

7.0 Stormwater Management



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7.1 Introduction and Background

OPA 289 policy 8.4.5 states that, “*The management of water resources within the North Oakville West Planning Area shall be undertaken in accordance with the directions established in the North Oakville Creeks Subwatershed Study (NOCSS). No amendments to the Secondary Plan shall be required to implement the recommendations of the Subwatershed Study or for changes to the number or location of stormwater management facilities in accordance with the policies of Section 8.6.2.2.a) of this Plan*”.

Section 6.0 of the NOCSS presents the recommended Management Strategy for the North Oakville Creeks Subwatershed. It includes strategies for land use management, SWM, terrestrial and wetland resources management, riparian corridor management, rehabilitation, remediation and monitoring plans. The goals, objectives, and targets of the Management Strategy are set out in NOCSS Section 6.2.

The NOCSS Section 6.3.6 discusses the SWM component of the Management Strategy. It includes discussion on hydrology, peak flow control, hydrogeology, water quality, fisheries protection, low impact development, source pollution prevention and various types of SWM measures.

In accordance with NOCSS and the EIR/FSS ToR, this section discusses required SWM related topics for the Subject Property within the 407 West Employment Area in details. It includes

- Stormwater Management Objectives and Proposed Strategy (Sections 7.2 and 7.3);
- Development of Hydrological Model (Section 7.4);
- Erosion Control Analysis (Section 7.5);
- Hydrologic Flow Regimes Analysis (Section 7.6);
- Topographic Depression Volumes (Section 7.7);
- Design of Stormwater Management Facilities (Section 7.8); and,
- Regional Storm Downstream Impacts (Section 7.9);

The following studies, technical guidelines and references were reviewed and used in the preparation of this section:

- North Oakville Creeks Subwatershed Study (NOCSS), 2006;
- North Oakville Creeks Subwatershed Study Addendum, 2007;
- River & Systems: Flooding Hazard Limit, Technical Guide, Ontario Ministry of Natural Resources, 2002;
- Stormwater Management Planning and Design Manual, MOECC, March 2003;
- Low Impact Development Stormwater Management Planning and Design Guide, CVC and TRCA, 2010;
- Guelph All-Weather Sequential-Events Runoff Model (GAWSER) Reference Manual, April 1996, Schroeter and Associates; and,
- Hydrology of Floods in Canada, A Guide to Planning and Design, NRC, 1989.

7.2 Stormwater Management Objectives

The SWM plan is intended to address multiple objectives including peak flow control (water quantity), water quality, erosion control and maintaining existing hydrologic water balance. The recommended SWM approach detailed in NOCSS is as follows:

- **Water Quantity:** The NOCSS recommends that SWM targets include control of the peak flow to pre-development levels for the 2-year to 100-year return period events and the Regional Storm. The modeling of pre-development conditions to establish unit flow rate targets for quantity control purposes has been completed as part of NOCSS and presented in Revised Target Unit Area Peak Flow Rates Table (07.06.27) included in the NOCSS Addendum, 2007. The proposed SWM facilities within the study area are designed to provide Regional Storm controls.
Addressed in Sections 7.4, 7.8 and 7.9.
- **Infiltration:** The NOCSS recommends that infiltration levels be maintained as close to current levels as possible to reduce impacts on groundwater systems. The overall strategy is to provide as many opportunities for infiltration as possible in the developed areas. This can be achieved by various techniques that take advantage of physical settings (i.e., soil conditions and topography), as well as, best available technology and management practices.
Addressed in Section 4.0.
- **Water Quality:** The NOCSS recommends meeting MOECC's Enhanced Level of water quality protection (Level 1) when sizing SWM facilities for water quality control, phosphorus control and fisheries protection. The Town requires that there be no-net increase in phosphorus loadings as a result of development. This objective will be met with the use of enhanced Level SWM facilities and as a result, there is no requirement to further analyze phosphorus loadings during development approvals. Temperature controls at SWM facility outlets can be provided through bottom draw outlets and rock filtration measures combined with shading.
Addressed in Section 7.8.
- **Erosion Control SWM Facility Sizing:** The NOCSS recommends that detailed erosion threshold analyses be required as part of an EIR/FSS so that existing channel erosion or aggradation is not exacerbated by development. The continuous hydrologic simulation was conducted using the Guelph All-Weather Sequential-Events Runoff (GAWSER Version 6.9.25) software to evaluate and confirm the appropriate extended detention storages are provided by the SWM facilities.
Addressed in Sections 7.4 and 7.5.
- **Topographic Depressions:** The NOCSS recommends that the storage within the topographic depressions be refined and checked against the storage within proposed SWM facilities in the EIR subcatchment area to verify that the SWM facility storage accounts for the depression storage.
Addressed in Section 7.7.
- **Stormwater Management Applications:** The NOCSS recommends the use of a hierarchy of stormwater controls with preference for source control, followed by conveyance system control, with less reliance on end-of-pipe control. In addition, where feasible, the use of infiltration measures, including the diversion of drainage to pervious surfaces, as well as, designed infiltration facilities,

surface retention, and storage is encouraged, to help maintain pre-development water balance conditions. NOCSS identifies the requirement for end-of-pipe SWM facilities for water quality and quantity control, it also recommends that consideration be given to alternative management measures to meet the SWM objectives and targets. In this regard, the NOCSS discusses alternative low impact development techniques, various source pollution protection programs and alternative SWM practices to be considered.

Addressed in Section 7.8.

7.3 Stormwater Management Alternatives and Proposed Approach

7.3.1 Evaluation of Stormwater Management Alternatives

NOCSS identifies the requirement for end-of-pipe SWM facilities for water quality and quantity control, it also recommends that consideration be given to alternative management measures to meet the SWM objectives and targets. In this regard, the NOCSS discusses alternative low impact development techniques, various source pollution protection programs and alternative SWM practices to be considered. As required by NOCSS and the EIR/FSS ToR, in this section the alternative SWM Practices are described, evaluated for application in the development, and a preferred approach is selected to satisfy NOCSS SWM goals, objectives, and targets.

7.3.1.1 Stormwater Management Practices

Stormwater Management Practices (SWMPs) can be applied locally at the lot level, along conveyance systems, or as end-of-pipe facilities. Each SWMP varies in its effectiveness and utility to address different water quality and quantity concerns.

Lot level SWMPs applicable to the concept plan for the subject lands include discharging clean roof runoff to pervious surfaces, infiltration swales, or soakaway pits, rain gardens, pervious pavements, green roofs and reduced lot gradings. These SWMPs encourage infiltration to groundwater; help to reduce the volume of water travelling to the major and minor systems and help to preserve hydrologic regime. Infiltration techniques are also effective for reducing runoff temperature increases and removing sediment, heavy metals and nutrients from runoff. Generally, infiltration techniques are recommended in areas where the minimum infiltration rate is equal to or greater than 15 mm/hr. Where practical, this is a preferable SWMP.

Conveyance SWMPs include drainage swales, infiltration trenches, exfiltration/filtration system and filter strips. Drainage swales and filter strips rely on various forms of vegetation to enhance the pollutant removal, habitat value and aesthetics of a development. Without other SWMPs they can effectively treat stormwater runoff from small areas (< 2 ha).

End-of-pipe SWMPs, such as extended detention ponds, artificial wetlands and infiltration basins can remove moderate to high levels of sediment from stormwater. Extended detention wetlands are also efficient in removing nutrients from stormwater during the summer months. These facilities can be effective for a wide variety of land areas.

7.3.1.2 Evaluation

In reviewing these options for inclusion in the proposed SWM Plan, these alternatives were evaluated on the basis of capabilities, limitations, anticipated risk profiles and physical constraints associated with their implementation. This analysis included the following factors:

- Ability to meet SWM goals, objectives and targets;
- Suitability of soils and groundwater conditions;
- Site topography and size of contributing drainage areas;
- Compatibility with urban form and natural features; and,
- Municipal servicing requirements.

The evaluation of alternative SWMPs has made use of relevant guidelines in the *MOECC Stormwater Management Planning and Design Manual, March 2003*, and the *Low Impact Development Stormwater Management Planning and Design Guide, CVC and TRCA, 2010*.

Lot Level SWMPs

The NOCSS recommends that the existing infiltration over the site be maintained but acknowledges that implementation is difficult (NOCSS analysis report Section 5.5.2, page 5-11). An approach to maintaining water balance has been implemented with infiltration techniques. These SWMPs are generally less feasible for the proposed industrial/commercial development within the West Branch of Fourteen Mile Creek watershed (north of Dundas Street) as:

- The soils have been characterized as clay loams that have a relatively low infiltration potential; and,
- Industrial/commercial development typically has a high imperviousness for viable site plans; therefore, there would be minimal opportunity to implement infiltration techniques.

To address the shortfall in infiltration over the developed blocks, infiltration trenches along the watercourse corridor limits have been established that will collect surface drainage from non-developed areas and allow it to infiltrate along the banks and outer meander belts of the watercourses, providing passive infiltration and water sources that will mimic pre-development hydrology inputs to baseflow maintenance. Please refer to Section 4.0 for the post-development water balance analysis, demonstrating a pre- to post-development match in overall infiltration targets to meet the NOCSS objectives.

However, the following opportunities to integrate SWMPs at the lot level will be considered at the detailed design stage. While they will be less effective at promoting pre-development rates of infiltration due to the tight soils, they will assist in the conversion of rainfall to evapotranspiration rather than runoff:

- Harvesting of rainwater from rooftops for non-potable uses (e.g., irrigation, toilet flushing) using rain barrels or cisterns;
- Installation of green roofs;
- Integration of soakaways (e.g., infiltration trenches or chambers) below landscaped areas. Section 4.4.4.4 discusses infiltration swales proposed along the edge of the buffers to the natural features to infiltrate a portion of the roof runoff; and,
- Incorporation of bioretention areas, rain gardens or biofilters into the landscape plans for the site.

When combined with the infiltration trenches at the boundaries of the watercourse corridors, post-development water balance has been demonstrated to address the NOCSS targets. Therefore, the techniques listed above should form recommendations for the individual block SWM plans to be followed at the site plan application stage.

Conveyance SWMPs

Conveyance SWMPs, such as pervious pipe systems and wide-bottom swales are not practical for the proposed industrial/commercial development within the West Branch of Fourteen Mile Creek watershed (north of Dundas Street) as the soils have been characterized as clay loams that have a low infiltration potential and they may result in overly wide rights of way or inefficient roadway longevity due to imperfect drainage.

Incorporation of vegetated filter strips and narrower open channel swales to intercept and treat parking lot and road runoff will be considered on the site where opportunities exist during the detailed design stage and implemented as recommended techniques at the site plan application stage for the individual development blocks.

End-of-Pipe SWMPs

SWM wet pond (i.e., extended detention ponds) can remove moderate to high levels of sediment from stormwater. When properly designed, SWM wet ponds can effectively achieve peak flow control (water quantity), provide water quality and erosion control, and maintain existing hydrologic water balance.

7.3.2 Proposed Stormwater Management Approach

As indicated previously, the objective of the SWM plan is to provide satisfactory storm drainage from the site and ensure the long-term sustainability of the receiving watercourses. The primary factor from a water quality perspective is to remove sediment and associated pollutants from stormwater runoff, thus preventing them from entering the receiving watercourse. Based upon the considerations discussed in the previous section, two extended detention wet ponds (i.e., Ponds 2 and 3) are proposed for the development area within the Subject Property. Two additional extended detention wet ponds (i.e., Ponds 1 and 5) are recommended for the adjacent development areas to the west. For study purposes, an additional wet pond located west of Tremaine Road and south of Highway 407 is also included in the analysis to reflect the ultimate development conditions as per Tremaine and Dundas Secondary Plan Subwatershed Study, 2009.

It is recognized that the complete development of the site and the construction of the infrastructures (e.g., SWM facilities, roads, culverts, etc.) will not occur simultaneously. Therefore, progressive **development phases** are adopted for the proposed study. **Table 7.3.1** summarizes the description of the development phases with their associated construction of infrastructures.

Table 7.3.1 – Development Phases

Development Phases	Estimated Development Area (ha) ¹⁾	SWM Facilities (Ponds)	Channel Realignment (Reach)	Roads ³⁾	Crossings ³⁾ (Reach)
Phase 1A	17.8 ²⁾	Pond 2	None	• Section of Avenue Two • Section of Burnhamthorpe Road Extension	None
Phase 1B	33.4	Pond 2 Pond 3	14W-21 14W-22	• Section of Avenue Two • Section of Avenue Three • Burnhamthorpe Road Extension between Avenue Two and Avenue Three	• Burnhamthorpe Road Extension at 14W-16 • Burnhamthorpe Road Extension at 14W-22
Phase 2	60.0	Pond 2 Pond 3	14W-21 14W-22 14W-23	• Section of Avenue Two • Section of Avenue Three • Burnhamthorpe Road Extension between Avenue Two and Avenue Three • Section of Avenue One	• Burnhamthorpe Road Extension at 14W-16 • Burnhamthorpe Road Extension at 14W-22 • Avenue One at 14W-22 • Avenue One at 14W-16
Ultimate Phase	103.7	Pond 2 Pond 3 Pond 1 Pond 5 Tremaine Pond	14W-21 14W-22 14W-23	• Avenue One • Avenue Two • Avenue Three • Burnhamthorpe Road Extension between Tremaine Road and Avenue Three	• Burnhamthorpe Road Extension at 14W-16 • Burnhamthorpe Road Extension at 14W-22 • Avenue One at 14W-22 • Avenue One at 14W-16

1) Includes SWM Facility (Pond) Blocks

2) Includes a Section of Dundas Street Expansion

3) Conceptual information is provided for reference purposes only.

All the proposed facilities will be wet ponds designed to provide stormwater quality (enhanced level), erosion and quantity control (up to Regional event) in accordance with the criteria referred to in the NOCSS. Additional details of the proposed wet ponds are provided later in the chapter. As discussed, lot level controls and conveyance controls will be used to a limited degree as initial means of control but as the level of implementation cannot be determined until further into the development process, the end of pipe SWM facilities have provided full redundancy to ensure the required level of treatment are provided.

Note that in order to allow a uniform and sustained level of flow to be maintained in the Reach 14W-12A channel, it is required to divert flows from rooftops of the proposed buildings. To properly utilize the flows from rooftop, it is assumed that the roof drains will be installed to provide a controlled unit flow rate of 41 L/s/ha at a maximum water depth of 0.15 m on the rooftops. The rooftop storages have been incorporated in the hydrological models for the proposed conditions. Details are discussed in Section 7.4.3. Information on the conceptual design of rooftop controls are included in the **Appendix 7.6**. The results of the proposed roof controls, including the roof release rates, water ponding depths, utilized ponding storages for the various storm events from 2- to 100- year and Regional storm, are included in the **Appendix 7.6**.

The computational hydrological simulation was performed by using the GAWSER (Guelph All-Weather Sequential-Events Runoff) hydrologic model. The existing GAWSER model for the West Branch of Fourteen Mile Creek, north of Dundas Street developed as part of the NOCSS study was obtained from CH. The obtained model was revised and updated as required by NOCSS and the EIR/FSS ToR to evaluate the hydrological impact and size the proposed SWM facilities for the subject study.

Low Impact Development and Water Balance

Infiltration measurements, such as infiltration swales, are proposed to maintain existing water balance target for the developments of the subject property. Based on the local soil conditions, infiltration could be used as one effective low impact development (LID) measures in the SWM practice. Together with other lot level, conveyance and end of pipe controls, a treatment train approach will be established for the proposed development. An example of treatment train approach for the proposed development includes a combination of green roof, infiltration trench, bioswale and SWM wet pond. Section 4.0 of hydrogeology study discusses the post-development water balance mitigation strategy in detail. The locations of the proposed infiltration measurements are shown in **Figure 4.7**.

The application of these mitigation measures was constrained by site conditions and final design proposal (site plan). Detailed analysis on the implementation of the infiltration and other LIDs will be refined during detailed engineering design stage based on the best available design guidance information available at the time.

Furthermore, since most hydrological models have limitations to quantify infiltration measurements, consequently, it is conservative not to incorporate infiltration measurements in the GAWSER hydrological model for the subject study.

7.4 Development of GAWSER Hydrologic Model

7.4.1 *Modelling Methodology*

The GAWSER is a modified version of the popular HYMO program developed by the USDA in the early 1970's and was originally developed by the University of Guelph in 1977. In 1996, it was updated and further developed principally by Dr. H. Schroeter. The model has physically-based computational procedures that has evolved over the past 35 years as an available 'Canadian-made' hydrologic modelling tool for water management, planning and operations. It has seen wide application in more than 90 watershed studies in Ontario, including the North Oakville Creeks.

The GAWSER model is a deterministic storm-event hydrologic model which can be used to simulate major hydrologic processes or stream flow hydrographs resulting from precipitation inputs for the purpose of planning, design or evaluating the effects of physical changes in the drainage basin. In GAWSER, precipitation inputs can be defined in terms of rainfall, snowmelt or a combination of both. Recent developments of GAWSER model include features for the statistical analyses of flow data for the determination of erosion indices.

By applying the single storm events (2- to 100- year and Regional event), the GAWSER model was used to determine the design flow hydrographs along water courses and size the proposed SWM facilities for the subject study area. The model was also used for continuous simulations for the purposes of erosion control analysis and hydrologic flow regime analysis. The meteorological input data (hourly rainfall series, temperature, and wind speed) for continuous simulation was directly retrieved from the original GAWSER model provided by CH. It was obtained from the Hamilton Royal Botanical Garden (Hamilton RBG) gauge station (#615330) and extends from 1962 to 1992.

7.4.2 *Existing Conditions*

The North Oakville EIR/FSS ToR states that, “The modeling of predevelopment conditions to establish unit flow rate targets for quantity (flood) control (2-year through Regional Storm flows) purposes has been completed as part of NOCSS. Further modelling of predevelopment conditions is not required for this purpose. SWM ponds are to be sized to meet unit flow rate targets.”

However, it has been recognized that the background data to support the model for predevelopment conditions used in NOCSS is outdated. Therefore, it is prudent to revise the original GAWSER model to reflect the latest existing sub-catchment drainage boundaries for the West Branch of Fourteen Mile Creek from NOCSS based on 2002 Town’s topographic mapping. Furthermore, the drainage areas from the existing Highway 407 corridor and the existing Regional Road No.5 (Dundas Street) were also delineated and included in the revised existing conditions model. The resulting peak flows (2-year through Regional Storm flows) from the revised model are deemed to be the target flows for the proposed developments.

The comparison of the drainage areas to all EIR Flow Nodes (including Culverts at Highway 407 and Culverts at Dundas Street) between the original NOCSS and the current study are presented in **Table 7.4.1**. **Table 7.4.2** lists the original Unit Flow Rates (UFRs) and Peak Flow Rates (PFRs) from NOCSS. The revised UFRs and PFRs for all EIR Flow Nodes are summarized in **Table 7.4.3**. **Figure 7.4.1** shows the revised existing catchments. The GAWSER model schematic for existing conditions is shown in **Figure 7.4.1-SCH**. All other detailed information, including catchment parameters are included in **Appendix 7.2**. **Appendix 7.2** also includes a comprehensive flow table (**Table APP-7.2**) including the existing peak flows from the GAWSER model at all EIR Flow Nodes (Culverts at Highway 407 and Dundas Street) and all Reference Flow Nodes.

Table 7.4.1 – Comparison of Catchment Areas between Original NOCSS and Updated Study

To EIR Nodes		Existing Drainage Area (ha)		Difference (%)	EIR Sub-catchment
		NOCSS	WSP*		
Culverts at HWY 407	FM-1	149.4	118.5	-21%	FM 1001
	FM-2	29.4	27.3	-7%	FM 1002
	FM-3	125.7	119.1	-5%	FM 1003A, FM 1003B
	FM-4	7.3	6.8	-7%	FM 1004
	FM-5	30.3	35.6	17%	FM 1005
	FM-6	33.5	33.6	0%	FM 1006
	FM-7	162.8	170.8	5%	FM 1007A, FM1007B, FM1007C, FM1007D
	FM-8	5.3	5.9	12%	FM 1008
Culverts at Dundas Street	FM-D2	46.6	31.4	-33%	FM1102
	FM-D3	11.7	14.4	23%	FM 1103
	FM-D4	424.0	397.2	-6%	FM1001, FM1002, FM1104, FM1003A, FM1003B, FM1004, FM 1105
	FM-D4a	15.2	16.5	9%	FM1106
	FM-D5	340.0	350.5	3%	FM1005, FM1006, FM1007A, FM1007B, FM1007C, FM1007D, FM1008, FM1108, FM1107, FM1109
Total		1381.2	1327.6	-4%	

* WSP updated drainage areas based on 2002 Town of Oakville topographic mapping.

Table 7.4.2 – Unit Flow Rates and Peak Flow Rates from Original NOCSS

EIR Node		Original NOCSS Drainage Area (ha)	Flow Type ¹	Return Period (Year)						
				2	5	10	25	50	100	Regional
Culverts at HWY 407	FM-1	149.4	UFR (m ³ /s/ha)	0.006	0.010	0.012	0.015	0.017	0.020	0.049
			PFR (m ³ /s)	0.94	1.48	1.79	2.27	2.59	2.93	7.32
	FM-2	29.4	UFR (m ³ /s/ha)	0.008	0.012	0.015	0.019	0.021	0.024	0.056
			PFR (m ³ /s)	0.23	0.36	0.43	0.55	0.63	0.71	1.65
	FM-3	125.7	UFR (m ³ /s/ha)	0.006	0.009	0.011	0.014	0.016	0.018	0.047
			PFR (m ³ /s)	0.71	1.14	1.40	1.79	2.05	2.32	5.95
	FM-4	7.3	UFR (m ³ /s/ha)	0.001	0.004	0.006	0.008	0.010	0.012	0.041
			PFR (m ³ /s)	0.01	0.03	0.04	0.06	0.08	0.09	0.30
	FM-5	30.3	UFR (m ³ /s/ha)	0.004	0.008	0.011	0.014	0.017	0.020	0.052
			PFR (m ³ /s)	0.13	0.25	0.33	0.44	0.51	0.59	1.57
	FM-6	33.5	UFR (m ³ /s/ha)	0.005	0.009	0.011	0.015	0.018	0.021	0.055
			PFR (m ³ /s)	0.15	0.29	0.38	0.51	0.60	0.69	1.83
	FM-7	162.8	UFR (m ³ /s/ha)	0.006	0.010	0.013	0.016	0.019	0.021	0.053
			PFR (m ³ /s)	0.99	1.64	2.05	2.65	3.05	3.48	8.68
	FM-8	5.3	UFR (m ³ /s/ha)	0.001	0.008	0.013	0.019	0.024	0.029	0.073
			PFR (m ³ /s)	0.01	0.04	0.07	0.10	0.13	0.15	0.39
Culverts at Dundas Street	FM-D2	46.6	UFR (m ³ /s/ha)	0.007	0.011	0.013	0.017	0.020	0.022	0.054
			PFR (m ³ /s)	0.31	0.51	0.62	0.80	0.92	1.04	2.50
	FM-D3	11.7	UFR (m ³ /s/ha)	0.010	0.016	0.019	0.024	0.028	0.031	0.065
			PFR (m ³ /s)	0.12	0.19	0.23	0.28	0.32	0.36	0.76
	FM-D4	424.0	UFR (m ³ /s/ha)	0.006	0.010	0.012	0.015	0.017	0.020	0.049
			PFR (m ³ /s)	2.62	4.17	5.09	6.49	7.42	8.39	20.96
	FM-D4a ²	15.2	UFR (m ³ /s/ha)	0.013	0.020	0.024	0.030	0.035	0.039	0.073
			PFR (m ³ /s)	0.20	0.31	0.37	0.46	0.53	0.59	1.11
	FM-D5	340.0	UFR (m ³ /s/ha)	0.006	0.010	0.013	0.017	0.019	0.022	0.055
			PFR (m ³ /s)	2.01	3.43	4.35	5.68	6.60	7.56	18.73

1) UFR = Unit Flow Rate, PFR = Peak Flow Rate

2) Since UFR at culvert FM-D4A is not specified in NOCSS, the UFR based on Existing Flow from Original NOCSS Model Catchment FM-1106 is used

Table 7.4.3 – Revised Unit Flow Rates and Peak Flow Rates from Current Study

EIR Node		Original NOCSS Drainage Area (ha)	Flow Type ¹	Return Period (Year)						
				2	5	10	25	50	100	Regional
Culverts at HWY 407	FM-1	118.5	UFR (m ³ /s/ha)	0.006	0.010	0.012	0.015	0.017	0.020	0.049
			PFR (m ³ /s)	0.75	1.17	1.42	1.80	2.05	2.32	5.80
	FM-2	27.3	UFR (m ³ /s/ha)	0.008	0.012	0.015	0.019	0.021	0.024	0.056
			PFR (m ³ /s)	0.21	0.33	0.40	0.51	0.58	0.66	1.53
	FM-3	119.1	UFR (m ³ /s/ha)	0.006	0.009	0.011	0.014	0.016	0.018	0.047
			PFR (m ³ /s)	0.68	1.08	1.32	1.69	1.94	2.20	5.64
	FM-4	6.8	UFR (m ³ /s/ha)	0.001	0.004	0.006	0.008	0.010	0.012	0.041
			PFR (m ³ /s)	0.01	0.03	0.04	0.06	0.07	0.08	0.28
	FM-5	35.6	UFR (m ³ /s/ha)	0.004	0.008	0.011	0.014	0.017	0.020	0.052
			PFR (m ³ /s)	0.16	0.29	0.38	0.51	0.60	0.70	1.84
	FM-6	33.6	UFR (m ³ /s/ha)	0.005	0.009	0.011	0.015	0.018	0.021	0.055
			PFR (m ³ /s)	0.15	0.29	0.38	0.51	0.60	0.69	1.83
	FM-7	170.8	UFR (m ³ /s/ha)	0.006	0.010	0.013	0.016	0.019	0.021	0.053
			PFR (m ³ /s)	1.04	1.73	2.15	2.78	3.20	3.65	9.11
	FM-8	5.9	UFR (m ³ /s/ha)	0.001	0.008	0.013	0.019	0.024	0.029	0.073
			PFR (m ³ /s)	0.01	0.05	0.08	0.11	0.14	0.17	0.44
Culverts at Dundas Street	FM-D2	31.4	UFR (m ³ /s/ha)	0.007	0.011	0.013	0.017	0.020	0.022	0.054
			PFR (m ³ /s)	0.21	0.34	0.42	0.54	0.62	0.70	1.69
	FM-D3	14.4	UFR (m ³ /s/ha)	0.010	0.016	0.019	0.024	0.028	0.031	0.065
			PFR (m ³ /s)	0.15	0.23	0.28	0.35	0.40	0.44	0.93
	FM-D4	397.2	UFR (m ³ /s/ha)	0.006	0.010	0.012	0.015	0.017	0.020	0.049
			PFR (m ³ /s)	2.46	3.90	4.77	6.08	6.95	7.86	19.63
	FM-D4a ²	16.5	UFR (m ³ /s/ha)	0.013	0.020	0.024	0.030	0.035	0.039	0.073
			PFR (m ³ /s)	0.21	0.33	0.40	0.50	0.57	0.64	1.20
	FM-D5	350.5	UFR (m ³ /s/ha)	0.006	0.010	0.013	0.017	0.019	0.022	0.055
			PFR (m ³ /s)	2.07	3.54	4.48	5.85	6.80	7.80	19.31

1) UFR = Unit Flow Rate, PFR = Peak Flow Rate

2) Since UFR at culvert FM-D4A is not specified in NOCSS, the UFR based on Existing Flow from Original NOCSS Model Catchment FM-1106 is used

7.4.3 Post-Development Conditions

7.4.3.1 Development Phasing

As previously discussed in Section 7.3.2, since the development of the site and the construction of the infrastructures (e.g., SWM facilities, roads, culverts, etc.) will not occur simultaneously, progressive development phases are adopted for the proposed study. The detailed description of the development phases is summarized in **Table 7.3.1**.

7.4.3.2 Post-Development Drainage Boundaries

A preliminary grading plan for the proposed development was prepared and discussed in Section 8.0. The proposed grading was developed to ensure integration with neighbouring lands, cores, linkages and receiving watercourses through all interim development phases and under ultimate development condition. Post-development sub-catchment boundaries have been delineated based on this preliminary grading plan and are shown in **Figures 7.4.2, 7.4.3, 7.4.4 and 7.4.5** for the Phase 1A, Phase 1B, Phase 2 and Ultimate Development Condition respectively. Note that the drainage areas reflecting the proposed expansion of Regional Road No.5 (Dundas Street) are included in post-development models. A comparison of the drainage area between the existing and all proposed development conditions at EIR Flow Nodes located at Culverts at Dundas Street West is provided in **Table 7.4.4**.

Table 7.4.4 – Comparison of Existing and Post-Development Drainage Areas

To EIR Nodes	Existing Drainage Area (ha) *	Phase 1A		Phase 1B		Phase 2		Ultimate Condition	
		Drainage Area (ha)	Difference (ha) from Existing	Drainage Area (ha)	Difference (ha) from Existing	Drainage Area (ha)	Difference (ha) from Existing	Drainage Area (ha)	Difference (ha) from Existing
FM-D2	31.4	31.3	-0.1	31.3	-0.1	31.3	-0.1	40.1	8.7
FM-D3	14.4	0.0	-14.4	0.0	-14.4	0.0	-14.4	0.0	-14.4
FM-D4	397.2	413.5	16.3	413.2	16.0	420.0	22.7	410.8	13.5
FM-D4a	16.5	16.5	0.0	16.7	0.2	14.3	-2.2	14.3	-2.2
FM-D5	350.5	350.5	0.0	350.5	0.0	346.1	-4.4	346.1	-4.4
Total	810.0	811.8	1.8	811.7	1.7	811.7	1.6	811.3	1.2

* WSP updated drainage areas based on 2002 Town of Oakville topographic mapping.

As indicated in **Table 7.4.4**, the differences of the overall drainage areas between the existing conditions and all interim phases and ultimate conditions are negligible. However, note that under existing conditions, culvert FM-D3 conveys runoff from Catchment 1103 and drains across Dundas St. to a municipal ditch on the road's south side, which discharges to Fourteen Mile Creek immediately south of Culvert FM-D4. Since this ditch functions as conveyance only with no natural heritage features requiring surface flow contributions, it will be eliminated during the proposed development conditions. Under the proposed conditions (all interim phases and ultimate conditions), drainage to culvert FM-D3 together with flows from the section of the proposed Dundas Street expansion (Catchment 1502) will be redirected a proposed Pond 2 and ultimately discharge to Fourteen Mile Creek located upstream of Dundas Street and pass through culvert FM-D4. The detailed discussions on the proposed SWM plans developed for all interim phases and ultimate conditions are provided in the following sections.

7.4.3.3 Conveyance of Minor System Flows

The Subject Property will be serviced by a conventional storm sewer system designed in accordance with Town's standards. The storm sewers will be sized using a 5-year return frequency and the current Town's IDF curves. All runoff from the development areas will be conveyed to the proposed SWM facilities for the study area. The collected runoff will be treated for water quality and quantity control with extended detention for erosion control. The detailed discussions on the proposed SWM design developed for all interim phases and ultimate conditions are provided in the following sections.

7.4.3.4 Conveyance of Major Storm Flows

A design for the overland flow route has been provided throughout the FSS Study Area in order to safely convey major storm system flows in excess of the minor system up to the Regional event. Excess flows will be contained within either the roadway right-of-way or by other lands such as flow easements under the Town's control. For all classes of roads, the product of depth of water at the gutter times the velocity of flow shall not exceed 0.65 m/s. All overland flow routes will be directed to the SWM facilities. Should the major system flow exceed the conveyance capacity of any given road, the storm sewer will be sized to accommodate the excess flows such that the road capacity is not exceeded. A detailed storm sewer design is discussed in Section 8.0.

7.4.3.5 Post-Development Hydrologic Analysis

In the GAWSER hydrologic model, each catchment is divided into impervious and pervious zones where the pervious zone can be further characterized by a maximum of four different soil types. Each soil type is then divided into two zones, which represent the proportionate contribution of that soil type to sub-surface and groundwater flows. Runoff from the sub-catchments is routed using a form of the area/time versus time method. As previously indicated in Section 7.4.2, the West Branch of Fourteen Mile Creek catchments north of Dundas Street were sub-divided into several sub-catchments to reflect the revised existing conditions in the NOCSS study, as shown in **Figure 7.4.1**. This model was updated to represent the post-development catchment boundaries as shown in **Figures 7.4.2, 7.4.3, 7.4.4 and 7.4.5** for Phases 1A, 1B, 2 and Ultimate Conditions, respectively. This was accomplished by changing GAWSER input parameters representing sub-catchment areas and hydrograph parameters. The GAWSER model schematics for the proposed conditions are illustrated in **Figures 7.4.2-SCH, 7.4.3-SCH, 7.4.4-SCH and 7.4.5-SCH** for Phases 1A, 1B, 2 and Ultimate Conditions, respectively. The catchment parameters for the proposed conditions are included in **Appendix 7.2**.

The post-development drainage plans as simulated in the GAWSER model for each interim phases and ultimate development conditions are summarized as follows. Section 7.8 discuss the design of the proposed SWM facilities. The drainage areas associated with each proposed SWM facility are summarized in **Table 7.8.1**.

Phase 1A (Figure 7.4.2)

A section of Avenue Two and a section of Burnhamthorpe Road Extension will be constructed in Phase 1A. The proposed SWM Pond 2 will be constructed to control runoff from the proposed development west of Reach 14W-12 (Catchment 3090) and a section of proposed expansion of Dundas Street West (Catchment

1502) for the storms up to Regional event. Note that flows from Catchment 3000 will also be diverted to Pond 2.

Phase 1B (Figure 7.4.3)

In addition to the development plan of Phase 1A, re-alignment of Reach 14W-21 and 1Reach 4W-22 will occur during Phase 1B. A section of Avenue Three will also be completed. The proposed SWM Pond 3 will be structured to provide controls for the development east of Reach 14W-22 and Reach 14W-12 (Catchments 3200 and 3201). Flows from Catchment 3100 located north of Burnhamthorpe Road Extension will also be diverted to Pond 3. The proposed SWM Pond 3 was design to provide controls for storms up to Regional event.

In order to allow a uniform and sustained level of flow to be maintained in the Reach 14W-12A channel, flows from rooftops of the proposed buildings (2.56ha, Catchment 2309) together with runoff from the existing undeveloped Catchments 2399 (7.68 ha) and local drainage Catchment 4011 will bypass the proposed SWM Pond 3 and be diverted to Reach 14W-12A. Section 7.6 discusses hydrologic flow regime analysis in details. Note that, it has been widely recognized that the runoffs from the building rooftop are considered as clean; therefore, oil-grit separators (OGS) are not required to provide treatment for such areas. Since the application of such strategy requires the detailed locations of the building rooftops, for the conceptual design purposes, it is assumed that the roof drains will be installed at rooftops of the proposed buildings to provide a controlled unit flow rate of 41 L/s/ha at a maximum water depth of 0.15 m on the rooftops. Information on the conceptual design of rooftop controls are included in the **Appendix 7.6**. The results of the proposed roof controls, including the roof release rates, water ponding depths, utilized ponding storages for the various storm events from 2- to 100- year and Regional storm, are included in the **Appendix 7.6**. It confirms that it is feasible to implement the required rooftop controls for the subject developments. Detailed design of roof drains and STM connections will be provided during detailed engineering design stage.

Phase 2 (Figure 7.4.4)

The entire lands of Subject Property including Block C3-4 (Azevedo Property) will be developed during Phase 2. Reach 14W-23 will be re-aligned and a section of Avenue One will be completed. The proposed SWM Pond 3 will capture and control flows from the entire development east of Reach 14W-22 and Reach 14W-12 (Catchment 3100 and 3200). Again, the proposed SWM Pond 3 was designed to provide controls for storms up to Regional event.

Note that flows from rooftops of the proposed buildings with a total area of 5.12 ha (Catchment 2309) will be diverted to Reach 14W-12A directly to allow a uniform and sustained level of flow to be maintained in the subject receiving reach. As described previously, it is assumed that the roof drains will be installed at rooftops of the proposed buildings to provide a controlled unit flow rate of 41 L/s/ha at a maximum water depth of 0.15 m on the rooftops. **Section 7.6** discusses hydrologic flow regime analysis in details. Again, because the strategy requires the site plan to indicate the final proposed locations of the building rooftops, detailed design on roof drains and STM connections will be provided during detailed engineering design stage. The results of the proposed roof controls, including the roof release rates, water ponding depths, utilized ponding storages for the various storm events from 2- to 100- year and Regional storm, are included in the **Appendix 7.6**. It confirms that it is feasible to implement the required rooftop controls for the subject developments.

Ultimate Development Conditions (Figure 7.4.5)

Under ultimate development conditions, the lands located west of the Subject Property will be developed (Catchments 3000, 3060, 3050). All associated roads will be completed. These developments will be controlled by additional three SWM Facilities, including Pond 1, Pond 5 and Tremaine Pond (as per Tremaine and Dundas Secondary Plan Subwatershed Study, 2009). Similar to Phase 2, flows from rooftops of the proposed buildings with a total area of 5.12 ha (Catchment 2309) will be diverted to Reach 14W-12A directly to allow a uniform and sustained level of flow to be maintained in the subject receiving reach. Note that it is assumed that the roof drains will be installed at rooftops of the proposed buildings to provide a controlled unit flow rate of 41 L/s/ha at a maximum water depth of 0.15 m on the rooftops. The drainage from the area west of Tremaine Road (Subcatchment 3300) may be diverted to Culvert FM-D1 if it is supported by the detailed analysis in conjunction with the development on the west side of Tremaine Road. However, in the current study, the flows from such Subcatchment 3300 are assumed to continue draining to the east side of Tremaine and discharge to Culvert FM-D2.

Similarly, information on the conceptual design of rooftop controls are included in the **Appendix 7.6**. The results of the proposed roof controls, including the roof release rates, water ponding depths, utilized ponding storages for the various storm events from 2- to 100- year and Regional storm, are included in the **Appendix 7.6**. It confirms that it is feasible to implement the required rooftop controls for the subject developments. Detailed design of roof drains and STM connections will be provided during detailed engineering design stage.

7.4.3.6 Results of GAWSER model for Post-Development Conditions

All interim development phases and ultimate development conditions are simulated by using GAWSER hydrological model by incorporating:

- Water quantity controls by the proposed SWM facilities together with the identified rooftop storage for the proposed buildings (during Phase 1B, 2 and Ultimate) for events up to Regional storm through design of the proposed SWM facilities (Section 7.8);
- Erosion controls by providing adequate erosion storage with orifice controls of the proposed SWM facilities through an erosion control analysis (Section 7.5); and,
- Division of clean runoff from proposed building rooftops to allow a uniform and sustained level of flow to be maintained in the Reach 14W-12A channel through a hydrologic flow regime analysis (Section 7.6).

For illustration purposes, **Table 7.4.5** provides a comparison of resulting Regional flows between the existing conditions and post-development conditions, including interim phases and ultimate conditions at their respective EIR Flow Nodes along Dundas Street and all Reference Flow Nodes within the study area. The post-development flows for all other storm events (including 2- to 100- year and Region storm) are included in a comprehensive flow table (**Table APP-7.2**) in the **Appendix 7.2**.

Table 7.4.5 – Comparison of Existing and Post-Development Peak Flow Rates for Regional Events

Flow Node Name		Peak Flow Rates for Regional Event (m ³ /s)				
		Existing Condition	Phase 1A	Phase 1B	Phase 2	Ultimate Condition
EIR Flow Nodes	FM-D2	1.73	1.73	1.73	1.73	1.40
	FM-D3	0.93	-	-	-	-
	FM-D4	19.34	19.14	18.49	18.94	17.76
	FM-D4A	1.23	1.23	1.25	1.07	1.07
	FM-D5	19.71	19.71	19.71	19.33	19.33
Reference Flow Nodes	1	10.29	10.21	10.06	10.04	9.44
	1A	10.20	10.10	10.00	9.98	9.39
	1B	8.22	8.22	8.11	8.10	8.02
	2	8.23	8.23	0.86	0.22	0.22
	2C	7.88	7.88	-	-	-
	2B	-	-	6.68	6.67	6.50
	2A	-	-	7.29	6.89	6.72
	3A	18.45	18.36	17.29	16.83	16.11
	3B	19.22	18.70	18.04	18.36	17.34
	3	19.22	19.00	18.34	18.83	17.64
	4	20.34	19.14	18.49	18.94	17.76
	5	3.79	3.79	3.79	3.38	3.38
	6	-	-	-	6.52	6.35
	7	-	-	-	7.95	7.87
	8	-	-	-	0.52	0.52
	9	3.06	3.06	3.06	2.69	2.69

As indicated in **Table 7.4.5** and **Table APP-7.2** in the **Appendix 7.2**, with proposed SWM facilities, the peak flows for all post-development conditions will be controlled to those under the existing conditions for all design storms including 2 to 100-year and Regional events. Consequently, the investigation of the potential increases to flood risk for the entire downstream watercourse to its outlet at Lake Ontario is not required by NOCSS. However, we understand that the quantity controls of upstream reaches would delay the peak flows to correspond more closely with the timing of the peak flow in the main branch; thereby, causing the potential increases in peak flows in the receiving watercourse downstream. As a prudent measure, a hydrological analysis for the entire Fourteen Mile Creek subwatershed was carried out to determine the flows at downstream locations and ensure there would be no impact due to the development at subject lands located at upstream of the subwatershed during Regional storm conditions. The Regional control downstream impact is discussed in Section 7.9.

7.5 Erosion Control Analysis

As noted in the North Oakville EIR and FSS ToR, erosion thresholds are meant to be integrated into a SWM system design in such a manner that existing channel erosion or aggradation is not exacerbated. The recommended method for assessing pre- and post-development erosion potential is to perform erosion threshold modelling. Erosion threshold represents the point at which sustained flows will tend to entrain and transport sediments. The critical shear stress represents the erosion threshold value, which when exceeded, creates erosive tendencies in the channel bed and banks.

The 14 Mile Creek Tributaries Fluvial Geomorphological and Erosion Threshold Assessment report completed by Water's Edge on October 21, 2013 (included in **Appendix 7.1**) assessed the Fourteen Mile Creek tributaries and summarized critical shear stresses and critical flows for relevant reaches. The summarized values in the report were used to establish erosion control in previous WSP EIR/FSS submissions. As part of the erosion control analysis, critical flow values of 0.25 m³/s for Reach C and 1.48 m³/s for Reach B were used. Both values generated exceedances that were not acceptable to CH. CH required that the differences in erosion exceedances (duration and frequency) between existing conditions and proposed conditions (including all interim conditions and ultimate development conditions) are within a 5% limit.

Following a peer review memo by Ecosystem Recovery, Water's Edge was requested to clarify and confirm their erosion threshold recommendations, which were deemed inconsistent and unclear by the peer reviewer. As a result, Water's Edge prepared a memo to WPS dated March 10, 2016 (included in **Appendix 7.1**), recommending to run the erosion control analysis based on Reach D (critical flow = 0.96 m³/s), which was determined to be the most sensitive reach based on the findings of both Ecosystem Recovery and Water's Edge. Another memo was issued by Water's Edge (May 3, 2017) to address comments from CH and the Town, concerning the selection of critical flows, reaches, and erosion threshold calculations (included in **Appendix 7.1**).

To perform the erosion control analysis, a continuous simulation was performed by using the GAWSER hydrologic model. Continuous simulation models can determine the potential cumulative impacts of multiple SWM facilities on the upstream and downstream development areas and calculate erosion indices by examining the historical rainfall records and the flow rates in the watercourse. For the subject study, the meteorological input data (hourly rainfall series, temperature, and wind speed) for the GAWSER model was obtained from the Hamilton RBG gauge station (#615330) with a 30-year of records from 1962 to 1992.

Erosion exceedance analysis was conducted using a critical flow of 0.96 m³/s for Reach D upstream of Dundas Street (Reference Node 3). The modelling process adopts a trial and error approach which includes a considerable number of attempts by adjusting outlet structures of the proposed SWM facilities (e.g., sizes and inverts of the orifice plates) to achieve the computational target (i.e., 5% of exceedance difference).

A summary of the erosion exceedance analysis results is shown in **Table 7.5.1**. The determined detention times required to release the 25 mm erosion control storages for each proposed SWM facility for all interim phases and ultimate conditions are summarized in **Table 7.5.2**. All other related information is included in **Appendix 7.3**.

Table 7.5.1 – Results of Erosion Control Analysis for Reach D Upstream of Dundas Street (Reference Node 3)

Development Conditions	Drainage Area (ha)	Hours		Pulses	
		Exceedance	Difference	Exceedance	Difference
Existing	395.4	719	N/A	99	N/A
Phase 1A	400.3	755	5.01%	102	3.03%
Phase 1B	400.0	731	1.67%	97	-2.02%
Phase 2	406.7	752	4.59%	97	-2.02%
Ultimate Condition	406.7	754	4.87%	94	-5.05%

Table 7.5.2 – SWM Ponds Detention Times for Erosion Controls

Pond #	Detention Time for Erosion Control (Hr)	
	Phase 1A, Phase 1B and Phase 2	Ultimate Conditions
Pond 2	47.3	47.3
Pond 3	53.4	41.3
Pond 5	N/A	46.9
Pond 1	N/A	42.4

As shown in **Table 7.5.1**, by utilizing the SWM facilities during all interim phases and under ultimate development conditions, all the duration and frequency exceedances are within the 5% margin specified in the EIR/FSS ToR. Therefore, it can be concluded that no further erosion analysis is needed. Impacts of these changes on the pre-development flow regime, including aquatic habitat and sediment transport are discussed in Section 7.6 of Hydrologic Flow Regimes Analysis.

Note that, to achieve the specified exceedances limits for all interim phase and ultimate development conditions, an adjustment of the outlet structure to reduce the detention time of Pond 3 will be required under ultimate development conditions (**Table 7.5.2**). Section 7.8 will discuss the design of the SWM facilities in detail.

7.6 Hydrologic Flow Regimes Analysis

The Hydrologic Flow Regime Analysis is included in **Appendix 7.4 – Hydrologic Flow Regimes Analysis Calculations**. The analysis discusses impacts on streamflow regime during three typical years: Dry Year, Wet Year, and Average Year. Monthly flows have been compared based on magnitude, duration and frequency. For impacts on Reach 14W-12A, Reach 14W-22, and Reach 14-W23, the results show the following key findings:

- For Reach 14W-12A, impact on streamflow regime is inevitable due to the proposed development. To allow a uniform and sustained level of flow to be maintained in the Reach 14W-12A channel, measurements are proposed to provide required surface runoff compensation to the Reach. It includes diverting flows from rooftops of the proposed buildings and/or from the existing undeveloped catchments. **Table 7.6.1** provides a summary of the required surface runoff compensation measurements to the Reach 14W-12A during all development phases.

Table 7.6.1 – Summary of Surface Water Compensation to 14W-12A

Phase	Runoff Compensation Measures								
	Flow from Local NHS Area			Flow from Rooftop of Proposed Building			Flow from Pre-Development Surface Drainage Area (Open Space)		
	Area ID	Area (ha)	Imp (%)	Area ID	Area (ha)	Imp (%)	Area ID	Area (ha)	Imp (%)
1a	Not Req.								
1b	4011	0.57	2	2309	2.56	100	2399	7.68	5
2	4011	0.57	2	2309	5.12	100	Not Req.		
Ultimate	4011	0.57	2	2309	5.12	100	Not Req.		

The Flow Regimes Analysis indicates that with the proposed measurements, the peak flows under all development phases are capable of filling the wetted perimeter and sustaining continuity downstream to Reach 14W-12. This has been presented via cross sections from the HEC RAS model, to confirm hydraulic conditions. The channel is not sustained by sediment entrainment flows and geomorphic functions as noted in **Appendix 7.1** (i.e., field investigation by WSP staff);

- For Reach 14W-22, there is no impact to duration and frequency of monthly flows. Magnitude of peak flows on a monthly basis are expected to decrease by 15 to 20%, which is a minor impact, due to a reduced drainage area. The peak flows under all development phases are capable of filling the wetted perimeter and sustaining continuity downstream to Reach 14W-12. This has been presented via cross sections from the HEC RAS model, to confirm hydraulic conditions. Sediment entrainment flows (10% exceedance) are maintained; and,
- For Reach 14W-23, there is no impact to duration and frequency of monthly flows. Magnitude of peak flows on monthly basis are expected to decrease by 15 to 20%, which is a minor impact due to a reduced drainage area. The peak flows under all development phases are capable of filling the wetted perimeter and sustaining continuity downstream. This has been presented via cross sections from the HEC RAS model, to confirm hydraulic conditions. Sediment entrainment flows (10% exceedance) are maintained

7.7 Topographic Depression Volumes

The NOCSS document recommends that surface storage volumes in area topographic depressions be identified and comparisons be made to SWM facility storage. Further clarification was provided as part of a mediation agreement, provided Mediation Item: Depressional Storage (May 30, 2007). As outlined in this agreement the principle intent of evaluating the existing depression storage was to ensure that the natural depression storage is maintained in the SWM system. Artificially (man-made) created storage was to be excluded from this evaluation. As also described in this agreement, to ensure that the storage volume of the depression areas is maintained, the calculated depression volume was to be compared to the proposed SWM facility volume within the same drainage area. If the depressional volume is determined to be less than or equal to the SWM facility volume then no additional analysis or change to SWM facility design would be required. The depressions identified in the NOCSS study within the subject development property are shown in **Figure 7.7.1** and listed in **Table 7.7.1**.

Table 7.7.1 – Topographic Depressions in the Subject Property Development Area

ID	Drainage Direction	NOCSS Classification	Proposed Development (Ultimate)
1	Fourteen Mile Creek West: Culvert FM-D4	Topographic Depression	Replaced by Employment Development Block
2	Fourteen Mile Creek West: Culvert FM-D4	Topographic Depression	Replaced by Employment Development Block
3	Fourteen Mile Creek West: Culvert FM-D4	Topographic Depression	Replaced by Employment Development Block
4	Fourteen Mile Creek West: Culvert FM-D4	Hydrologic Feature 'A'	Replaced by Employment Development Block Realignment of Reach 14W-14
5	Fourteen Mile Creek West: Culvert FM-D4	Hydrologic Feature 'A'	Replaced by Proposed SWM Pond 3
6	Fourteen Mile Creek West: Culvert FM-D4	Hydrologic Feature 'A'	Remain Existing Reach 14W-16
7	Fourteen Mile Creek West: Culvert FM-D4	Hydrologic Feature 'B'	Remain Existing Reach 14W-16
8	Fourteen Mile Creek West: Culvert FM-D4	Hydrologic Feature 'B'	Replaced by Proposed SWM Pond 5
9	Fourteen Mile Creek West: Culvert FM-D3	Hydrologic Feature 'B'	Replaced by Employment Development Block

A detailed topographic survey of the development area was carried out by WSP (formally MMM) in 2011 including topographic depressions. As indicated in **Table 7.7.1**, depression storage analysis will not be carried out for Depressions 6 and 7 as they will be kept its original condition in the post-development conditions. Depression storage analysis will not also be carried out for Depressions 5 and 8 as they will be replaced by proposed SWM facilities. In addition, Depression 1 is located in relatively flat area and could not be identified during the topographic survey. Therefore, topographic depressional storage analysis was carried out for Depressions 2, 3, 4 and 9 within the Subject Property development area. The topographic depressional storages were estimated using Civil3D software and are presented in **Table 7.7.2**.

Table 7.7.2 – Depressional Storage Calculations

ID	Calculated Dep. Storage (m ³)	Within Drainage Area of Pond No.	Available Storage in SWM Facilities
2	18	SWM Pond 3	Permanent Pool Volume = 28,690 m ³
3	5		Active Storage (100-Year) - Simulated/Modelled = 24,381 m ³
4	21		Active Storage (Regional) - Simulated/Modelled = 64,583 m ³
9	120	SWM Pond 2	Permanent Pool Volume = 10,431 m ³ Active Storage (100-Year) = 13,981 m ³ Active Storage (Regional Control) = 40,505 m ³

The total volume of Depression 2, 3, 4 and 9 is significantly less than the proposed SWM facility volume as presented in **Table 7.7.2**. Therefore, no additional analysis or change to the SWM facility design is required.

7.8 Design of Stormwater Management Facilities

7.8.1 General

The hydrological models developed for the subject study area include a total of five (5) SWM facilities (wet ponds) under ultimate development conditions. As shown in **Figure 7.4.5**, the locations of the wet ponds have been sited based on the principle of maintaining existing drainage patterns and flow regimes in the undisturbed channel reaches to the extent practicable. As previously indicated in Section 7.3.2 and shown in **Table 7.3.1**, the proposed SWM facilities will be constructed progressively in accordance with the development phases. Section 7.4.3.5 discusses the modelling approach for these proposed SWM facilities during all interim phase and ultimate development conditions. The drainage areas for the proposed SWM wet ponds in the subject study is provided in **Table 7.8.1**. A brief summary of each proposed SWM wet pond is provided as follows:

- **Pond 2:** It will be constructed during Phase 1A and located on the south-west side of Reach 14W-12. It will provide all required controls for the developments within the Subject Property and a section of the Dundas Street expansion area (refer to Section 7.8.2 for details). During ultimate development conditions, Pond 2 will also provide controls for an area located west of the Subject Property and south of the proposed Pond 1;
- **Pond 3:** It will be constructed from Phase 1B and located on the east side of Reach 14W-12. It will replace the existing depression #5 Hydrologic Feature 'A' as shown in **Figure 7.7.1**. Pond 3 will provide all required controls for the developments within the Subject Property. During ultimate development conditions, Pond 3 will also provide controls for an area immediately outside of the property boundary, located west of Reach 14W-22 and south of Highway 407 (Catchment 3080). Note that, during interim phases and ultimate development conditions, flows from certain pond contributing areas (e.g., rooftops of the proposed buildings and/or undeveloped existing areas) will be diverted to bypass the Pond 3 and discharge directly to Reach 14W-12A to allow a uniform and sustained level of flow to be maintained in the receiving reach;
- **Pond 5:** It will be constructed under ultimate conditions. Pond 5 will provide controls for the development immediately outside of the Subject Property boundary located west of Reach 14W-16 and east of Tremaine Road, south of Highway 407. A portion of Pond 5 Block will also replace an existing depression #8 Hydrologic Feature 'B' as shown in **Figure 7.7.1**;
- **Pond 1:** It will be constructed under ultimate conditions. Pond 1 will provide controls for the development immediately outside of the Subject Property boundary located between the proposed Avenue Two and Tremaine Road north of Dundas Street. Note that an area located south of the Pond 1 where flows cannot be captured by the Pond 1 will drain to proposed Pond 2 for the required treatments; and,

Tremaine Pond: It will be constructed during ultimate development conditions. The pond will be located west of Tremaine Road and south of Highway 407. It is one of the three SWM facilities recommended in the Tremaine and Dundas Secondary Plan Subwatershed Study (2009) for the West Branch of Fourteen Mile Creek drainage areas west of Tremaine Road and south of Highway 407. For the modelling purposes, Tremaine Pond is included in the GAWSER model to properly

determine the post-development flows along the downstream watercourses under ultimate development conditions. Since the Tremaine Pond will be located within the Tremaine and Dundas Secondary Plan Subwatershed Study (2009) area, the design of such pond will be carried out in the future study and is not considered in this study.

Table 7.8.1 – Drainage Areas for the Proposed SWM Ponds

Pond #	Phase 1A		Phase 1B		Phase 2		Ultimate	
	Drainage Area (ha)	Imperviousness (%)	Drainage Area (ha)	Imperviousness (%)	Drainage Area (ha)	Imperviousness (%)	Drainage Area (ha)	Imperviousness (%)
Pond 2	17.8	89	17.8	89	17.8	89	20.8	89
Pond 3	-	-	24.6	50	37.0	90	39.9	90
Pond 1	-	-	-	-	-	-	23.6	88
Pond 5	-	-	-	-	-	-	14.4	88

This section of the report documents the outline design work completed for SWM Ponds 2, 3, 5 and 1, which includes preliminary grading proposals, design of the outlet control structures, and hydrologic modelling to verify compliance with the target discharge rates at all downstream EIS flow nodes and reference flow nodes. Preliminary calculations have been undertaken to confirm that the allowances of the blocks of these proposed SWM facilities are adequate. All the proposed facilities will be designed to provide stormwater quality (enhanced level), erosion and quantity control (up to Regional event) in accordance with the criteria referred to in the NOCSS.

7.8.2 Dundas Street Expansion

As previously discussed in Section 7.4, the proposed expansion of Regional Road No.5 (Dundas Street) were included in the post-development model for the subject study. All related data required for the modelling purposes regarding the design of the proposed expansion are obtained from the Environmental Study Report (ESR) - Dundas Street Class EA Study Brant Street to Bronte Road, MMM Group, May 2015. On-site controls will be provided for the most sections of the proposed Dundas Street expansion, except a section of the area (Catchment 1502), which will drain and be treated by the proposed Pond 2 located within the Subject Property. All related information regarding the Dundas Street Expansion catchments, including the catchment parameters, modelling results and the relevant sections/drawings of the ESR report are included in the **Appendix 7.5**.

Note that the existing culvert FM-D2 (C21A) at Dundas Street is an 825 mm diameter CSP culvert. As indicated in ESR (MMM, May 2015), during the Regional Storm event, Dundas Street will be overtopped. Therefore, the existing culvert FM-D2 (C21A) will be replaced by a new 1050 mm diameter concrete culvert to convey the Regional Storm flow.

7.8.3 Pond Design Overview and Control Criteria

The SWM facilities were designed as 'wet ponds' in accordance with the Section 4.6.2 of the 2003 Ontario Ministry of the Environment (MOE) Stormwater Management Planning and Design Manual (SWMPDM). The specific SWM facilities requirements provided in the Town of Oakville Development Engineering Procedures

& Guidelines Manual were also adhered to. Major and minor stormwater flows will be conveyed to the pond facilities by the proposed storm sewer and overland flow system. Section 8.0 discusses the sewer design in detail. All required facility functions are provided in a single depression with an internal barrier berm to separate the facility into a forebay and a main treatment cell. Major and minor flow from the drainage area will enter the forebay for gross particulate settling and erosive energy dissipation. Flow then enters the main cell where a combination of treatment actions, similar to plug flow and continuous flow reactor dynamics allow for water quality treatment of raw stormwater flows. Treated flow exits the facility at the downstream end through the outlet control structure. The outlet structure consists of a flow control MH configured with an orifice-controlled reverse slope pipe draining flow from the deeper pool areas of the main cell, a flow weir and an orifice-controlled outflow pipe draining to the downstream. An emergency spillway provides capability for each SWM ponds to pass the uncontrolled Regional flows if the outlet structure is blocked, or otherwise non-functional. Further details on the design targets for the outlet control structures are set out in the following sections.

The identified control criteria of the pond design are summarized as follows:

- **Water Quality** (Section 7.8.4): The proposed SWM facilities will be designed to meet MOECC's Enhanced Level of water quality protection (Level 1) for water quality control, phosphorus control and fisheries protection;
- **Erosion Control** (Section 7.8.5): In accordance with the results from the detailed erosion threshold analyses, the proposed SWM facilities will be designed to provide erosion controls extended detention so that the existing channel erosion or aggradation is not exacerbated by development;
- **Water Quantity** (Section 7.8.6): The proposed SWM facilities will be designed to provide water quantity (flood) controls for the 2-year to 100-year return period events and the Regional storm;
- **Water Balance:** No specific SWM facility functions are required at this design stage to address site-wide water balance issues. Please see Section 4.0 for specific information on how water balance issues are addressed.

A summary of the key SWM facilities design features, alongside the minimum requirements set out in the MOECC and Town's guidance documents, is provided at the end of Section 7.8.8 in **Table 7.8.9**. The following sections discuss the designs of the proposed SWM facilities in details.

7.8.4 Water Quality Control

Table 3.2 of the MOECC's SWM Planning & Design Manual (2003) sets out the volumetric storage requirements for various types of SWM facility to achieve different levels of water quality protection. The proposed SWM facilities in the subject study are targeting an 'Enhanced Level' of water quality treatment (long-term removal of 80% of total suspended solids), in line with MOE recommendations. **Table 7.8.2** below summarises the calculations undertaken to determine the required permanent pool volumes in each pond, and demonstrates that the volumes provided in the preliminary grading design exceed these requirements. Note that all facilities for water quality controls are designed based on ultimate development conditions.

Table 7.8.2 – Summary of SWM Pond Permanent Pool Requirements under Ultimate Development Conditions

Element	Pond 2	Pond 3	Pond 1	Pond 5
Drainage Area (ha) ¹	20.8	39.9	23.6	14.4
Imperviousness (%)	89	90	88	88
Storage Criteria for Enhanced Level Treatment (m ³ /ha) ²	257	258	255	255
Storage Required for Enhanced Level Treatment (m ³)	5,331	10,295	6,005	3,672
Permanent Pool Storage Required (m ³)	4,501	8,701	5,063	3,096
Water Quality Extended Detention (m ³) ³	830	1,594	942	576
Permanent Pool Volume Provided (m ³)	9,497	23,678	14,564	3,679

1) Drainage areas for each SWM ponds are based on the identified proposed development areas and exclude the rooftop areas which will bypass the SWM pond and drain to the receiving water courses directly.

2) Based on interpolation of storage volumes from MOECC Table 3.2 – includes permanent pool volume and 40 m³/ha allowance for extended detention.

3) Based on 40 m³/ha as per MOECC Table 3.2 – typically exceeded by Erosion Control Extended Detention requirements.

It is confirmed that the permanent pool volumes provided by all proposed SWM facilities exceed the minimum requirements for 'Enhanced Level' treatment calculated in accordance with the MOECC SWM Planning & Design Manual. All detailed SWM facility calculations are included in **Appendix 7.6**.

An operations and maintenance manual will be provided at the detailed design stage for those who will assume responsibility for maintaining the facilities. It will cover the appropriate monitoring and maintenance activities that should be followed to keep the facility in good working order. It will also cover the requirements for removal of accumulated sediment/TSS at the base of the pond. A preliminary monitoring program has been outlined in **Appendix 7.8**.

Note that to allow a uniform and sustained level of flow to be maintained in the Reach 14W-12A channel, starting Phase 1B through ultimate development conditions, flows from rooftops of the proposed buildings and/or runoff from the existing pre-developed areas will bypass the proposed SWM Pond 3 and be diverted to Reach 14W-12A. Since it has been widely recognized that the runoff from the building rooftops is considered as clean, additional water quality treatment (e.g., oil-grit separator) is not required for such areas.

According to the North Oakville Subwatershed Management Strategy (July 12, 2007), mediation item on total phosphorus, a “no net increase in the total phosphorus” can be ensured using SWM facilities providing enhanced level of control. Therefore, if ponds are designed with enhanced level of control, there will be no requirement to analyze total phosphorus in subsequent studies.

According to the North Oakville Subwatershed Management Strategy (July 12, 2007), mediation item on DO, a target of 6 mg/L was set, which is the provincial standard. Excerpts from the mediation item indicate: *“With respect to DO, Enhanced Level requirements for SWM facility design do not specifically address DO performance levels. Therefore, DO targets and monitoring are for the purposes of collecting data on the functioning of these facilities from a DO perspective. It will be analyzed to determine their function with respect to DO, but results will not impact pond assumption.”*

7.8.5 Extended Detention Erosion Control

CH required that the differences in erosion exceedances (duration and frequency) between existing conditions and proposed conditions (including all interim conditions and ultimate development conditions) are within a 5% limit, so that the existing channel erosion or aggradation is not exacerbated by development. To meet such criteria, an erosion control analysis was undertaken for the receiving watercourses (refer to Section 7.5 for details). Furthermore, based on discussion with the Town staff, the drawdown times for the 25 mm storm event in each SWM facility are required to be between 24 to 48 hrs. This requirement dictated the size of the orifice plates that control discharge from the reverse-slope pipes. The required sizes were calculated using the falling head orifice equation – and the results are summarised below in Table 7.8.3. Full calculations are provided in Appendix 7.6.

Table 7.8.3 –Summary of SWM Pond Erosion Control Design

Element	Pond 2	Pond 3		Pond 1	Pond 5
Required Extended Detention Storage for Erosion Control (m ³) ¹	4,162	7,994		4,639	2,837
Provided Extended Detention Storage for Erosion Control (m ³)	7,155	12,381		7,552	4,926
Permanent Pool Elevation (m)	146.69	146.20		152.00	153.00
Extended Detention Elevation (m)	147.64	147.00		153.00	154.20
Max. Extended Detention Depth (m)	0.95	0.80		1.00	1.20
Development Phases	Ultimate Conditions	Phase 1A, Phase 1B and Phase 2	Ultimate Conditions	Ultimate Conditions	Ultimate Conditions
Orifice Diameter (mm)	200	255	290	215	155
Peak Outflow Rate (m ³ /s)	0.078	0.117	0.149	0.096	0.054
Drawdown Time (hours)	47.3	53.4	41.3	42.4	46.9

1) Calculated runoff volume during a 25 mm rainfall event provided by hydrologic modelling.

As shown in above **Table 7.8.3**, with properly sized orifice plates, all proposed SWM facilities will be able to retain adequate erosion control volumes and release them for 24 to 48 hours during the ultimate development conditions. Note that in order to maintain the differences in erosion exceedances between existing conditions and proposed conditions (including all interim conditions and ultimate development conditions) within a 5% limit (**Table 7.5.1**), the orifice plate for Pond 3 needs to be adjusted under ultimate development conditions

(change orifice plate from 255 mm diameter to 290 mm diameter). Furthermore, as previously mentioned in Section 7.5, the modelling process adopts trial and error approach. Best efforts have been made during the process to achieve the target of the 5% of exceedance difference. The final results of the erosion analysis modeling show that the drawdown time for Pond 3 during interim conditions (i.e., Phases 1A, 1B and 2) has to be relatively longer than 48 hours (i.e., 53.4 hours) in order to meet the 5% limits. Such increase is acceptable, since it is insignificant and will only occur during interim phases.

Note that the drawdown times for the 25 mm storm event in each facility were calculated using the falling head orifice equation and based on Equation 4.10 in SWM Planning and Design Manual (MOECC, March 2003) for erosion control purposes. During less frequent events (e.g., 5 to 100-year events), excess water will flow through high flow control structures (e.g., weir and spillway as later described in Section 7.8.6). The times required to drain the water for each facility through high flow control structures were obtained from the outflow hydrographs as the outputs from the GAWSER model. **Appendix 7.6** includes summary tables of the calculated total drawdown times (including both flows through low flow orifice for erosion control and high flow weir and spillway for quantity controls) related to each design storm for SWM Ponds 2 and 3.

7.8.6 SWM Forebay Design

A sediment forebay is required at inlet of each SWM ponds to settle out most of the sediment load within an area that can be accessed for maintenance. According to MOECC's design manual, the forebay length should be greater than or equal to the larger of the settling and dispersion lengths. **Table 7.8.4** provides a summary of the minimum forebay dimensions for each SWM ponds.

Table 7.8.4 – Summary of SWM Pond Forebay Design

Element		Pond 2	Pond 3	Pond 1	Pond 5
Sediment Forebay	Minimum Forebay Length (m)	21.56	22.58	22.54	15.36
	Minimum Bottom Width (m)	2.70	2.82	2.82	1.92
	Maximum Forebay Surface Area	33% of Total Pond Area	33% of Total Pond Area	33% of Total Pond Area	33% of Total Pond Area

The location of the forebay for each pond is shown in the drawings of the servicing plans included in the Section 8.0. Detailed forebay calculations are included in **Appendix 7.6**.

7.8.7 Water Quantity Control

All proposed SWM facilities were design to provide quantity/flood control for storm events up to the Regional storm. For design purposes, the required quantity control storages for all facilities were determined based on the ultimate development conditions and were evaluated and confirmed through the model simulations for all interim conditions. The target discharge rates for each SWM facility was established based on the existing flows at the relevant locations during ultimate development conditions. Once the designs were complete for the facility grading, the stage-storage-discharge relationships (facility rating curves) were determined based on the designed flow control structures. The rating curves were then tested in the hydrologic model to verify compliance with the target flow rates as established in Section 7.4.2. It was achieved by comparing the post-development flows for all interim phases and ultimate development conditions with the defined target flows at at the respective EIR Flow Nodes along Dundas Street and all Reference Flow Nodes within the study area.

The control structure for each of the facilities includes three elements which are described as follows:

- 1) **Low Flow Control (Erosion Control):** a reversed pipe connecting the bottom of the outlet to a control manhole (integrated with flow weirs) where an orifice plate installed at the invert of the reversed pipe (upstream side of control manhole) was sized to meet erosion control criteria;
- 2) **High Flow Control (Flood Control up to Regional Storm Event):** a control manhole integrated with one (1) or two (2) flow weirs. Such control manhole is installed at the extended detention (erosion control) elevation. A properly sized pipe connecting the control manhole to the downstream is required for water quantity controls and to ensure the flow can route through the facility during major storm events up to Regional storm event. A second orifice plate is installed at the pipe invert located at the downstream side of the control manhole; and,
- 3) **Emergency Spill:** The emergency spillway is constructed 0.10 m above the designed Regional flow elevations to guard against the possibility that the outlet structures are blocked, or otherwise non-functional. As per identified design criteria, a freeboard allowance of a minimum 0.30 m above the designed Regional flow elevations is also provided.

Tailwater Assessment

Due to the site constraints, some proposed SWM Ponds will have submerged outlet conditions, which will potentially impact the pond design and result in reduced water heads on the outlets (i.e., parameter h in the orifice equation and weir equation). To properly evaluate the potential outlet blockages associated with the ponds, tailwater conditions have been assessed and incorporated in the required pond design for the proposed development. Based on the assessment, water heads of Pond 3 were reduced to reflect the submerged outlet conditions during the Regional storm event. This was achieved by giving a higher invert elevation (to reflect the reduced water head values) of the orifice at outlet control MH in the pond design.

Details of the outlet control structures are summarised below in **Table 7.8.5**, and full calculations for the facility design including stage-storage-discharge relationship are included in **Appendix 7.6**. Details on a conceptual design of the outlet structure, including locations, dimensions, pond cross sections, etc., are

provided in the drawings included in Section 8.0. Note that the current study provides preliminary designs for each proposed SWM facility servicing the developments in the subject study areas as per the EIR/FSS ToR. Detailed SWM designs will be required during consequent development application process.

Table 7.8.5 – Summary of SWM Pond Outlet Control Structures

Element		Pond 2	Pond 3	Pond 1	Pond 5
Low Flow Control	Reversed Pipe Orifice Invert - Permanent Pool Elevation (masl)	146.69	146.20 (at Permanent Pool); 146.39 (for Tailwater Analysis)	152.00	153.00
	Reversed Pipe Orifice Diameter (mm)	200	290 (Ultimate) 255 (All Interim)	215	155
Flow Control MH (Flood Control up to Regional Event)	Weir Invert - Erosion Control Extended Detention - (masl)	147.64	147.00	153.00	154.20
	Weir Width/Opening (m)	2.00	2.00	2.00	2.00
	Invert of a Pipe Connecting the Control MH to D/S	146.69	146.20 (at Permanent Pool); 146.39 (for Tailwater Analysis)	152.00	153.00
	D/S Pipe Orifice Diameter (mm)	270	400	350	240
	High Flow Weir (2nd Weir) - Width/Opening (m)	Not required	Not required	Not required	0.80
	High Flow Weir (2nd Weir) - Invert (masl)	Not required	Not required	Not required	157.10
Emergency Spill (to pass uncontrolled Regional Flows)	Spillway Invert - (masl)	150.84	149.88	156.10	157.36
	Spillway Width (m)	15.00	20.00	15.00	10.00
Freeboard	Top of Regional Storm Storage Elevation (masl) - Provided	150.74	149.78	156.00	157.26
	Top of Regional Storm Storage Elevation (masl) - Simulated/Modelled	150.63	149.47	155.84	157.26
	Top of Pond Elevation including Freeboard (masl)	151.44	151.00	156.75	157.80
	Freeboard Depth (m)	0.81	1.53	0.91	0.54

1) For Phases 1A, 1B and 2

2) Under Ultimate Development Condition

Appendix 7.6 contains calculations that determine the stage-storage-discharge relationship for the outlet control configurations described above. These discharge relationships were then used to run the hydrologic model for all post-development conditions, and verify that the target discharge rates were being achieved. The following **Tables 7.8.6, 7.8.7, 7.8.8 and 7.8.9** summarise the modelled inflow, simulated outflow rates and utilized active storage volumes in Ponds 2, 3, 1 and 5 respectively by using the GAWSER model, for each return period under ultimate development conditions.

Table 7.8.6 – Model Results of Proposed Pond 2 under Ultimate Development Conditions

Storm Events	Inflow (m ³ /s)	Target Outflow (m ³ /s)	Simulated Outflow (m ³ /s)	Utilized Storage (ha*m)	Elevations (m)
2-Yr	0.698	0.107	0.071	0.6166	147.84
5-Yr	0.974	0.165	0.078	0.8721	147.87
10-Yr	1.146	0.198	0.094	0.9939	147.95
25-Yr	1.361	0.250	0.133	1.1700	148.13
50-Yr	1.523	0.284	0.157	1.2759	148.25
100-Yr	1.683	0.319	0.184	1.3981	148.38
Regional	1.842	0.787	0.301	4.0505	150.63

Table 7.8.7 – Model Results of Proposed Pond 3 under Ultimate Development Conditions

Storm Events	Inflow (m ³ /s)	Target Outflow (m ³ /s)	Simulated Outflow (m ³ /s)	Utilized Storage (ha*m)	Elevations (m)
2-Yr	0.925	0.253	0.118	1.1951	146.97
5-Yr	1.281	0.390	0.184	1.5270	147.15
10-Yr	1.492	0.467	0.231	1.7237	147.27
25-Yr	1.788	0.590	0.300	2.0154	147.44
50-Yr	1.996	0.670	0.329	2.1959	147.54
100-Yr	2.207	0.754	0.344	2.4381	147.66
Regional	3.233	1.857	1.438	6.4583	149.47

Table 7.8.8 – Model Results of Proposed Pond 1 under Ultimate Development Conditions

Storm Events	Inflow (m ³ /s)	Target Outflow (m ³ /s)	Simulated Outflow (m ³ /s)	Utilized Storage (ha*m)	Elevations (m)
2-Yr	0.796	0.165	0.087	0.6808	152.90
5-Yr	1.111	0.269	0.150	0.8843	153.14
10-Yr	1.306	0.332	0.198	0.9930	153.25
25-Yr	1.555	0.426	0.269	1.1543	153.43
50-Yr	1.739	0.490	0.303	1.2553	153.53
100-Yr	1.923	0.557	0.318	1.3884	153.68
Regional	2.266	1.299	0.497	3.9624	155.84

Table 7.8.9 – Model Results of Proposed Pond 5 under Ultimate Development Conditions

Storm Events	Inflow (m ³ /s)	Target Outflow (m ³ /s)	Simulated Outflow (m ³ /s)	Utilized Storage (ha*m)	Elevations (m)
2-Yr	0.487	0.100	0.049	0.4302	154.10
5-Yr	0.679	0.153	0.067	0.5756	154.40
10-Yr	0.799	0.182	0.090	0.6501	154.54
25-Yr	0.951	0.229	0.125	0.7598	154.74
50-Yr	1.063	0.260	0.145	0.8262	154.87
100-Yr	1.176	0.292	0.168	0.9028	155.01
Regional	1.385	0.715	0.342	2.5943	157.26

A comparison of resulting Regional flows between the existing conditions and controlled post-development conditions, including interim phases and ultimate conditions at the respective EIR Flow Nodes along Dundas Street and all Reference Flow Nodes within the study area is previously shown in **Table 7.4.5**. The resulting post-development flows for all other storm events (including 2 to 100-year and the Regional storm) are included in a comprehensive flow table (**Table APP-7.2**) in the **Appendix 7.2**. Preliminary details of the proposed SWM facilities are provided in Section 8.0.

Note that, as indicated previously, rooftop storages have been proposed for the proposed developments for the purpose of hydrologic flow regimes analysis for the Reach 14W-12A channel. Such rooftop storages were incorporated in the hydrological models for the proposed conditions. Details are discussed in Section 7.4.3. Information on the conceptual design of rooftop controls are included in the **Appendix 7.6**. The results of the proposed roof controls, including the roof release rates, water ponding depths, utilized ponding storages for the various storm events from 2- to 100- year and Regional storm, are included in the **Appendix 7.6**.

7.8.8 Thermal Mitigation

According to the North Oakville Subwatershed Management Strategy (July 12, 2007), mediation item on temperature and DO targets, a conservative maximum daily temperature target of 20°C needs to be maintained. Excerpts from the mediation item indicate:

“SWM facilities will incorporate measures to address temperature reduction where feasible and practical. It is agreed that there are limited measures available to be used for temperature reduction including pond, outfall and creek plantings, bottom draw outlets, pond configuration and outfall cooling trenches.....Should post construction show that temperature targets are not being met where these types of measures are included in pond design, it is recognized that there may be very limited opportunities to further reduce temperatures.....In other words, best efforts use of acceptable measures for temperature reduction on SWM facilities is required.”

All proposed SWM facilities within the subject study area are designed to incorporate 3.0 m permanent pool depths. Based on discussions with CH, MNRF and the Town staff, it has been recognized that deeper permanent pool will provide benefits for the thermal mitigation as required by the NOCSS.

7.8.9 Summary of SWM Pond Design

Preliminary designs have been completed for the four (4) SWM facilities (wet ponds) associated with the subject study area. The designs are in line with the MOECC and the Town’s design guidelines. Each facility will provide water quality treatment (to ‘Enhanced Level’), erosion control (24- to 48-hour detention of the 25 mm storm event and maintain the differences of erosion exceedance within 5% for post-development conditions), and water quantity control functions – such that controlled release rates for all events from the 2 to 100-year return periods and Regional storm are within the targets set by hydrologic modelling, in order to maintain existing flow regimes in the receiving reach. **Table 7.8.10** below provides a summary of the design features of each SWM facility, and compares them to the criteria suggested by the MOECC and the Town’s design guidelines.

The proposed facilities meet all the ‘minimum criteria’, plus many of the ‘preferred criteria’ as well.

Table 7.8.10 – Summary of Pond Design Criteria

Design Element	MOE Minimum Criteria	MOE Preferred Criteria	Town of Oakville Criteria	Pond 2	Pond 3	Pond 1	Pond 5
Drainage Area	5 ha.	> 10 ha.	-	17.8 ha ^{1) 2) 3)} 20.8 ha ⁴⁾	24.6 ²⁾ 37.0 ³⁾ 39.9 ⁴⁾	23.6 ⁴⁾	14.4 ⁴⁾
Treatment Volume ⁵⁾	As per table 3.2.	Increase PP by max. expected ice vol.	-	PP vol. approx. 132% above minimum MOE Table 3.2 requirement.	PP vol. approx. 230% above minimum MOE Table 3.2 requirement.	PP vol. approx. 188% above minimum MOE Table 3.2 requirement.	PP vol. approx. 19% above minimum MOE Table 3.2 requirement.
		Increase active storage to 25% of total volume		Active storage 72% of total vol.	Active storage 55% of total vol.	Active storage 63% of total vol.	Active storage 74% of total vol.
Active Storage Detention	24 hours	24 hours		47.3 hrs ⁴⁾	53.4 hrs ^{1) 2) 3)} 41.3 hrs ⁴⁾	42.4 ⁴⁾	46.9 ⁴⁾
Forebay	Min. Depth 1.0 m	Min. Depth 1.5 m	Min. Depth 1.5 m	Depth 3.0 m	Depth 3.0 m	Depth 3.0 m	Depth 3.0 m
	Min. Forebay Length based on Settling and Dispersion Calculations (m)			21.56	22.58	22.54	15.36
	Min. Forebay Bottom Width based on 1/8 of the Forebay Length (m)			2.70	2.82	2.82	1.92
	Max. Area 33% of PP			Max. Area 33% of PP	Max. Area 33% of PP	Max. Area 33% of PP	Max. Area 33% of PP
Length to Width Ratio	Overall Min. 3:1	From 4:1 to 5:1	Min. 3:1	Forebay approx. 3:1	Forebay approx. 2:1	Forebay approx. 3:1	Forebay approx. 3:1
	Forebay Min. 2:1			Overall approx. 4:1	Overall approx. 4:1	Overall approx. 4:1	Overall approx. 4:1
Permanent Pool Depth	Max. Depth 3 m	Max. Depth 2.5 m	Min. Depth 1.2 m	3.00 m	3.00 m	3.00 m	3.00 m
	Mean Depth 1-2 m	Mean Depth 1-2 m					
Active Storage Depth	Water Quality & Erosion Control Max. 1.5 m	Water Quality & Erosion Control Max. 1.0 m		Water Quality & Erosion Control: 0.95 m	Water Quality & Erosion Control: 0.80 m	Water Quality & Erosion Control: 1.00 m	Water Quality & Erosion Control: 1.20 m
	Total Depth 2.0 m (100-yr event)	Total Depth 2.0 m (100-yr event)	Max. Depth 2.0 m (100-yr event)	100-Year Depth: 1.69 m ⁶⁾ Regional Depth: 3.94 m ⁷⁾	100-Year Depth: 1.46 m ⁶⁾ Regional Depth: 3.27 m ⁷⁾	100-Year Depth: 1.68 m ⁶⁾ Regional Depth: 3.84 m ⁷⁾	100-Year Depth: 2.01 m ⁶⁾ Regional Depth: 4.26 m ⁷⁾

Design Element	MOE Minimum Criteria	MOE Preferred Criteria	Town of Oakville Criteria	Pond 2	Pond 3	Pond 1	Pond 5
Side Slopes	5:1 for 3 m either side of PP Max. 3:1 elsewhere	7:1 near normal water level plus use of 0.3 m steps 4:1 elsewhere	Min. 7:1 3 m either side of PP Min. 5:1 to upper limit of extended detention Min. 4:1 below the '7:1' zone Min. 3:1 elsewhere	As per Oakville criteria (exceeds MOE requirements)	As per Oakville criteria (exceeds MOE requirements)	As per MOE criteria	As per MOE criteria
Inlet	Min. Pipe Dia. 450 mm Slope > 1%	-	At PP elevation or max. 0.3 m above	Exceeds minimum criteria.	Exceeds minimum criteria.	Exceeds minimum criteria.	Exceeds minimum criteria.
Outlet	Min. Pipe Dia. 450 mm Reverse Slope Pipe Min. Dia. 150 mm Slope > 1% Orifice Min. Dia. 75 mm	Orifice Min. Dia. 100 mm	Primary outlet to be bottom draw	Exceeds minimum criteria. Orifice 200 mm dia.	Exceeds minimum criteria. Orifice 255 mm dia. 1) 2) 3) 290 4)	Exceeds minimum criteria. Orifice 215 mm dia.	Exceeds minimum criteria. Orifice 155 mm dia.
Buffer	Min. 7.5 m above max. water quality/erosion control level Min. 3 m above water quantity control level	-	7.5 m buffer beyond 100-year high water level (max. buffer grade 10%)	As per Oakville criteria (exceeds MOE requirements)	As per Oakville criteria (exceeds MOE requirements)	As per Oakville criteria (exceeds MOE requirements)	As per Oakville criteria (exceeds MOE requirements)

1) Phase 1A

2) Phase 1B

3) Phase 2

4) Ultimate Development Conditions

5) Ultimate Development Conditions as Most Conservative Scenarios

6) 100-Year depth based on the calculated utilized storage volume from the model for 100-year simulations

7) Regional Storm depth based on the calculated utilized storage volume from the model for Regional storm simulations

A pre- and post-construction monitoring plan for the different SWM facilities, realigned reaches, and different municipal infrastructures is provided in **Appendix 7.8**. The monitoring plan outlined in **Appendix 7.8** describes the monitoring strategy and the items to be monitored.

7.9 Downstream Impacts for Regional Storm

Policy 8.4.13.2 of OPA 289 states, “*The North Oakville Creeks Subwatershed Study recommends that stormwater targets include control of the peak flow to predevelopment levels for various return periods, including the regional storm. Through the land development application process, an investigation of the potential increase to flood risk may be carried out to confirm if Regional Storm controls are necessary, in accordance with the directions established in the North Oakville Creeks Subwatershed Study.*”

The proposed SWM facilities within the Subject Property are designed to provide Regional controls for the developments. Therefore, an investigation of the potential increase to flood risk is not required as per NOCSS. However, we understand that the quantity controls of upstream tributaries would delay the peak flows to correspond more closely with the timing of the peak flow in the main branch, thereby, causing the potential increases in peak flows in the receiving watercourse downstream. As a prudent measure, a hydrological analysis for the entire Fourteen Mile Creek subwatershed was carried out to determine the flows at downstream locations and ensure there would be no impact due to the development at subject lands located at upstream of the subwatershed during Regional storm conditions.

The original hydrological model was obtained from the “*Fourteen Mile Creek and McCraney Creek System Flood Mitigation Opportunities Study*” provided the Town. The model was developed by using PCSWMM model for the entire Fourteen Mile Creek sub-watershed. The resulting post-development hydrographs at Reference Node 4 (immediate downstream of culvert FM-D3 at Dundas Street) from GAWSER model for the Regional event scenarios were input to the same flow node location in PCSWMM model (Conduit # 3660.527, PCSWMM Node # DS1 as shown in **Figures APP-7.7.1 and APP-7.7.2**). The results of the Regional peak discharge rates at downstream locations along Fourteen Mile Creek from PCSWMM simulations are summarized in **Table 7.9.1**. **Table 7.9.1** also provides a comparison between existing, uncontrolled and controlled conditions for all interim and ultimate development scenarios.

Table 7.9.1 – Downstream Regional Flow Comparison

PCSWMM Downstream Flow Node			DS1	DS2	DS3	DS4	DS5	DS6
PCSWMM Catchment			215	210	110	108	107	101
Location			Dundas Rd	Richview Blvd	Bronte Rd	Fourteen Mile Lands	HWY 403	Lake Ontario
Area (ha)			408	457	1063	1605	2318	3118
Existing		Regional Flow (m ³ /s)	20.3	23.2	89.9	139.8	192.6	243.2
		% difference						
PH1A	Uncontrolled	Regional Flow (m ³ /s)	20.6	23.8	90.5	140.4	193.2	243.7
		% difference	1.3%	2.7%	0.7%	0.5%	0.3%	0.2%
	Controlled	Regional Flow (m ³ /s)	19.1	21.9	88.7	138.6	191.5	242.2
		% difference	-5.9%	-5.8%	-1.3%	-0.9%	-0.6%	-0.4%
PH1B	Uncontrolled	Regional Flow (m ³ /s)	20.5	23.9	90.6	140.5	193.3	243.8
		% difference	0.5%	3.0%	0.8%	0.5%	0.4%	0.2%
	Controlled	Regional Flow (m ³ /s)	18.5	21.5	88.5	138.3	191.3	242.0
		% difference	-9.1%	-7.3%	-1.6%	-1.0%	-0.7%	-0.5%
PH2	Uncontrolled	Regional Flow (m ³ /s)	22.1	26.2	92.9	142.8	195.4	245.8
		% difference	8.8%	12.9%	3.3%	2.1%	1.4%	1.0%
	Controlled	Regional Flow (m ³ /s)	18.2	20.9	87.9	137.7	190.8	241.5
		% difference	-10.8%	-10.0%	-2.3%	-1.5%	-1.0%	-0.7%
Ultimate	Uncontrolled	Regional Flow (m ³ /s)	23.2	27.5	94.3	144.1	196.6	246.8
		% difference	14.0%	18.5%	4.8%	3.1%	2.1%	1.5%
	Controlled	Regional Flow (m ³ /s)	17.4	20.1	87.2	137.0	190.2	241.1
		% difference	-14.5%	-13.2%	-3.0%	-2.0%	-1.2%	-0.9%

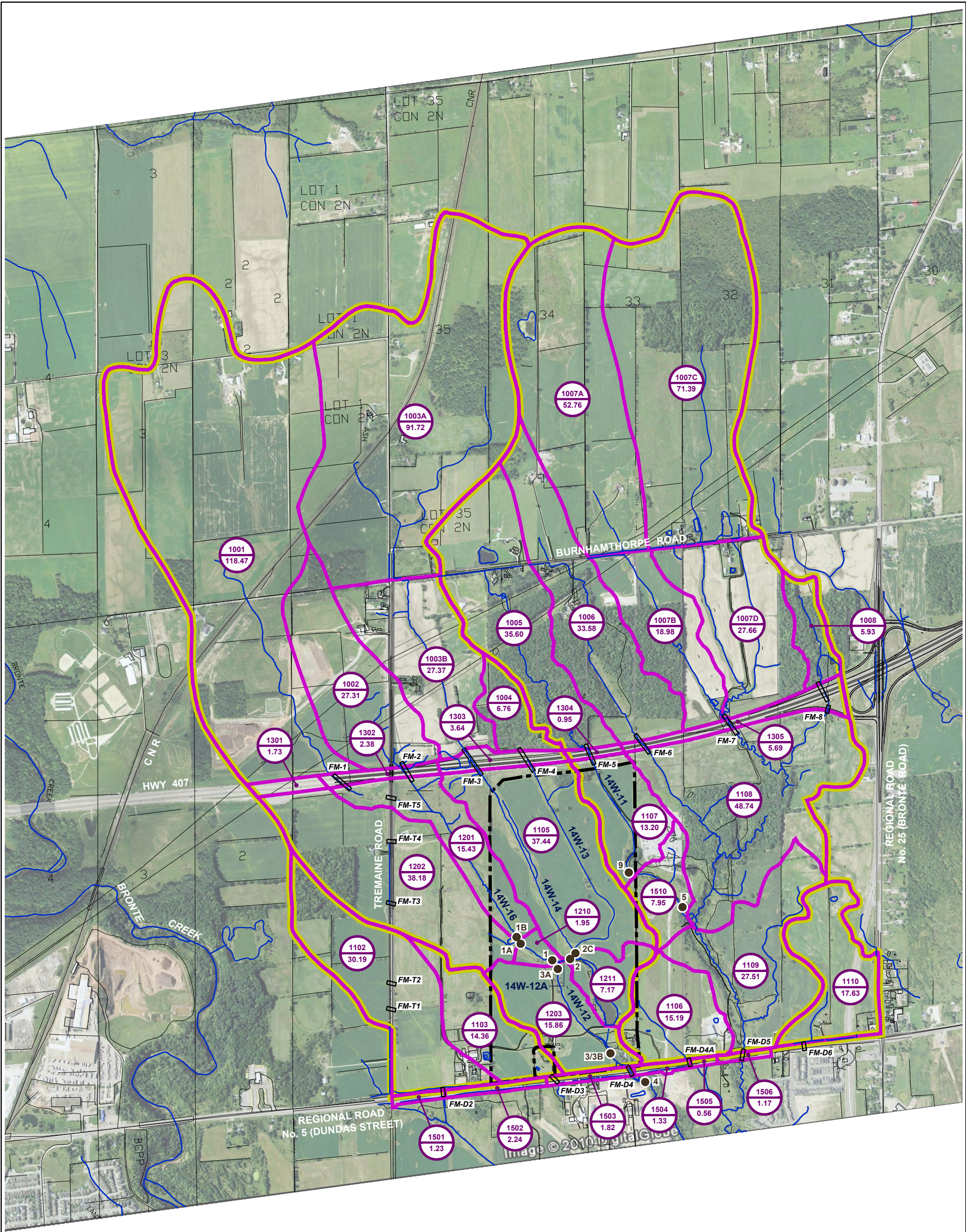
As shown in **Table 7.9.1**, with the proposed SWM facilities providing Regional controls for the developments within the study area, peak flows at all downstream flow nodes along Fourteen Mile Creek to its outlet at Lake Ontario will be lower than the existing levels during Regional event. Consequently, it is confirmed that there will be no impact to the downstream watercourses due to the development of the Subject Property located upstream of the sub-watershed.

Details of this analysis are provided in **Appendix 7.7**. **Appendix 7.7** also includes **Figure APP-7.7.1** which shows the PCSWMM sub-catchment boundary plan. PCSWMM model schematic is also included in **Figure APP-7.7.2** in **Appendix 7.7**.

7.10 Summary

In accordance with NOCSS and the EIR/FSS ToR, a SWM plan has been developed for the Subject Property within the 407 West Employment Area. The following provides a summary for the developed SWM plan.

- **Water Quantity:** The SWM facilities are sized to control of post-development peak flows to pre-development levels for the 2 to 100-year return period events and the Regional storm.
- **Water Quality:** The SWM facilities are designed to meet MOECC's Enhanced Level of water quality protection (Level 1) for water quality control, phosphorus control and fisheries protection.
- **Erosion Control:** The detailed erosion threshold analyses including a fluvial geomorphological study were performed to ensure the proposed SWM facilities would provide adequate erosion control protection for the downstream watercourses, so that existing channel erosion or aggradation is not exacerbated by development.
- **Hydrologic Flow Regimes Analysis:** A comprehensive investigation of impact of development has been carried out on all flow nodes within the Subject Property. Where reaches were to be realigned or where habitat concerns had been communicated with the study team, detailed assessments were incorporated. Specifically, the magnitude of peak flows will decrease by approximately 15 to 20% from existing conditions for Reach 14W-22, and Reach 14W-23, and the duration and frequency will be similar. For 14W-12A, although significant reductions in streamflows are anticipated, the wetted perimeter and continuity of the flows will be maintained.
- **Topographic Depression Volumes:** Evaluation of the existing depression storage was performed to ensure that the natural depression storage would be maintained in the SWM system.
- **SWM Pond Design:** The SWM facilities are design to meet all the criteria as enforced by the MOECC and in accordance with the Town's design guidelines.
- **Downstream Impacts for Regional Storm:** The proposed SWM facilities will provide Regional controls for the developments within the subject lands, and as such, there will be no impact to the downstream watercourses due to the developments of the Subject Property located at upstream of the sub-watershed.



Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

Existing Drainage Boundaries and Reference Nodes (Revised from NOCSS by MMM)

LEGEND

- SUBJECT PROPERTY
- SUB-CATCHMENT BOUNDARY
- EIR SUB-CATCHMENT BOUNDARY
- EXISTING CREEK
- CULVERTS (EIR NODES)
- SUB-CATCHMENT No.
- AREA (ha)
- REFERENCE NODE

1001
118.47

1

Scale

1 : 15,000

0 0.1 0.25 0.5 0.75km

Client

Prepared by

Date

April 2018

Project No.

09M-00013-01-WR1

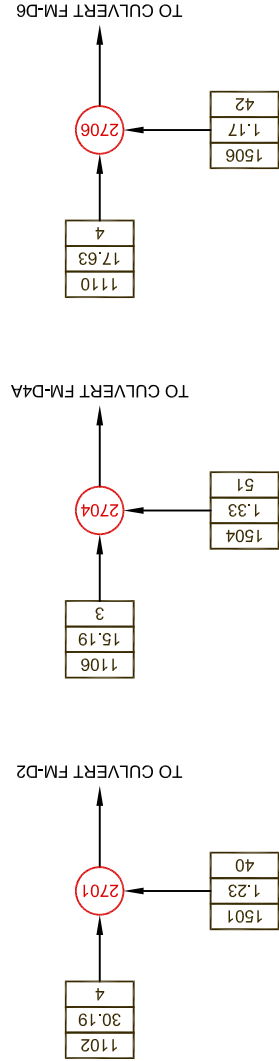
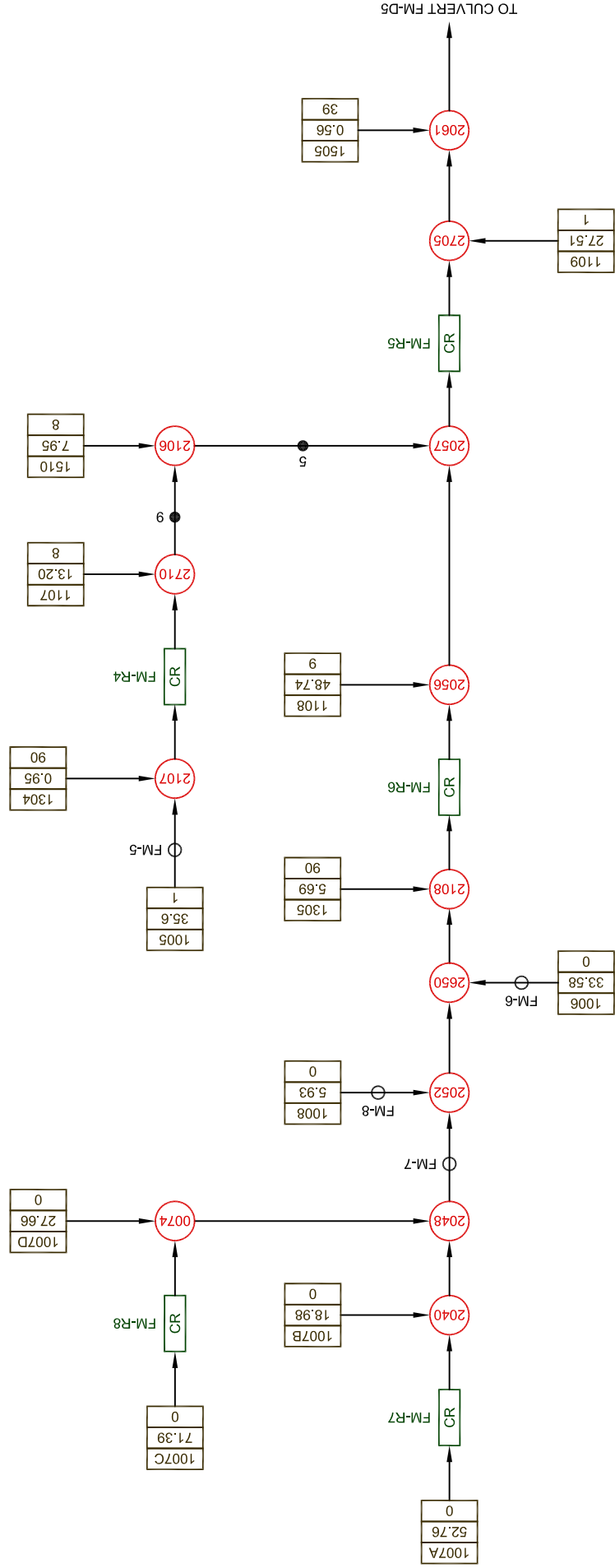
Aerial Photo

© DigitalGlobe 2010, Google 2009

QuadReal

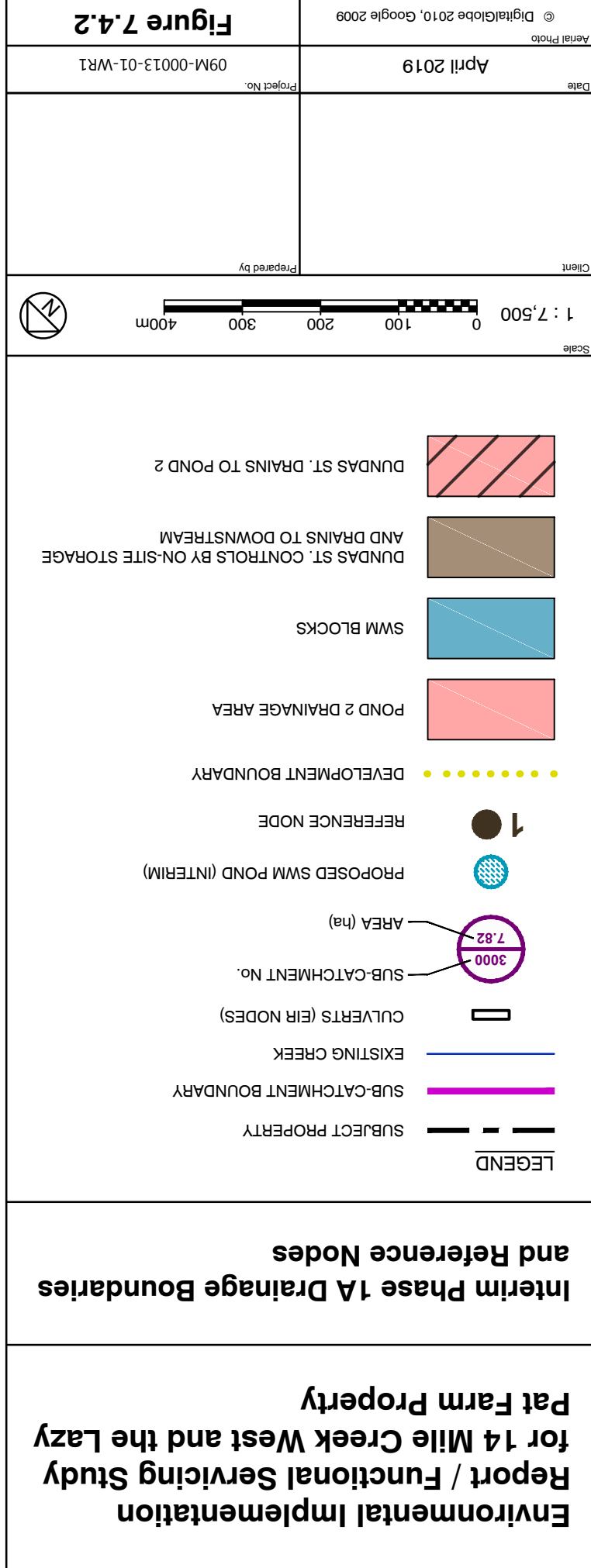
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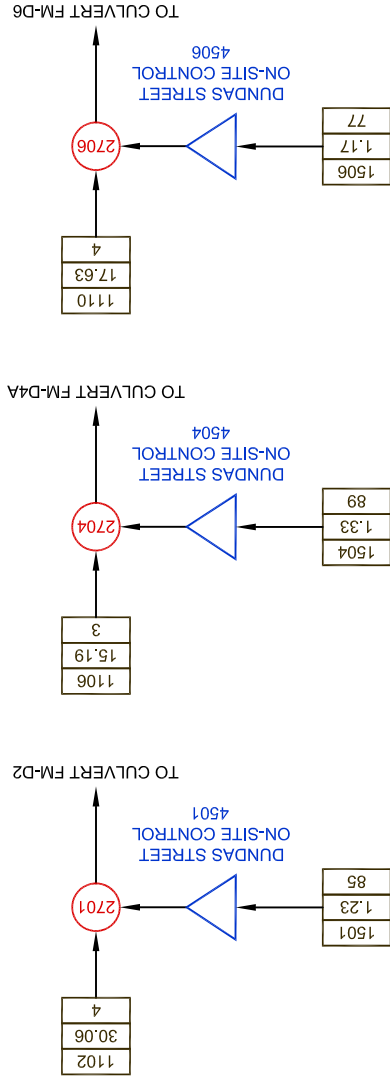
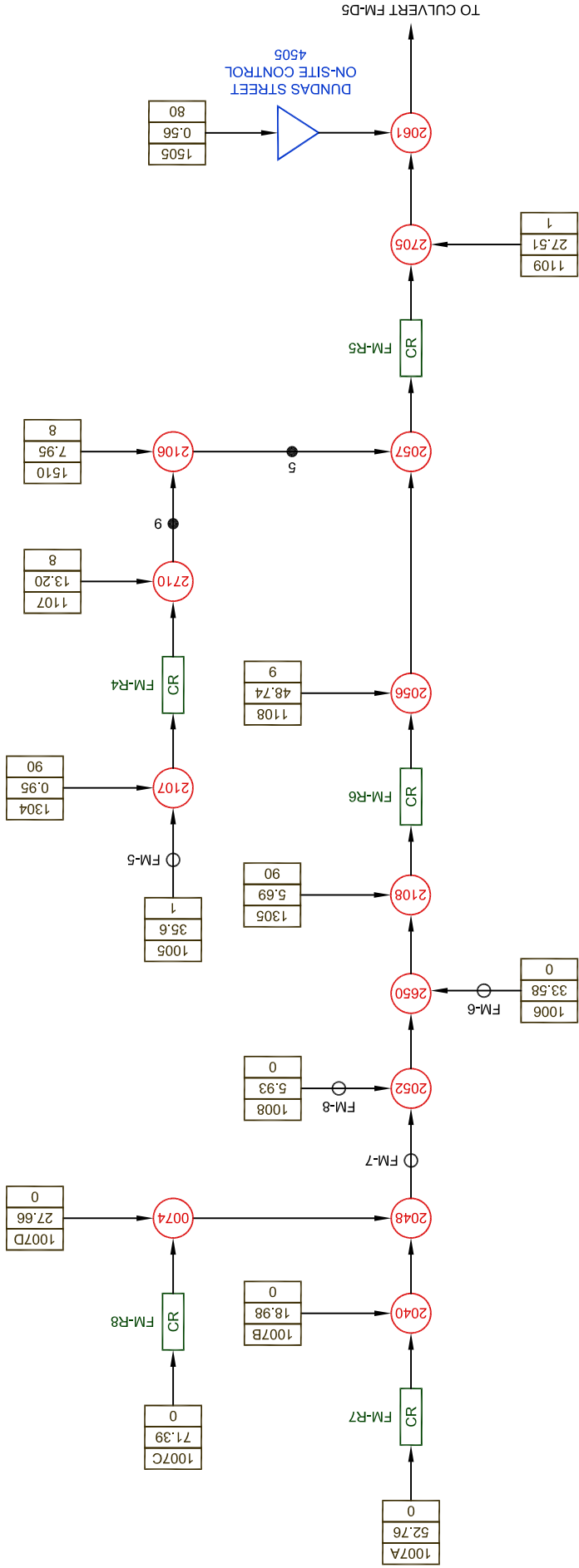
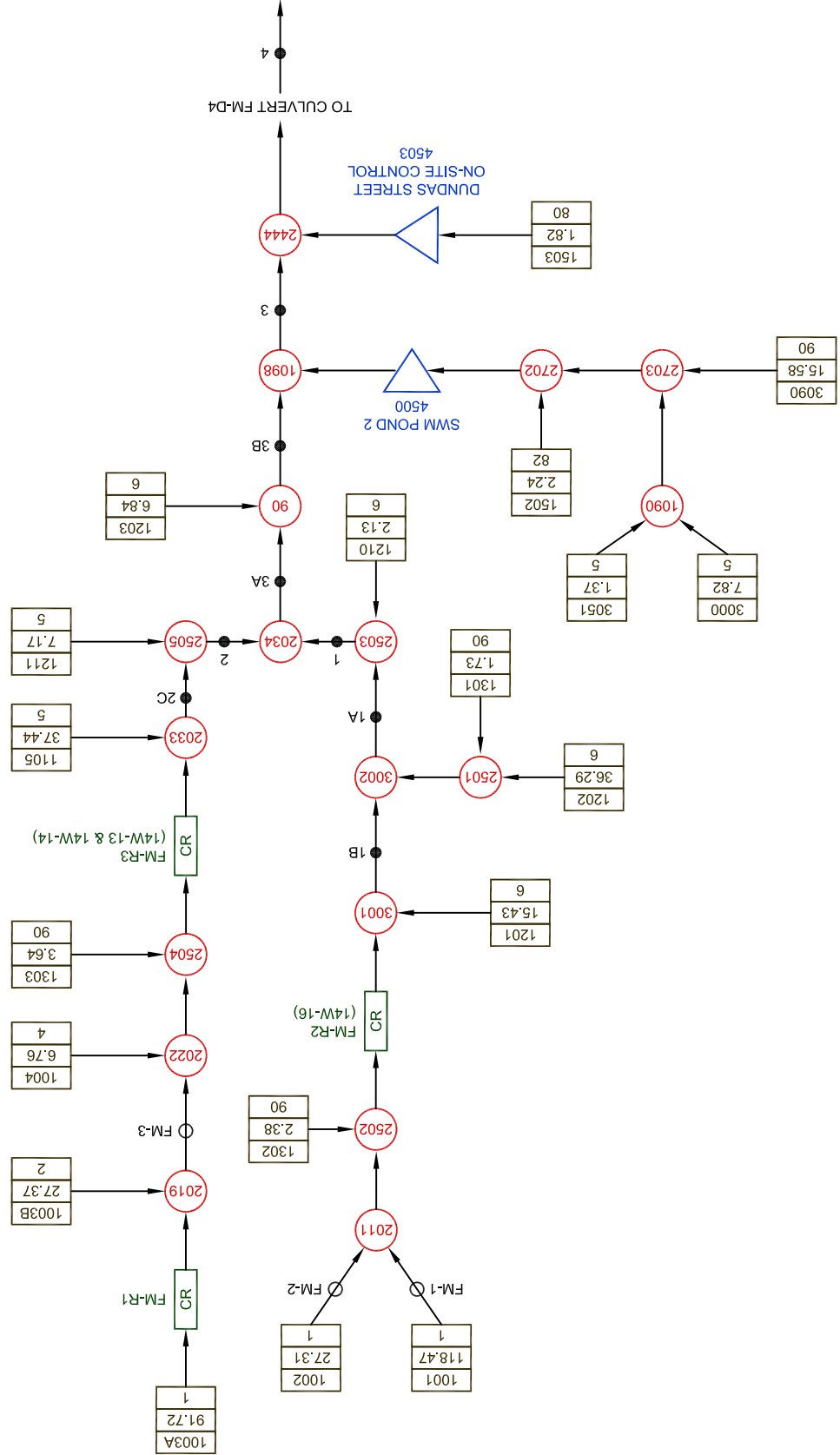
Figure 7.4.1



**Environmental Implementation
Report / Functional Servicing Study
for 14 Mile Creek West and the Lazy
Pat Farm Property**

Interim Phase 1A Drainage Boundaries and Reference Nodes





Pat Farm Property

Report / Functional Servicing Study

for 14 Mile Creek West and the Lazy

Environmental Implementation

Modelling Schematic -

Interim Phase 1A Conditions

1105

CATCHMENT ID.

37.44

CATCHMENT AREA (ha)

5

IMPERVIOUSNESS (%)

○

CULVERT (EIR)

●

REFERENCE

○

NODE

CR

CHANNEL ROUTING

△

ROUTE RESERVOIR

DIV

DIVERSION

○

ADDHYD

CR

CHANNEL ROUTING

△

ROUTE RESERVOIR

DIV

DIVERSION

Client

Prepared by

WSP

Quaddreal

Date

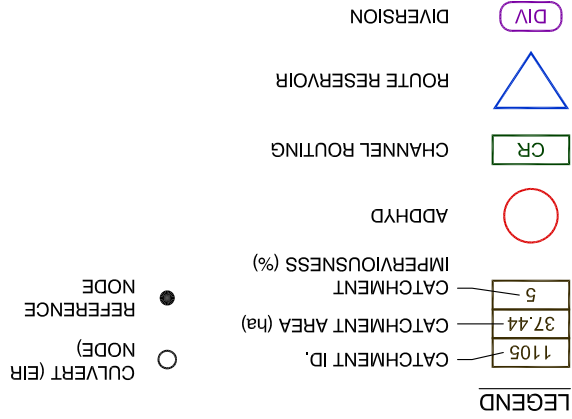
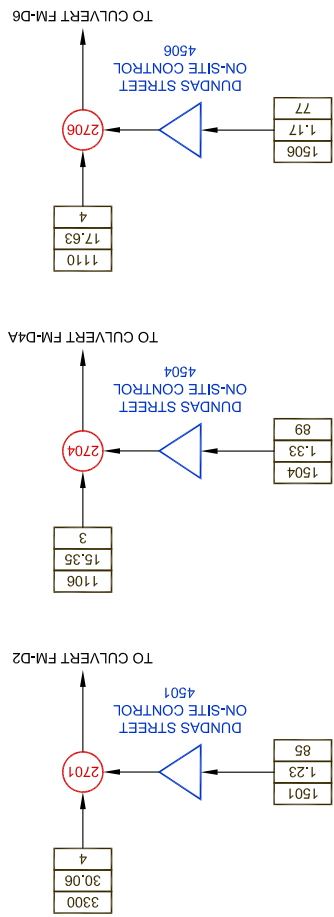
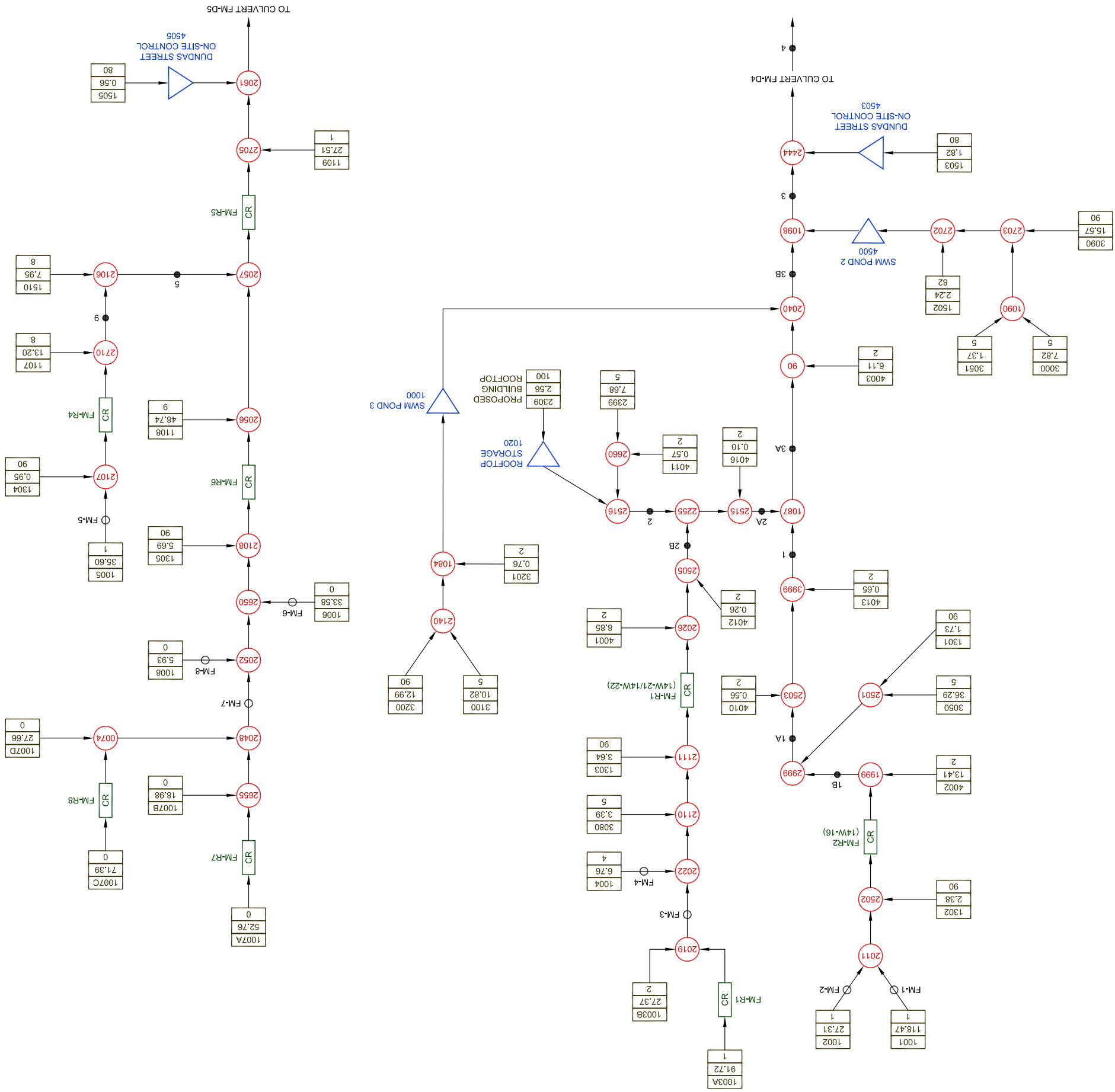
April 2018

Project No.

09M-00013-01-WR1

Figure 7.4.2-SCH

Aerial Photo



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Prepared by



Project No.

April 2018

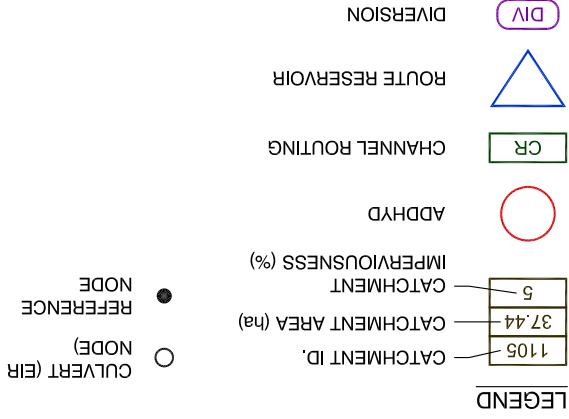
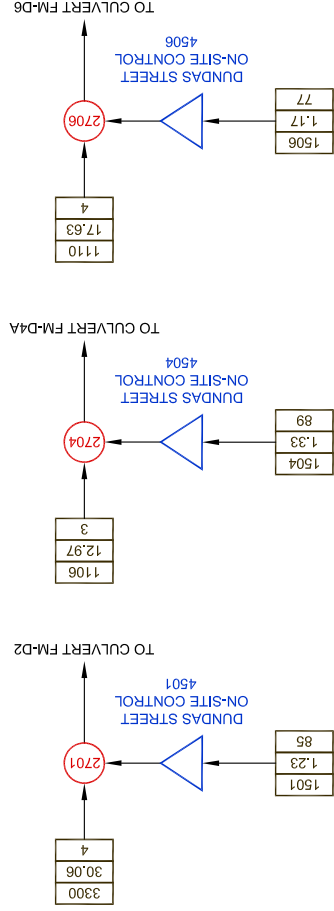
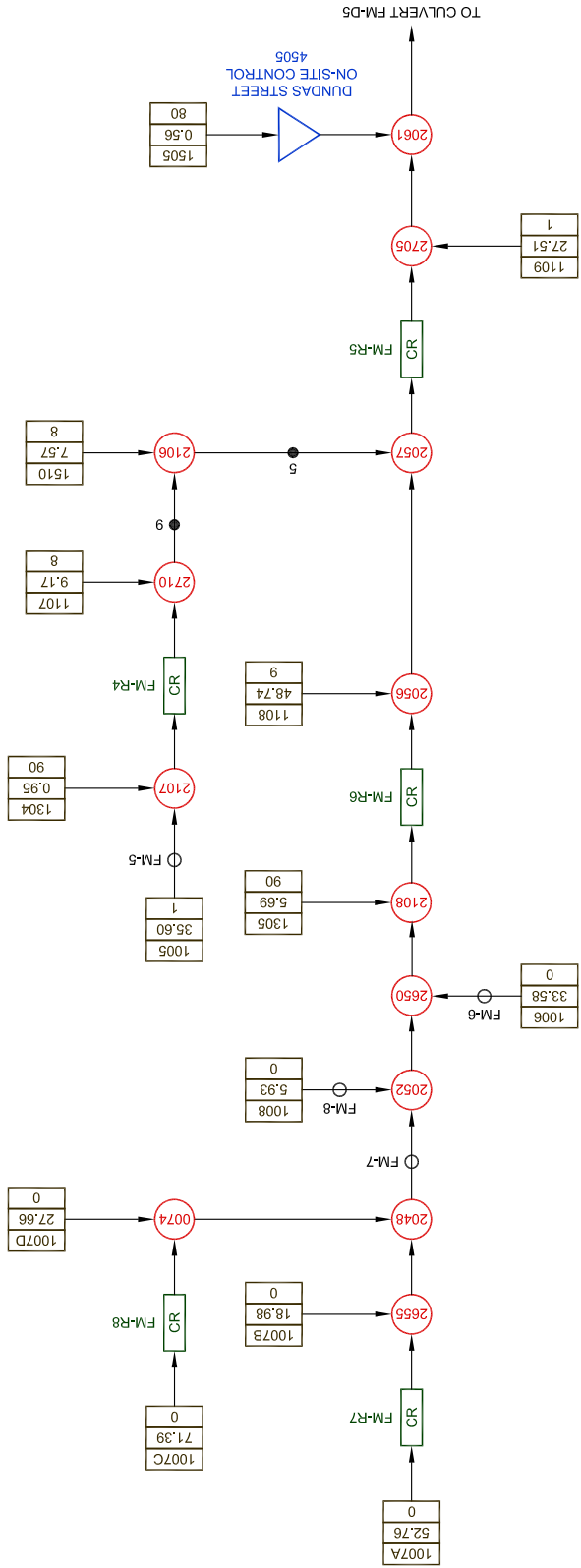
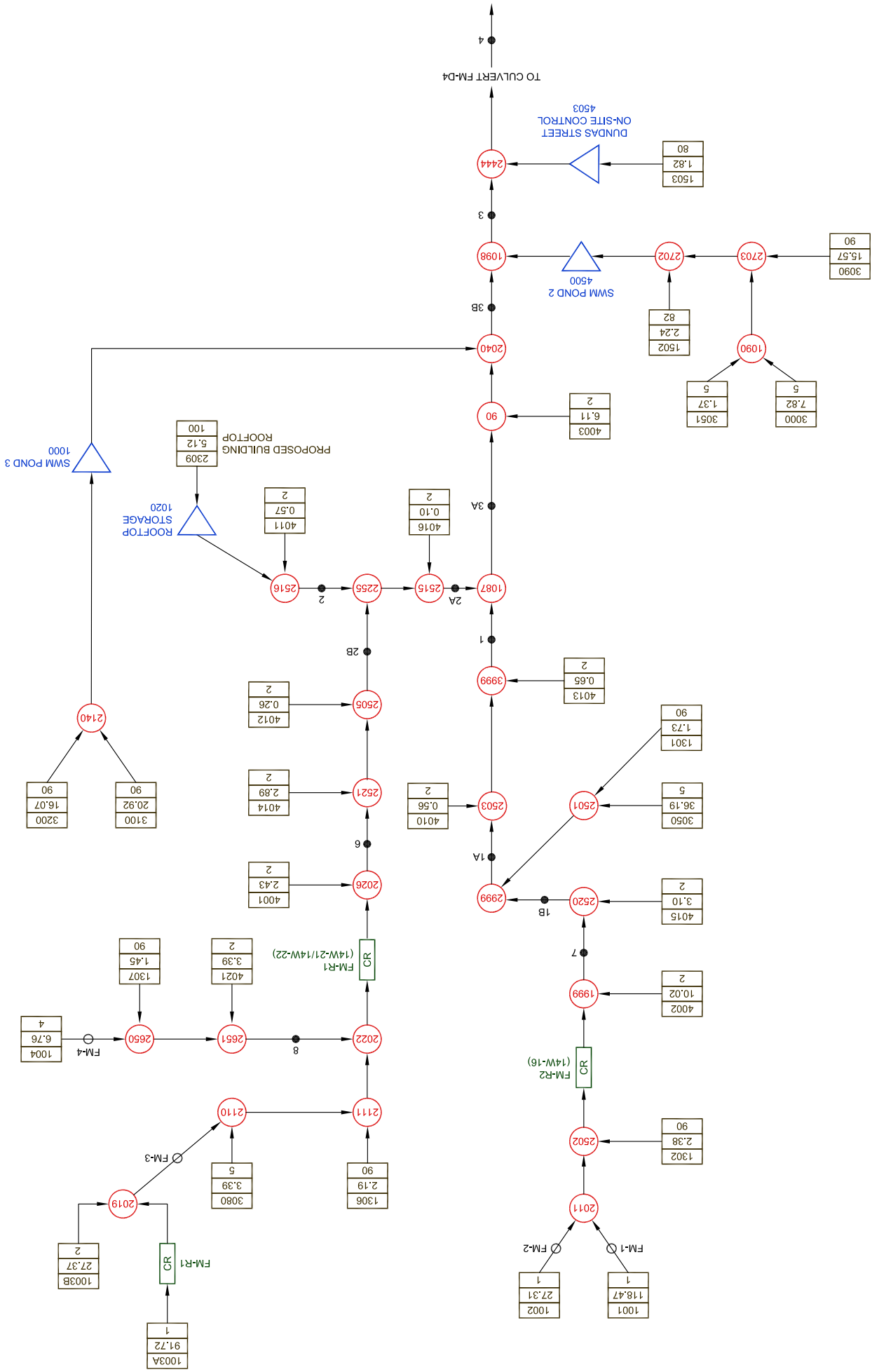
09M-00013-01-WR1

erial Photo

Figure 7.4.3-SCH

**Environmental Implementation
Report / Functional Servicing Study
for 14 Mile Creek West and the Lazy
Pat Farm Property**

Modelling Schematic - Interim Phase 1B Conditions



Modelling Schematic - Interim Phase 2 Conditions

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property



Client: N.T.S.

Scale:

Prepared by:

Date: April 2018

Project No.: 09M-00013-01-WR1

Figure 7.4.4-SCH

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

Ultimate Drainage Boundaries and Reference Nodes

LEGEND

- SUBJECT PROPERTY
- SUB-CATCHMENT BOUNDARY
- EXISTING CREEK
- PROPOSED REALIGNED CREEK
- CULVERTS (GIR NODES)
- SUB-CATCHMENT No. 3000
- AREA (ha) 7.82
- PROPOSED SWM POND (INTERIM)
- REFERENCE NODE
- DEVELOPMENT BOUNDARY
- POND 2 DRAINAGE AREA
- SWM BLOCKS
- DUNDAS ST. CONTROLS BY ON-SITE STORAGE AND DRAINS TO DOWNSTREAM
- DUNDAS ST. DRAINS TO POND 2
- POND 3 DRAINAGE AREA
- AREA DRAINS TO 14W-12A (REF. NODE 2)
- TREMAINE POND DRAINAGE AREA
- POND 5 DRAINAGE AREA
- POND 1 DRAINAGE AREA

Scale 1 : 10,000

0 0.1 0.2 0.3 0.4 0.5km

Client Prepared by

Date June 2018

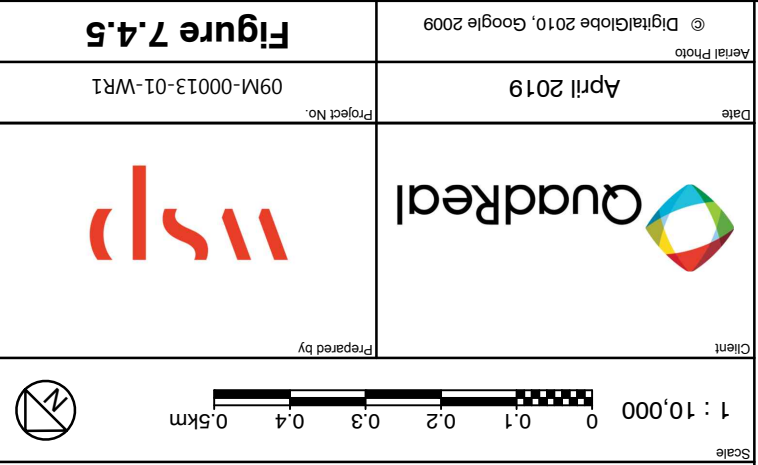
Project No. 09M-00013-01-WR1

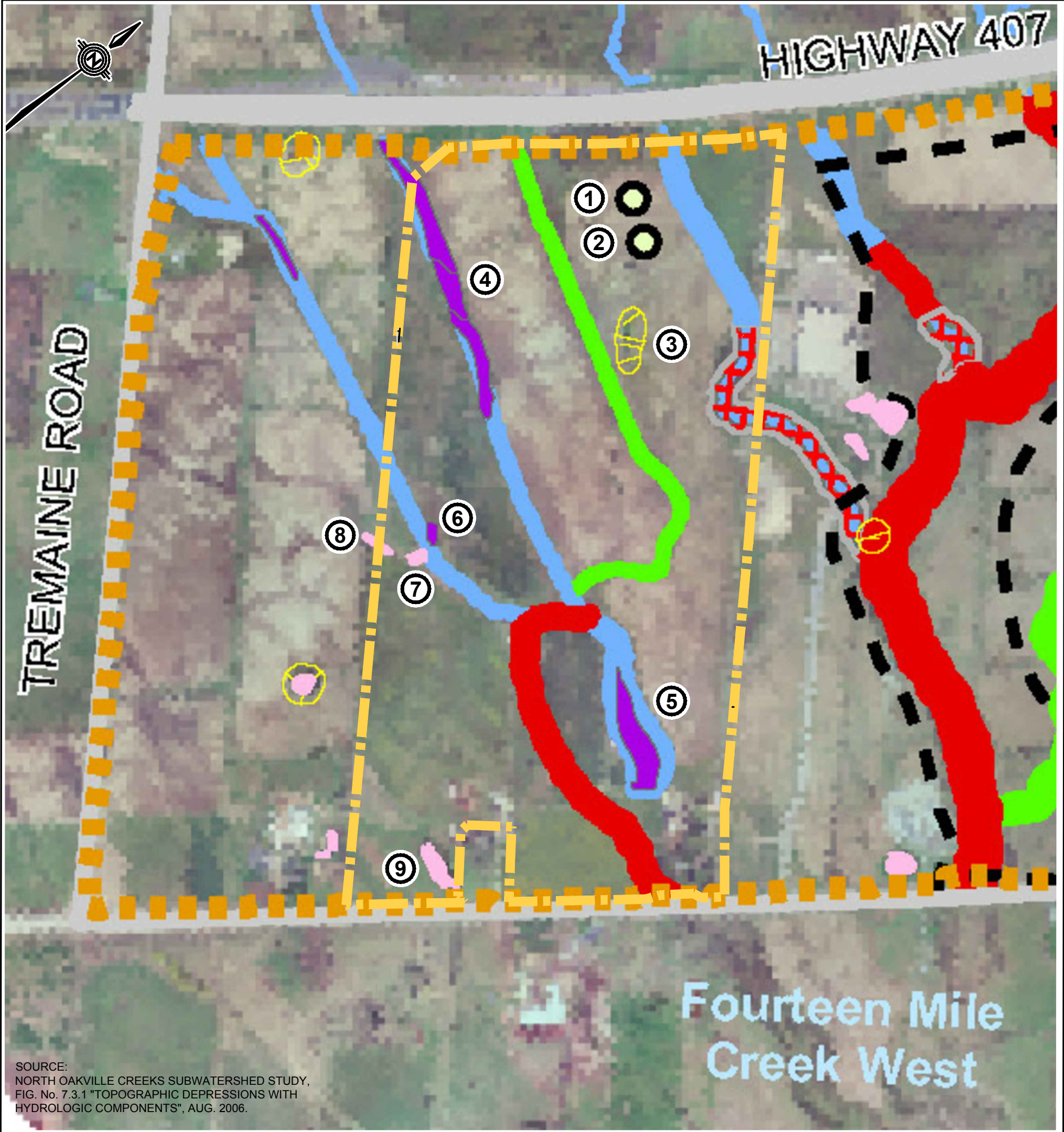
Figure 7.4.5



Ultimate Drainage Boundaries and Reference Nodes

Ultimate Drainage Boundaries and Reference Nodes





SOURCE:
NORTH OAKVILLE CREEKS SUBWATERSHED STUDY,
FIG. No. 7.3.1 "TOPOGRAPHIC DEPRESSIONS WITH
HYDROLOGIC COMPONENTS", AUG. 2006.

Environmental Implementation
Report / Functional Servicing
Study for 14 Mile Creek West
and the Lazy Pat Farm Property

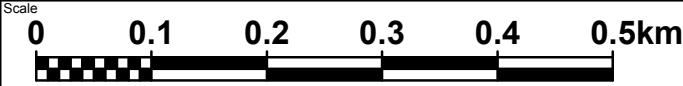
Topographic Depression in the
Development Area

LEGEND

- Secondary Plan Boundary
- Core
- Linkage
- Hydrologic Feature 'A'
- Hydrologic Feature 'B'
- Topographic Depressions (with Hydrologic Component)
- Topographic Depressions
- Watercourse

Stream Corridor

- High Constraint
- High Constraint - Requiring Rehabilitation
- Medium Constraint
- Low Constraint



Client	Prepared by

Date	Project No.
April 2018	09M-00013-01-WR1

Aerial Photo	Figure 7.7.1
© DigitalGlobe 2010, Google 2009	

8.0 Municipal Services



8.0 Municipal Services

8.1 Introduction

The municipal services component of the EIR/FSS addresses the servicing (water, wastewater, and stormwater) and grading requirements to develop the FSS Study Area. The servicing design is intended to work in conjunction with the SWM plan and other considerations as detailed in Section 7.0.

The municipal servicing design for the EIR/FSS has been conceptually developed utilizing the land use concept plan as its basis. The land use concept plan illustrates natural features, the pattern of major roads and the land uses. The proposed municipal servicing design generally follows the proposed road layout while taking into account the natural features and topography of the site. This is appropriate at the EIR/FSS level.

To develop the municipal servicing design the Region's Water and Wastewater Master Plan (WWMP) and the SWM Plan in Section 7.0 were reviewed and utilized. The design and documentation has been developed in conjunction with the 407 West Employment Lands – Area Servicing Plan (ASP) prepared by WSP (formerly MMM Group), which was approved by the Region (June 2014). The ASP generally covers the lands bounded by Dundas Street West, Bronte Road, the Highway 407 and Tremaine Road. The EIR/FSS is consistent with the ASP and does not create undue constraints on the servicing of other lands within or external to the ASP boundary. The Region's Master Plan indicates that the servicing of the lands to the east and west are to be self-contained. Specifically, the lands to the west known as the Evergreen Secondary Plan (formerly Tremaine and Dundas Secondary Plan, are currently in the Public Consultation Phase. A draft Land Use Plan and Secondary Plan was prepared for consultation in April 2017. We have reviewed the Water and Wastewater Servicing Assessment prepared by AECOM (September 2009) for the lands. This report indicates that water servicing is to be provided from the proposed watermain on Tremaine Road, sanitary servicing is tributary to the trunk sewer on Dundas Street West, and the Secondary Plan options included in the report indicate that SWM will be provided onsite. This is generally in conformance with the servicing strategy outlined in the ASP and in this EIR/FSS.

In the WWMP and the subsequent update, the Region considered a wide variety of possible strategies to service the expected growth throughout the entire Region for both water and wastewater servicing. The conclusions of the Region's work with respect to treatment and conveyance (including conveyance options) as it affects the FSS Study Area are summarized in the following Wastewater and Water Sections.

The SWM Plan for the 407 West Employment Area is generally consistent with NOCSS. NOCSS sets the standards and requirements for the treatment of stormwater throughout North Oakville.

8.2 Conceptual Wastewater Servicing Strategy

8.2.1 General

The Region provides the Town with Wastewater Treatment, Sewage Pumping Stations and Wastewater collection services. The wastewater infrastructure requirements are outlined below.

8.2.2 Treatment

Wastewater treatment for Oakville is provided at three Wastewater Treatment Plants (WWTP). These plants are Mid-Halton, Oakville South East, and Oakville South West. The recommended alternative in the Master Plan proposed that all wastewater treatment for growth in Oakville and Milton be at the Mid-Halton WWTP.

The first phase of Mid-Halton WWTP was constructed in 1991 with a rated capacity of 20,000 m³/d. It was subsequently re-rated to 25,000m³/d. The Region completed an expansion of the plant to 50,000 m³/d in 2003 and has subsequently completed the next expansion to 75,000 m³/d to service anticipated growth within Milton and Oakville. A subsequent expansion to 125,000 m³/d is currently underway; and construction is expected to be completed soon.

The Region has planned the Mid-Halton Wastewater plant and has sufficient land to allow it to be expanded in an orderly and predictable fashion. These expansions would be timed so that the capacity is available when required by development throughout the region. Expansion of capacity will trigger the need for various other changes or improvements such as biosolids handling and a new outfall (currently underway).

8.2.3 Collection System

Figure 8.1 is a representation of the portion of the Region's proposed wastewater collection and pumping system that are intended to service the 407 West Employment Area, the lands to their east and a portion of the expected growth in Milton.

The 407 West Employment Area generally slopes from north to south and towards the centre of the lands, approximately 180 m to the east of Colonel William Parkway at Dundas Street. The Master Plan provides for a series of local sewers that will drain from north to south connecting to a new Trunk Sewer System on Dundas Street. The Region's Master Plan does not show the sewers within the 407 West Employment Area. One of the purposes of this report is to apply the Master Plan concept to the Land Use Plan road and development scheme and recommend a specific plan for the sewer system. At Dundas Street, a trunk system which directs the flows to the existing system at Colonel William Parkway is proposed. The existing system will intercept flow and divert it south towards the Mid-Halton Plant in a manner that minimizes impact to the existing residents of the Town of Oakville.

More specifically, the lands to the west of the western natural heritage area and east of Tremaine Road will be conveyed south by an internal wastewater sewer along an internal Avenue 2 to the Trunk Sewer on Dundas Street West which will then connect to the existing wastewater system at Colonel William Parkway.

The external area from the lands west of Tremaine Road can connect to the system at the future Burnhamthorpe Road Extension, Avenue 3 or on Dundas Street West. The wastewater flows from the lands between the two natural heritage areas will be conveyed south along an internal Avenue 3 and discharged to the existing wastewater system on the south side of Dundas Street at Colonel William Parkway. The lands east of the Fourteen Mile Creek NHS and west of Bronte Road will be conveyed south along Avenue 3 and will connect to the Trunk Sewer on Dundas Street West which will then discharge to the existing system at the south side of Dundas Street at Colonel William Parkway. The trunk sewer and downstream system on Colonel William Parkway has been designed to accommodate an external area of 362 ha and an equivalent residential population of 8,145 people. Please refer to the design calculations and drainage plan for the Bronte Creek Community completed by Stantec in Appendix 8.1.

There is the possibility for the flows from a portion of the lands between the Fourteen Mile Creek NHS and Bronte Road to discharge to the existing system at Valleyridge Drive. The Region has advised that the sewers on Valleyridge Drive were designed to accommodate an external area of 80 ha with an equivalent population of 10,000 people. As shown on Figure 8.2, the 407 West Employment Area lands that would be tributary to this sewer have an area of approximately 40 ha and an equivalent population of 4,500 people. This alternative servicing option would require extending the existing sewer to Dundas Street and would eliminate the need for a sewer on Dundas crossing the Fourteen Mile Creek NHS. This is shown on Figure 8.2 as an alternative wastewater sewer alignment.

The existing system eventually discharges to the Mid-Halton WWTP and PS. No upgrades to the existing wastewater sewers are anticipated to accommodate future development. The need for a local or regional pumping station appears not to be required.

8.2.4 Region's Timing of Required Wastewater Infrastructure

Regional project 6911 is required to service the FSS Study Area. This project can be completed as development requires. The connection point at Colonel William Parkway is in place with available capacity to service the FSS Study Area.

8.2.5 Expected Sewage Generation

In this section sewage generated in the 407 West Employment Area Land Use Plans has been assessed and compared to the Master Plan. The design criteria that the Region has utilized in the Master Plan are used in this analysis. To develop the estimated sewage generation, the system design criteria is first set out and then applied to the proposed development statistics from Figure 3.1.

The Region's wastewater system criteria are as follows in Table 8.1 below:

Table 8.1 – Average Day Wastewater Flow

Land Use	Unit	Collection System	Treatment
Residential	L/cap/d	275	365
Commercial	m ³ /ha/d	24.75	26.00
Industrial	m ³ /ha/d	34.38	17.63
Institutional	m ³ /ha/d	10.94	10.94

The modified Harmon Peaking Factor equation is used to determine the peak flows for the collection system. The average day wastewater flow criteria for wastewater treatment include an allowance for infiltration. An infiltration allowance of 0.286 L/s/ha is added to the peak system flows for designing the collection system.

The treatment capacity flow generated by the 407 West Employment Area is illustrated in Table 8.2. Commercial land use demands have been applied to the entire site to provide a conservative estimate and to allow for flexibility with respect to the ultimate land use mix.

Table 8.2 – Generated WWTP Flows: 407 West Employment Area Land Use Plan Projections

	Residential ML/d	Commercial ML/d	Industrial ML/d	Institutional ML/d	Total ML/d
Average Daily Flow	0.0	4.1	0.0	0.0	4.0

Pumping stations and sewers are designed based upon peak flows. Flows will increase as various sub-catchment areas are connected to the Trunk Sewer. Table 8.3, which follows, estimates the peak flow to the existing wastewater sewer on Colonel William Parkway. This is the full flow from the 407 West Employment Area.

The difference in the peak flows between the Region's projections and from those generated from the 407 West Employment Area Land Use Plan combined with the proposed increase in pipe slope in some instances will impact the sizing of the Dundas Street trunk sewer by one pipe size in some locations. Industrial land use demands have been applied to the entire site to provide a conservative estimate and to allow for flexibility with respect to the ultimate land use mix.

Table 8.3 – Peak Generated Collection System at Colonel William Parkway Trunk Sewer: 407 West Employment Area Only

	Residential L/s	Commercial L/s	Industrial L/s	Institutional L/s	Sub total L/s
Average Flow	0.0	0.0	62.3	0.0	62.3
Peaking Factor	4.3	4.3	2.662	4.3	
K	0.80	0.80	0.80	0.80	0.80
Infiltration	0.0	0.0	44.8	0.0	44.8
Sub total	0.0	0.0	177.5	0.0	177.5

8.2.6 Region's Concept Plan Applied to the FSS Study Area

The proposed sewer system to service the FSS Study Area is described in this section and, as well as, the proposed drainage boundaries as illustrated on Figure 8.2.

8.2.6.1 Dundas Street Wastewater Sewer

As discussed in the above sections, the Region proposes that all wastewater flows from the FSS Study Area drain to a trunk wastewater sewer system along Dundas Street West. As plans were being developed for the FSS Study Area, alternative locations for this wastewater sewer were considered. At this stage it has been determined that Dundas Street West would be a feasible alignment for the wastewater sewer.

The Master Plan recommends that the Dundas Street West gravity wastewater sewer directing flows from the entire 407 West Employment Area to the gravity wastewater sewer on Dundas Street West and ultimately to the existing wastewater sewer on Colonel William Parkway. This study generally supports that conclusion.

A conceptual design has been undertaken for the trunk wastewater sewer on Dundas Street West, which is presented on the attached drawings, P1-P17.

8.2.6.2 Internal Collection Systems

To convey wastewater drainage from the Subject Property to the Dundas Street West sewer collection system, various alternative system layouts were evaluated. The common elements of the system layouts were:

- All sewers are located on proposed road alignments; and
- All crossings of watercourses on natural features follow proposed road alignments.

Several factors were considered that would influence the proposed alternatives. The factors include environmental features, existing topography, proposed road patterns, SWM facilities, and relative ease of sewer construction.

While Figure 8.2 shows the preferred alignment for the internal sewers, there is flexibility in the location of these sewers and the corresponding drainage boundaries. The sizing of the Dundas Street sewer has considered this flexibility, and as such, will allow the plan to evolve as it moves forward over time. The costs for local sewers are not considered to be DC recoverable.

A local or regional pumping station appears to be unnecessary.

8.2.6.3 External Drainage Areas

Two other tributary areas will connect to the Dundas Street trunk sewer at various locations. Due to their shorter lengths and smaller tributary areas they are considered to be local sewers. Together they service an approximate area of 69 ha of developable lands with an estimated equivalent population of 3,805 people. These local sewers allow the Dundas Street trunk to be kept at a nominal depth.

The adjacent Tremaine and Dundas Secondary Plan Area, City of Burlington will discharge wastewater flows to the Dundas Street Trunk system. A Secondary Plan is currently being prepared for the area to determine the preferred land use concept. According to the City of Burlington's, the Secondary Plan has been adopted by the City but is pending acceptance from the Region. At present there are 3 land use options proposed which includes approximately 56 ha of developable land. For the purpose of this study the most conservative approach was assumed to evaluate the downstream wastewater sewer. It was assumed that the entire lands would develop as residential. The wastewater flows generated by the Tremaine-Dundas Community are shown in Table 8.4.

Table 8.4 – Peak Generated Collection System Flows to Dundas Trunk Sewer: Tremaine Dundas Community

	Residential L/S	Commercial L/S	Industrial L/S	Institutional L/S	Sub Total L/S
Average Flow	6.9	0.0	0.0	0.0	6.9
Peaking Factor	3.560	3.560	3.560	3.560	3.560
Harmon Peaking Factor (K)	1.0	1.0	1.0	1.0	1.0
Infiltration	16.0	0.0	0.0	0.0	16.0
Sub Total	40.6	0.0	40.6	0.0	40.6

The Region has expressed concerns relating to the servicing of the lands on the east side of Old Bronte Road, north of Dundas. This area includes approximately 13 ha of existing residential development with an equivalent population of 715 people, and is currently serviced by septic systems. In servicing these lands in the future, it is expected that wastewater flows cannot be conveyed to the trunk sewer on Grand Oak Trail, as this would require a crossing of the NHS to the east. As such, it is proposed that a new wastewater sewer be constructed on Old Bronte Road connecting to the existing 825mm trunk sewer on Old Bronte Road north of Dundas Street West. Construction of this local sewer will have to be coordinated with the Region. The wastewater flows generated by the existing Old Bronte Road residential development are shown in Table 8.5.

Table 8.5 – Peak Generated Collection System Flows to Dundas Trunk Sewer – Existing Old Bronte Road Residential Development

	Residential L/S	Commercial L/S	Industrial L/S	Institutional L/S	Sub Total L/S
Average Flow	2.3	0.0	0.0	0.0	2.3
Peaking Factor	3.89	4.3	4.3	4.3	
K	1.0	1.0	1.0	1.0	1.0
Infiltration	4.1	0.0	0.0	0.0	4.1
Sub Total	12.9	0.0	0.0	0.0	12.9

It is expected that the Sixteen Hollow Lands to the east of the 407 West Employment Lands will be serviced by the existing wastewater sewer located at approximately Third Line and Dundas Street West and will not impact any of the sewers utilized by the 407 West Employment Lands.

8.2.7 Sewer Sizing and Technical Analysis

Flows and sewer sizes were developed using Regional design criteria. Detailed design sheets are provided in Appendix 8.1. The existing system along Colonel William Parkway from Dundas Street West to the south limit of the Bronte Creek Community has been designed to accommodate 362 ha of development with an equivalent population of 8,145. The design sheet and drainage plan for the Bronte Creek Community are available in Appendix 8.1.

The Bronte Creek Community design sheets have been used to assist with the analysis of the proposed development of the 407 West Employment Area. The proposed 407 West Employment Area and the external contributors (Tremaine Neighbourhood and the Old Bronte Road properties) that contribute to the Colonel William Parkway collect from a total area of 206 ha and an equivalent population of 18,869 people with a combined wastewater flow of approximately 197 L/s at the Colonel William Parkway and Dundas Street West manhole and 225 L/s at the south limit of the Bronte Creek Community. In contrast, the Bronte Creek Community analysis completed by Stantec shows a total flow contribution of approximately 180 L/s at the Colonel William Parkway and Dundas Street West manhole and 210 L/s at the south limit of the Bronte Creek Community. This does represent an increase of approximately 17 L/s at the Colonel William Parkway and Dundas Street West manhole, and 15 L/s at the south limit of the Bronte Creek Community; however, at no point is the capacity of any leg of sewer greater than 75%. It should also be noted that the wastewater generation values used for both the Tremaine Subdivision and the 407 West Employment Area are the highest values possible. This wastewater analysis is truly a worst-case scenario.

Conceptual Plan-Profiles of the Wastewater Sewer design are provided in Appendix 8.3.

8.2.8 Mitigation Measures for Wastewater Crossings of Watercourses and Natural Heritage

To provide a service connection to Blocks P1 and P3, it will be necessary for the wastewater sewer to cross watercourses and the NHS at two locations. Wherever possible, the wastewater sewer alignment will be kept within the proposed right-of-way (ROW) and will go over the culvert structure.

In circumstances where a bridge will be used or where going over the culvert is not possible, the wastewater sewer will be installed using a trenchless technology, such as, jack and bore or directional drilling below the watercourse or natural feature preferably within the ROW. The sewer should be installed using a steel liner or another acceptable form of protective casing, with the launching and receiving pits positioned as far as practically possible from the watercourse or natural feature. All crossings should be a minimum of 3.0 m below the watercourse or natural feature and must follow all geotechnical recommendations. This will have a carry through effect of lowering the entire wastewater sewer system and therefore, needs to be taken into consideration during the early stages of detail design.

Where the culvert or bridge structure requires piles or other deep foundations and the use of trenchless construction is not feasible within the ROW, the wastewater sewer alignment will be moved outside of the ROW to a point where it will not influence the structure foundation. The launching and receiving pits will be located as far as practically possible from the watercourse or natural feature. The alternative alignments and profiles of the proposed wastewater collection system with respect to the crossings are shown on Drawings P1-17.

Should the existing culverts along Dundas Street be replaced with bridges, Option 2 will be implemented, as indicated on Exhibit 3.6 and the Plan-Profiles in Appendix C. In Option 2, the sanitary drainage area from Avenue 5 will be directed to the existing system on Valleyridge Drive to avoid the crossing of the watercourse. The sanitary sewer along Dundas Street between Avenue 3 and Avenue 5 will be routed around the bridge structure and installed using trenchless technology with a minimum depth of 3.0 m below the existing watercourse.

The crossing of any Redside Dace habitat watercourse will require review and approval from the MNRF prior to the construction of the crossing infrastructure. Each crossing permit will have specific requirements; however, for the crossings within the 407 West Employment Lands, it should be expected that the following will be required:

- Construction will be during the permissible Redside Dace in-water construction window of July 1 to September 15;
- Construction will utilize trenchless construction methods;
- Utilities will be installed at a depth of at least 3 m below the bottom of a Redside Dace habitat watercourse;
- A specific contingency plan will need to be prepared for each crossing to address all concerns of the construction methodology proposed;
- All disturbed soils will need to be stabilized using a methodology approved by the MNRF; and
- Erosion and sediment control measures will remain in place until final restoration has been completed.

8.3 Conceptual Water Servicing Strategy

The Region's 'Sustainable Halton Water and Wastewater Master Plan' dated September 2011 set out a strategy for the long term and orderly development of the Region's infrastructure. This report was prepared in response to the new Official Plan (ROPA 38) and Phasing (ROPA 39). In the case of water, this report addressed supply, pressure districts, storage and distribution. This report also addresses timing. It provided conceptual information on the location of proposed infrastructure; however, this is subject to more detailed review when considering the servicing corridors available through the road network that is proposed as part of the Land Use Plan for the 407 West Employment Area.

This EIR/FSS report has been prepared to develop on and complement the Region's plans by providing more specific information on how it can be implemented in the context of the specific plans for the FSS Study Area. Therefore, to provide appropriate context, the Region's Plan as it relates to the FSS Study Area is summarized in this section.

8.3.1 Supply

Historically water supply for South Halton has come from three main sources, the Burlington Water Purification Plant, the Oakville Water Purification Plant, and wells within Milton (to service specific areas of Milton).

The 2002 Master Plan concluded that the long-term growth of Halton would require the construction in stages of a new water treatment plant that will have an ultimate capacity of 220 ML/d. The first stage of this new plant (Burloak) is now complete and commissioned.

This new supply is critical to meet the Region's medium and long-term growth projections for both the 407 West Employment Area and the Region as a whole.

8.3.2 Pressure Districts

The Subject Property, consistent with Section 8.2.6, is located within the Oakville pressure district identified as Zone 3 or O-3. Zone 3 in Oakville includes all lands with an elevation of 128 to 166 m. The zone boundary is generally parallel to Sixteen Mile Creek on the east, along Highway 407 to the north, along Tremaine Road to the west, and generally in between Upper Middle Road and the Q.E.W. to the south as shown on Figure 8.3.

Supply for Zone 3 is currently via a booster pumping station at Eighth Line and Upper Middle Road and the Kitchen Reservoir and Pump Station at Regional Road 25 and Upper Middle Road. Storage is provided at the Moore Reservoir on Sixth Line north of Burnhamthorpe Road (north of Dundas Street).

The Region has recently constructed a 1200 mm watermain on Dundas Street from Tremaine Road to Bronte Road. This watermain will directly supply the 407 West Employment Area. We understand that the supply to Zone 3 has been augmented via a 1200 mm watermain connection on Dundas Street from 400 m east of Bronte Road to Neyagawa Boulevard. This supply is connected to the existing Zone 3 water supply (Moore Reservoir on Sixth Line) via the existing 600 mm watermain on Dundas Street connecting to the existing Sixth Line main which links the Eighth Line Water Booster Pumping Station (WBPS) with the Moore Reservoir. The 407 West Employment Area will also receive supply from the Burlington Zone B3 via a 900 mm watermain on Dundas Street from Appleby Line to Tremaine Road.

8.3.3 Storage

Storage for Oakville Zone 3 is currently provided at the R.J. Moore Reservoir on Sixth Line. Until 2002, Zone 3 also provided the storage for Zone 4, where it was pumped to Zone 4 on an as required basis. In 2002, an elevated storage tank was constructed in Zone 4 on Trafalgar Road north of Burnhamthorpe Road.

The existing storage available in Oakville Zone 3 is sufficient for long term build-out of the FSS Study Area, as well as, all other lands serviced by Zone 3.

8.3.4 Distribution

Development in Oakville is currently serviced via a series of trunk watermains that connect sources of supply, pumping, and storage to a local distribution network.

To support growth, the Region proposes a series of new trunk watermains that interconnect with and expand the existing system and connect to the new proposed sources of supply, pumping and storage as described above. The FSS Study Area will connect to the existing Zone 3 system at Dundas Street and Bronte Road, looped along Tremaine Road and internally through the FSS Study Area to ultimately connect to the future

watermain (Regional Project #5854) in the adjacent Sixteen Hollow Lands. Water service will be distributed to the local network from the Dundas Street and the internal regional watermain.

8.3.5 Region's Timing

The infrastructure described above will be constructed on an as-required basis for each phase of development. In many instances works such as the treatment plants, storage, and pumping stations will be constructed incrementally. In the case of linear infrastructure, it will be extended incrementally to provide local service connectivity and looping.

8.3.6 Expected Water Demand

In this section, water demands under various conditions have been assessed using the design criteria that the Region has utilized in the Master Plan and supplemented with the Region's Design Criteria as required. To develop the estimated demands, the system design criteria is first set out and then applied to the proposed development statistics from Figure 3.1.

The flow demand, storage volume requirements, and pumping station capacities are similar to those used by the Region in developing the Master Plan. Any difference in the demand estimated in the Master Plan is due to an increase of undevelopable natural heritage areas throughout the 407 West Employment Area.

Table 8.6 sets out the system unit demands. Table 8.7 summarizes the Water System Design Criteria.

Table 8.6 – System Unit Demands

	Residential L/cap/day	Commercial m³/ha/day	Industrial m³/ha/day	Institutional m³/ha/day
Average Day Demand	314	26.00	17.63	10.94
Maximum Day Peaking Factor	1.9	1.9	1.9	1.9
Peak Hour Peaking Factor	3.00	3.00	3.00	3.00

Table 8.7 – Water System Design Criteria

Component	Condition/Description	Criteria
Pumping Stations	With adequate zone storage available	Maximum day flow to zone and all subsequent zones
	Without adequate storage available	The greater of peak hour flow or maximum day plus fire to the zone and the maximum day flow to all subsequent higher zones
Storage	Balancing storage	25% of maximum day demand
	Fire storage	Largest expected fire zone (based on land use)
	Total	125% of Balancing + Fire (allows for 25% Emergency Storage)

Fire flow	Minimum flow (single family residential)	5,500 L/min for 2 hours @ minimum 140 kPa (20 psi)
	Minimum flow (industrial/commercial/institutional)	15,000 L/min for 3 hours @ minimum 140 Pa (20 psi)
System pressure	Normal operating conditions	280 kPa (40 psi) to 700 kPa (100 psi)

Table 8.8 summarizes the projected demands under various conditions for the 407 West Employment Area at build-out by applying the above criteria to the development statistics described in Figure 3.1. Commercial land use demands have been applied to the entire site to provide a conservative estimate and to allow for flexibility with respect to the ultimate land use mix.

Table 8.8 – Flow Demands: Linear Infrastructure (407 West Employment Area Land Use Plan Population Projections)

	Residential (ML/d)	Commercial (ML/d)	Industrial (ML/d)	Institutional (ML/d)	Total (ML/d)
Average Day Demand	0.0	4.1	0.0	0.0	4.1
Maximum Day	0.0	7.7	0.0	0.0	7.7
Peak Hour	0.0	12.2	0.0	0.0	12.2

8.3.7 Region's Concept Plan Applied to the FSS Study Area

One of the important purposes of this report is to apply the Region's Master Plan Update water distribution concept to the approved Secondary Plan for the FSS Study Area. As stated at the outset of this Report, the Report's purpose is to adapt the Region's servicing concept to the approved Secondary Plan, not to modify it. As a result of this principle, and because the estimated demand based upon the approved Secondary Plan is similar to the demand assumed by the Region, no changes are recommended to the Region's proposed supply, pumping, or storage system network.

The development of a community plan has; however, created the opportunity, and in fact the need, for a 'plan specific' trunk water main distribution network to be developed to replace the generic one that the Region applied in the absence of a Secondary Plan.

The proposed ASP water distribution network is illustrated in Figure 8.4. To address environmental sensitivities and minimize impact, all mains are proposed to be located on existing or proposed road allowances.

The proposed ASP water distribution system is essentially the same as the distribution from the Region's Master Plan Update (MPU) with minor changes based on the outcome of land uses proposed by the approved Secondary Plan. The following key elements of the proposed distribution network that are the same as the Region's MPU water system include:

- The 1200 mm PD3 supply main on Dundas Street from Bronte Road to Tremaine Road (completed); and

- The 600 mm PD3 watermain on Tremaine Road from Dundas Street to the proposed East-West Collector through the FSS Study Area (Region Project #5853).

Changes to the network to respond to the proposed Secondary Plan and road pattern include the following minor changes to the Region's MPU.

- A slight relocation of the east-west 600 mm watermain to better match the proposed road alignment (Region Project # 5627).

Finally, to maintain required fire flows and adequate pressure during all phases of development, the local north-south watermain should connect to the 1200 mm diameter watermain on Dundas Street.

The proposed changes to the distribution system will have no change to the development charge projects.

8.3.8 Water Distribution Modeling Analysis

The Region provided a copy of the Region's Water Distribution Model dated August 12, 2008 to assist WSP in modeling the proposed Area Servicing Plan watermain system. The following recommendations are based on the update of the Region's model to include the proposed ASP watermain system shown in Figure 8.4.

8.3.9 Water Distribution Modeling Results for Peak Hour and Maximum Day

The proposed FSS Study Area system was incorporated into the Region's Water Distribution Model to determine if the proposed FSS Study Area water system would be adequate to service the FSS Study Area. Table 8.9 summarizes the results of the distribution modeling. Copies of the Peak Hour and Maximum Day model results have been included in Appendix 8.2.

Table 8.9 – Results of Water Distribution Modeling for Proposed ASP Water System

	Peak Hour	Maximum Day
Minimum HGL	190.27 m	196.00 m
Node for Minimum HGL	WJ-1152-O	NO-248
Maximum HGL	190.97 m	196.24 m
Node for Maximum HGL	NO-245, 246, 251, 252, 253 WJ-3114-O, & WJ-3116-O	NO-241, 245, 246, 251, 252, 253, WJ-1150-O, WJ3114-O, & WJ-3116-O
Minimum System Pressure (psi)	41.16 psi	48.66 psi
Node for Minimum System Pressure	NO-239	NO-239
Maximum System Pressure (psi)	62.51 psi	70.00 psi

Node for Maximum System Pressure	NO-252	NO-252
----------------------------------	--------	--------

The results of the distribution modeling show that the proposed ASP water system will provide adequate flow and pressure to all locations in the ultimate development condition. It should be noted that the maximum head loss in the proposed system is only 0.08 m during a maximum day demand and 0.18 m during a peak hour demand. The low head loss in the overall system indicates that the watermains are adequately sized and that increasing the watermain sizes from the proposed 1200/600 mm trunk watermains and the 300 mm distribution watermains is not required.

During the initial phases of development, all attempts will be made to provide full looping of the internal water distribution systems; this may require interim or temporary watermains. In cases where looping is not possible, a regular flushing program will be required at all dead ends.

8.3.10 Water Distribution Modeling Results for Maximum Day plus Fire

The proposed system was also modeled to determine if the proposed water distribution system could meet the Region's fire requirements of 5,500 L/minute for residential development and 15,000 L/minute for commercial/institutional/industrial development.

The results of the maximum day plus fire modeling indicates that the fire flow of 15,000 L/minute at a residual pressure of 20 psi is available at all nodes within the FSS Study Area. Therefore, it can be concluded that the proposed ASP water system is adequately sized for the maximum day plus fire demands. All office towers and large scale industrial buildings will likely require boosters to provide proper fire protection; this needs to be evaluated at the detailed design stage on a case-by-case basis.

8.3.11 Additional Design Considerations

8.3.11.1 Local Service Watermains

The 407 West Employment Area Land Use Plan proposes developments that front onto external roads such as Dundas Street, Tremaine Road, and Bronte Road where Regional DC watermains are proposed. These proposed developments will require water services and in some cases may require local watermains to service these developments. The FSS primarily addresses the watermain sizes for the transmission and major distribution watermains. Local distribution mains have been conceptually sized in this study but will need to be addressed in more detail through the Functional Servicing Reports supporting the various Draft Plans of Subdivision and Site Plan Applications and will be in accordance with the Region's published standards for water connections. Each proposed building will need to be evaluated at the detailed design stage to determine if a booster is required for that development to meet the water demand and fire service requirements. All service connections will be designed in accordance with the Region's published standards for water connections.

8.3.11.2 Mitigation Measures for Single Feed Watermain Supplies

The ultimate water distribution is a well-designed network of interconnected watermains with multiple loops to ensure security and flexibility in servicing the full build out of the proposed FSS Study Area and the rest of

the FSS Study Area. While it is a priority to loop systems where possible and as soon as the opportunity is available, it may be necessary to service development areas with single feed watermain during various phases of development until the future watermain loops can be constructed. The phasing will likely move to the north from Dundas Street West, which may create circumstances where there are single feed, dead-end watermain. During the interim condition of single feed watermain, a regular flushing practice will be required to maintain water quality. Autoflushers are to be utilized on all interim single feed watermain. The autoflushers will be placed in locations and programmed to ensure that adequate water quality is maintained until full looping of watermain occurs. The flushing program, including the flushing frequency and discharge locations, will be determined in conjunction with Region staff.

Wherever possible during interim conditions, interim watermain loops will be utilized. This may include the use of private watermain to close the loop. Upon completion of the full water distribution system, any private watermain used exclusively for looping will be decommissioned in accordance with Region's requirements.

During the development process, each phase will be evaluated on a case by case basis to determine how adequate water quality will be maintained in the water distribution system.

8.3.11.3 Mitigation Measures for Watermain Crossings of Watercourses and Natural Heritage

In order to provide a robust water distribution system with complete looping, it will be necessary for the watermain to cross watercourses and the NHS. Wherever possible, the watermain alignment will be kept within the proposed ROW and will go over the culvert structure or hung on a bridge structure if a bridge is utilized.

In circumstances where going over the culvert or hanging the watermain on a bridge is not possible, the watermain will be installed using a trenchless technology such as jack and bore or directional drilling below the watercourse or natural feature preferably within ROW. The watermain should be installed using a steel liner or another acceptable form of protective casing, with the launching and receiving pits positioned as far as practically possible from the watercourse or natural feature. All crossings should be a minimum of 3.0 m below the watercourse or natural feature and must follow all geotechnical recommendations.

Where the culvert or bridge structure requires piles or other deep foundations and the use of trenchless construction is not feasible within the ROW, the watermain alignment will be moved outside of the ROW to a point where it will not influence the structure foundation. The launching and receiving pits will be located as far as practically possible from the watercourse or natural feature. The alternative alignments and profiles of the proposed water distribution system with respect to the crossings are shown on Drawings P1-17.

The crossing of any Redside Dace habitat watercourse will require review and approval from the MNRF prior to the construction of the crossing infrastructure. Each crossing permit will have specific requirements from the MNRF; however for the crossings within the FSS Study Area, it should be expected that the following will be required:

- Construction will be during the permissible Redside Dace in-water construction window of July 1 to September 15;
- Construction will utilize trenchless construction methods;

- Utilities will be installed at a depth of at least 3 m below the bottom of a Redside Dace habitat watercourse;
- A specific contingency plan will need to be prepared for each crossing to address all concerns of the construction methodology proposed;
- All disturbed soils will need to be stabilized using a methodology approved by the MNRF; and
- Erosion and sediment control measures will remain in place until final restoration has been completed.

8.4 Stormwater

8.4.1 General

There are currently no storm sewers or SWM facilities to service the FSS Study Area. The existing conditions currently drain overland primarily by sheet flow into the existing watercourses onsite. These watercourses then drain offsite to the south under Dundas Street West via 3 culverts. There is currently no treatment of the stormwater from the Subject Property.

Both the minor and major storm systems have been designed to conform to the SWM Plan presented in Section 7.0.

8.4.2 Minor Storm System (Sewers)

The conceptual minor storm system has been designed to convey up to the 1:5 year storm event to the appropriate SWM facility. The intention of the minor storm system is to match as closely as possible with the existing drainage boundaries of each of the existing culverts. The minor storm system will consist of gravity sewers that will discharge to a SWM facility for treatment. The majority of the storm sewers are within the conceptual road network with some of the sewers discharging to the SWM facility within an easement on a development block.

The conceptual minor storm system is shown on Figure 8.5. The proposed storm sewer system has been designed to the current Town's design criteria. Please note that the storm drainage illustrated to the east of Avenue 3 within Subcatchments FM108, FM1109, FM1110 and FM1110.1 on Figure 8.5 is conceptual only and had not been evaluated by the review agencies with respect to feasibility and NHS impacts and as such, is subject to change. The storm design sheets are available in Appendix 8.3. The Plan-Profiles of the storm design are available in Appendix 8.4.

8.4.3 Major Storm System (Overland Flow)

The proposed major storm system is in accordance with SWM plan as described in Section 7.0 of this report. The overland flow route will convey flows up to the 1:100 year storm event to the SWM facility containing the major flows within the road ROW. The overland flow route is shown on Figure 8.5.

8.5 Grading

The existing topography of the FSS Study Area generally slopes from north to south and towards the centre of the lands approximately 180 m to the east of Colonel William Parkway at Dundas Street West. The general

philosophy of the conceptual grading plan is to match existing grades as closely as possible while still maintaining necessary elements of the SWM Plan in Section 7.0. Due to the relatively shallow bedrock in some locations it may be advantageous to increase the height of fill in some areas to provide less complicated construction of the municipal servicing.

The conceptual road grades all meet the Town of Oakville criteria, all road grades fall between 0.5% and 3.5%. The conceptual lot grading will generally slope towards the right of way so that the stormwater can be treated in the stormwater management facilities. Both the conceptual road grades and lot grading will be further refined as the development goes through the subdivision and site plan process. Final road and lot grading will be completed in accordance with the Town of Oakville's grading criteria. As noted in Section 4.0, at the detailed design stage, landscaped areas adjacent to natural features will be graded towards these features wherever possible.

The conceptual grading plan is illustrated in Figure 8.6. Please note that the grading illustrated to the east of Avenue Three within Subcatchments FM-1108, FM-1109, FM-1110 and FM-1110.1 on Figure 8.5 is conceptual only and had not been evaluated by the review agencies with respect to feasibility and Natural Heritage System impacts and as such is subject to change.

8.6 Phasing

Consideration has been given to the incremental installation of municipal services to coincide with the proposed phasing strategy. This may require the construction of temporary services within easements or temporary ROWs in the event that services cannot be installed in their ultimate alignments (i.e., on the lands of other owners). As noted previously, to ensure security and redundancy in the water distribution system, temporary private watermains may be required to provide looping. Conceptual phased servicing plans are illustrated in Figure 8.7.1, 8.7.2 and 8.7.3. Similarly, the site grading will be performed on a phased basis to coincide with the areas to be developed. Road crossings of the NHS systems will be installed only when required to service development. Conceptual phased grading plans are illustrated in Figure 8.8.1, 8.8.2 and 8.8.3. Further consideration for the phased construction of services and grading will be completed at the time of development of each phase.

8.7 Summary

The Municipal Servicing Section of the EIR/FSS provides a conceptual design of the wastewater, water and storm servicing, the road and lot grading and the overland flow route associated with the development of the FSS Study Area.

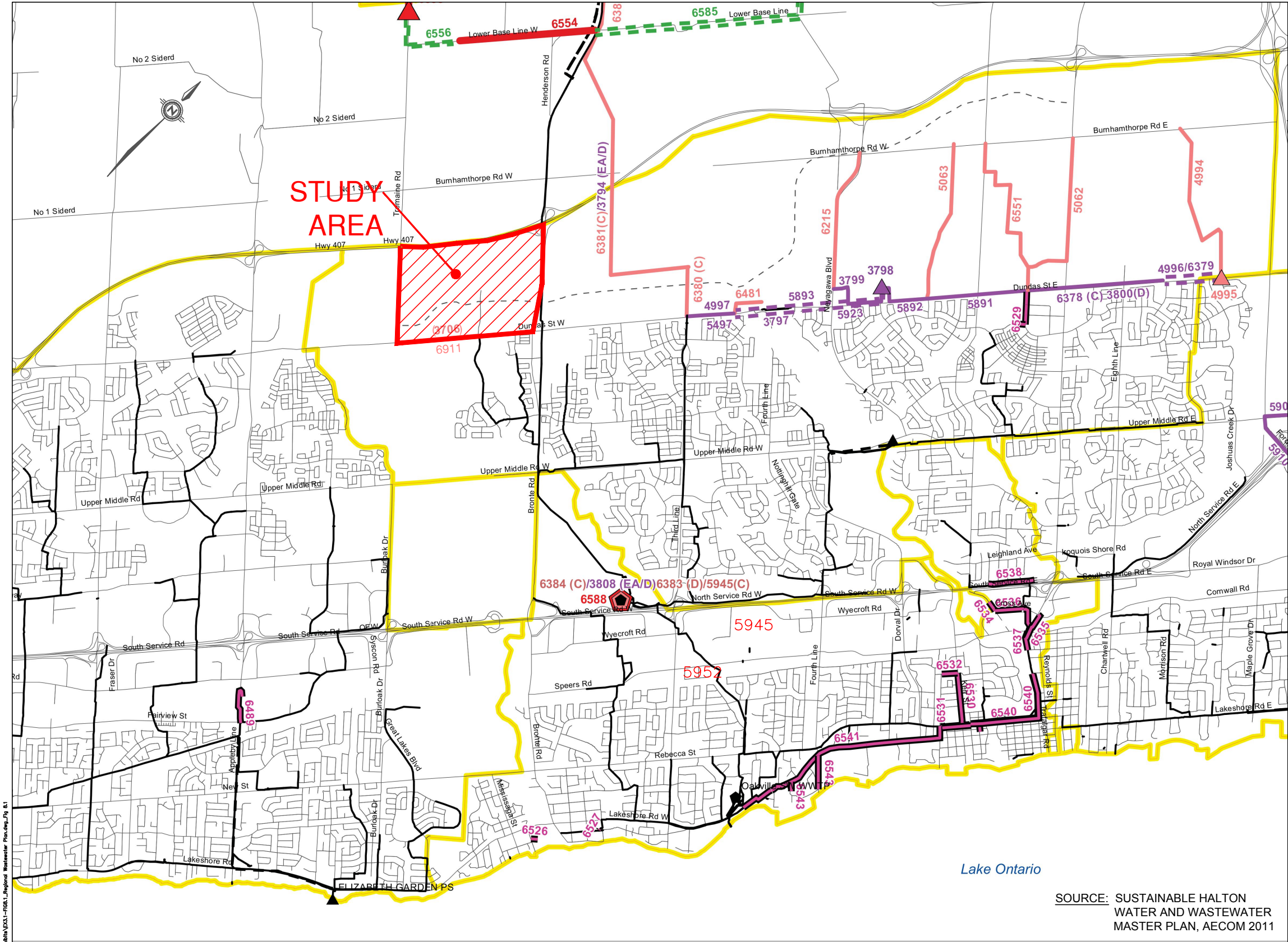
The wastewater design will incorporate a gravity sewer system within the conceptual road network that generally drains north to south and outlets into the proposed Dundas Street Trunk Sewer which in turn discharges to the existing wastewater sewer on Colonel William Parkway. The Wastewater design sheets are available in Appendix 8.1. The conceptual wastewater servicing design is illustrated on Figure 8.2.

The water distribution system will consist of a network of local and trunk watermains within the conceptual road network. A water distribution model of the proposed watermain was completed to size the watermains within the FSS Study Area. The results of the water model for the FSS Study Area are available in Appendix 8.2. The conceptual water servicing design is illustrated on Figure 8.4.

The minor storm system will consist of gravity sewers within the conceptual road network that will discharge to a SWM facility for treatment based on the catchment areas indicated in Section 7.0. The major storm system will convey the major storm flows via an overland flow route along the road ROWs to the designated SWM facility. The storm design sheets are available in Appendix 8.3. The conceptual minor and major storm system design are illustrated on Figure 8.5.

The conceptual road and lot grading is designed with the intention of matching existing grades as closely as possible while still maintaining necessary elements of the SWM Plan detailed in Section 7.0. The conceptual grading plan is illustrated on Figure 8.6.

The overall servicing and grading design has considered the proposed phasing of the development. Municipal services, roads and grading works will be installed incrementally as needed to service development. Conceptual phasing of the servicing and grading are illustrated on Figures 8.7.1, 8.7.2, 8.7.3, 8.8.1, 8.8.2 and 8.8.3.



Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

REGIONAL WASTEWATER PLAN

Legend

Existing Infrastructure

- ▲ Wastewater Pumping Station
- ◆ Wastewater Treatment Plant
- Wastewater Main
- - - Wastewater Forcemain

Previously Approved Infrastructure

- ▲ Wastewater Pumping Station
- ◆ Wastewater Treatment Plant
- Wastewater Main
- Distribution-Built Boundary Wastewater main

Proposed/Upgrade Infrastructure

- ▲ Wastewater Pumping Station
- ◆ Wastewater Treatment Plant
- Proposed Alignment - Wastewater Main
- - - Wastewater Forcemain

Funded Infrastructure

- ▲ Wastewater Pumping Station
- ◆ Wastewater Treatment Plant
- Wastewater Main
- - - Wastewater Forcemain

- WWTP Drainage Area
- ▨ Proposed Lake Base Service Area

Scale		NTS	
Client		Prepared by	
Date		Project No.	
APRIL 2019		1409222.001	
		FIGURE 8.1	

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

WASTEWATER DRAINAGE PLAN

LEGEND

SANITARY SEWER

SANITARY MANHOLE

SANITARY DRAINAGE BOUNDARY

ALTERNATIVE SANITARY SEWER ALIGNMENT

EXTERNAL SANITARY SEWER (B.O.)

EXISTING SANITARY SEWER

39.40

4533

TRIBUTARY AREA (Ha)

TRIBUTARY POPULATION

13.0

715

EXTERNAL TRIBUTARY AREA (Ha)

EXTERNAL TRIBUTARY POPULATION

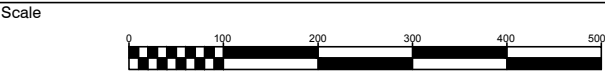
FUTURE CULVERT CROSSING

407 WEST EMPLOYMENT AREA

PROPERTY BOUNDARIES

NOTES:

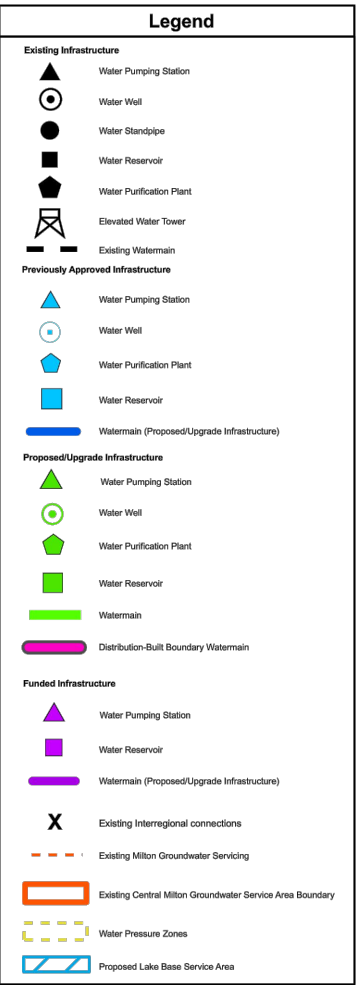
- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
- Right-of-way requirements for future 407 Transitway to be determined
- The Natural Heritage System on lands owned by others is conceptual, as shown in the North Oakville West Secondary Plan, and is subject to further study.



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	FIGURE 8.2

FILE NAME: X:\DIV10\14-09222\MUN\ASP-FSS Exhibits\EX3.6-Fig8.2_Wastewater Drainage Plan.dwg_Fig 8.2

REGIONAL WATER PLAN



NTS



QuadReal



APRIL 2019

1409222.001

SOURCE: SUSTAINABLE HALTON
WATER AND WASTEWATER
MASTER PLAN, AECOM 2013

FIGURE 8.3

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

WATER DISTRIBUTION PLAN

LEGEND

- EXISTING REGIONAL WATERMAIN
- PROPOSED REGIONAL WATERMAIN
- PROPOSED LOCAL WATERMAIN
- FUTURE CULVERT CROSSING
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES

NOTES:

- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
- Right-of-way requirements for future 407 Transitway to be determined
- The Natural Heritage System on lands owned by others is conceptual, as shown in the North Oakville West Secondary Plan, and is subject to further study.



Date

APRIL 2019

Project No.

1409222.001

FIGURE 8.4

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

STORMWATER DRAINAGE PLAN

LEGEND

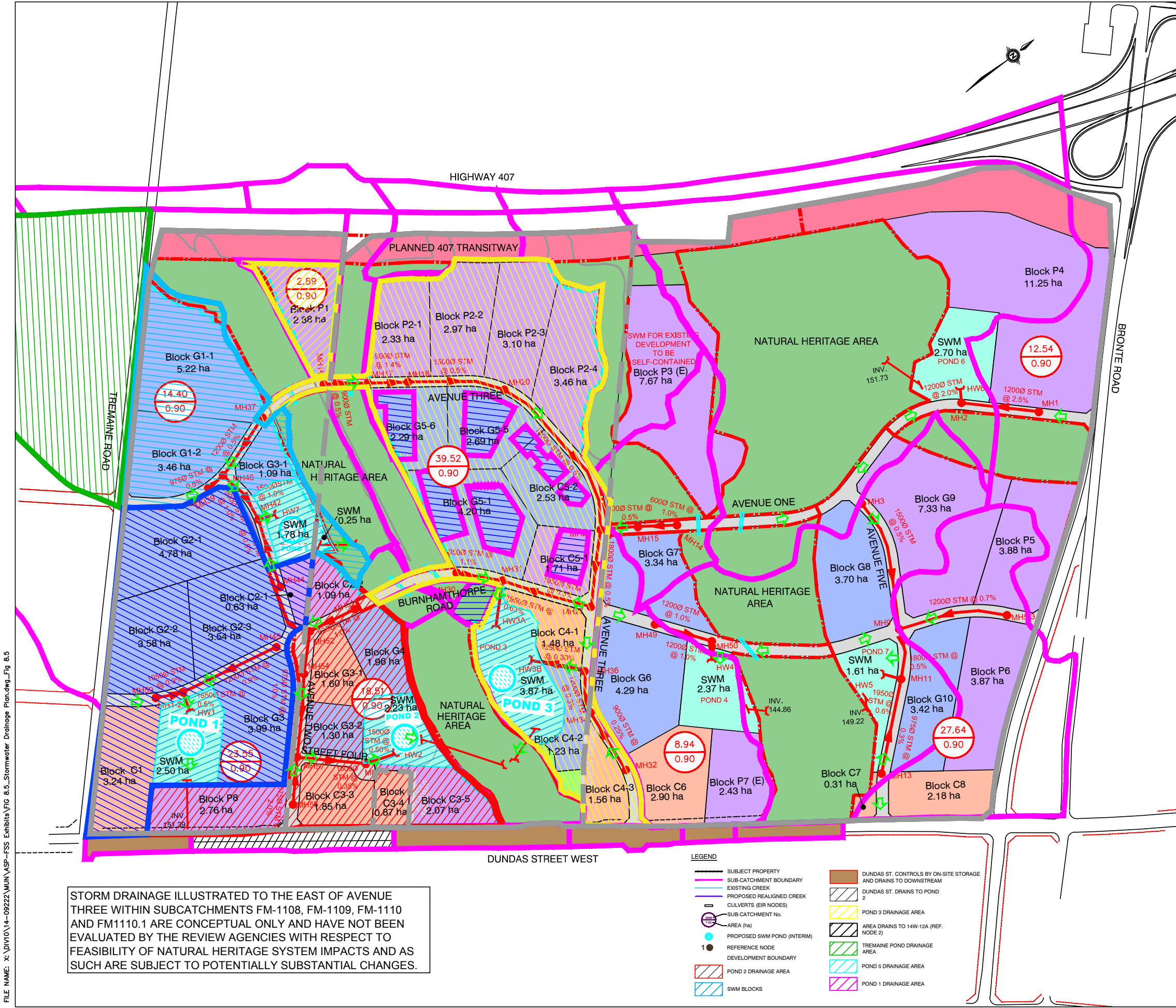
- STORM SEWER
- STORM MANHOLE
- STORM DRAINAGE BOUNDARY
- OVERLAND FLOW ROUTE
- TRIBUTARY AREA (Ha)
- RUN-OFF CO-EFFICIENT
- FUTURE CULVERT CROSSING
- PROPOSED INFILTRATION SWALE
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES

NOTES:

- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
- Right-of-way requirements for future 407 Transitway to be determined
- The Natural Heritage System on lands owned by others is conceptual, as shown in the North Oakville West Secondary Plan, and is subject to further study.



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Date	Project No.
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	FIGURE 8.5

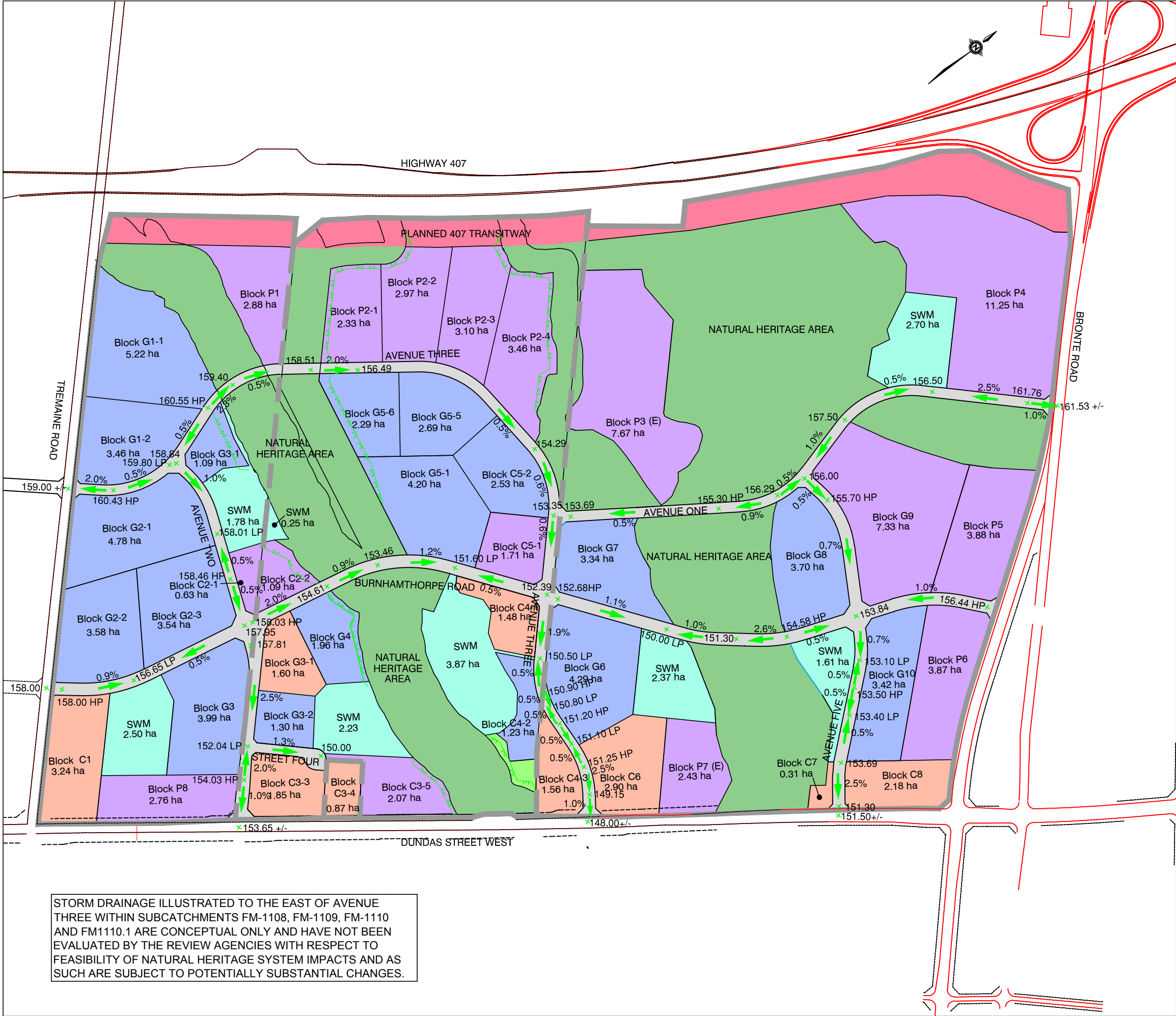


FILE NAME: x:\010\14-09222\MUN\ASP-FSS Exhibits\Fig 8.5_Stormwater Drainage Plan.dwg_Fig 8.5

STORM DRAINAGE ILLUSTRATED TO THE EAST OF AVENUE THREE WITHIN SUBCATCHMENTS FM-1108, FM-1109, FM-1110 AND FM1110.1 ARE CONCEPTUAL ONLY AND HAVE NOT BEEN EVALUATED BY THE REVIEW AGENCIES WITH RESPECT TO FEASIBILITY OF NATURAL HERITAGE SYSTEM IMPACTS AND AS SUCH ARE SUBJECT TO POTENTIALLY SUBSTANTIAL CHANGES.

Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

CONCEPTUAL GRADING PLAN

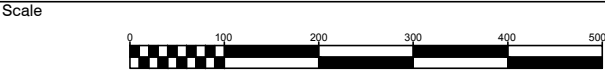


LEGEND

- 2.0% PROPOSED ROAD GRADE
- 240.70 PROPOSED ROAD ELEVATION
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- 3:1 SLOPING

NOTES:

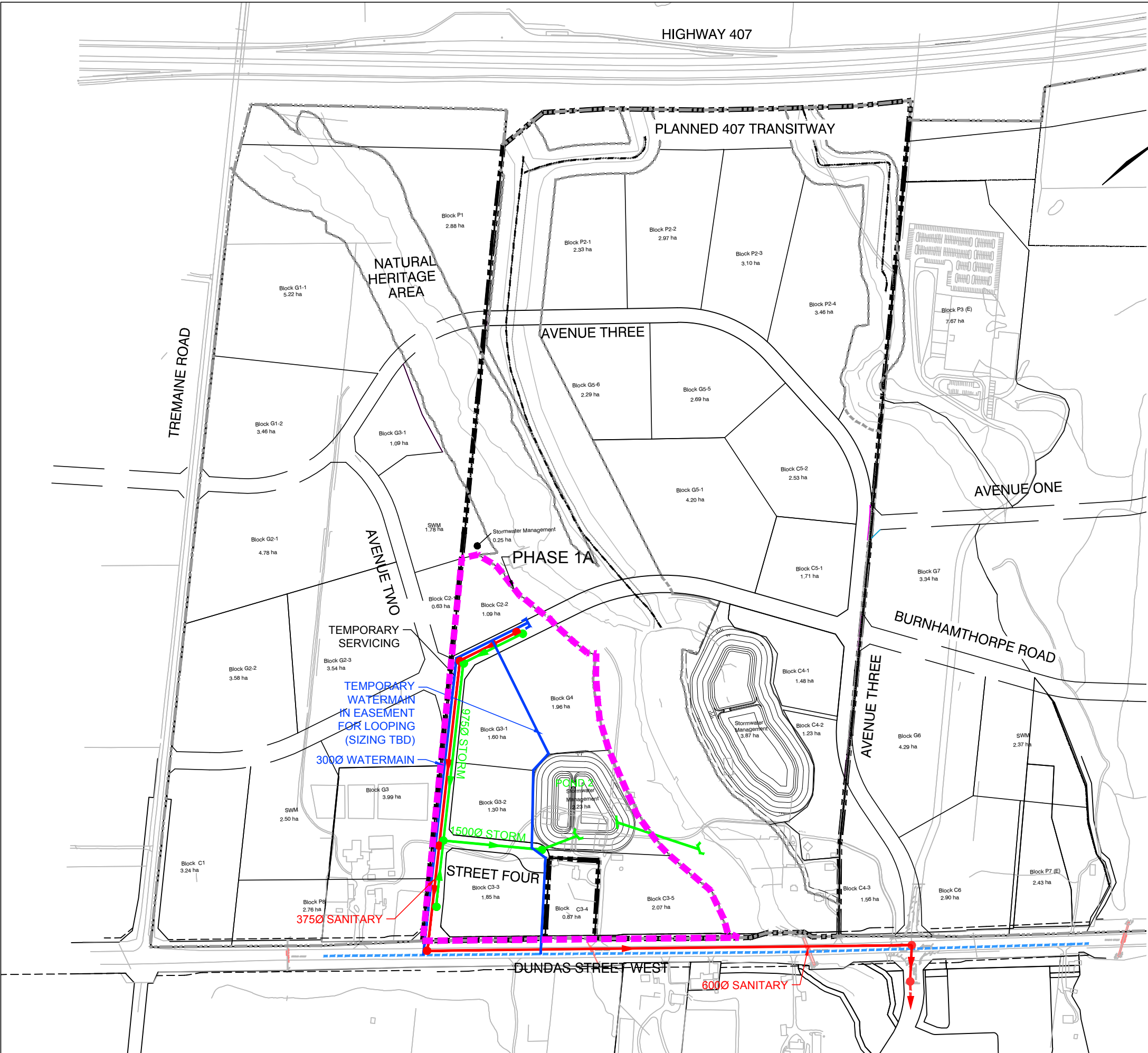
- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
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FIGURE 8.6	

STORM DRAINAGE ILLUSTRATED TO THE EAST OF AVENUE THREE WITHIN SUBCATCHMENTS FM-1108, FM-1109, FM-1110 AND FM1110.1 ARE CONCEPTUAL ONLY AND HAVE NOT BEEN EVALUATED BY THE REVIEW AGENCIES WITH RESPECT TO FEASIBILITY OF NATURAL HERITAGE SYSTEM IMPACTS AND AS SUCH ARE SUBJECT TO POTENTIALLY SUBSTANTIAL CHANGES.

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Environmental Implementation
Report / Functional Servicing Study
for 14 Mile Creek West and the Lazy
Pat Farm Property

INTERIM SERVICING PLAN
PHASE 1A

LEGEND

- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER
- PROPOSED STORM SEWER
- EXISTING STORM SEWER
- PROPOSED WATERMAIN
- EXISTING WATERMAIN
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- PHASE LIMITS

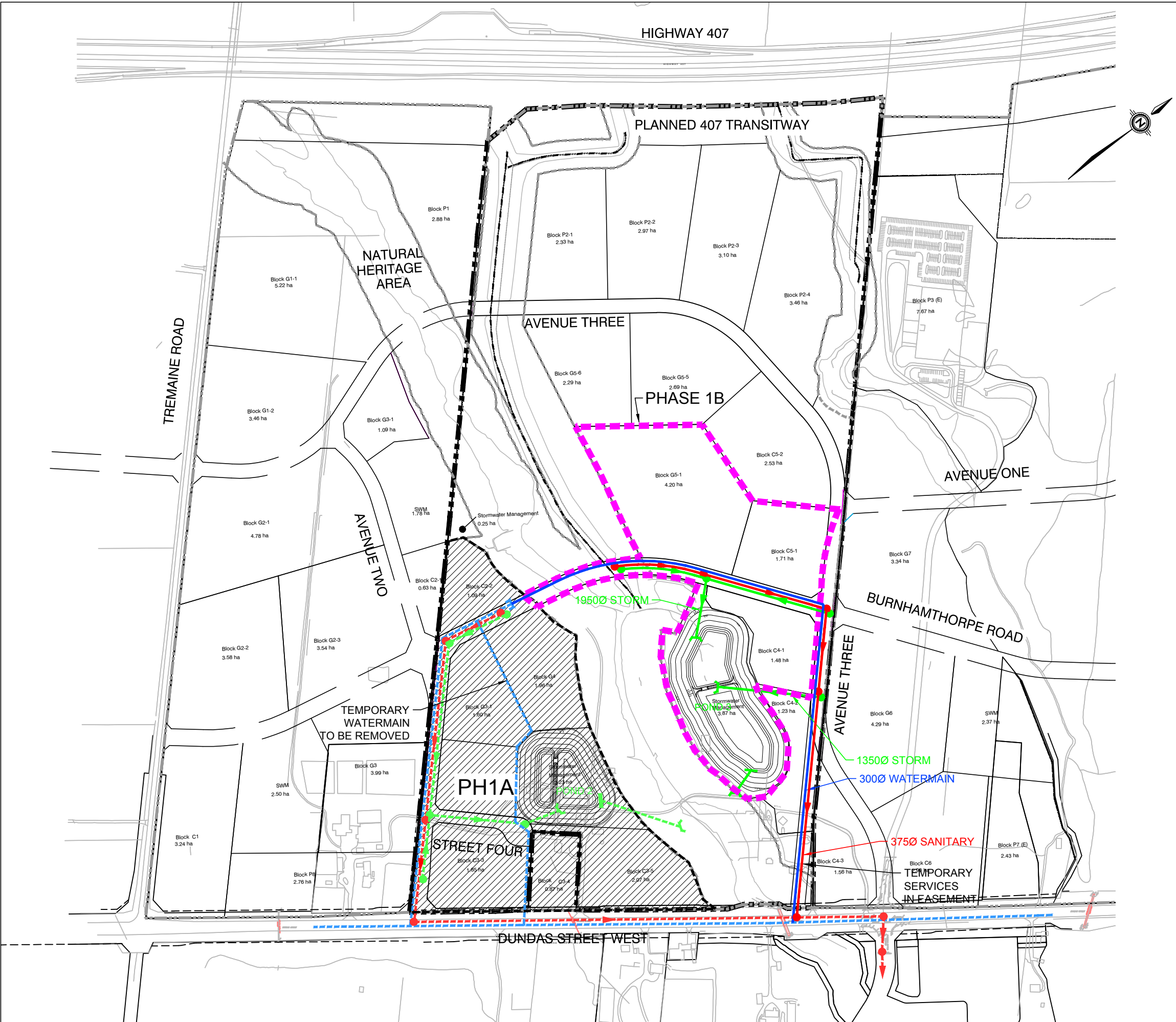
NOTES:

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	FIGURE 8.7.1

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Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

INTERIM SERVICING PLAN PHASE 1B

LEGEND

- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER
- PROPOSED STORM SEWER
- EXISTING STORM SEWER
- PROPOSED WATERMAIN
- EXISTING WATERMAIN
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- PHASE LIMITS

NOTES:

- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
- Right-of-way requirements for future 407 Transitway to be determined
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Scale

0 100 200 300 400 500m

Client

Prepared by

QuadReal

WSP

Date

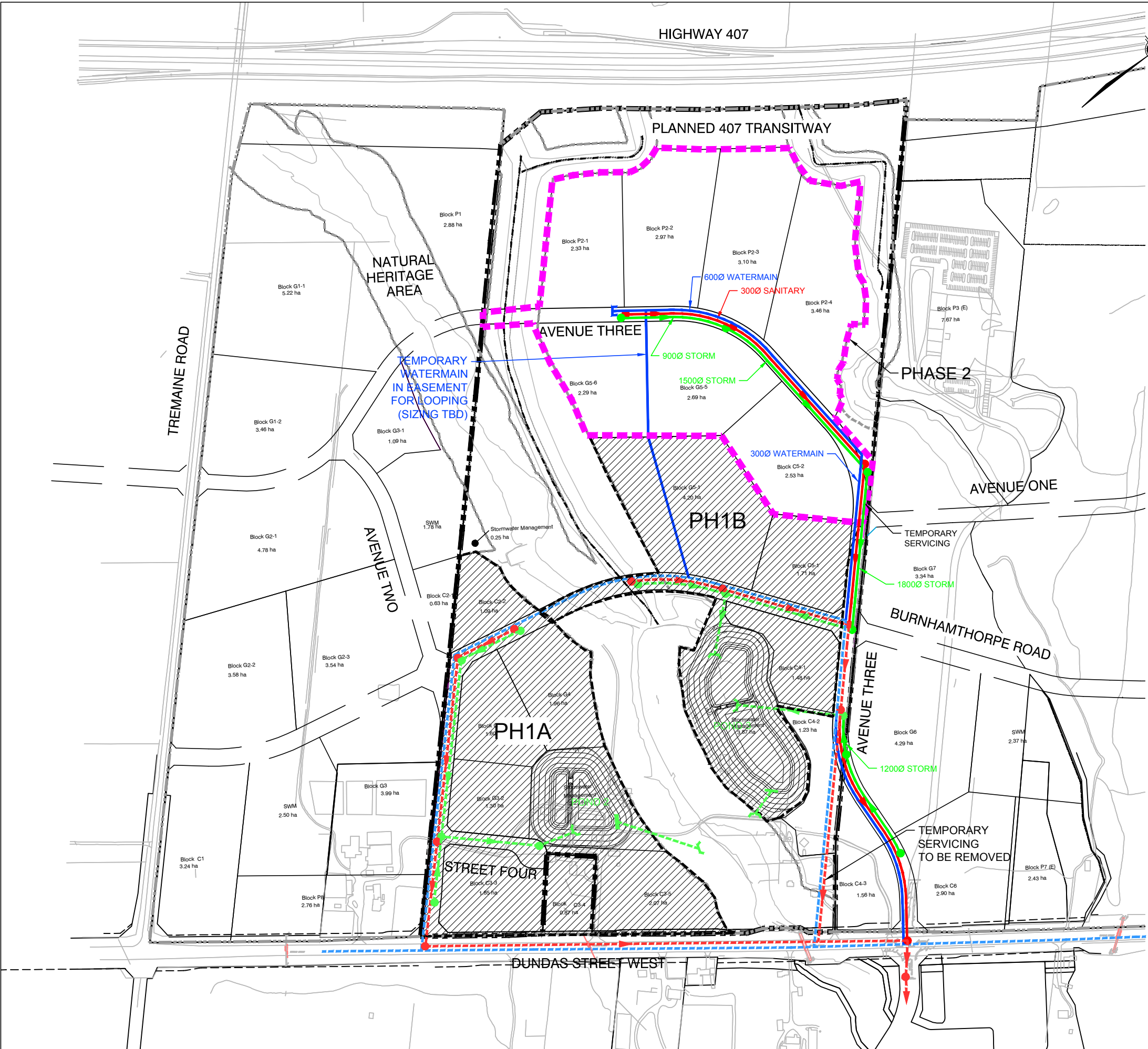
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FIGURE 8.7.2

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Environmental Implementation Report / Functional Servicing Study for 14 Mile Creek West and the Lazy Pat Farm Property

INTERIM SERVICING PLAN PHASE 2

LEGEND

- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER
- PROPOSED STORM SEWER
- EXISTING STORM SEWER
- PROPOSED WATERMAIN
- EXISTING WATERMAIN
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- PHASE LIMITS

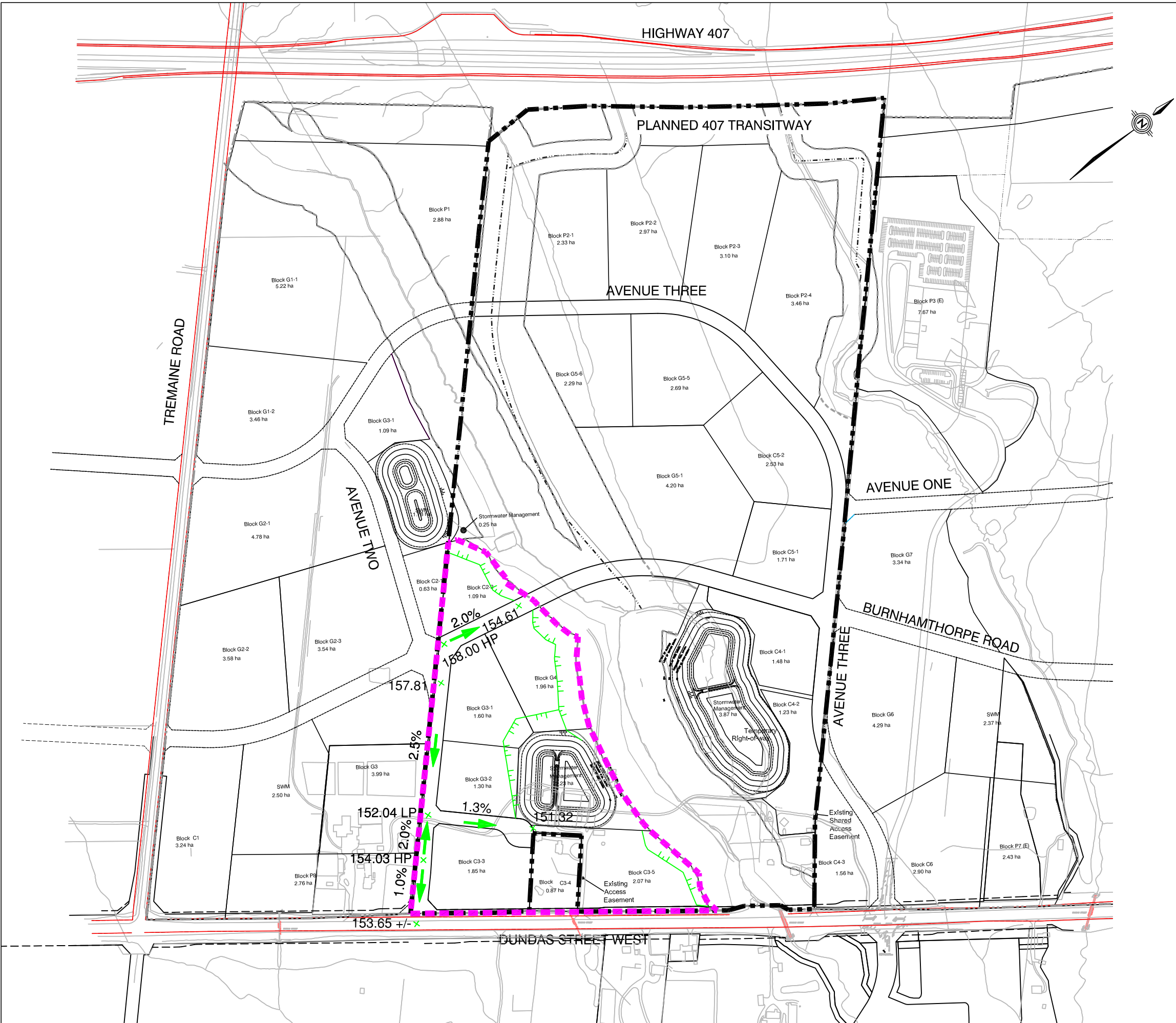
NOTES:

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Client	Prepared by
QuadReal	WSP
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	FIGURE 8.7.3

FILE NAME: x:\DIV\014-09222\MUN\ASP-FSS Exhibits\09022_Fig 8.8.1_2_3.dwg_Fig 8.8.1



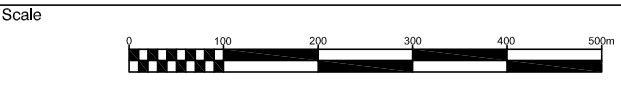
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INTERIM GRADING PLAN PHASE 1A

LEGEND

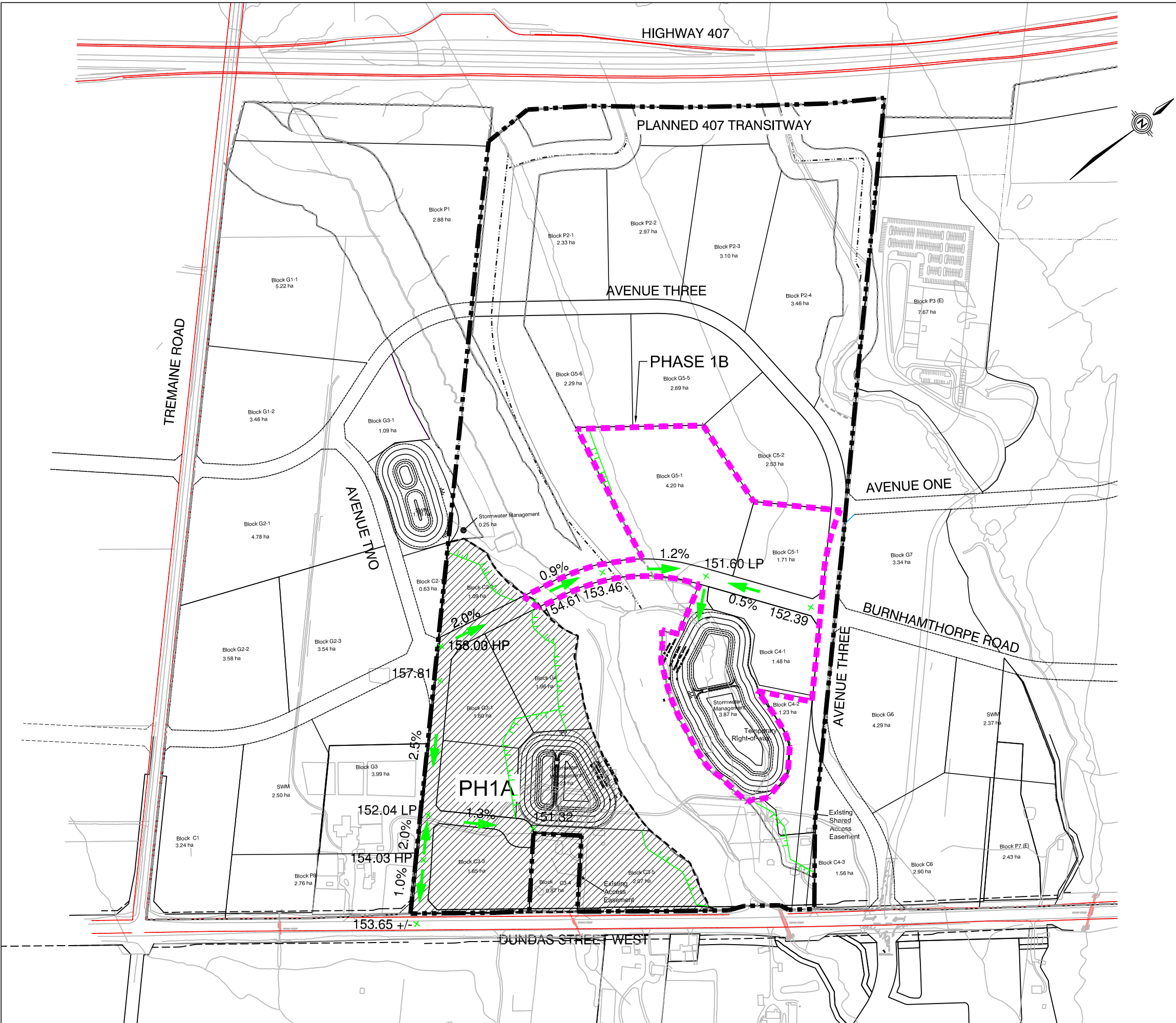
- 2.0% PROPOSED ROAD GRADE
- 240.70 PROPOSED ROAD ELEVATION
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- 3:1 SLOPING

- NOTES:
- For the purposes of the analysis land use, natural heritage and storm water sizing and location assumptions have been made for the entire 407 West Employment Area
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FIGURE 8.8.1	

FILE NAME: X:\DIV\014-09222\MUN\ASP-FSS Exhibits\09022_Fig 8.8.1_2_3.dwg_Fig 8.8.2



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INTERIM GRADING PLAN PHASE 1B

LEGEND

- 2.0% PROPOSED ROAD GRADE
- 240.70 PROPOSED ROAD ELEVATION
- 407 WEST EMPLOYMENT AREA
- PROPERTY BOUNDARIES
- 3:1 SLOPING

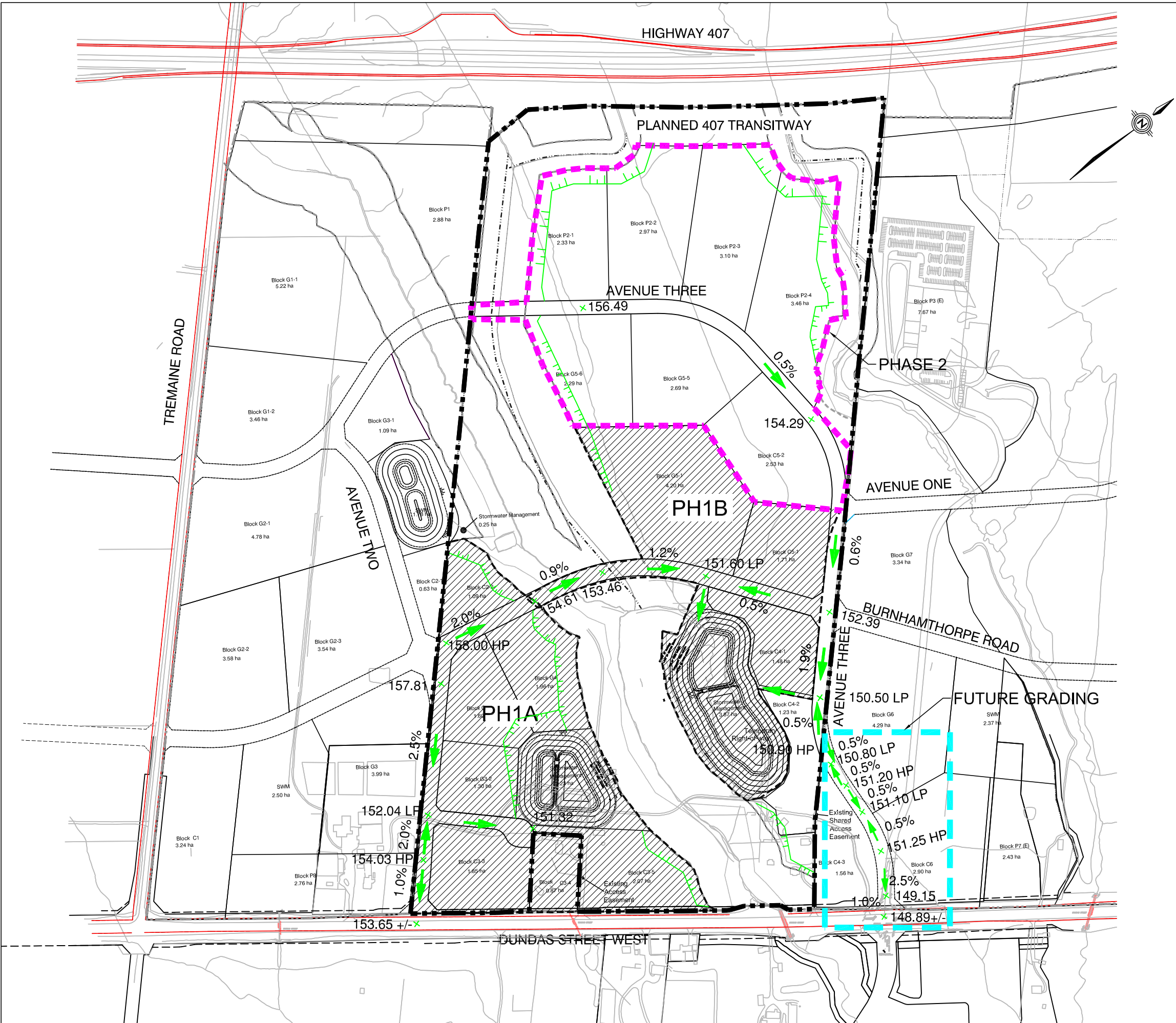
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FIGURE 8.8.2	

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INTERIM GRADING PLAN PHASE 2

- LEGEND**
- 2.0% PROPOSED ROAD GRADE
 - 240.70 PROPOSED ROAD ELEVATION
 - 407 WEST EMPLOYMENT AREA
 - PROPERTY BOUNDARIES
 - 3:1 SLOPING

NOTES:

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	FIGURE 8.8.3