Servicing Report

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1. STORMWATER MANAGEMENT

This conceptual stormwater management servicing report has been prepared as technical background for the East Woodstock Secondary Plan and Design Study.

The study area lands are located within the eastern part of the City of Woodstock and are bordered by Blandford Road to the east, Oxford County Road No. 2 to the south, Township Road No. 3 to the north and just west of the future Chandaria Way. Highway 401 transects the eastern portion of the study area on a diagonal running northeast to southwest approximately. The future Toyota Motor Engineering & Manufacturing North America (TEMA) facility is situated in the center of the study area. To the northwest, a portion of lands owned by the Upper Thames River Conservation Authority are also included in the study area and are adjacent to the Pittock Reservoir.

The extent of the study area is shown as Appendix C – Figure 1.

For the purposes of this report the stormwater management for the proposed Toyota site was not addressed specifically and was assumed to be as per the approved SWM report prepared by Giffels Associates Limited on behalf of Toyota Motor Engineering and Manufacturing.

The objective of the stormwater management servicing report is to prepare a conceptual stormwater management plan that services proposed growth in the area while addressing water quantity/quality requirements and erosion control requirements.

1.1. Methodology

1.1.1. Hydrologic Modeling

The hydrologic event simulation model SWMHYMO was used to quantify hydrographs and estimate peak flow and storage requirements for storm runoff from the subject site. SWMHYMO is a hydrologic model used for the simulation and management of stormwater runoff in either small or large rural and urban areas.

Rural areas were modeled using the NASHYD procedure of SWMHYMO, which uses the SCS unit hydrograph approach. The model employs estimates of SCS CN values (based on soil types and land use), along with estimates of the drainage time of concentration to simulate runoff from rainfall inputs into the model.

Urban areas were modeled using the STANDHYD procedure of SWMHYMO. The model employs estimates of SCS CN values (based on soil types and land use). For the purpose of this model, the ratio of total impervious area (TIMP) and the ratio of directly impervious area (XIMP) were considered the same. Calculations for percent imperviousness under post-development conditions are included in Appendix A – Table 1.

The hydrologic response of the area was evaluated using 25-mm, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year and 250-year design storms. All storm hyetographs were generated using a 4-hour Chicago type distribution using IDF curves developed from City of Woodstock data. The Chicago storm event was compared to a 24-hour SCS storm and was found to be the critical storm event since it produced the lowest pre-development flows and highest post-development flows. The Chicago type distribution was used to evaluate peak flows and storage volumes.

The following table provides the IDF parameters used for the Chicago storm events.

Table 1.1 IDF Curve Parameters for Woodstock

Return Period (years)	Woodstock		
	a	b	c
2	630.345	5.326	0.780
5	765.427	4.634	0.758
10	849.300	4.236	0.747
25	984.812	4.236	0.742
50	1072.922	4.067	0.737
100	1152.683	3.871	0.732
250	3162.78	15.65	0.857

$$i = \frac{a}{(t_c+b)^C}$$

Where

i = rainfall intensity (mm/hr)

t_c = time of concentration (minutes)

1.1.2. Hydraulic Modeling

Hydraulic modeling uses the flow estimates to assess various conveyance concerns in the study area watercourses, including water levels for design storm events, velocities, erosion concerns, and culvert capacity.

The study uses a combination of manual calculations (i.e. Manning's equation for steady flow in uniform open channels), culvert calculations, and HEC-RAS hydraulic modeling.

Cross section, channel slopes, and culvert characteristics were obtained through a combination of a review of available topographic mapping and site inspections. In the hydraulic calculations, Manning's n value was assumed to be 0.035 for the main channel and 0.045 for the overbank areas.

1.2. Pre-development conditions

1.2.1. Drainage Areas

The existing pre-development catchment areas to the Lampman and Balls Drain are included in Appendix C - Figure 2 and summarized in the table below.

Table 1.2 Pre-Development Catchment Areas

Catchment ID	Catchment Area (ha)
A	88
B	158
C	221
D	90
E	44

1.2.2. Soils

The soils in the proposed development area consist of sandy loam, loamy sand and clay loam. The Lampman Drain consists of a recent alluvian deposit. A soils map for the study area is provided in Appendix C - Figure 3. For the purposes of hydrologic modeling, the soils were classified as hydrologic soil types B and C.

1.2.3. Land Use

Current land uses are comprised mainly of active and inactive farm lands with wetland areas. Schedule B-1 of Oxford County's Official Plan currently identifies these areas as Agricultural Reserve and Environmental Protection. As well, Schedule C-1 of the same OP further designates the Environmental Protection areas as Provincially Significant Wetlands, Locally Significant Natural Heritage Features and Conservation Authority Land. Isolated woodland patches also occur scattered throughout the study area, comprised largely of dense deciduous cover.

Existing land use in the study area is shown in an aerial photograph provided in Appendix C - Figure 1.

Hydrologic modeling of existing rural conditions also requires an estimate of the time to peak for each sub-catchment. The Bransby Williams formula was used to calculate the time to peak.

$Tp = 0.67*(0.057*L/(Sw^{0.2}*A^{0.1}))$

Where

Tp = time to peak

L = catchment length

Sw = slope

A = catchment area

The table below presents the CN values used given the soil type and cover material and the time to peak used for calculating peak flows in the pre-development model.

Table 1.3 SWMHYMO Pre-Development Input Parameters

Drainage Area	Total Area ha	Soil Type	Cover	CN	Initial Abstraction mm	Catchment Length m	Catchment Slope %	Time to Peak hrs
A	87.8	C	Agricultural	78	7.0	1600	0.30	0.83
B	158.1	B	Agricultural	70	7.0	2500	0.40	1.15
C	145	C	Agricultural	78	7.0	2300	0.50	1.02
C1	75.8	C	Agricultural	78	7.0	1655	0.80	0.71
D	89.7	B	Agricultural	70	5.0	1300	1.10	0.52
E	44.3	B	Agricultural	70	5.0	2200	0.80	1.00
F	53.3	B	Agricultural	70	5.0	1400	0.40	0.72
F1	13.8	B	Agricultural	70	5.0	425	2.70	0.17
G	167.1	C	Agricultural	78	7.0	2000	1.00	0.76
G1	15.8	B	Agricultural	70	6.0	720	0.20	0.48
G2	18.6	C	Agricultural	78	5.0	1300	0.30	0.79
G3	208	B	Woods	60	11.0	2810	0.40	1.26
G4	40.3	C	Agricultural	78	6.5	1115	0.40	0.59
G5	41.1	C	Agricultural	78	6.0	800	0.40	0.42
G6	34.9	C	Agricultural	78	8.0	1180	0.80	0.55
G7	15.9	C	Industrial	86	5.0			
M	48.2	B	Brush	60	10.0	1400	0.70	0.65

1.2.4. Model Results

The following table summarizes the pre-development peak flows for each of the study area catchments.

Table 1.4 Pre-Development Peak Flow Rates

Storm Event	Peak Flow (m^3/s)				
	Catchment A	Catchment B	Catchment C	Catchment D	Catchment E
2-year	0.79	0.82	2.04	0.91	0.29
5-year	1.52	1.62	3.84	1.72	0.55
10-year	2.07	2.25	5.21	2.36	0.75
25-year	2.86	3.15	7.12	3.26	1.03
50-year	3.48	3.88	8.63	3.99	1.26
100-year	4.11	4.63	10.14	4.74	1.50
250-year	6.91				

The pre-development SWMHYMO hydrologic modeling input and output files have been included in Appendix A.

1.3. SWM Design Criteria

1.3.1. Storm Water Management Targets

1.3.1.1. Quality

Based on the Natural Heritage Background Study, September 2006, the fish habitat within the Lampman-Lock Drain between the Gordon Pittock Reservoir and TEMA site has been classified as a Type 2 fish habitat and a Level 3 protection area. The pre-development condition in the Lampman/Locke Drain has exhibited clean groundwater inflows that have provided water quality that is sufficient for northern pike and creek chub spawning. Due to the significance of this drain to the spawning habitats of the local fish, it is recommended that a normal protection level be used when designing the storm water management facilities.

The Balls drain, part of the Grand River watershed, consists of an intermittent watercourse, swales and enclosed tile drains. Due to the length of some of these tile drains, surrounding agricultural activity, and distance away from nearest fish bearing waters, none of these aquatic features represent fish habitat directly or indirectly. Although no fish habitat is directly or indirectly affected, it is recommended that a normal protection level be used when designing the storm water management facility, to protect watercourses downstream of the facility.

The MOE Stormwater Management Planning and Design Manual provides the following storage volume requirements for a wet pond with an enhanced protection level. The water quality storage volume represents the minimum permanent pool size, plus an additional 40 m3/ha for extended detention.

Table 1.5 MOE Normal Protection Water Quality Storage Requirements

Impervious Level	Storage Volume (m^3/ha)*
35%	90
55%	110
70%	130
85%	150

***Includes 40m^3/ha**

1.3.1.2. Erosion

To evaluate the erosion control requirements in the Lampman Drain, continuous modeling was performed using the SWMHYMO model developed for pre and post-development conditions. Continuous modeling facilitates an assessment of an erosion index over long periods, characterizing the magnitude and duration that the velocity of the stormwater in the channel will exceed a critical threshold velocity. For the purposes of the assessment, the critical velocity is taken as 0.17 m/s for a 0.5 mm size of particle (medium sand), which is typical in the bed of the Lampman Drain. The continuous model was performed for a one year period (April to November). Modeling used hourly rainfall data from the London Airport for the year 1985, which was selected as an average year with several significant storm events.

The continuous model output provides a time series of half hour discharges into the channel. The discharge is converted to velocity using rating curves developed from a specific cross-section from an existing HEC-RAS model for the Lampman Drain. In the Lampman Drain, the rating curve (i.e. the relation between depth, flow, and velocity) is influenced by the operating elevation of the Pittock Reservoir downstream. As a result, four different rating curves were developed, based on reservoir levels maintained at different times of the year. Rating curves for the Lampman Drain are provided in Appendix A.

The erosion index for the post-development condition is evaluated by comparing the results to the pre-development conditions to ensure that the erosion index in the post-development model is equal to or less than the erosion index in the pre-development model.

The following table provides the average operating levels of the Pittock Reservoir for the April to November time period.

Table 1.6 Average Operating Levels of the Pittock Reservoir

Time Period	Average Operating Elevation (m)
Apr-May	284.8
May-Jul	286.6
Jul-Sept	285.9
Sept-Oct	284.8
Oct-Nov	284.0

The analysis for the erosion index was reviewed at two different cross-sections of the Lampman Drain as follows.

Cross Section Location 1 (9972) Along the Lampman Drain midway between County Road No. 4 and Township Road No. 3.

Cross Section Location 2 (9937) Along the Lampman Drain just north of Township Road No. 3.

The erosion index was calculated under pre-development conditions and post-development conditions with the proposed stormwater management facilities recommended by this report. For the proposed facilities, a minimum extended detention volume of 125 m^3/ha was considered for erosion control purposes.

The results for the two reaches are summarized in the following table.

Table 1.7 Erosion Index

Location	Erosion Index	
	Pre-Development (m)	Post-Development (m)
1	424	37
2	3254	1725

The results from the continuous model indicate that using 125 m^3/ha are sufficient to prevent an increase in the erosion of the channels downstream of the proposed SWM facilities discharging to the Lampman Drain.

For purposes of this report, it was assumed that 125 m^3/ha of storage will be utilized for the stormwater management facility (SWMF A) discharging to the Balls Drains at the east side of the study area. This assumption can be verified at the time of the detailed design of SWMF A if necessary when additional detailed information is available on the watercourse downstream of SWMF A.

1.3.1.3. Peak Flows

The existing Lampman Drain outlets into the Pittock Reservoir and has a catchment area in excess of 1175 ha. The Pittock Reservoir provides quantity control for areas downstream of the East Woodstock Lands Study Area. The peak flow criteria for the SWM facilities discharging into the Lampman Drain is to ensure that flood elevations in the drain do not increase under post-development conditions. 250-year floodlines in the Lampman Drain are at an elevation of 292.0m. To evaluate peak flow attenuation requirements, a HEC-RAS model was developed for the Lampman Drain from County Road No. 4 to the Pittock Reservoir. The results indicated that uncontrolled peak flows released from the SWM facilities D and E, discharging to the Lampman Drain did not increase downstream flood elevations above 292.0 m. As a result, quantity control storage is not required for these facilities (subject to adequate conveyance to the Lampman Drain). The HEC-RAS outputs are included in Appendix A.

1.3.1.4. Summary of Stormwater Management Design Criteria

The assessment conducted for this report result in the following design criteria for stormwater management facilities.

Table 1.8 Stormwater Management Facility Design Criteria

Storm Water Management Facility	Impervious Level	Water Quality Volume (m^3/ha)	Catchment Area (ha)	Extended Detention Volume (m^3/ha)	Peak Flow Attenuation Level
A	77%	139	88	125	2yr to 250yr
B	69%	129	92	125	2yr to 100yr
C	76%	138	145	125	2yr to 100yr
D	73%	134	104	125	None
E	80%	143	97*	125	None

*The catchment area for SWMF E is 44.3 ha for Catchment E and 53.3 ha for Catchment F.

1.4. Post-development conditions

1.4.1. Land Use

The proposed land use for the entire study area will be a combination of industrial, commercial, residential and open space as indicated in Appendix C – Figure 5.

The table below presents the parameters used for calculating peak flows in the post-development model.

Table 1.9 SWMHYMO Post-Development Input Parameters

Drainage Area	Total Area ha	Soil Type	Cover	Curve Number	Initial Abstraction mm	Catchment Length m	Catchment Slope %	Time to Peak hrs
A	87.8	C	Industrial/Commercial	86	5.0			
B	92.3	B	Industrial	79	5.0			
B1	41.5	B	Agricultural	70	7.0	1650	0.400	0.87
C	145	C	Industrial/Commercial	86	5.0			
C1	75.8	C	Agricultural	78	8.0	1100	0.800	0.47
D	104	B	Industrial/Commercial	79	5.0			
E	44.3	B	Industrial/Commercial	79	5.0			
F	53.3	B	Industrial/Commercial	79	5.0			
F1	13.8	B	Industrial/Commercial	79	5.0			
G	284	C	Industrial	79	5.0			
G1	16.2	B	Open Space/Industrial	69	6.0	1130	0.200	0.75
G2	18.6	C	Open Space/Industrial	79	5.0	1300	0.300	0.79
G3	208.2	B	Brush	60	11.0	2810	0.195	1.45
G4	12.3	C	Open Space/Industrial	79	6.5	800	0.400	0.48
G5	7.5	C	Open Space/Industrial	79	6.0	1115	0.400	0.70
G6	10	C	Open Space/Industrial	79	8.0	500	0.050	0.46
M	48.2	B	Brush	60	10.0	1400	0.700	0.65

The post-development SWMHYMO hydrologic modeling input and output files have been included in Appendix A.

1.4.2. Proposed SWM Facilities

The location of the proposed SWM facilities is shown on the proposed storm servicing plan in Appendix C – Figure 5.

1.4.2.1. SWMF A

SWMF A, servicing the south-east area of the Woodstock East Lands Study under post-development conditions, will service approximately 80 ha of mostly industrial land use area. SWMF A will provide water quality, erosion control and peak flow attenuation for a total catchment area of approximately 88 ha. Flows from the facility will discharge into the existing culvert crossing under Blandford Road north of Oxford County Road No. 2 connecting into Balls Drain.

A 375 mm diameter orifice at invert elevation 295.50 m will be used to provide the water quality and erosion control extended detention storage which will control the average discharge to approximately 121 L/s over a 34 hour period. The SWM facility will attenuate the 2 to 250-year post-development peak flow rates to pre-development levels. This attenuation will be provided by a 2.4m long weir, set at an elevation of 296.18 m.

The following table summarizes the post-development stormwater peak flows at the SWM facility inlet and outlet for various design storms and the required storage.

Table 1.10 SWMF A Proposed Design

Design Storm	Inflow (m^3/s)	Discharge (m^3/s)	Storage (m^3)*
2-year	7.33	0.49	20360
5-year	10.42	1.19	26140
10-year	13.74	1.75	29790
25-year	16.91	2.55	34420
50-year	19.23	3.19	37800
100-year	21.81	3.87	41170
250-year	28.61	6.85	54600

* Not including permanent pool storage of 9,163 m^3

Additional details on SWMF-A are contained in Appendix B.

1.4.2.2. SWMF – B

SWM Facility B, servicing the south area of the Woodstock East Lands Study under post-development conditions, will service approximately 75 ha of mostly industrial land use area. SWMF B will provide water quality, erosion control and peak flow attenuation for a total catchment area of approximately 92 ha, and is also designed to convey pre-development flows from an external area of 42 ha south of the study area. The facility will discharge to an existing culvert under Highway 401 east of Oxford County Road No. 2 connecting into the East Perimeter By-Pass Channel that will be constructed within the Toyota facility site which will subsequently discharge to the Lampman Drain system.

A 375mm diameter orifice at elevation 295.00 m will be used to provide the water quality and erosion control extended detention storage which will control the average discharge to approximately 121 L/s over a 27 hour period. The SWM facility will attenuate the 2 to 100-year post-development peak flow rates to pre-development levels. This attenuation will be provided by a 1.2m long weir, set at an elevation of 295.60 m.

The following table summarizes the post-development stormwater peak flows at the SWM facility inlet and outlet under various design storms and the required storage.

Table 1.11 SWMF B Proposed Design

Design Storm	Inflow (m^3/s)	Discharge (m^3/s)	Storage (m^3)*
2-year	6.96	0.74	19050
5-year	9.97	1.46	25780
10-year	13.14	2.00	30110
25-year	16.18	2.76	35660
50-year	18.45	3.35	39780
100-year	20.71	3.94	43800

* Not including permanent pool storage of 8,513 m^3

Additional details on SWMF-B are contained in Appendix B.

1.4.2.3. SWMF C

SWMF C, servicing the south area of the Woodstock East Lands Study under post-development conditions, will service approximately 131 ha of mostly industrial land use area. SWMF C will provide water quality, erosion control and peak flow attenuation for a total catchment area of approximately 145 ha and is also designed to convey pre-development flows from an external area of 75 ha south of the study area. It is being proposed that an adjacent wetland be utilized for quantity control storage for this facility. The SWM facility will provide the quality and erosion control outside the limits of the wetland and will discharge into the proposed wetland where the wetland will provide the quantity control required to control 2 to 100 year storm events to pre-development levels. The wetland will drain to an existing culvert under Oxford County Road No. 4 north of Oxford County Road No. 2 connecting into the West Perimeter By-Pass Channel that will be constructed within the Toyota facility site which will connect into the Lampman Drain.

A 525 mm diameter orifice at elevation 293.50 m will be used to provide the water quality and erosion control extended detention storage which will control the average discharge to approximately 292 L/s over a 28 hour period. The SWM facility will attenuate the 2 to 100-year post-development peak flow rates to pre-development peak flow levels. This attenuation will be provided by a 4.8m long weir, set at an elevation of 294.45 m.

The following table summaries the post-development stormwater flow conditions at the SWM facility inlet and outlet under various design storms.

Table 1.12 SWMF C Proposed Design

Design Storm	Inflow (m^3/s)	Discharge (m^3/s)	Storage (m^3)*
2-year	14.18	1.12	36530
5-year	20.18	3.00	46310
10-year	24.48	4.49	52300
25-year	29.83	6.55	59650
50-year	34.50	8.22	65110
100-year	38.69	9.99	70310

* Not including permanent pool storage of 26,660 m^3

Additional details on SWMF-C are contained in Appendix B.

1.4.2.4. SWMF D

SWMF D, servicing the north-west area of the Woodstock East Lands Study under post-development conditions will service approximately 93 ha of mostly commercial and industrial land use area with some minor residential land use also. SWMF D will include water quality and erosion controls for a total catchment area of approximately 104 ha. SWMF D does not require 2 to 100 year peak attenuation controls, since the facility discharges to the Lampman Drain.

A catch-basin at elevation 294.50 m will be used to provide the water quality and erosion control extended detention storage which will control the average discharge to approximately 125 L/s over a 30 hour period. During large storm events up to the 100-year design storm stormwater will be directed to the SWM facilities via the minor and major systems. At the high water level of the extended detention storage a 7.2m long weir, set at an elevation of 295.50 m, will allow flows from the facility to discharge to the Lampman Drain.

The following table summaries the post-development stormwater flow conditions at the SWM facility inlet and outlet under various design storms.

Table 1.13 SWMF D Proposed Design

Design Storm	Inflow (m^3/s)	Discharge (m^3/s)	Storage (m^3)*
2-year	8.23	1.94	17200
5-year	11.72	4.38	20500
10-year	15.50	6.21	22600
25-year	19.02	8.80	25300
50-year	21.65	10.73	27300
100-year	24.24	12.67	29100

* Not including permanent pool storage of 9,839 m^3

Additional details on SWMF-D are contained in Appendix B.

1.4.2.5. SWMF E

SWMF E, servicing the northern area of the Woodstock East Lands Study under post-development conditions will service approximately 93 ha of mostly industrial land use catchment area, with minor system and major system servicing. SWMF E will include water quality and erosion controls for a total catchment area of approximately 97 ha. Major overland flows will be split in the catchment, with 44 ha being conveyed will be conveyed to SWMF E, while the remaining 53 ha will conveyed north into a channel traveling under Township Road No. 3 and Oxford County Road No. 4, ultimately discharging to the Lampman Drain. The channel from Oxford County Road No. 4 to the Lampman Drain will require enhancements to allow 7.5 m^3/s of flow to drain into the Lampman Drain during a 100-year event to prevent the overflow and flooding of the channel. Alternatively, a peak flow attenuation facility could be constructed at SWMF F located on the east or west side of County Road 4, as indicated in Appendix C – Figure 5, to control post-development peak flows to pre-development levels. If the overland flows are not controlled to pre-development flow rates by the construction of the SWMF F on the east side of County Road 4, the culverts under Oxford County Road No. 4 and Township Road No. 3 will require replacement to convey the overland flow from the 53 ha to the Lampman Drain. SWMF E does not provide 2 to 100 year peak attenuation controls, since the facility discharges to the Lampman Drain.

A single catch-basin at an elevation of 295.00 m will be used to provide the water quality and erosion control extended detention storage which will control the average discharge of approximately 125 L/s over a 27 hour period. At the high water level of the extended detention storage a 6.0m long weir, set at an elevation of 296.00 m, will allow flows from the facility to discharge to the Lampman Drain.

The following table summaries the post-development stormwater flow conditions at the SWM facility inlet and outlet under various design storms.

Table 1.14 SWMF E Proposed Design

Design Storm	Inflow (m^3/s)	Discharge (m^3/s)	Storage (m^3)*
2-year	7.37	1.77	15900
5-year	8.92	3.60	18600
10-year	9.96	4.72	20000
25-year	12.09	6.00	21600
50-year	13.30	6.86	22600
100-year	14.44	7.67	23500

* Not including permanent pool storage of 10,507 m^3

Additional details on SWMF-E are contained in Appendix B.

1.4.3. Onsite Controls

There are two locations within the study area as indicated on the storm servicing plan in Appendix C – Figure 5 where the use of individual onsite controls in lieu of regional facilities are warranted due to topography and direct access to the Lampman Drain. These two locations are as follows:

- The southwest corner of County Road No. 4 and Township Road No. 3.
- The southwest corner of Blandford Road and Old Township Road No. 2.

Typical on site controls would include the following:

Water Quality (Normal)

- BMP's
- Stormceptor or reviewed equivalent
- Wet ponds
- Combination of above

Water Quantity (2-100 year)

- Parking lot surface storage
- Roof top storage
- Underground storage
- Infiltration (if applicable)
- Dry ponds
- Combination of above

In most instances the on site controls will be approved through site plan approval process and will be under private ownership and maintenance/operation.

Water quantity on site controls would be relevant to only the location at the southwest corner of Blandford Road and Old Township Road No. 2 as the other location has direct access to the Lampman Drain.

1.4.4. Summary Results

Stormwater Management Facilities A through C are to provide peak attenuation controls for storms up to the 100 year storm event, while SWM facilities D and E do not require peak flow attenuation. The table below summarizes the modeled pre and post-development flow rates.

Table 1.15 Pre-Development and Post-Development Results

Design Storm	SWM Pond A		SWM Pond B		SWM Pond C		SWM Pond D		SWM Pond E	
	Pre (m^3/s)	Post (m^3/s)	Pre (m^3/s)	Post (m^3/s)	Pre (m^3/s)	Post (m^3/s)	Pre (m^3/s)	Post (m^3/s)	Pre (m^3/s)	Post (m^3/s)
2-year	0.79	0.49	0.82	0.74	2.04	1.12	0.91	1.94	0.29	1.77
5-year	1.52	1.19	1.62	1.46	3.84	3.00	1.72	4.38	0.55	3.60
10-year	2.07	1.75	2.25	2.00	5.21	4.49	2.36	6.21	0.75	4.72
25-year	2.86	2.55	3.15	2.76	7.12	6.55	3.26	8.80	1.03	6.00
50-year	3.48	3.19	3.88	3.35	8.63	8.22	3.99	10.73	1.26	6.86
100-year	4.11	3.87	4.63	3.94	10.14	9.99	4.74	12.67	1.50	7.67
250-year	6.91	6.85								

To provide both quality and quantity control for the East Woodstock Lands Secondary Plan area, the following approximate land area will be required for the various SWM facilities:

Table 1.16 SWM Facility Area Requirements

SWM Facility	Area (ha)
A	3.8
B	3.1
C	4.7
D	2.2
E	2.0

The SWM facility area calculations are based upon a maximum ponding depth of approximately 3 m including a 1.0 m deep sediment forebay/permanent pool and also allows for 5:1 side slopes, a 2 m wide aquatic safety bench and a 15 m buffer around the perimeter of the facility. The actual area of each SWM facility land area requirement will be refined at the time of preparing draft plans of subdivisions containing the specific SWM facility.

The elevation information provided in this report for the various SWM facilities have been based upon available topographic information and will be refined at the time of the detailed design of each specific SWM facility.

1.5. SEDIMENT AND EROSION CONTROLS

The construction of the SWM facilities and servicing/grading of the lands within the study area should include a rigorous program of sediment and erosion controls to protect downstream watercourses and significant natural areas. These sediment and erosion controls should be detailed on the engineering drawings for the draft plans of subdivisions within the study area and include, but not be limited to, the following.

- light duty and heavy duty silt fencing;
- strawbale filters;
- rock check dams;
- minimizing areas of topsoil stripping;
- temporary sedimentation ponds.

A regular program of inspection and maintenance of the sediment and erosion controls throughout the duration of the construction process and site development must be undertaken.

1.6. MONITORING

1.6.1. SWM Facility Monitoring

Each SWM facility will require an MOE Certificate of Approval. These certificates of approval are applied for during the detailed design process of the SWM facility and construction of the SWM facility cannot commence until such time as the certificate of approval has been obtained.

Typically, each SWM facility certificate of approval has a standard condition that requires the owner to undertake a monitoring process of the SWM facility generally as follows.

- Regular review of water levels in the facility both during rainfall events and afterwards to ensure that the facility is functioning as designed.

- Inspection of sediment accumulation in the sediment forebay once a year.

- General inspection of the facility's condition once a year noting any deficiencies such as side slope erosion, vegetation cover, outlet/inlet structures.

All of these inspections should be logged in a maintenance/inspection log book along with the details of any maintenance or sediment removal that was undertaken.

1.6.2. Study Area Monitoring

To monitor the performance of the SWM facilities for the entire study area that drains towards the Lampman Drain, a pressure gauge can be installed within the cast in place concrete culvert running under Oxford County Road No. 4. The pressure gauge will measure the depth of flow within the culvert, from which average flows can be calculated. This data can be compared to historical data within the site to determine the performance of the SWM facilities for conditions of the Lampman Drain upstream of

Oxford County Road No. 4. This location was selected since the normal operating levels of the Gordon Pittock Reservoir will not affect water elevation within the culvert. Another pressure gauge can be installed within the cast in place concrete culvert running under Blandford Road to monitor the performance for lands draining towards the Balls Drain.

To monitor the water quality of the study area, water tests can be performed in accordance with MOE guidelines. Water tests should be taken at the inlet and outlets of each SWM facility to determine the efficiency of the SWM facility.

1.7. Conclusions

The principal objectives of this report were to provide a systematic review of stormwater management practices for the East Woodstock Lands Study Area and provide preliminary design details and engineering calculations supporting the proposed stormwater management practices and to illustrate that there is sufficient SWM land allocated in the proposed secondary plan of the East Woodstock Lands Study.

The principal conclusions of the report are as follows:

- Construct five stormwater management facilities (SWMF A-E) within the East Woodstock Lands Study Area (providing enhanced water quality and erosion control of 125m^3/ha) as “end-of-pipe” stormwater controls. Three of these facilities, SWMF A-C, are also to include peak flow attenuation storage, Facility A for 2 to 250 year storm events and Facilities B and C for 2 to 100 year storm events.

- SWMF F can be constructed in lieu of constructing an enhanced channel from Oxford County Road No. 4 to the Lampman Drain.

- Implement on site controls on two small areas within the East Woodstock Study Area are warranted due to location and topography.

- A total of approximately 15.8 ha within the East Woodstock Study Area should be dedicated for SWM facilities.

2. SANITARY SERVICING

The sanitary servicing for the study area is shown in Appendix D - Figure 1. The direction of flow and sewer sizing for the various gravity sewers are shown on the drawing. Sanitary pumping stations (SPS 1-3) are also shown on County Road No. 4 just north of Dundas Street, on County Road No. 4 in the vicinity of the Lampman Drain and Blandford Road just north of Dundas Street. Forcemains from the sanitary pumping stations will convey the sewage from the sanitary pumping stations to the existing/proposed City of Woodstock gravity sewers.

2.1. Wastewater Flow Projections

The following assumptions were used to generate the approximate wastewater flow projections for the lands in the study area. The assumptions are consistent with the previously issued County of Oxford Highway 401 and County Road 4 Area Servicing Study by R.V. Anderson Associates Limited (RVA).

- Business park/traditional industrial 0.19 L/s/ha average flow.
- Business park/traditional industrial peaking factor 2.0.
- Residential 0.22 L/s/ha average flow based upon 20 units per ha, 3 person per unit and 320 Lpcd usage.
- Residential peaking factor Harmon.
- Infiltration 0.10 L/s/ha.

The Toyota site was assumed to produce a peak flow of 54 L/s in accordance with information provided by their consultants.

2.2. Catchment Area No. 1

Catchment Area No. 1 being approximately 211 ha (60 ha of which is external to the study area) of residential/service commercial/business park/traditional industrial land use west of County Road No. 4 and south of County Road No. 35 and an additional 50 ha of land on the south side of Dundas Street and east of County Road No. 4 external to the study area will be serviced via gravity sewers as shown in Appendix D - Figure 1 and conveyed to a sanitary pumping station (SPS 1) to be constructed on the east side of County Road No. 4 just north of Dundas Street. This particular sanitary pumping station will also serve as the sanitary outlet for the sanitary servicing of the Toyota site.

It is anticipated that the peak flow from Catchment Area No. 1 will be approximately 114.7 L/s (not including the Toyota site). The Toyota site will apparently produce a peak flow to the sanitary pumping station of approximately 54 L/s. The projected flows and minimum pipe sizes (based upon 0.50% minimum grade unless otherwise noted) for Catchment Area No. 1 are summarized in the following chart.

Table 2.1 Catchment Area No. 1 – Projected Flows and Pipe Data

Node*	Node*	Area (Ha)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
1	2	23	11.0	200	3.0 – 4.0
3	2	33	29.3	250	4.0 – 5.0
2	4	91	57.1	300	5.0
5	4	22	10.6	250**	3.0 – 6.0
4	6	113	67.7	375**	6.0
6	7	113	67.7	375**	6.0
8	9	20	9.6	200	3.0 – 4.0
9	10	20	9.6	200	3.0 – 4.0
11	12	9	4.3	200	3.0 – 4.0
12	10	9	4.3	200	3.0 – 4.0
10	13	98	47.0	300	4.0 – 5.0
13	7	98	47.0	300	4.0 – 5.0
7	SPS1	211	114.7	450**	6.0

*Refer to Appendix D - Figure 1 for node locations.
**Assumed 0.30% grade.

The sanitary pumping station to be constructed on the east side of County Road No. 4 just north of Dundas Street has recently been designed/tendered by the County of Oxford and is scheduled to commence construction in the fall of 2006. The RVA servicing report on which the sanitary pumping station design was based, indicates that the design flow to the sanitary pumping station is 168 L/s which is consistent with the projected flows from Catchment Area No. 1.

The discharge of approximately 168.7 L/s from the sanitary pumping station will be conveyed via a 400 mm diameter forcemain northerly on County Road No. 4 and then westerly on County Road No. 35 (Devonshire Road). A connection to the existing City of Woodstock gravity sewer system will be made at or near Cree Avenue. This forcemain has recently been designed/tendered by the County of Oxford and is scheduled to commence construction in the fall of 2006.

2.3. Catchment Area No. 2

Catchment Area No. 2 being approximately 221 ha of residential/service commercial/business park/traditional industrial land use north of County Road No. 35 and the Toyota site and east and west of County Road No. 4 will be serviced via gravity sewers as shown in Appendix D - Figure 1 and conveyed to a sanitary pumping station (SPS 2) to be constructed in the vicinity of the Lampman Drain near County Road No. 4. The sanitary pumping station should be located outside the limits of the regional floodlines of the Lampman Drain or otherwise be adequately floodproofed.

It is anticipated that the peak flow from Catchment Area No. 2 to the sanitary pumping station will be approximately 114 L/s. The projected flows and minimum pipe sizes (based upon 0.50% grade, unless otherwise noted) for Catchment Area No. 2 are summarized in the following chart.

Table 2-2 Catchment Area No. 2 – Projected Flows and Pipe Data

Node*	Node*	Area (Ha)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
15	16	30	14.4	200	3.0 – 4.0
17	16	25	15.4	200	4.0 – 5.0
16	18	65	34.6	250	5.0
19	18	20	9.6	200	3.0 – 4.0
18	21	85	44.2	300**	4.0
19	21	9	4.3	200	3.0 – 4.0
21	22	100	51.4	300**	5.0
22	23	100	51.4	300	3.0 – 5.0
50	24	20	9.6	200	3.0 – 4.0
24	23	28	13.4	200	3.0 – 4.0
23	SPS 2	128	64.9	300	4.0
25	26	18	8.6	200	3.0 – 4.0
26	28	27	13.0	200	3.0 – 4.0
28	29	27	13.0	200	3.0 – 4.0
30	29	27	13.0	200	4.0
29	31	54	25.9	250**	4.0 – 5.0
31	32	54	25.9	250**	5.0
32	34	85	40.8	300**	5.0 – 6.0
33	34	18	8.6	200	3.0 – 4.0
34	SPS 2	103	49.4	300**	6.0

* Refer to Appendix D - Figure 1 for node locations.
** Assumed 0.30% grade

The discharge of approximately 114.3 L/s from the sanitary pumping station will be conveyed via a 300 mm diameter forcemain northerly on County Road No. 4 and westerly on Lansdowne Avenue to the existing City of Woodstock gravity sewer system. The forcemain will likely have to cross under the Lampman Drain along Lansdowne Avenue using open cut or trenchless methods, or alternatively be connected to the existing culvert and be insulated.

2.4. Catchment Area No. 3

Catchment Area No. 3 being approximately 172 ha of service commercial/business park/traditional industrial land uses north of Dundas Street and west of Blandford Road to Highway 401 will be serviced via gravity sewers as shown in Appendix D - Figure 1 and conveyed to a sanitary pumping station (SPS 3) to be constructed on the west side of Blandford Road just north of Dundas Street.

It is anticipated that the peak flow from Catchment Area No. 3 to the pumping station will be approximately 82 L/s. The projected flows and minimum pipe sizes (based upon 0.50% grade, unless otherwise noted) for Catchment Area No. 3 are summarized in the following chart.

Table 2.3 Catchment Area No. 3 – Projected Flows and Pipe Data

Node*	Node*	Area (Ha)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
35	36	10	4.8	200	3.0 – 4.0
36	37	10	4.8	200	4.0
38	39	14	6.7	200	3.0 – 4.0
39	37	30	14.4	200	4.0
37	40	40	19.2	200	4.0 – 5.0
41	42	13	6.2	200	3.0 – 4.0
42	45	32	15.4	200	4.0 – 5.0
43	44	19	9.1	200	3.0 – 4.0
44	45	39	18.7	200	4.0 – 5.0
45	47	71	34.0	250**	5.0
46	47	23	11.0	200	3.0 – 4.0
47	48	104	49.9	300**	5.0
49	48	14	6.7	200	3.0 – 4.0
48	40	132	63.4	300**	5.0 – 6.0
40	SPS 3	172	82.5	375**	6.0

* Refer to Appendix D - Figure 1 for node locations.

** Assumed 0.30% grade

The discharge of approximately 82.5 L/s from the sanitary pumping station is currently being proposed to be conveyed via a 250 mm diameter forcemain southerly on Blandford Road and westerly on Dundas Street connecting to the proposed gravity sewer system to service Catchment Area No. 1 on Dundas Street just west of Highway 401. MTO approval of the installation of the forcemain under Highway 401 will be required. In the event the discharge from SPS 3 is conveyed to and through Catchment Area No. 1 to SPS 1, then it is likely some upgrades to SPS 1 would be needed in that SPS 1 is currently only designed for flows from Catchment Area No. 1 and the Toyota site and 200 and 300 mm diameter trunk sanitary sewers on Dundas Street and County Road No. 4, respectively, would have to be increased to 375 mm diameter..

Alternatively the discharge from Catchment Area No. 3 and SPS 3 could be directed southerly and then westerly through future extensions of the City of Woodstock gravity sewer system as development south of Dundas Street and Highway 401 occur.

2.5. Existing Downstream Trunk Sanitary Sewers

As noted, the sanitary flows from the study area will be directed to existing City of Woodstock trunk sanitary sewers on Devonshire Avenue and Lansdowne Avenue. As noted in the RVA report, a portion of the existing northeast trunk sanitary sewer along Jack Poole Drive and the Thames River Valley from the manhole located adjacent to the terminals of Wellington Street to the pollution control plant will need to be twinned to accommodate full build-out of the study area. Similarly, it is understood that a portion of the existing northeast trunk sanitary sewer directly west of Lansdowne Avenue will need to be replaced or twinned to accommodate the planned residential development in the Lansdowne Avenue/Devonshire Avenue area.

As noted in the RVA report, the City/County should undertake a monitoring program of these existing trunk sanitary sewers in order to better define the existing flows and available surplus capacity.

3. STORM SEWER SERVICING

Stormwater runoff from the study area and any external areas will be conveyed to approved outlets via storm sewers for minor system and overland flow routes for major system. The minor system being the storm sewer system is shown on Appendix D - Figure 2 complete with pipe sizes, direction of flow and catchment areas.

Stormwater management was addressed in earlier sections of this report.

3.1. Design Criteria

The City of Woodstock uses the Rationale method for the design of the storm sewers and assumes the following:

- a = 712.74 b = 5.326 c = 0.797
- initial inlet time = 10 minutes
- c = 0.70 for industrial/business park

The vast majority of the catchment areas are industrial/commercial thus a constant C = 0.70 has been used in the review of the storm sewer system.

3.2. Projected Storm Sewer Flows

The following is a summary of projected minor storm flows for the storm sewer systems conveying flows to SWMF A to E based upon an assumed pipe grade of 0.50% and an average pipe velocity of 1.5 m/s.

Table 3.1 SWMF A – Projected Minor System Flows

Node*	Node*	Area (Ha)	Tc (mins)	I (mm/hr)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
25	26	14	14.4	66	179.8	675-900	2.5-3.0
26	27	39	23.2	49	3719	1050-1350	3.5-4.5
28	27	5	13.3	69	671	675-750	2.5
27	29	49	27.0	45	4290	1350-1500	4.5-5.0
30	29	13	14.4	66	1670	675-1050	2.5-3.0
29	31	80	33.7	38	5916	1500-1650	5.0
31	SWMF	80	33.5	37	5760	1650	5.0

*Refer to Appendix D - Figure 2 for node locations.

Table 3.2 SWMF B – Projected Minor System Flows

Node*	Node*	Area (Ha)	Tc (mins)	I (mm/hr)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
32	33	20	15.5	63	2452	675-1200	2.5-3.0
33	34	43	25.5	46	4107**	1200-1500	3.0-3.5
34	35	43	26.2	45	4023**	1500	3.5
36	35	12	15.0	65	1517	675-975	2.5-3.0
35	SWMF	55	26.5	45	5074**	1500	4.0

*Refer to Appendix D - Figure 2 for node locations.
**Includes 258 L/s offsite discharge.

Table 3.3 SWMF C – Projected Minor System Flows

Node*	Node*	Area (Ha)	Tc (mins)	I (mm/hr)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
1	2	23	17.3	59	2640	675-1200	2.5-3.0
2	3	57	24.0	48	5136	1200-1500	3.0-3.5
4	3	22	13.9	67	2867	675-1200	2.5-3.0
3	SWMF	79	26.0	45	6741	1650	4.0
5	6	9	22.3	50	875	675-825	2.5-3.0
37	6	20	22.2	51	1985	675-1050	2.5-3.0
6	8	29	29.9	42	3170**	1350	3.0-3.5
8	9	48	37.0	36	4162**	1500	3.0-4.0
9	SWMF	48	37.2	35	4069**	1500	3.0-4.0

*Refer to Appendix D - Figure 2 for node locations.
**Includes 800 L/s offsite discharge.

Table 3.4 SWMF D – Projected Minor System Flows

Node*	Node*	Area (Ha)	Tc (mins)	I (mm/hr)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
10	11	30	20.5	53	3117	675-1350	2.5-3.0
12	11	25	16.6	61	2960	825-1350	2.5-3.0
11	13	61	27.1	45	5290	1500	3.5
14	13	21	17.8	58	2370	675-1200	2.5-3.0
13	15	82	29.8	42	6670	1650	4.0
16	15	12	13.3	69	1611	675-975	2.5-3.0
15	SWMF	101	31.0	41	8058	1800	4.0

*Refer to Appendix D - Figure 2 for node locations.

Table 3.5 SWMF E – Projected Minor System Flows

Node*	Node*	Area (Ha)	Tc (mins)	I (mm/hr)	Peak Flow (L/s)	Pipe Dia. (mm)	Typ. Depth (m)
17	18	15	16.1	62	1760	675-1050	2.5-3.0
18	19	27	22.2	51	2679	1200	3.5
20	21	15	15.5	64	1868	675-1050	2.5-3.0
21	19	27	20.0	54	2837	1200	3.5
19	22	54	27.7	44	4623	1500	4.0
22	23	75	35.5	37	5400	1650	4.5
24	23	18	15.5	63	2206	675-1200	2.5-3.0
23	SWMF	93	35.0	37	6696	1650	4.5

*Refer to Appendix D - Figure 2 for node locations.

3.3. Existing Storm Drainage Tile

It should be noted that a significant amount of existing tile drainage pipe ranging in size from 300 – 1050 mm diameter exists on County Road No. 4 both north and south of Dundas Street. This existing drainage tile may upon further detailed review of condition and inverts are sufficient to remain in place and provide for the minor system drainage along County Road No. 4 to SWMF C.

4. WATER SERVICING

The water servicing, both trunk and local watermains, for the study area is shown on Appendix D - Figure 3. The water servicing for the study area will consist of:

- Connections to and extensions of existing trunk watermains.
- New trunk watermains.
- New local watermains.
- New water tower.

4.1. Design Criteria

The water distribution system should be designed in accordance with MOE Design Guidelines and the requirements of the County of Oxford. Typical design criteria would be as follows:

- Pressures should be greater than 275 kPa during peak hour demand conditions;
- Pressures should be greater than 140 kPa under simultaneous maximum day demand and fire flow conditions;
- Pressures should be less than 700 kPa under all demand conditions;
- Pressures should be between 350 kPa and 550 kPa during maximum day demand conditions;
- The system should be sized to supply a fire flow of 159 L/s for a duration of 3 hours.

The water distribution system will also have to have due regard to such detailed design criteria such as maximum velocities, water age/turnover, etc. which are also provided in the MOE Design Guidelines.

4.2. Proposed Water Tower

The County of Oxford will be proceeding shortly with the construction of a new water tower on the north side of Dundas Street just west of Highway 401 as shown in Appendix D - Figure 3. The water tower will have a maximum high water elevation of approximately 353.0 m which according to the County of Oxford will be sufficient to provide adequate pressure to the entire study area.

4.3. Trunk Watermains

A network of trunk watermains as shown in Appendix D - Figure 3 will be proposed along the main roads within the study area generally as follows.

- Dundas Street: 400/500 mm dia. trunk watermain from the existing 300 mm watermain west of County Road No. 4 to the proposed water tower.
- Dundas Street 300 or 400 mm dia. trunk watermain from the proposed water tower to Blandford Road.
- Blandford Road 300 or 400 mm dia. trunk watermain from Dundas Street to Township Road No. 3.
- Township Road No. 3 300 or 400 mm dia. trunk watermain from Blandford Road to County Road No. 4.
- County Road No. 4 400 mm dia. trunk watermain from Township Road No. 3 to Dundas Street.
- Devonshire Avenue 400 mm dia. trunk watermain from the existing 400 mm watermain at Landsdowne Avenue to County Road No. 4.
- Landsdowne Avenue 300 or 400 mm dia. trunk watermain from the existing system to County Road No. 4.

The network of trunk watermains will provide for the efficient distribution of water for domestic and fire protection purposes to the various local watermains to service the various phases of the study area.

MTO approval will be required in regards to the installation of the trunk watermain installations under Highway 401 at Dundas Street and Township Road No. 3. The MTO has indicated that proposed highway crossings should occur outside of the main interchange location. The trunk watermain installations on both County Road No. 4 and Landsdowne Avenue at the Lampman Drain will require the watermain to be installed under the Lampman Drain via open cut or trenchless installation methods or alternatively be connected to the large concrete culverts and be insulated.

The County of Oxford will be proceeding shortly with the construction of the 400/500 mm diameter trunk watermains on Dundas Street, the 400 mm diameter trunk watermain on Devonshire Avenue and the 400 mm diameter trunk watermain on County Road No. 4 from Dundas Street to Devonshire Avenue.

The County of Oxford have noted that further hydraulic modeling of the trunk watermains will be required in the future as further information becomes available on the water demands of the entire water distribution system servicing the City of Woodstock and possible adjacent local interests.

4.4. Local Watermains

A network of local watermains as shown in Appendix D - Figure 3 will be required to service the various phases of the study area. The local watermains will generally range in size between 150 – 300 mm diameter and will provide both domestic and fire protection servicing.

5. CONSTRUCTION COST ESTIMATES

The construction cost estimates for roads, sanitary sewers, storm sewers, stormwater management facilities, watermains, sanitary pumping stations and forcemains based upon 2006 construction costs are included in Appendix E.