DETAILS ON DRAWING M1.

65M-3830

No.	DESCRIPTION	DATE	BY	TOWN APP.
9.	AS CONSTRUCTED COMPLETE	OCT 2008	B.T.B.	
8.	AS CONSTRUCTED UNDERGROUNDS	DEC 06	B.T.B.	
7.	DELETED VC @ STA 1+015 PER TOWN COMMENT	29 OCT 04	M.A.E.	S.A.
6.	REVISED PER REGION OF YORK COMMENTS	25 OCT 04	M.A.E.	
5.	ISSUED FOR ABOVE GROUND APPROVAL	15 OCT 04	M.A.E.	
4.	REVISED TRENCH PLUGS PER TOWN	16 AUG 04	M.A.E.	
3.	THIRD SUBMISSION	7 JULY 04	T.R.	
2.	SECOND SUBMISSION	13 APR 04	M.A.E.	
1.	FIRST SUBMISSION	8 JAN 04	M.A.E.	

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
B.T. BURKE
PROVINCE OF ONTARIO

Cathedral Town Ltd. and
404 Developments Inc.
Cathedraltown - Phase 1

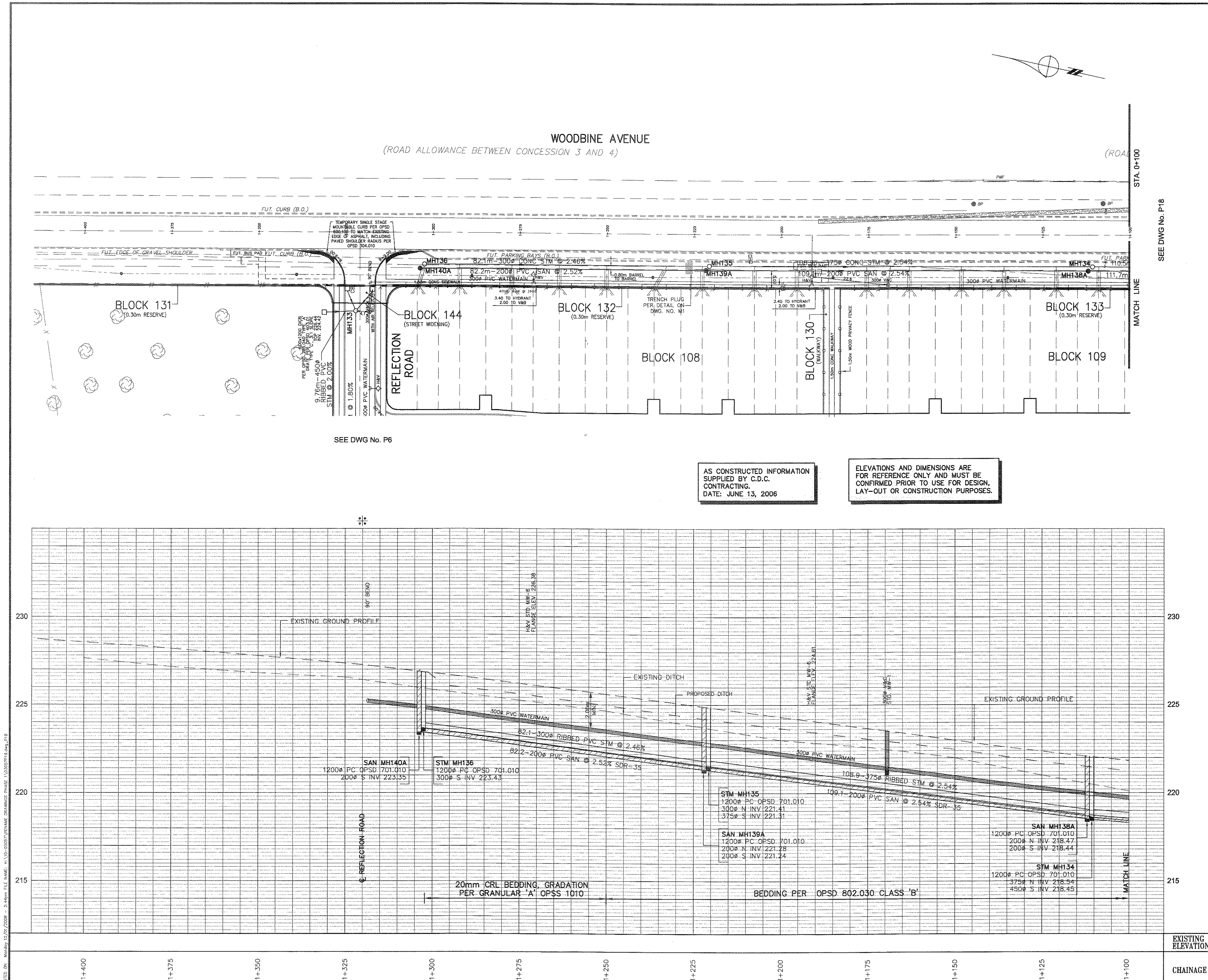
WOODBINE AVENUE

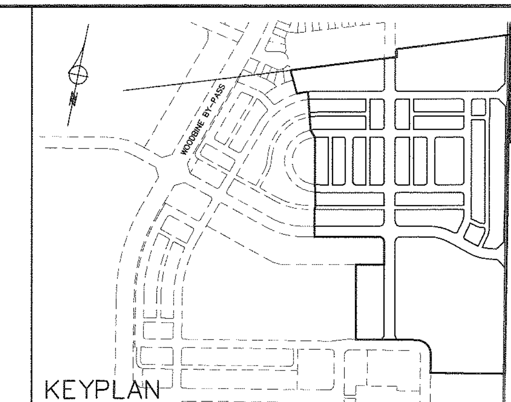
STA. 0+805.99 TO STA. 1+100

Marshall Macklin Monaghan
PROJECT MANAGERS · ENGINEERS · SURVEYORS · PLANNERS

DESIGN BY: T.Q.	CHECKED BY: M.A.E.	PROJECT No. 10-03057
DRAWN BY: 10/12 CAD	CHECKED BY: T.R.	
SCALES: HOR. 1:500 VER. 1:100	DATE: OCTOBER 2003	
Approved as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification. STEVE ANDREWS — Steven Andrews, P.Eng, Manager of Engineering Development	30 NOV/04 Date	DRAWING No. P18

TOWN FILE No.1022 PH1





KEYPLAN
N.T.S.

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65M-3830

No.	DESCRIPTION	DATE	BY	TOWN APP.
9.	AS CONSTRUCTED COMPLETE	OCT 2008	B.T.B.	
8.	AS CONSTRUCTED UNDERGROUNDS	DEC 06	B.T.B.	
7.	DELETED VC @ STA. 1+310 PER TOWN COMMENTS	29 OCT 04	M.A.E.	S.A.
6.	REVISED PER REGION OF YORK COMMENTS	25 OCT 04	M.A.E.	
5.	ISSUED FOR ABOVE GROUND APPROVAL	15 OCT 04	M.A.E.	
4.	REVISED TRENCH PLUGS PER TOWN	16 AUG 04	M.A.E.	
3.	THIRD SUBMISSION	7 JULY 04	T.R.	
2.	SECOND SUBMISSION	13 APR 04	M.A.E.	
1.	FIRST SUBMISSION	8 JAN 04	M.A.E.	

REVISIONS

MARKHAM

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

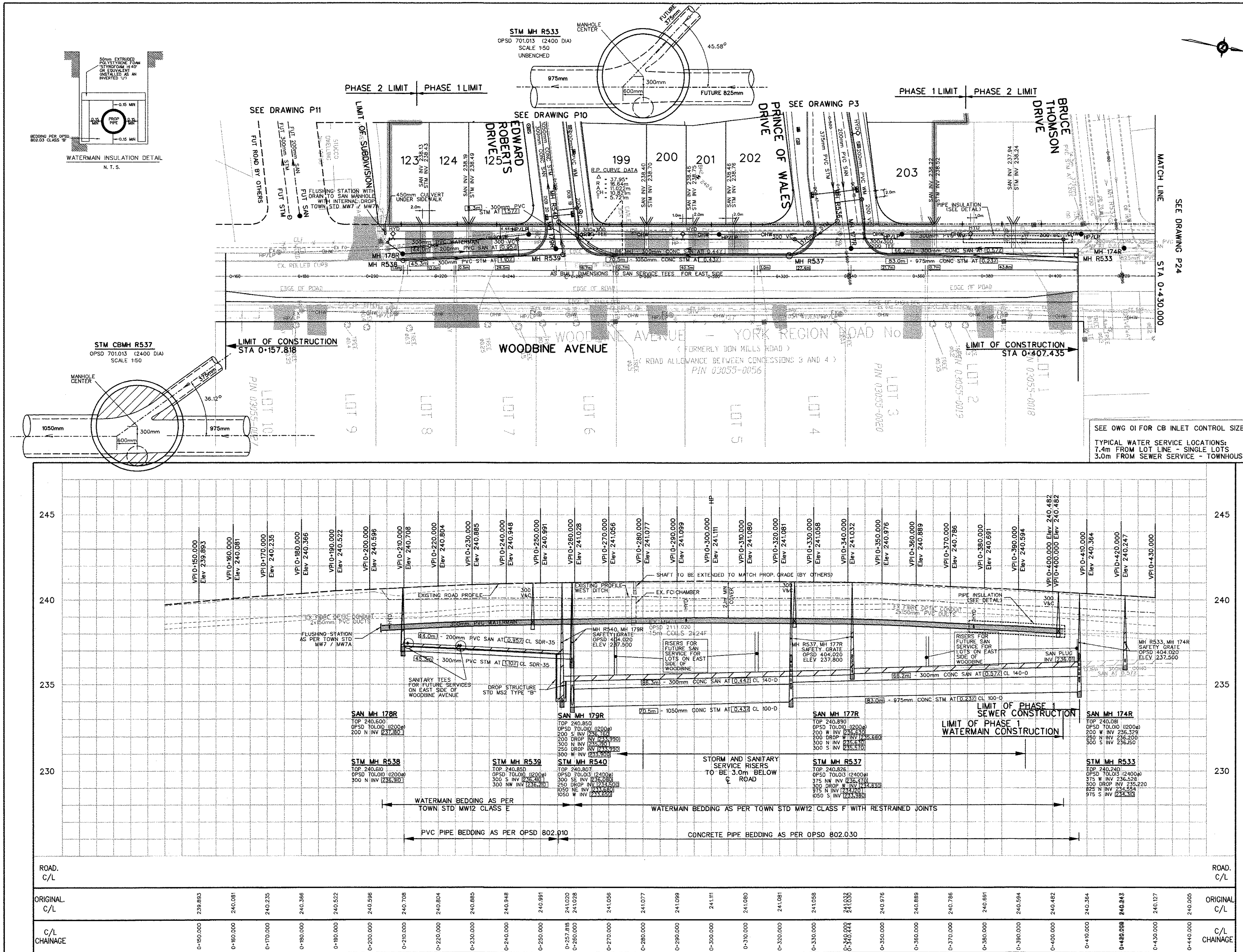
LICENSED PROFESSIONAL ENGINEER
B.T. BURKE
PROVINCE OF ONTARIO

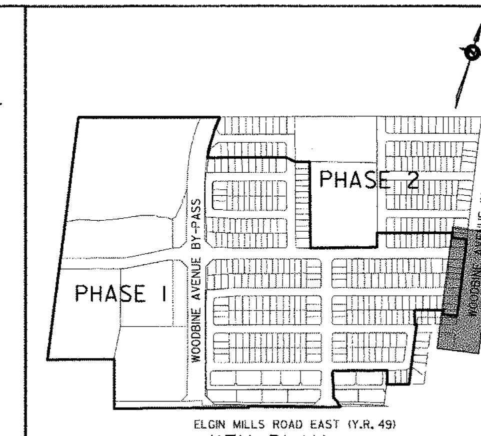
Cathedral Town Ltd. and
404 Developments Inc.
Cathedraltown - Phase 1
WOODBINE AVENUE
STA. 1+100 TO STA. 1+400

Marshall Macklin Monaghan
PROJECT MANAGERS · ENGINEERS · SURVEYORS · PLANNERS

EXISTING ELEVATIONS	DESIGN BY: T.Q.	CHECKED BY: M.A.E.	PROJECT No. 10-03057
	DRAWN BY: 10/12 CAD	CHECKED BY: T.R.	
	SCALES: HOR. 1:500 VER. 1:100	DATE: OCTOBER 2003	
CHAINAGE	Approved as to form in reliance upon the professional skill and ability of Marshall Macklin Monaghan Consulting Engineers as to design and specification. STEVE ANDREWS — Steven Andrews, P.Eng, Manager of Engineering Development	30 NOV/04 — Date	DRAWING No. P19

TOWN FILE No.1022 PH1





ELGIN MILLS ROAD EAST (Y.R. 49)

KEY PLAN

N.T.S.

BENCHMARK

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO TOWN OF MARKHAM MONUMENT No. M-11-003

ELEVATION 236.786m

TWO STOREY BRICK HOUSE ON WEST SIDE OF WOODBINE AVE 39.6m S OF JUNCTION OF WOODBINE AVE AND ELGIN MILLS ROAD IN THE HAMLET OF VICTORIA SQUARE AND 27.9m WEST OF CENTRELINE OF WOODBINE AVE. TABLET SET HORIZONTALLY IN S FACE OF STONE FOUNDATION, 27cm WEST OF SOUTHEAST CORNER AND 30cm BELOW BRICKWORK.

REFER TO DRAWING N1 FOR GENERAL NOTES

LEGEND

- WATERMAIN AND HYDRANT LOCATION
- DOUBLE STORM AND TWO SINGLE SANITARY SERVICE
- SINGLE STORM AND SINGLE SANITARY SERVICE
- STREET LIGHTS
- BELL PEDESTAL
- T.V. CABLE PEDESTAL
- TRANSFORMER
- WATER BOX LOCATION
- SANITARY MANHOLE
- STORM MANHOLE
- CATCH BASIN - NO ICD
- CATCH BASIN - TYPE 'B' ICD = 28.3 L/S
- CATCH BASIN - TYPE 'D' ICD = 54.0 L/S
- VALVE CHAMBER
- 233.15 — MIN. BASEMENT ELEVATION OR SUMP PUMP TO CONNECT TO STORM SERVICE
- PHASE I LIMIT
- 300 W INV 239.642 — AS-CONSTRUCTED INFORMATION

AS-CONSTRUCTED

No.	DESCRIPTION	TOWN	DATE	BY
2	AS-CONSTRUCTED INFO AS PER TOWN COMMENTS		SEPT. 13, 2012	
1	AS-CONSTRUCTED DRAWINGS		SEPT. 24, 2007	

REVISIONS

MARKHAM the MARK of EXCELLENCE

THE CORPORATION OF THE TOWN OF MARKHAM
ENGINEERING DEPARTMENT

HERITAGE AT VICTORIA SQUARE
MONARCH CORPORATION
WOODBINE AVENUE
STA 0+160.000 TO 0+430.000

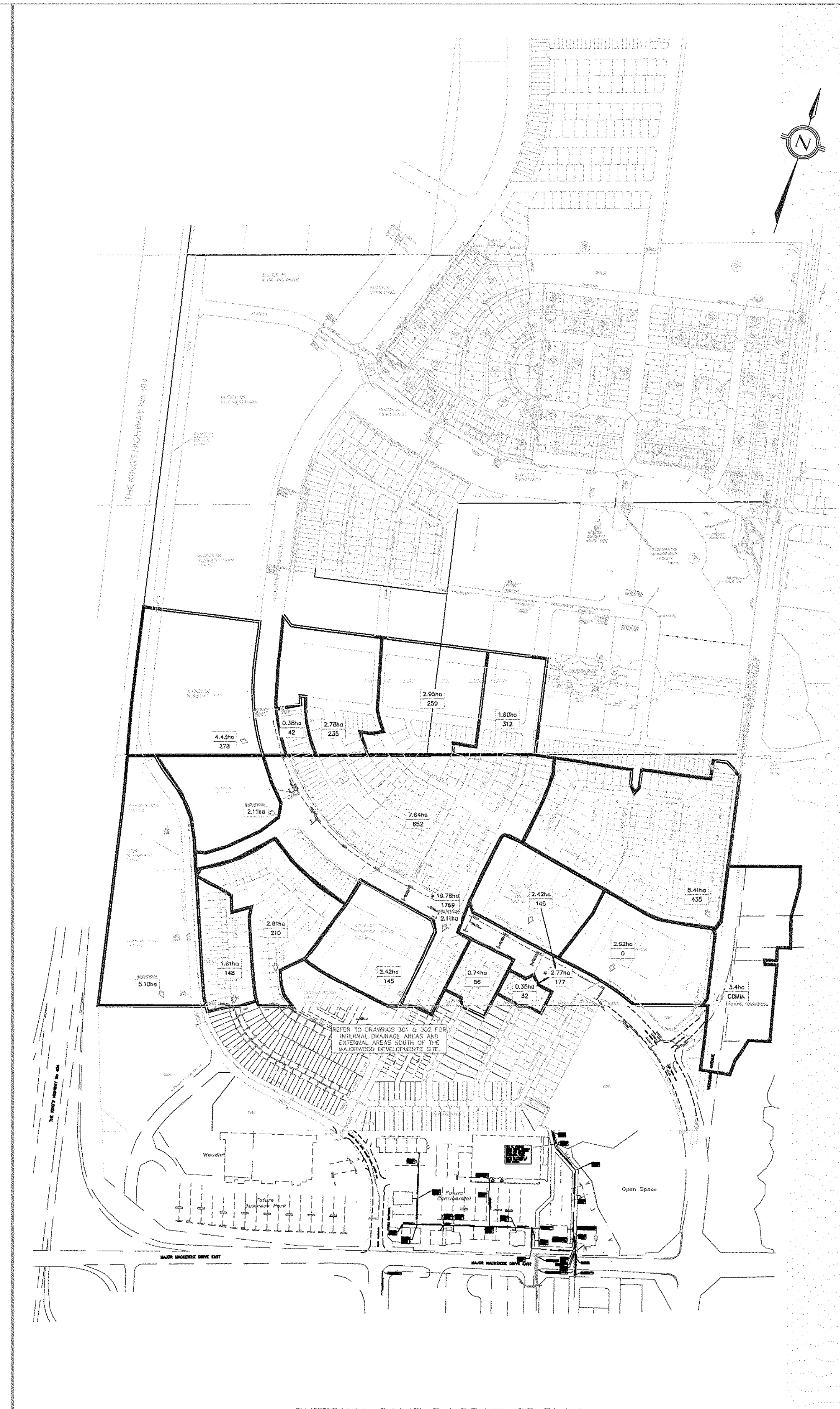
Cole Engineering Group Ltd.
100 Renfrew Drive - Suite 100 - Markham - Ontario - L3R 9R6
Phone: (416) 987-6161, (905) 940-6161 Fax: (905) 940-2064

DESIGN BY: AR/KA	DRAWN BY: RM/LMR/MJC	PROJECT No. M03-115
SCALE: 1:500 (HORIZ) 1:100 (VERT)	DATE: FEBRUARY/2006	
Approved as to form in reliance upon the professional skill and ability of Cole Engineering Group Limited, Consulting Engineers, as to design and specification. Orig signed by Brian Lee, P.Eng. — Brian Lee, P.Eng. Manager of Development Engineering	August 28, 2007 — Date	DRAWING No. P23
		FILE No.: 1025

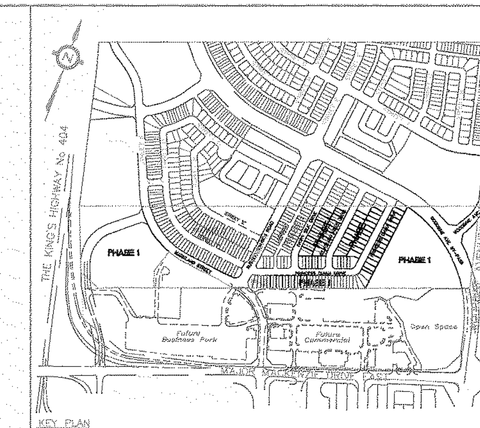
PHASE 1 APPROVAL ONLY

D19-06.007.M

EXTERNAL STORM DRAINAGE PLAN (POST DEVELOPMENT)



EXTERNAL SANITARY DRAINAGE PLAN



KEY PLAN
NTS

BENCHMARK: ELEV. 212.010m

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO THE MTO BENCHMARK No. 00819698479 HAVING AN ELEVATION OF 212.01 METRES. LOCATED ON TWO AND ONE-HALF STOREY WHITE STUCCO HOUSE ON EAST SIDE OF LESLIE STREET, 0.3 KM SOUTH OF STOUFFVILLE ROAD AND 16.5 M EAST OF CENTRELINE OF LESLIE STREET. TABLET IS SET HORIZONTALLY IN NORTH FACE OF STONE FOUNDATION, 15 CM EAST OF NORTHWEST CORNER AND 61 CM BELOW STUCCO WORK.

STORM LEGEND

- DRAINAGE AREA BOUNDARY
- 1.14ha — DRAINAGE AREA (ha)
- 0.50 — RUNOFF COEFFICIENT
- TC=10.50 — TIME OF CONCENTRATION
- MAJOR SYSTEM FLOW DIRECTION
- * COMBINED DRAINAGE AREA

SANITARY LEGEND

- DRAINAGE AREA BOUNDARY
- 2.22ha — CATCHMENT AREA (ha)
- 222 — TOTAL POPULATION
- * COMBINED DRAINAGE AREA

AS RECORDED

No.	DESCRIPTION	DATE	BY	APPROVED
1.	AS RECORDED SUBMISSION	24/02/09	M.D.	

REVISIONS

MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
H-F.H. WONG
PROVINCE OF ONTARIO

MAJORWOOD DEVELOPMENTS INC.
LAKEVIEW HOMES INC. (Phase 1)

EXTERNAL DRAINAGE PLANS

Stantec Consulting Ltd.
7270 WOODBINE AVE., SUITE 300
MARKHAM, ONTARIO L3R 4B9
TELEPHONE: (905) 474-0455
FAX: (905) 474-9889

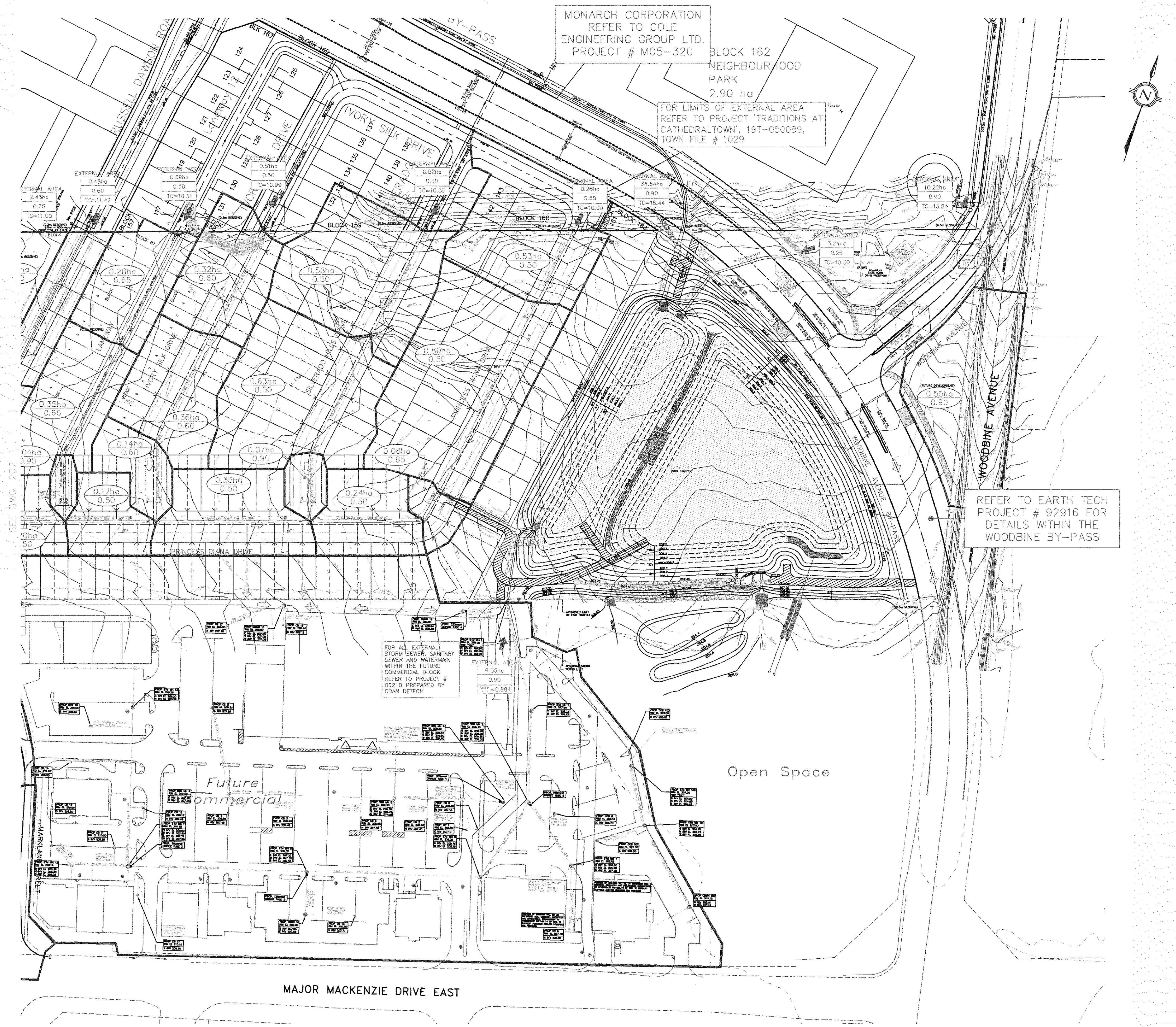
DESIGN BY: M.D. / P.M.	CHECKED BY: H.W.	PROJECT No.
DRAWN BY: M.D.	CHECKED BY: P.M.	160621169
SCALE : 1:4000	DATE: AUGUST 2006	

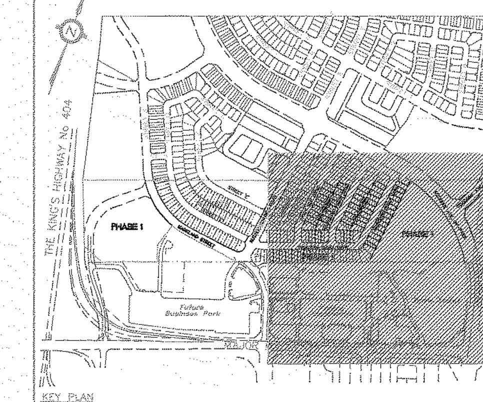
Approved as to form in reliance upon the professional skill and ability of Stantec Consulting Engineers as to design and specification.

ORIGINALS SIGNED
Brian Lee, P.Eng.
Manager of Development Engineering

JULY 27, 2007
Date

DRAWING No.
200





BENCHMARK: ELEV. 212.010m

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO THE MTO BENCHMARK No. 00819698479 HAVING AN ELEVATION OF 212.01 METRES. LOCATED ON TWO AND ONE-HALF STOREY WHITE STUCCO HOUSE ON EAST SIDE OF LESLIE STREET, 0.3 KM SOUTH OF STOUFFVILLE ROAD AND 16.5 M EAST OF CENTRELINE OF LESLIE STREET. TABLET IS SET HORIZONTALLY IN NORTH FACE OF STONE FOUNDATION, 15 CM EAST OF NORTHWEST CORNER AND 61 CM BELOW STUCCO WORK.

LEGEND

- DRAINAGE AREA BOUNDARY
- LIMIT OF PHASE 1
- SINGLE SERVICE CONNECTION
- DUAL SERVICE CONNECTION
- STORM SEWER AND MANHOLE
- SINGLE CATCHBASIN
- DOUBLE CATCHBASIN
- 0.32ha — DRAINAGE AREA (ha)
- 0.50 — RUNOFF COEFFICIENT

EXTERNAL AREA
- 1.14ha — DRAINAGE AREA (ha)
- 0.50 — RUNOFF COEFFICIENT
- TC=10.50 — TIME OF CONCENTRATION
- MAJOR SYSTEM FLOW DIRECTION
- TEMPORARY DEAD END BARRICADE

INLET CONTROLS FOR ROAD LOCATIONS ONLY

- NO ICD REQUIRED
- CB EQUIPPED WITH-19.8 l/s ICD (TYPE'A')
- CB EQUIPPED WITH-28.3 l/s ICD (TYPE'B')
- CB EQUIPPED WITH-36.8 l/s ICD (TYPE'C')

AS RECORDED

No.	DESCRIPTION	DATE	BY	APPROVED
1.	AS RECORDED SUBMISSION	24/02/09	M.D.	

REVISIONS

MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
H-F.H. WONG
PROVINCE OF ONTARIO

MAJORWOOD DEVELOPMENTS INC.
LAKEVIEW HOMES INC. (Phase 1)

STORM DRAINAGE PLAN 1

Stantec Consulting Ltd.
7270 WOODBINE AVE., SUITE 300
MARKHAM, ONTARIO L3R 4B9
TELEPHONE: (905) 474-0455
FAX: (905) 474-9889

DESIGN BY: M.D. / P.M.	CHECKED BY: H.W.	PROJECT No. 160621169
DRAWN BY: M.D.	CHECKED BY: P.M.	
SCALE : 1:1000	DATE: AUGUST 2006	

Approved as to form in reliance upon the professional skill and ability of Stantec Consulting Engineers as to design and specification.

ORIGINALS SIGNED — Brian Lee, P.Eng. Manager of Development Engineering

JULY 27, 2007 — Date

DRAWING No. 201

KEY PLAN
SCALE 1:7500

LEGEND

- EXISTING CONTOUR
- EXISTING STORM MANHOLE
- EXISTING SANITARY MANHOLE
- EXISTING CATCH BASIN
- PROPOSED STORM MANHOLE
- PROPOSED CATCH BASIN/DOUBLE CATCH BASIN
- PROPOSED STORM SEWER
- PROPOSED STORM SERVICE
- PROPOSED DITCH OR SWALE
- PROPOSED 3:1 MAX SLOPE
- PROPOSED MINOR STORM DRAINAGE BOUNDARY
- EXISTING MINOR STORM DRAINAGE BOUNDARY
- FUTURE MINOR STORM DRAINAGE BOUNDARY
- MAJOR OVERLAND FLOW DIRECTION
- MINOR SYSTEM FLOW DIRECTION
- PROPOSED MAJOR STORM DRAINAGE AREA BOUNDARY
- LIMIT OF SUBDIVISION PHASE 2
- EXTERNAL AREA MINOR STORM SYSTEM FLOW DIRECTION

PHASE 1		PHASE 2
0.33ha	DRAINAGE AREA (ha)	0.33ha
0.90	COMPOSITE RUNOFF COEFFICIENT	0.90
2.13	EXTERNAL DRAINAGE AREA (ha) BY OTHERS	0.03
0.90	COMPOSITE RUNOFF COEFFICIENT	0.90

LEGEND (AS-CONSTRUCTED)

PHASE 2 AS-CONSTRUCTED STORM & SANITARY SEWER 10.5m

BENCHMARK
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO CITY ELEVATION 202.911m
CONCRETE BOX CULVERT UNDER MAJOR MACKENZIE DRIVE 81.0m EAST OF EXISTING CENTRELINE OF LESLIE ST TABLET IS SET VERTICALLY ON TOP OF CONCRETE DECK, 25cm WEST AND 15cm SOUTH OF THE NORTHEAST CORNER OF THE CULVERT.

No.	DESCRIPTION	DATE	BY
6.	AS-CONSTRUCTED SUBMISSION TO CITY OF MARKHAM (CITY COMMENTS ADDRESSED)	03/14/2014	NK
5.	AS-CONSTRUCTED SUBMISSION TO CITY OF MARKHAM	03/25/2013	NK
4.	SUBMISSION FOR FINAL APPROVAL	12/01/2010	NK
3.	THIRD SUBMISSION	09/28/2010	NK
2.	SECOND SUBMISSION	08/30/2010	NK
1.	FIRST SUBMISSION	03/24/2010	NK

REVISIONS

MARKHAM
THE CORPORATION OF THE CITY OF MARKHAM
ENGINEERING DEPARTMENT

LICENSED PROFESSIONAL ENGINEER
N. KARAKIS
100037074
Apr 30/14
PROVINCE OF ONTARIO

HERITAGE AT CATHEDRALTOWN PH.2
MONARCH CORPORATION
STORM DRAINAGE AREA PLAN
SHEET 2 OF 4

COLE ENGINEERING

DESIGN BY: RM/SS	DRAWN BY: JM/PT	PROJECT No. M05-320
SCALE: 1:1000	DATE: AUGUST 2010	

Approved as to form in reliance upon the professional skill and ability of Cole Engineering Group Limited, Consulting Engineers, as to design and specification.

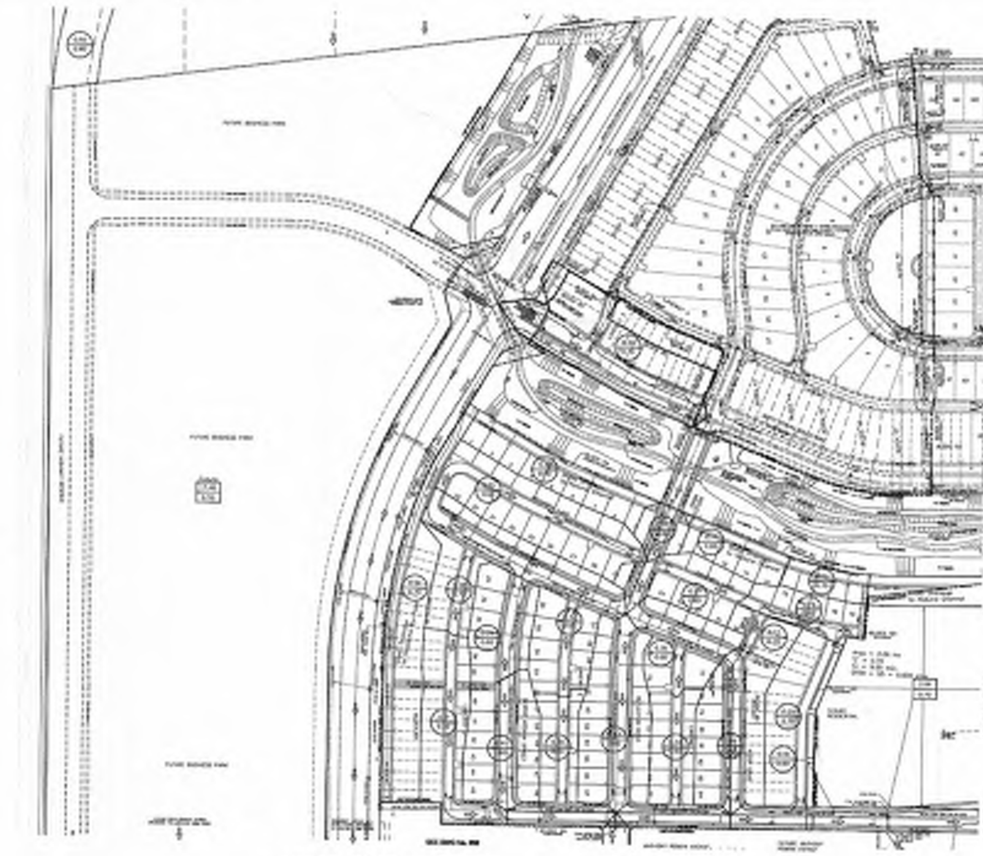
Brian Lee, P.Eng. — Date
Manager of Development Engineering & Transportation

DRAWING No. 202
CITY FILE No.: 1029

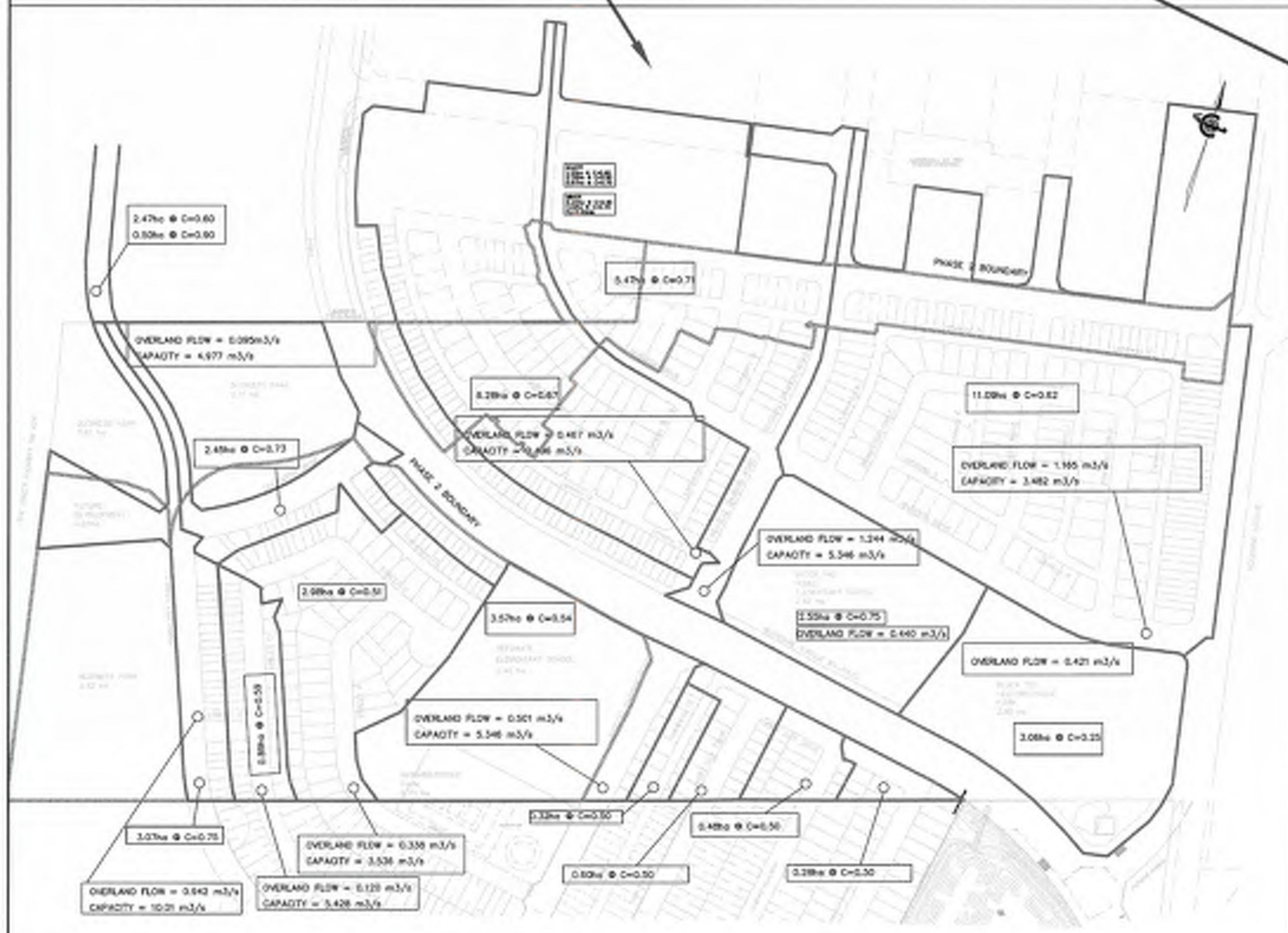
PHASE 2
19TM-05009

SU #: 05 011381
TECH 11 122056

MINOR STORM AREAS
MMM FILE 10-08-011 DWG 203



MINOR STORM AREAS
MMM FILE 10-08-011 DWG 201



MINOR STORM AREAS
MMM FILE 10-08-011 DWG 202

NOTE: INFORMATION SHOWN BASED ON PRELIMINARY PLANS ISSUED BY MMM GROUP AUG 2010, REFLECTING DESIGN STAGE AS OF MARCH 2010.

KEY PLAN
SCALE 1:7500

AS CONSTRUCTED

BENCHMARK
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO CITY ELEVATION 202.911m
CONCRETE BOX CULVERT UNDER MAJOR MACKENZIE DRIVE 81.0m EAST OF EXISTING CENTRELINE OF LESLIE ST TABLET IS SET VERTICALLY ON TOP OF CONCRETE DECK, 25cm WEST AND 15cm SOUTH OF THE NORTHEAST CORNER OF THE CULVERT.

No.	DESCRIPTION	DATE	BY
6.	AS-CONSTRUCTED SUBMISSION TO CITY OF MARKHAM (CITY COMMENTS ADDRESSED)	03/04/2014	AK
5	AS-CONSTRUCTED SUBMISSION TO CITY OF MARKHAM	03/05/2013	AK
4.	SUBMISSION FOR FINAL APPROVAL	12/01/2010	AK
3.	THIRD SUBMISSION	09/08/2010	AK
2.	SECOND SUBMISSION	08/05/2010	AK
1.	FIRST SUBMISSION	03/04/2010	AK

REVISIONS

MARKHAM
THE CORPORATION OF THE CITY OF MARKHAM
ENGINEERING DEPARTMENT

HERITAGE AT CATHEDRALTOWN PH.2
MONARCH CORPORATION
OVERALL STORM DRAINAGE PLAN
SHEET 4 OF 4

COLE ENGINEERING

DESIGN BY: RM/SG	DRAWN BY: JM/PT	PROJECT No. M05-320
SCALE: 1:1000	DATE: AUGUST 2010	

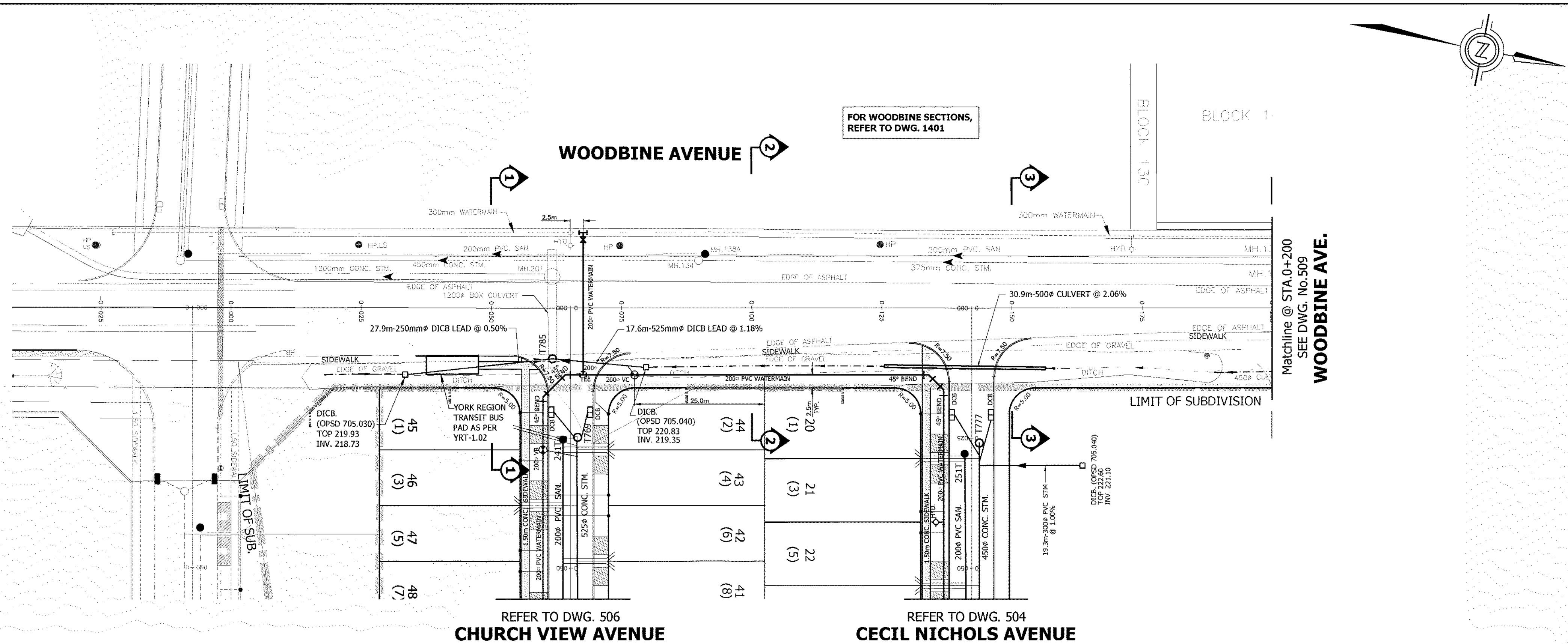
Approved as to form in reliance upon the professional skill and ability of Cole Engineering Group Limited, Consulting Engineers, as to design and specification.

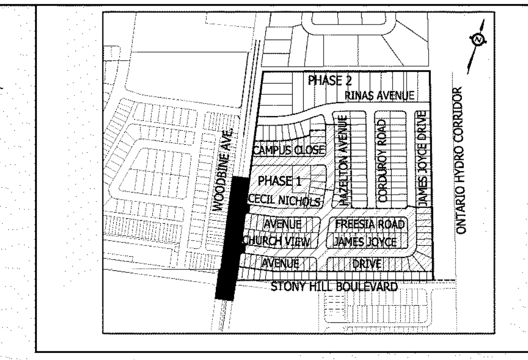
Brian Lee, P.Eng. — Date
Manager of Development Engineering & Transportation

DRAWING No. 204
CITY FILE No.: 1029

PHASE 2
19TM-05009

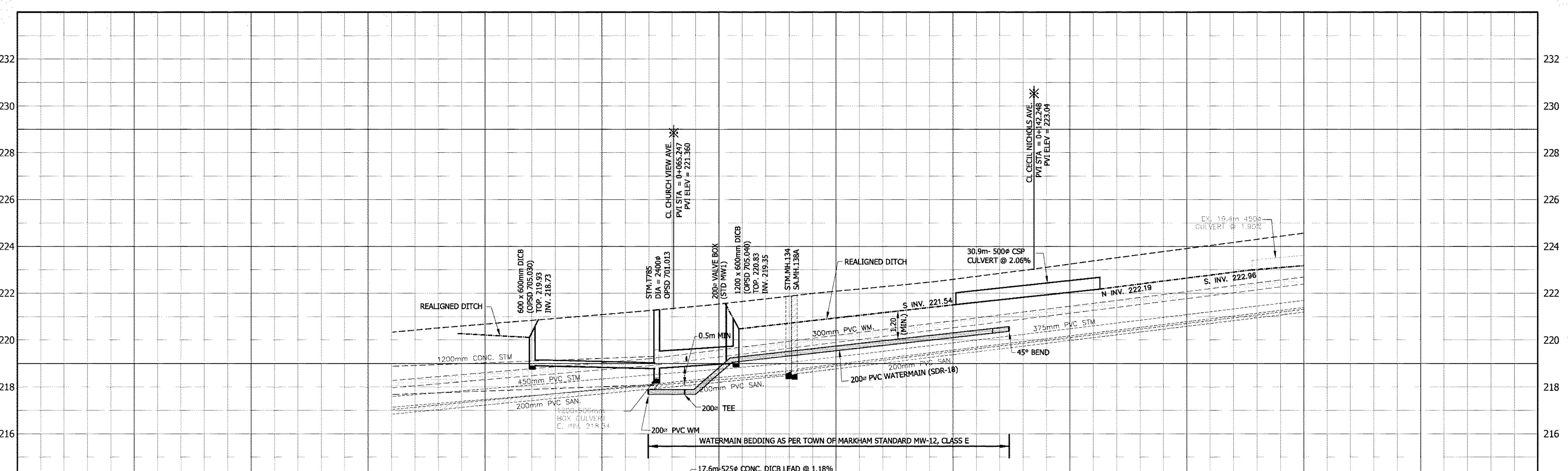
SU #: 05 011381
TECH 11 122056





LEGEND

- STORM SEWER AND MANHOLE
- SANITARY SEWER AND MANHOLE
- SINGLE CATCHBASIN
- DOUBLE CATCHBASIN
- HYDRANT & VALVE
- VALVE CHAMBER
- VALVE BOX
- DENOTES LIGHT POLE
- DENOTES POWER STREAM TRANSFORMER
- DENOTES COMMUNITY MAIL BOX LOCATION
- DENOTES RETAINING WALL
- DENOTES CHAIN LINK FENCE
- DENOTES 1.8m HIGH NOISE FENCE
- 221.40 MINIMUM BASEMENT ELEVATION



STORM SEWER INVERT	N218.938 · S218.798 · W218.480 · S218.798 · S218.955
SANITARY SEWER INVERT	
CENTRELINE ELEVATION	220.335 · 220.682 · 221.088 · 221.540 · 222.088 · 222.596 · 223.248 · 223.914 · 224.572
CENTRELINE CHAINAGE	0+000 · 0+025 · 0+050 · 0+075 · 0+100 · 0+125 · 0+150 · 0+175 · 0+200

ROAD BASE THICKNESS

INTERNAL	150mm COMPACTED DEPTH 50mm CRUSHER RUN LIMESTONE
RESIDENTIAL	100mm COMPACTED DEPTH 20mm CRUSHER RUN LIMESTONE
ROADWAYS	75mm COMPACTED DEPTH HL-8
	40mm COMPACTED DEPTH HL-3

BENCH MARK NO. 458-69, ELEV. 207.672

458-69: FRAME BARN (OWNED BY MR. A. COWLEY) ON THE EAST SIDE OF WOODBINE AVENUE (REGIONAL RD. 8), 1.9km SOUTH OF JCT. OF WOODBINE AVENUE AND 18th AVENUE IN THE HAMLET OF VICTORIA SQUARE, 4.2km NORTH OF JCT. OF HWY. 7 AND WOODBINE AVENUE, 132.9m NORTH OF 17th AVENUE AND 159.7m EAST OF CENTRELINE OF WOODBINE AVENUE. TABLET IS SET HORIZONTALLY IN WEST FACE OF STONE FOUNDATION, 2.83m SOUTH OF N.W. CORNER, 2.13m BELOW FRAMEWORK AND 40cm ABOVE GROUND LEVEL.

AS-CONSTRUCTED

No.	DESCRIPTION	TOWN	DATE	BY
4	ISSUED FOR ACCEPTANCE OF MAINTENANCE		NOV 1 2013	O.S.
3	REPLACED TEMP. FLUSING STA. WITH TEMP. FLUSING HYDRANT & 50mm BLOW OFF ON WOODBINE		SEPT 24 2010	J.O.
2	REVISED WM CROSSING ON WOODBINE TO OPEN CUT		SEPT 24 2010	J.O.
1	RELOCATED WATERMAIN AND WOODBINE		SEPT 24 2010	J.O.

REVISIONS

MARKHAM

TOWN FILE: 2179
19TM-06007
SU 06 108 473

LICENSED PROFESSIONAL ENGINEER J. ORMONDE 100089999 NOV. 1, 2013 PROVINCE OF ONTARIO

MONARCH WALMARK DEVELOPMENT LTD.
EAST CATHEDRAL COMMUNITY
AS-CONSTRUCTED

WOODBINE AVENUE
FROM STA. 0+000 TO STA. 0+200

urbantech
Urbantech Consulting, A Division of Leighton-Zec Ltd.

DESIGN BY: E.T.	CHECKED BY: J.O.	PROJECT No. 06-200
DRAWN BY: J.S.	CHECKED BY: D.Z.	
SCALES: HORZ. 1:500 VERT. 1:100	DATE: OCTOBER 2010	DRAWING No. 508

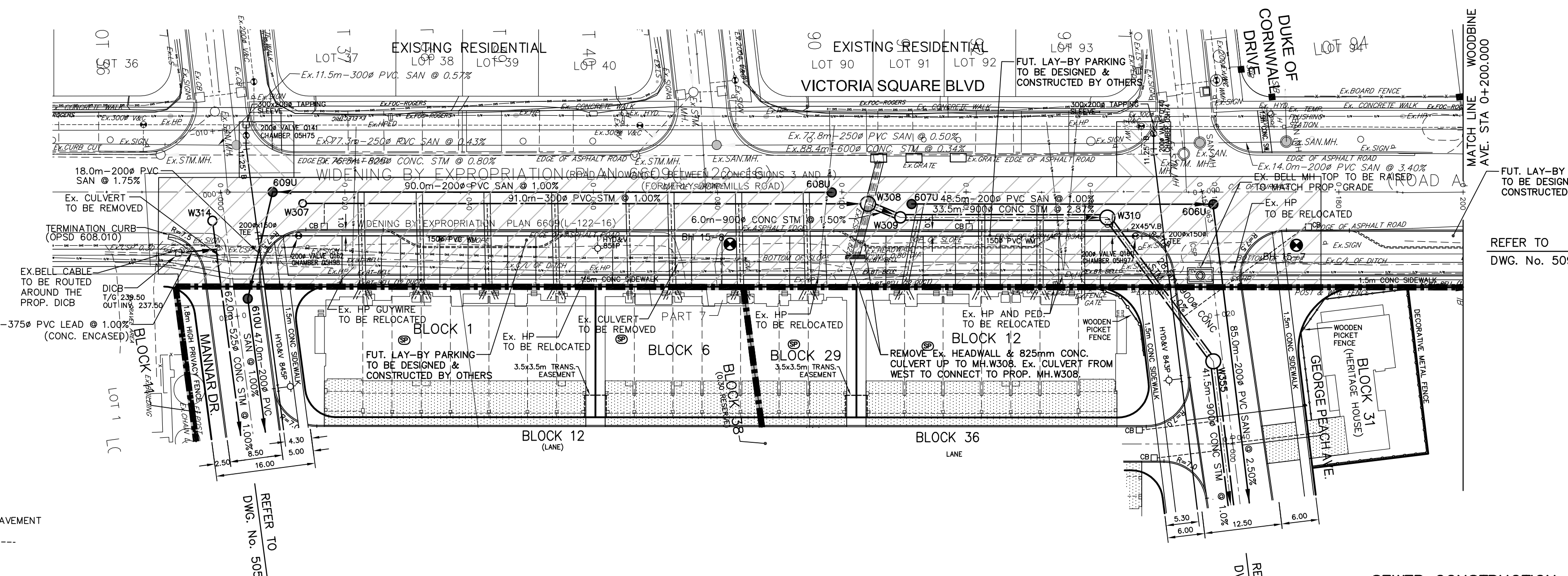
APPROVED AS TO FORM IN RELIANCE UPON THE PROFESSIONAL SKILL AND ABILITY OF URBANTECH CONSULTING LTD. CONSULTING ENGINEERS AS TO DESIGN AND SPECIFICATION.

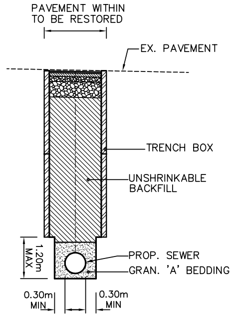
ORIGINALLY APPROVED ON JULY 23, 2010
BRAIN LEE, MANAGER, DEVELOPMENT ENGINEERING & TRANSPORATION — DATE

File: P:\Projects\08-200\Drawings\As Constructed\500 – Plan & Profiles\08-108473 – 508_WOODBINE AVE – PAR.dwg – Revised by <SRIEMER> : Tue, Oct 29 2013 – 11:08am TOWN ENG. FILE No. 2179

SPECIAL NOTES:

a) 100YR HGL IS THE OBVERT OF STORM PIPE UNLESS OTHERWISE NOTED.
b) 100YR HGL IS MINIMUM 0.5 m BELOW BOTTOM OF BASEMENT SLAB ELEVATION.
c) IN CASES WHERE WATERMAIN IS ABOVE NATIVE SOIL, IT SHALL BE CONSTRUCTED IN ENGINEERED FILL WITH RESTRAINED JOINTS AND BEDDING CLASS 'F'.
d) WHERE SUBGRADE IS LOOSE AND SOFT, A GEOWEB CELLULAR CONFINEMENT SYSTEM AS SPECIFIED IN THE GEO-TECHNICAL REPORT SHOULD BE CONSTRUCTED AT THE TRENCH BASE AND THEN CONVENTIONAL BEDDING CAN BE USED ON TOP OF GEOWEB.
e) ENGINEERED FILL SHALL BE IN ACCORDANCE WITH THE RECOMMENDATIONS OF THE GEOTECHNICAL REPORT.
f) ALL EXISTING PLUG INVERTS TO BE VERIFIED ON SITE.
g) DEPTH OF Ex. UTILITIES SHOULD BE VERIFIED ON SITE PRIOR TO SEWER INSTALLATION. IF NECESSARY, Ex. UTILITIES SHALL BE RAISED TO AVOID CONFLICTS WITH PROPOSED WORKS.
h) PRIOR TO CONSTRUCTION COMMENCEMENT PROVIDE A PRE-CONSTRUCTION CONDITIONS SURVEY FOR ALL THE EX. HOMES.





VERTICAL TRENCH AND RESTORATION DETAIL

N.T.S.

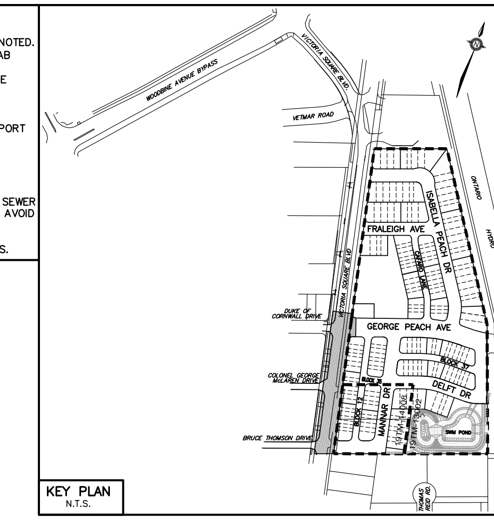
VERTICAL TRENCH BOX SPECIAL NOTES:

1) MAX DEPTH OF VERTICAL TRENCH BOX TO BE 5.0m.
2) WHEN DEPTH OF SEWER EXCEED 5.0m, TOP OF TRENCH TO BE OVER EXCAVATED TO CREATE MIN 5.0m WIDE PLATFORM. THE EXCAVATED MATERIAL SHOULD BE PLACED MIN 7.0m AWAY FROM THE LIMIT OF TRENCH.
3) DE-WATERING SHOULD BE CONDUCT BY EDUCTORS AS PER SOILS AND HYDROGEOLOGY REPORT.
4) DURING EXCAVATION AND TRENCHING SOIL ENGINEER SHOULD BE CONSULTED FOR THE STABILITY AND SAFETY OF THE TRENCH.

SEWER CONSTRUCTION SEQUENCE:

1. SET UP STAGE 1 CONSTRUCTION AREA PER TRAFFIC CONTROL PLAN DWG. No.1303.
2. CONSTRUCT SANITARY SEWER AND STORM SEWER ON WOODBINE AVE BY VERTICAL TRENCH.
3. INSTALL MH.W308 OVER EXISTING 800mm CULVERT. PLUG AND ABANDON EXISTING CULVERT FROM MH.W308 TO EXISTING HEADWALL AND REMOVE HEADWALL.
4. CONSTRUCT SERVICE CONNECTIONS AND WATERMAIN UP TO STAGE 1 LIMIT BY VERTICAL TRENCH.
5. RESTORE ROAD CUTS PER RESTORATION DETAIL ON THIS DRAWING.
6. SET UP STAGE 2 CONSTRUCTION AREA PER TRAFFIC CONTROL PLAN DWG. No.1304.
7. CONSTRUCT REMAINING WATERMAIN FROM STAGE 1 CONSTRUCTION LIMIT TO EXISTING 300mm WATERMAIN.
8. RESTORE ROAD CUTS PER RESTORATION DETAIL ON THIS DRAWING
9. SHAVE 50MM OF SURFACE ASPHALT AND PAVE ALL AREAS DISTURBED (PAVEMENT FROM STREET "D" TO STREET "A" BY THE SEWER CONSTRUCTION AND PROVIDE PAVEMENT MARKING PER DWGs. No.801 & No.802.

DENOTES STAGE 1 CONSTRUCTION
DENOTES STAGE 2 CONSTRUCTION



KEY PLAN
N.T.S.

MINIMUM COLLECTOR ROAD BASE THICKNESS

450mm COMPACTED DEPTH 50mm COMMERCIAL CRUSHER RUN LIMESTONE
150mm COMPACTED DEPTH 20mm COMMERCIAL CRUSHER RUN LIMESTONE
100mm COMPACTED DEPTH HL-8 (2 LIFTS)
50mm COMPACTED DEPTH HL-3

LEGEND:

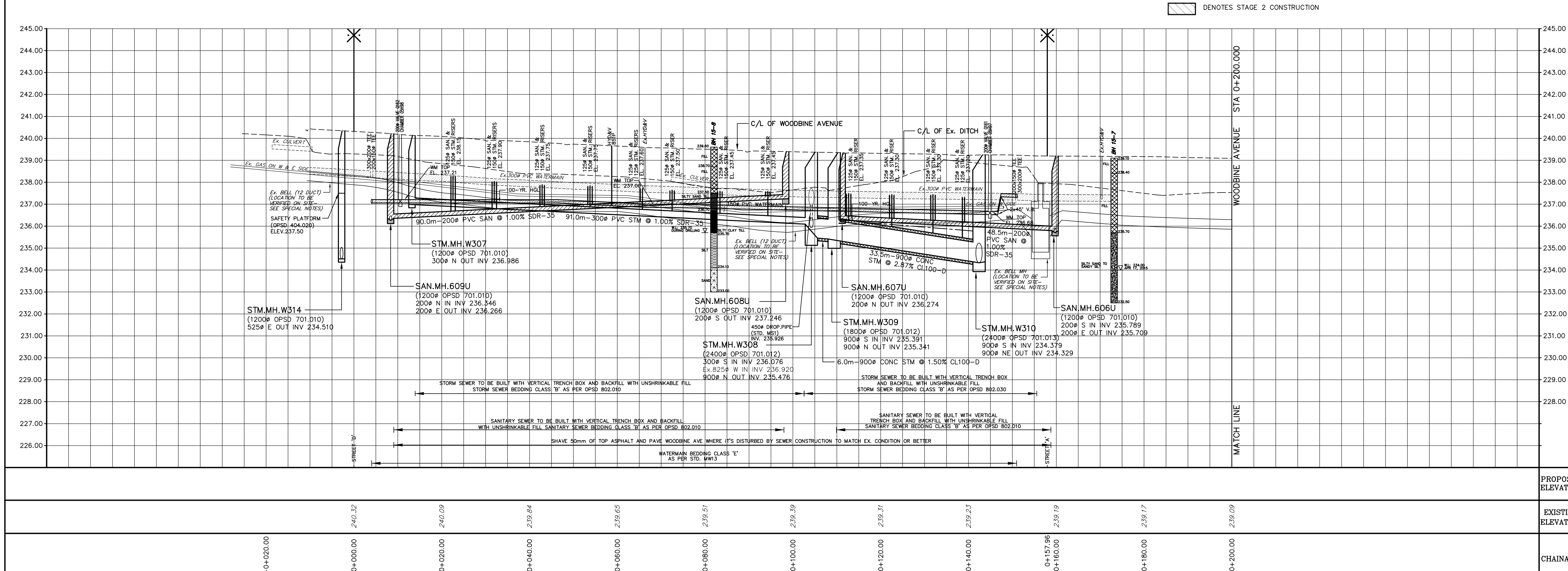
DENOTES WATER SERVICE
DENOTES SINGLE CATCHBASIN
DENOTES DOUBLE CATCHBASIN
DENOTES SANITARY MANHOLE
DENOTES STORM MANHOLE
DENOTES VALVE AND CHAMBER
DENOTES HYDRANTS
DENOTES ASPHALT DRIVEWAY
DENOTES SANITARY & STORM DOUBLE CONNECTION
DENOTES SANITARY & STORM SINGLE CONNECTION
DENOTES 2.0m HIGH ACOUSTIC FENCE
DENOTES UPGRADED WOODEN FENCE
DENOTES 1.8m HIGH PRIVACY FENCE
DENOTES 1.5m HIGH CHAIN LINK FENCE
DENOTES DECORATIVE METAL FENCE
DENOTES WOODEN PICKET FENCE
DENOTES RETAINING WALL
DENOTES LIMIT OF SUBDIVISION
DENOTES BLOCK WITH SUMP PUMPS
DENOTES BLOCK WITH SAN GRINDER PUMPS

NOTES:

1. ALL CONCRETE STORM AND SANITARY SEWER PIPES 300mm DIAMETER AND OVER EQUAL TO C.S.A. SPECIFICATIONS A-257-2 CL65-D OR LATEST AMENDMENT UNLESS OTHERWISE NOTED.
2. ALL SANITARY MANHOLES SERVICING 200mm SEWER LINES SHALL BE PROVIDED WITH MINIMUM 250mm BENCHING THROUGHOUT.
3. STORM MANHOLE BENCHING TO BE FULL, AS PER CITY STANDARDS. THE CONCRETE CHANNEL WALL SHALL BE EXTENDED VERTICALLY FROM THE SPRING LINE TO THE CROWN OF THE PIPE.
4. ALL PIPES ATTACHED TO MANHOLES TO HAVE TYPE 3 BEDDING FROM MANHOLE TO THE FIRST JOINT.
5. STORM AND SANITARY SEWER BEDDING TO BE CLASS "B" UNLESS OTHERWISE NOTED.
6. ALL MANHOLE AND CATCHBASIN EXCAVATIONS TO BE BACKFILLED WITH GRANULAR "B"COMPACTED TO 95% PROCTOR DENSITY.
7. "MODULOC" OR APPROVED MANHOLE AND CATCHBASIN ADJUSTERS TO BE USED IN LIEU OF BRICKING.
8. ALL RLCB TO BE CONCRETE ENCASED ADJACENT TO BUILDING ENVELOPE.
9. ALL PVC LATERAL SEWER PIPES (SDR 28) SHALL BE EQUAL TO A.S.T.M. SPECIFICATIONS D-3034 OR LATEST AMENDMENT UNLESS OTHERWISE NOTED. ALL PVC GRAVITY SEWER PIPES (SDR 35) SHALL BE EQUAL TO A.S.T.M. SPECIFICATIONS D-3034 OR LATEST AMENDMENT UNLESS OTHERWISE NOTED.
10. WATER SERVICES TO BE COPPER WITH SADDLE AND ANODE PROTECTION (0.45kg)
11. WATERMAIN SHALL BE POLYVINYL CHLORIDE (PVC) CLASS 150 (DR-18) CONFORMING TO AWWA C-900 WITH TRACING WIRE PER CITY OF MARKHAM STANDARDS.
12. ALL WATER SERVICES TO BE 19mm DIAMETER AND INSTALLED AS PER STD. MW8, UNLESS OTHERWISE NOTED.
13. ALL WATER BOXES SHALL BE INSTALLED IN SUCH A WAY, THAT THEY DO NOT CONFLICT WITH FUTURE DRIVEWAYS. IN CASE OF CONFLICT, THE BUILDER SHALL BE RESPONSIBLE FOR RELOCATING THE WATER BOX OUT OF DRIVEWAY.
14. HYDRANT TO BE CONSTRUCTED AS PER CITY STD. MW6.
15. ALL HYDRANTS TO BE EQUIPPED WITH STORZ TYPE PUMPER NOZZLES.
16. ALL WATERMAIN IRON FITTINGS TO BE COVERED WITH DENSOPASTE AND WRAPPED IN DENSOTAPE.
17. WATERMAIN BEDDING AS PER CITY STD. MW13.
18. THRUST BLOCKING FOR WATERMAIN TO BE AS PER CITY STD. MW16A.
19. WATERWORKS MATERIAL AS PER SPECIFICATIONS DATED LATEST EDITION.
20. ALL CURBS TO BE DEPRESSED AT SIDEWALKS.
21. ALL RESTORATION AND RELOCATIONS TO BE COMPLETED TO THE SATISFACTION OF THE DIRECTOR OF ENGINEERING.
22. FOR ALL CONSTRUCTION DETAILS NOT SHOWN ON THIS PLAN, REFERENCE SHALL BE MADE TO THE DESIGN STANDARDS AND CONSTRUCTION GUIDELINES OF THE CITY OF MARKHAM.

BENCHMARKS

ELEVATIONS RELATED TO GEODETIC DATUM
1) DERIVED FROM TOWN OF MARKHAM BENCHMARK No. 092073005, HAVING A PUBLISHED ELEVATION OF 227.828 METRES.
2) DERIVED FROM TOWN OF MARKHAM BENCHMARK No. M-12-008, HAVING A PUBLISHED ELEVATION OF 239.520 METRES.



No.	DESCRIPTION	CITY	DATE	BY
6	SUBMISSION TO CITY FOR SIGNING		Apr. 2017	K.Sh.
5	5th SUBMISSION		Aug. 2016	K.Sh.
4	4th SUBMISSION		May 2016	K.Sh.
3	3rd SUBMISSION		Apr. 2016	K.Sh.
2	2nd SUBMISSION		Nov. 2015	K.Sh.
1	1st SUBMISSION		Apr. 2015	K.Sh.

REVISIONS

MARKHAM ENGINEERING DEPARTMENT

REGISTERED PROFESSIONAL ENGINEER H. O. TOZCU Apr.13, 2017 PROVINCE OF ONTARIO

EATON SQUARE DEVELOPMENT RESIDENTIAL SUBDIVISION

PLAN AND PROFILE

VICTORIA SQUARE BLVD

(FROM STA. 0+000.000 TO STA. 0+200.000)

SCHAEFFERS CONSULTING ENGINEERS

DESIGN BY: P.Ch.	CHECKED BY: K.S.	PROJECT No. 2014-4052
DRAWN BY: V.B.	CHECKED BY: H.T.	
SCALE: HORZ. 1:500 VERT. 1:100	DATE: FEB. 2014	DRAWING No. 508

ACCEPTED AS TO FORM IN RELIANCE UPON THE PROFESSIONAL SKILL AND ABILITY OF SCHAEFFER & ASSOCIATES LTD. CONSULTING ENGINEERS AS TO DESIGN AND SPECIFICATION.

REZA FANI, P.ENG. MANAGER OF DEVELOPMENT ENGINEERING DATE

TECH 15 113916
TECH 15 136727

19TM-13002
19TM-14008

Environmental & Stormwater Management Plan

West Cathedral Community

URS

Cole Sherman

MARKHAM
the MARK of EXCELLENCE

Gartner Lee Limited

Superseded

~~October 24, 2002~~
~~CN30702080~~
~~(33014172)~~

Revised January 2006

* X Section Number	Existing Condition	Proposed Condition
205.31	216.73	216.73
205.32	216.74	216.74
205.33	216.74	216.72

* Refer to Figure 4.7 for location

4.6.5 Carlton Creek Drainage Improvements

The proposed stormwater management plan includes the construction of a channel extending from Elgin Mills Road to 70m upstream of Woodbine Avenue. The new channel would be based on natural channel design principles. A detailed landscape plan would also be implemented to enhance the ecological function of the drainage corridor. This new channel will replace the depressional draw that currently exists.

From Elgin Mills Road to the intersection of Street 50 and Woodbine Bypass, the new channel will convey only runoff from the proposed stormwater management facility to be located immediately north of Elgin Mills Road. Downstream of the Woodbine Bypass, the channel would convey also the major system flow from the lands to the north. As described in Section 4.6.3.2, this additional flow would, at a downstream location, be directed to the stormwater management pond identified as E3 on Figure 4.3 for attenuation.

The proposed natural channel design section for the study reach was designed following Rosgen's approach to natural channel design. Based on a proposed channel slope of 0.5%, a local soil type of silt-clay (CPM, 1996) and a 2 year design flow of 0.8 m^3/s, a C6 stream was selected as the most appropriate type for the study reach. A typical channel section would have a 0.5m bottom width, 4:1 vegetated side slopes and a depth of 0.55m. The low flow channel would meander within a valley bottom width of 15m. Step pools would be provide periodically along the system to maintain the design slope. The proposed channel would be located within a 35 m drainage easement. A plan of the proposed channel is provided in Figure 4.8. Detailed calculations in support of the selected design are given in Appendix E.

There are four crossings of the proposed channel. At each of these crossings a 8.5 m span by 1.5 m Con/Span structure is proposed. A typical section of the proposed structure is given in Figure 4.9. Based on the HEC-2 simulations these structures are capable of handling the 100-year flow without overtopping. Two cross-sections for the channel are given in Figures 4.11 and 4.12.

4.7 Mitigation Impacts Associated with SWMP

4.7.1 Quality Control

The permanent pool and extended detention volumes provided for quality control in each of the stormwater management ponds, as summarized in Table 4.4, were based on the storage volumes for wet ponds as identified in the MOEE, 1994 document (Table 6.1). In the design, deep pools should be provided at both the inlet (sediment forebay) and outlet structures for improved growth of emergent vegetation and sediment control, and to mitigate thermal impacts on the receiving watershed from the stored runoff. The volumes to be provided are based on MNR Level 1 fisheries protection and the land use and catchment areas as identified on Figure 4.3. It is recognized that as specific details are made available, volume requirements as summarized in Table 4.4 would have to be refined.

4.7.2 Quantity Control Benefits

All stormwater management facilities being proposed would be designed to provide sufficient quantity control to limit, as a minimum, post-development flows to pre-development levels for all events up to and including the 100-year design storm. In order to complete the analysis the developed QUALHYMO model was modified to include adequate storage to attenuate peak outflows to predevelopment levels for the 2- to 100-year design storm events. Specific design details such as slopes, inlet and outlet structure details, landscaping, site location and size would be identified at the draft plan submission stage.

The established quantity control volumes and release rates and quantity control volumes for the various design storms are summarized in Table 4.4. A condensed QUALHYMO output is included in Appendix B.

Results of the analysis confirmed that there would be no significant impacts on the flood hazard condition associated with the recipient drainage systems.

Table 4.3

SWMP	WATERSHED	CATCHMENT AREA (ha)		TYPE OF FACILITY	COMMENTS
		EXISTING	PROPOSED		
	Berczy Creek	13.49	10.0	Uncontrolled Release	• Release of flows greater than 5 year up to 100 year storm to be restricted to predevelopment rates • No release for storms up to 5-year event
E4	Carlton Creek East Branch	49.0	60.0	Wetpond	Facility to provide erosion, quality and quantity control for all residential areas. Employment lands to be quantity controlled for runoff in excess of the 5-year storm
E3	Carlton Creek East Branch	85.0	80.5	Wetpond	Floodplain and channel to be realigned to accommodate pond layout Facility to provide erosion, quality and quantity control for all residential areas. Employment lands to be quantity controlled for runoff in excess of the 5-year storm
W1	Carlton Creek West Branch	71.0	100.0	Wetpond	• Facility to provide erosion, quality and quantity control for all residential areas. Employment lands to be quantity controlled for runoff in excess of the 5-year storm
	Applewood Creek	8.0	8.0	Uncontrolled Release	• Uncontrolled release as per previously accepted reports (CPM 1996)

Table 4.4

Summary of Facility Performance Characteristics

SWM Pond	Design Storm	Pre-Dev	Post-Dev		
		Release Rates (cms)	Release Rates (cms)	Pond Volume (m^3)	Pond Depth (m)
E4	Permanent Pool	-	-	5,300	2.00
	Erosion Control	0.47	0.47	6,811	2.56
	2-Year	0.82	0.82	8,748	2.71
	5-Year	1.24	0.82	11,137	2.89
	10-Year	1.65	0.82	14,714	3.15
	25-Year	2.28	0.82	18,151	3.39
	100-Year	3.45	0.82	19,919	3.51
E3	Permanent Pool	-	-	6,365	2.00
	Erosion Control	0.86	0.86	8,975	2.60
	2-Year	1.52	1.52	11,372	2.75
	5-Year	2.32	1.52	15,675	3.01
	10-Year	3.03	1.52	20,171	3.27
	25-Year	4.10	1.52	25,234	3.55
	100-Year	6.05	1.52	28,993	3.75
W1	Permanent Pool	-	-	14,150	2.00
	Erosion Control	0.76	0.76	20,270	2.95
	2-Year	1.36	1.36	23,740	3.10
	5-Year	2.07	1.36	24,678	3.14
	10-Year	2.72	1.36	26,332	3.21
	25-Year	3.70	1.36	27,763	3.27
	100-Year	5.49	1.36	36,358	3.62

4.7.3 Erosion Assessment

The Toronto Regional Conservation Authority has adopted the criteria for erosion protection for the subject site as retention of the 30 mm rainfall event for a 40 hour time period. To provide the required protection, storage volumes of 320 m^3/impervious hectare would have to be provided at SWM ponds. The average release rates and storage capacities are summarized in Table 4.4.

4.7.4 Temporary Erosion Control

To ensure stormwater quality control during construction, erosion and sediment measures will be carried out prior to any site-disturbing activities. The following works may be included:

Stormwater Management Pond Design Report
Interim Pond B3

404 North Secondary Plan (OPA 149)
Town of Markham

For

404 North Developers Group

August 2008

MASONGSONG ASSOCIATES ENGINEERING LIMITED
1151 Denison Street • Unit 15
Markham, Ontario • L3R 3Y4
T: (905) 944-0162 F: (905) 944-0165
maeng@maeng.ca
www.maeng.ca

MAEL PROJECT: 07331

The proposed development must comply with the servicing and stormwater management guideliness outlined in *the Master Servicing Study and Stormwater Management Study for 404 North Secondary Plan Area (OPA149)*, November 2007, (MSS) prepared by SCS Consulting Group Ltd.

The *MMS* describes an ultimate Pond B3 required as a communal end-of-pipe stormwater management wet-pond facility located to the east of the subject site and north of proposed Woodbine Bypass, designed to treat the minor and major stormwater runoff from the central portion of the study area of approximately 66 ha.

Due to land development phasing, the *MSS* proposes the construction of an Interim Pond B3 which provides the required level of treatment to the first phases of development while avoiding encroaching into non-participating properties. Refer to Figures 3.1, 3.2 and 4.1 of the *SMS* (included in Appendix A) for a location of Ultimate Pond B3 and Interim Pond B3.

This design report provides detailed modelling and design geometry of **Interim Pond B3**. Interim Pond B3 is to provide quality and quantity stormwater controls from the central portion of the study area of approximately 58 ha, also referred to as the West Tributary of Berczy Creek Watershed. The discharge for Interim Pond B3 (referenced as Flow Node C in this report) will be maintained at the same pre-development location: conveyance through an existing 1500mm x 900mm Concrete Box Culvert running west-to-east under Woodbine Avenue.

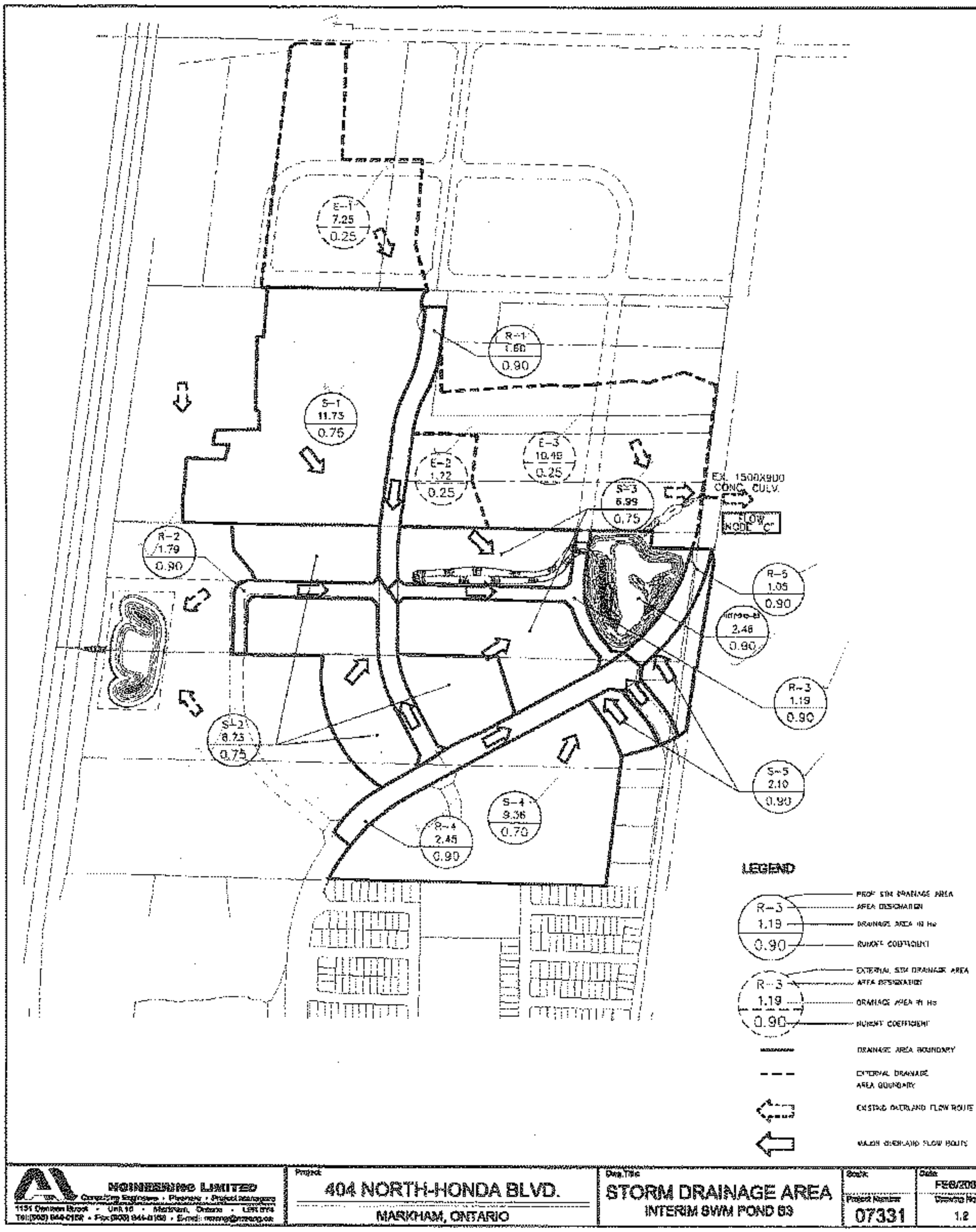
The proposed land use tributary to Interim Pond B3 is primarily industrial park land, with some residential and community amenity uses south of the Woodbine By-Pass. In accordance with Town of Markham and Toronto Region Conservation Authority guidelines, all industrial/commercial sites will require on-site quantity controls to attenuate site runoff to a maximum of 180 L/s/ha during any storm event. As the study area is tributary to Berczy Creek which eventually crosses the Highway 407 (MTO corridor) downstream of the proposed development, the Ministry will only support flow-control measures which are below-grade, require little or no maintenance, and are difficult to remove or alter. This will need to be emphasized through each subsequent site plan development within the Ministry's jurisdictional corridor.

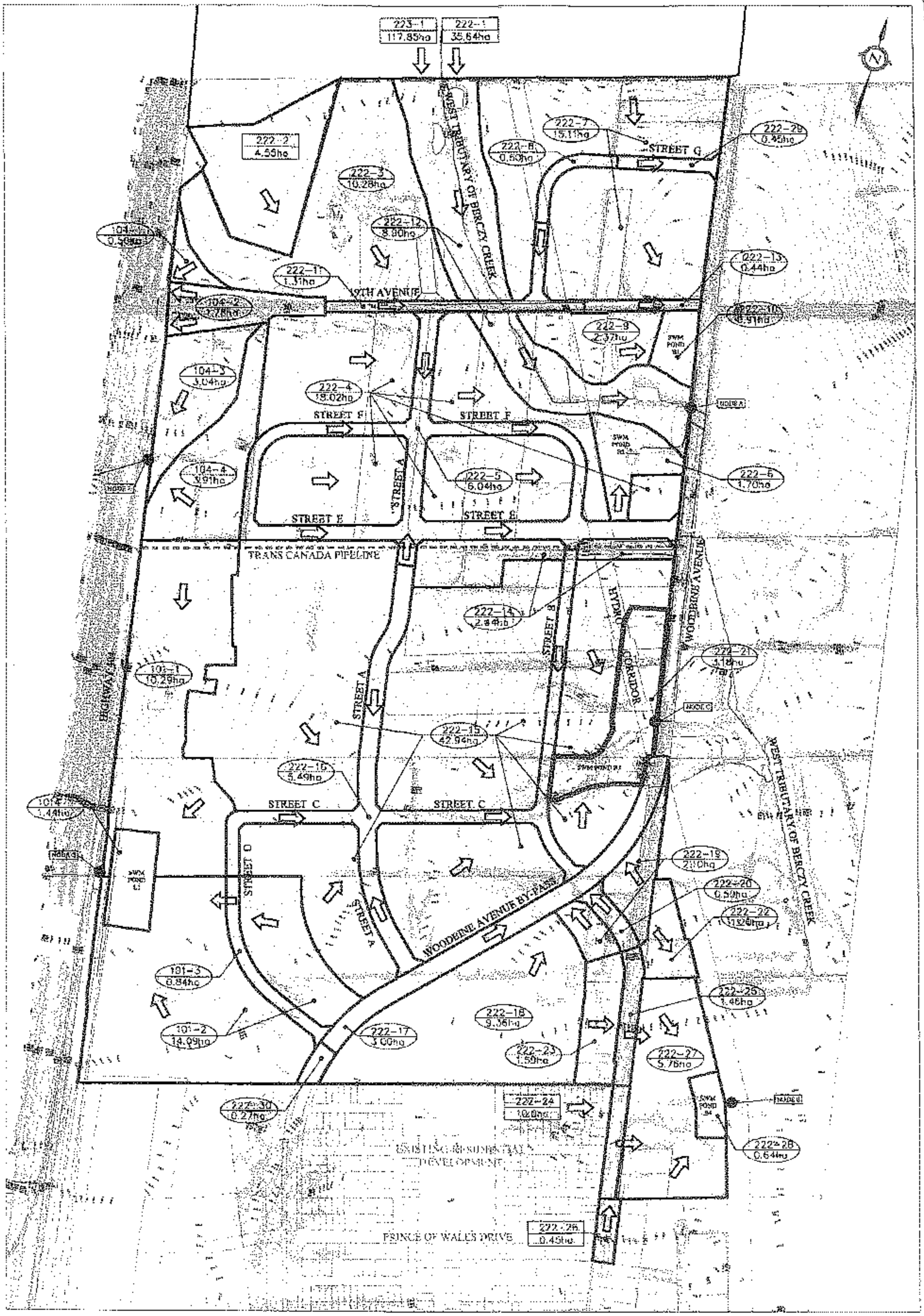
Presently, the Interim Pond B3 tributary area includes 11.73 ha of the Honda Campus site, or approximately the easterly two-thirds of the site plan (refer to Figure 1.2). While the Honda Campus has strived to implement low-impact stormwater quality and quantity controls, MTO does not recognize the quantity attenuation measures derived from bio-swale storage. An uncontrolled post-development scenario was therefore analysed to simulate failure of the on-site bio-swale control system at the Honda Campus, and has been incorporated in the pond sizing and geometry of Interim Pond B3.

Specific criteria to be applied in the Interim Pond B3 design are as follows:

- Extended detention for erosion control- 25 mm event detained with a 48 hr drawdown time

- Water quality control – MOE *Level 1* or *Enhanced* Protection
- Water quantity control to pre-development runoff levels for the 2 through 100 year storm events.





404 NORTH DEVELOPERS GROUP

MARKHAM

SCS

LEGEND:

STUDY BOUNDARY

PROPOSED DRAINAGE BOUNDARY

DRAINAGE DIRECTION

222-2
10.28ha
CATCHMENT ID
PROPOSED DRAINAGE AREA (ha)

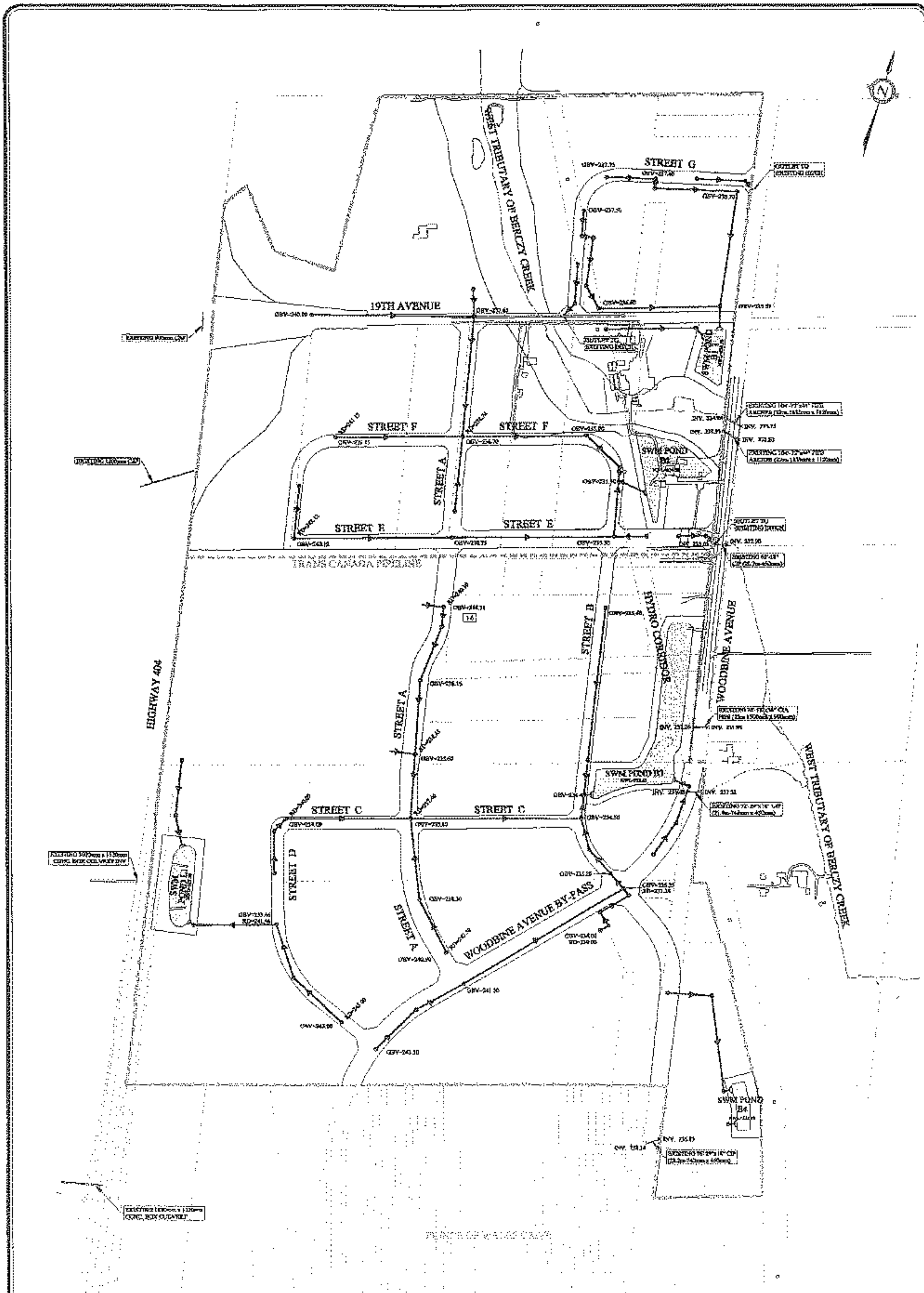
222-1
4.54ha
EXTERNAL CATCHMENT ID
EXTERNAL DRAINAGE AREA (ha)

404 NORTH (OPA 149)
STORMWATER MANAGEMENT STUDY

POST-DEVELOPMENT DRAINAGE PLAN

1120

3.1



[redacted]

404 NORTH DEVELOPERS GROUP

MARKHAM

SCS

LEGEND:

OBV=243.30

RD=243.50

404 NORTH (OPA 149)
STORMWATER MANAGEMENT STUDY

PRELIMINARY STORM SERVICING PLAN

1120

3.2

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WEST CATHEDRAL COMMUNITY

Cathedraltown Phase 3 Development

STORMWATER MANAGEMENT REPORT

Town Of Markham

March 2006

Prepared for:
Fram Building Group

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2.0 STORMWATER MANAGEMENT PLAN

2.1. STORM DRAINAGE PLAN

The lands located within the Cathedraltown Phase 3 Part 2 Development will drain to Pond E3 as per the recommendations identified in the West Cathedral ESMP. The lands associated with Phase 3 are located for the most part within Basin 2056 of the previously developed QUALHYMO model. A small area of residential lands located at the northeast corner of the Woodbine Avenue Bypass and Betty Roman Boulevard, which was located in Basin 2056 of the original ESMP, is also included in the Phase 3 Part 1 development. Although the southerly catchment area associated with Basin 2055 has been refined based on the results of the detailed sewer design for Phase 3 Part 1 as completed by Marshal Macklin Monaghan (MMM), the total catchment area to Pond E3 remains unchanged. The refined catchment limit associated with Basin 2056 is identified on Figure 2.

Details of the proposed storm sewer system for Phase 3 a developed by MMM are provided in Appendix A. The information provided includes a plan view of the system and the storm sewer design calculations. The proposed storm sewer for Phase 3 Part 1 would require the construction of a new outlet to Pond E3. It is to be located at the southwest corner of the existing facility. This is consistent with the recommendations of the ESMP. The location of the storm sewer outlet and the proposed sediment forebay are shown on Figure 3.

Major system flow from the Phase 3 Part 1 development will follow the roadways and outlet to Pond E3 at the same location as the proposed storm sewer outfall. An intake structure would be required within the roadway to intercept the surface runoff and safely convey it to the quantity cell of Pond E3. This could be accomplished by the construction of a new conveyance system that would be designed to intercept and convey the 100-year minus the 5-year flow. Alternatively, it could be combined with the 5-year minor storm sewer system. If the systems were combined than a flow splitter would be required within the pond block to direct the 5-year flow to the sediment forebay and the excess to the quantity cell.

It is noted that depending on the final road grades, the storm sewer system to service Phase 3 Part 1 may have to be designed to capture the entire 100 year flow from localized areas within the Phase 3 development limits. This is a result of potential grading constraints associated with the roadway design and the cut and fill requirements of the site. Alternatively, the 100-year major system over land flow from the lands that could not be drained by gravity to Pond E3 could be directed southerly, to SWM Pond W1. That pond would, however, have to be designed to accommodate the excess runoff. The development schedule of any portion of Phase 3 that does not have its minor and major system flows attenuated within Pond E3 would be dependent on the timing of construction associated with Pond W1. For the current analysis, the pond has been designed to accept both the minor and major system flows from the entire Phase 3 Part 1 development area. This represents the worst-case scenario for the design of SWM Pond E3.

Pond E3 constructed in 2004 was based on satisfying the requirements of the entire development upstream as identified in the West Cathedral ESMP. The outlet control structure was partially constructed plus the north sediment forebay. To accommodate Phase 3 Part 1 the southerly forebay as identified on Figure 4 would have to be constructed which includes the access road. This is in addition to the new storm sewer minor and major systems outflow pipes. The pond outlet control structure would also have to be finalized as per the requirement of the ultimate design.

2.2. IMPACT ASSESSMENT

The impacts of the Phase 3 development on the peak outflows and the effectiveness of the SWM Pond E3 in attenuating the peak outflows to the design release rates as identified in the MESP were assessed based on the use of Rational Method and the QUALHYMO model. The Rational Method which was used to size the proposed storm sewer system (refer to Appendix A for plan and support calculations) confirmed that a 1.5m diameter pipe would be required to convey the 5-year flow from Phase 3 Part 1 lands to the sediment forebay.

As previously noted the construction of Phase 3 Part 1 of Cathedraltown requires that Pond E3 be retrofitted to include a second sediment forebay. This forebay is to be designed specifically to accommodate the 5-year flow from Subcatchment 2056. Based on the QUALHYMO model the 5-year outflow is 3.1 cubic metres. Based on the settling/dispersion equations as provided in the MOEE Planning and Design Manual, the required forebay length is 79 metres. The length provided as shown on Figure 3 is 80 m. The support calculations are provided in Appendix B.

To assess the impacts of the proposed Phase 3 Part 1 development on Pond E3 the existing QUALHYMO model, which included Phases 1 and 2 of the King David Development, was modified to include a more detailed discretization for the Phase 3 lands. As shown on Figure 4, the Phase 3 catchment area has been refined to include a total of five subcatchment areas. Utilizing the developed model, rainfall runoff simulations were performed for the 2 to 100 year storm events. Results of the analysis as summarized in Table 1 confirmed that under the 100-year storm event the maximum increase in the pond level would be 1.6m (Elev. 216.6m). The top of the flood control berm associated with the pond is at an elevation of 217.5m. The total storage utilized in the pond is 25,390 cu.m., which is significantly less than the available storage of 40,530 cu.m.. With development of Phases 1,2 and 3 Part 1 and implementation of the proposed stormwater management plan, the peak outflows from Pond E3 ranged from 0.24 cu.m. to 1.54 cu.m. for the 2 to 100 year events respectively. These flows are less than the maximum permissible release rates as defined in the MESP, which ranged from 1.54 cu.m. to 6.34 cu.m. for the 2 to 100 year events respectively. A comparison of the release rates, storage volumes utilized within the pond and depth of ponding is provided in Table 1. The performance characteristics of Pond E3, which includes the stage/storage/discharge curves, are given in Appendix C.

The potential impacts of the proposed stormwater management plan on the East Carleton Creek peak flows at Woodbine Avenue are provided in Table 2. In summary, with the implementation of the proposed plan the peak flows downstream of the subject site would be less than that of the existing condition. The 100 Year QUALHYMO output and a schematic of the model are included in Appendix B.

The existing stormwater management wetpond facility is designed to provide erosion control to protect the downstream reaches of East Carleton Creek. Results of the analysis confirmed that the drawdown time for Phases 1, 2 and 3 Part 1 would be in the order of 65 hours.

The proposed wet pond also provides quality control. For Phases 1 through 3 inclusive the required permanent pool volume is estimated to be 6750 m^3. The permanent pool volume provided in the pond in the interim design is approximately 17000 m^3.

3.0 SUMMARY

The Cathedraltown Phase 3 Part 1 development is located within the catchment area associated with Pond E3. The proposed stormwater management plan, which directs all runoff to Pond E3 for quality and quantity control, is therefore consistent with the recommendations of the West Cathedral Community Plan ESMP.

Results of the detailed hydrologic and hydraulic calculations as described in this technical report confirmed that the quality and quantity control requirements of the Phase 3 Part 1 development can be adequately addressed by the existing wetpond facility. New minor and major system outflow sewers would however be required in order to safely convey the runoff to the pond. A new sediment forebay would also have to be constructed at the south limit of the facility in order to treat the minor system flows from Phase 3.

The benefits associated with the modified Pond E3 facility are identified in Table 1. In summary it was found that the existing wetpond facility provides the required storage to attenuate the peak outflows associated with the 2 to 100 year design storm events to rates less than that of the predevelopment condition.

The peak outflows along the East Carleton Creek at Woodbine Avenue, as summarized in Table 2 would also be less than that of the pre-development condition.

The stormwater requirements associated with Phase 3 Part 2 will be provided for in Pond W1. That pond will service a number of landowners within the West Cathdraltown Community. Specific details of that facility and all support calculations will be provided in a separate document.

Report Prepared By:

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Brian Plazek, P.Eng.
Manager, Water Resources Group

Table 1
POND E3 Performance Characteristic

Stom Event	Ultimate				Phase 3			
	Release	*Storage	Depth	Elevation	Release[2]	*Storage	Depth	Elevation
Erosion[1]	0.38	40500	1.55	216.55	0.19	25500	0.63	215.63
2	1.54	42300	1.64	216.64	0.24	28700	0.85	215.85
5	2.14	46200	1.85	216.85	0.27	32500	1.09	216.09
10	2.92	49200	2.00	217.00	0.30	36210	1.31	216.31
25	4.13	52200	2.15	217.15	0.91	40700	1.56	216.56
50	4.91	54330	2.25	217.25	1.42	41410	1.60	216.60
100	6.34	57600	2.40	217.4	1.54	42460	1.65	216.65

[1] Refers to drawdown time of 40h for 30mm storm, as defined by CPM's 1997 EMSP for the Cathedral Community

[2] Refers to cumulative release from pond.

* Includes a permanent pool storage volume of 17,070 cu.m.

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Table 2
Comparison Of Peak Flows
East Carleton Creek at
Woodbine Avenue

Storm Event	Peak Flow, m^3/s	
	Predevelopment Condition	*Postdevelopment Condition
2	1.68	1.25
5	2.51	1.81
10	3.24	2.29
25	4.34	3.00
50	5.11	3.85
100	6.37	5.16

* Includes King David Phases 1, 2 and 3.

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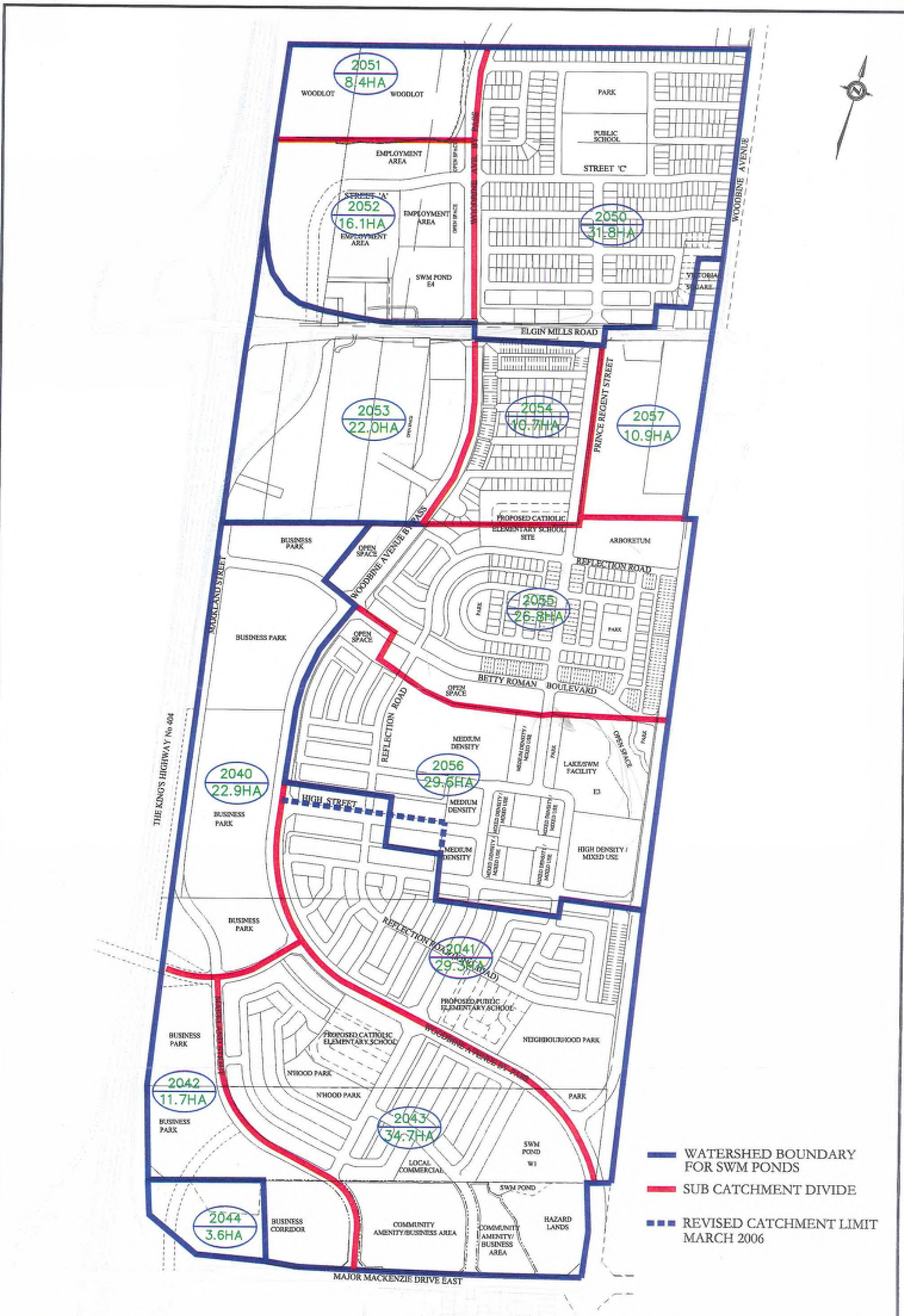

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PROJECT	TITLE		FIGURE No.
ENVIRONMENTAL AND STORMWATER MANAGEMENT PLAN WEST CATHEDRAL COMMUNITY	POST DEVELOPMENT CONDITION WATERSHED DISCRETIZATION	DATE NOV. 2004 SCALE 1:7500 PROJECT No. 02080	FIGURE 2

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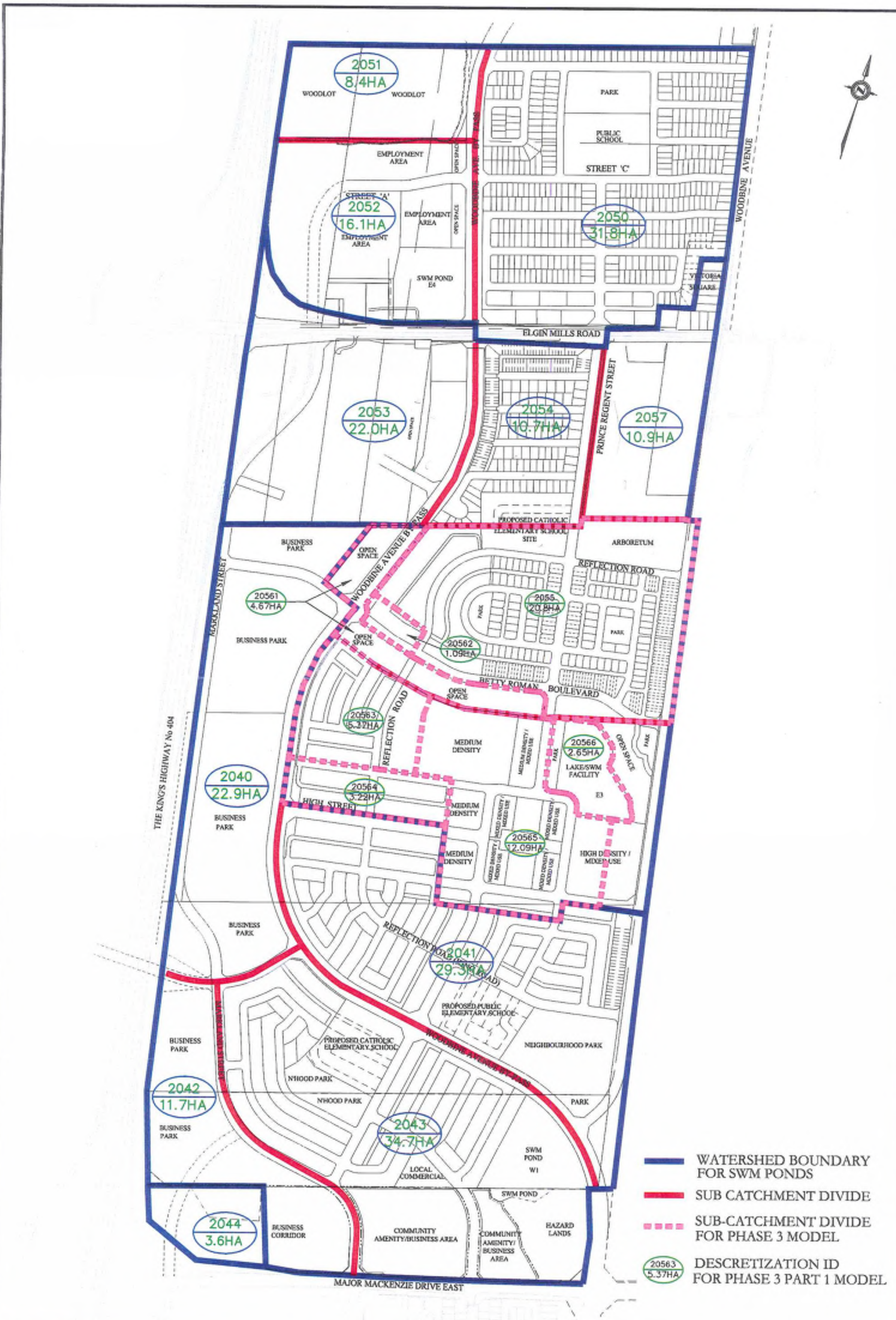
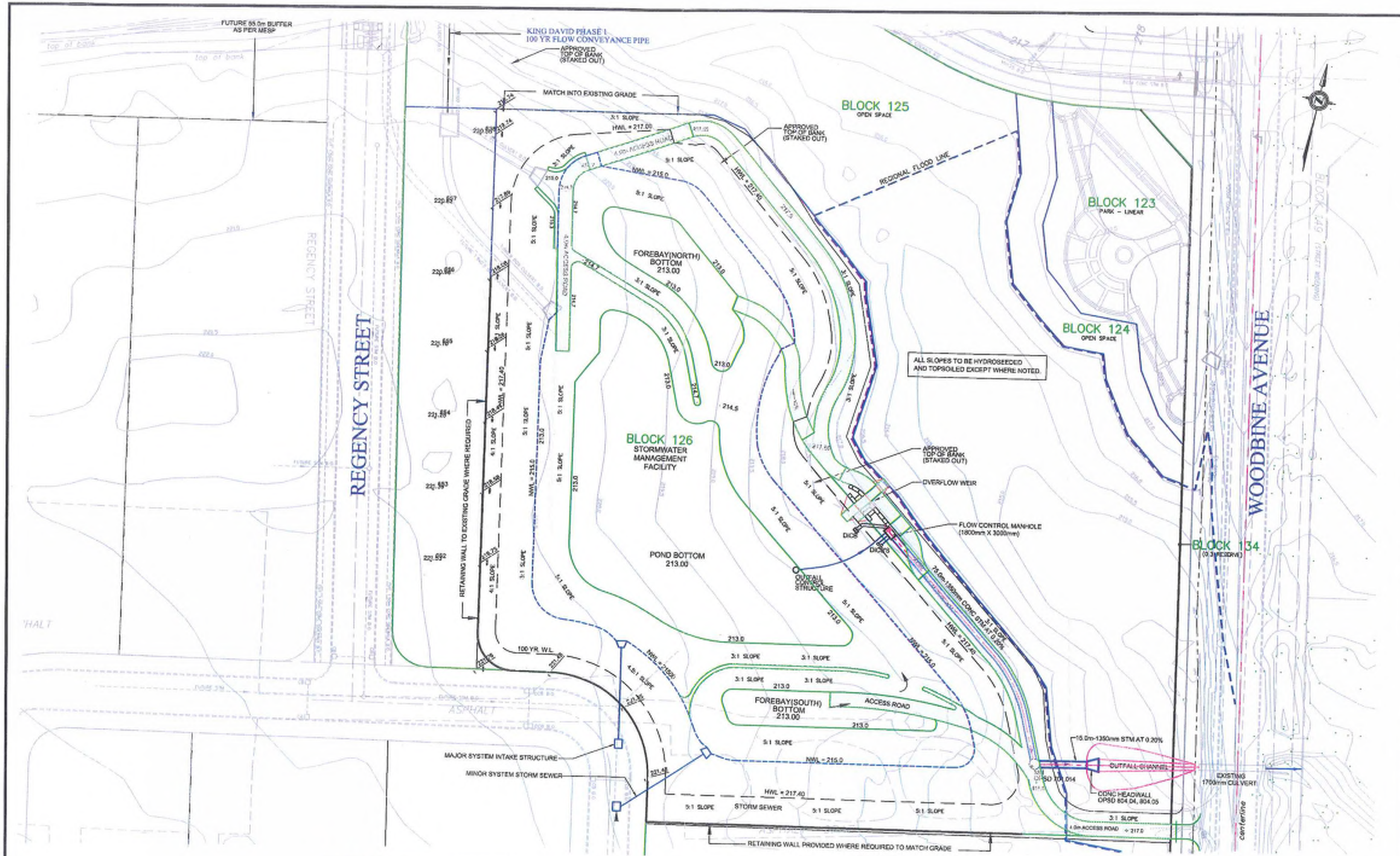


MARKHAM the MARK of EXCELLENCE | URS

PROJECT	TITLE	DATE / SCALE / PROJECT No.	FIGURE No.
ENVIRONMENTAL AND STORMWATER MANAGEMENT PLAN WEST CATHEDRAL COMMUNITY	POST DEVELOPMENT CONDITION WATERSHED DISCRETIZATION	MAR. 2006 / 1:7500 / 02080	FIGURE 3



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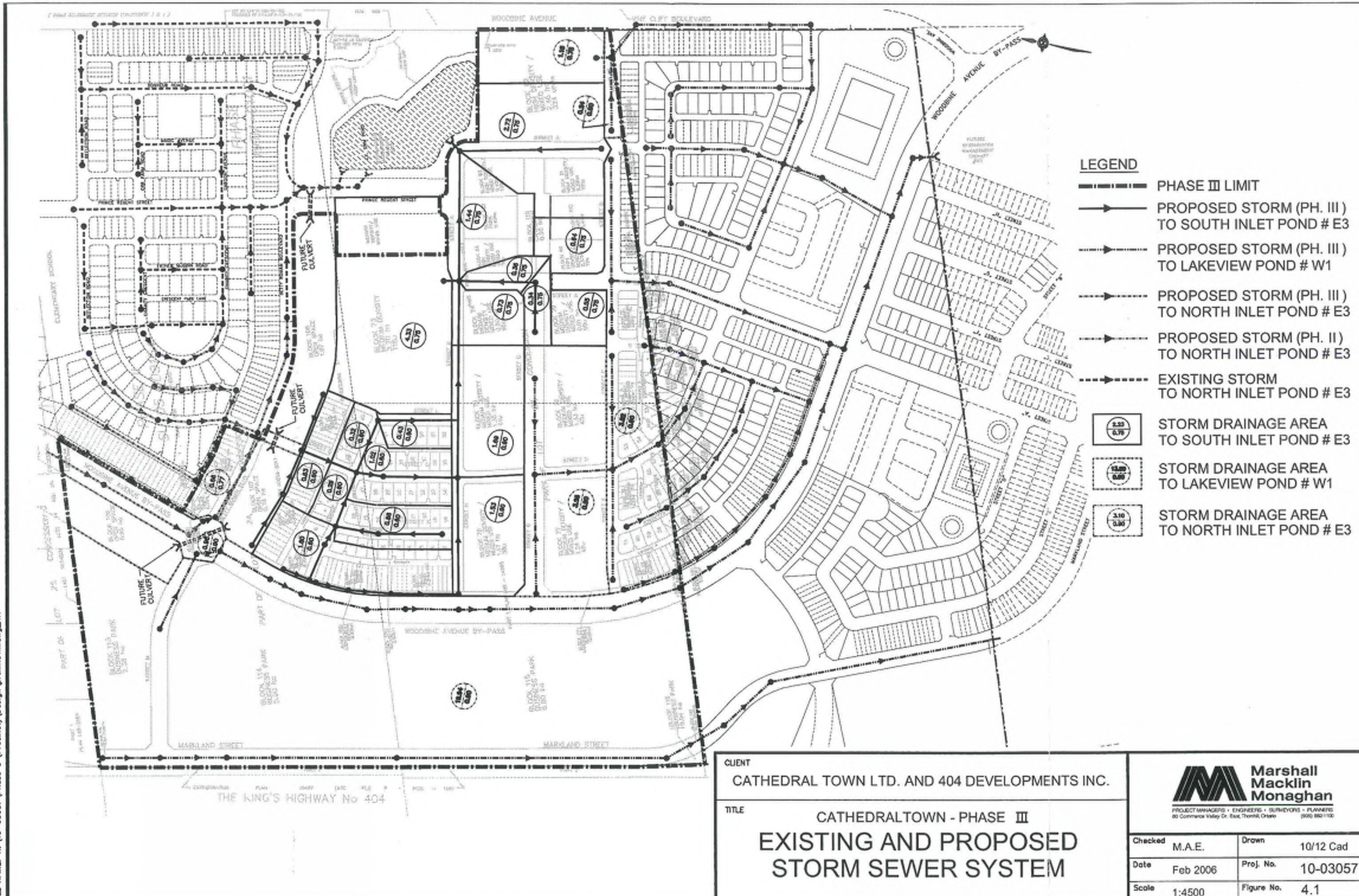
PROJECT	TITLE	DATE	FIGURE No.
ENVIRONMENTAL AND STORMWATER MANAGEMENT PLAN WEST CATHEDRAL COMMUNITY	STORMWATER MANAGEMENT POND E3	NOV. 2004	FIGURE 4
		SCALE 1:1000	
		PROJECT No. 02080	



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DETAILED DESIGN REPORT
STORMWATER MANAGEMENT POND W1

West Cathedral Community
Town of Markham

September 2006

Revised July 2007

Prepared for:
Majorwood Developments Inc.

Prepared by:
Stantec Consulting Ltd.
7270 Woodbine Avenue
Markham ON L3R 4B9

File No. 160621169

4.0 Proposed Hydrology

The proposed drainage area to the W1 SWM Pond includes residential and commercial lands located south of Elgin Mills, west of Woodbine, east of Highway 404 and north of Major Mackenzie (see **Figure 3.1**). The total drainage area to the W1 SWM pond is approximately 100.1 ha in size (including the 2.7 ha pond block area). The proposed grading of the drainage area has resulted in the following drainage patterns. Refer to **Figure 4.0** for the overall proposed storm drainage plan to Pond W1 and drawings 200, 201 and 202 for Storm Drainage Plans for the Majorwood subdivision. Minor system flows from approximately 100.1 ha of the proposed drainage area (Catchments 2040, 2041, 2042, and 2043) will drain to the proposed SWM facility W1.

Major system flows from approximately 28.9 ha (Catchment 2040, 2042) will be controlled on site through a combination of rooftop storage, underground storage and parking lot storage to maintain a discharge rate of 174 l/s/ha (Catchment 2040) and 168 l/s/ha (Catchment 2042), in accordance with the 2006 ESMP.

Major system flows from approximately 71.2 ha (Catchment 2041, and 2043) will be conveyed overland via the road network and ultimately into the downstream stormwater management facility (Pond W1) located within the Majorwood Development lands. However, no overland flow is permitted along the Woodbine Avenue Bypass. Therefore, major system flows from the Woodbine Bypass and areas north of the Bypass will be piped to Pond W1.

The QHRouge hydrology model completed as part of the ESMP has been updated to reflect the design of the catchment areas tributary to SWM Pond W1. The revised ESMP model and the detailed design model are both provided in **Appendix A** along with a summary of catchment input parameters for the entire model and output flows for the areas tributary to pond W1. **Table 6.6** (in Section 6.6) summarizes the inflow to and outflow from SWM Pond W1 from the detailed design QHRouge model, as

Stantec

well as the subsequent storage and water level for storm events ranging from an extended detention (30mm) event to the 100-year storm.

6.0 Pond Operating Characteristics

6.1 PROPOSED STORMWATER MANAGEMENT SCHEME

Pond W1 is a quantity/quality pond with a minor system drainage area of approximately 100.1 ha (including the 2.7 ha pond block), a major system drainage area of approximately 71.2 ha (including the pond block) and a normal water level of 205.2m. The pond will attenuate post development flows to predevelopment flows for all storms up to and including the 100-year storm event.

The outflow from the pond will be conveyed to the Carleton Creek via a storm sewer outfall pipe and overflow spillway. **Drawings 901**, **902**, and **903** illustrate the details for the proposed Pond W1 and the outfall.

The pond is located within the Majorwood Phase 1 Lands. Phase boundaries are on **Drawings 101 and 102** of the subdivision drawing set.

6.2 TARGET RELEASE RATES AND REQUIRED VOLUMES

The QHROUGE model was obtained from URS and has been updated to reflect the minor changes in drainage areas and imperviousness for each of the four sub catchments. In addition, the model was revised to account for the rooftop and parking lot storage provided within the industrial sub catchments. Finally the pond volumes were confirmed utilizing the pre development peak flow rates provided in the ESMP and **Section 3.0** of this report. **Table 6.2** provides a summary of the required design volumes and the pre development peak flow rates (target outflow) for the pond.

TABLE 6.2 –Target Outflow & Required Volumes for W1 (Node 1121)

Return Event	Target Peak Outflow (m^3/s)	Required Volume (m^3)
Permanent Pool	0	19,097
Baseflow Emulation	0	6,536*
30mm	0.10	22,000
2-Year	1.36	23,150
5-Year	2.07	25,200
10-Year	2.72	27,400
20-Year	3.70	30,200
100-Year	5.49	36,200

Note: * Baseflow emulation volume is the upper 0.5m of the permanent pool volume.

6.3 INLET CONFIGURATION

Based on the QHROUGE modeling, the peak 30 mm storm flow to the pond is 6.55 m^3/s (see **Appendix A**). The required volume for the 4 hour 25 mm rainfall event is 22,679m^3, which occurs at a maximum depth of 1.32 m above the normal water level at an elevation of 206.52 m (see **Appendix C**).

The 5 year peak flow (minor system) from the 100.1 ha drainage area will be directed towards the pond via a 2100 mm diameter storm sewer pipe and a 1350 mm storm sewer pipe both from the north and an 1800 mm storm sewer pipe from the south. Based on the storm sewer design sheets and using the Modified Rational Method, the 1350mm diameter and the 1800 mm diameter inlet pipes will convey 5-year peak flows of 2.12m^3/s and 5.64m^3/s respectively, while the 2100 mm diameter inlet pipe will convey 100-year peak flow of 11.28m^3/s into the pond.

Flows in excess of the 5 year peak flows (major system) will be conveyed along the streets and outlet into the pond overland with the following exceptions:

- Major system flows from sub catchments 2040 and 2042 will be attenuated on site through roof top controls and parking lot storage.
- No overland flows are allowed to enter the Woodbine By-Pass; therefore, major system flows for drainage areas north of the Woodbine Bypass will be captured and piped to SWM Pond W1.

6.4 OUTLET CONFIGURATION

As shown on **Drawing 901**, **902** and **903**, the quantity control outlet for Pond W1 consists of:

i) a submerged 300 mm diameter perforated horizontal pipe;

ii) a 300 mm diameter concrete pipe to the 1200mm diameter Control MH1 (invert 205.2);

iii) a 205 mm diameter orifice plate attached to the 300 mm diameter pipe entering the 1200mm diameter Control MH1 at an elevation of 205.2 m;

iv) a 2.4 m wide x 0.65 m high weir cut into the face of the 3000mm x 1650mm Control MH2 at 206.55 m in elevation, including a 900 mm diameter outlet pipe and headwall;

v) a 2.85 m long trapezoidal weir with an invert elevation of 206.95 m; and

vi) a 70 m long trapezoidal weir with an invert elevation of 207.40 m designed as an emergency spillway with the capacity to convey the 100 year un-attenuated flow. Flow capacity calculations are provided in **Appendix C**.

In addition, the water balance outlet for Pond W1 consists of:

i) a 100 mm diameter perforated subdrain wrapped in filter cloth connected to a 200 mm diameter pipe;

ii) a removable 60 mm diameter orifice plate attached to the 100 mm diameter entering the 1500mm diameter Control MH3 at an elevation of 204.7 m;

iii) 2 – 200 mm diameter perforated incoming pipes located below 2 – 200 mm diameter perforated outgoing pipes all situated within an 11 m long x 2.0 m wide x 1.5 m high exfiltration trench filled with pea gravel and wrapped in filter cloth;

iv) 2 – 200 mm diameter gate valves and valve boxes located on 2 – 200mm pvc stm bypass pipes.

v) a 200 mm diameter pipe entering a 600 mm x 600 mm ditch inlet at an elevation of 203.2 m, with the ditch inlet grate elevation at 204.7 m.

There are two outlet pipes exiting from the 1200mm diameter Control MH3, each one connected to a separate exfiltration trench and overflow outlet. Each outlet and trench has been sized to meet and double the sizing requirements specified within the *Water Balance Report*. In addition, a second outlet and trench are in reserve, should the other outlet be blocked or require maintenance, thereby providing a factor of safety 4 times the design specified within the *Water Balance Report*. To prevent clogging of the exfiltration trench during construction, the trenches will not be activated until 3 years following base asphalt placement. To ensure that this occurs, a solid aluminum plate (with no opening) will be installed during construction, and will be replaced with the 60mmdiameter orifice plate within Control MH3 at the appropriate time. Notes specifying timing and plate installation are included on the design drawings.

Details are provided on **Drawings 901, 902**, and **903**. The detailed pond storage volume and release rate calculations for the water balance outlet are provided in **Appendix B.** Also included in this Appendix are the allowable release rate and minimum trench sizing calculations from the *Water Balance Report.*

6.5 THERMAL IMPACTS

Thermal impacts have been addressed in three ways. A bottom draw outlet configuration, which draws water from the cooler areas, located at the bottom of the pond, is utilized. The normal water level will be shaded by the proposed landscaping to further minimize the thermal impacts associated with pond water ultimately entering the Carleton Creek. Also, the release of the upper 0.5m of the permanent pool through the exfiltration system will further enhance thermal impacts to the Creek.

6.6 EXTENDED DETENTION STORAGE

The initial 1.32 m of extended detention fluctuation within the pond will outlet through the submerged 300 mm diameter perforated pipe. The 2.0 m perforated section of pipe will have approximately 78 - 75 mm diameter openings with a combined opening area of 0.345 m^2 (see **Appendix F**). The perforated pipe outlet will be controlled by a 205 mm diameter orifice plate located in the Control Structure at an invert elevation of 205.2 m (see details on **Drawings 901, 902** and **903**). The 205 mm diameter orifice is sized to discharge the 30 mm runoff volume of 22,679 m^3 at an elevation of 206.52 m over a 140 hour period with a peak flow of 0.10 m^3/s (see calculations in **Appendix C**).

Table 6.6 summarizes the stage/storage/inflow/outflow characteristics of Pond W1.

Table 6.6: Stage/Storage/inflow/Outflow for Pond W1

Return Period (year)	Target Peak Outflow (m^3/s)	Required Storage (m^3)	Total Peak Flow into W1* (m^3/s)	Design Peak Outflow (m^3/s)	Provided Storage (m^3)	Elevation (m) (NWL = 205.2 m)
30 mm	0.10	22,000	6.55	0.10	22,679	206.52
2	1.36	23,150	8.7	0.63	27,683	206.78
5	2.07	25,200	10.97	1.27	30,868	206.94
10	2.72	27,400	12.82	1.92	32,971	207.05
25	3.70	30,200	15.45	3.06	35,504	207.18
100	5.49	36,200	19.96	5.48	39,789	207.38

* Peak flows taken from QHRouge Modeling

6.7 POND GRADING

To provide a natural appearance, the proposed grading within the pond block will utilize slopes varying from 3:1 to 6:1 (horizontal to vertical). For public safety, an area with a gentler slope of 6:1 having a width of 3 m immediately above the normal water level of 205.20 m has been provided. Immediately above the 6:1 sloped area to an elevation of 207.75 m, slopes of 4:1 are provided and 3:1 slopes are provided from an elevation of 207.75 m to an elevation of 211.0 m. The slopes in the permanent pool will be graded with 5:1 slopes. The proposed pond grading is shown on **Drawing 901**.

6.8 PERMANENT POOL AND SEDIMENT FOREBAYS

The sizing of the permanent pool is based on Level 1 (Enhanced) protection with a total weighted site imperviousness of approximately 75% for areas draining to Pond W1. The total required permanent pool volume is 18,872 m^3. The total permanent pool provided is 22,183 m^3. Refer to **Appendix C** for pond stage-storage calculations.

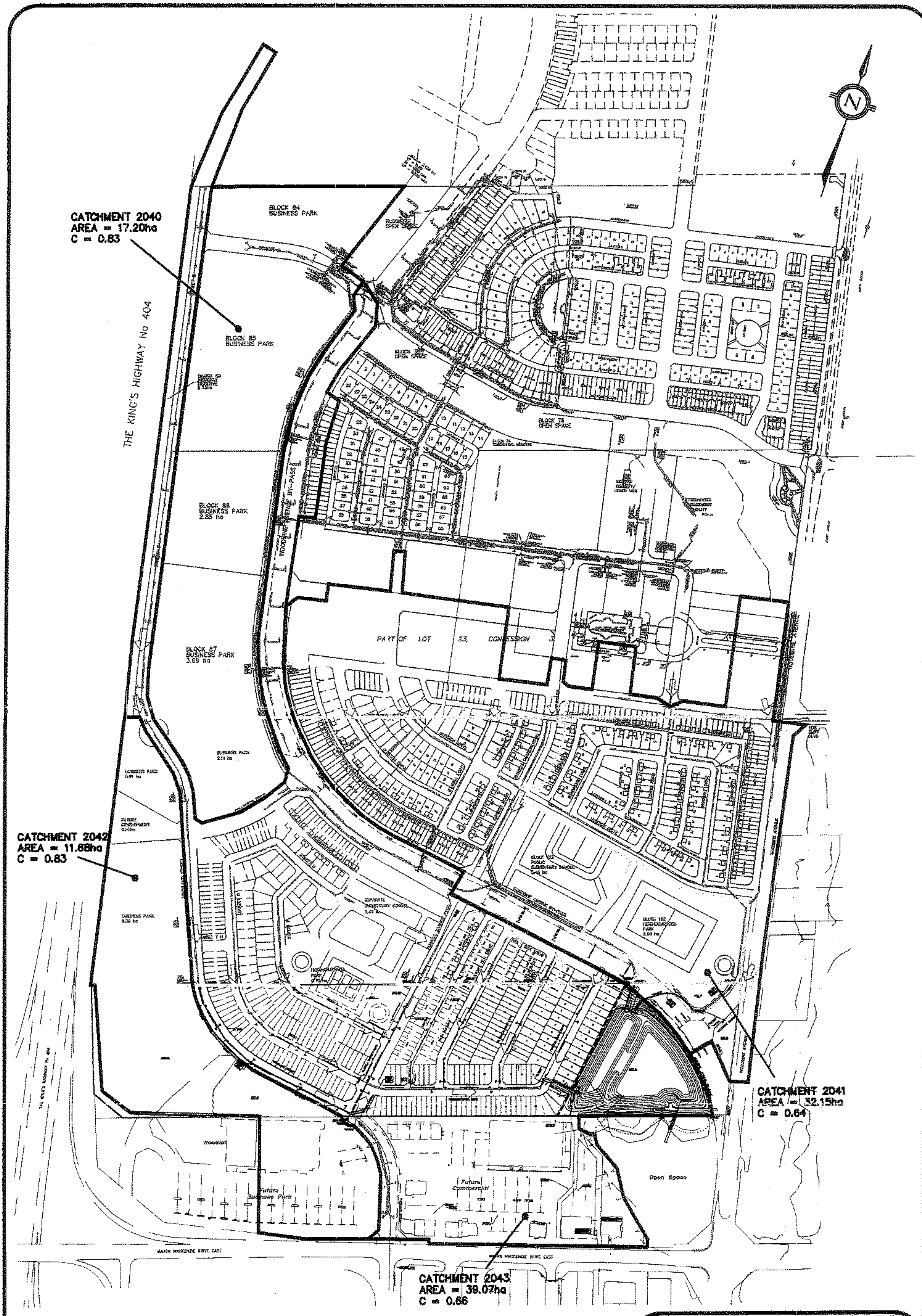
A sediment forebay has been provided in the permanent pool with all the three headwalls connected (refer to **Appendix D** for sizing calculations). The forebay was graded 2.0 m deep. The sediment forebay has also been designed with a length to width ratio of at least 2:1.

6.9 MAINTENANCE ACCESS

Maintenance access has been provided to the sediment forebay, pond inlets and pond outlets via 4.0 m wide access roads at maximum grades of 8:1. The access roads will be constructed as per the details provided on **Drawings 903**.

6.10 SUBDRAIN CALCULATIONS

An infiltration trench has been incorporated into the design of the pond to allow additional base flow to the Carleton Creek. A 200 mm diameter subdrain is proposed at an elevation of 204.8 m (0.4 m below the normal water level) and is connected to the 1500mm Control MH3. Mounted to the inside of the control manhole is a 60 mm diameter orifice plate set at an invert elevation of 204.7 m. This orifice plate has been sized to release the top 0.5 m of water below the normal water level. The required baseflow emulation volume is 6,536 m^3 and the available volume is 6,970



OVERLAND FLOW ROUTE

MINOR SYSTEM DRAINAGE AREA

CATCHMENT BOUNDARIES

Stantec Consulting Ltd.
7270 WOODBINE AVE., SUITE 300
MARKHAM, ONTARIO L3R 4B9
TELEPHONE: (905) 474-0455
FAX: (905) 474-9889

MAJORWOOD
(FORMERLY BARROWCREST)

EXTERNAL STORM DRAINAGE PLAN

DATE:	JULY 2007	DRAWING No.
PROJECT:	160621169	4
SCALE:	N.T.S.	

STORMWATER MANAGEMENT AND POND DESIGN REPORT

Town of Markham

MONARCH CORPORATION

TRADITIONS AT VICTORIA SQUARE

Monarch
by Taylor Woodrow

July 2007

Cole Engineering

100 Renfrew Drive, Suite 100, Markham, Ontario L3R 9R6 Tel: 416.987.6161 Fax: 905.940.2064

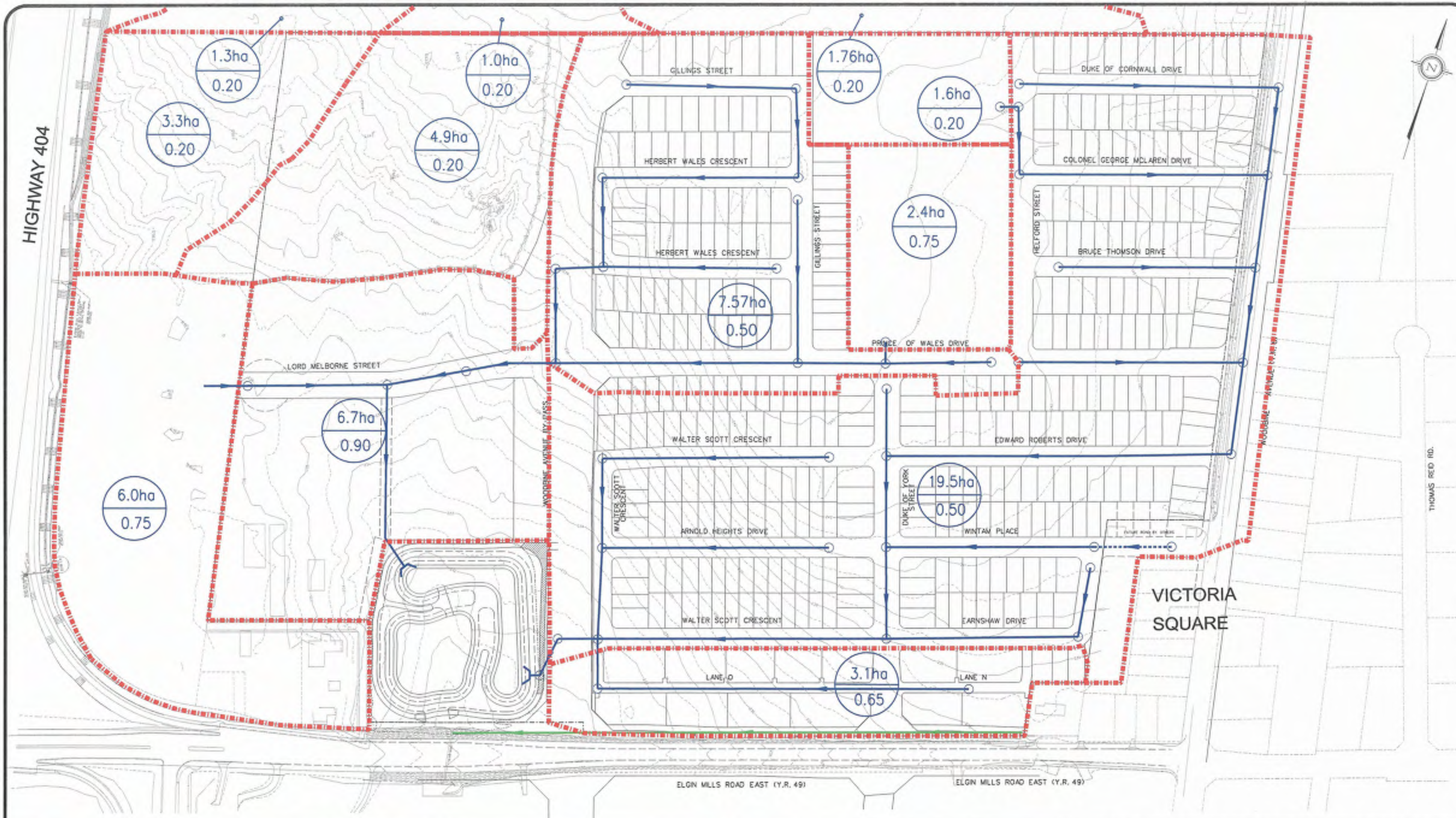
2.0 PROPOSED CONDITIONS

The proposed land use pattern, road and lot layouts are based on information provided by Bousfields that incorporates the design team and client inputs. Three employment area blocks, totalling about 5.7 hectares, are located to the west side of the plan, west of the proposed Woodbine Avenue By-Pass. The residential component is located to the east side of the Woodbine Avenue By-Pass, and consists of single and townhouse lots sited along road right of ways of various widths. Provision for parkland, a school block, open space, and a stormwater management pond block have also been accommodated on the plan. Refer to Figure 2 for the Draft Plan.

Street B will be the main east-west midblock collector between Woodbine Avenue and the Woodbine Avenue By-Pass. Streets I, J, K, B and L will connect to Woodbine Avenue directly, which will be downgraded from a Regional Road to a minor local collector road after the Woodbine Avenue By-Pass is opened to traffic. Access to the site from Elgin Mills Road East will be provided at Street C and the Woodbine Avenue By-Pass.

The storm water management facility will be located at the south west edge of the property, at the natural lowest part of this site.

The grading and storm sewers were designed to support the layout and location of the storm water management pond. The major and minor drainage systems were planned around the existing topographical layout of the site to attain a self contained system, adhering to the principle that drainage areas contained in natural watersheds are functionally not to affect any external lands.



Cole Engineering Group Ltd.
100 Renfrew Drive - Suite 100 - Markham - Ontario - L3R 9R6
Phone: (416) 987-6161, (905) 940-6161 Fax: (905) 940-2064

LEGEND:

- 1.0ha / 0.20 — DRAINAGE AREA / COMPOSITE RUNOFF COEFFICIENT
- STORM DRAINAGE AREA
- FDC SEWERS
- MINOR STORM SEWER SYSTEM
- MINOR STORM SEWER DIRECTION
- POTENTIAL FDC/SUMP PUMP AREA

POST-DEVELOPMENT MINOR STORM DRAINAGE PLAN
TRADITIONS AT VICTORIA SQUARE
MARKHAM, ONTARIO
MONARCH CORPORATION
STORMWATER MANAGEMENT REPORT

DATE:	JULY 2007	PROJECT No.:	M03-115
SCALE:	1:3000	FIGURE No.:	STM-2

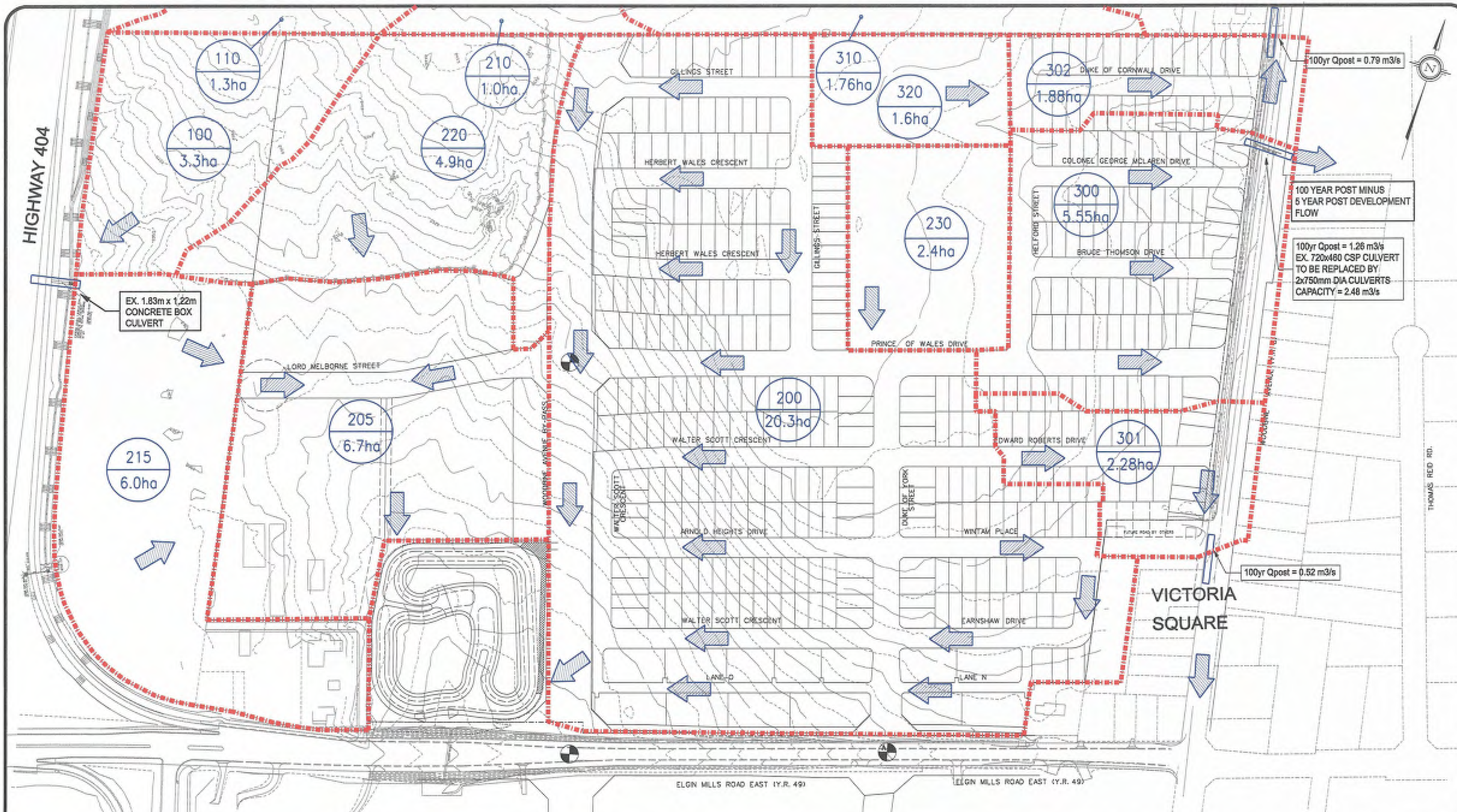
3.0 HYDROLOGIC MODEL

To determine the SWM control targets and to assess the success of potential SWM measures, hydrologic models of the subject lands in the pre-development and post-development conditions were prepared. The hydrologic models were prepared in Visual OTTHYMO version 2, which is a commonly used and accepted model within Ontario. Pre- and post-development drainage areas are shown on Figures STM-1, STM-2 and STM-3 respectively. SCS Curve Numbers for calculating pervious area losses were derived from the TRCA Rouge River Model, while other input parameters were calculated based on the site characteristics in the pre-development and post-development conditions. Input parameters are provided in Appendix A, as well as hydrologic model input and output data. Table 3.1 below shows the pre-development flows to the east (Woodbine Avenue), west (Highway 404 ditch) and to the south (Elgin Mills culvert).

Table 3.1

Pre-development Peak Flow Summary

Return Period Storm	Woodbine North Outlet	Woodbine East Outlet	Woodbine South Outlet	West Outlet	South Outlet
(yr)	m^3/s	M^3/s	m^3/s	m^3/s	m^3/s
2	0.109	0.349	0.077	0.136	0.665
5	0.214	0.692	0.154	0.278	1.344
10	0.284	0.925	0.207	0.378	1.816
25	0.403	1.320	0.297	0.551	2.626
100	0.619	2.044	0.464	0.877	4.148
Total Drainage Area to Outlet (ha)	2.97	10.98	2.65	4.60	39.2



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LEGEND:

- MAJOR OVERLAND FLOW ROUTE
- STORM DRAINAGE BOUNDARY
- 100YR STORM SYSTEM
- WATERSHED ID (100)
- DRAINAGE AREA (11.5ha)

POST-DEVELOPMENT MAJOR STORM DRAINAGE PLAN

TRADITIONS AT VICTORIA SQUARE
MARKHAM, ONTARIO
MONARCH CORPORATION
STORMWATER MANAGEMENT REPORT

DATE:	JULY 2007	PROJECT No.:	M03-115
SCALE:	1:3000	FIGURE No.:	STM-3

4.0 STORMWATER MANAGEMENT PLAN

The proposed works will result in a change in land use within the subject site, with an increase in the amount and extent of impervious surfaces due to conversion of agricultural fields into roads, sidewalks, lot paving and other similar types associated with residential development.

The urbanization of the subject lands will affect the local hydrologic cycle, with an increase in the rate and volume of surface runoff following a storm event. Without appropriate mitigation, these increases would adversely affect the local drainage regime by increasing frequency and duration of flows that may cause flooding, by increasing erosion rates in watercourses, and by detrimentally impacting water quality and habitat in the aquatic ecosystems.

To minimize the impact to the local drainage regime and aquatic ecosystems, it is proposed to mitigate the impact of development by implementation of appropriate stormwater quantity and quality controls, as described in the following sections.

A dual drainage approach to managing surface runoff from the subject lands is proposed, utilizing minor and major flow systems in conjunction with stormwater detention and controlled release. The onsite storm sewer system is designed to accept and convey runoff from the more frequent storm events (5-year or 10-year storm event). Flows exceeding the capacity of the storm sewer system will flow along the internal road network and collect at the lower part of the site where it will enter the stormwater management pond. The SWM pond is designed to provide extended detention and controlled release of stormwater runoff to mitigate and minimize impacts to the local drainage regime and aquatic ecosystems. In addition, lot level controls such as roof leaders directed to pervious areas will be utilized on all lots.

Best management practices, to encourage the reduction of sediments and their related pollutants from the stormwater before it enters the sewer system, are implemented. This includes the disconnection of roof water leaders and the use of overland flow over grassed yards and swales. All street catchbasins have sumps to promote settling of suspended solids in stormwater runoff. Good "house keeping" techniques are to be implemented throughout the construction process.

The capacity of the existing culverts under Elgin Mills Road and Woodbine Avenue has been assessed, and the results are presented below. Calculations are provided in Appendix D.

The Elgin Mills culvert capacity is equal to 4.66 m3/s (Regional storm release rate) when the water level north of Elgin Mills builds up to 232.29, and 2.92 m3/s (100 year storm release rate) when the water level north of Elgin Mills builds up to 231.69.

The existing 720 x 460 CSP culvert that crosses Woodbine Avenue has a capacity equal to 0.65 m3/s, which is insufficient to convey the predevelopment peak flows towards the east outlet and it suspected that this culvert overtops Woodbine Avenue during the 100 year storm. As such during Phase 2 of the development, this culvert is to be replaced with twin 750mm diameter concrete culverts to provide conveyance for the post development peak flow which are set at 100 year flow minus the 10 year flow resulting in a total major flow to Woodbine Avenue of 2.38

m^3/s. This proposed culvert configuration will be detailed during the Phase 2 engineering submission and have a capacity of 2.48 m^3/s.

The backwater discharge characteristics for the culvert under Elgin Mills, was entered into the pond outlet stage-storage calculations so that an accurate stage discharge curve could be entered into the post development model. The pond design calculations are provided in Appendix B.

Stormwater Drainage Design Criteria

Stormwater quantity and quality control measures are designed in accordance with the following criteria as defined by municipal standards and the various regulatory agencies.

Quantity Control Design Criteria

Based on consultation with the Town of Markham and the Toronto and Region Conservation Authority, the quantity control criteria for the development is as follows:

- The post-development peak flow rates, for the 2 year to the 100 year storm events shall be controlled to a rate less than or equal to the pre-development rate.

Water Quality Criteria

- All new development shall meet Enhanced (Level 1) MOEE 2003, quality control criteria.
- All new developments are to provide 48 hour detention for runoff from a 25mm storm event.

4.1. Storm Sewers

Storm sewers are designed to convey up to the 5 year or 10 year Town of Markham design storm, with adequate size and depth in accordance with the most recent Town of Markham municipal design standards and specifications. The storm sewer network will discharge into the proposed SWM pond.

As identified in the Environmental and Stormwater Management Plan – West Cathedral Community, dated March 2005, prepared by URS, an existing area of 13.5ha in the north east corner of Traditions at Victoria Square drains easterly towards Berczy Creek. As part of the URS Plan, under post-development conditions, the area draining to Berczy Creek was modified to 10ha. In order to maintain post-development flows to pre-development levels for this area, the flows greater than the 5 year design storm up to the 100 year design storm were proposed to continue to flow towards Berczy Creek. The flows up to the 5 year design storm were proposed to be conveyed into the Traditions at Victoria Square stormwater management pond.

As part of the detailed design which we did for Traditions at Victoria Square, the proposed grading required an increase to the area which will continue to flow towards Berczy Creek. To compensate for the increase in drainage area, and to maintain pre-development flows into Berczy Creek, the storm sewers serving areas 300 and 320 as identified on Figure STM-3 are designed to convey flows up to the 10 year design storm, into the stormwater management pond. Post-development flows greater than the 10 year storm and up to the 100 year storm will spill to Berczy Creek along the existing overland flow route.

The storm sewers for the remainder of the site are designed to carry the 5 year storm flows. The storm sewer system is designed to minimize instances of surcharging within the sewer pipes, and to maintain a separation of at least 0.5m between finished basement elevations and the 100-year hydraulic grade line within the pipes to reduce the risk of basement flooding. In some cases, where road and intersection designs warrant, inlet restriction devices are to be installed. Refer to Figure STM-2 for layout information.

In order to convey building foundation weeper drainage, a sewer connection from the sewer main to the edge of the road allowance will be constructed for each lot or building block in the plan of subdivision.

4.2. Overland flow

The 100 year design storm, less the 5 year or 10 year flows will flow along the roadways as overland flow to the SWM pond at the south edge of the site, or to the east of the central ridge, (through a proposed twin 750 mm culvert across Woodbine Avenue which will replace the existing 720 x 460 CSP culvert and which eventually drain into the Berczy Creek) respectively. Overland flow from the existing woodlot and external drainage area which currently drains to the west will continue to discharge to the existing concrete box culvert crossing Highway 404. The road network is designed to safely convey the 100 year minus 10 year flows for the land draining to the east and 100 year minus 5 year flows for the remainder of the site.

Roadway conveyance calculations were done to determine the depth of flooding on the roadways and boulevards during the 100 year storm event. During the 100 year storm event, flow depths on the roadways do not exceed 0.25 m, which satisfies the municipal and provincial criteria for roadway flooding.

To ensure the hydraulic grade in the storm sewer system is maintained below or at the obvert of the pipes where required, inlet control devices (ICDs) are to be installed at the storm sewer system inlets. Inlet control devices are selected to ensure the minor system capacity and capture rates are not exceeded, under expected depths of ponding in sag areas or expected depth of flooding on the roadway. Use of ICDs would cause temporary ponding and detention of stormwater runoff, and controlled release into the storm sewer system.

4.3. Foundation Drainage System

Lots fronting on Elgin Mills Road will have a separate foundation drain collector (FDC) sewer which will be constructed in order to avoid potential surcharge of the footing weeper. The FDC sewer will discharge directly to the Elgin Mills Road culvert.

4.4. Stormwater Management Pond

A SWM wet-pond facility is proposed to be located along the south edge of the site on a 2.29 hectare block. The SWM pond outlet will discharge to the existing culvert beneath Elgin Mills Road. The proposed SWM pond is intended to provide water quality enhancement of stormwater runoff, as well as provide extended detention and controlled release of stormwater to maintain the local drainage regime and mitigate impacts on the environment. The pond is designed as a conventional wet pond incorporating a permanent pool, sediment forebay, and an extended detention and flood control component, with the layout of the pond satisfying the MOE geometric guidelines for wet ponds.

The SWM pond is designed according to the following criteria:

- The permanent pool is sized to provide Enhanced (Level 1) water quality treatment.
- Suggested bank slope, 7:1 slope from 0.5m above the normal water level to 0.5m below and 4:1 slope below 0.5m of the normal water level.
- Sediment Forebays are designed to MOE 2003 SWMP Manual standards.
- Extended detention of the 25mm storm for a minimum of 48 hours is to be provided.
- The maximum increase in water level, for the 100 year storm over the permanent water level, should be no more than 3.0m.
- Suggested bank slope, above the permanent water level, is 5:1
- A minimum of 0.3 m of freeboard will exist between the Regional Storm water level and the top of the pond.

4.4.1. Stormwater Management Pond Volumes

Based on the proposed land use in the post-development condition, the lumped imperviousness of the site is 55.2%. To achieve Enhanced Level water quality treatment (80 % suspended solids removal), 151 m^3/ha is required which results in a permanent pool requirement of 8,528 m^3. A permanent pool volume of 9,686 m^3 has been provided at a pond water surface elevation of 230.0 m. Calculations for permanent pool volumetric requirements are provided in Appendix C.

Further water quality enhancements will be realized by extended detention of the runoff from a 25mm storm. Based on the post-development hydrologic model, the volumetric runoff coefficient for the 54.34 ha area tributary to the pond is 15.40 mm, which would require an extended detention volume of 8,368 m^3 at elevation 230.66. The proposed outlet structure configuration and discharge characteristics, controls the 2 to 100 year storm to predevelopment levels, requiring 24,385 m^3 of active storage at elevation 231.69. The access road is set at an elevation of 232.00, which provides 0.32m of freeboard between the top of pond and the 100 year pond elevation. Hydrologic model results are provided in Appendix C.

Above the 100 year storm event, the facility will provide for safe conveyance of the regional flood, at elevation 232.29. Although the access road is submerged in some locations during the regional flood, the flood is contained within the pond block without adversely affecting adjacent properties or roads.

The adequacy of the proposed facility has been verified using the detailed design grading information. Model results are summarized in Table 4.5.1.1 and 4.5.1.2, and confirm that post-development flow rates will not exceed the pre-development rates at each site outlet.

Table 4.5.1.1

Post-development Peak Flow Summary

Return Period Storm	Woodbine North Outlet (m^3/s)	Woodbine East Outlet (m^3/s)	Woodbine South Outlet (m^3/s)	West Outlet (m^3/s)	South Outlet (SWM Pond) (m^3/s)
2 year	0	0	0	0.136	0.21
5 year	0	0	0	0.278	0.43
10 year	0.09	0.19	0.06	0.378	0.67
25 year	0.34	0.53	0.23	0.551	1.06
100 year	0.79	1.26	0.52	0.877	2.92

Table 4.5.1.2

SWM Pond Performance Summary

Return Period Storm	Inflow (m^3/s)	Outflow (m^3/s)	Storage Used (m^3)
25 mm	4.93	0.08	7,927
2 year	7.54	0.21	10,534
5 year	10.94	0.43	13,322
10 year	13.20	0.67	15,825
25 year	16.60	1.06	19,479
100 year	22.08	2.92	24,385
Total Storage Available = 39,123 m^3 at 232.50			

The SWM facility volumes have also been confirmed using the TRCA AES 12-hour watershed storms, and the provided volumes are adequate for control of the 100 year storm runoff to maintain peak discharge rates at the south outlet (and by aggregation from the site as a whole) to less than the pre-development level. Model results are provided in Appendix A, and summarized below in Table 4.5.1.3.

Table 4.5.1.3

SWM Pond Performance Summary (AES 12-Hour Storms)

Return Period Storm (yr)	Peak Flows at South Outlet (SWM Pond)		Storage Used (m^3)
	Pre-Development (m^3/s)	Post-Development (m^3/s)	
2	0.606	0.39	12,814
5	0.964	0.55	14,579
10	1.232	0.76	16,681
25	1.594	1.02	19,167
50	1.878	1.25	21,066
100	2.172	1.56	22,650
Total Storage Provided = 39,123 m^3 at 232.50			

4.4.2. Forebay Design

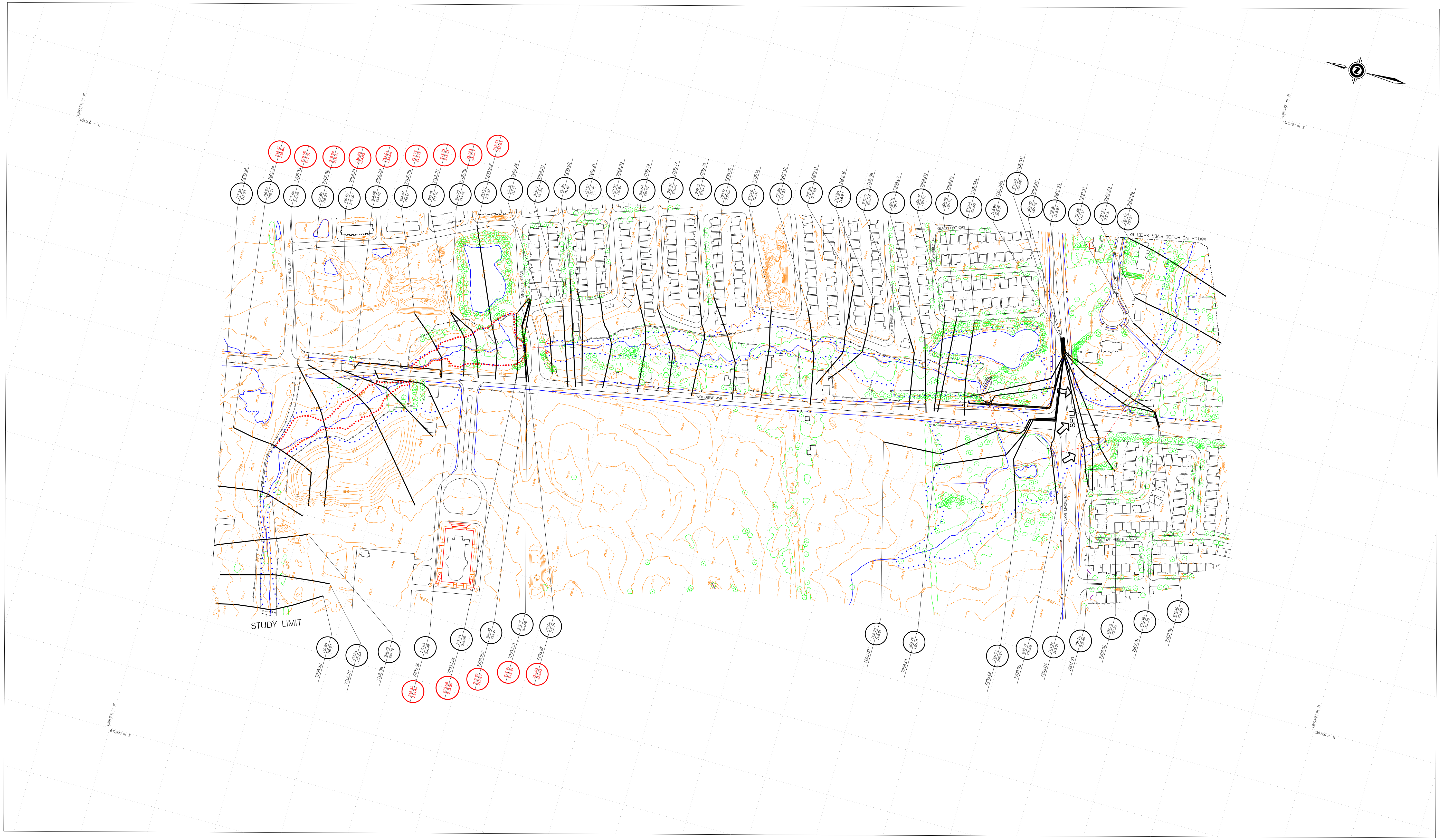
The proposed pond incorporates a 1.2 m deep sediment forebay for pre-treatment of stormwater runoff. Flows will enter the sediment forebay and be allowed to settle within the permanent pool. The sediment forebay is connected to the main wet cell by means of a submerged berm at elevation 229.7 m. The forebay layout results in greater than a 4:1 length to width ratio, which will minimize the potential for short-circuiting of flows. Based on expected sediment loadings, in excess of 10 years capacity will be provided within the lower 0.6 m of the forebay. Calculations are provided in Appendix C.

4.4.3. Outlet Structure Design

The outlet control structure will provide bottom draw extended detention, quantity control, and emergency outlet. The piped SWM pond outlet will discharge to the existing Elgin Mills Road culvert at controlled release rates less than the pre-development rates. Flows up to and including the 25 mm storm flows are retained in the pond and discharged by means of a 225 mm diameter orifice plate installed in a control manhole and set at an elevation equal to the permanent pool. The orifice plate will provide controlled release such that the drawdown time for the 25 mm

F

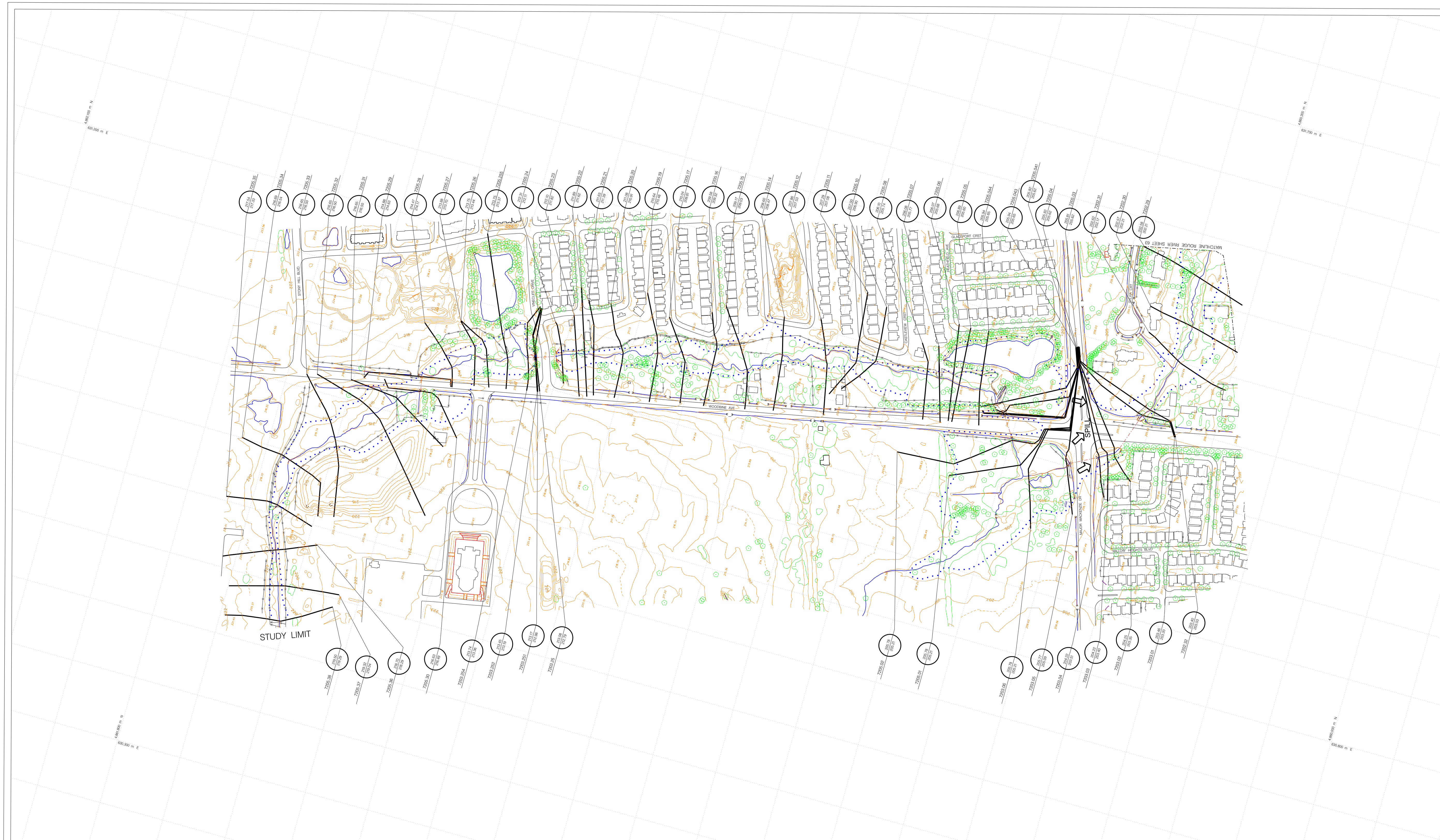
Carlton Creek Crossing Floodplain Mapping



Flood Plan Mapping
Victoria Square Boulevard Class EA
Woodbine Avenue (North Intersection) to
Woodbine Avenue (South Intersection)

Proposed Flood Line
Existing Flood Line

LEGEND
Cross–Section Label
Cross–Section Leader Line
Regional Flood Elevation (m)
Cross–Section Number 14.060 172.00
100 Year Existing Flood Elevation (m) 172.00
Cross–Section Length
REGULATORY FLOOD ELEVATION IS THE HIGHER OF THE TWO ELEVATIONS DISPLAYED
212.87
212.57
Proposed Regional Flood Elevation (m)
Proposed 100 Year Flood Elevation (m)



REVISIONS			
NO.	DESCRIPTION	BY	DATE
1	Regional Storm elevation revised at cross-sections 7202.31 and 7202.30	M.T.	2011-03-24

LEGEND

Cross-Section Label

Cross-Section Leader Line

Regional Flood Elevation (m)

Cross-Section Number 14.060

172.00

172.00

100 Year Existing Flood Elevation (m)

Cross-Section Length

REGULATORY FLOOD ELEVATION IS THE HIGHER OF THE TWO ELEVATIONS DISPLAYED

LEGEND

Contour Index
Contour Intermediate
Contour Auxiliary
Contour Depression
Contour Text 100.20
Spot Height . 100.10
Road
Parking Lot
Race Track
Wall
Retaining Wall
Rail Line
Runway
Silo, Smoke, Tank
Marsh Symbol
Marsh Boundary
Township Fabric
Trail
Bridge
Wooded Area
Tree
Hedge
Fence
Water Feature
Culvert Symbol
Culvert to Scale
Dam
Pool
Building
Pit
Pile
Regulatory Flood Line

This map was compiled photogrammetrically from 1/10 000 aerial photograhy exposed in April 2005.

The vertical datum is mean sea level as established by the Geodetic Survey of Canada.

North American Datum 1983, U.T.M. 6° projection Zone 17, Central Meridian 81° W.

Grid Interval 100 metres.

PLEASE NOTE:
FLOODLINE ELEVATIONS ARE SUBJECT TO CHANGE DUE TO REVISED INFORMATION.

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ASSOCIATION OF ONTARIO LAND SURVEYORS
CR99

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PLEASE NOTE:
THE PROFESSIONAL ENGINEER'S STAMP VERIFIES THE FLOOD LINE AND ASSOCIATED DATA NOT THE MAP DATA UNLESS OTHERWISE NOTED

LICENSED PROFESSIONAL ENGINEER
DANIEL R. MILLER
PROVINCE OF ONTARIO

FLOOD PLAIN MAPPING PROGRAM

FLOODLINE APPROVED DATE: 2006-03-28

TORONTO AND REGION Conservation for The Living City

5 Shoreham Drive Downsview Ontario M3N 1S4 (416) 661-6600

Scale 1:2000

CONTOUR INTERVAL 1.0 METRES:

ROUGE RIVER

SHEET No. 64