REPORT

Ninth Line Stormwater Management Report

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1.0 INTRODUCTION

In April 2016, Golder Associated Ltd. (“Golder”) was retained as a sub consultant to CIMA+ for the Halton Region (“Region”) to prepare a stormwater management report in support of the “P-639-15 Class Environmental Assessment Study for Ninth Line (Regional Road 13) Transportation Corridor Improvements From Dundas Street (Regional Road 5) to 407 ETR (Express Toll Route)“.

The proposed work involves the widening of the existing 3.8 km long section of Ninth Line (“the Site”) from a 2-lane rural road to a 4 lane urban road section with a turning lane (where required), bike lanes, and multi-use pathways (Figure 1).

The preferred alternative for the roadway widening includes a right of way between 24 m and 37 m wide, with road crossing culverts, Low Impact Development features for managing water quality and quantity, a storm sewer system to convey RIGHT OF WAY flows, and a ditching system to convey external flows.

2.0 EXISTING CONDITIONS

The existing stormwater controls for the Ninth Line corridor are discussed below. This section includes the general layout of the road, the adjacent drainage ditches and stormwater management systems, as well as the exiting road crossing culverts and estimates of their ability to convey estimated target flowrates.

2.1 Road Drainage

The Site is located along Ninth Line in the Region of Halton, between Dundas Street and the 407 ETR interchange (Figure 1). Starting at the Dundas Street intersection and moving north:

- At the Dundas Street intersection, Ninth Line is a 4-lane urban cross section with a concrete median and two additional turning lanes from the southbound Ninth Line, with catch basins along the curb and ditch inlet catch basins to either site of the road.
- 200 m north of the Dundas Street intersection, Ninth Line narrows to a 2-lane rural section with a gravel shoulder and ditches on both sides of the road.
- At Burnhamthorpe Road, 2.1 km north of the Dundas Street intersection, the existing intersection is being moved 100 m southwest and converted to a two-lane roundabout as part of the William Halton Parkway extension. As part of the roundabout construction (scheduled for completion in late 2019), approximately 200m of Ninth Line north and south of the roundabout will be widened to 4 lanes, connecting back to the existing road alignment. There is a proposed system of catch basins at the roundabout, draining to the existing crossing 150 m southwest along Burnhamthorpe Road.
- North of Burnhamthorpe Road, Ninth Line crosses the 407 ETR with a series of two overpasses and two underpasses. At these locations, the ditching to either side is reduced or eliminated, and gravel shoulder is reduced (for underpasses) or eliminated (on overpasses). There are no deck drains on the overpasses, and runoff drains along the concrete barriers on either side of the road to the edge of the overpass. Drainage in this section is to 407 ETR SWM ponds at the intersection (either directly or through crossing culverts).

Runoff from the existing road right of way generally flows uncontrolled into roadside ditches, which in turn convey the flow to nearby road crossing culverts.

2.2 Road Crossing Culverts

There is a total of twelve road crossing culverts at the Site (Figure 2), with culvert sizes and materials listed in Table 1 below. The majority of culverts are corrugated steel pipes (CSP) while two are concrete box culverts. Culvert information is generally taken from the topographic survey provided by CIMA+; in three cases (CC#1, CC#1.5 and CC#3.5), some information for the culverts was not shown on the survey and the culvert hydraulics could not be assessed.

Table 1: Existing Crossing Culverts

Culvert ID#	Culvert Size and Material	Current Length (m)
CC#1	Triple Barrel CSP	-
CC#1.5	4.0 m Wide Concrete Box	-
CC#2	1600 mm CSP	75
CC#3	800 mm CSP	29
CC#3.5	500 mm CSP	-
CC#4	750 mm CSP	24.3
CC#5	700 mm CSP	25.0
CC#6	400 mm CSP	14.8
CC#7	600 mm CSP	15.3
CC#8	1200 mm CSP	18.3
CC#9	1.9 m x 1.1 m Concrete Box[1]	17.9
CC#10	500 mm CSP	14.7

[1] Culvert height measured from obvert to top of sediment at culvert downstream end.

2.2.1 Hydrology

The Region provided GIS catchment areas for the existing crossing culverts, which are shown on Figure 2. The catchments provided do not include drainage to the culverts at the 407 ETR. The remaining areas (CC#4 through CC#10) drain to the same catchment, which is a tributary of Joshua Creek.

Flow in this catchment

- originates near the 407 ETR;
- crosses Ninth Line a first time at CC#4;
- flows through a swale before crossing Burnhamthorpe Road to the southwest of the Ninth Line / Burnhamthorpe Road intersection;

- passes through the wetland west of Ninth Line before flowing into the roadside ditch west of the road in front of the Fern Hill school;
- crosses Ninth Line a second time at CC#8;
- flows across the fields east of the road before emptying into a defined floodplain (becoming the Joshua Creek tributary at this point); and,
- crossing Ninth Line a third time through the CC#9 concrete box culvert and flows south across the cemetery property towards Joshua Creek.

As regional road, road crossing culverts are required to be able to pass the peak runoff from both the 1:100 year and Regional storm events without overtopping the road. Peak runoff rates for each road crossing culvert were estimated using either the Rational method or EPA SWMM5 peak runoff values.

Rational Method:

Peak flow using the rational method were estimated using the formula:

$Q = 2.78 / 1000 \times C \times i \times A$

Where:

Q is the peak flow (m^3/s)

C is the runoff coefficient (unitless) for the 1:100yr storm, assumed from Design Chart 1.07 in the Ontario Ministry of Transportation "Drainage Management Manual" (MTO, 1997), conservatively assuming a silty clay soil and adding 25% for the 1:100yr storm

i is the peak rainfall intensity (mm/hr), assumed using:

- the rainfall intensity formula in the Town of Oakville "Development Engineering Procedures and Guidelines" (Oakville, 2012) for 1:100yr storm using a Bramsby-Williams estimate for time to peak (minimum of 5 min); or,
- the 53 mm/hr peak rainfall from Design Chart 1.03 of the MTO Drainage Manual (MTO, 1997) for the Regional Storm.

A is the catchment area (ha) as determined using the Region catchment map.

EPA SWMM5 Peak Runoff:

The United States Environmental Protection Agency (EPA) Storm Water Management Model Version 5 (SWMM 5) is a dynamic rainfall-runoff-subsurface runoff model that provides single event or continuous simulation of surface and/or subsurface runoff quantity and quality. The model also provides a method to estimate changes in hydrographs as a result of the flood wave flowing along a channel (such as delays in the peak flow arrival time based on the velocity and the length of the channel). This feature of the model is particularly important to the project, given that upstream flows for the watercourse crossing locations at CC#4, CC#5, CC#6, and CC#7 directly contribute to the corresponding flows for the watercourse crossing locations at CC#8 and CC#9 (CC#10 meanwhile is parallel to CC#9 and does not contribute to either CC#8 or CC#9 in the existing condition).

The EPA SWMM 5 model for the project was developed with consideration of the following information and assumptions:

- Distribution and total rainfall depth of 98.1 mm over 24 hours were assumed for the 1:100-year flow event based on the City of Oakville IDF curve information;
- Distribution and total rainfall depth of 212 mm over 12 hours were selected for the Regional Storm (Hurricane Hazel) based on the literature from the Ontario Ministry of Transport (MTO, 1997);
- Catchments were delineated by soil type, slope and land use using available mapping from MNRF, aerial imagery from Google Earth, and the future development plans for the project, noting that the following:
  - Soil types for undeveloped areas were assumed to be clayey silt, with a selected US Soil Conservation Service (SCS) curve numbers of 81 to 90 for 1:100-year AMCII 'normal' conditions and 91 to 96 for Regional AMCIII 'saturated ground' conditions;
  - Paved areas were assumed to be impervious; and
  - Land slopes for all areas were assumed to be between 1.0% and 2.5%.
  - Subcatchment widths were assumed as the length of the roadside ditch fronting each subcatchment along Ninth Line for crossing culverts CC#4, CC#5, CC#6, CC#7, and CC#9 (ranging from 200m to 430m), and the approximate length of the creeks within the catchment for CC#8 and CC#9 (500m and 550m, respectively)
- Culverts and weirs (simulating road overtopping) were entered based on survey results
- Channel connections between the relevant watercourse crossing locations were modelled based on the defined drainage pattern of the tributary system, noting that channels were assumed to be trapezoidal in shape, with 1 m bottom width and, 3:1 side slopes.

The EPA SWMM 5 estimated flows under 1:100 year and Regional Storm conditions were then compared to the associated flow estimates from the draft stormwater management report.

Peak Flow Results

The resulting peak runoff rate estimates are shown in Table 1 below, with the peak flow for each crossing (Rational Method and EPA SWMM5, 1:100yr and Regional Storm) highlighted.

Table 2: Estimated Peak Flows at Watercourse Crossing Locations based on EPA SWMM5 and Rational Method

Watercourse Crossing and Culvert ID	Catchment Area (ha)	EPA SWMM 5 Estimated Peak Flows from Supplementary Analyses		Rational Method Estimated Peak Flows from Draft Stormwater Management Report	
		1:100yr Peak Flow (m^3/s)	Regional Storm Peak Flow (m^3/s)	1:100yr Peak Flow (m^3/s)	Regional Storm Peak Flow (m^3/s)
		Values in Bold Denote Highest Flow Rate for the Selected Methods and Event			
CC#4	3.0	0.90	0.44	**2.16**	0.41
CC#5	3.4	0.99	0.50	**2.45**	0.47
CC#6	4.4	0.64	0.61	**1.7**	0.45
CC#7	8.4	1.29	2.71	**3.32**	0.86
CC#8	78.2	3.53	**8.96**	7.98	8.17
CC#9	119.7	5.56	**12.95**	10.67	7.98 (1)
CC#10	5.8	0.52	0.75	**3.14**	0.59

(1) Peak flow information for Regional Storm conditions at CC#9 were obtain from the Town of Oakville model.

A comparison of the EPA SWMM 5 peak flow estimates relative to the corresponding estimates from the draft stormwater management report showed the following:

- EPA SWMM 5 estimated flows under Regional Storm conditions generated the highest flow rates for CC#8 and CC#9 (i.e., 62% higher than the associated flows from the Town of Oakville model in the case of CC#9); and,
- Rational method estimated flows under 1:100-year conditions generated the highest flow rates for CC#4, CC#5, CC#6, CC#7, and CC#10 (i.e., as much as 6 times higher than the corresponding EPA SWMM 5 estimated flows).

Based on these results, and, in the interest of advancing a conservative design for the crossing structures, the following approach will be used to update the hydrology and hydraulic assessment for the draft stormwater management report:

- The rational method estimated flows will be relied on for assessing capacity for CC#4, CC#5, CC#6, CC#7, and CC#10; and
- The EPA SWMM 5 estimated flows will be relied on for assessing capacity for CC#8 and CC#9.

2.2.2 Hydraulics

The hydraulic characteristics of the existing culverts were estimated using HEC-RAS hydraulic models.

- In the case of CC#9, the existing HEC-RAS model for Joshua Creek (which includes the CC#9 culvert) was used. This included updating the model based on the topographic survey results as well as the measured channel and floodplain cross sections taken during the fluvial geomorphology fieldwork.
- For the remaining culverts CC#4 through CC#8, and CC#10, individual culvert models were created in HEC-RAS using available topographic survey results and assumptions concerning downstream channels and boundary conditions.
- The roadway was entered as broad spillways at the elevation of the surveyed centerline, allowing water to flow over the road if the centerline elevation was exceeded.

The results, shown in Table 3 below, suggest that the existing crossing culvert are not able to pass the estimated design peak flows (1:100yr or Regional) without overtopping the existing road, and that water would spill over the road in all cases.

Table 3: Existing Road Crossing Culvert Hydraulics

Culvert ID#	Culvert Size and Material	Estimated Peak Design Flow Rate (m^3/s)	Modelled Upstream Water Level (masl)	Existing Road Centerline Elevation (masl)	Overtopping Road
CC#4	750 mm CSP	2.16	181.32	181.20	Yes
CC#5	700 mm CSP	2.45	181.74	181.63	Yes
CC#6	400 mm CSP	1.70	179.88	179.57	Yes
CC#7	600 mm CSP	3.32	176.45	175.95	Yes
CC#8	1,200 mm CSP	8.96	174.37	173.73	Yes
CC#9	1.9 m x 1.1 Concrete Box	12.95	169.98	169.75[1]	Yes
CC#10	500 mm CSP	3.14	171.07	170.50	Yes

[1] Elevation represents low point approximately 40 m south of Culvert CC#9

2.3 Existing Stormwater Features

2.3.1 407 ETR

Discussions between CIMA+ and 407 ETR engineers suggest that the highway drainage system at the 407 ETR interchange (including ditches, culverts, and ponds) has been designed to accommodate widening of Ninth Line (Figure 3). Stormwater plans for the interchange were not provided by 407 ETR. Future Ninth Line road drainage in that area (i.e. between the overpass over the 407/403 east-west connection and the north limit of the Site) will be captured in proposed catch basins in the new RIGHT OF WAY and routed to the nearest 407 ETR ditch or crossing culvert. The capacity of the existing interchange stormwater features to receive this flow will be verified during detailed design in consultation with 407 ETR.

2.3.2 William Halton Parkway Roundabout

Drawings obtained for the planned William Halton Parkway roundabout indicate the intersection will be drained by a system of catch basins that collect runoff and convey it 100 m southwest to an existing crossing culvert and channel (Figure 4). The channel flows south towards the wetland and ultimately through CC#8 and CC#9. In addition, the proposed design includes the replacement of the existing Culvert CC#5 with a 900mm diameter concrete culvert.

Profile drawings provided by CIMA+ show the existing road elevation of 181.7 masl at CC#5 (140 m north of the intersection) and approximately 184 masl at the intersection. Furthermore, the "Stormwater Management Design Report – William Halton Parkway, Ninth Line to Trafalgar Road" (Stantec, 2016) shows minor drainage from the intersection routed to an oil grit separator sized for only the 0.79 ha roundabout catchment. It is therefore unlikely that any storm sewer system for Ninth Line north of the William Halton Parkway can be connected to the proposed William Halton Parkway system. The section of the ninth Line storm sewer north of William Halton Parkway would therefore have to discharge to the local low point (in this case, the channel downstream of CC#5).

2.3.3 Dundas Street Intersection

Currently a portion of the ditches on the east and west side of Ninth Line at Dundas Street drain to ditch inlet catch basins on both sides of Ninth Line, located approximately 120 m north of the Dundas Street intersection (Figure 5). The elevation and inverts of the catch basins was not surveyed as part of Ninth Line topographic survey. Profile drawings of Ninth Line provided by CIMA+ show a relatively flat slope between Dundas Street and CC#9.

3.0 PROPOSED CONDITIONS

The proposed stormwater management design for the Ninth Line corridor is discussed below. This section includes the basis for the design, the proposed stormwater management system for managing flows within the right of way, the proposed road crossing culverts and associated channel work, and the proposed external ditching to capture flows outside the right of way.

3.1 Basis of Design

The proposed stormwater management is designed to satisfy targets set from various stakeholders, including Halton Region, the Town of Oakville (via the North Oakville Creek Subwatershed Study), Conservation Halton, and the Ontario Ministry of Natural Resources and Forestry (MNRF).

North Oakville Creek Subwatershed Study

The North Oakville Creek Subwatershed Study ("NOCSS") was prepared for the Town of Oakville in August 2006. The goal of the subwatershed study was to "develop a subwatershed plan that allows sustainable development while ensuring the maximum benefits to the natural and human environments on a watershed basis" (Oakville, 2006). In the Implementation Report for the study, Table 7.4.1 sets target area unit peak flow rates for various catchments and storm events, the targets for the JC-D2 catchment of Joshua Creek (of which the Site is a part) are shown in Table 4 below.

Table 4: NOCSS Unit Peak Flow Targets

	Target Unit Peak Flow Rate ($m^3/s/ha$)						
	1:2-year storm	1:5-year storm	1:10-year storm	1:25-year storm	1:50-year storm	1:100-year storm	Regional Storm
JC-D2	0.004	0.007	0.009	0.012	0.013	0.015	0.043

Halton Region

Halton Region requires that the stormwater management system for the proposed Ninth Line be designed to convey runoff from both the 1:100 year and Regional storm events. This includes both the management of stormwater within the road right of way as well as flows in the road crossing culverts.

Conservation Halton

Conservation Halton has expressed (November 24, 2016) a desire to see infiltration and water quality improvements along the Ninth Line corridor, in the form of low impact development features such as infiltration trenches or bioretention features. They have also suggested opportunities for increasing the size of the proposed wetland culvert (CC#7) and the construction of dry benches in the culvert to facilitate wildlife passage Conservation Halton has also requested that new or replacement culverts be sized at 3 times the bankfull width, in particular the Joshua Creek tributary CC#9..

MNRF

Natural Heritage field work has identified Barn Swallow nests within the existing CC#9 culvert. Barn Swallows have been designated as “threatened” in Ontario, and the proposed recovery strategy includes protection of nesting sites. As a result, several different options for the CC#9 culvert were evaluated and discussed with respect to maintain barn swallow habitat.

3.2 Right of Way Stormwater Management

The proposed stormwater management system for the Ninth Line right of way is designed to meet the design basis above while leveraging the existing stormwater features and considering physical constraints of property and existing infrastructure.

A hydrologic/hydraulic model of the proposed right of way stormwater management system was created in EPA SWMM5. For the model, the proposed right of way was divided into sections between 70 m and 275 m long (based on cross section and right of way width), with two catchments for each section (one for each side of the road). Catchment characteristics were defined from the function design drawings for the preferred alternative provided by CIMA+ in May 2017.

3.2.1 Design Overview

Considering the existing features and site topography, the proposed SWM design divides the area into four sections (Figure 6):

- The section at the 407 ETR will continue to drain to the existing stormwater features;

- The section between the 407 ETR and the proposed William Halton Parkway roundabout will drain (via LID features and stormwater pipes) to a discharge point at the downstream end of the proposed CC#5;
- The section between the proposed William Halton Parkway roundabout and sports fields north of Dundas Street will drain (via LID features and stormwater pipes) to a discharge point at the downstream end of the proposed CC#9; and,
- The section between the sports fields north of Dundas Street and the Dundas Street intersection will connect (via LID features and stormwater pipes) to the existing SWM infrastructure at the intersection.

3.2.2 Low Impact Development Options

Both bioretention and infiltration trench systems were considered for the proposed stormwater system south of the 407 ETR. Both systems were discussed and modeled (along with stormwater pipe storage), and the discussion and results are presented below.

Bioretention Systems

The proposed bioretention system design follows the Credit Valley Conservation Authority's "Low Impact Development Stormwater Management Planning and Design Guide" (CVC, 2010). The system would occupy a 2 m wide landscaped border between the curb and edge of the multi-use pathway or right of way boundary (Figure 7). Flows would enter the bioretention system either via sheet-flow from the outside portions of the right of way or via curb cuts along the road. A 0.3 m deep surface storage area would be underlain with a 1.0 m deep filter media containing vegetation plantings. This in turn would be underlain with a storage layer to allow any water passing through filter layer to slowly seep out of the bioretention system. An overflow pipe with an opening set above the surface storage level would allow excess water to flow into a storm sewer system under the road. Seepage through the filter layer is assumed as 118.5 mm/hr (assuming a fine sand), and seepage from the bottom of the storage layer is assumed as 34 mm/hr, reflecting a conservative estimate of percolation for a silty clay soil (based on initial field results from the geotechnical investigations by Golder in 2017).

Infiltration Trench System

The proposed infiltration trench system design follows the same Credit Valley Guidelines (CVC, 2010). The system would occupy a maximum width of 2.4 m (Figure 8), however a portion or all of that width can lie under the multi-use pathway, allowing for implementation in a constrained right of way. In this system, runoff flow from the right of way would be directed towards catch basins along the curb on either side of the road; the catch basins would include a catch basin inset sediment trap providing pre-treatment of flows (to reduce sediment and clogging of the trench). A low flow would connect the pre-treated catch-basing flows to a perforated distribution pipe within the trench, and a high flow pipe also connected to the catch basin (with an invert below the top of the trench) would route flows into a storm sewer system under the road once the infiltration trench is full. The planned trench is 1.5 m deep and filled with crushed gravel surrounded by a water-permeable geotextile. As with the bioretention feature, seepage from the bottom of the storage layer is assumed as 34 mm/hr.

LID Selection

The primary constraint for use of this system along Ninth Line is available space. A bioretention system requires 2 m of landscaped area at the surface, however the preferred road option from the wetland south to Dundas Street was designed to limit property requirements and the disturbed footprint and thus does not allow for 2 m of landscaped area between the road and the multi-use pathway. Furthermore, initial modelling in SWMM5 suggested that because of the percent of impervious surface in the right of way where there is room for bioretention (roughly 80% in these sections), the 0.3 m storage depth above the filter layer would likely be unable to store/infiltrate sufficient runoff to achieve the NOCSS targets without significant downstream storage. Bioretention systems where therefore not included in the proposed conceptual design.

With the infiltration trench, the space constraint is lessened in that the system can be built underneath the multi-use pathway. This allows the system to be built along most of the proposed road alignment south of the 407 ETR. In some areas however, the infiltration trench system could not be implemented, including at road crossing culverts (which cross the right of way and would intersect the trenches) and along the existing gas line west of Ninth Line. High groundwater levels may also affect operations of proposed infiltration trenches. Although preliminary results from geotechnical investigations (Golder, 2017) suggest that the groundwater level is below the proposed infiltration trenches (groundwater was encountered only at a single borehole, in a silty sand layer over 5 m below ground surface), further testing during detailed design will be needed to confirm groundwater depth and percolation rates from the proposed trenches.

The lifespan of the proposed infiltration trench will be affected by the level pre-treatment of runoff from the ROW. The selection of a catch basin or inlet designs that will maximize the removal of trash and suspended sediment (that would otherwise clog the infiltration trench) will be an important consideration at the detail design stage of the project. Furthermore, a plan for regular inspection, cleanout and maintenance of the inlets, sumps, and infiltration trenches should be described at the detailed design stage. The final design of the trenches will need to take into account life cycle costing for the system, including replacement of the trenches once they reach their design lifespan.

Superpipes

In addition to the preferred Infiltration Trench option discussed above, the proposed design would include large diameter storage pipes (‘superpipes’) to provide quantity control. These pipes would be located immediately upstream of the discharge points for the system:

- immediately upstream of the proposed outfall north of William Halton Parkway (CC#5),
- immediately upstream of the proposed outfall at the Joshua Creek Tributary (CC#9); and,
- immediately upstream of the proposed connection to the Dundas Street intersection storm sewer.

These pipes (show on Figures 9-11) would be between 1200mm and 1800mm in diameter and include orifice plates at the discharge point to control peak flows. The orifice plates would be sized to limit the flow to the NOCSS targets for both the 1:100yr storm and Regional storm.

Modelling Results

Modelling in SWMM5 (shown in Table 5 below) suggests that the storage provided by infiltration trenches along each side of the road would likely be able to store/infiltrate sufficient runoff to achieve the NOCSS targets for both the 1:100 year and Regional storms with a storage superpipe included in the design. Additional measurements of in-situ infiltration at the proposed infiltration footprints, as well as assessment of the runoff and infiltration trenches with respect to groundwater quality and detailed hydraulic modelling of the inlet, conveyance, superpipe, and orifices will be conducted at the detailed design stage.

Table 5: Stormwater Management System Outfall Controls

Outlet Location	Contributing Area (ha)	1:100yr Storm		Regional Storm	
		Peak Discharge (m^3/s)	Peak Discharge ($m^3/s/ha$)	Peak Discharge (m^3/s)	Peak Discharge ($m^3/s/ha$)
North of William Halton Parkway (at CC#5)	2.23	0.03	0.013	0.055	0.025
Joshua Creek Tributary (CC#9)	5.38	0.081	0.015	0.187	0.035
Dundas Street Storm Sewer	0.70	0.008	0.011	0.02	0.029

In additional comments provided for the project, Conservation Halton has requested that the superpipe design consider "storm stacking effect", or the effect of back-to-back small storms. This was done by confirming the drain time of the superpipes for the 1:2 yr storm event is less than 48 hours so that storage volume is available should a second 1:2 yr storm occur immediately after the first storm. Results from the modelling suggest that the majority of the storm runoff is captured in the infiltration trenches and that the superpipe drain time is less than 48 hours at all three outlets (see Table 6 below). Additional design and assessment of the superpipe storage, discharge, and drain time will be conducted at the detailed design stage.

Table 6: Drawdown Time for 1:2 yr Storm

Outlet Location	Drain Time for 1:2 yr Storm (hours)
North of William Halton Parkway (at CC#5)	30
Joshua Creek Tributary (CC#9)	25
Dundas Street Storm Sewer	24

3.2.3 Dundas Street Intersection

As noted, the area directly north of the Dundas Street intersection currently drains to ditch inlet catch basis (one on either side of the Ninth Line). and ultimately connect to the storm the Dundas Street storm sewer system. Profile drawings of Ninth Line provided by CIMA+ show a relatively flat slope between Dundas Street and CC#9, suggesting that redirecting runoff from the section of Ninth Line currently draining towards the Dundas Street intersection (so that it would instead flow back towards CC#9) may be difficult. It is therefore proposed that this portion of Ninth Line would continue to drain towards the Dundas Street intersection storm sewer system via a direct connection from the proposed LID features.

The ditch inlet catch basins would be removed as part of the widening of the road and replaced with the proposed roadside catch basin / infiltration trench / superpipe system proposed above. This proposed system would capture runoff from the same areas as the existing ditch inlet catch basins and would connect to the came point in the Dundas street storm sewer system. Based on the modelling results in Table 5 above, the proposed infiltration system and the superpipes is expected to meet the NOCSS targets for peak discharge rate; given that there are currently no controls prior to the ditch inlets, this is expected to result in a decrease in peak flows in the proposed condition as compared to the existing condition. Additional detailed hydraulic modelling of the inlet, conveyance, superpipe, and orifices will be conducted at the detailed design stage.

3.2.4 Water Quality

In additional comments provided for the project, Conservation Halton has requested a discussion of how the proposed conceptual design will affect water quality. The comments provided note Total Suspended Solids (TSS), Total Phosphorus (TP), and Temperature as parameters of interest.

A preliminary assessment of the proposed conceptual design suggests the following:

- Total Suspended Solids: There is no removal of TSS from road runoff before it reaches the roadside ditches in the existing condition. In the proposed condition, removal of suspended sediments is expected to occur in both the catch basin inserts (proposed for all catch basins) and the infiltration trenches, with no further removal of TSS expected in the superpipes. The CVC LID Design Guide (CVC, 2010) does not provide guidance for catch basins inserts, however Table 4.4.3 of the Guide suggests a TSS removal of approximately 80% for an infiltration system.
- Total Phosphorus: There is no removal of TP from road runoff before it reaches the roadside ditches in the existing condition. In the proposed condition, removal of TP is expected to occur in the infiltration trenches, with no further removal of TP expected from either the catch basin inserts or the superpipes. Table 4.4.3 of the CVC LID Design Guide (CVC, 2010) suggests a total phosphorus removal of approximately 80% for an infiltration system.
- Temperature: There is no measures to control temperature of road runoff before it reaches the roadside ditches in the existing condition. In the proposed condition, the infiltration trenches and superpipes are expected to result in some reduction of temperature as the soils absorb some heat from the water. The infiltrated water would also not discharge to the surface water.

Additional detailed estimates of the impact to water quality will be conducted at the detailed design stage.

3.2.5 Future Technological Improvements

Construction for the project is not scheduled until 2025; it is possible that ongoing evolution of stormwater management and low impact development may result in new technologies and best management practices becoming available prior to the detailed design phase. These advances may include improved materials, construction methods, and monitoring capabilities, as well as new cost-effective solutions for improving water quality and tailored filtration media. In particular, improvements in water quality treatment at stormwater inlets may serve to extend the life of the system by removing additional sediment before it enters the infiltration trenches. The future detailed design phase of this project should include a review of currently available stormwater management technologies and, if more appropriate methods are available, update of the approach presented here.

3.3 External Drainage

3.3.1 Crossing Culverts

A culvert sizing exercise was undertaken for the overtopping culverts to determine required sizing given the widened roadway. This analysis used the HEC-RAS models previously developed for assessing the existing culverts, and culverts lengths provided by CIMA+.

Crossing Culvert CC#4

Culvert CC#4, located 350 m south of the Highway 407 interchange, is to be replaced as part of that proposed work. Modelling for the crossing suggests a new culvert should be a minimum 1200 mm diameter concrete pipe with a length of 46 m to both span the right of way and pass the 1:100yr and Regional storm peak flows without overtopping. As with the existing culvert, the replacement culvert would discharge to a swale on the west side of Ninth Line, leading southwest towards the downstream end of Culvert CC#5.

Crossing Culvert CC#5

Culvert CC#5, located 140 m north of the William Halton Parkway roundabout, is to be replaced as part of that proposed work. Modelling for the crossing suggests a new culvert should be a minimum 1200 mm diameter concrete pipe with a length of 50.5 m to both span the right of way and pass the 1:100yr and Regional storm peak flows without overtopping. As with the existing culvert, the replacement culvert would discharge to a swale on the west side of Ninth Line, leading southwest along William Halton Parkway.

Crossing Culvert CC#6

Culvert CC#6, located 400 m south of the William Halton Parkway roundabout, is to be replaced as part of that proposed work. Modelling for the crossing suggests a new culvert should be a minimum twin 900 mm diameter concrete pipes with a length of 43 m to both span the right of way and pass the 1:100yr and Regional storm peak flows without overtopping. As with the existing culvert, the replacement culverts would discharge to an existing swale on the west side of Ninth Line, flowing west towards the wetland feature.

Wetland Culvert CC#7

During a field visit in summer 2016, Conservation Halton staff noted that the culvert at the wetland (CC#7), which was suggested to convey flows between the two wetland segments on either side of the road, was actually 40 m south of the expected position between the two wetland sections (Figure 2). During a subsequent meeting with Conservation Halton, Halton, CIMA+, and Golder on November 24, 2016, Conservation Halton expressed support for constructing the proposed replacement culvert north of the existing location to connect the two wetland sections. Conservation Halton also expressed an interest in seeing the culvert height increased to provide wildlife passage between the wetland segments and suggested the use of a concrete box culvert (as opposed to a CSP pipe at the wetland) with an internal dry bench structure. This would include a 0.5 m wide area of soil material raised 0.3 m above the culvert invert adjacent to the wall of the culvert (with the proposed culvert modelled as blocked up to the 0.3 m depth). Based on Golder's experience, an appropriate culvert height at this location would be 0.9 m (0.3 m bench and 0.6 m clearance) for passage of small mammals such as skunks and raccoons, and amphibians and reptiles such as snakes and frogs. The proposed box culvert height was therefore maintained as 1.0 m as per the sizing recommendation. This will require raising the road profile through the wetland area, requiring retaining walls along the road through the wetland (Figure 2). Retaining walls are considered preferable in this case to grassed fill slopes falling away to either side of the road, which would result in additional wetland encroachment.

Modelling for the crossing with the proposed bench suggests the new culvert should be a minimum 3.0 m wide by 1.0 m tall open-bottom concrete box with a length of 45 m to both span the right of way and pass the 1:100yr and Regional storm peak flows without overtopping. As noted by Conservation Halton, the proposed culvert would be slightly north of the existing culvert to be more in-line with the wetland location.

Wetland Outflow and CC#8

The topographic survey showed the highpoint of the ditch south of the wetland on the east side to be 174.42 metres above sea level (masl) (20 m south of existing Culvert CC#7), while the high point of the ditch on the east side is shown as 175.38 masl (75 m south of existing Culvert CC#7). This, along with the observed cattails in the ditch to the west of the road and the 900 mm diameter driveway culverts at the Fern Hill School (170 m south of Culvert CC#7) in an area where the adjacent land generally drains away from the road (i.e., less runoff to the ditch), suggests that the wetland outflow generally follows the west ditch south from the wetland, through the 900 mm diameter driveway culverts, and crosses Ninth Line at the existing Culvert CC#8 (1,200 mm diameter CSP).

The proposed road design includes a narrow right of way cross section designed to minimize property impacts at the Fern Hill School, as well as wetland impacts to the east of the road. The proposed design calls for the construction of a new crossing culvert immediately north of the school (Figure 12). This new culvert further reduces the footprint of the road in this area.

- The proposed culvert would be a 3 m wide by 1.5 m tall open bottom concrete box culvert, with a length of 45 m and a 0.6% slope, crossing Ninth Line immediately north of the school. The low height of the culvert is intended to limit changes to the vertical alignment of the road at this location. The culvert itself is designed to pass the 8.17 m^3/s peak flow expected for the existing CC#8 (which includes the flow from the wetland). Both existing and proposed crossing culverts connect roadside ditches; since these are not natural channels, there is no meander belt requirement to be met with the proposed culvert width.
- The upstream end (west side) of the culvert would be linked to the wetland (at approximately the proposed location for Culvert CC#7) by a 200 m long, 0.5 m wide trapezoidal swale with 3:1 side slopes and a longitudinal slope between 0.5% and 2%.
- The downstream end of the culvert (east side) would be linked to the existing drainage ditch away from the road at the existing Culvert CC#8 by a 200 m long, 0.5 m wide trapezoidal swale with 3:1 side slopes and a 0.5% slope.

With the wetland outflow diverted north of the school, and runoff from the school itself is shown to drain away from the road (based on the Region catchments), a 5 ha area between the school and the existing CC#8 crossing (largely a portion of the soccer field south of the school) would now drain to the Ninth Line roadside ditch and ultimately south toward the downstream end of the Joshua Creek CC#9 culvert (Figure 12). The proposed catchment to CC#8 without this 5 ha portion was modelled using the previously-created Regional storm SWMM5 hydrologic model, and the results suggest that there would be an approximate 1.5 m^3/s decrease in peak flow during the Regional storm event (from 8.96 m^3/s to 7.46 m^3/s, or a 17% decrease). Ultimately, the proposed 3 m by 1.5 m concrete box culvert was found to pass either of the peak flows without overtopping the road.

Joshua Creek Tributary CC#9 and CC#10

Natural Heritage field investigations by Golder has identified the Joshua Creek Tributary crossing Culvert CC#9 as barn swallow habitat based on the presence of barn swallow nests in the culvert. Based on the MNRF “threatened” designation for Barn Swallows, additional alternatives for extending or replacing CC#9 were considered. These included:

- **Option 1: Cleanout and Extension:** Cleanout of 0.1 m of sediment from the existing 1.9 m x 1.1 m concrete box culvert and extension as required by the road widening.
- **Option 2: Extension and Overflow Culvert:** Cleanout and extension of the existing 1.9 m x 1.1 m concrete box culvert, twinned with an additional 2.4 m x 1.2 m open bottom concrete box culvert to provide additional capacity and barn swallow nesting habitat.
- **Option 3: Replacement:** Replacement of the existing culvert with a 4.0 m x 1.2 m open bottom concrete box culvert which would provide replacement capacity and barn swallow nesting habitat.
- CIMA+ noted that an increase in elevation of the road in that area was being forced by the proposed CC#10, which has a proposed obvert of 169.9 masl (compared to the proposed obvert of CC#9, which is 168.7 masl). It has therefore been proposed to eliminate CC#10, route the additional flow through a roadside ditch to the upstream face of the proposed CC#9, and increase the width of the proposed CC#9 culvert to pass the flows. The previously-created Regional storm SWMM5 model was used to model the effect of both the decrease in catchment to CC#8 (which is upstream and connected to CC#9 in the model) and the diverted flow from CC#10 to the upstream end of CC#9; the result was a peak flow of 12.51 m^3/s at CC#9 (down 0.44 m^3/s or 3% from the existing condition).

Results from the modelling are shown in Table 7 below; they suggest that:

- The Option 1 cleanout and extension of the existing culvert is not sufficient to pass the flow without overtopping the road; and
- The proposed Options 2: Overflow Culvert and Option 3: Replacement provide sufficient capacity to pass the peak flows.

Table 7: Joshua Creek Tributary Culvert Modelling Results

Scenario	Culvert Length (m)	Culvert Opening Size	Estimated Upstream Water Level (m)	Overtopping Road during Regional Flow
Option 1: Culvert Cleanout and Extension	36.5	1.9 m x 1.1 m	170.87	Yes
Proposed: Culvert Extension and Overflow Culvert	36.5	1.9 m x 1.1 m 2.4 m x 1.2 m	169.07	No
Option 3: Culvert Replacement	36.5	4.0 m x 1.2 m	169.08	No

Based on hydraulics and ease of construction, as well as the Conservation Halton request to provide a culvert 3 times the bankfull width (where the bankfull width is shown as between 0.5 m and 1 m downstream of the existing culvert in the fluvial geomorphology report), Option 3: Culvert Replacement is suggested as the preferred option for Culvert CC#9 in the proposed condition. This option is expected to provide sufficient hydraulic capacity and replacement barn swallow habitat.

Results

The results (shown in Table 8) suggest significant increases in culvert size would be required to convey the peak design flow rate.

Table 8: Capacity Assessment of Proposed Crossing Culverts

Element ID	Estimated Design Flow Rate (m^3/s)	Optimized Culvert Dimensions (mm)	Optimized Culvert Length (m)	Proposed Road Centerline Elevation (masl)	Estimated Upstream Water Level (m)	Overtopping Road
CC#4	2.16	Circular 1,200	46	181.6	181.08	No
CC#5	2.45	Circular 1,200	50.5	181.76	181.38	No
CC#6	1.70	2 x Circular 900	43	179.4	179.11	No
CC#7	3.32	Open Bottom Concrete Box 3,000 x 1,000	45	176.74	176.04	No
CC#8	7.46	Open Bottom Concrete Box 3,000 x 1,500	40.5	175.78	174.51	No
CC#9	12.51	Open Bottom Concrete Box 4,000 x 1,200	36.5	170.2	169.08	No

Upstream Water Levels

In additional comments provided for the project, Conservation Halton has requested a detailed review to assess any increases in flood elevation or velocity on privately owned properties resulting from the new road construction. In order to address that concern, the differences in the modeled upstream water levels at each of the crossing culverts is shown in Table 9 below.

- With the exception of Culvert CC#8, they show the water level during the peak flow event dropping by between 0.24 m and 0.9 m, suggesting the proposed crossing culverts will result in lower water levels and reduced risk of flooding compared to the existing culverts. This is largely the result of the increase in culvert sizes associated with the proposed condition and the fact that the larger culverts no longer result in overtopping of the road during the peak flow.

- Additionally, although Culvert CC#8 shows an increase in water level of 0.14 m, the location of the culvert has been moved roughly 200 m north of its current location. Based on the modelling, the peak water level at the proposed culvert location is contained within the proposed ditch running south from the wetland.

Table 9: Differences in Modelled Water Levels Upstream of Crossing Culverts

Element ID	Estimated Upstream Water Level (m)	
	Existing Condition	Proposed Condition
CC#4	181.32	181.08
CC#5	181.74	181.38
CC#6	179.88	179.11
CC#7	176.45	176.04
CC#8	174.37	174.51
CC#9	169.98	169.08

Additional detailed hydraulic modelling of the crossing culverts to confirm upstream water levels, velocities, and flooding impacts will be conducted at the detailed design stage.

3.4 External Ditches

The existing road is generally flanked on either side by ditches which convey both right of way and external drainage. While the proposed 35 m-wide urban cross-section includes a storm sewer system to convey the right-of-way runoff, outside ditching at the edge of proposed 35 m right of way will be required to convey runoff from areas draining towards the proposed road.

Preliminary sizing of the ditches was carried out by using Figure 2 to estimate contributing surface runoff towards the right of way.

- In areas where the ditches are only intercepting sheet flow from adjacent land area, the proposed ditch size was a 1.0 m deep trapezoidal ditch with a 1.5 m bottom width and 2:1 side slopes.
- Between CC#4 and CC#5, the proposed ditch would need to convey the flow discharging from CC#4, and the proposed ditch was increased to 1 m deep with a 1.5 m bottom width. A discussion at detailed design will determine where it is possible to convert part of this ditch to a pipe system.
- The proposed ditch connecting the wetland to the proposed CC#8 and beyond, a trapezoidal channel up to 3.0 m deep was proposed, with a 0.5 m bottom width and 3:1 side slopes.

Approximate sizing for the ditches is shown on Figure 12.

In additional comments provided for the project, Conservation Halton has requested that roadside ditches be converted into enhanced ditches. These are vegetated ditches where the runoff from the 25 mm design storm does not exceed 100 mm deep, or 2/3 the height of the tallest vegetation in the ditch. Preliminary modelling of the 25 mm design storm flows for the conceptual ditches was carried out in SWMM5 for the intercepting ditches upstream of crossing culverts which, as well the ditches between CC#4 and CC#5 and upstream and downstream of CC#8. Other ditches were not modeled as the assumed catchment area for these ditches (where land generally appears to drain away from the road) was assumed to be negligible. Results (shown in Table 10 below) suggest that;

- The smaller catchments contributing to the intercepting ditches (typically less than 3 ha per ditch) and correspondingly lower peak flows (maximum of 0.17 m^3/s) allow the intercepting ditches to meet the target of 100 mm maximum flow depth during the 25 mm design storm event.
- The ditch between CC#4 and CC#5 results in a peak flow of 0.048 m^3/s, however the relatively low slope of the ditch (less than 0.1% based on existing topography) results in a peak flow depth of 0.19 m. This ditch may still meet the criteria for an enhanced swale based on the ultimate height of the vegetation in the ditch.
- The significantly larger contributing catchments to the ditches upstream and downstream of CC#8 (73.2 ha) results in a correspondingly higher peak flow compared to other ditches (0.4 m^3/s). Consequently, the peak ditch flow depth is higher in this ditch (0.20 m) compared to other ditches. The larger flow and backwater from the channel downstream of this ditch (which crosses private property east of Ninth Line) is expected to make meeting the 0.1 m flow depth target difficult for this ditch.

Table 10: 25 mm Design Storm Ditch Flows

Location	Maximum Flow Rate for 25 mm Design Storm (m^3/s)	Maximum Flow Velocity for 25 mm Design Storm (m/s)	Maximum Flow Depth for 25 mm Design Storm (mm)
Upstream of CC#4	0.043	0.31	0.08
Between CC#4 and CC#5	0.048	0.14	0.19
Upstream of CC#5	0.038	0.35	0.08
Upstream of CC#6	0.027	0.65	0.03
Upstream of CC#7	0.025	0.71	0.06
Upstream and Downstream of CC#8	0.473	0.84	0.20
Upstream of CC#9	0.005	0.29	0.02

It should be noted that in all of these cases, the flow to the ditch is almost entirely from privately-owned land in adjacent catchments (which currently flows via sheet flow towards Ninth Line), and not from the proposed Right of Way (which would drain internally to the SWM system and be treated by the infiltration trenches and superpipe systems). Any additional treatment provided by enhanced ditches would therefore apply to runoff from these privately-owned adjacent lands, not the Right of Way runoff (which for the most part does not flow through these ditches). Future development of these adjacent privately-owned lands in the future would likely require on-site quality control for their own runoff, superseding the effect of the enhanced ditches.

4.0 CONCLUSIONS

The proposed design is expected to meet the requirements of the NOCSS, Halton Region, Conservation Halton, and MNRF. It incorporates the existing SWM features at the 407 ETR, while proposing new features including infiltration trenches (with catch basin inserts providing pre-treatment), storm sewers for conveyance and storage, a realigned culvert at the wetland designed for animal passage, and a Joshua Creek tributary culvert to provide replacement barn swallow habitat. During detail design, some elements will be confirmed, including design of SWM features at the 407 ETR, detail design for the LID system (including pre-treatment and lifecycle costing for the various elements), and delineation of the full extent of infiltration trenches with respect to percolation and service locates.

5.0 LIMITATIONS

The stormwater design and results provided here reflects a conceptual design based on information and road designs made available to Golder at this time. This conceptual design was prepared for the Environmental Assessment Study for Ninth Line and should not be used beyond providing context for that study. Designs, drawings, and results presented in this report should in no way be relied on for future design work (including detailed design or construction drawings for the future Ninth Line). All recommendations and requirements resulting from this work that need to be addressed and implemented in future design work are summarized in the Environmental Assessment document.

6.0 REFERENCES

City of Oakville Planning and Development Commission, Development Engineering Department "Development Engineering Procedures and Guidelines" (2012)

Credit Valley Conservation Authority and Toronto and Region Conservation Authority "Low Impact Development Stormwater Management Planning and Design Guide: Version 1.0" (2010)

Golder Associates Ltd. "Geotechnical Explorations and Testing – Ninth Line (Regional Road 13) Transportation Corridor Improvements from Dundas Street (Regional Road 5) to 407 Express Toll Route, Regional Municipality of Halton, Ontario" (August 2017)

Ontario Ministry of Transportation, Drainage and Hydrology Section, Transportation Engineering Branch, Quality and Standards Division "MTO Drainage Management Manual" (1997)

Stantec "Stormwater Management Design Report – William Halton Parkway, Ninth Line to Trafalgar Road" (November 2016)

Signature Page

Golder Associates Ltd.

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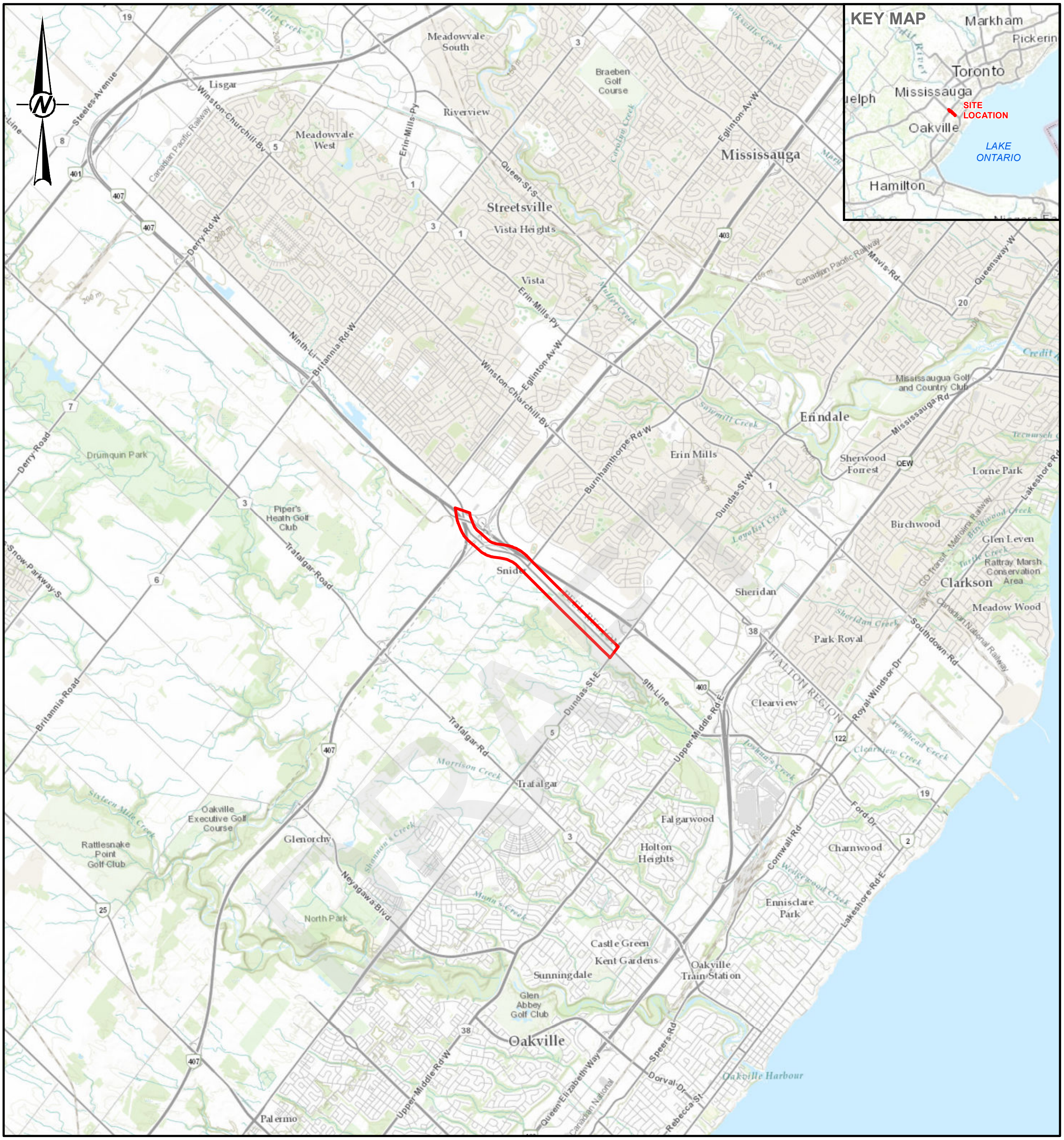
FIGURES



LEGEND

- SITE LOCATION
- ROAD
- WATERCOURSE
- WATERBODY
- WETLAND
- WOODED AREA

0 2 4 6

1:100,000 KILOMETRES

REFERENCE(S)
BASE DATA - MNR LIO, OBTAINED 2016; ESRI, HERE, DELORME, TOMTOM, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSTOPO, MAPMYINDIA, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
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PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17

CLIENT
REGION OF HALTON

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
LOCATION PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-08-30
DESIGNED	JR
PREPARED	JR
REVIEWED	CD
APPROVED	DK

PROJECT NO.	CONTROL	REV.	FIGURE
1648031	0003	A	1

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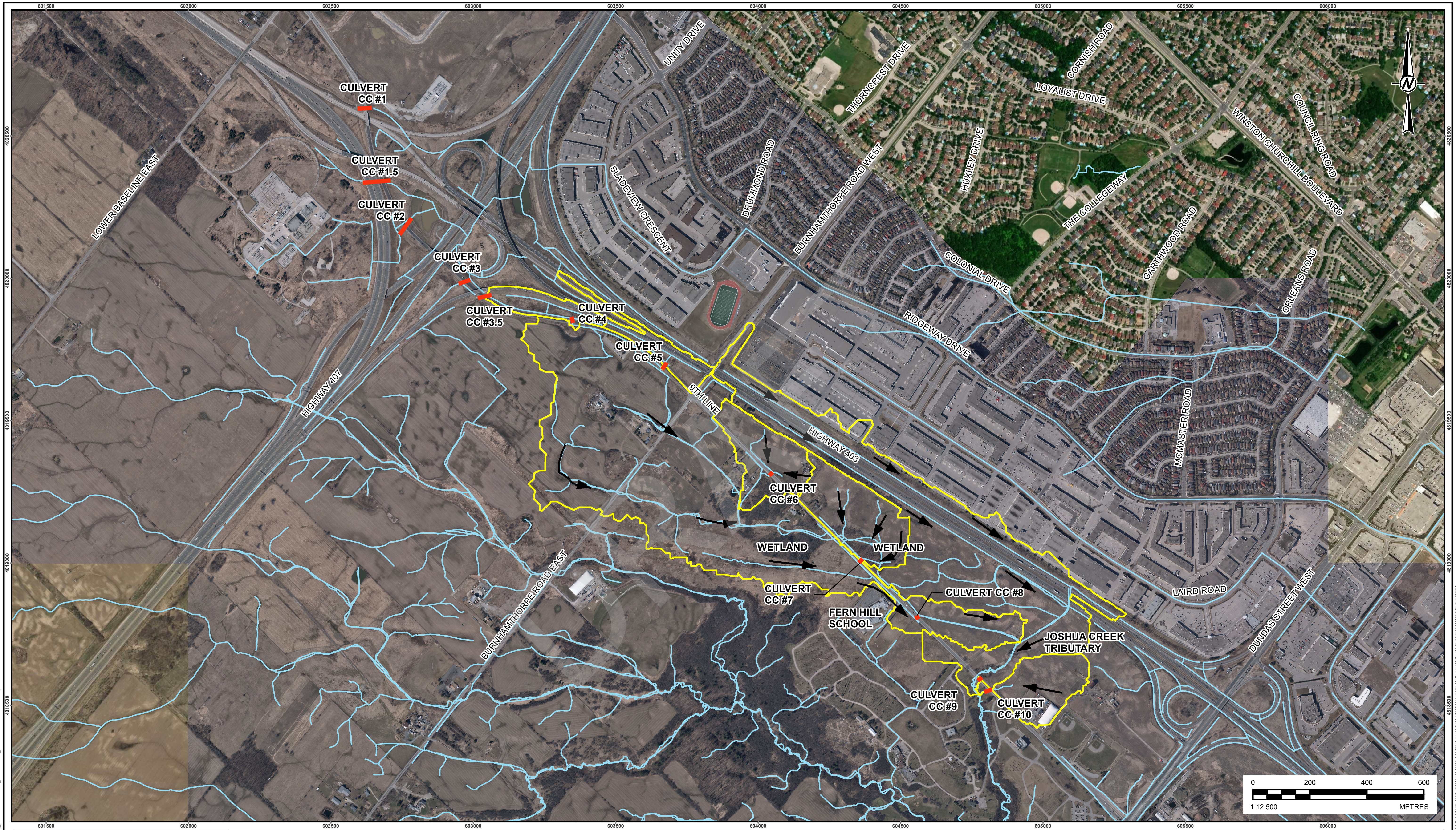

LEGEND

- FLOW DIRECTION
- CULVERT
- CATCHMENT
- WATERCOURSE

REFERENCES

BASE DATA - MNR LIO, OBTAINED 2016
IMAGERY - PROVIDED BY THE REGION OF HALTON, 2016; ESRI, DIGITALGLOBE, GEOEYE, EARTHSTAR GEOGRAPHICS, CNES/AIRBUS DS, USDA, USGS, AEROGRID, IGN, AND THE GIS USER COMMUNITY
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CLIENT
REGION OF HALTON

CONSULTANT

GOLDER

YYYY-MM-DD	2019-08-23
PREPARED	JR
DESIGN	JR/RA
REVIEW	
APPROVED	

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
EXISTING CROSSING CULVERTS AND CATCHMENTS

PROJECT NO.	PHASE	REV.	FIGURE
1648031	0003	A	2

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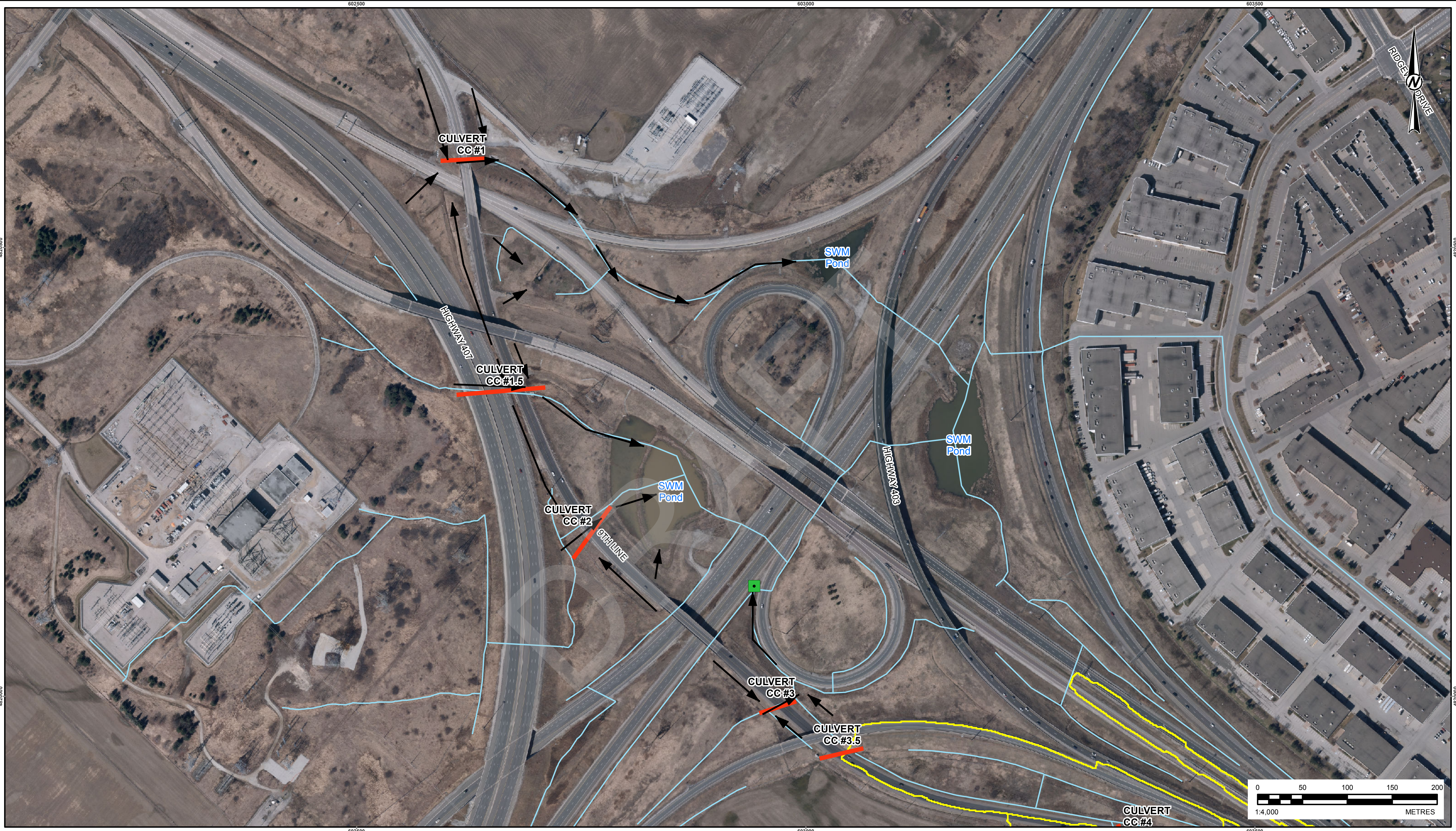

LEGEND

- DICB
- FLOW DIRECTION
- CULVERT
- CATCHMENT
- WATERCOURSE

REFERENCES

BASE DATA - MNR LIO, OBTAINED 2016
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REGION OF HALTON

CONSULTANT

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YYYY-MM-DD	2017-08-30
PREPARED	JR
DESIGN	JR/RA
REVIEW	CD
APPROVED	DK

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
407 ETR DRAINAGE

PROJECT NO.	PHASE	REV.	FIGURE
1648031	0003	A	3

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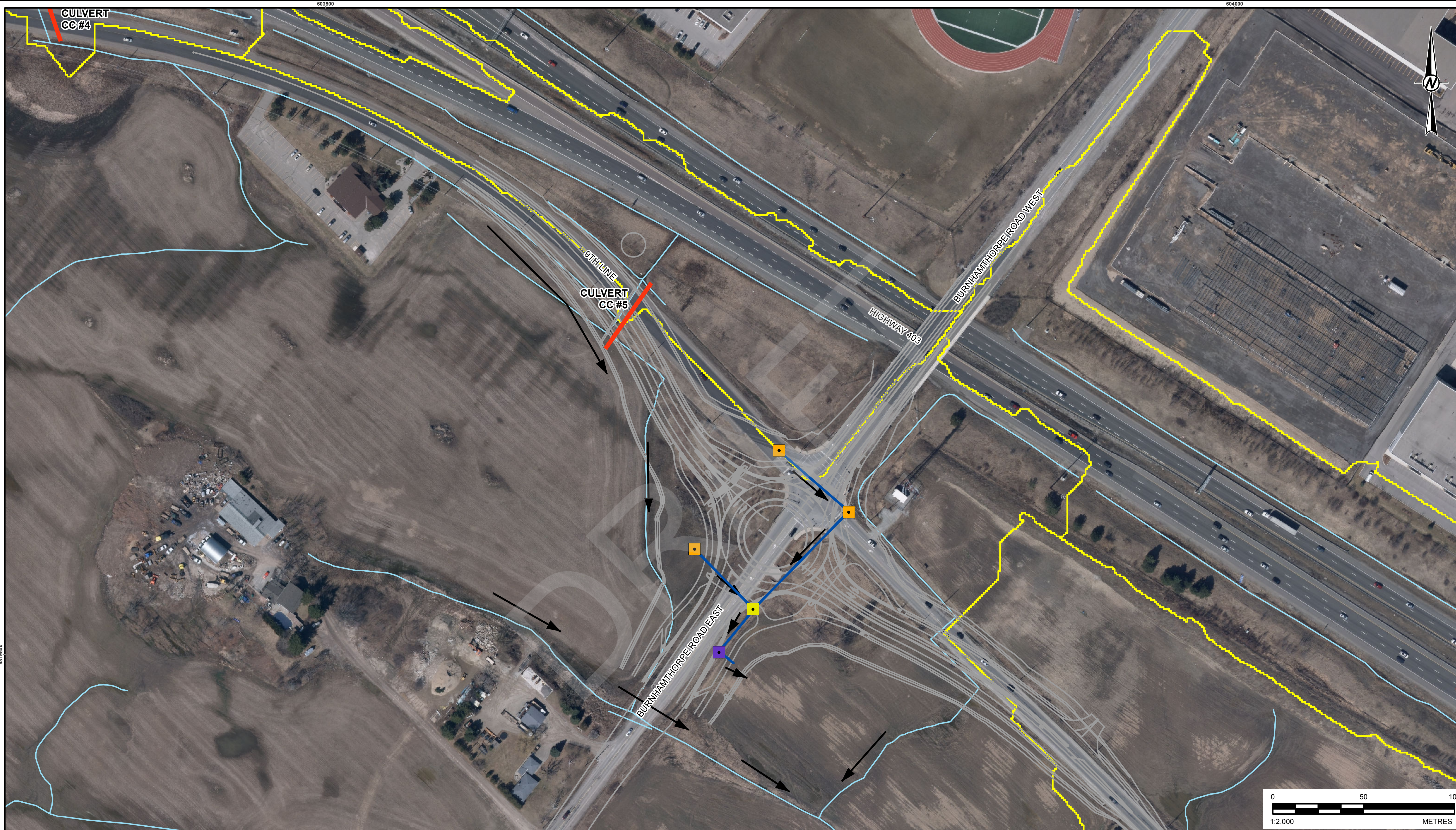

LEGEND

- CB
- CBMH
- OGS
- FLOW DIRECTION
- CULVERT
- CATCHMENT
- WATERCOURSE
- WILLIAM HALTON PARKWAY ROUNDABOUT
- STORM SEWER PIPES

REFERENCES

BASE DATA - MNR LIO, OBTAINED 2016
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REGION OF HALTON

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YYYY-MM-DD	2017-08-30
PREPARED	JR
DESIGN	JR/RA
REVIEW	CD
APPROVED	DK

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
WILLIAM HALTON PARKWAY DRAINAGE

PROJECT NO.	PHASE	REV.	FIGURE
1648031	0003	A	**4**

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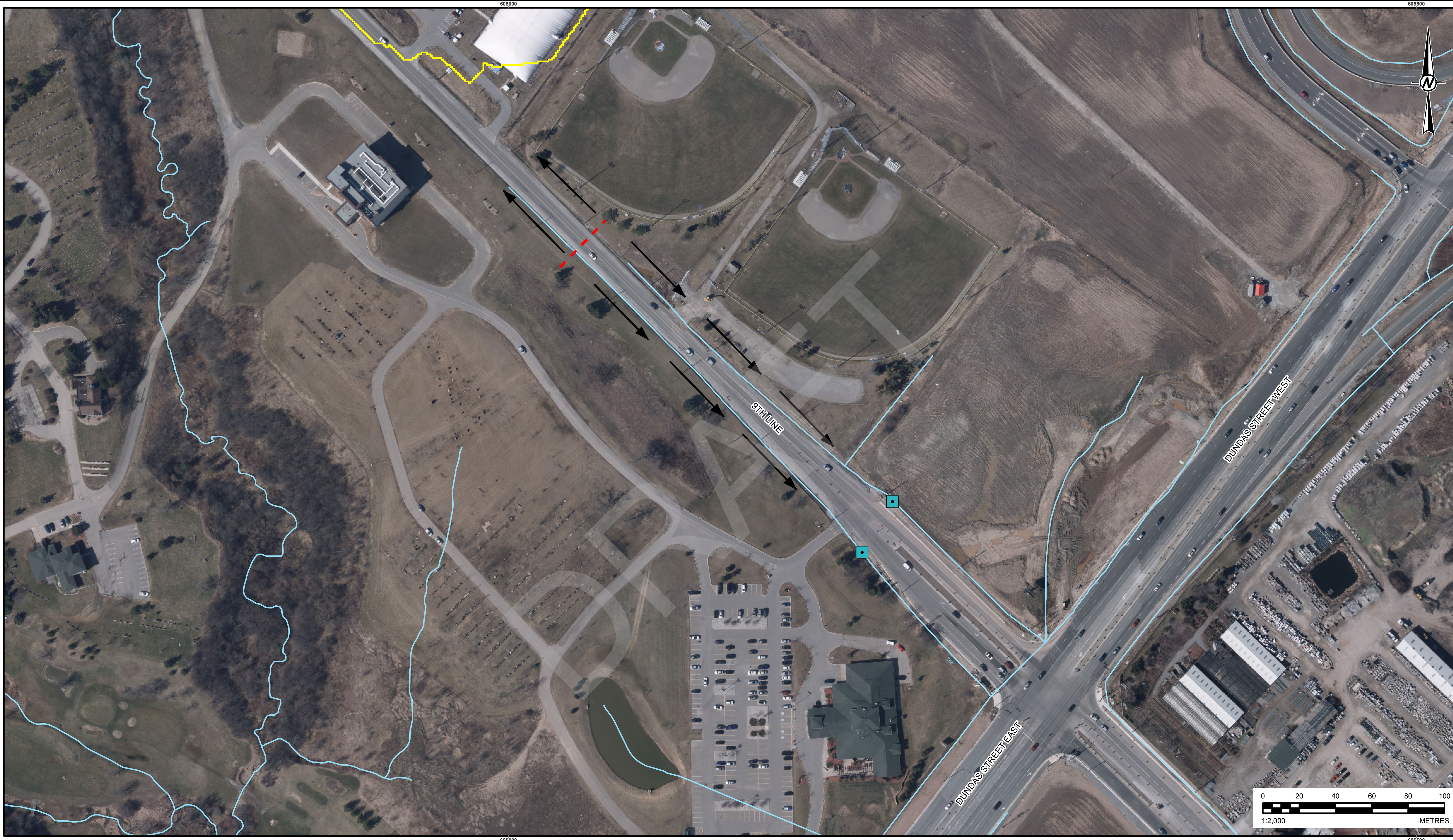

LEGEND

- EXISTING DICB
- FLOW DIRECTION
- CULVERT
- DRAINAGE DIVIDE
- CATCHMENT
- WATERCOURSE

REFERENCES

BASE DATA - MNR LIO, OBTAINED 2016
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CLIENT
REGION OF HALTON

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-08-30
PREPARED	JR
DESIGN	JR
REVIEW	CD
APPROVED	DK

TITLE
DUNDAS STREET DRAINAGE

PROJECT NO.	PHASE	REV.	FIGURE
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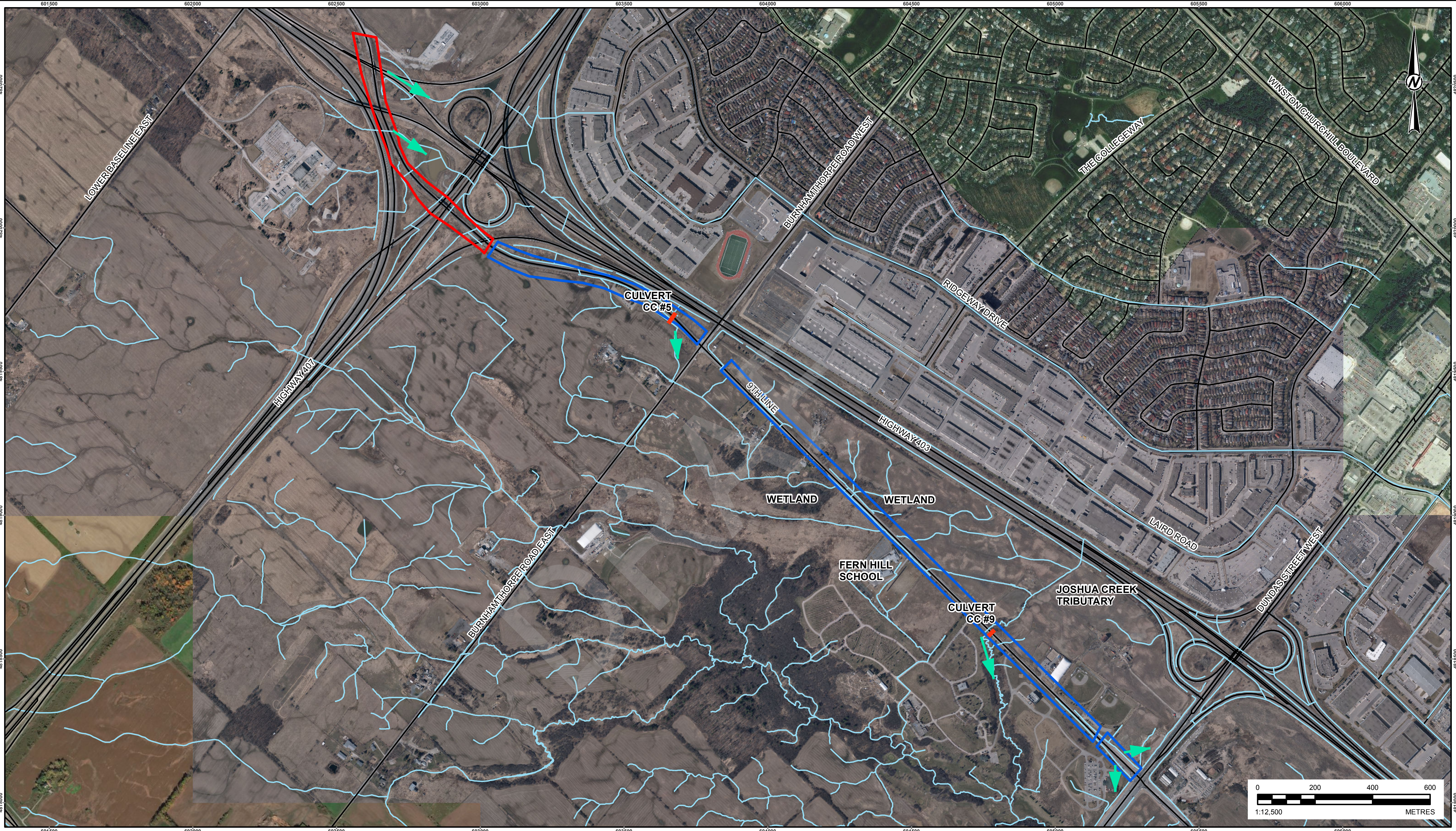

LEGEND

- PROPOSED OUTLET LOCATION
- CULVERT
- CATCHMENT
- WATERCOURSE
- EXISTING 407 ETR SWM FEATURES
- PROPOSED INFILTRATION TRENCHES AND SUPERPIPES

REFERENCES

BASE DATA - MNR LIO, OBTAINED 2016
IMAGERY - PROVIDED BY THE REGION OF HALTON, 2016; ESRI, DIGITALGLOBE, GEOEYE, EARTHSTAR GEOGRAPHICS, CNES/AIRBUS DS, USDA, USGS, AEROGRID, IGN, AND THE GIS USER COMMUNITY
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PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17

CLIENT
REGION OF HALTON

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-08-30
PREPARED	JR
DESIGN	JR/RA
REVIEW	CD
APPROVED	DK

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
PROPOSED STORMWATER SEGMENTS

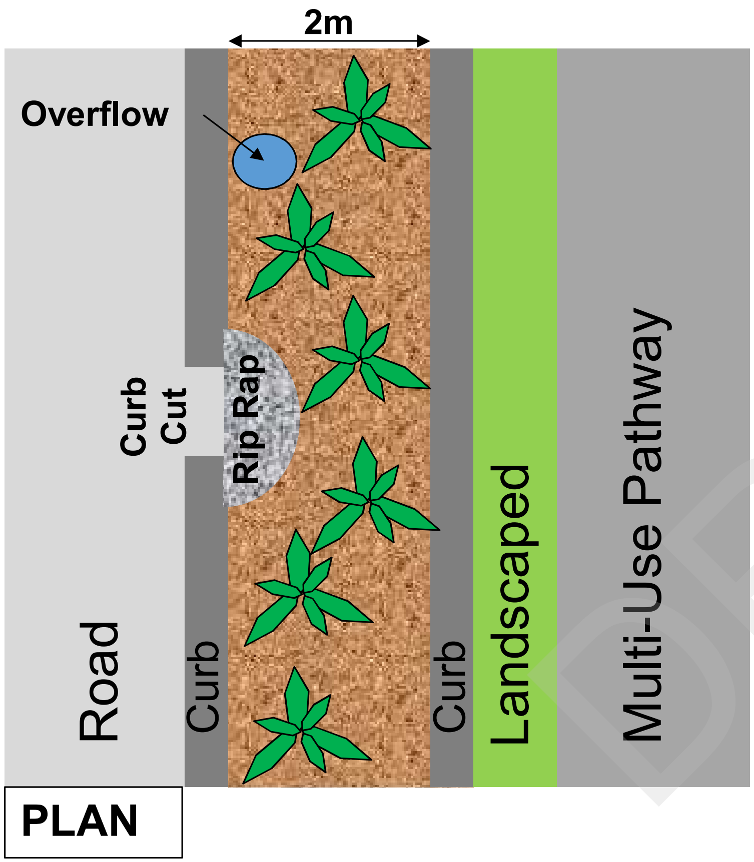
PROJECT NO.	PHASE	REV.	FIGURE
1648031	0003	A	6

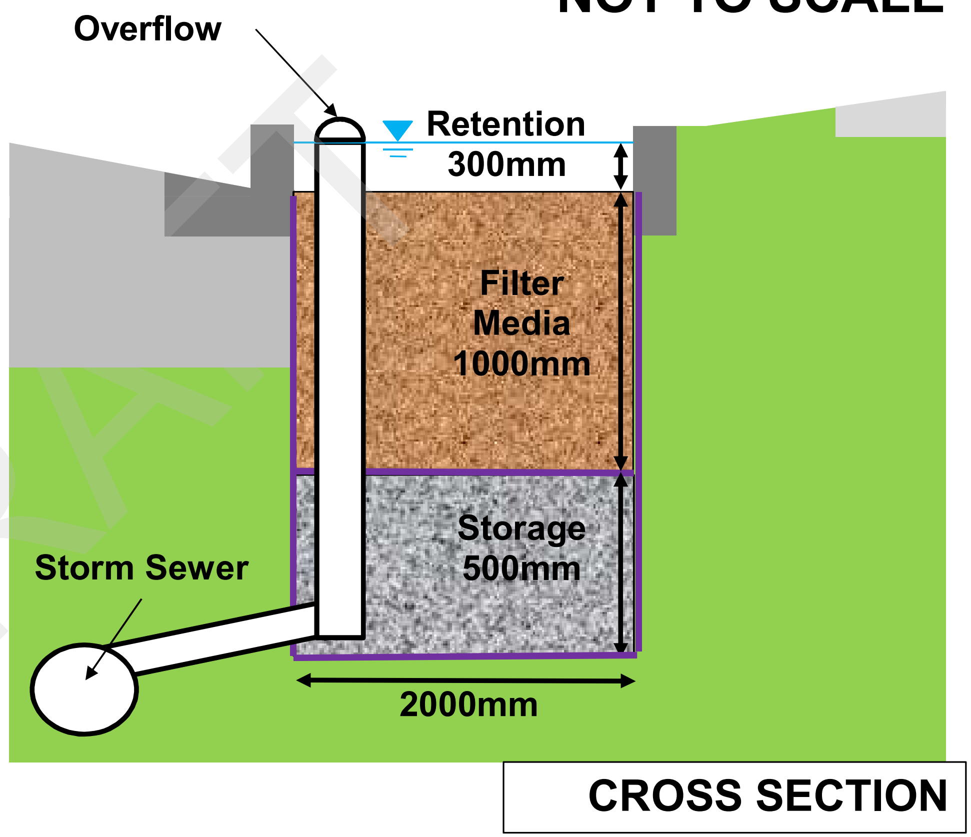
25mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:

S:\Clients\Region_of_Halton\Ninth_Line_Oakville\99_PROJ\1648031\40_PROD\0002_SurfaceWaterConstraintReport\1648031-0002-CS-0001.mxd

Figure 7: Bioretention Schematic

NOT TO SCALE

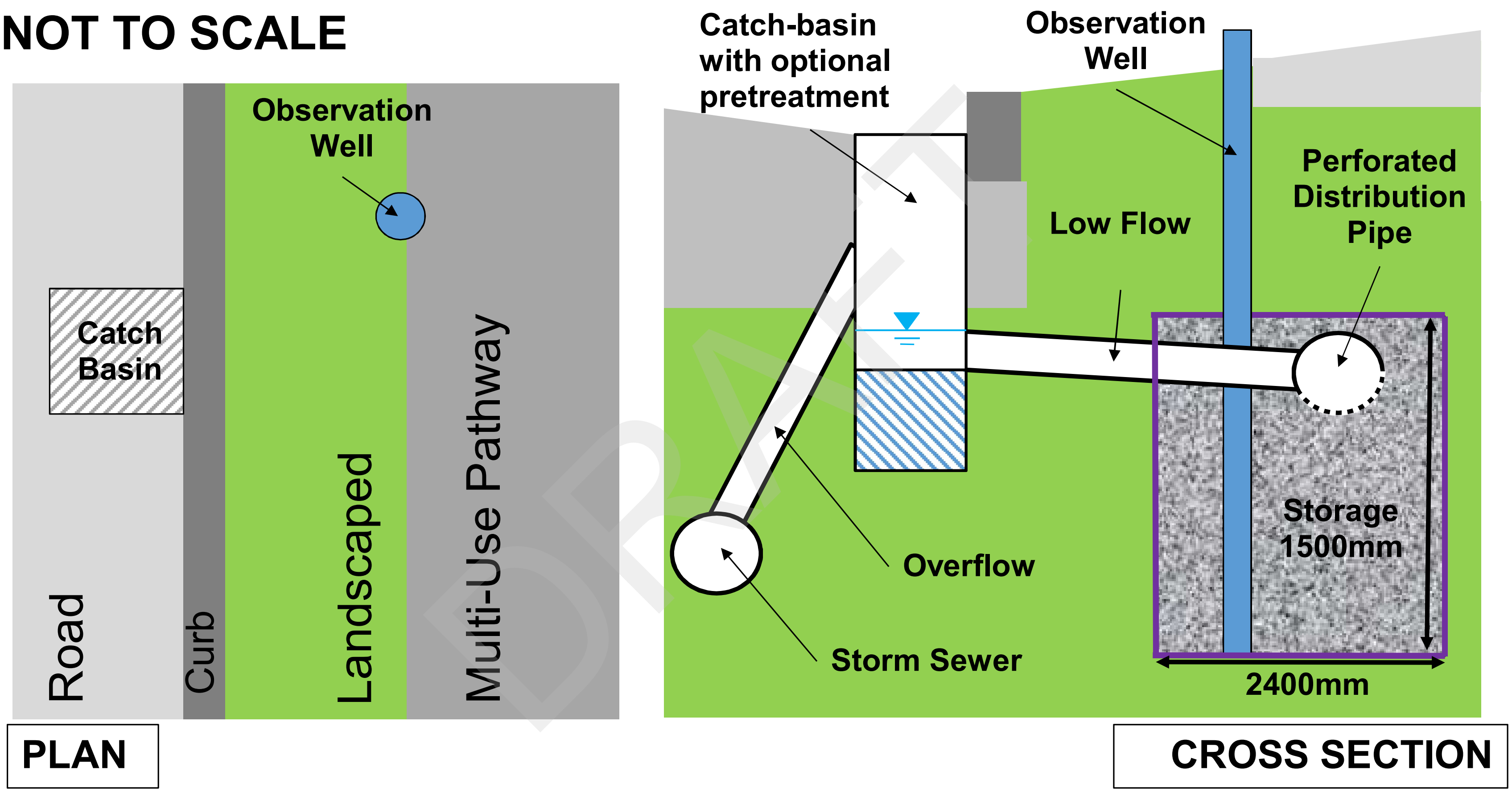




Project: 1648031	Golder Associates	26-May-17

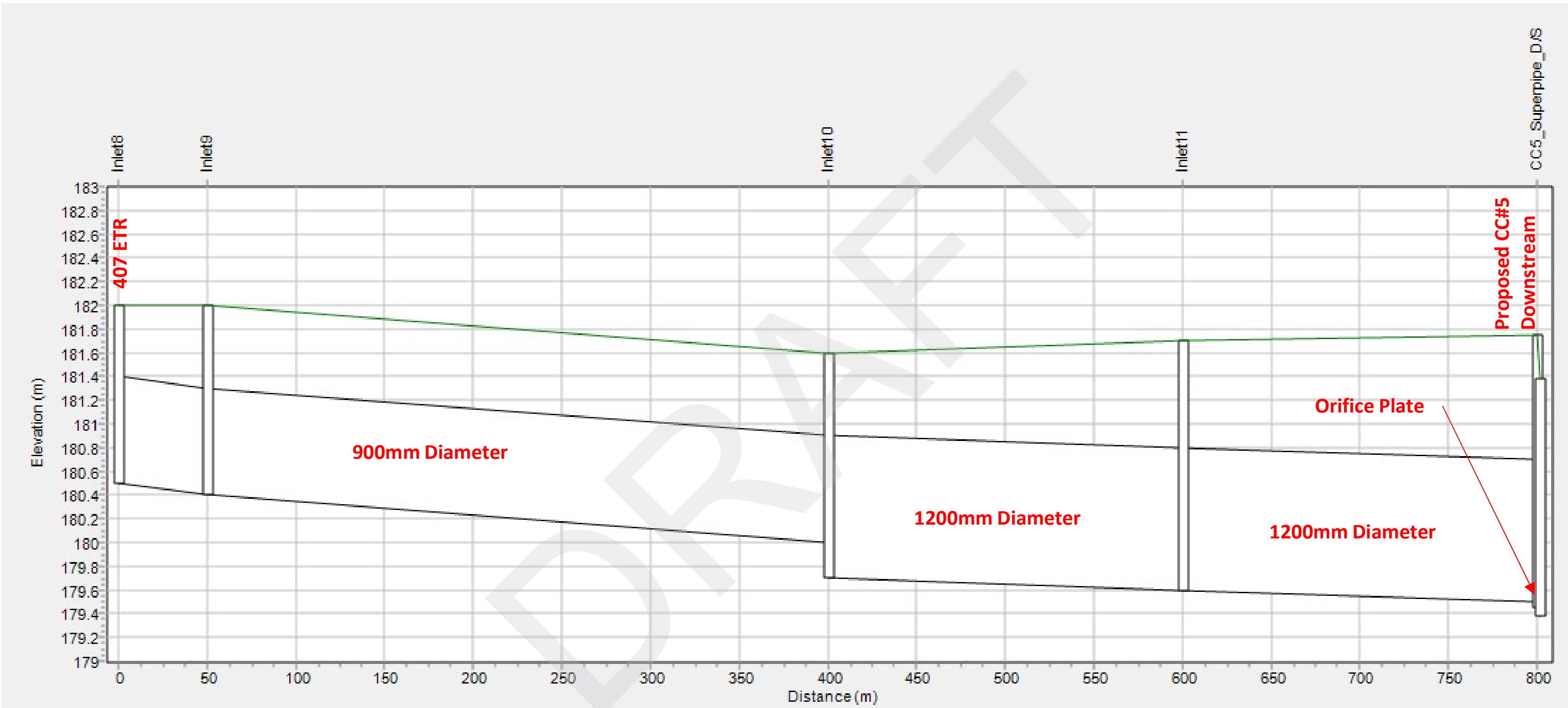
Figure 8: Infiltration Trench Schematic

NOT TO SCALE



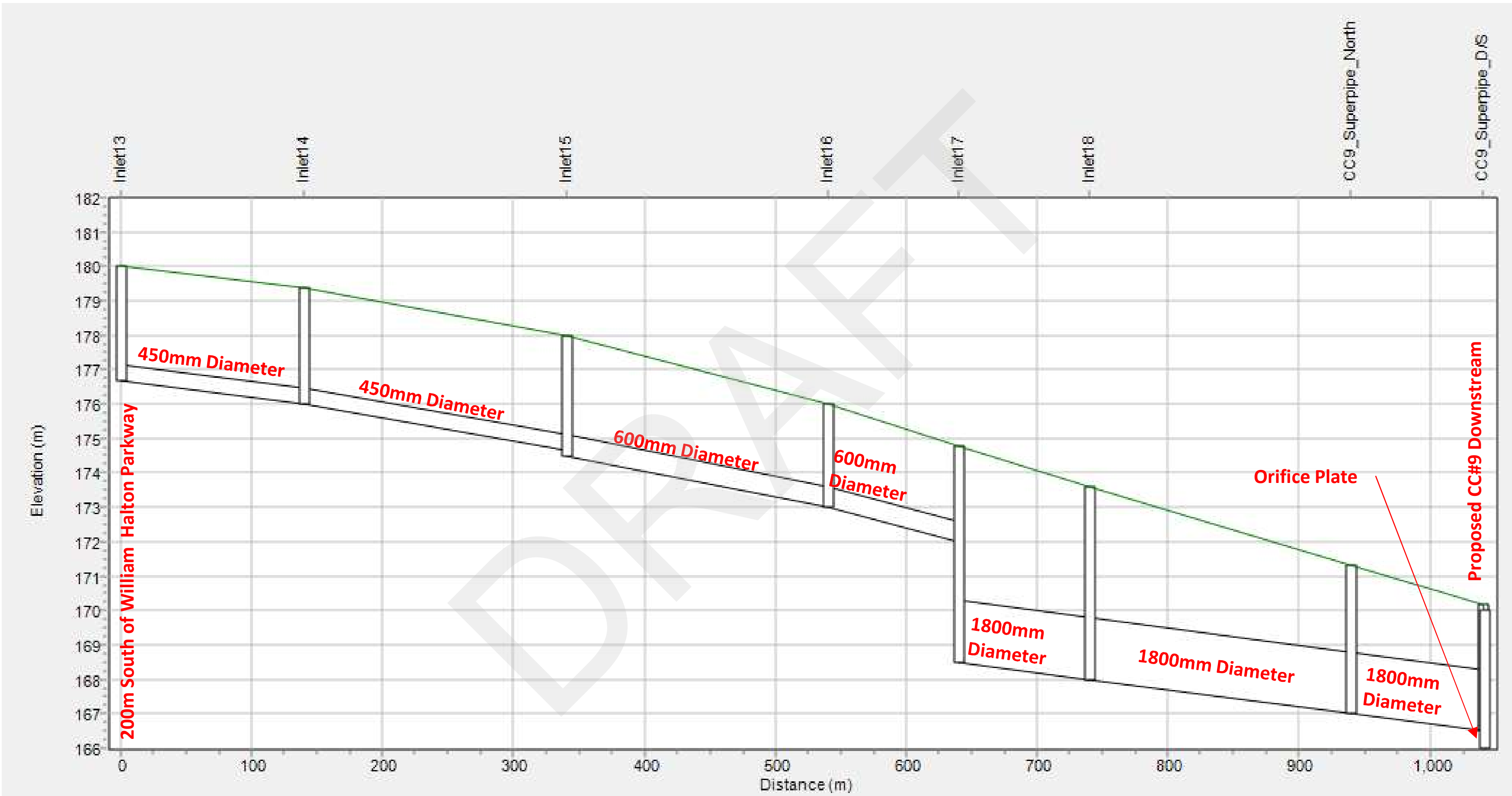
Project: 1648031	Golder Associates	26-May-17

Figure 9: Proposed Storm Sewer Profile North of William Halton Parkway



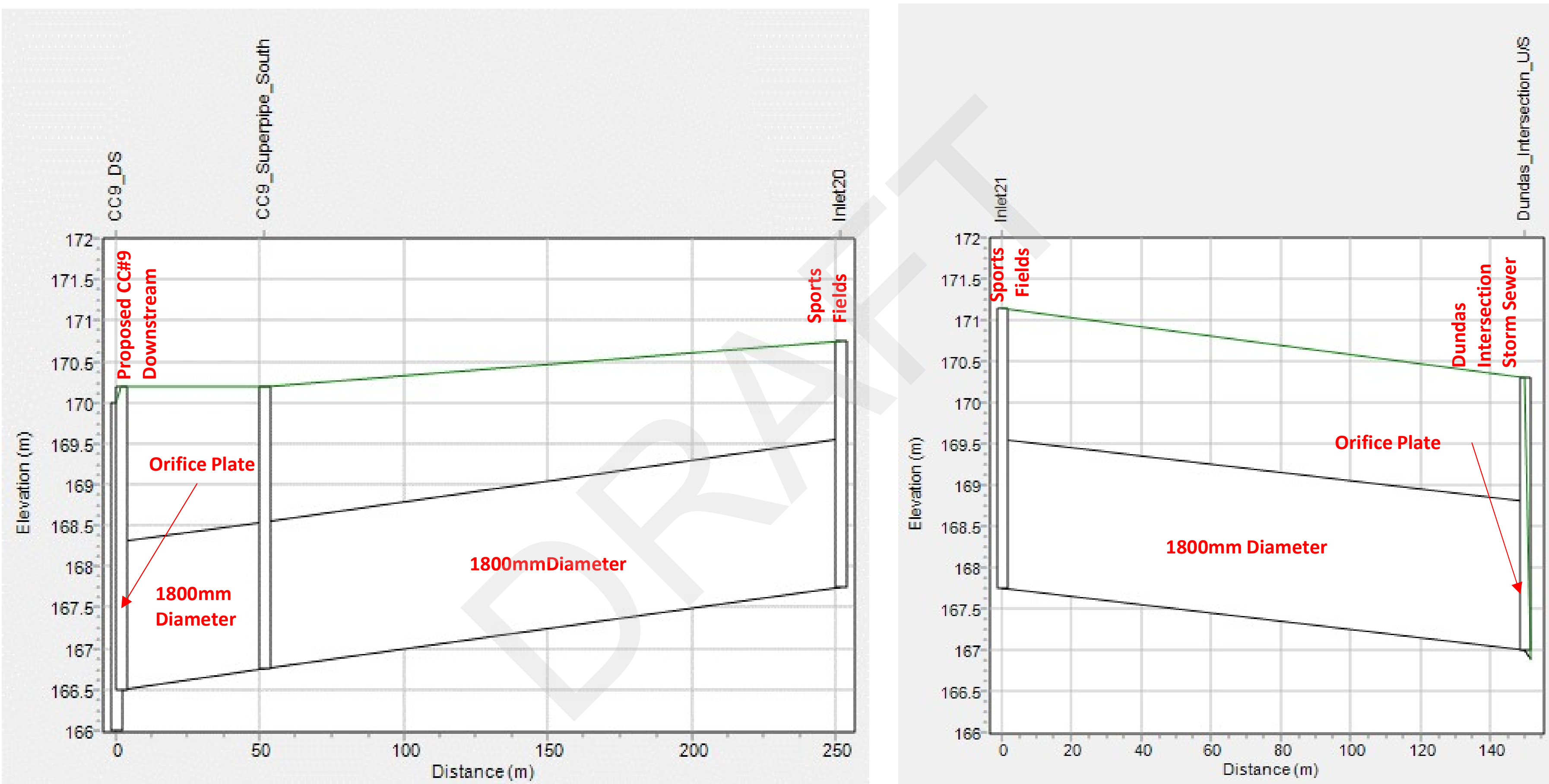
Project: 1648031	Golder Associates	26-Aug-19

Figure 10: Proposed Storm Sewer Profile South of William Halton Parkway

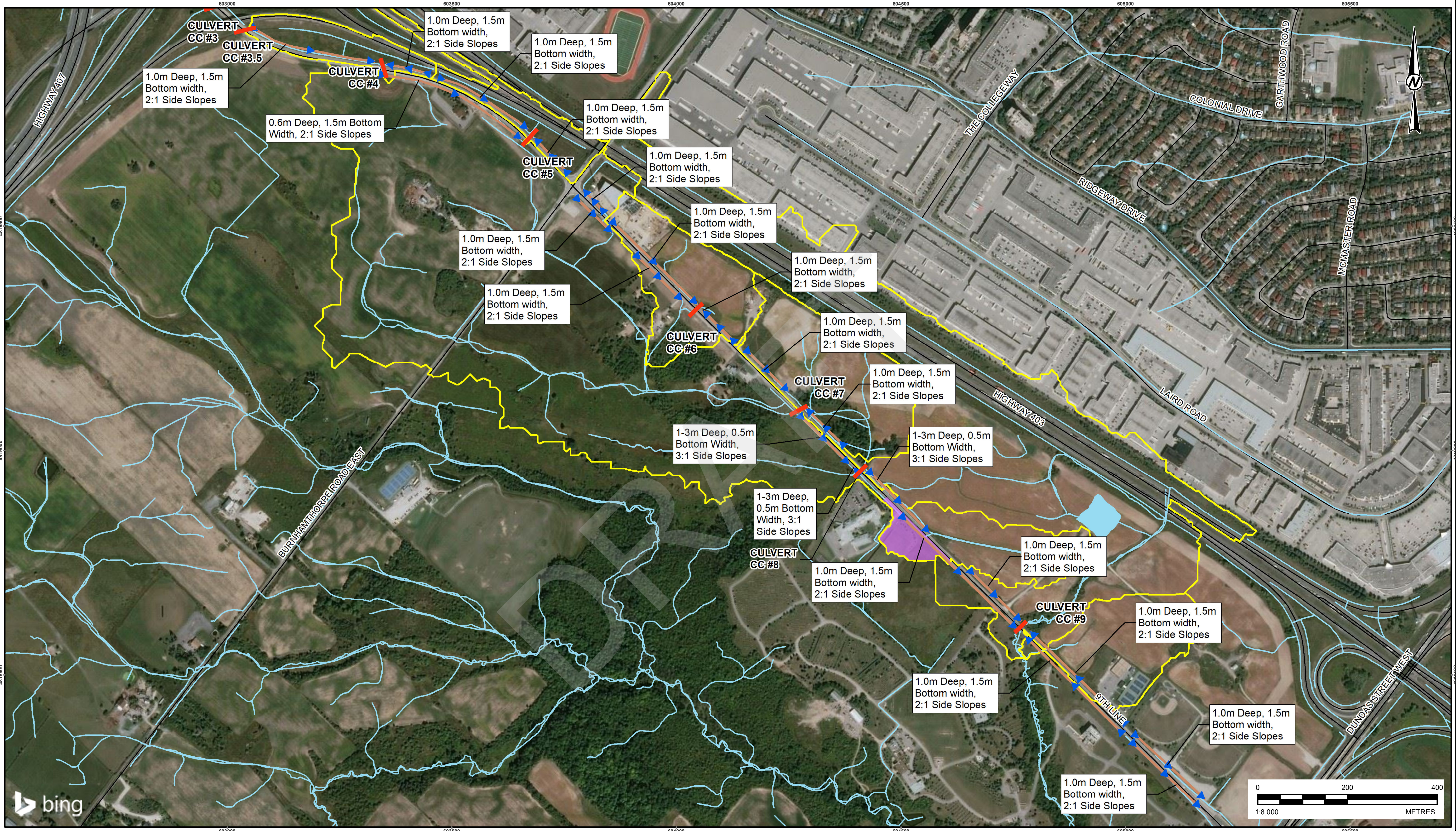


Project: 1648031	Golder Associates	26-Aug-19

Figure 11: Proposed Storm Sewer Profile Between CC#9 and Dundas St.



Project: 1648031	Golder Associates	26-Aug-19



LEGEND

- CULVERT (APPROXIMATE)
- DITCH AND FLOW DIRECTION
- WATERCOURSE
- ROAD
- WATERCOURSE
- WATERBODY
- REDIRECTED FLOW TO DOWNSTREAM END OF CC#9
- CATCHMENT

REFERENCES
BASE DATA - MNR LIO, OBTAINED 2016
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PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17

CLIENT
REGION OF HALTON

CONSULTANT
GOLDER

YYYY-MM-DD	2019-08-23
PREPARED	JR
DESIGN	JR
REVIEW	
APPROVED	

PROJECT
STORMWATER MANAGEMENT REPORT FOR HALTON 9TH LINE

TITLE
PROPOSED CROSSING CULVERTS AND CATCHMENTS

PROJECT NO.	PHASE	REV.	FIGURE
1648031	0003	A	12

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: 25mm

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