Lakeshore Road Transit Project Assessment Process (TPAP) and Preliminary Design

Part A - Etobicoke Creek to East Avenue

Drainage and Stormwater Management Report

City of Mississauga

May 24, 2023

Client Project Team

Project Manager	Gino Dela Cruz, City of Mississauga

HDR Project Team

Project Manager	Andrew Shea, P.Eng.
Technical Team	Dante Mawji, P.Eng.
	Janice Look, P.Eng.
	Mahsa Khodadadi, EIT
Quality Control	Anthony Reitmeier, P.Eng.

Disclaimer

The material in this report reflects HDR's professional judgment considering the scope, schedule and other limitations stated in the document and in the contract between HDR and the client. The opinions in the document are based on conditions and information existing at the time the document was published and do not consider any subsequent changes.

In preparing this report, HDR relied, in whole or in part, on data and information provided by the Client and third parties that was current at the time of such usage, which information has not been independently verified by HDR and which HDR has assumed to be accurate, complete, reliable, and current. Therefore, while HDR has utilized its best efforts in preparing this report, HDR does not warrant or guarantee the conclusions set forth in this report which are dependent or based upon data, information or statements supplied by third parties or the client, or that the data and information have not changed since being provided in the report. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that HDR shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party resulting from decisions made or actions taken based on this document.

Contents

Tables

Figures

Appendices

This page is intentionally left blank.

1 Introduction

The City of Mississauga's Transportation and Works Department intends to build upon the recent completion of the Lakeshore Road Transportation Master Plan and Implementation Strategy (2019) that was carried out under the Municipal Class Environmental Assessment (EA) process and complete the outstanding Class EA Study processes and approvals for the Lakeshore Corridor. Part A of this assessment includes Transit Project Assessment Process (TPAP) and Preliminary Design for a two (2) km section of Lakeshore Road East from Etobicoke Creek to East Avenue. HDR has been retained by the City of Mississauga to conduct the Lakeshore Road TPAP study and preliminary design.

This Drainage and Stormwater Management Report has been prepared in support of the TPAP study and complies with the Ministry of the Environment, Conservation and Parks (MECP), Credit Valley Conservation (CVC), Toronto and Region Conservation Authority (TRCA), and the City of Mississauga's policies, regulations, and standards. The Lakeshore Road TPAP Part A study corridor is illustrated in **Figure 1-1**.

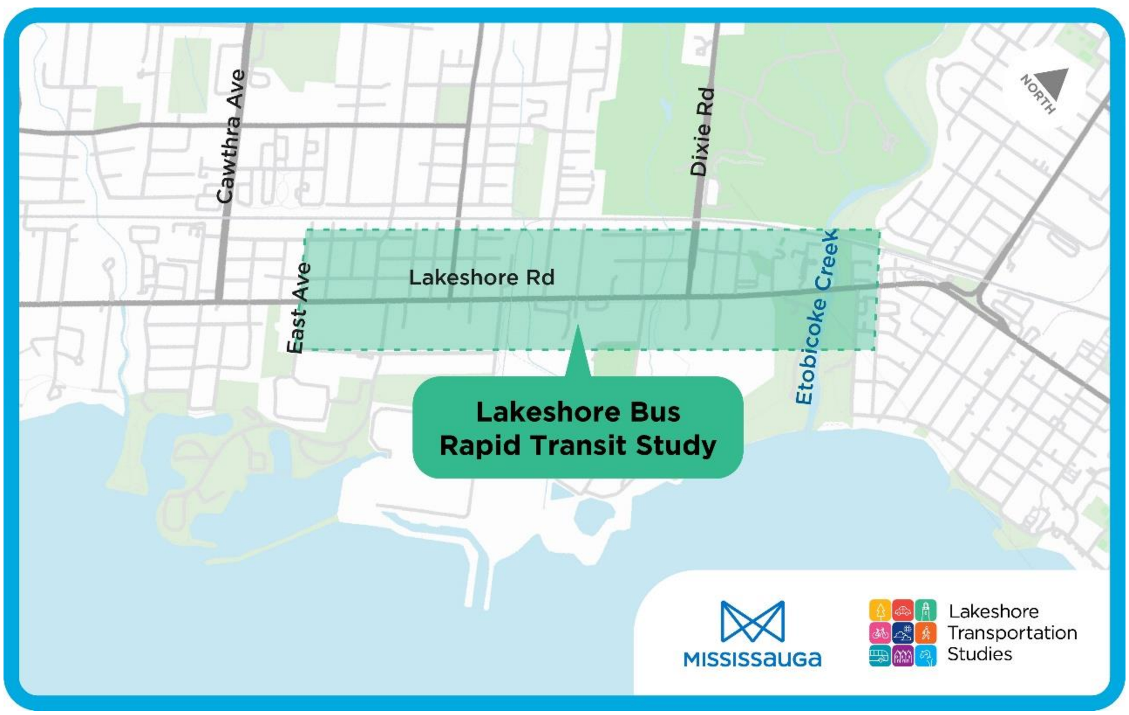


Figure 1-1: Study Corridor

The Lakeshore Road TPAP Part A study corridor spans approximately 2.3 km in the City of Mississauga. Lakeshore Road is an urban two-lane east-west arterial road and intersects with several existing and future local roads and entrances within the project limits. The existing right-of-way and land use vary throughout the corridor.

There are two regulated watercourses that cross Lakeshore Road within the project limits, namely Applewood Creek and Serson Creek. These watercourses are regulated by the Credit Valley Conservation authority (CVC). Refer to the Drainage Area Plans in **Appendix A** for the location of the crossings.

The objective of the Drainage and Stormwater Management Report is to develop a strategic approach to the development of the proposed project that will:

- Review available drainage information for existing conditions, including storm drainage area plans, reports and previous studies, plan-and-profile drawings and hydraulic models;
- Identify and evaluate existing drainage patterns and transverse watercourse crossings;
- Identify and evaluate the existing stormwater and drainage conditions in the study area, including sensitive areas and potential issues;
- Establish design criteria for stormwater management to meet the requirements of the various authoritative bodies; and
- Identify potential stormwater runoff quality and quantity impacts to the receiving watercourses/ storm sewer systems resulting from changes to the roadway cross-section (i.e. increased pavement area) and develop a mitigation strategy.

1.1 Background information

In preparation of the Lakeshore Road TPAP Drainage and Stormwater Management Report, the following essential documents were obtained and reviewed:

1. City of Mississauga Storm Drainage Design Requirements, January 2020;
2. Ministry of the Environment, Conservation and Parks (MECP) Stormwater Management Practices Planning and Design Manual, March 2003;
3. Credit Valley Conservation (CVC) Stormwater Management Criteria, August 2012;
4. Credit Valley Conservation (CVC) Technical Guidelines for Watercourse Crossings, 2019;
5. Toronto and Region Conservation Authority (TRCA) Stormwater Management Criteria, August 2012;
6. Sustainable Technologies Evaluation Program (STEP) Low Impact Development Stormwater Management (LID SWM) Planning and Design Guide, 2020;
7. Toronto and Region Conservation Authority (TRCA) Erosion and Sediment Control Guide for Urban Construction, 2019;
8. Natural Environment Assessment Report for the Lakeshore Transportation Studies, Matrix Solution Inc., November 2021;
9. Existing Condition Report for the Lakeshore Connecting Communities, City of Mississauga, HDR, October 2016;
10. Draft Geotechnical Investigation for the Transit Project Assessment Process (TPAP) for Lakeshore Road – Part A – Pavement (Etobicoke Creek to East Avenue), April 2023;
11. Class Environmental Assessment for Culvert and Creek Improvements on Lakeshore Road East Over Applewood Creek, AECOM, June 2015;
12. Biennial Inspection Report for Lakeshore Road East Over Applewood Creek Culvert, Engineered Management System, September 2019;

13. Biennial Inspection Report for Lakeshore Road East Over Serson Creek Culvert, Engineered Management System, September 2019; and
14. MECP Response to Notice of Commencement Letter dated October 12, 2021.

2 Existing Drainage Conditions

2.1 Watershed and Subwatershed

The Credit Valley Conservation Authority (CVC) has jurisdiction with respect to drainage and stormwater management of the Credit River watershed within the majority of the Lakeshore Road TPAP study corridor. Within this watershed, the study area crosses the Lake Ontario Shoreline East subwatershed. The far eastern portion of the study corridor is located within the Toronto and Region Conservation Authority (TRCA) jurisdiction (Etobicoke Creek watershed); therefore, the TRCA criteria for stormwater management will be applied to the catchment that is draining to Etobicoke Creek.

The study area also falls under the jurisdiction of the Ministry of Natural Resources and Forestry (MNRF) Aurora district. There are two (2) regulated watercourse crossing within the project limits, both located within the Credit River watershed. Refer to the Drainage Plans in **Appendix A** for the crossing locations.

2.2 Land Use

Based on the site investigation and the available background information, the existing land use along Lakeshore Road East varies along the study corridor and includes mixed used properties, residential lots, commercial areas, park space and watercourse valley lands.

2.3 Hydrogeological Conditions

A Geotechnical Investigation was completed for the Lakeshore Road TPAP Part A by Frontop Engineering Ltd. in April, 2023. As part of this investigation, forty-two (42) boreholes were drilled, and eighteen (18) monitoring wells were installed within the study corridor to measure groundwater levels and soil material properties.

Based on the information from the Geotechnical Investigation Report (dated April 17, 2023), the soil material at the locations where low impact development (LID) measures are proposed can be classified as sandy silt, clayey silt, and silty clay. As a conservative approach, the soil with the lowest estimated hydraulic conductivity was used for further calculation. The estimated hydraulic conductivity of silty clay is 0.51 mm/hr, or 1.42×10^{-5} cm/s (Rawls, W.J. et al., 1983)[1]. This approximately corresponds to an infiltration rate of 31 mm/hr, as per Table C1 in Appendix C of the CVC/TRCA LID SWM Planning and Design Guide (2010). A safety correction factor of 3.0 was applied to estimate the soil infiltration rate at the base of the proposed BMPs. Accordingly, the percolation rate of the native soil is estimated to be 10.3 mm/hr.

As part of the Geotechnical Investigation, groundwater levels were measured from the monitoring wells in November 2022, February 2023, and March 2023. Based on the measurements, the groundwater level ranges from approximately 1.1 m to 2.5 m below the existing ground surface where LID measures are generally proposed. Throughout the entire project corridor, the groundwater levels range from 1.1 m to 3.0 m below the ground surface where groundwater was observed.

[1] Rawls, W.J., Brakensiek, D., & Miller, N. (1983). Green-ampt Infiltration Parameters from Soils Data. Journal of Hydraulic Engineering, 109, 62-70.

After the geotechnical investigation is completed, the groundwater level and soil types will be confirmed. During the detailed design stage, borehole investigations and in-situ infiltration rate measurements should be completed at all proposed LID locations to confirm the soil infiltration rates and groundwater levels.

2.4 Existing Drainage Pattern

Lakeshore Road between Etobicoke Creek and East Avenue has primarily an urban cross-section and stormwater runoff is primarily managed by an underground storm sewer system. The corridor runs along the natural drainage gradient towards the east. The majority of the corridor directly discharges either to Serson Creek, Applewood Creek, or Etobicoke Creek via storm sewer outfalls. The remainder of the corridor ultimately discharges into Lake Ontario. Refer to the Drainage Plans in **Appendix A** for additional details.

The approximate location and catchment areas for each drainage area in the existing conditions are summarized in **Table 2-1.**

Table 2-1: Summary of Existing Drainage Areas

Drainage Area ID	Description	Drainage Area (ha)	From Station	To Station	Discharge Location
A-1	From 13 m east of West Avenue to 33 m west of Alexandra Avenue	1.18	9+718	10+194	Existing storm sewer system on Goodwin Road (ultimate discharge to Lake Ontario)
A-2	From 80 m east of West Avenue to 33 m west of Alexandra Avenue	0.82	9+780	10+194	Existing storm sewer system on East Avenue South (ultimate discharge to Lake Ontario)
A-3	From 33 west of Alexandra Avenue to Meredith Avenue	0.92	10+194	10+385	Existing storm sewer system on Lakefront Promenade (ultimate discharge to Lake Ontario)
A-4	From Meredith Avenue to 95 m east of Orchard Road	3.11	10+385	11+215	Serson Creek
A-5	From 5 m east of Ogden Avenue to Hydro Road	0.42	10+575	10+795	Existing storm sewer system on Hydro Road (ultimate discharge to Lake Ontario)
A-6	From 95 m east of Orchard Road to Deta Road	2.81	11+215	11+805	Applewood Creek
A-7	From Deta Road to Etobicoke Creek	2.01	11+805	12+180	Etobicoke Creek

2.4.1 External Areas

Existing catchment areas and outlet locations along the corridor are identified in **Appendix A**. There are several storm sewer connections from areas outside the Right-of-Way to the storm sewer system along the corridor, as shown in the Drainage Area Plans in **Appendix A**.

2.5 Aquatic Resources

The two watercourses that exist within the project limits, namely Serson Creek and Applewood Creek, are within the Credit River watershed and are under the jurisdiction of Credit Valley Conservation's (CVC) and the Ministry of Natural Resources and Forestry (MNRF) Aurora district.

Applewood Creek is classified as a warmwater system, which contains a pollution tolerant mix of cyprinid species. The fish species within Applewood Creek are common and secure in Ontario and no

SAR (Species At Risk) or SCC (Species of Conservation Concern) are identified. Fish survey competed by CVC in 2011 and 2021 did not yield any fish species in Serson Creek.

A portion of the study corridor discharges to Etobicoke Creek, which is under the jurisdiction of TRCA and MNRF Aurora District. The Etobicoke Creek is a warmwater system and the fisheries data shows except for one SAR species (American Eel), the fish species within Etobicoke Creek are common and secure in Ontario.

2.6 Transverse Drainage Crossings

There are two (2) regulated watercourse crossings within the project limits, which are culvert crossings at Serson Creek and Applewood Creek. In addition, there are two unregulated culvert crossings within the project limits that convey the local drainage.

A summary of the size, type, and location of the existing culvert structures can be found in **Table 2-2**. Refer to the Drainage Plans in **Appendix A** for additional details.

Table 2-2: Summary of Transverse Crossings

Crossing (Watercourse)	Crossing Location	Culvert Dimensions (Span x Rise, mm)	Culvert Description	Culvert Length (m)
Local Drainage	East of Greaves Avenue	600 x 900	CSP Culvert	17.87
Local Drainage	West of Alexandra Avenue	600 x 900	CSP Culvert	21.75
Serson Creek	0.1 km west of Haig Boulevard	8100 x 1400	Concrete Box Culvert	27.56
Applewood Creek	0.15 km east of Fergus Road	2-3100 x 1200	Twin Concrete Box Culverts	32.40

2.6.1 Existing Condition Summary

Biennial Inspection Report for Lakeshore Road East culverts by Engineered Management Systems Inc. (September 2019), provides detailed information regarding the condition of the crossing structures at Serson Creek and Applewood Creek.

The structural conditions of the culverts at the time of inspection are described in **Table 2-3**.

Table 2-3: Summary of Transverse Crossings Condition Assessment

Crossing (Watercourse)	Material/ Crossing Type	Overall Condition[1]
Serson Creek	Concrete Box Culvert	Good condition
Applewood Creek	Twin Concrete Box Culverts	Good condition

Note 1: Per Inspection Report by Engineered Management Systems Inc. (September 2019)

2.6.2 Assessment Criteria

Hydraulic assessments of the transverse crossings within the project limits were undertaken in accordance with the City of Mississauga Storm Drainage Design Requirements (2020).

Design Flows

Based on the City of Mississauga Storm Drainage Design Requirements (2020), the design for arterial roads is the larger of the 100 year or Regional Storm events flows.

Minimum Culvert Sizes

The minimum culvert size for a driveway culvert is 300 mm in diameter with a precast headwall as per the City of Mississauga's Storm Drainage Design Requirements.

2.6.3 Hydraulic Assessment of Transverse Crossings

The design peak flows for the culvert crossings were obtained from the existing hydraulic models (HEC-RAS) for Serson Creek and Applewood Creek provided by CVC. The hydraulic model (HEC-RAS) provided by CVC for Applewood Creek was developed for the preferred option by AECOM in 2015, as part of an EA study for culvert and creek improvements on Lakeshore Road East over Applewood Creek. This model is updated and used as the base condition for this study. There are no existing hydraulic models for the two unregulated crossings within the project limits.

It is recommended that during detailed design, the design flows be reviewed and verified to confirm any changes to the land-use, channel geometry and associated hydrologic information that may affect the peak flows presented in this study.

A summary of the design storm peak flows for the transverse crossings is presented in **Table 2-4**.

Table 2-4: Design Peak Flows - Transverse Crossings

Watercourse/ Drainage Crossing	Type	Peak Flow (m^3/s)		
		50 Yr	100 Yr	Regional
Serson Creek	Culvert	16.7	19.2	19.1
Applewood Creek	Culvert	43.1	51.3	53.4

A hydraulic assessment of the regulated culvert crossings was conducted to determine the hydraulic performance under the existing conditions.

The culvert capacities were assessed based on the 100 year and Regional design storm as per the City of Mississauga Storm Drainage Design Requirements. There is a 1600 mm trunk sanitary sewer crossing Serson creek just downstream of the culvert. The existing Serson Creek hydraulic model was updated to include this sewer as an obstruction within the channel. **Table 2-5** summarizes the hydraulic analysis results for the crossings within the project limits. All hydraulic assessment output files are provided in **Appendix B**.

Table 2-5: Hydraulic Analysis Results for the Transverse Culverts (Existing Condition)

Crossing	Type	U/S Invert (m)	D/S Invert (m)	Length (m)	Road Elev. (m)	Water Surface Elev. (m)			Remarks
						50 Yr	100 Yr	Reg.	
Serson Creek	Culvert	81.69	81.54	27.56	84.50	83.38	83.53	83.52	100 year and Regional flows do not overtop road
Applewood Creek	Culvert	78.85 79.15	78.41 78.71	28.02	83.68	81.84	82.26	82.38	100 year and Regional flows do not overtop road

The results presented in **Table 2-5** indicate that the 100 year and Regional Storm events do not overtop the road at Serson Creek and Applewood Creek crossings.

3 Proposed Drainage Conditions

3.1 Roadway Drainage System

The preferred alternative design concept for Lakeshore Road East from Etobicoke Creek to East Avenue recommends widening the road, as well as the addition of an exclusive transit median and in-boulevard cycle tracks and sidewalks on both sides of the road. The design concept also includes intersection improvements at all signalized intersections and streetscaping opportunities along the corridor. As part of the proposed roadway design, localized high points and low points are introduced in the roadway profile. Overall, the existing drainage patterns will not be altered as per the proposed roadway improvements, except for minor localized changes as a result of the proposed roadway profile and widening. However, some existing discharge locations will be redirected as the result of replacing the existing drainage swales located south of Lakeshore Road with underground storm sewers.

The approximate location and catchment areas for each drainage area in the proposed conditions are summarized in **Table 3-1.**

Table 3-1: Summary of Proposed Drainage Areas

Drainage Area ID	Description	Drainage Area (ha)	From Station	To Station	Discharge Location
B-1	From 13 m east of West Avenue to 33 m west of Alexandra Avenue	2.00	9+718	10+194	Existing storm sewer system on Goodwin Road (ultimate discharge to Lake Ontario)
B-2	From 33 west of Alexandra Avenue to Meredith Avenue	0.92	10+194	10+385	Existing storm sewer system on Lakefront Promenade (ultimate discharge to Lake Ontario)
B-3	From Meredith Avenue to 95 m east of Orchard Road	3.53	10+385	11+215	Serson Creek
B-4	From 95 m east of Orchard Road to Deta Road	2.81	11+215	11+805	Applewood Creek
B-5	From Deta Road to Etobicoke Creek	2.01	11+805	12+180	Etobicoke Creek

3.1.1 Minor Drainage System

The overall drainage pattern will generally be consistent with the existing conditions. To accommodate the proposed roadway widening, storm sewer upsizing and catchbasin relocations are anticipated. The existing drainage swales located south of Lakeshore Road will be replaced by underground storm sewers.

The storm sewer system for the ultimate roadway configuration is to be designed at the detailed design stage for a 10-year storm event as per the City of Mississauga Storm Drainage Design Requirements. Roadway drainage will be collected by a series of catchbasins and will be conveyed by storm sewers to the existing storm outlet locations. There are several existing outlets for the runoff from Lakeshore Road East within the project limits. For the existing storm sewer discharge locations, refer to the Drainage Plans in **Appendix A**.

3.1.2 Major Drainage System

The roadway design should ensure that the major system runoff up to the 100-year storm event can be safely conveyed to outlet locations, and the depth of water shall not exceed the crown of the road, as per City of Mississauga Storm Drainage Design Requirements. At these locations, major system inlets will capture the 100-year flow and direct it to the appropriate outlet. A spread analysis should be completed at the detailed design stage to ensure that the ponding at low points does not exceed the crown of the road.

For major system flow directions, refer to the Drainage Plans in **Appendix A**.

3.2 Transverse Crossings

There are two (2) regulated watercourse crossings within the study corridor, which are culvert crossings at Serson Creek and Applewood Creek. The proposed size, structure and location of the crossings were determined based on the existing structures condition assessment, fluvial geomorphologic assessment, proposed roadway geometry, grading impacts, and hydraulic performance. Extension of the culvert at Applewood Creek and replacement of the culvert at Serson Creek crossing is required to accommodate the proposed roadway modifications. The objective of this assessment is to evaluate the potential impact of the proposed extensions on the hydraulic capacity of the culverts. Details regarding the hydraulic modelling of the two transverse crossings are provided in the Hydraulic Modelling Memo for Applewood and Serson Creeks in **Appendix C**.

A summary of the recommended approach for upgrades at the watercourse crossing is provided in **Table 3-2**.

Table 3-2: Transverse Crossing Recommendations

Crossing (Watercourse)	Location	Recommendations for Culvert Upgrades
Serson Creek	Sta. 21+700	**Culvert replacement** is required to accommodate roadway improvements. The required replacement is 47.00 m long and 11 meters wide.
Applewood Creek	Sta. 22+265	**Culvert extension** is required to accommodate roadway improvements. The required extension is 12.0 m on south of the crossing, for a total culvert length of 40.02 m.

3.2.1 Hydraulic Assessment of Proposed Transverse Crossings

Under proposed conditions, the drainage boundary and design peak flow values for the transverse crossings are considered to remain unchanged compared to the existing conditions. The increase in the pavement area as a result of the corridor improvements is very small in comparison to the large external drainage areas contributing to the watercourse crossing location. Therefore, the design peak flows based on the current land use conditions (obtained from CVC's HEC-RAS model) were used to assess the hydraulic performance of the proposed crossings.

The hydraulic assessments for the proposed crossings are based on the preliminary proposed horizontal road design and vertical structure profile. Note that the proposed inverts of the crossing culverts are to be confirmed during detailed design to accommodate the road design and the roadside ditch grading.

Serson Creek Crossing

Under proposed conditions, the existing concrete box culvert is recommended to be replaced to accommodate the proposed roadway widening. The existing 1600 mm trunk sanitary sewer crossing Serson Creek just downstream of the existing culvert will be located inside the proposed culvert and create an obstruction within the culvert. While relocating this sewer outside the culvert, so that it will not obstruct the flow and reduce the culvert's hydraulic capacity was the primary recommendation, it has been determined through engagement with stakeholders that this is not a feasible option. To evaluate its impact, the existing hydraulic model provided by CVC was updated to allow modeling of the proposed culvert with the obstruction, which required an unsteady-state simulation. Additionally, the existing hydraulic model provided by CVC was updated to include additional channel geometry details, including additional cross sections, spatial modifications of existing cross sections, and adjustments to the channel geometry.

The hydraulic modeling results show that replacing and upsizing the culvert will result in a decrease of 0.03 m in the immediate upstream 100-year and Regional flood levels as shown in **Table 3-3**. Under existing and proposed conditions, the 100 year and Regional Storm events do not overtop Lakeshore Road at the Serson Creek crossing.

The proposed culvert will result in an increase in channel velocities immediately upstream of the crossing. Adequate erosion protection measures should be designed in the detailed design stage to mitigate the increased erosion hazard.

Applewood Creek Crossing

Under proposed conditions, the existing twin concrete box culvert is recommended to be extended to accommodate the proposed roadway widening. The existing hydraulic model provided by CVC was updated to include additional channel geometry details required to model the proposed culvert extensions. These updates include additional cross sections, spatial modifications of existing cross sections, and adjustments to the channel geometry.

The hydraulic modeling results show that extending the length of the culvert to accommodate the proposed road widening will result in an increase of 0.07 m in the immediate upstream 100 year and Regional flood level as shown in **Table 3-3**. This increase in water surface elevation is transient and entirely contained by the channel valley banks, resulting in no additional flooding impact to adjacent properties. Under existing and proposed conditions, the 100 year and Regional Storm events do not overtop Lakeshore Road at Applewood Creek crossing.

Table 3-3: Hydraulic Analysis Results for the Transverse Crossings (Proposed Conditions)

Crossing	Type	Road Elev. (m)	U/S XS ID	Immediate U/S Water Surface Elev. (m)						Remarks
				Updated Ex.			Proposed			
				50 Yr	100 Yr	Reg.	50 Yr	100 Yr	Reg.	
Serson Creek	Culvert	84.50	11137	83.36	83.50	83.49	83.27	83.47	83.46	100 year and Regional flows do not overtop road
Applewood Creek	Culvert	83.67	10914	81.84	82.26	82.38	81.89	82.33	82.45	100 year and Regional flows do not overtop road

4 Stormwater Management Strategy

4.1 Stormwater Management Criteria

The stormwater management plan for the study area shall be developed to comply with the policies, regulations, and standards of Credit Valley Conservation (CVC), Toronto and Region Conservation Authority (TRCA), Ministry of Environment, Conservation and Parks (MECP), and City of Mississauga.

4.1.1 Water Quality Control

Watercourses within the CVC and TRCA's jurisdiction are classified as requiring an "Enhanced" level of protection, which equates to 80% Total Suspended Solids (TSS) removal.

Water quality management measures within the study limits will be designed to provide "Enhanced" water quality treatment, as a minimum, for the increased pavement area as a result of roadway widening/improvements, as per the MECP Response to Notice of Commencement Letter dated October 12, 2021.

4.1.2 Water Quantity Control

Storm Sewer Systems

Within the project limits, the stormwater runoff from Lakeshore Road East discharges either into the existing storm sewer systems or outlets at the watercourse crossings. For locations where the runoff discharges into an existing system, the minor system design storm (10-year storm) peak flows must be controlled to the existing peak flows, for which the receiving system was designed. The receiving storm sewer systems within the project limits are City of Mississauga municipal systems, which would have been designed based on a 10-year design storm.

Watercourse Crossings

CVC and TRCA has established quantity control targets for the watersheds under their jurisdiction. For the storm outlets at Serson Creek and Applewood Creek, CVC requires 100-year post-development peak flows to be controlled to 2 year pre-development levels.

For the storm outlets at Etobicoke Creek at Lakeshore Road, quantity control is not required according to the TRCA Stormwater Management Criteria (2012).

4.1.3 Water Balance and Erosion Control

The CVC and TRCA criteria for water balance and erosion control requires retention of 5 mm of rainfall. This criterion is applicable to increased pavement area as a result of roadway widening/improvements.

4.2 Hydrologic Modeling

A hydrologic analysis was conducted using the Rational Method to calculate the surface runoff under the 2- to 100-year storm events for both the existing and proposed condition scenarios. The Modified Rational Method was then be used to calculate the storage volumes required to control the post-development peak flows for the design storm events to the allowable release rates.

City of Mississauga Intensity Duration Frequency (IDF) curves were applied to calculate the peak flows under both existing and proposed conditions, using a minimum inlet time (T_c) of 15 minutes.

4.3 Pavement Area Analysis

A pavement area analysis was performed to determine the increase in impervious surface, which will result from the roadway widening, the addition of exclusive transit median, and construction of new cycle tracks and sidewalks.

As a Low Impact Development measure, it is recommended that the boulevard and median areas outside of the transit and active transportation facilities be covered with permeable material (e.g. grass, permeable pavement, etc.) to minimize the overall increase in impervious area along the Lakeshore Road corridor. Since these are not load bearing surfaces, the use of permeable material will not impact the functionality of the proposed design but will provide water quality and quantity control benefits through runoff reduction. Therefore, the proposed stormwater strategy was developed considering the boulevard and median areas outside of the transit and active transportation facilities as pervious. Additional details and specifications for the permeable material are to be included in the detailed design stage.

It was determined that the proposed roadway improvements will result in an additional 2.52 hectare (34%) increase in pavement area within the Lakeshore Road study corridor. The results are summarized in **Table 4-1**.

Table 4-1: Pavement Area Analysis

Study Corridor	Existing Pavement Area (ha)	Proposed Pavement Area (ha)	Increased Pavement Area (ha)	Percentage Increase
Lakeshore Rd.	7.38	9.59	2.52	34%

4.4 Stormwater Best Management Practice Options

Various Best Management Practices (BMPs) for stormwater management were reviewed and assessed for their applicability on this project. Due to the nature of this facility (i.e. linear transportation corridor) and the limited space within the roadway right-of-way, a series of bioretention cells integrated with the proposed streetscaping are proposed to provide quality treatment, erosion control, and water balance. To provide quantity control throughout the Lakeshore Road corridor, online storage pipes are proposed.

4.4.1 Bioretention Cells

Bioretention systems allow for stormwater filtration, infiltration and evapotranspiration from trees and vegetative plantings. For roadway applications, these can take the form of sub-surface modular units that are filled with lightly compacted soil within a trench situated beneath the roadway boulevard areas. The trench unit consists of a filter bed which is a mixture of sand, fines and organic material to support vegetation and promote evapotranspiration by allowing surface runoff to route through a distribution pipe via gravity within the trench. Soil filtration, bioremediation, infiltration, and evapotranspiration will occur as water filtrates through the soil from the perforated distribution pipe.

The facility will also be lined with geotextile fabric and clean granular fill (50 mm clear stone) will be placed below the filter bed for storage and infiltration of roadway runoff. In addition to removing TSS particles, the facility reduces water temperature impact and enhances water balance through infiltration. A perforated underdrain pipe can be incorporated in the granular layer for soils with low infiltration rate to collect and direct the excess runoff to an existing storm sewer system. The bioretention cell also contributes to controlling downstream erosion through extended detention and reducing flow velocities.

Discharging the runoff directly into the bioretention systems has the following advantages:

- Boulevard landscaping will receive a supply of rainwater during every rainfall event, thus sustaining their health and reducing maintenance requirements;
- Stormwater runoff from the roadways could potentially see significant detention within the bioretention systems, which will result in runoff reduction;
- Water quality treatment will be achieved since stormwater can be routed through the bioretention filter media; and
- For smaller rainfall events, the bioretention system can provide (in the long-term) for complete capture of the runoff through infiltration and evapotranspiration.

The design criteria specified in the SWM Planning and Design Guide (MECP, 2003) and LID SWM Planning and Design Guide (STEP, 2020) were applied to determine the depth and footprint area for the bioretention cells. The maximum allowable depth of the stone reservoir below the underdrain pipe can be calculated using the following formula:

$$d_{r\ max} = i * t_s / V_r$$

where i is the infiltration rate of the native soils, which was estimated to be 10.3 mm/hr within the project limits based on the Hydrogeological Investigation (**Section 2.3**); t_s is time to drain, which is recommended to be 48 hours; and V_r is void space ratio of the aggregate used, which is typically 0.4 for clear stone. Accordingly, the maximum allowable depth of the reservoir can be calculated to be d_{max} = 1236 mm.

For this project, 2.2 m wide bioretention cells with a 0.5 m filter bed layer, a 0.1 m pea gravel choking layer, and a 0.5 m deep gravel storage layer, including a 0.2 m diameter underdrain pipe, are proposed, for a total facility depth of 1.1 m. Conceptual plan and profiles of the proposed bioretention cells are provided in **Appendix D**. The footprint area of the bioretention cells can be calculated using the following formula:

$$A_f = WQV / (d_c * V_r)$$

where *WQV* is the required water quality volume to meet the ‘Enhanced’ level protection (80% TSS removal), which is determined based on the contributing drainage area and the imperviousness using Table 3.2 of the SWM Planning and Design Manual (MECP, 2003); d_c is the depth of the bioretention cell, and V_r is the void space ratio for the filter bed and gravel storage layer, which is typically 0.4. In addition to providing quality treatment, the provided gravel storage volume beneath the invert of the underdrain pipe will retain water to meet the water balance and erosion control targets. Additionally, the ratio of the impervious drainage area to footprint area of the bioretention cells should be between 5:1 and 20:1 to limit the rate of accumulation of fine sediments and thereby prevent clogging.

The bottom of the bioretention cells should be one (1) metre above the seasonally high goundwater table. Based on the results of the Geotechnical Investigation (**Section 2.3**) and the proposed roadway profile, the LID measures have been proposed in locations where a 1.0 m separation from the bottom of the proposed facilities can be attained. Further investigation should be completed during the detailed design stage to confirm adequate separation from the proposed facilities at each location and determine the percolation rate of the native soils using in-situ infiltration testing to ensure the maximum allowable depth of the reservoir is not exceeded.

The bioretention cells are proposed for all drainage areas. In addition to providing 'Enhanced' level protection (80% TSS removal), the provided storage volume within the bioretention cells includes the volume required to retain the first 5 mm of rainfall to meet the CVC and TRCA water balance and erosion control target. Pre-treatment of the runoff directed to the bioretention cells using catchbasin inserts (e.g. CB Shield) is recommended.

The bioretention cells are designed to provide water quality treatment for pavement areas equivalent to the total increase in pavement area along the Lakeshore Road corridor. Due to physical constraints of the roadway layout and limited space within the right-of-way, the required bioretention length for Drainage Area B2 and B3 cannot be accommodated in the design. Consequently, the bioretention length provided for B1, B4, and B5 are increased beyond the required length such that the total required bioretention cell length for water quality treatment across the corridor can be achieved.

Table 4-2 lists the details of the bioretention cells proposed along the Lakeshore Road corridor. For locations of the proposed bioretention cells, refer to the Drainage Plans provided in **Appendix A**. Detailed calculations are provided in **Appendix E**.

Table 4-2: Summary of Proposed Water Quality Treatment Strategy

Drainage Area ID	Proposed Pavement Area (ha)	Additional Pavement Area (ha)	Req'd Water Quality Volume (m^3)	Req'd Water Balance Storage[1] (m^3)	Prop-osed Length (m)	Treated Pavement Area[2] (m^2)	Provided Water Balance Volume[3] (m^3)	Req'd Erosion Control Volume[4] (m^3)	Provided Quality and Erosion Control Volume (m^3)
B1	1.68	0.54	31	27	220	0.54	58	108	213
B2	0.86	0.19	12	10	25	0.19	7	38	24
B3	3.31	1.05	65	53	145	1.05	38	211	140
B4	2.63	0.63	40	32	160	0.63	42	127	155
B5	1.10	0.10	5	5	80	0.10	21	20	77
Total	**9.59**	**2.52**	**153**	**126**	**630**	**2.52**	**166**	**504**	**610**

[1] Based on the retention of the first 5 mm of rainfall for increased pavement area
[2] Pavement area considered to receive water quality treatment
[3] Provided storage volume below underdrain
[4] Storage volume in addition to water balance volume to meet 25 mm retention for increased pavement area

Through the proposed water quality treatment strategy, a total of 2.52 ha of pavement area, which is the increase in pavement area across the Lakeshore Road study corridor, is considered to receive water quality control using the bioretention facilities. A total of 166 m^3 and 610 m^3 of water balance and water quality/erosion control storage volumes are respectively provided using the facilities, which exceeds the required storage volumes based on MECP and CVC/TRCA criteria. During detailed design, the location and performance characteristics of the bioretention facilities will need to be confirmed to ensure that all bioretention cell design criteria can be met.

4.4.2 Online Storage Pipes

At existing outlet locations, consideration should be given to providing over-sized storage pipes with flow control devices (e.g., orifice plate) upstream of the discharge location to provide peak flow control in combination with allowable surface ponding for major flows.

For locations where the runoff discharges into an existing system, the minor system design storm (10-year storm) peak flows must be controlled to the existing peak flows. In addition, CVC requires the 100-year post-development peak flows to be controlled to 2-year pre-development levels for the full range of storm events for Serson Creek and Applewood Creek. Due to the limited area available within the Lakeshore Road right-of-way, the storage required to meet the CVC criteria for Serson Creek and Applewood Creek cannot be provided. Therefore, as a best-efforts approach, the proposed peak flows will be controlled to their existing levels at these locations. The required storage is considered as the largest of the storage required to control the peak flow from all storm events, up to the 100-year storm event, to the existing levels. The required storage can be provided as a combination of underground storage and surface ponding for major flows. As indicated in the TRCA Stormwater Management Criteria (2012), no quantity control is required for Etobicoke Creek.

The required storage volumes to achieve the quantity control targets for each catchment are summarized in **Table 4-3**. Online storage pipes shall be designed in combination with surface ponding to provide the required storage in the detailed design stage. Detailed calculations are provided in **Appendix E**.

Table 4-3: Summary of Proposed Water Quantity Treatment Strategy

Drainage Area ID	Drainage Area (ha)	Existing Pavement Area (ha)	Additional Pavement Area (ha)	Required Storage to Control Minor Flows[1] (m^3)	Required Storage to Control Major Flows[2] (m^3)
B1	2.00	1.14	0.57	97	171
B2	0.92	0.67	0.18	22	38
B3	3.53	2.26	0.99	138	242
B4	2.81	2.00	0.51	72	126
B5	2.01	1.00	0.27	N/A	N/A
Total	**11.27**	**7.07**	**2.52**	**328**	**577**

[1] Based on the capacity of the receiving storm sewer system (up to 10-year storm)
[2] Based on controlling post to pre-development peak flow rates (up to 100-year storm)

Through the proposed water quantity control strategy, a total of 328 m^3 of storage volume will need to be provided to attenuate minor peak flows and a total of 577 m^3 will need to be provided to attenuate major peak flows to existing levels. During detailed design, the location, pipe sizing, and orifice sizing of the online storage pipes will need to be determined to ensure that the water quantity control criteria can be met.

4.4.3 Supplemental BMP Measures

Through discussions with MNRF, CVC and TRCA, opportunities to implement supplemental stormwater best management practice (BMP) measures to augment the treatment proposed by the bioretention cells using a treatment train approach, including measures to mitigate water temperature impacts, can be considered.

The supplemental BMP measures shall be designed based on the site conditions and further geotechnical and hydrogeological investigations are to be undertaken during the next phase of design. Any low impact development measures shall meet the design criteria as per the CVC/TRCA Low Impact Development Stormwater Management Planning and Design Guide (2010).

A list of potential LID measures to support the treatment train approach that may be considered for implementation within the study corridor during the detailed design is provided as follows:

Infiltration Trenches

Infiltration trenches are linear conveyance facilities lined with geotextile fabric and clean granular fill (50 mm clear stone) for quality treatment of roadway runoff. In addition to removing TSS particles, the granular filter within the trench reduces water temperature impact and enhances water balance through infiltration. It also contributes to controlling downstream erosion by reducing flow velocities.

Vegetated Filter Strips and Plunge Pool

Vegetated filter strips operate through a combination of sedimentation and infiltration. Shallow flows are routed over grassed areas, which allow the filter strips to function by slowing down the runoff velocity and filter out suspended sediment and associated pollutants and allowing infiltration into underlying soils. Filter strips are applicable where there are low, flat vegetated areas that will allow runoff to disperse over a wide area.

Plunge pools are designated depression areas at the base of storm outfalls to prevent scouring and erosion due to the high velocity of the flow at the outfall pipe locations. The plunge pool also functions as a level spreader that reduces the concentrated flow from the outfall and spreads the flow onto a natural vegetated floodplain area.

Vegetative filter strips and plunge pools can be considered at the storm outfall locations to disperse the energy of the flow and to provide additional water quality control in series with the bioretention cells as a treatment train system.

Oil-Grit Separator Units

Oil/grit separator (OGS) units combine a storage chamber for sediment trapping and oil separation with drainage inlets for intercepting or receiving roadway stormwater runoff. At locations where the roadway drainage area is less than 2.0 ha, oil-grit separator units can be used for water quality control. It should be noted that some agencies only accept a sediment removal efficiency of 50% for OGS units. Consequently, additional mitigation measures shall be considered in series with each OGS to achieve the “Enhanced” protection (Level 1) water quality target.

4.5 Erosion and Sediment Control during Construction

Erosion and sediment control (ESC) measures should be implemented and monitored through the construction period in accordance with the TRCA ESC Guide for Urban Construction (2019). Construction activities should be conducted during periods that are least likely to result in in-stream impacts to fish habitat.

Detailed erosion and sediment control plans will be required as part of the detailed design component for all phases of the construction. The erosion and sediment control plans will be subject to review and approval by the various external agencies involved in the project, including the Conservation Authorities.

During construction, disturbances to watercourse riparian vegetation should be minimized. If riparian vegetation is removed or disturbed, erosion and sediment control measures such as silt fences, rock flow check dams and sedimentation ponds should be utilized to provide a maximum protection of local and downstream aquatic resources. These measures should be maintained during construction and until disturbed areas have been stabilized with seed and mulch. Additionally, topsoil should not be stockpiled close to the watercourses and water should not be withdrawn from these sensitive streams for construction purposes.

The site engineer and contractor will be responsible for delineating work areas and ensuring that erosion and sediment control measures are functional. In addition, the engineer will ensure that provisions related to fisheries and watercourse protection is met and that any required fish habitat compensation measures are implemented in accordance with the terms and conditions of the Fisheries Act Authorization.

4.6 Stormwater Management Plan Summary

The proposed stormwater management plan for the project has been developed by examining the opportunities and constraints within the entire study corridor. Runoff from the paved roadway area will be conveyed to the proposed bioretention systems and roadway storm sewer systems and discharge into either existing storm sewer systems or watercourses. As per **Section 4.3**, the total roadway pavement area will increase by 2.52 ha, including the cycle tracks and sidewalks within the boulevard areas. Enhanced level water quality, water balance, and erosion control treatment will be provided for 2.52 ha of pavement area, meeting the MECP requirement of providing treatment to the increased pavement area.

The stormwater management plan for this project is presented on the Drainage Plans in **Appendix A**. **Table 4-4** provides a summary of the water quality treatment and quantity control strategies proposed to reduce the impacts from the increase in impervious surface within the project limits, where road widening is proposed.

Table 4-4: Summary of Stormwater Management Plan

Drainage Area ID	Existing Pavement Area (ha)	Additional Pavement Area (ha)	Pavement Area Considered to Receive Quality Treatment (ha)	Quality Storage Volume Provided (m^3)	Required Storage to Control Minor Flows (m^3)	Required Storage to Control Major Flows (m^3)
B1	1.14	0.54	0.54	213	97	171
B2	0.67	0.19	0.19	24	22	38
B3	2.26	1.05	1.05	140	138	242
B4	2.00	0.63	0.63	155	72	126
B5	1.00	0.10	0.10	77	N/A	N/A
Total	**7.07**	**2.52**	**2.52**	**610**	328	577

5 Conclusions

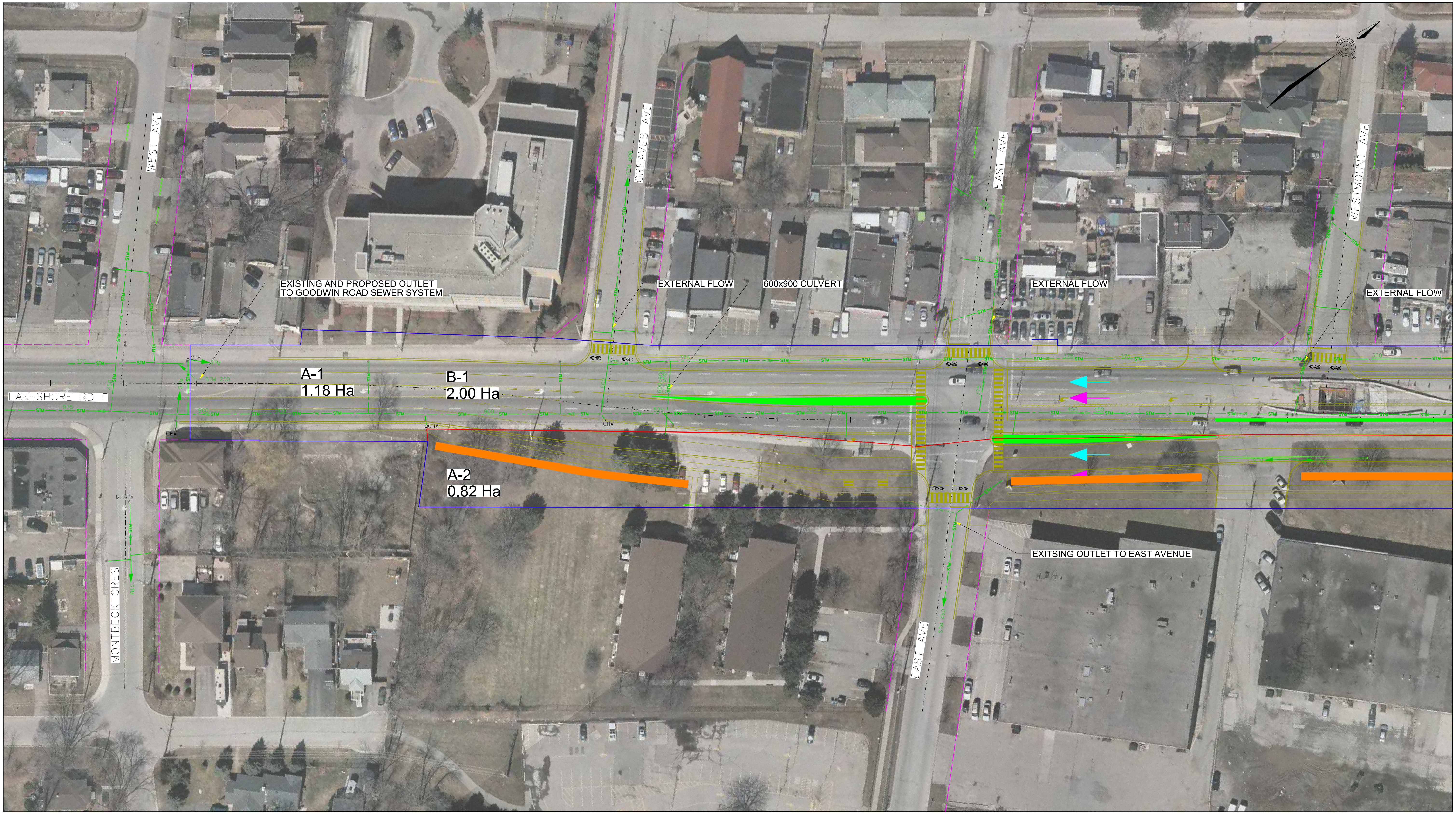
The Lakeshore Road East corridor between Etobicoke Creek and East Avenue is proposed to be widened, with addition of an exclusive transit median, and in-boulevard cycle tracks and sidewalks on both sides of the road.

Majority of the study corridor is within the area regulated by CVC, except for the east portion of the corridor being within the TRCA jurisdiction. There are two (2) watercourse crossings within the Lakeshore Road East Part A project limits, which are located at Serson Creek and Applewood Creek. Hydraulic assessment of these two crossings using available CVC models indicated that the 100 year and Regional Storm events do not overtop the road at those crossings. Hydraulic assessment of the proposed culverts has been carried out to quantify the impacts to the upstream areas. The results of these analysis indicated a small transient increase in the upstream water surface elevations at Applewood Creek crossing and a small transient decrease in upstream water surface elevations at Serson Creek crossing. However, a flood hazard analysis indicated that the changes would remain confined within the channel valley and would not result in any additional adverse flooding impacts to adjacent properties or infrastructure.

Stormwater best management practices, including catchbasin inserts bioretention systems, and online storage pipes are proposed to provide storm water quality treatment, water balance, erosion control, and quantity control of the increased runoff from the roadway right-of-way. The proposed road improvements will result in an additional pavement area of 2.52 ha. As part of the SWM strategy and in accordance with MECP requirements, a total of 2.52 ha of pavement area is considered to receive quality treatment through the proposed bioretention cells. The water balance and water quality and erosion control storage volumes provided within the proposed bioretention cells exceed the required volumes determined by TRCA and CVC criteria. Quantity control will be provided through the proposed online storage pipes. Due to the limited area available within the Lakeshore Road right-of-way, the storage required to meet the CVC criteria for Serson Creek and Applewood Creek cannot be provided. Therefore, as a best effort approach, the proposed peak flows will be controlled to their existing levels at these locations.

Opportunities to implement supplemental BMP measures to support a treatment train approach can be considered during the next phases of design in series with the proposed measures to enhance the overall water quality objectives.

Appendix A: Drainage Area Plans



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

	EXISTING PROPERTY LINE		EXISTING CATCHMENT AREA	A-X	EXISTING DRAINAGE AREA ID
	EXISTING ALIGNMENT CL		PROPOSED CATCHMENT AREA	B-X	PROPOSED DRAINAGE AREA ID
STM	EXISTING STORM SEWER		PROPOSED BIORETENTION CELL		EXISTING OVERLAND FLOW DIRECTION
	EXISTING CULVERT		ROADWAY DESIGN AND GRADING		PROPOSED OVERLAND FLOW DIRECTION
	EXISTING FLOW ARROW		WATERCOURSE CROSSING		
	EXISTING STORM CONTINUATION ARROW		PROPERTY REQUIREMENT		

HDR

MISSISSAUGA

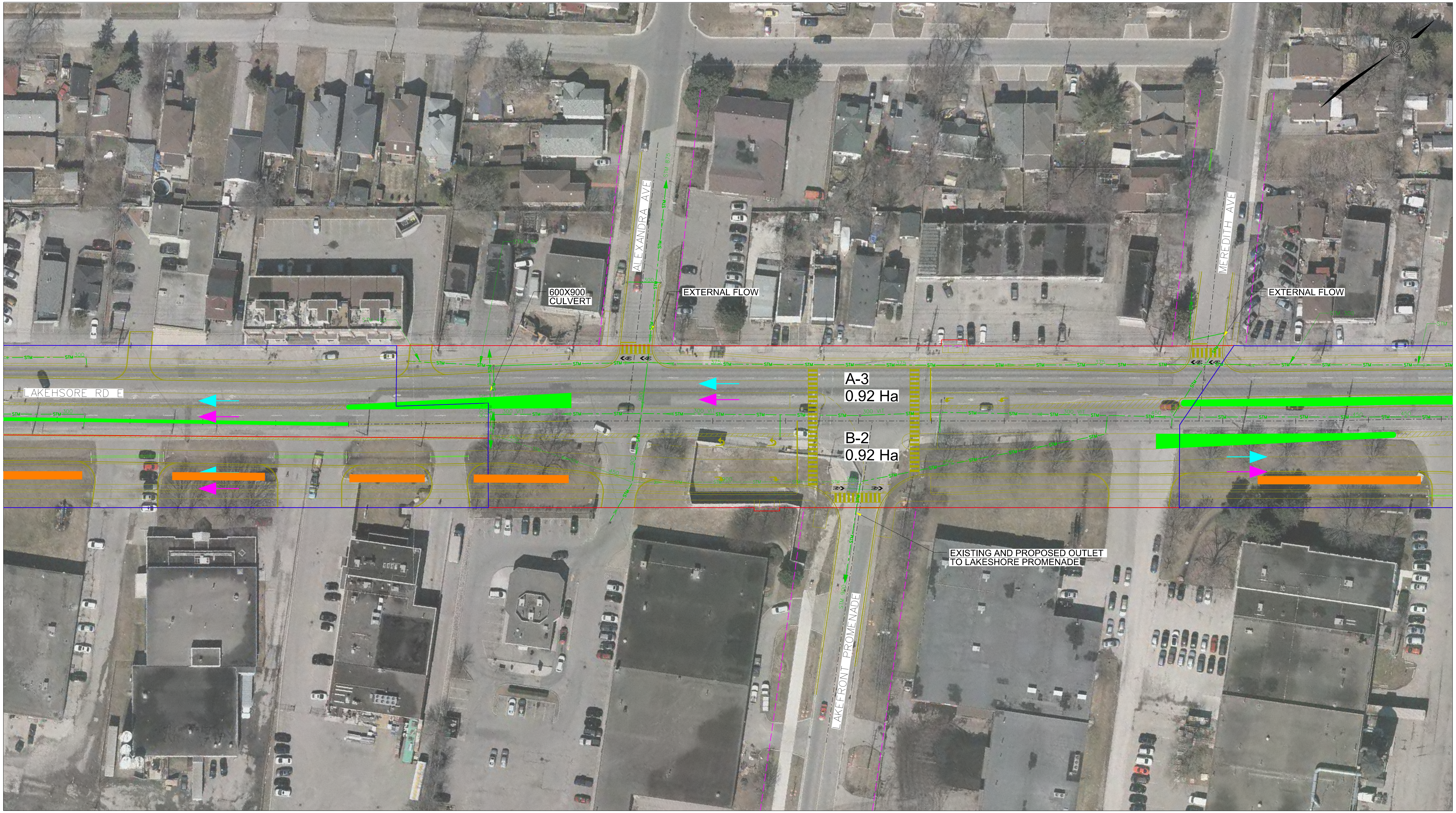
LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 20+546.07 TO STA 20+859.83

PLAN NO. 01

SCALE H 1:1000

DATE APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

EXISTING PROPERTY LINE
EXISTING ALIGNMENT CL
EXISTING STORM SEWER
EXISTING CULVERT
EXISTING FLOW ARROW
EXISTING STORM CONTINUATION ARROW

EXISTING CATCHMENT AREA
PROPOSED CATCHMENT AREA
PROPOSED BIORETENTION CELL
ROADWAY DESIGN AND GRADING
WATERCOURSE CROSSING
PROPERTY REQUIREMENT

A-X EXISTING DRAINAGE AREA ID
B-X PROPOSED DRAINAGE AREA ID
EXISTING OVERLAND FLOW DIRECTION
PROPOSED OVERLAND FLOW DIRECTION

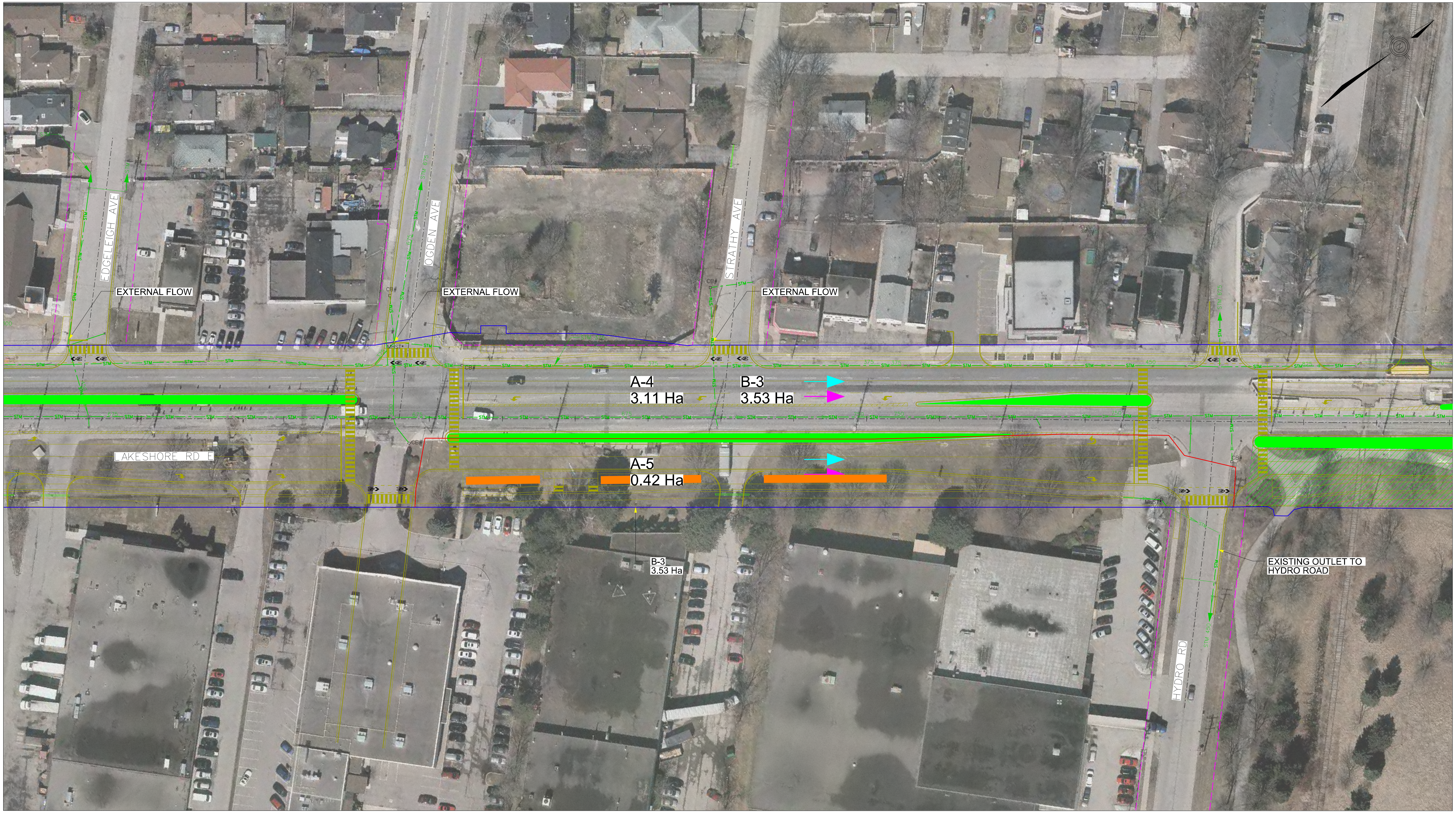
HDR
MISSISSAUGA

LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN
STA 20+859.83 TO STA 21+259.93

PLAN NO.
02

SCALE
H 1:1000

DATE
APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

EXISTING PROPERTY LINE
EXISTING ALIGNMENT CL
EXISTING STORM SEWER
EXISTING CULVERT
EXISTING FLOW ARROW
EXISTING STORM CONTINUATION ARROW

EXISTING CATCHMENT AREA
PROPOSED CATCHMENT AREA
PROPOSED BIORETENTION CELL
ROADWAY DESIGN AND GRADING
WATERCOURSE CROSSING
PROPERTY REQUIREMENT

A-X EXISTING DRAINAGE AREA ID
B-X PROPOSED DRAINAGE AREA ID
EXISTING OVERLAND FLOW DIRECTION
PROPOSED OVERLAND FLOW DIRECTION

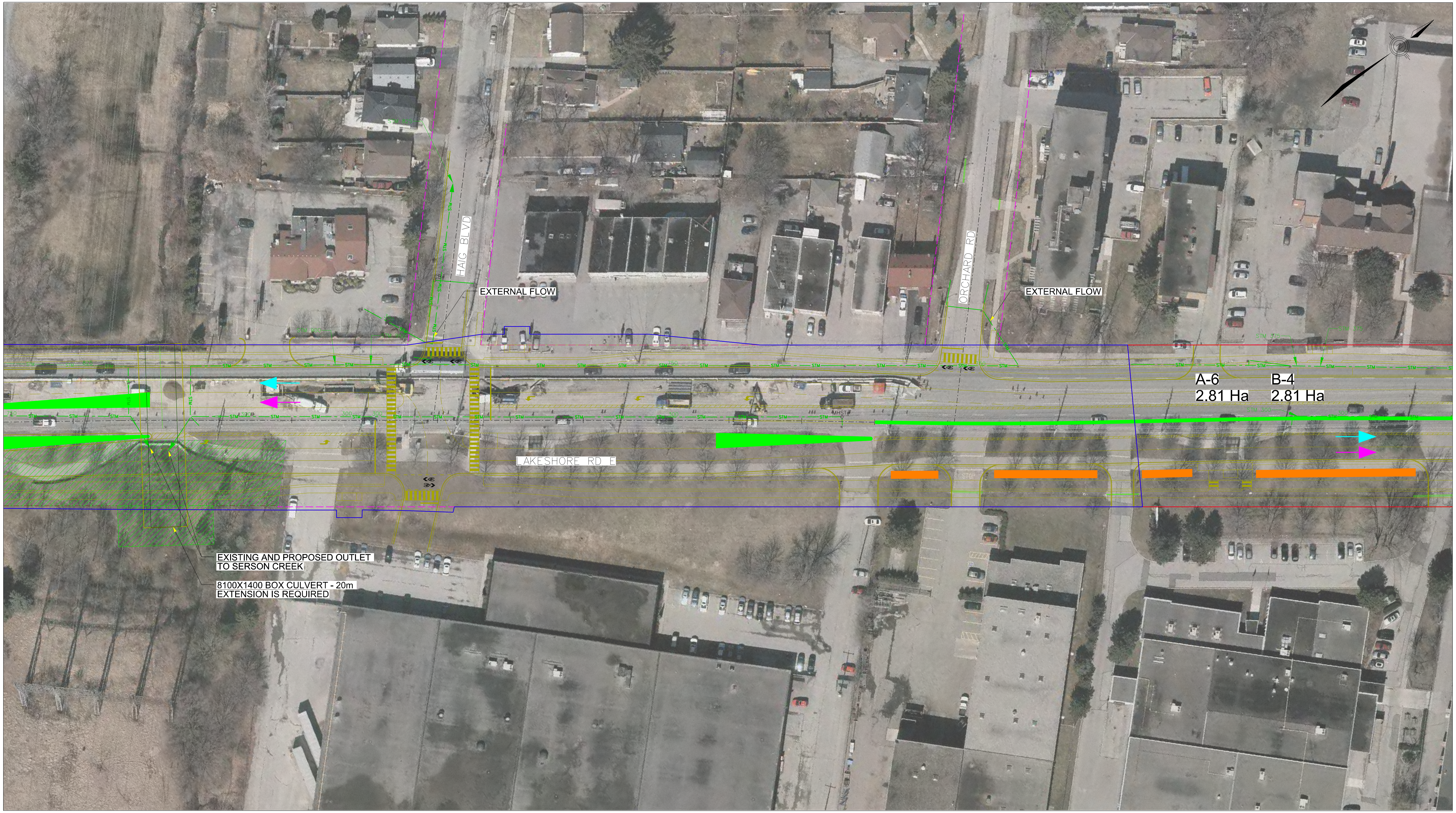
LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 21+259.93 TO STA 21+657.49

PLAN NO.
03

SCALE
H 1:1000

DATE
APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

	EXISTING PROPERTY LINE		EXISTING CATCHMENT AREA	A-X	EXISTING DRAINAGE AREA ID
	EXISTING ALIGNMENT CL		PROPOSED CATCHMENT AREA	B-X	PROPOSED DRAINAGE AREA ID
STM	EXISTING STORM SEWER		PROPOSED BIORETENTION CELL		EXISTING OVERLAND FLOW DIRECTION
	EXISTING CULVERT		ROADWAY DESIGN AND GRADING		PROPOSED OVERLAND FLOW DIRECTION
	EXISTING FLOW ARROW		WATERCOURSE CROSSING		
	EXISTING STORM CONTINUATION ARROW		PROPERTY REQUIREMENT		

HDR

MISSISSAUGA

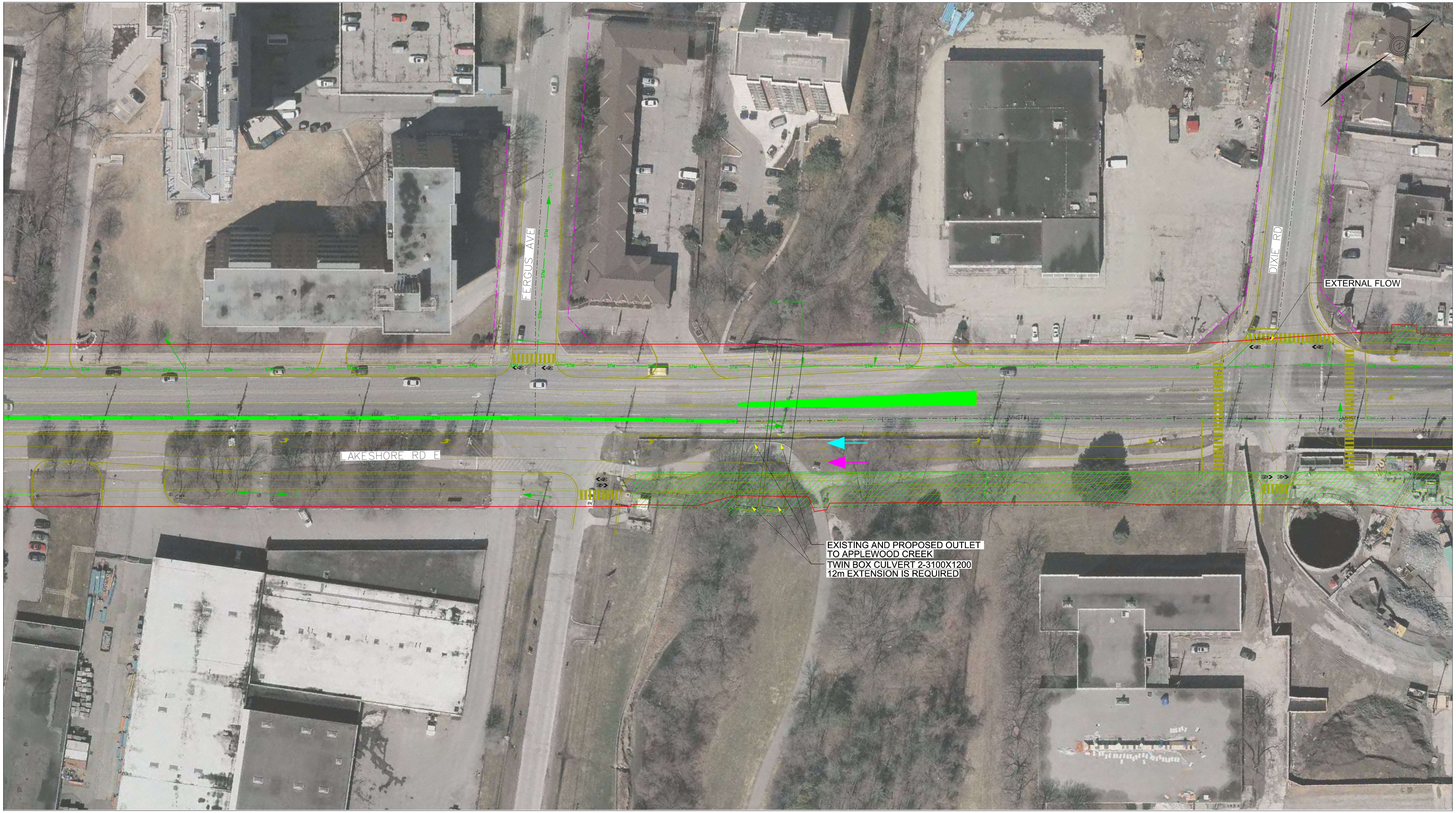
LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 21+657.49 TO STA 22+024.35

PLAN NO.
04

SCALE
H 1:1000

DATE
APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

	EXISTING PROPERTY LINE		EXISTING CATCHMENT AREA	A-X	EXISTING DRAINAGE AREA ID
	EXISTING ALIGNMENT CL		PROPOSED CATCHMENT AREA	B-X	PROPOSED DRAINAGE AREA ID
STM	EXISTING STORM SEWER		PROPOSED BIORETENTION CELL		EXISTING OVERLAND FLOW DIRECTION
	EXISTING CULVERT		ROADWAY DESIGN AND GRADING		PROPOSED OVERLAND FLOW DIRECTION
	EXISTING FLOW ARROW		WATERCOURSE CROSSING		
	EXISTING STORM CONTINUATION ARROW		PROPERTY REQUIREMENT		

HDR

MISSISSAUGA

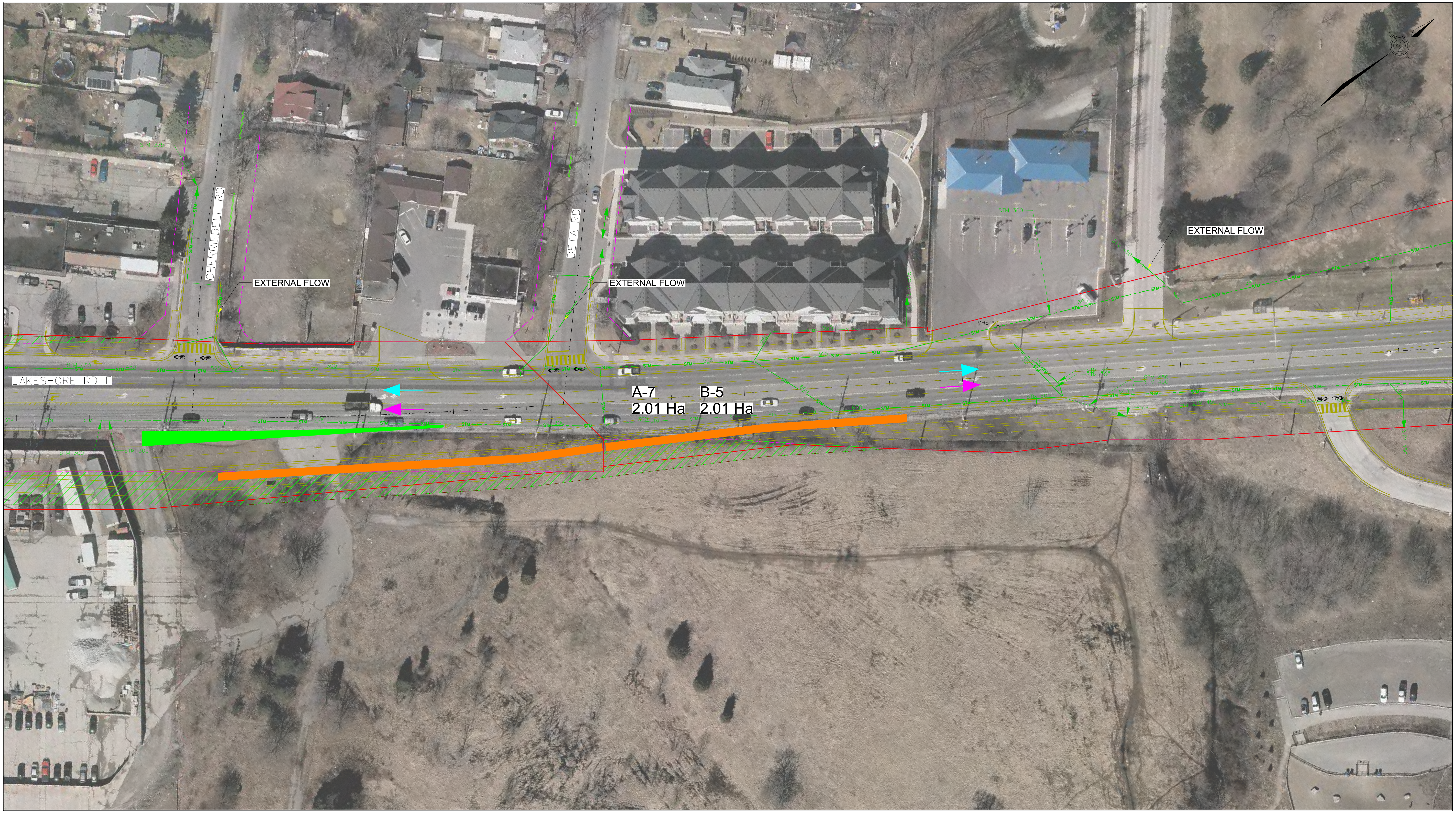
LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 22+024.35 TO STA 22+400.00

PLAN NO. 05

SCALE H 1:1000

DATE APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

	EXISTING PROPERTY LINE		EXISTING CATCHMENT AREA	A-X	EXISTING DRAINAGE AREA ID
	EXISTING ALIGNMENT CL		PROPOSED CATCHMENT AREA	B-X	PROPOSED DRAINAGE AREA ID
STM	EXISTING STORM SEWER		PROPOSED BIORETENTION CELL		EXISTING OVERLAND FLOW DIRECTION
	EXISTING CULVERT		ROADWAY DESIGN AND GRADING		PROPOSED OVERLAND FLOW DIRECTION
	EXISTING FLOW ARROW		WATERCOURSE CROSSING		
	EXISTING STORM CONTINUATION ARROW		PROPERTY REQUIREMENT		

HDR

MISSISSAUGA

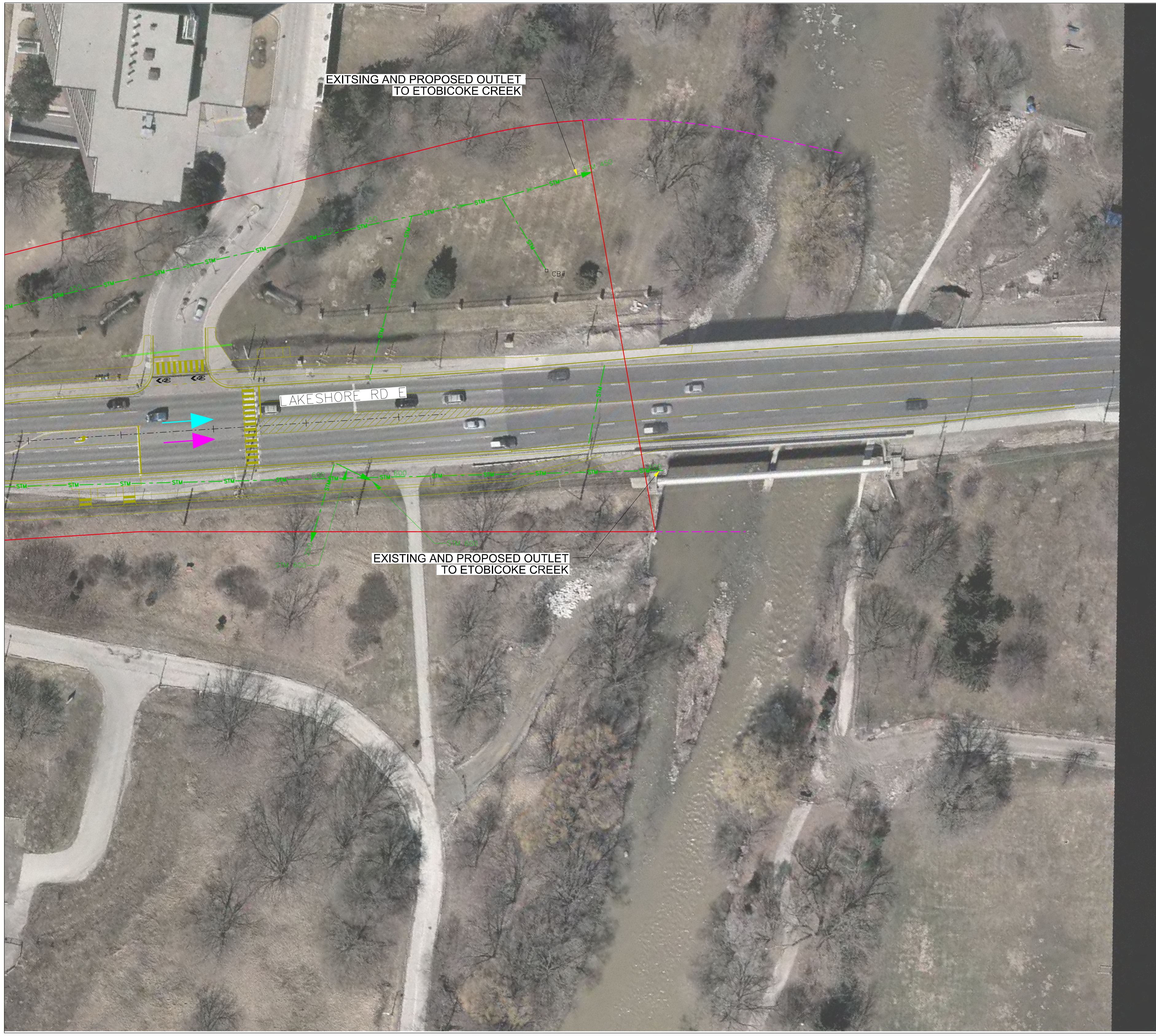
LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 22+400.00 TO STA 22+800.00

PLAN NO.
06

SCALE
H 1:1000

DATE
APRIL 2022



NOTE: PROPOSED BOULEVARD TREE LOCATIONS AND SPACING ARE CONCEPTUAL AND WILL BE CONFIRMED DURING DETAILED DESIGN

EXISTING PROPERTY LINE	EXISTING CATCHMENT AREA	A-X EXISTING DRAINAGE AREA ID
EXISTING ALIGNMENT CL	PROPOSED CATCHMENT AREA	B-X PROPOSED DRAINAGE AREA ID
EXISTING STORM SEWER	PROPOSED BIORETENTION CELL	EXISTING OVERLAND FLOW DIRECTION
EXISTING CULVERT	ROADWAY DESIGN AND GRADING	PROPOSED OVERLAND FLOW DIRECTION
EXISTING FLOW ARROW	WATERCOURSE CROSSING	
EXISTING STORM CONTINUATION ARROW	PROPERTY REQUIREMENT	

HDR

MISSISSAUGA

LAKESHORE ROAD
(PART A)
(EAST AVE W TO ETOBICOKE CREEK)
DRAINAGE PLAN
PLAN

STA 22+800.00 TO STA 22+950.15

PLAN NO. 07

SCALE H 1:1000

DATE APRIL 2022

Appendix B: Hydraulic Model Output

hdrinc.com 100 York Boulevard, Suite 300, Richmond Hill, ON, CA L4B 1J8
(289) 695-4600

Applewood Creek - HEC-RAS Output
(All Design Storm Events)

HEC-RAS River: Applewood Reach: 2241													
Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
2241	10962	2yr	Updated Ex	13.30	81.25	82.28	82.28	82.53	0.023683	2.18	6.09	12.60	1.00
2241	10962	2yr	Prop	13.30	81.25	82.28	82.28	82.53	0.023683	2.18	6.09	12.60	1.00
2241	10962	5yr	Updated Ex	20.80	81.25	82.46	82.46	82.77	0.021879	2.46	8.44	13.59	1.00
2241	10962	5yr	Prop	20.80	81.25	82.46	82.46	82.77	0.021879	2.46	8.44	13.59	1.00
2241	10962	10yr	Updated Ex	28.50	81.25	82.62	82.62	82.99	0.020868	2.69	10.61	14.40	1.00
2241	10962	10yr	Prop	28.50	81.25	82.62	82.62	82.99	0.020868	2.69	10.61	14.40	1.00
2241	10962	25yr	Updated Ex	35.60	81.25	82.74	82.74	83.16	0.020126	2.86	12.47	14.94	1.00
2241	10962	25yr	Prop	35.60	81.25	82.74	82.74	83.16	0.020126	2.86	12.47	14.94	1.00
2241	10962	50yr	Updated Ex	43.00	81.25	82.86	82.86	83.33	0.019718	3.01	14.26	15.43	1.00
2241	10962	50yr	Prop	43.00	81.25	82.86	82.86	83.33	0.019718	3.01	14.26	15.43	1.00
2241	10962	100yr	Updated Ex	51.20	81.25	82.96	82.99	83.50	0.020796	3.25	15.77	15.84	1.04
2241	10962	100yr	Prop	51.20	81.25	82.96	82.99	83.50	0.020796	3.25	15.77	15.84	1.04
2241	10962	Regional	Updated Ex	52.60	81.25	82.98	83.01	83.52	0.020906	3.28	16.04	15.91	1.04
2241	10962	Regional	Prop	52.60	81.25	82.98	83.01	83.52	0.020906	3.28	16.04	15.91	1.04
2241	10914	2yr	Updated Ex	13.40	78.85	80.45	79.66	80.49	0.001136	0.81	16.57	14.20	0.24
2241	10914	2yr	Prop	13.40	78.85	80.46	79.66	80.49	0.001122	0.81	16.64	14.21	0.24
2241	10914	5yr	Updated Ex	20.90	78.85	80.81	79.84	80.86	0.001229	0.96	21.78	14.92	0.25
2241	10914	5yr	Prop	20.90	78.85	80.82	79.84	80.87	0.001200	0.95	21.96	14.94	0.25
2241	10914	10yr	Updated Ex	28.70	78.85	81.17	80.00	81.23	0.001202	1.05	27.30	15.78	0.25
2241	10914	10yr	Prop	28.70	78.85	81.19	80.00	81.25	0.001165	1.04	27.64	15.96	0.25
2241	10914	25yr	Updated Ex	35.80	78.85	81.50	80.13	81.56	0.001104	1.10	32.65	17.28	0.25
2241	10914	25yr	Prop	35.80	78.85	81.54	80.13	81.59	0.001047	1.08	33.21	17.38	0.24
2241	10914	50yr	Updated Ex	43.10	78.85	81.84	80.25	81.91	0.000967	1.12	38.37	18.30	0.24
2241	10914	50yr	Prop	43.10	78.85	81.89	80.25	81.95	0.000905	1.10	39.20	18.44	0.23
2241	10914	100yr	Updated Ex	51.30	78.85	82.26	80.38	82.33	0.000815	1.13	45.55	19.72	0.22
2241	10914	100yr	Prop	51.30	78.85	82.33	80.38	82.39	0.000754	1.10	46.75	19.98	0.21
2241	10914	Regional	Updated Ex	53.40	78.85	82.38	80.41	82.44	0.000778	1.12	47.54	20.14	0.22
2241	10914	Regional	Prop	53.40	78.85	82.45	80.41	82.51	0.000718	1.09	48.85	20.41	0.21
2241	10898 1-Lakeshore Rd E			Culvert									
2241	10862	2yr	Updated Ex	13.40	78.41	80.42	79.36	80.44	0.000492	0.60	22.32	17.76	0.17
2241	10862	2yr	Prop	13.40	78.41	80.42	79.36	80.44	0.000492	0.60	22.32	17.76	0.17
2241	10862	5yr	Updated Ex	20.90	78.41	80.70	79.55	80.73	0.000664	0.76	27.36	18.78	0.20
2241	10862	5yr	Prop	20.90	78.41	80.70	79.55	80.73	0.000664	0.76	27.36	18.78	0.20
2241	10862	10yr	Updated Ex	28.70	78.41	80.92	79.72	80.96	0.000821	0.91	31.71	19.61	0.23
2241	10862	10yr	Prop	28.70	78.41	80.92	79.72	80.96	0.000821	0.91	31.71	19.61	0.23
2241	10862	25yr	Updated Ex	35.80	78.41	81.09	79.84	81.14	0.000967	1.02	34.97	20.21	0.25
2241	10862	25yr	Prop	35.80	78.41	81.09	79.84	81.14	0.000967	1.02	34.97	20.21	0.25
2241	10862	50yr	Updated Ex	43.10	78.41	81.22	79.96	81.28	0.001137	1.14	37.65	20.69	0.27
2241	10862	50yr	Prop	43.10	78.41	81.22	79.96	81.28	0.001137	1.14	37.65	20.69	0.27
2241	10862	100yr	Updated Ex	51.30	78.41	81.35	80.07	81.43	0.001327	1.27	40.37	21.22	0.29
2241	10862	100yr	Prop	51.30	78.41	81.35	80.07	81.43	0.001327	1.27	40.37	21.22	0.29
2241	10862	Regional	Updated Ex	53.40	78.41	81.38	80.10	81.46	0.001376	1.30	41.02	21.36	0.30
2241	10862	Regional	Prop	53.40	78.41	81.38	80.10	81.46	0.001376	1.30	41.02	21.36	0.30
2241	10856	2yr	Updated Ex	13.40	78.48	80.41		80.43	0.000902	0.63	21.11	18.19	0.19
2241	10856	2yr	Prop	13.40	78.48	80.41		80.43	0.000902	0.63	21.11	18.19	0.19
2241	10856	5yr	Updated Ex	20.90	78.48	80.69		80.72	0.001204	0.80	26.25	19.35	0.22
2241	10856	5yr	Prop	20.90	78.48	80.69		80.72	0.001204	0.80	26.25	19.35	0.22
2241	10856	10yr	Updated Ex	28.70	78.48	80.91		80.96	0.001480	0.93	30.70	20.29	0.24
2241	10856	10yr	Prop	28.70	78.48	80.91		80.96	0.001480	0.93	30.70	20.29	0.24
2241	10856	25yr	Updated Ex	35.80	78.48	81.07		81.13	0.001739	1.05	34.05	20.97	0.26

Serson Creek - HEC-RAS Output
(50-Year Design Storm Event)

HEC-RAS River: Serson Reach: 2231 Profile: Max WS

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
2231	11456	Max WS	ue_50y_uns	14.50	83.93	86.22	85.17	86.27	0.000580	0.99	17.63	44.19	0.24
2231	11456	Max WS	prop_50y_uns	14.50	83.93	86.25	85.17	86.29	0.000550	0.97	17.93	44.51	0.23
2231	11441 2-CN Spur Line			Bridge									
2231	11427	Max WS	ue_50y_uns	14.50	83.75	84.82	84.82	85.04	0.013662	2.39	7.90	61.22	0.96
2231	11427	Max WS	prop_50y_uns	14.50	83.75	84.82	84.82	85.04	0.013662	2.39	7.90	61.22	0.96
2231	11390	Max WS	ue_50y_uns	14.50	83.36	84.60		84.68	0.005121	1.64	16.52	46.72	0.60
2231	11390	Max WS	prop_50y_uns	14.50	83.36	84.60		84.68	0.005121	1.64	16.52	46.72	0.60
2231	11333	Max WS	ue_50y_uns	14.50	83.30	84.37		84.44	0.004383	1.59	18.90	55.40	0.57
2231	11333	Max WS	prop_50y_uns	14.50	83.30	84.37		84.43	0.004266	1.57	18.78	55.23	0.56
2231	11290	Max WS	ue_50y_uns	14.48	83.02	84.04		84.14	0.010275	1.84	12.50	39.97	0.80
2231	11290	Max WS	prop_50y_uns	14.50	83.02	84.04		84.14	0.010131	1.83	12.58	40.12	0.80
2231	11244	Max WS	ue_50y_uns	14.50	82.57	83.66		83.77	0.006134	1.87	13.07	67.73	0.67
2231	11244	Max WS	prop_50y_uns	14.50	82.57	83.65		83.77	0.006606	1.92	12.71	67.15	0.70
2231	11193	Max WS	ue_50y_uns	14.50	82.05	83.47		83.53	0.002732	1.36	19.26	75.95	0.45
2231	11193	Max WS	prop_50y_uns	14.50	82.05	83.42		83.50	0.003831	1.55	16.90	71.97	0.53
2231	11137	Max WS	ue_50y_uns	16.70	81.70	83.36		83.41	0.000951	1.05	18.32	39.02	0.29
2231	11137	Max WS	prop_50y_uns	16.70	81.70	83.27		83.33	0.001204	1.14	16.83	37.05	0.32
2231	11130.44	Max WS	ue_50y_uns	16.70	81.78	83.26	82.58	83.39	0.006100	1.59	10.52		0.42
2231	11130.44	Max WS	prop_50y_uns	16.70	81.75	83.24	82.58	83.33	0.001601	0.90	14.97		0.34
2231	11102.88	Max WS	ue_50y_uns	16.70	81.54	83.08	82.34	83.21	0.006826	1.62	10.32		0.42
2231	11102.88	Max WS	prop_50y_uns	16.70	81.54	83.18	82.25	83.25	0.003313	1.15	14.53		0.29
2231	11098.45	Max WS	ue_50y_uns	16.70	81.63	83.14		83.19	0.000971	1.02	17.16	14.23	0.29
2231	11098.45	Max WS	prop_50y_uns	16.70	81.46	83.17	82.22	83.24	0.002664	1.10	14.80		0.28
2231	11096.43	Max WS	ue_50y_uns	16.70	81.85	83.13		83.19	0.001312	1.11	15.69	14.20	0.33
2231	11096.43	Max WS	prop_50y_uns	16.70	81.85	83.13	82.47	83.23	0.005518	1.37	12.08		0.39
2231	11094.38	Max WS	ue_50y_uns	16.70	81.85	83.13		83.19	0.001323	1.12	15.65	14.19	0.33
2231	11094.38	Max WS	prop_50y_uns	16.70	81.85	83.12	82.46	83.22	0.005481	1.37	12.12		0.39
2231	11092.36	Max WS	ue_50y_uns	16.70	81.63	83.13		83.18	0.000991	1.02	17.05	14.21	0.29
2231	11092.36	Max WS	prop_50y_uns	16.70	81.39	83.14	82.22	83.21	0.002396	1.06	15.00		0.28
2231	11082.9	Max WS	ue_50y_uns	16.70	81.63	83.12		83.17	0.001032	1.04	16.83	14.17	0.30
2231	11082.9	Max WS	prop_50y_uns	16.70	81.33	83.11	82.20	83.18	0.002418	1.07	15.09		0.27
2231	11076	Max WS	ue_50y_uns	16.70	81.51	83.11		83.16	0.000949	1.01	17.24	14.16	0.28
2231	11076	Max WS	prop_50y_uns	16.70	81.31	83.11		83.16	0.000895	0.99	17.57	14.16	0.28
2231	11041	Max WS	ue_50y_uns	16.70	81.30	83.05		83.12	0.002023	1.33	17.65	38.70	0.40
2231	11041	Max WS	prop_50y_uns	16.70	81.30	83.05		83.12	0.002023	1.33	17.65	38.70	0.40

Serson Creek - HEC-RAS Output
(100-Year Design Storm Event)

HEC-RAS River: Serson Reach: 2231 Profile: Max WS

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
2231	11456	Max WS	UE_100y_UNS	16.60	83.93	86.76	85.24	86.80	0.000307	0.86	23.37	50.25	0.18
2231	11456	Max WS	UNS_Prop_100y	16.60	83.93	86.77	85.24	86.81	0.000302	0.85	23.47	50.36	0.18
2231	11441 2-CN Spur Line			Bridge									
2231	11427	Max WS	UE_100y_UNS	16.60	83.75	84.87	84.86	85.11	0.013928	2.52	8.55	61.80	0.98
2231	11427	Max WS	UNS_Prop_100y	16.60	83.75	84.87	84.86	85.11	0.013928	2.52	8.55	61.80	0.98
2231	11390	Max WS	UE_100y_UNS	16.60	83.36	84.64		84.73	0.005025	1.69	18.57	48.77	0.60
2231	11390	Max WS	UNS_Prop_100y	16.60	83.36	84.64		84.73	0.005024	1.69	18.57	48.77	0.60
2231	11333	Max WS	UE_100y_UNS	16.59	83.30	84.41		84.47	0.004350	1.63	20.91	55.85	0.57
2231	11333	Max WS	UNS_Prop_100y	16.60	83.30	84.41		84.47	0.004353	1.63	20.91	55.85	0.57
2231	11290	Max WS	UE_100y_UNS	16.60	83.02	84.08		84.18	0.010198	1.88	14.07	42.54	0.81
2231	11290	Max WS	UNS_Prop_100y	16.42	83.02	84.08		84.18	0.009786	1.85	14.18	42.90	0.79
2231	11244	Max WS	UE_100y_UNS	16.60	82.57	83.73		83.84	0.005117	1.81	15.43	71.30	0.62
2231	11244	Max WS	UNS_Prop_100y	16.60	82.57	83.72		83.83	0.005436	1.85	15.10	70.91	0.64
2231	11193	Max WS	UE_100y_UNS	16.60	82.05	83.60		83.64	0.001837	1.22	24.81	80.88	0.38
2231	11193	Max WS	UNS_Prop_100y	16.60	82.05	83.58		83.62	0.002056	1.27	23.81	80.32	0.40
2231	11137	Max WS	UE_100y_UNS	19.20	81.70	83.50		83.55	0.000887	1.07	20.72	40.03	0.28
2231	11137	Max WS	UNS_Prop_100y	19.20	81.70	83.47		83.52	0.000953	1.10	20.20	39.82	0.29
2231	11130.44	Max WS	UE_100y_UNS	19.20	81.78	83.37	82.65	83.54	0.008063	1.83	10.52		0.46
2231	11130.44	Max WS	UNS_Prop_100y	19.20	81.75	83.36	82.64	83.48	0.002116	1.04	14.97		0.38
2231	11102.88	Max WS	UE_100y_UNS	19.20	81.54	83.12	82.42	83.30	0.009023	1.86	10.32		0.47
2231	11102.88	Max WS	UNS_Prop_100y	17.90	81.54	83.45	82.28	83.53	0.003805	1.23	14.53		0.28
2231	11098.45	Max WS	UE_100y_UNS	19.20	81.63	83.21		83.27	0.001081	1.11	18.12	14.39	0.31
2231	11098.45	Max WS	UNS_Prop_100y	17.90	81.46	83.42	82.26	83.50	0.003059	1.18	14.80		0.28
2231	11096.43	Max WS	UE_100y_UNS	19.20	81.85	83.19		83.27	0.001444	1.21	16.63	14.36	0.35
2231	11096.43	Max WS	UNS_Prop_100y	17.90	81.85	83.35	82.49	83.47	0.006337	1.47	12.08		0.39
2231	11094.38	Max WS	UE_100y_UNS	19.20	81.85	83.19		83.26	0.001457	1.21	16.58	14.35	0.35
2231	11094.38	Max WS	UNS_Prop_100y	17.90	81.85	83.32	82.49	83.43	0.006295	1.47	12.12		0.39
2231	11092.36	Max WS	UE_100y_UNS	19.20	81.63	83.20		83.26	0.001106	1.12	17.99	14.37	0.31
2231	11092.36	Max WS	UNS_Prop_100y	17.90	81.39	83.32	82.24	83.40	0.002752	1.13	15.00		0.29
2231	11082.9	Max WS	UE_100y_UNS	19.20	81.63	83.18		83.24	0.001155	1.13	17.74	14.33	0.32
2231	11082.9	Max WS	UNS_Prop_100y	19.20	81.33	83.16	82.26	83.25	0.003196	1.23	15.09		0.31
2231	11076	Max WS	UE_100y_UNS	19.20	81.51	83.18		83.24	0.001068	1.11	18.14	14.32	0.30
2231	11076	Max WS	UNS_Prop_100y	19.20	81.31	83.18		83.24	0.001009	1.09	18.47	14.32	0.30
2231	11041	Max WS	UE_100y_UNS	19.20	81.30	83.13		83.19	0.001818	1.32	20.70	41.09	0.38
2231	11041	Max WS	UNS_Prop_100y	19.20	81.30	83.13		83.19	0.001818	1.32	20.70	41.09	0.38

Serson Creek - HEC-RAS Output
(Regional Design Storm Event)

HEC-RAS River: Serson Reach: 2231 Profile: Max WS													
Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
2231	11456	Max WS	PROP_REG_REPL	15.90	83.93	86.58	85.22	86.62	0.000373	0.90	21.42	48.19	0.20
2231	11441 2-CN Spur Line			Bridge									
2231	11427	Max WS	UE_REG_UNS	15.90	83.75	84.85	84.85	85.08	0.013867	2.48	8.33	61.70	0.97
2231	11427	Max WS	PROP_REG_REPL	15.90	83.75	84.85	84.85	85.08	0.013867	2.48	8.33	61.70	0.97
2231	11390	Max WS	UE_REG_UNS	15.90	83.36	84.63		84.71	0.005053	1.67	17.89	48.04	0.60
2231	11390	Max WS	PROP_REG_REPL	15.71	83.36	84.63		84.71	0.004902	1.65	17.94	48.09	0.59
2231	11333	Max WS	UE_REG_UNS	15.89	83.30	84.39		84.46	0.004340	1.62	20.28	55.71	0.57
2231	11333	Max WS	PROP_REG_REPL	15.90	83.30	84.39		84.46	0.004343	1.62	20.28	55.71	0.57
2231	11290	Max WS	UE_REG_UNS	15.90	83.02	84.06		84.17	0.010335	1.88	13.49	41.39	0.81
2231	11290	Max WS	PROP_REG_REPL	15.80	83.02	84.07		84.17	0.009893	1.84	13.67	41.60	0.79
2231	11244	Max WS	UE_REG_UNS	15.90	82.57	83.72		83.82	0.004979	1.77	15.11	70.92	0.61
2231	11244	Max WS	PROP_REG_REPL	15.90	82.57	83.71		83.82	0.005295	1.81	14.77	70.53	0.63
2231	11193	Max WS	UE_REG_UNS	15.90	82.05	83.59		83.63	0.001725	1.18	24.60	80.77	0.37
2231	11193	Max WS	PROP_REG_REPL	15.90	82.05	83.57		83.62	0.001929	1.23	23.61	80.17	0.39
2231	11137	Max WS	UE_REG_UNS	19.10	81.70	83.49		83.55	0.000889	1.07	20.62	39.99	0.28
2231	11137	Max WS	PROP_REG_REPL	19.10	81.70	83.46		83.52	0.000956	1.10	20.11	39.78	0.29
2231	11130.44	Max WS	UE_REG_UNS	19.10	81.78	83.36	82.65	83.53	0.007979	1.82	10.52		0.46
2231	11130.44	Max WS	PROP_REG_REPL	19.10	81.75	83.36	82.64	83.47	0.002094	1.03	14.97		0.38
2231	11102.88	Max WS	UE_REG_UNS	19.10	81.54	83.12	82.41	83.30	0.008929	1.85	10.32		0.47
2231	11102.88	Max WS	PROP_REG_REPL	17.89	81.54	83.45	82.27	83.53	0.003802	1.23	14.53		0.28
2231	11098.45	Max WS	UE_REG_UNS	19.10	81.63	83.20		83.27	0.001077	1.11	18.08	14.39	0.31
2231	11098.45	Max WS	PROP_REG_REPL	17.89	81.46	83.42	82.26	83.50	0.003056	1.18	14.80		0.28
2231	11096.43	Max WS	UE_REG_UNS	19.10	81.85	83.19		83.26	0.001438	1.21	16.59	14.35	0.35
2231	11096.43	Max WS	PROP_REG_REPL	17.89	81.85	83.35	82.49	83.47	0.006331	1.46	12.08		0.39
2231	11094.38	Max WS	UE_REG_UNS	19.10	81.85	83.19		83.26	0.001451	1.21	16.54	14.35	0.35
2231	11094.38	Max WS	PROP_REG_REPL	17.89	81.85	83.32	82.49	83.43	0.006289	1.46	12.12		0.39
2231	11092.36	Max WS	UE_REG_UNS	19.10	81.63	83.20		83.26	0.001101	1.11	17.96	14.37	0.31
2231	11092.36	Max WS	PROP_REG_REPL	17.89	81.39	83.32	82.24	83.40	0.002750	1.13	15.00		0.29
2231	11082.9	Max WS	UE_REG_UNS	19.10	81.63	83.18		83.24	0.001150	1.13	17.71	14.32	0.31
2231	11082.9	Max WS	PROP_REG_REPL	19.10	81.33	83.16	82.26	83.24	0.003163	1.22	15.09		0.31
2231	11076	Max WS	UE_REG_UNS	19.10	81.51	83.17		83.23	0.001062	1.10	18.11	14.31	0.30
2231	11076	Max WS	PROP_REG_REPL	19.10	81.31	83.17		83.23	0.001004	1.08	18.44	14.31	0.29
2231	11041	Max WS	UE_REG_UNS	19.10	81.30	83.12		83.19	0.001821	1.32	20.60	41.01	0.38
2231	11041	Max WS	PROP_REG_REPL	19.10	81.30	83.12		83.19	0.001821	1.32	20.60	41.01	0.38

Appendix C: Hydraulic Modelling Memo for Applewood and Serson Creeks

hdrinc.com 100 York Boulevard, Suite 300, Richmond Hill, ON, CA L4B 1J8
(289) 695-4600

Memo

Date:	Tuesday, May 23, 2023
Project:	Lakeshore Road TPAP
To:	CVC
From:	Dante Mawji, P.Eng.
Subject:	Hydraulic Modelling of Lakeshore Road Crossing Extension at Applewood and Serson Creeks

Introduction

As part of the Transit Project Assessment Process (TPAP) for the proposed Lakeshore Road widening and improvements, HDR has conducted a hydraulic assessment for the proposed extension of the Applewood Creek culvert and replacement of Serson Creek culvert crossings. The purpose of this memo is to outline the changes made to the existing hydraulic models provided by CVC and to summarize the results of the analysis.

Applewood Creek

The existing crossing at Applewood Creek is a twin box culvert running 28.02 m along the length of the channel as shown in **Figure 1**. With the widening of Lakeshore Road, the culvert will require a 12-meter extension to a final length of 40.02 meters as shown in **Figure 2**. The modeling of this structure required no additional modifications or special consideration.

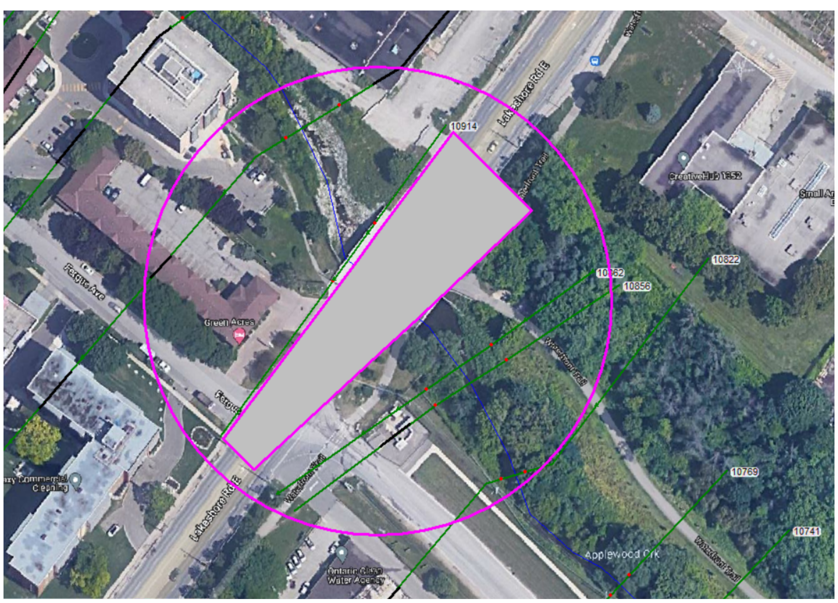


Figure 1: Existing Applewood Creek Crossing at Lakeshore Road (HEC-RAS)

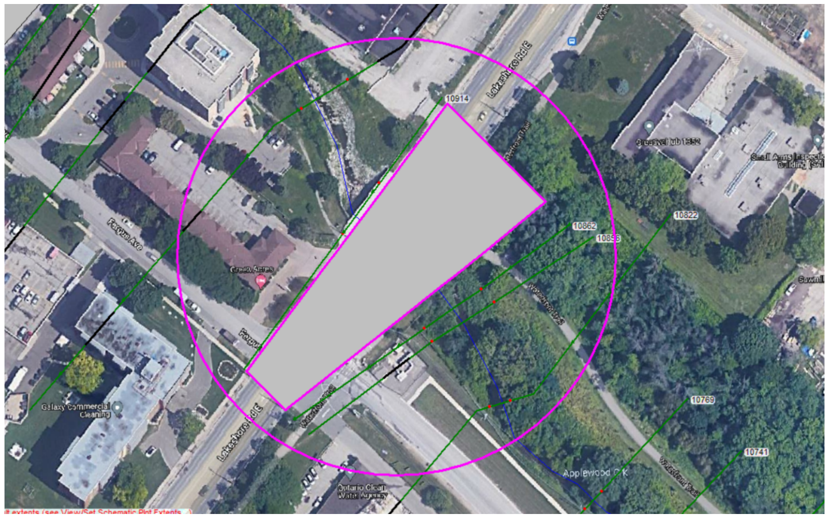


Figure 2: Proposed Applewood Creek Crossing at Lakeshore Road (HEC-RAS)

Serson Creek

The existing crossing of Lakeshore Road at Serson Creek is an 8-meter span concrete box culvert running 27.56 meters along the length of the channel as shown in **Figure 3.** A 1650 mm sanitary sewer runs across the downstream reach and will be enclosed by the proposed extension of lakeshore road. This enclosure will create a bottleneck, increasing upstream flood levels significantly. To reduce the effect of the obstruction on the upstream water surface elevations, replacement of the existing culvert is proposed. The existing culvert will be replaced with a 47 meter long, 11-meter span culvert, as shown in **Figure 4** and **Figure 5.**

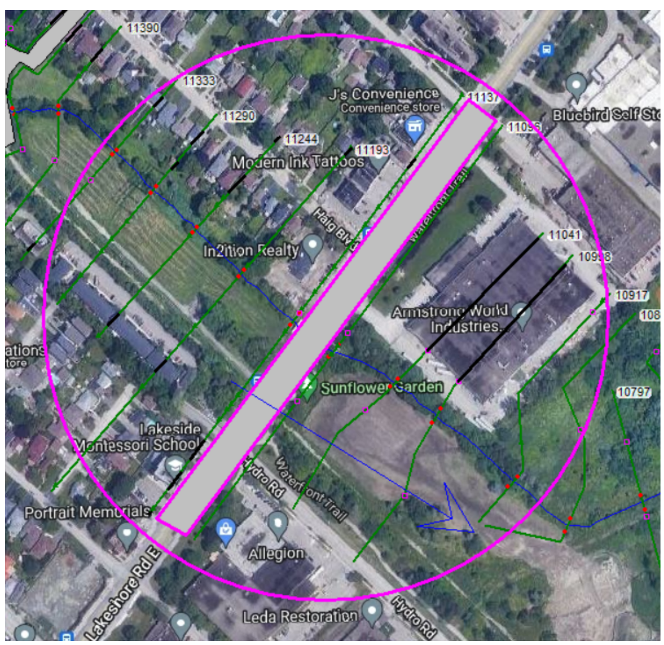


Figure 3: Existing Serson Creek Crossing at Lakeshore Road (HEC-RAS)

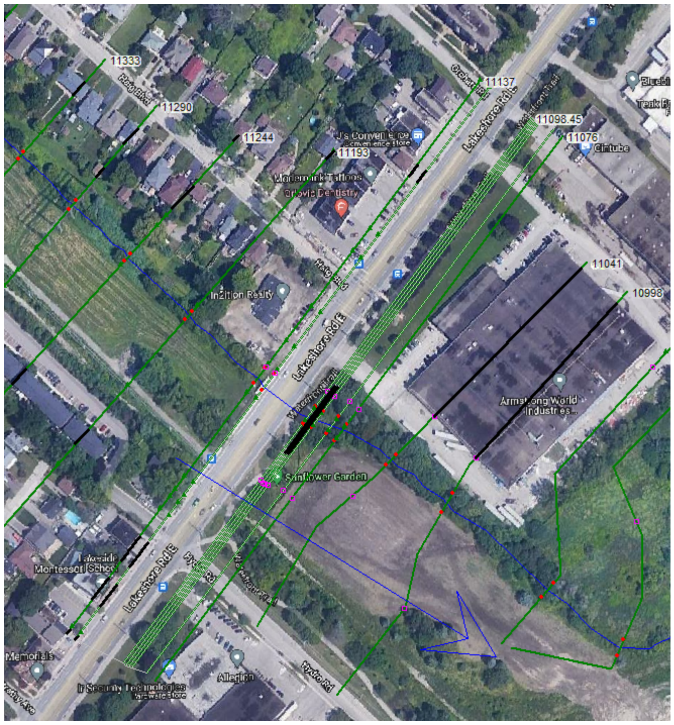


Figure 4: Proposed Serson Creek Crossing at Lakeshore Road (HEC-RAS) [1]

[1] Proposed Serson Creek Structure shown in **Figure 4** is modeled as a Lidded Cross Section which is not visually represented in plan view as the structures modeled in the existing condition.

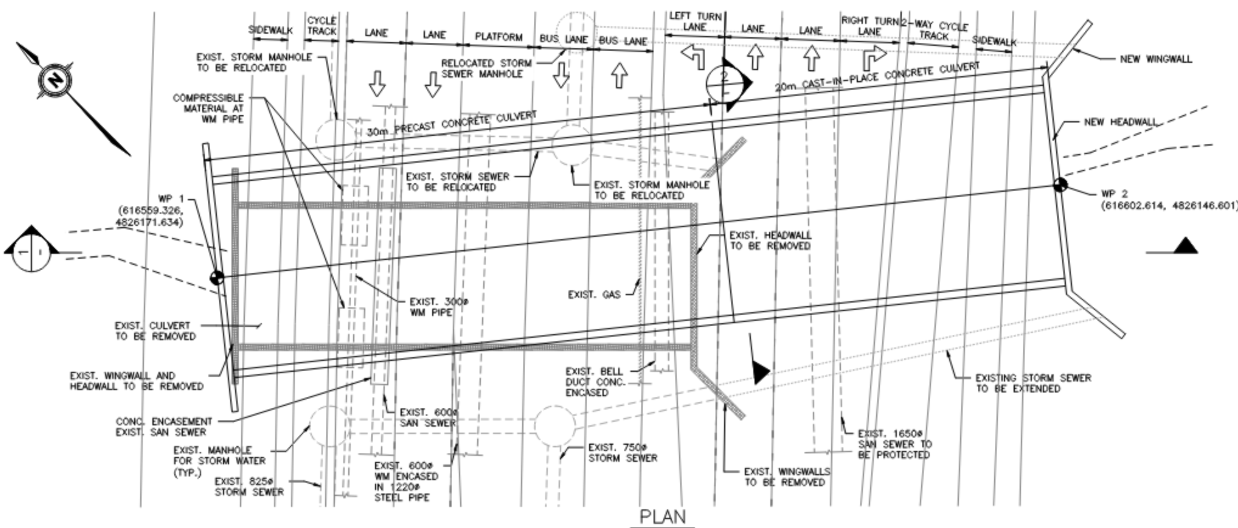


Figure 5: Proposed Serson Creek Crossing at Lakeshore Road (General Arrangement Plan)

The downstream channel will require subsequent grading to accommodate the extended structure. The existing and proposed downstream grading is shown in **Figure 6** and **Figure 7**.

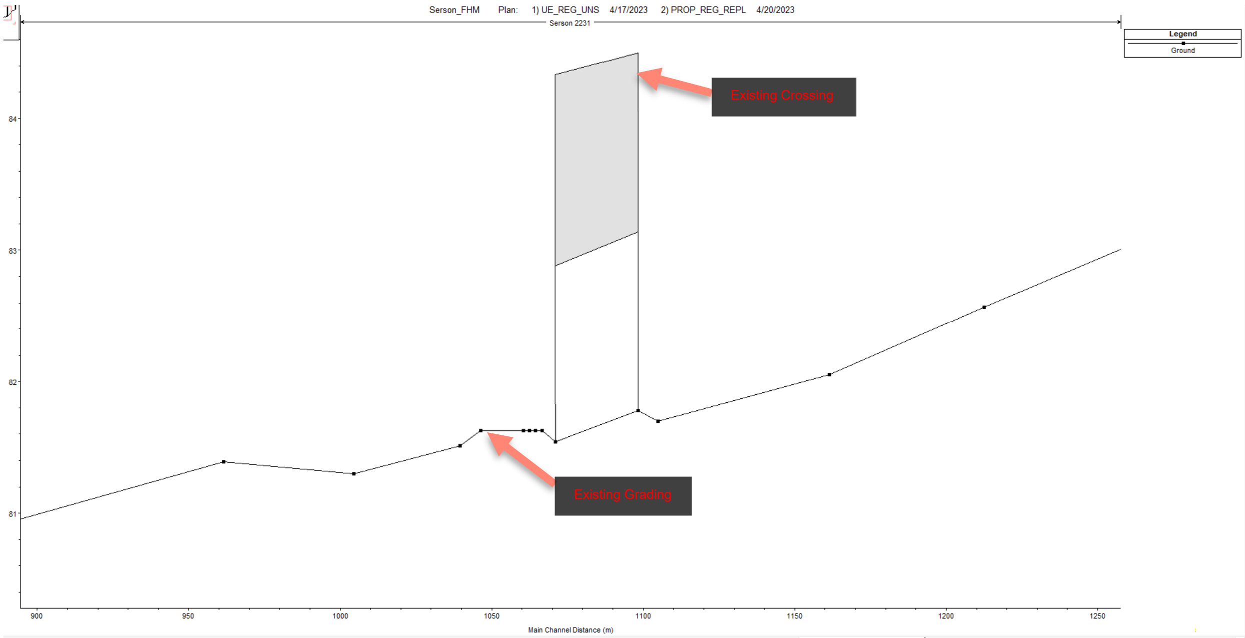


Figure 6: Existing Serson Creek Ground Profile (HEC-RAS)

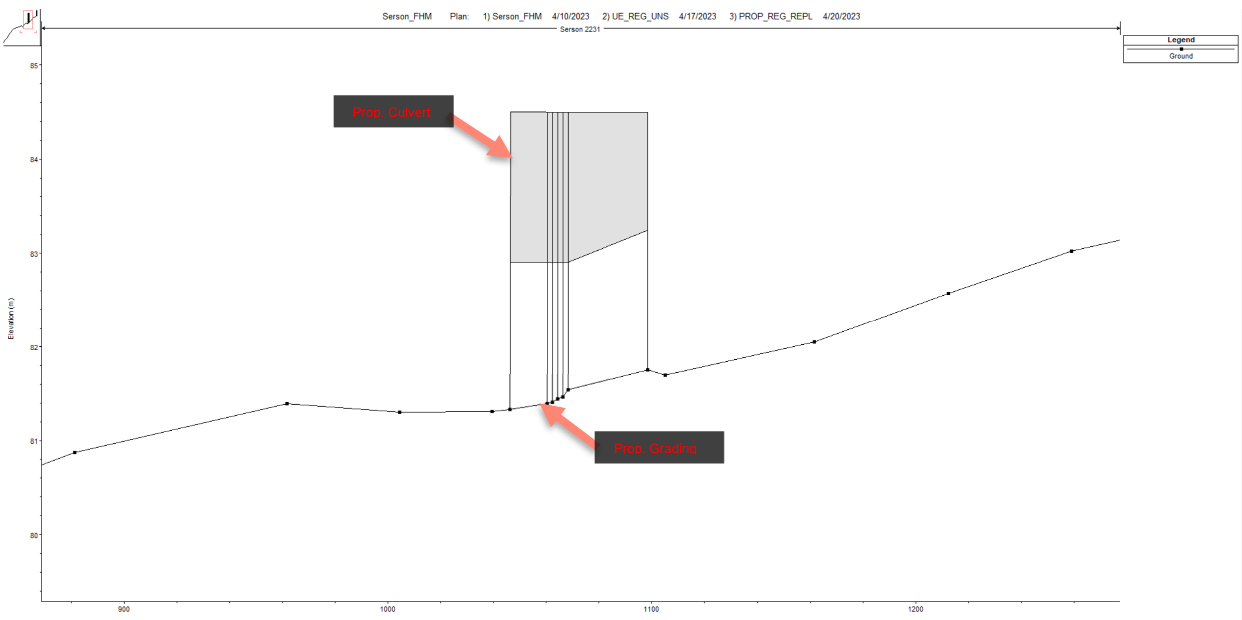


Figure 7: Proposed Serson Creek Ground Profile (HEC-RAS)

In the existing condition, the centerline of a 1650 mm sanitary sewer runs perpendicular to the watercourse, 7.47 m downstream of the outlet. In the proposed condition, the sanitary sewer is encased in 150mm concrete and 50mm of compressible material for reinforcement and enveloped within the proposed culvert as seen in **Figure 8**. Modeling the culvert with the sanitary sewer remaining in place will result in an obstruction within the culvert opening, requiring special modeling considerations, as described in the following section.

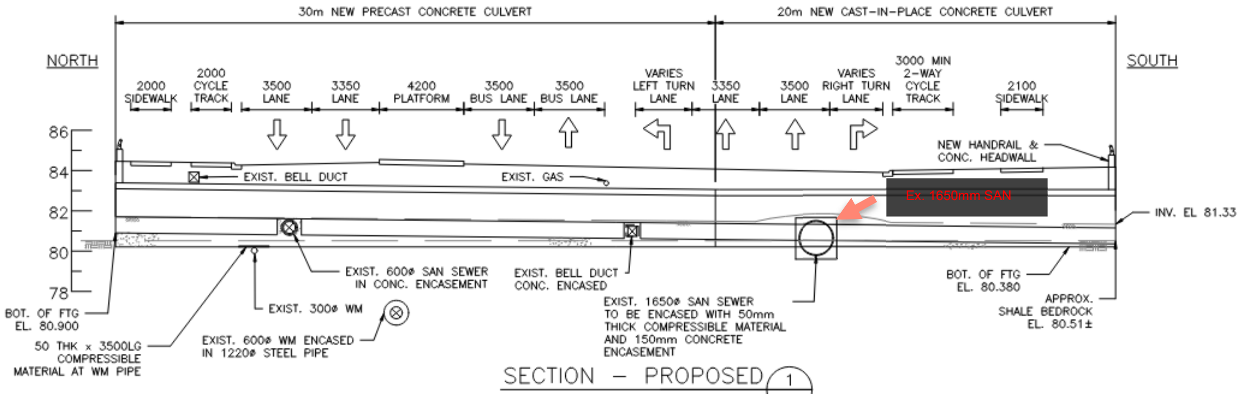


Figure 8: Proposed Serson Creek Profile (General Arrangement)

Serson Updated Models

Building the proposed culvert over the existing sanitary sewer will result in an irregular cross section that will vary through the length of the structure. HEC-RAS cannot model structures to contain internal variations under steady state conditions. To represent the opening variations caused by the sanitary sewer, an unsteady state model proposed model was required. To produce results in comparable terms with existing conditions, an unsteady state model of the existing and updated existing conditions was also required.

To model the irregularity, the crossing structure was converted from a normal HEC-RAS structure into a lidded cross section structure for all scenarios. Lidded cross sections were required as normal HEC-RAS structures cannot model internal geometric variance caused by the existing sanitary sewer. A summary of the different model scenarios is presented in **Table 1**.

Table 1: Water Surface Elevation Comparison

Model Scenario	Computation State	Structure Type	Number of Defining XS for Structure	Sanitary Sewer Obstruction Defined
Existing (original)	Steady	Normal Structure	2	No
Existing (modified)	Unsteady	Lidded bounding XS	4	No
Updated Existing	Unsteady	Lidded bounding XS	8	Yes
Proposed	Unsteady	Lidded bounding XS	8	Yes

The unsteady model of the Serson Creek crossing contains a total of 8 cross sections compared to the existing models 2 cross sections. Of the additional 6 cross sections, 4 were used to define the obstruction by the existing sanitary sewer, and the remaining 2 were added to redefine the crossing structure as a lidded cross section. Due to the expected regrading and relatively short distances along the channel, the existing section's geometry were extrapolated to produce the geometry of the proposed sections. The extrapolated sections channel bottom was then adjusted to reflect the proposed design grading.

As the number of model elements increase, so does the complexity of the unsteady simulation. This complexity introduces increased instability and potential error. Model elements further upstream of cross section 11471 have no impact on the subject model area and were removed to reduce complexity and improve model stability.

Steady state flows provided in the existing model were converted into unsteady flow format and applied to the unsteady state model for the 50-year, 100-year and Regional storm events. The existing steady state flow node at cross section 11471 was input as a Flow Hydrograph at the upstream boundary condition for the unsteady model. In contrast to cumulative flows represented in steady state format, unsteady flows through the system are summative with their downstream flows. The subsequent downstream flows were converted to unsteady (summative) format and added to sections 11193 and 10718 as Lateral Hydrographs. The downstream boundary condition is defined by Normal Depth at cross section 10037 with a standard friction slope of 0.00023. A summary of the existing and proposed flow nodes is provided in **Table 2** and **Table 3** respectively.

Table 2: Existing Model Steady State Flows

River Station	50-Year Flow	100-Year Flow	Regional Flow
11471	14.5	16.6	15.9
11137	16.7	19.2	19.1
10718	15.9	18.3	20.5

Table 3: Proposed Model Unsteady Flows

River Station	50-Year Flow[2]	100-Year Flow[2]	Regional Flow[2]
11471	14.5[3]	16.6[3]	15.9[3]
11193	2.2[4]	2.6[4]	3.2[4]
10718	-0.8[4]	-0.9[4]	1.4[4]

[2] Unsteady flows are input as hydrographs into HEC-RAS. Values shown represent the maximum value of the stepwise function hydrographs produced from existing steady state flows.
[3] Upstream boundary condition flow represents total system flow at that flow node.
[4] Flow represents the change in flow from upstream node.

Results

A summary of the existing and proposed water surface elevations at the upstream cross section with the greatest impact can be found in **Table 4.**

Table 4: Water Surface Elevation Comparison

River	River St.		Water Surface Elev. (m)		
			50 Year	100 Year	Regional
Applewood Creek	10914	Existing	81.84	82.26	82.38
		Proposed	81.89	82.33	82.45
		Difference	0.05	0.07	0.07
Serson Creek	11137	Existing	83.36	83.50	83.49
		Proposed	83.27	83.47	83.46
		Difference	-0.09	-0.03	-0.03

The maximum increase in water surface elevation for the regional event is 7 cm for the Applewood Creek structure. Water surface elevations decrease for all modeled events upstream of the Serson Creek proposed structure.

Analysis

For the Applewood Creek Crossing, the increase in water surface elevation caused by the proposed extension is greatest at the cross section immediately upstream of the structure and dissipate further upstream as shown in **Figure 9.** There are no downstream impacts to the water surface elevation for the proposed crossing.

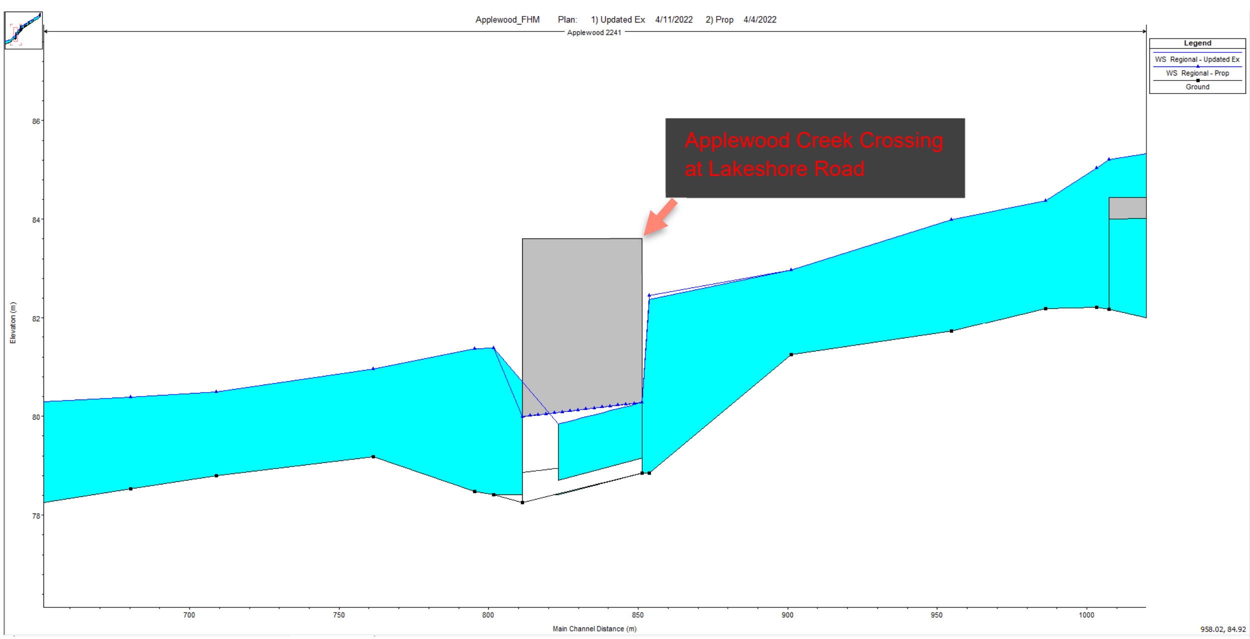


Figure 9: Applewood Creek Updated Existing and Proposed Regional Water Surface Profile Plot

The regional floodplain map for the existing and proposed condition is shown in **Figure 10.** The 7 cm increase in water surface elevation caused by the proposed development has no significant impact to the extents of the regional floodplain.

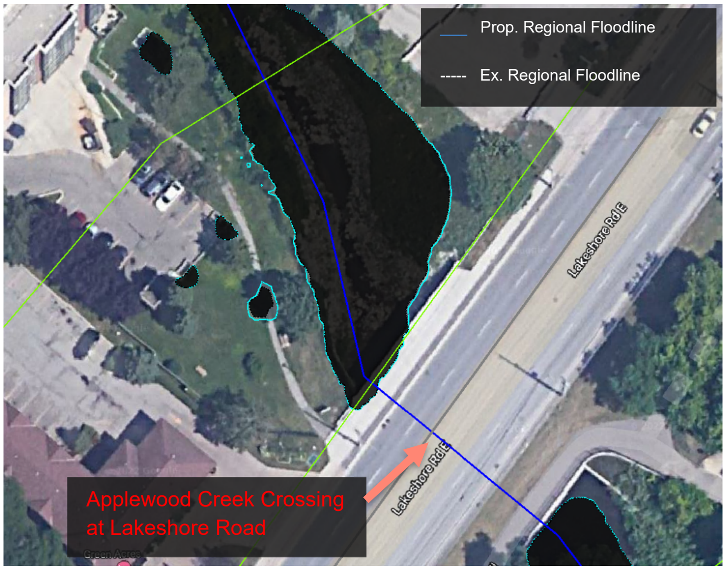


Figure 10: Applewood Creek Updated Existing and Proposed Regional Floodplain Map

For the Serson Creek Crossing, the decrease in water surface elevation caused by the proposed culvert replacement is observed for three cross sections immediately upstream before returning to the existing levels as shown in **Figure 11.** There are no downstream impacts to the water surface elevation from the proposed crossing.

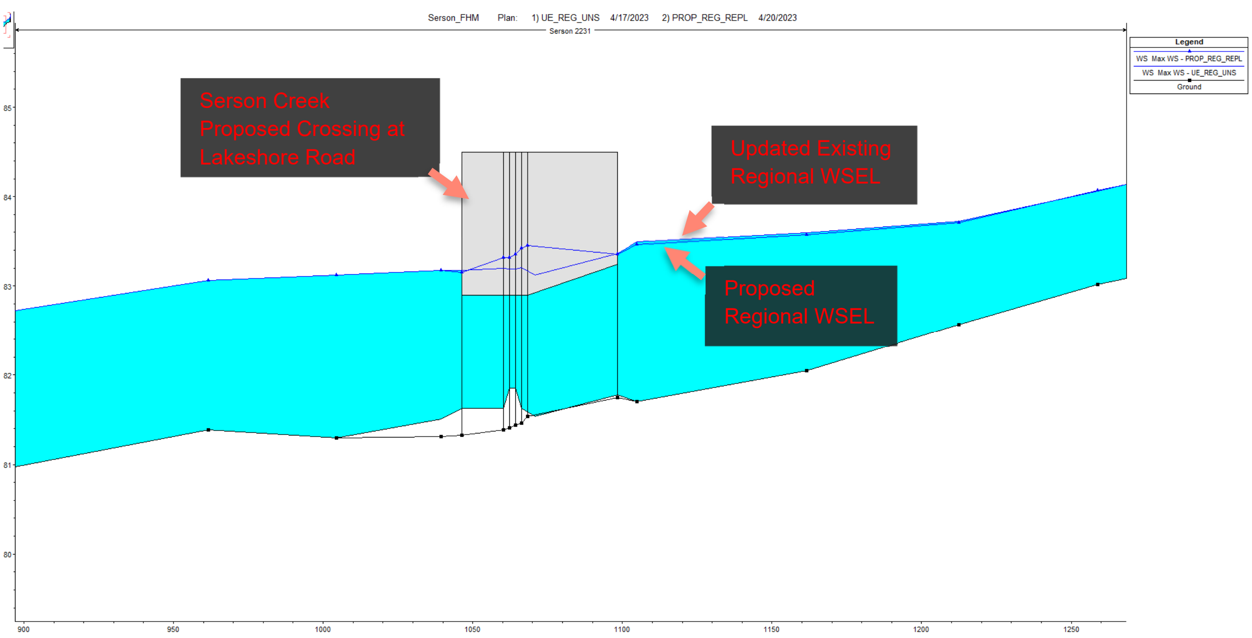


Figure 11: Serson Creek Updated Existing and Proposed Regional Water Surface Profile Plot[1]

[1] The water surface elevation line inside the culvert is not considered as valid water surface elevations. Within the bounds of a structure modeled as lidded cross sections, the projected "water surface elevation" represents the hydraulic grade line.

The updated floodplain map for the proposed culvert is shown in **Figure 12.**

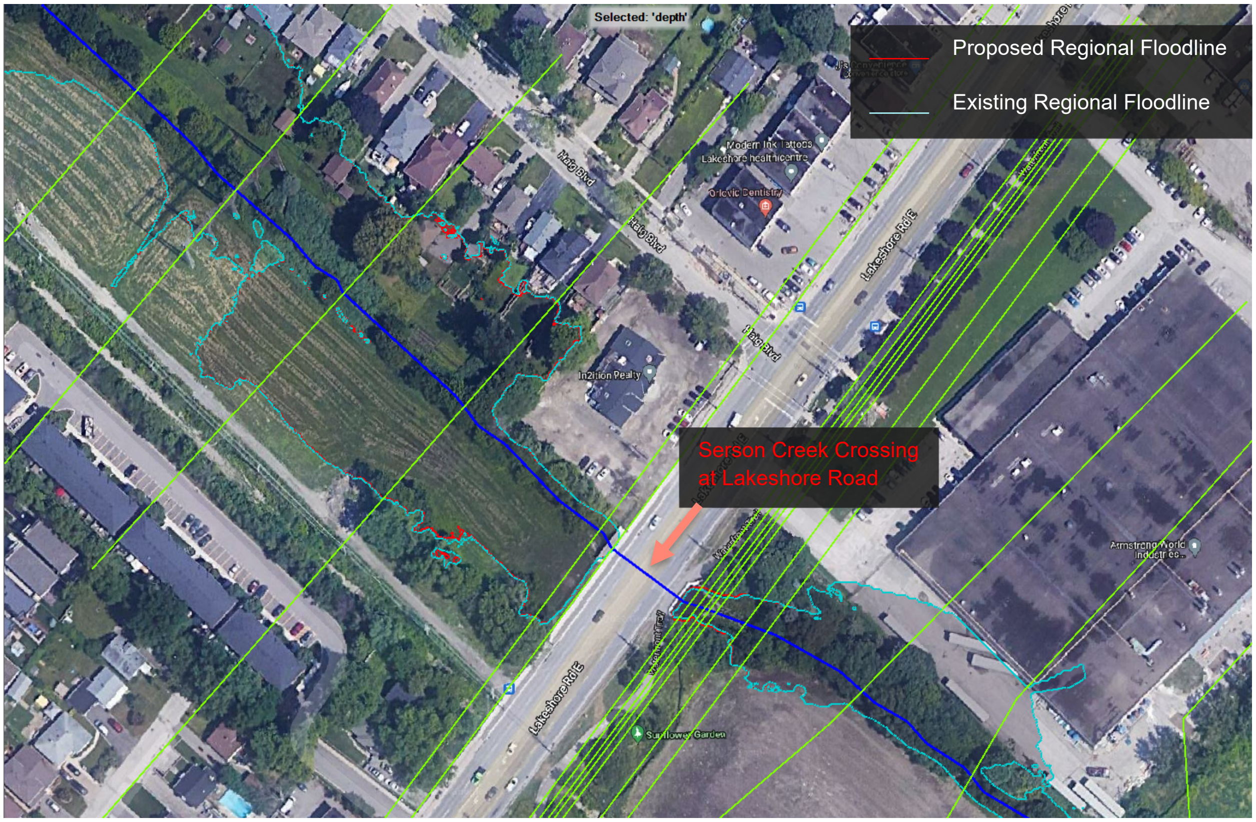


Figure 12: Serson Creek Updated Existing and Proposed Regional Floodplain Map

The updated floodplain mapping shows reductions in water surface elevations caused by the proposed culvert replacement and the corresponding decrease in regional floodplain limits. The decrease in regional floodplain limits is notable at the three cross sections immediately upstream of the structure and dissipates further upstream.

Model Validation

An additional unsteady state existing condition model was produced to validate the results of the updated unsteady models. This validation model was an identical copy of the existing condition steady state model converted to run using an unsteady flow simulation. To produce this model, the irrelevant geometry from sections upstream of 11471 was removed and the flows were converted to unsteady state format. The results of this simulation show acceptable deviations between the existing steady and existing unsteady models with respect to the subject crossing. A comparison of the water surface elevations between the existing steady state model and existing unsteady state validation model is provided in **Table 5.**

Table 5: Water Surface Elevation Comparison Between Existing Steady State and Existing Unsteady Model

River St.	Location Relative to Serson Creek Crossing	Simulation State	Water Surface Elev. (m)		
			50 Year	100 Year	Regional
11193	Upstream	Existing Steady	83.38	83.50	83.50
		Existing Unsteady	83.46	83.56	83.56
		Difference	0.08	0.06	0.06
11137	Immediately Upstream	Existing Steady	83.29	83.41	83.41
		Existing Unsteady	83.34	83.45	83.44
		Difference	0.05	0.04	0.03
11096	Immediately Downstream	Existing Steady	83.14	83.20	83.20
		Existing Unsteady	83.12	83.17	83.17
		Difference	-0.02	-0.03	-0.03

Conclusion

As part of the Lakeshore Road expansion TPAP, extension of the existing crossing at Applewood and replacement of the existing culvert at Serson Creek are proposed. Existing hydraulic models for the two crossings were provided by CVC and updated by HDR to include the proposed structure modifications and analyze the impacts to various flood levels.

The Applewood Creek Crossing is proposed to be extended 12.00 meters, from an initial length of 28.02 meters to a proposed length of 40.02 meters. This extension results in a maximum increase in water surface elevation of 5 cm, 7 cm and 7 cm for the 50-year, 100-year and Regional storm events respectively. This increase in water surface elevation does not result in any significant impact to floodplain limits. There were no impacts to the watercourse observed downstream of the proposed structure. No further updates to this model were required.

The 27.56-meter long, 8-meter wide Serson Creek Crossing is proposed to be replaced with a 47.00-meter long, 11-meters wide culvert crossing. The expansion of Lakeshore Road envelopes an existing sanitary sewer that runs perpendicular to the crossing and will cause some obstruction of flow to the proposed culvert. The irregularities caused by the obstructing sanitary sewer could not be modeled in HEC-RAS using ordinary structures in steady state. Existing and proposed scenarios were converted into unsteady state models to analyze the impacts of the proposed structure to the watercourse. A summary of the changes made to the existing, updated existing and proposed models are listed below:

- Removal of irrelevant model elements upstream of section 11471 for improved stability.
- Conversion of steady state flows into unsteady flow hydrographs and placed at corresponding flow node locations.
- Removal of original HEC-RAS structure.
- Addition of bounding cross sections to define structure as lidded section.

- Application of structure road deck and opening geometry as lid for relevant cross sections to define structure.
- Production of unsteady simulation plan files using new geometry and unsteady flow data.

The following changes were only made in the **updated existing** and **proposed** conditions models:

- Addition of four cross sections downstream of crossing to define existing sanitary sewer.
- Addition of blocked obstruction elements at the channel bottom for two of the additional sections to represent the existing sanitary sewer.

The following changes were only made in the **proposed** condition model:

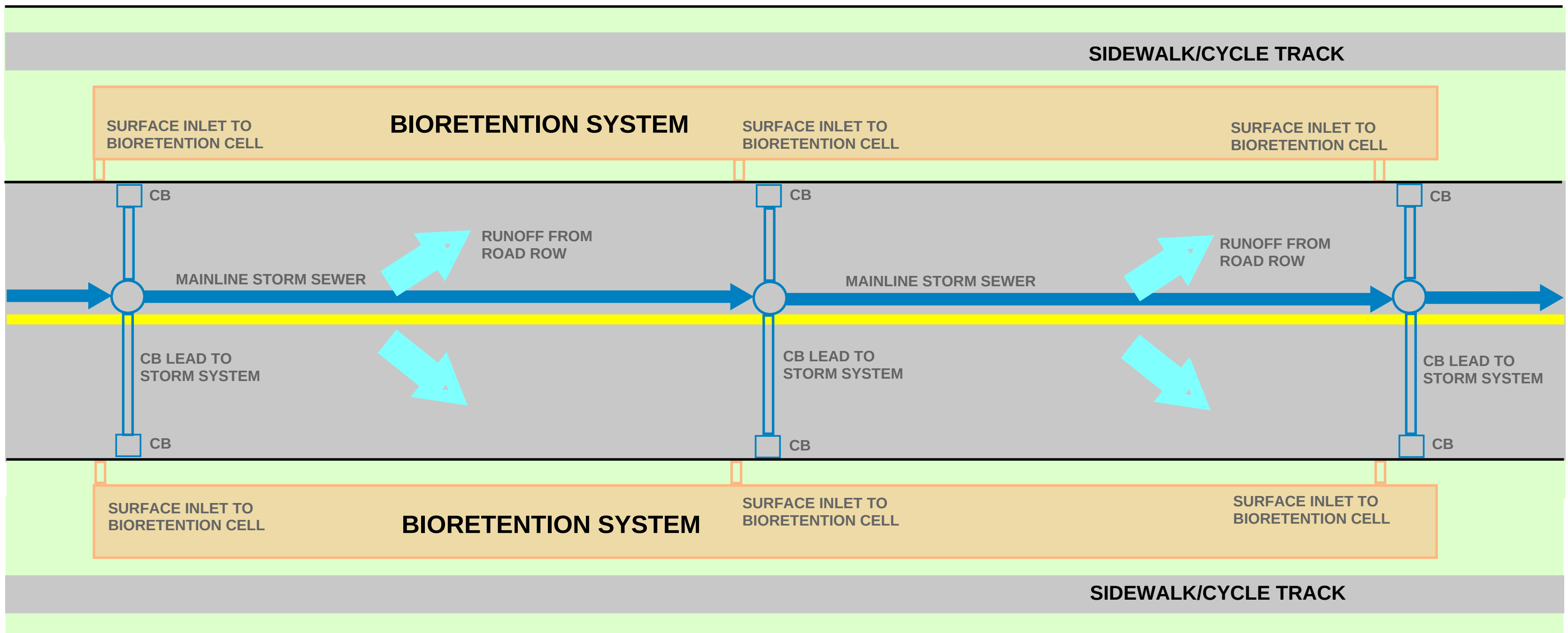
- Updating the modeled structure based on design structure.
- Adjustment of channel bed to match proposed design grades.
- Application of structure road deck and opening geometry as lid for new cross sections.

The culvert replacement results in a decrease in water surface elevation of 9 cm, 3 cm, and 3 cm for the 50-year, 100-year, and Regional water surface elevations respectively. The decrease in water surface elevation results in a decrease in floodplain limits at the three cross sections immediately upstream of the proposed culvert. There were no impacts to the watercourse observed downstream of the proposed structure.

Appendix D: Proposed Bioretention Cells Schematic

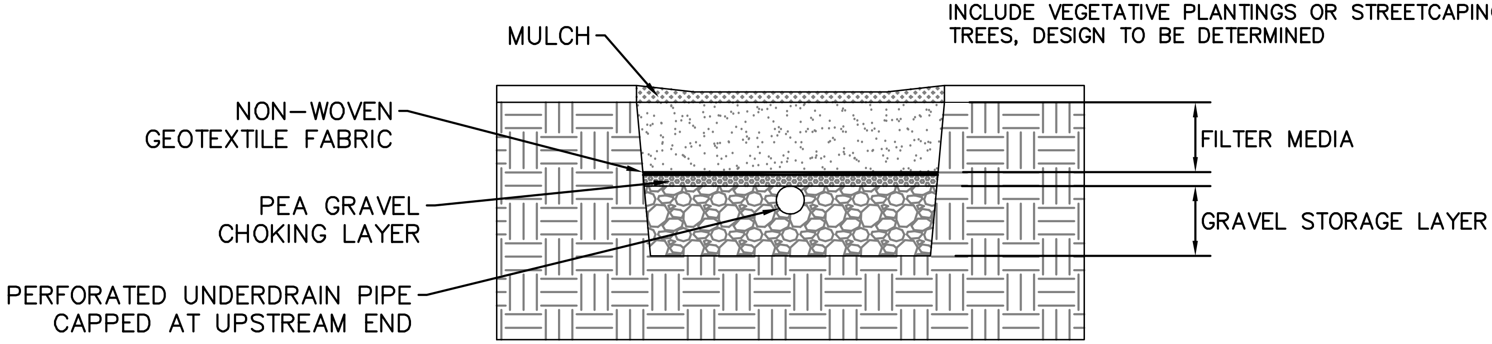
SCHEMATIC OF LINEAR LID BIORETENTION CELL FEATURE - FIGURE NOT TO SCALE

PLAN VIEW: ROAD RIGHT-OF-WAY LID IMPLEMENTATION



SECTION VIEW: SUBSURFACE DESIGN

NOTE: BIORETENTION SYSTEM CAN POTENTIALLY INCLUDE VEGETATIVE PLANTINGS OR STREETCAPING TREES, DESIGN TO BE DETERMINED



Appendix E: Detailed Calculations

hdrinc.com 100 York Boulevard, Suite 300, Richmond Hill, ON, CA L4B 1J8
(289) 695-4600

Project	Lakeshore TPAP Part A, City of Mississauga				
Date	27-Apr-23	No.	--	Page	
By	J. Look	Checked	A. Reitmeier		

Stormwater Management Calculations

TABLE 01
QUALITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	Drainage Area (ha)	Existing			Proposed			Increased Paved Area (ha)	Contributing Pavement Area (ha)	Required Treatment Volume[1] (m^3)	Required Water Balance Storage[3] (m^3)	Total Required Storage (m^3)	Required Bioretention Area[2] (m^2)	Required Bioretention Length (m)	Bioretention Facilities Width (m)	Proposed Bioretention Cell Length (m)	Provided Water Balance Storage Volume (m^3)	Required Erosion Control Storage Volume[4] (m^3)	Provided Water Quality and Erosion Control Storage Volume (m^3)
		Paved Area (ha)	% Impervious	Req. Volume (m^3)	Paved Area (ha)	% Impervious	Req. Volume (m^3)												
B-1	2.00	1.14	57%	35.59	1.68	84%	66.18	0.54	0.54	31	27	31	271	123	2.2	220	58	108	213
B-2	0.92	0.67	73%	24.11	0.86	94%	36.37	0.19	0.19	12	10	12	102	46	2.2	25	7	38	24
B-3	3.53	2.26	64%	75.34	3.31	94%	140.34	1.05	1.05	65	53	65	542	246	2.2	145	38	211	140
B-4	2.81	2.00	71%	70.98	2.63	94%	111.32	0.63	0.63	40	32	40	336	153	2.2	160	42	127	155
B-5	2.01	1.00	50%	29.03	1.10	55%	33.56	0.10	0.10	5	5	5	50	23	2.2	80	21	20	77
Total	**11.27**	**7.07**	**63%**		**9.59**	**85%**		**2.52**	**2.52**	**153**	**126**	**153**	**1300**	**591**		**630**	**166**	**504**	**610**

[1] From Table 3.2 of MOE SWM Planning and Design Manual (2003)
[2] 5% of the contributing pavement area
[3] Based on TRCA target of 5 mm retention
[4] Storage volume in addition to water balance volume to meet 25 mm retention

MOE Table 3.2

Impervious Level (%)	W.Q. Storage Volume (m^3/ha)
35%	25
55%	30
70%	35
85%	40

Bioretention Cell Dimensions

Hydraulic Conductivity [5]	1.42E-05 cm/s
Infiltration Rate, i =	31 mm/hr
Safety Factor =	3
Infilt. with Safety Factor	10.3 mm/hr
d_p =	100 mm
t_s =	48 hr
V_r =	0.4
$d_{r\ max}$ =	1237 mm
d_r =	0.3 m
Perforated Pipe	0.20 m
$d_{filter} = d_{f\ minimum}$	0.50 m
$d_{pea\ gravel}$ =	0.1 m
d_{total} =	1.10 m

LID SWM GUIDE Table C1

Kfs cm/s	T min/cm	1/T mm/hr
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Stormwater Management Calculations

Project	Lakeshore TPAP Part A, City of Mississauga	No.	--
By	J. Look	Date	27-Apr-23
Checked	A. Reitmeier	Checked	--

TABLE 02

QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	Existing			Proposed			Increased Paved Area (ha)	10 Year			100 Year			Discharge Location
	Drainage Area (ha)	Paved Area (ha)	Runoff Coefficient	Drainage Area (ha)	Paved Area (ha)	Runoff Coefficient		Existing Flow (m^3/s)	Proposed Flow (m^3/s)	Req'd Storage Vol. (m^3)	Existing Flow (m^3/s)	Proposed Flow (m^3/s)	Req'd Storage Vol. (m^3)	
B-1	1.18	1.14	0.88	2.00	1.68	0.80	0.54	0.43	0.66	97	0.76	1.16	171	Existing storm sewer
B-2	0.92	0.67	0.72	0.92	0.86	0.86	0.19	0.28	0.33	22	0.49	0.58	38	Existing storm sewer
B-3	3.11	2.26	0.72	3.53	3.31	0.86	1.05	0.93	1.26	138	1.64	2.22	242	Serson Creek
B-4	2.81	2.00	0.71	2.81	2.63	0.86	0.63	0.83	1.00	72	1.46	1.76	126	Applewood Creek
B-5	2.01	1.00	0.57	2.01	1.10	0.61	0.10	0.48	0.51	11	0.84	0.89	20	Etobicoke Creek [1]
Total	**10.03**	**7.07**		**11.27**	**9.59**		**2.52**			**339**			**597**	

Note 1 No quantity control is required for Drainage Area B5, volume shown is for information purposes only

Project	Lakeshore TPAP Part A, City of Mississauga			
Date	27-Apr-23	No.	--	Page
By	J. Look	Checked	A. Reitmeier	

Stormwater Management Calculations

TABLE 03
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B1**	
Existing Drainage Area	1.18	ha
Existing Pavement Area	1.14	ha
Existing Runoff Coefficient	0.88	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	2.00	ha
Proposed Pavement Area	1.68	ha
Proposed Runoff Coefficient	0.80	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	610	4.6	0.78	1	90.17	259.69
5-yr	820	4.6	0.78	1	121.21	349.09
10-yr	**1010**	**4.6**	**0.78**	**1**	**149.29**	**429.98**
25-yr	1160	4.6	0.78	1.1	188.61	543.22
50-yr	1300	4.7	0.78	1.2	229.05	659.70
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**756.41**

Storage Volume Calculation - 10 Year Post to 10 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	149.29	661.02	277.63	180.59	97.04
15	99.17	439.07	395.16	386.98	8.18
20	83.06	367.76	441.31	515.98	0.00
25	71.90	318.33	477.50	644.97	0.00
30	63.66	281.85	507.32	773.96	0.00
40	52.22	231.21	554.91	1031.95	0.00
50	44.60	197.46	592.38	1289.94	0.00
60	39.11	173.18	623.46	1547.93	0.00
70	34.96	154.79	650.13	1805.92	0.00
80	31.69	140.33	673.57	2063.91	0.00
90	29.05	128.62	694.53	2321.89	0.00
100	26.86	118.92	713.52	2579.88	0.00
120	23.43	103.75	747.00	3095.86	0.00
360	10.14	44.90	969.90	9287.58	0.00
720	5.94	26.28	1135.26	18575.15	0.00
1440	3.47	15.34	1325.56	37150.31	0.00
Required Storage Volume:			**97.04**		

Storage Volume Calculation - 100 Year Post to 100 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	1162.84	488.39	317.69	170.70
15	175.86	778.65	700.78	680.77	20.02
20	147.65	653.75	784.50	907.69	0.00
25	128.01	566.79	850.19	1134.61	0.00
30	113.47	502.39	904.31	1361.53	0.00
40	93.22	412.76	990.62	1815.38	0.00
50	79.69	352.84	1058.53	2269.22	0.00
60	69.94	309.67	1114.80	2723.07	0.00
70	62.54	276.92	1163.05	3176.91	0.00
80	56.72	251.13	1205.42	3630.76	0.00
90	52.00	230.24	1243.29	4084.60	0.00
100	48.09	212.93	1277.59	4538.45	0.00
120	41.97	185.84	1338.01	5446.14	0.00
360	18.19	80.53	1739.42	16338.41	0.00
720	10.65	47.14	2036.64	32676.81	0.00
1440	6.22	27.53	2378.41	65353.62	0.00
Required Storage Volume:			**170.70**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**0.661**	m^3/s	10 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.430**	m^3/s	10 Year Existing Flow
Required Storage	**97.04**	m^3	
Uncontrolled Discharge Flow Rate	**1.163**	m^3/s	100 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.756**	m^3/s	100 Year Existing Flow
Required Pipe Storage	**170.70**	m^3	

Project	Lakeshore TPAP Part A, City of Mississauga				
Date	27-Apr-23	No.	--	Page	
By	J. Look	Checked	A. Reitmeier		

Stormwater Management Calculations

TABLE 04
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B2**	
Existing Drainage Area	0.92	ha
Existing Pavement Area	0.67	ha
Existing Runoff Coefficient	0.72	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	0.92	ha
Proposed Pavement Area	0.86	ha
Proposed Runoff Coefficient	0.86	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	610	4.6	0.78	1	90.17	166.82
5-yr	820	4.6	0.78	1	121.21	224.25
10-yr	**1010**	**4.6**	**0.78**	**1**	**149.29**	**276.21**
25-yr	1160	4.6	0.78	1.1	188.61	348.95
50-yr	1300	4.7	0.78	1.2	229.05	423.77
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**485.90**

Storage Volume Calculation - 10 Year Post to 10 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	149.29	327.74	137.65	116.01	21.64
15	99.17	217.69	195.93	248.59	0.00
20	83.06	182.34	218.81	331.45	0.00
25	71.90	157.83	236.75	414.31	0.00
30	63.66	139.74	251.54	497.18	0.00
40	52.22	114.64	275.13	662.90	0.00
50	44.60	97.90	293.71	828.63	0.00
60	39.11	85.87	309.12	994.35	0.00
70	34.96	76.75	322.35	1160.08	0.00
80	31.69	69.58	333.96	1325.80	0.00
90	29.05	63.77	344.36	1491.53	0.00
100	26.86	58.96	353.77	1657.25	0.00
120	23.43	51.44	370.37	1988.70	0.00
360	10.14	22.26	480.89	5966.10	0.00
720	5.94	13.03	562.88	11932.21	0.00
1440	3.47	7.61	657.23	23864.41	0.00
Required Storage Volume:			**21.64**		

Storage Volume Calculation - 100 Year Post to 100 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	576.55	242.15	204.08	38.07
15	175.86	386.06	347.46	437.31	0.00
20	147.65	324.14	388.96	583.08	0.00
25	128.01	281.02	421.53	728.85	0.00
30	113.47	249.09	448.37	874.61	0.00
40	93.22	204.65	491.16	1166.15	0.00
50	79.69	174.94	524.83	1457.69	0.00
60	69.94	153.54	552.73	1749.23	0.00
70	62.54	137.30	576.66	2040.77	0.00
80	56.72	124.51	597.67	2332.31	0.00
90	52.00	114.16	616.44	2623.84	0.00
100	48.09	105.57	633.45	2915.38	0.00
120	41.97	92.14	663.40	3498.46	0.00
360	18.19	39.93	862.43	10495.38	0.00
720	10.65	23.37	1009.79	20990.75	0.00
1440	6.22	13.65	1179.25	41981.50	0.00
Required Storage Volume:			**38.07**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**0.328**	m^3/s	10 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.276**	m^3/s	10 Year Existing Flow
Required Storage	**21.64**	m^3	
Uncontrolled Discharge Flow Rate	**0.577**	m^3/s	100 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.486**	m^3/s	100 Year Existing Flow
Required Pipe Storage	**38.07**	m^3	

Project	Lakeshore TPAP Part A, City of Mississauga				
Date	27-Apr-23	No.	--	Page	
By	J. Look	Checked	A. Reitmeier		

Stormwater Management Calculations

TABLE 05
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B3**	
Existing Drainage Area	3.11	ha
Existing Pavement Area	2.26	ha
Existing Runoff Coefficient	0.72	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	3.53	ha
Proposed Pavement Area	3.31	ha
Proposed Runoff Coefficient	0.86	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	610	4.6	0.78	1	90.17	563.12
5-yr	820	4.6	0.78	1	121.21	756.99
10-yr	**1010**	**4.6**	**0.78**	**1**	**149.29**	**932.39**
25-yr	1160	4.6	0.78	1.1	188.61	1177.94
50-yr	1300	4.7	0.78	1.2	229.05	1430.51
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**1640.22**

Storage Volume Calculation - 10 Year Post to 10 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	149.29	1260.47	529.40	391.60	137.80
15	99.17	837.24	753.52	839.15	0.00
20	83.06	701.26	841.51	1118.86	0.00
25	71.90	607.02	910.53	1398.58	0.00
30	63.66	537.44	967.39	1678.29	0.00
40	52.22	440.89	1058.13	2237.72	0.00
50	44.60	376.53	1129.59	2797.16	0.00
60	39.11	330.24	1188.86	3356.59	0.00
70	34.96	295.17	1239.72	3916.02	0.00
80	31.69	267.58	1284.40	4475.45	0.00
90	29.05	245.25	1324.37	5034.88	0.00
100	26.86	226.76	1360.59	5594.31	0.00
120	23.43	197.84	1424.42	6713.17	0.00
360	10.14	85.62	1849.47	20139.52	0.00
720	5.94	50.11	2164.79	40279.04	0.00
1440	3.47	29.26	2527.67	80558.08	0.00
Required Storage Volume:			**137.80**		

Storage Volume Calculation - 100 Year Post to 100 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	2217.38	931.30	688.89	242.41
15	175.86	1484.78	1336.30	1476.20	0.00
20	147.65	1246.61	1495.93	1968.27	0.00
25	128.01	1080.79	1621.19	2460.33	0.00
30	113.47	957.99	1724.39	2952.40	0.00
40	93.22	787.07	1888.97	3936.53	0.00
50	79.69	672.82	2018.46	4920.67	0.00
60	69.94	590.49	2125.78	5904.80	0.00
70	62.54	528.04	2217.79	6888.93	0.00
80	56.72	478.87	2298.58	7873.07	0.00
90	52.00	439.03	2370.79	8857.20	0.00
100	48.09	406.03	2436.20	9841.33	0.00
120	41.97	354.36	2551.41	11809.60	0.00
360	18.19	153.56	3316.84	35428.79	0.00
720	10.65	89.90	3883.59	70857.59	0.00
1440	6.22	52.49	4535.30	141715.17	0.00
Required Storage Volume:			**242.41**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**1.260**	m^3/s	10 Year Proposed Condiitions
Controlled Discharge Flow Rate	**0.932**	m^3/s	10 Year Existing Flow
Required Storage	**137.80**	m^3	
Uncontrolled Discharge Flow Rate	**2.217**	m^3/s	100 Year Proposed Condiitions
Controlled Discharge Flow Rate	**1.640**	m^3/s	100 Year Existing Flow
Required Pipe Storage	**242.41**	m^3	

Project	Lakeshore TPAP Part A, City of Mississauga			
Date	27-Apr-23	No.	--	Page
By	J. Look	Checked	A. Reitmeier	

Stormwater Management Calculations

TABLE 05A
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B3**	
Existing Drainage Area	3.11	ha
Existing Pavement Area	2.26	ha
Existing Runoff Coefficient	0.72	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	3.53	ha
Proposed Pavement Area	3.31	ha
Proposed Runoff Coefficient	0.86	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	**610**	**4.6**	**0.78**	**1**	**90.17**	**563.12**
5-yr	820	4.6	0.78	1	121.21	756.99
10-yr	1010	4.6	0.78	1	149.29	932.39
25-yr	1160	4.6	0.78	1.1	188.61	1177.94
50-yr	1300	4.7	0.78	1.2	229.05	1430.51
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**1640.22**

Storage Volume Calculation - 100 Year Post to 2 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	2217.38	931.30	236.51	694.79
15	175.86	1484.78	1336.30	506.81	829.49
20	147.65	1246.61	1495.93	675.75	820.18
25	128.01	1080.79	1621.19	844.69	776.50
30	113.47	957.99	1724.39	1013.62	710.76
40	93.22	787.07	1888.97	1351.50	537.47
50	79.69	672.82	2018.46	1689.37	329.09
60	69.94	590.49	2125.78	2027.25	98.53
70	62.54	528.04	2217.79	2365.12	0.00
80	56.72	478.87	2298.58	2702.99	0.00
90	52.00	439.03	2370.79	3040.87	0.00
100	48.09	406.03	2436.20	3378.74	0.00
120	41.97	354.36	2551.41	4054.49	0.00
360	18.19	153.56	3316.84	12163.47	0.00
720	10.65	89.90	3883.59	24326.94	0.00
1440	6.22	52.49	4535.30	48653.89	0.00
Required Storage Volume:			**829.49**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**2.217**	m^3/s	100 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.563**	m^3/s	2 Year Existing Flow
Required Storage	**829.49**	m^3	

Stormwater Management Calculations

Project	Lakeshore TPAP Part A, City of Mississauga			
Date	27-Apr-23	No.	--	Page
By	J. Look	Checked	A. Reitmeier	

TABLE 06
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B4**	
Existing Drainage Area	2.81	ha
Existing Pavement Area	2.00	ha
Existing Runoff Coefficient	0.71	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	2.81	ha
Proposed Pavement Area	2.63	ha
Proposed Runoff Coefficient	0.86	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	610	4.6	0.78	1	90.17	501.96
5-yr	820	4.6	0.78	1	121.21	674.77
10-yr	**1010**	**4.6**	**0.78**	**1**	**149.29**	**831.12**
25-yr	1160	4.6	0.78	1.1	188.61	1050.00
50-yr	1300	4.7	0.78	1.2	229.05	1275.14
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**1462.07**

Storage Volume Calculation - 10 Year Post to 10 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	149.29	1001.86	420.78	349.07	71.71
15	99.17	665.46	598.92	748.00	0.00
20	83.06	557.38	668.86	997.34	0.00
25	71.90	482.48	723.71	1246.67	0.00
30	63.66	427.17	768.91	1496.01	0.00
40	52.22	350.43	841.03	1994.68	0.00
50	44.60	299.28	897.83	2493.35	0.00
60	39.11	262.48	944.94	2992.02	0.00
70	34.96	234.61	985.36	3490.69	0.00
80	31.69	212.68	1020.88	3989.36	0.00
90	29.05	194.93	1052.64	4488.02	0.00
100	26.86	180.24	1081.43	4986.69	0.00
120	23.43	157.25	1132.17	5984.03	0.00
360	10.14	68.06	1470.01	17952.10	0.00
720	5.94	39.83	1720.64	35904.20	0.00
1440	3.47	23.25	2009.06	71808.39	0.00
Required Storage Volume:			**71.71**		

Storage Volume Calculation - 100 Year Post to 100 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	1762.43	740.22	614.07	126.15
15	175.86	1180.14	1062.13	1315.86	0.00
20	147.65	990.84	1189.01	1754.49	0.00
25	128.01	859.04	1288.57	2193.11	0.00
30	113.47	761.44	1370.59	2631.73	0.00
40	93.22	625.59	1501.41	3508.97	0.00
50	79.69	534.78	1604.33	4386.22	0.00
60	69.94	469.34	1689.63	5263.46	0.00
70	62.54	419.70	1762.76	6140.70	0.00
80	56.72	380.62	1826.98	7017.94	0.00
90	52.00	348.96	1884.37	7895.19	0.00
100	48.09	322.73	1936.36	8772.43	0.00
120	41.97	281.66	2027.93	10526.92	0.00
360	18.19	122.05	2636.32	31580.75	0.00
720	10.65	71.45	3086.78	63161.50	0.00
1440	6.22	41.72	3604.79	126323.01	0.00
Required Storage Volume:			**126.15**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**1.002**	m^3/s	10 Year Proposed Condiitions
Controlled Discharge Flow Rate	**0.831**	m^3/s	10 Year Existing Flow
Required Storage	**71.71**	m^3	
Uncontrolled Discharge Flow Rate	**1.762**	m^3/s	100 Year Proposed Condiitions
Controlled Discharge Flow Rate	**1.462**	m^3/s	100 Year Existing Flow
Required Pipe Storage	**126.15**	m^3	

TABLE 06A
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B4**	
Existing Drainage Area	2.81	ha
Existing Pavement Area	2.00	ha
Existing Runoff Coefficient	0.71	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	2.81	ha
Proposed Pavement Area	2.63	ha
Proposed Runoff Coefficient	0.86	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$				Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
	A	B	C	C_f		
2-yr	**610**	**4.6**	**0.78**	**1**	**90.17**	**501.96**
5-yr	820	4.6	0.78	1	121.21	674.77
10-yr	1010	4.6	0.78	1	149.29	831.12
25-yr	1160	4.6	0.78	1.1	188.61	1050.00
50-yr	1300	4.7	0.78	1.2	229.05	1275.14
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**1462.07**

Storage Volume Calculation - 100 Year Post to 2 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	1762.43	740.22	210.82	529.40
15	175.86	1180.14	1062.13	451.76	610.36
20	147.65	990.84	1189.01	602.35	586.66
25	128.01	859.04	1288.57	752.94	535.63
30	113.47	761.44	1370.59	903.53	467.06
40	93.22	625.59	1501.41	1204.71	296.70
50	79.69	534.78	1604.33	1505.88	98.45
60	69.94	469.34	1689.63	1807.06	0.00
70	62.54	419.70	1762.76	2108.24	0.00
80	56.72	380.62	1826.98	2409.41	0.00
90	52.00	348.96	1884.37	2710.59	0.00
100	48.09	322.73	1936.36	3011.77	0.00
120	41.97	281.66	2027.93	3614.12	0.00
360	18.19	122.05	2636.32	10842.36	0.00
720	10.65	71.45	3086.78	21684.71	0.00
1440	6.22	41.72	3604.79	43369.43	0.00
Required Storage Volume:			**610.36**		

Uncontrolled Discharge Flow Rate	**1.762**	m^3/s	100 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.502**	m^3/s	2 Year Existing Flow
Required Pipe Storage	**610.36**	m^3	

Project	Lakeshore TPAP Part A, City of Mississauga			
Date	27-Apr-23	No.	--	Page
By	J. Look	Checked	A. Reitmeier	

Stormwater Management Calculations

TABLE 07
DRAINAGE AREA QUANTITY CONTROL REQUIREMENT CALCULATION

Drainage Area ID	**B5**	
Existing Drainage Area	2.01	ha
Existing Pavement Area	1.00	ha
Existing Runoff Coefficient	0.57	*Assume pavement C = 0.9, landscaped C = 0.25*
Proposed Drainage Area	2.01	ha
Proposed Pavement Area	1.10	ha
Proposed Runoff Coefficient	0.61	*Assume pavement C = 0.9, landscaped C = 0.25*
Time of Concentration	7	minute

City of Mississauga Rainfall Parameters

Return Period	IDF Parameters $i = C_f \times A / (Tc + B)^C$ A	B	C	C_f	Rainfall Intensity (mm/hr)	Allowable Release Rate (L/s)
2-yr	610	4.6	0.78	1	90.17	288.89
5-yr	820	4.6	0.78	1	121.21	388.35
10-yr	**1010**	**4.6**	**0.78**	**1**	**149.29**	**478.33**
25-yr	1160	4.6	0.78	1.1	188.61	604.31
50-yr	1300	4.7	0.78	1.2	229.05	733.88
100-yr	**1450**	**4.9**	**0.78**	**1.25**	**262.63**	**841.47**

Storage Volume Calculation - 10 Year Post to 10 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	149.29	505.15	212.16	200.90	11.26
15	99.17	335.53	301.98	430.50	0.00
20	83.06	281.04	337.25	574.00	0.00
25	71.90	243.27	364.90	717.50	0.00
30	63.66	215.39	387.69	861.00	0.00
40	52.22	176.69	424.06	1148.00	0.00
50	44.60	150.90	452.70	1435.00	0.00
60	39.11	132.35	476.45	1722.00	0.00
70	34.96	118.29	496.83	2009.00	0.00
80	31.69	107.24	514.74	2296.00	0.00
90	29.05	98.29	530.76	2583.00	0.00
100	26.86	90.88	545.27	2869.99	0.00
120	23.43	79.29	570.85	3443.99	0.00
360	10.14	34.31	741.20	10331.98	0.00
720	5.94	20.08	867.57	20663.96	0.00
1440	3.47	11.72	1012.99	41327.93	0.00
Required Storage Volume:			**11.26**		

Storage Volume Calculation - 100 Year Post to 100 Year Pre

Time (minutes)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)	Storm Runoff Volume (m^3)	Ex. Discharge Flow Vol. (m^3)	Required Storage Volume (m^3)
7	262.63	888.64	373.23	353.42	19.81
15	175.86	595.04	535.54	757.32	0.00
20	147.65	499.59	599.51	1009.76	0.00
25	128.01	433.14	649.71	1262.20	0.00
30	113.47	383.93	691.07	1514.64	0.00
40	93.22	315.43	757.03	2019.52	0.00
50	79.69	269.64	808.92	2524.40	0.00
60	69.94	236.65	851.93	3029.28	0.00
70	62.54	211.62	888.80	3534.16	0.00
80	56.72	191.91	921.18	4039.04	0.00
90	52.00	175.95	950.12	4543.92	0.00
100	48.09	162.72	976.34	5048.80	0.00
120	41.97	142.01	1022.51	6058.56	0.00
360	18.19	61.54	1329.26	18175.69	0.00
720	10.65	36.03	1556.39	36351.38	0.00
1440	6.22	21.04	1817.58	72702.75	0.00
Required Storage Volume:			**19.81**		

Required Storage Summary

Uncontrolled Discharge Flow Rate	**0.505**	m^3/s	10 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.478**	m^3/s	10 Year Existing Flow
Required Storage	**11.26**	m^3	
Uncontrolled Discharge Flow Rate	**0.889**	m^3/s	100 Year Proposed Conditions
Controlled Discharge Flow Rate	**0.841**	m^3/s	100 Year Existing Flow
Required Pipe Storage	**19.81**	m^3	