ENHANCED FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

1200 King Road

City of Burlington

Prepared for

Alinea Land Corporation
Project #: 22-287

OPA and ZBA Submission: February 2026

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1. INTRODUCTION

The purpose of this report is to outline the servicing details for the proposed storm drainage, sanitary and water distribution systems required to service the subject development in support of official plan amendment and rezoning applications.

The recommended servicing plans have been prepared in accordance with design criteria and requirements of the City of Burlington and Halton Region. The information in this report is intended to assist the regulatory agencies in their review of the application for the proposed development.

2. BACKGROUND

2.1. Location and Description

The current property owner is proposing the development of the existing grassed and wooded lot located at 1200 King Road in the City of Burlington. The subject lands are approximately 49.1 ha in size and the property is bounded by the following:

- To the north by Highway 403 and an existing hydro corridor.
- To the south by a CN rail line.
- To the east by King Road.
- To the west by the Aldershot GO station and Alayne Bryk Wy.

The property is described as Part of Lots 3, 4,5 and 6, Concession 1, Part 1 RP 20R -11103, and Part 1 RP 20R-15410, formerly of the Township of East Flamborough, now in the City of Burlington, Regional Municipality of Halton.

The site is located within three subwatersheds under Conservation Halton (CH) jurisdiction, the east portion lies within Indian Creek (29.3 ha), the central area is within Falcon Creek (10.3 ha) and the west portion of the site is within Grindstone Creek (9.7 ha).

2.2. Proposed Development

The development consists of:

- Residential (26.01 acres)
- Mixed-use residential (11.95 acres)
- Entertainment district (7.24 acres)
- Campus mixed-use (11.17 acres)
- Future Development (1.54 acres)
- Park land (5.03 acres)
- Landscaped Buffer (1.13 acres)
- Creek (4.63 acres)
- Stormwater pond (4.62 acres)
- NHS area (25.04 acres)
- MTO buffer (3.14 acres)
- Public roads (19.82 acres)

The proposed development will have street connections to King Road and Alayne Bryk Wy.

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2.3. Background Studies

The servicing and development concepts presented within this report are an extension of the information contained in the following reports:

- ZBA Architectural Plans Set, Giannone Petricone Associates, Feb 2026
- Environmental Impact Assessment, Natural Resource Solutions Inc., Feb 2026
- Fluvial Geomorphology Assessment: Grindstone Creek, Falcon Creek and Indian Creek, GEO Morphix, Feb 2026
- Preliminary Geotechnical Investigation, DS Consultants LTD., Feb 2026
- Preliminary Hydrogeological and Hydrological Assessment, Terra-Dynamics Inc., Feb 2026
- Pre-development Wetland Water Balance, Terra-Dynamics Inc., Feb 2026
- Plan of Survey, MacKay MacKay & Peters Limited
- Falcon Creek Hydrology and Hydraulics Study, Valdor Engineering Inc, May 2012.
- Grindstone Creek Watershed Study, CH, June 1998
- Guidelines for Stormwater Management Engineering Submissions, CH, June 2024
- Stormwater Management Design Guidelines, City of Burlington, May 2020
- Stormwater Management Planning and Design Manual, Ontario Ministry of the Environment, 2003
- Water and Wastewater Linear Design Manual, December 2025, Halton Region
- AMEC Indian Creek/Grade Separation Design, 2013
- 2022 Development Charges Update Water/Wastewater Technical Report, GMBluePlan, Sept 2021
- Region of Halton Integrated Master Plan Water and Sanitary Infrastructure Program, November 2025 (draft to Council)
- Burlington Major Transit Station Areas (MTSA) Area Specific Plans Functional Servicing Report, WSP, Jul 2023

3. EXISTING CONDITIONS

3.1. Land Use

Under existing conditions, the site consists of agricultural fields on the eastern section of the property. The lands to the south and west of the site are generally covered with trees.

3.2. Geotechnical and Hydrogeology

In support of the rezoning application, site specific geotechnical investigations were prepared by Soil-Mat Engineers and Consultants LTD; and hydrogeological investigations were prepared by Terra-Dynamics Consulting Inc.

The geotechnical investigation states that the site's soil stratigraphy is generally characterized by the following:

- The site topography slopes downwards from the northwest to southeast with a total relief of approximately 8 to 10 meters.
- There is a drainage ditch running north to south toward the east of the site.
- A surface veneer of topsoil approximately 100 to 300 mm in thickness was found at the borehole locations, with the exception of borehole No. 7.
- The fill material is generally brown and reddish brown, has occasional medium to coarse grained sand, and contains gravel.
- The fill material was found to extend to depths of up to approximately 1.3 to 1.4 meters below the existing grade.
- Deposits of silty sand and sandy silt were found interbedded in the silty clay deposit. Notably, these sandy layers were encountered from approximately 1.4 to 2.1 and 5.9 to 6.4 meters below the existing grade.
- A native Queenston Shale Bedrock was encountered beneath the silty clay till at depths of about 7.5 to 8.8 meters below grade in Boreholes Nos. 1, 2 and 5. The Queenston Shale was present to the termination of Borehole Nos. 1, 2, and 5 at depths of approximately 7.9 and 9.2 meters below grade.
- The conditions encountered in the boreholes are generally considered to be suitable for site development.

The hydrogeological review indicated the following:

- Groundwater flow in the clay-based overburden deposits is generally southward towards Hamilton Harbour and localized groundwater flow at very slow velocities of approximately 4 cm/year occurs towards both Indian and Falcon Creeks.
- Silty clay and silty clay with stones (Halton Till) overlie the Queenston Shale across the property.
- The geometric mean of nine hydraulic conductivity tests in the clay aquitard was calculated to be 9.3×10^{-9} m/s.

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4. GRADING DESIGN & ROADWAYS

4.1. Design Standards

A grading plan for the subject property has been prepared in conjunction with the storm, sanitary, and water servicing system design for the development. The grading information for the development is provided on the enclosed **GRD-1 and GRD-2**, included in **Appendix A**.

The site grading plan has been prepared with the following considerations:

1. Conforms to the City of Burlington design criteria,
2. Matches existing boundary conditions,
3. Provides safe overland flow conveyance for major storm conditions,
4. Raises grades where practical to mitigate ground water table related issues,
5. Ensures no depth of potential ponding greater than 0.30m, and
6. Provides appropriate cover for proposed servicing.

4.2. Grading Design

Refer to **Drawing GRD-1**, **GRD-2** and **GRD-3** for the conceptual grading designs for the site.

The site has the following grading characteristics:

- Road gradients in the range of 0.50% – 4.0%.
- Generally overland flow conveyance is to the proposed stormwater storage facilities.
- 100-year capture is proposed along Street D, which discharges to Pond 1 as a continuous overland flow route was not suitable.
- An emergency overland outlet has been provided from Street D through private property to Pond 2. The overland flow route will be contained in an easement.
- An open space channel has been provided to contain the regional flood flows associated with Indian Creek.
- Adjacent to Street D in the vicinity of Street A, a grading buffer is proposed to transition grades to the existing condition. Additionally, within the entertainment district, a similar grading buffer will be necessary. The purpose of the grading buffers is to ensure that the spill from Falcon/Indian Creek is removed from the subject lands.

5. WATER SERVICING

5.1. Design Criteria

The proposed watermain design will comply with the Halton Region design criteria (as outlined in the December 2025 Linear Design Manual) as follows:

- Average Day Demand: 230 L/person/day (residential), 190 L/employee/day (ICI)
- Peaking Factors:
  - Maximum Day Lake-Based Supply: 1.9
  - Maximum Hourly Demand: 3.0
- Normal operating pressure range in the distribution system: 275 kPa (40 psi) and 690 kPa (100 psi)
- Minimum pressure under max day plus fire flow: 140 kPa (20 psi)

Population estimates for the 1200 King Site are calculated based on an average 600 ppl/ha population density for net developable areas (per Region of Halton Linear Design Manual - Apartments in intensification areas). This aggregate density is representative of typical high-density redevelopment areas. Population estimates will be refined as planning proceeds. The 600 ppl/ha assumption is consistent with initial population totals at the block level.

5.2. Existing Conditions

The 1200 King Road property is currently located within the Region of Halton Pressure Zone B1A, with a top water level (TWL) of 140 m. This area of Burlington receives water from the Kingsway Booster Pumping Station (30 ML/d), with floating storage provided by the Waterdown Reservoir (7.5 ML capacity).

Existing water infrastructure local to the site includes a Zone B1A 350 mm diameter watermain located within the King Road ROW adjacent to the property, and a 600 mm watermain on Waterdown Road to the west. There is also a network of local watermains (150 mm to 300 mm) in the neighbourhood to the south.

North of the site, there is an existing 300/350 mm Zone B2 (TWL 168 m) watermain located on North Service Road, north of Highway 403.

Existing infrastructure is shown on **Drawing SERV-1.**

While the site has good access to existing water services, there are known distribution pressure issues in this area of Burlington, given the ground elevation relative to the rest of Zone B1A. Through previous correspondence regarding this site, the Region has noted that pressures in some areas of the 1200 King Road site may only be in the 35+ psi range, below the Regional minimum standard of 40 psi.

5.3. Proposed Water Supply and Distribution System

5.3.1 Region of Halton Master Plan Strategy

The Region of Halton is currently updating their Water and Wastewater Infrastructure Master Plan. The previous Master Plan was completed in 2011 (Sustainable Halton) and extends to the 2031 timeframe. The new Master Plan covers infrastructure needs to 2051. The Region intends to finalize the Master Plan in Q2 of 2026, but the overall capital program to 2051 was approved by Council in November 2025.

The Council approved capital plan includes the following proposed updates which impact the 1200 King Road development site:

- Zone B1A will be eliminated and transitioned to the higher Zone B2. This will be achieved through upgrades at the Kingsway Booster Pump Station and through extension of watermain infrastructure on King Road and on North Service Road.

- The existing B1A Waterdown Reservoir will be decommissioned. Floating storage for the new Zone B2 areas will be supplied by existing Zone B2 Reservoirs and supplemented by a new Zone B2 Reservoir on Waterdown Road.

- Capital infrastructure associated with the Zone B1A to Zone B2 pressure boundary re-alignment is scheduled for construction in the 2028 – 2031 timeframe.

When the pressure zone boundary re-alignment is complete, the 1200 King Road site will be serviced fully by Zone B2. This change in zone will eliminate the low pressure concerns on the west side of the site.

In addition to new Master Plan infrastructure, the Region has one existing project in their capital implementation plan to 2031 that impacts this area of Burlington:

- **ID5881:** New 600 mm WM (Zone B2) along North Servicing Road from King Road to the Waterdown Booster Pumping Station.

This project supports the Zone B1A to Zone B2 boundary adjustment. It is fully funded and approved through the Region's 2023 Allocation Program. The completion of this project is critical as it supports all development in North Aldershot.

5.3.2 1200 King Road – Proposed Water Servicing

The proposed water distribution plan for the 1200 King Road site is shown on **Drawing WM-1**. The site will be serviced by an internal network of local 300 mm pipes. The site is connected to the Regional water system as follows:

- At King Road, via two (2) connections to the existing 350 mm watermain (Street 'A' and Street 'B).
- At Waterdown Road, via a new 300 mm watermain connection, as an extension of Street 'A' along Alayne Bryk Way. The 300 mm watermain is planned to connect to the existing 600 mm B1A watermain on Waterdown Road.

The 300 mm watermain along Street 'A' (new service road) is the main watermain spine for the area, providing connection from King Road to Waterdown Road and looping for the system.

For the purposes of preliminary planning, the site is currently split into two development Phases, as shown in **Drawing WM-1**. The east portion of the site (Phase 1) is planned to proceed on Zone B1A in the near-

term. Hydraulic modeling (detailed in **Section 5.4**) shows that distribution pressures can be maintained above the Region's level-of-service requirements for areas generally less than 107 m in elevation (Phase 1). Fire flows can be achieved with the Region's existing system, in advance of any additional upgrades.

The central and west portions of the 1200 King Road site are currently shown in Phase 2, intended to develop after the pressure boundary re-alignment is complete. Elevations in these areas are such that distribution pressures cannot be met under existing conditions, but will be achieved once the area is transitioned to Zone B2. It is noted that fire flows for Phase 2 areas can be met regardless of the pressure zone. As planning proceeds, there may be adjustments to the Phasing delineation to accommodate priority areas (for example, the Event Centre and Mohawk College). This will be coordinated through discussions with the Region and the City.

The 300 mm watermain along Street 'A' is anticipated to proceed as part of Phase 1 (in coordination with the road works) to provide full looping for the site. As confirmed with hydraulic modeling (see **Section 5.4**, Scenario 1 and Scenario 3 results), the full watermain along Street 'A' is not required in order to proceed with development in Phase 1 (there is sufficient pressure and fire flow with only the King Road connection), but it does increase fire flow availability across the plan. See **Section 10** for additional information regarding phasing.

The following will be incorporated into the site design:

- Hydrant spacing and coverage will be provided in accordance with Region of Halton and Fire Underwriters Survey Guidelines.
- Proper cover and utility separation in accordance with Region of Halton Standards.
- Water infrastructure will be located within road right-of-ways, with the exception of the easement under the rail line. This is an existing corridor, however, with good access.
- The system will be properly looped to minimize dead end watermains. Consideration for interim conditions will be evaluated, depending on phasing.

5.4. Hydraulic Modelling Analysis

Region Staff have worked closely with Urbantech and the Landowners' Team to advance the water and wastewater servicing strategy for the 1200 King Road site. Given the ongoing Integrated Master Plan project and the neighbouring Aldershot GO Station Major Transit Station Area, an enhanced level of hydraulic modelling was necessary to properly assess the implications of development phasing and timing within the broader Regional system.

The Region coordinated with the City to confirm appropriate population projections and build-out horizons for use in the modelling for all neighbouring development areas, including the Aldershot GO MTSA. Unlike the initial submission of this study, where modelling was completed externally, the Region undertook the hydraulic analysis in-house. Through this process, Regional staff worked directly with Urbantech to refine phasing limits, confirm growth assumptions, and validate the overall water distribution strategy.

The Region's modelling results (refer to details in **Appendix C**) confirm that the proposed watermain configuration and phasing approach described in the preceding sections can be supported. In particular, the analysis demonstrates that:

- Phase 1 servicing within Pressure Zone B1A can proceed. Available distribution pressures and fire flows meet Regional requirements for the proposed form of development.
- No improvements to the external municipal water system are required to accommodate Phase 1.
- The transition from Zone B1A to B2 will facilitate full development of the 1200 King Road site.

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It is recognized that the hydraulic modelling completed to date reflects the best information available with respect to population, land use, building configuration, and system infrastructure. As planning and design advances, refinements to site layout, grading, unit counts, architectural design, fire protection measures, and Regional infrastructure assumptions may influence both demand distribution and required fire flow calculations. Accordingly, the results presented herein should be considered appropriate for the present stage of planning and will be updated, as necessary, through subsequent submissions to confirm system performance and to align with future Regional direction and available data. Further, additional information on construction timing, market priorities, or adjacent development may lead to minor refinements to phasing limits. These will be discussed, as required, in consultation with the Region and the City. Nevertheless, the analysis completed to date confirms that the initial phase is supportable within the existing network and that ultimate build-out is directly enabled by infrastructure already identified and approved within the Regional capital framework.

The following sections provide additional detail on the modeling work provided by Halton Region in support of this plan. Please see **Appendix C** for the full Region results.

5.4.1 Model Set-up

The Region of Halton uses a dynamic hydraulic model (InfoWater), calibrated and validated with flow monitoring information to assess development impacts to the Region's water distribution system. The model used was developed through the Integrated Master Plan and includes the most current design criteria (as outlined in **Section 5.1**) and level-of-service requirements.

Model population estimates, post-development site grading and preliminary pipe size / alignments were provided to the Region by Urbantech, taken from a previous version of the site plan. While there have been some minor modification made to blocks in the interim, the overall population totals and distribution is relatively constant. Demand allocations were assigned to model junctions in accordance with the proposed site plan and internal watermain configuration.

As summarized in the presentation material, Phase 1 consists of a residential population of approximately 5,300 persons and an ICI population of 640 persons, for a total demand of approximately 29.5 L/s under MDD conditions. At full build-out (Phase 2), total population increases to approximately 16,700 persons, corresponding to an overall MDD of approximately 81.9 L/s.

The model includes:

- Junction elevations derived from the grading information available at this stage;
- Distribution pipe lengths and diameters consistent with the proposed design drawings;
- Hazen-Williams roughness values consistent with Regional standards; and
- Pressure zone assignment reflecting existing and ultimate servicing strategies.

5.4.2 Modelling Scenarios

A series of staged scenarios were analyzed to understand system performance as development progresses and to review options for phasing boundaries.

Scenario 1: Phase 1 – Initial Development (B1A)
This scenario represents Phase 1 build-out, with watermains connected to the existing system along King Road, without additional trunk upgrades or pressure zone changes. This is consistent with the Phase 1 boundaries as shown in **Drawing WM-1**, but does not model the full 300 mm watermain connection from King Road to Waterdown Road across Street 'A'.

Scenario 2: Phase 1.5 – Interim Expansion (B1A)
This scenario introduces additional development to the west of Phase 1, incorporating the entertainment district and additional residential blocks, while maintaining supply from B1A.

Scenario 3: Phase 1.75 – Interim with Additional 300 mm Connection (B1A)
This sensitivity test incorporates the Scenario 2 development area but adds the proposed 300 mm watermain connection on Street 'A' between Waterdown Road and King Road. The intent is to quantify the benefit of additional looping while still remaining within Zone B1A.

Scenario 4: Phase 2 – Ultimate Build-Out (B2)
The ultimate scenario assumes the new planned Regional infrastructure is in place, the site is transitioned to Pressure Zone B2. All Phase 1 and Phase 2 demands are active.

5.4.3 Analysis and Findings

Scenario 1: Phase 1 – Initial Development (B1A)

Under Phase 1 Maximum Day Demand (MDD) conditions, simulated distribution pressures across the development are predicted to range from approximately 44 to 50 psi. Although at the lower end of the preferred range, these pressures remain within the Region's acceptable level-of-service criteria. It is also recognized that high-rise residential and mixed-use buildings are typically equipped with internal booster pumping systems, which will assist in supporting service pressures within individual buildings.

Available Fire Flow (AFF) at representative network nodes is predicted to range from approximately 224 to 257 L/s, which is sufficient to support the proposed built form within the Phase 1 development area.

There are no identified capacity restrictions or pressure issues associated with Scenario 1

Scenario 2: Phase 1.5 – Interim Expansion (B1A)

With an expansion of Phase 1 to the west, but without a transition to Zone B2, model results indicate that distribution pressures fall below the Region's 40 psi minimum criterion in portions of the plan area, particularly at the higher elevations in the northwest. Under this scenario, Available Fire Flow (AFF) is predicted to decline to approximately 174–207 L/s, which is insufficient to reliably meet the fire protection criteria anticipated for this form of development.

This modelling scenario confirms that there are practical limits to what Zone B1A can support on an interim basis when supply is derived solely from the King Road connection (i.e., in the absence of full system looping; refer to Scenario 3). Should additional lands be brought forward within Phase 1, it may be necessary to defer development of other blocks in order to maintain acceptable fire flow performance. In the northwest portion of the plan, ground elevations present an added constraint, and achieving adequate distribution pressures under Zone B1A will be challenging irrespective of fire flow availability.

Scenario 3: Phase 1.75 – Interim with Additional 300 mm Connection (B1A)

Introducing the proposed 300 mm watermain link between Waterdown Road and King Road via Street 'A' results in a measurable improvement to system performance. Under this configuration, Available Fire Flow (AFF) increases to approximately 241–277 L/s, representing an improvement over the Scenario 1 results. Notwithstanding this benefit, predicted distribution pressures in the northwest portion of the plan continue to remain below the Region's minimum criteria.

This modelling scenario demonstrates that there may be opportunity for some minor expansions to Phase 1 limits, provided that such expansion is confined to areas where ground elevations allow distribution

pressures to be maintained above 40 psi. Expansion of Phase 1 would also be predicated on coordination with the City and Region.

It is important to note that there are permutations of Scenarios 2 and Scenario 3 could support alternate Phase 1 delineations depending on how development is prioritized and staged. For example, certain blocks may be deferred to prioritize delivery of the entertainment district or other campus-related uses, etc. As planning advances and assumptions regarding land use, density, and timing are further refined, the phasing boundaries can be refined in consultation with the City and the Region

Scenario 4: Phase 2 – Ultimate Build-Out (B2)

Under the ultimate servicing configuration, where the development is supplied from Pressure Zone B2, overall hydraulic performance improves substantially. Maximum day demand pressures are predicted to increase to approximately 74–86 psi across the site and available fire flow ranges from 418 - 721 L/s. At these service levels, all blocks are expected to satisfy both minimum pressure objectives and fire protection requirements.

5.4.4 Overall Conclusions – Water Hydraulic Modelling

Water Hydraulic Modelling results clearly identify that there will be a servicing threshold associated with 1200 King Road that requires a phased approach to development. Modelling establishes that Phase 1 (as identified in **Drawing WM-1**) can proceed without upgrades to the Region's municipal system and on Pressure Zone B1A. The Phase 1 boundary may be increased or modified as planning proceeds, however localized fire flow limitations and distribution pressure constraints will need to be confirmed and addressed through subsequent design stages, in consultation with the Region and the City. Development beyond Phase 1 is more sensitive to elevation and location, and therefore requires additional investigation to define the practical limits of interim servicing.

Full build-out of 1200 King Road is dependent on transitioning the area to Pressure Zone B2. This conversion unlocks the pressure and fire flow capacity necessary to support higher densities and broader system reliability. The findings therefore reinforce the need to closely coordinate development phasing with the implementation schedule for Regional infrastructure upgrades required to facilitate the B1A to B2 transition

6. WASTEWATER SERVICING

6.1. Design Criteria

Wastewater sewers will be designed in accordance with Halton Region design criteria (as outlined in the December 2025 Linear Design Manual) as follows:

- Average Dry Weather Flow: 215 L/person/day (residential), 185 L/employee/day (ICI)
- Peaking Factor: Calculated from the modified Harmon Formula
- Inflow and Infiltration: 0.28 L/s/ha

As outlined in **Section 5.1**, population estimates for the 1200 King Site are calculated based on an average 600 ppl/ha population density for net developable areas (per Region of Halton Linear Design Manual - Apartments in intensification areas). This aggregate density is representative of typical high-density redevelopment areas. Population estimates will be refined as planning proceeds. The 600 ppl/ha assumption is consistent with initial population totals at the block level. Please refer to **Appendix D** for Sanitary Design Sheets.

6.2. Existing Conditions

The 1200 King Road development is within the West Burlington Trunk Sewer catchment, which ultimately drains to the Skyway Wastewater Treatment Plant. The West Trunk system is a series of 900 mm, 1200 mm and 1350 mm sewer components that collect wastewater from the area generally west of the QEW in Burlington. There are no wastewater pump stations within this trunk system; wastewater flows by gravity to the Treatment Plant.

Existing wastewater infrastructure local to the site includes the following:

- 450 mm - 525mm wastewater sub-trunk within King Road adjacent to the property.
- 450 mm wastewater sub-trunk located on St. Matthews Avenue. This wastewater sub-trunk crosses the rail line through an existing easement on the west side of the 1200 King Road property.

These sub-trunks connect downstream to the existing 900 mm West Trunk system. Existing infrastructure for the area is shown on **Drawing SAN-1** and **Drawing SAN-2**.

6.3. Proposed Sanitary Servicing

6.3.1 Region of Halton Master Plan Strategy

Similar to the water system, the wastewater network in this area of Burlington has been reviewed through the Region's Integrated Master Plan to 2051. The existing trunk network will be impacted as re-development occurs in the Aldershot MTSA, 1200 King Road and North Aldershot site (north of Highway 403).

The capital program mapping approved by Council in November 2025 does not indicate the need for any trunk sewer upgrades downstream of the 1200 King Road site to support intensification areas. As such, trunk capacity is anticipated to be available in the near-term for 1200 King Road development.

From a longer-term perspective, the Region intends to complete the following capital projects, which will increase the capacity of the broader sanitary system:

- **ID8463:** New 2400 mm Skyway Inlet Sewer (parallel to the QEW). Planned for construction starting in 2027. Financed through the Region's 2023 Allocation Program.

- Skyway Wastewater Treatment Plant Expansion (New Master Plan project). Proposed for in-service in the 2037 – 2041 timeframe. Not anticipated to impact timing for the 1200 King Road property.

6.3.2 1200 King Road

Local wastewater servicing for the east portion of the site, ranging in size from 250 mm to 450 mm in diameter, (east of Falcon Creek) will drain by gravity to the existing 450 mm municipal sewer on King Road. Wastewater servicing for the west portion of the site, ranging in size from 250 mm to 300 mm in diameter, will drain by gravity to the existing 450 mm sewer that runs under the railway line at the west boundary of the property and connects downstream at St. Matthews Avenue. The Sanitary Drainage Plan is shown in **Drawing SAN-1**.

Preliminary sewer design sheets (**Appendix D**) conform with design criteria outlined in **Section 6.1**. Sewer design sheets indicate adequate existing capacity in the adjacent sanitary sewers to accommodate the full 1200 King Road development. The design sheet includes assumptions regarding flow from upstream employment land and the closed Bayview Landfill Site.

Development for 1200 King Road will be phased (as outlined in **Section 10**) due to constraints in the water system. Phase 1 sanitary flow will drain only to the existing 450 mm sewer on King Road.

6.4. Hydraulic Modelling Analysis

Similar to the water system, Region Staff have worked closely with Urbantech and the Landowners' Team to confirm the wastewater servicing strategy for the 1200 King Road site. Given the ongoing Integrated Master Plan project and the neighbouring Aldershot GO Station Major Transit Station Area, an enhanced level of hydraulic modelling was necessary to properly assess the implications of these intensification areas as it relates to the Region's downstream trunk sewer system.

The Region coordinated with the City to confirm appropriate population projections and build-out horizons for use in the modelling for all neighbouring development areas, including the Aldershot GO MTSA. Unlike the initial submission of this study, where modelling was completed externally, the Region undertook the hydraulic analysis in-house. Through this process, Regional staff worked directly with Urbantech to confirm growth assumptions and validate the overall wastewater strategy.

The Region's modelling results (refer to details in **Appendix C**) confirm that the proposed sewer configuration and phasing approach described in the preceding sections can be supported. In particular, the analysis demonstrates that all downstream trunk sewers have sufficient capacity to support 1200 King Road growth, up to the 1800 mm Skyway Inlet Sewer. The Skyway Inlet Sewer is scheduled for upgrade in the 2027 timeframe to provide additional long-term capacity for growth. The Region has noted that there is capacity existing in the Skyway sewer to support planned development in Burlington to 2031.

The following sections provide additional detail on the modeling work provided by Halton Region in support of this plan. Please see **Appendix C** for the full Region results.

6.4.1 Model Set-up

The sanitary analysis was completed using the most recent InfoWorks ICM wastewater hydraulic model prepared in support of the Region's Integrated Master Plan. The model framework, boundary conditions, and design criteria are consistent with those applied by the Region for growth planning and consistent with those outlined in **Section 6.1**.

The development loading for the subject lands was based on the Phase 1 and Phase 2 population allocations provided by the project team. Flows were distributed to the network using sub-catchments aligned with the proposed site plan and internal servicing layout.

The wastewater model includes:

- Maintenance hole nodes with ground elevations derived from the grading and servicing drawings provided by Urbantech;
- Sub-catchment areas with associated population and road allowance inputs; and
- Conduits defined by proposed pipe diameters, lengths, slopes, and invert elevations.

In accordance with the Regional level of service criteria, performance was evaluated based on:

- risk of surcharge (d/D < 1) during the 5-year event; and
- risk of basement flooding, defined by hydraulic grade remaining at least 1.8 m below ground surface during the 10-year event.

The model network routes flows from the development through the downstream system toward the trunk infrastructure tributary to the Skyway Wastewater Treatment Plant.

6.4.2 Modelling Scenarios

The wastewater assessment focused on the ultimate build-out condition, representing full Phase 1 and Phase 2 development with the internal sewer network as outlined in **Drawing SAN-1**. This approach reflects the understanding that if the downstream system can accommodate ultimate flows, interim stages will also operate within capacity.

The model incorporates the updated discharge locations and invert adjustments shown on the latest servicing plans.

6.4.3 Analysis and Findings

Modelling results indicate that the sanitary system has sufficient capacity to convey flows from the proposed development to the downstream trunk network. Model outputs demonstrate that all pipes from the site to the 1800 mm inlet trunk toward the Skyway facility operate with a surcharge ratio (d/D) of less than 1. As such, the system meets the Region's criterion for acceptable performance under wet weather conditions.

Based on the simulated hydraulic grade lines, there is no predicted risk of basement flooding within the contributing area under the Regional design storm.

The results therefore confirm that, from a hydraulic capacity perspective, the downstream system can support the proposed ultimate build-out of the 1200 King Road development, without the need for any future upgrades. The modelling completed for the ultimate condition demonstrates that the sanitary network downstream of the site provides adequate conveyance capacity to accommodate projected flows from both Phase 1 and Phase 2 development.

7. STORM DRAINAGE AND STORMWATER MANAGEMENT

7.1. Design Criteria

Based on City of Burlington criteria, requirements from previous studies for the area and comments received from the regulatory agencies the design criteria for the site as follows:

1. Control post development flows to the existing conditions for all creeks (including Regional Event)
2. Ensure MECP enhanced (Level 1) stormwater quality treatment of runoff is provided
3. Minimum 24-48 hour extended detention

7.2. Existing conditions

Urbantech received the latest hydrology and hydraulic modelling for the three creeks within 1200 King Road in October 2025. The models were from the following studies:

- Indian Creek – SWMHYMO model prepared by AMEC as part of the King Road Aqueduct and Indian Creek Realignment (constructed in 2017).
- Falcon Creek – GAWSER model prepared by Valdor Engineering as part Falcon Creek Hydrology and Hydraulic Study in December 2012.
- Grindstone Creek – Visual Otthymo model prepared by Matrix Solutions Inc. as part of the Flood Hazard Mapping Report for the Grindstone Creek Watershed.

7.2.1 Indian Creek

As noted in Section 7.2 above, the Indian Creek hydrologic model was prepared when designing the King Road Indian Creek realignment and finalized in 2017. The SCS 24-hour storm distribution was used in the modelling and per Section 4.2.4 of the City of Burlington's Stormwater Management Design Guidelines has been used as part of this development to ensure consistency with the strategy used to design the existing downstream channel and aqueduct.

From the SWMHYMO model, a Visual Otthymo (VO6) model was derived as was discussed with Conservation Halton on September 30th 2025, where are the inputs from the SWMHYMO model were replicated in VO. To verify that the change did impact the results, the VO model results were compared to those of key nodes within the modelling. The original SWMHYMO model results for the Regional storm are provided in **Appendix F**.

Table 7-1: Pre-Development SWMHYMO and VO6 Flows

SWMHYMO ID/ VO Node Name	Area (ha)	SWMHYMO Flow (m^3/s)	VO Model Flow (m^3/s)
504	92.82	9.95	9.94
505	97.18	10.38	10.38
506	129.23	13.90	13.90
507	144.94	15.20	15.20
508	157.38	16.34	16.34
509	165.11	17.13	17.13
510	175.43	18.13	18.13

Model comparison results indicate that simulated flows are consistent at all assessment locations, with the exception of a single point (504) where the VO6 results are marginally lower than those generated using SWMHYMO, representing a slightly more conservative estimate. Given the overall agreement between the two modelling approaches and the absence of any material differences, it is considered appropriate to proceed with VO for subsequent analyses and design.

An additional VO6 scenario was set up to represent the site's existing conditions compared to the SWMHYMO scenario. The catchment areas for the existing conditions can be found on STM-1 and also below in **Table 7-2**.

Table 7-2: Indian Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
EX-4	19.00	7.1
EX-13	8.83	29.0
EX-14	30.73	7.1
EX-15	2.11	29.0
EX-16	12.70	7.1

Table 7-3 presents the flows at 3 key locations from the SWMHYMO model and the existing conditions VO6 model: the property line (NHYD 19), the aqueduct (NHYD 20) and the most downstream node in the model (NHYD 22). For all storm distributions, the SWMHYMO model flows were determined to be lower, and therefore will be the used as the target flows for post-development to be conservative. Additionally, since the aqueduct over King Road was designed based on the SWMHYMO model flows, using the SWMHYMO flows for the subject property would ensure consistency with design methodology employed for the existing downstream channel and aqueduct.

Table 7-3: Pre-Development SWMHYMO and VO6 Flows

NHYD	Scenario	Storm Distribution						
		2 (m^3/s)	5 (m^3/s)	10 (m^3/s)	25 (m^3/s)	50 (m^3/s)	100 (m^3/s)	Reg (m^3/s)
19	SWMHYMO	3.08	4.68	5.75	7.06	8.17	9.56	17.13
	Existing	3.17	4.84	5.97	7.34	8.49	9.91	17.73
20	SWMHYMO	2.92	4.65	5.71	7.02	8.11	9.42	17.11
	Existing	3.03	4.80	5.92	7.29	8.41	9.74	17.70
22	SWMHYMO	3.07	4.89	5.92	7.23	8.34	9.68	18.13
	Existing	3.18	5.05	6.14	7.50	8.65	10.01	18.71

7.2.2 Falcon Creek

Flacon creek is located within the centre of the property, and is located in catchment 109 as shown on Figure 4.1 from the original Falcon Creek study by Valdor Engineering, included in **Appendix F**. In the original existing conditions model all of catchment 109 was modelled as undeveloped (including the highway and railway corridor), and through topography it was determined that approximately 10 ha of the 35.7 ha in catchment area 109 is actually being conveyed to the adjacent Grindstone and Indian Creeks. As this is a substantial change in area compared to the original catchment size, and there has been development north of the highway, the ultimate condition GAWSER model has been adopted as the existing conditions, with catchment 109 revised to represent the present day conditions for the highway and railway, and the lands within 1200 King being modelled as undeveloped for the purpose of setting

targets. **Table 7-4** presents the parameters used for the modelling of the development area draining towards Falcon Creek. To see how the catchments are divided refer to **Drawing STM-1**.

Table 7-4: Falcon Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
EX-3	4.93	7.1

The SCS Type II 6-hour storm distribution was used in the modelling and per Section 4.2.4 of the City of Burlington's Stormwater Management Design Guidelines has been used as part of this development to ensure consistency with the other works being done within the watershed.

Targets have been set at node 5 from the GAWSER model. **Table 7-5** below outlines the flows to Falcon Creek.

Table 7-5: Existing Flows to Falcon Creek

Creek Name	Flow Location	Flow (m^3/s)						
		2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	Regional
Falcon Creek	Node 5	2.22	6.60	9.89	13.56	16.17	19.11	23.25

Please refer to **Appendix B** for the GAWSER modelling.

7.2.3 Grindstone Creek

The subject lands are tributary to two subcatchments within the Grindstone Creek watershed, one located to the north-west of the site, and the other to the south. The AES 12-hour storm distribution was used in the modelling and per Section 4.2.4 of the City of Burlington's Stormwater Management Design Guidelines has been used as part of this development to ensure consistency with the other works being done within the watershed. **Table 7-6** presents the parameters of the two subcatchments. To see how the catchments are divided refer to **Drawing STM-1**.

Table 7-6: Grindstone Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
EX-1	4.93	7.1
EX-2	6.35	7.1

Table 7-7 outlines the flows to Grindstone Creek. It is to be noted that although the campus/residential block has an area of 3.06 ha, only 2.46 ha is within the Grindstone Creek catchment; the remaining 0.6 ha drains towards Falcon Creek under existing conditions but will be diverted in post-development. Therefore, an area of 2.46 ha is used to calculate the target flows in the table below.

Table 7-7: Existing Unit Flow Rate to Grindstone Creek

NHYD	Area (ha)	Flow Rate	Return Period (Years)						Reg Storm
			2	5	10	25	50	100	
S_140-2	25.30	Unit Flow Rate (m^3/s per ha)	0.02	0.03	0.03	0.04	0.05	0.05	0.14
	0.80	Target Flow Rate (m^3/s)	0.02	0.02	0.03	0.03	0.04	0.04	0.12
S_130-1	26.80	Unit Flow Rate (m^3/s per ha)	0.02	0.03	0.03	0.04	0.04	0.05	0.14
	2.46	Target Flow Rate (m^3/s)	0.05	0.06	0.08	0.09	0.10	0.12	0.35

7.3. Existing Conditions and Surrounding Drainage Patterns

The existing combined sewer network in the vicinity of the site includes:

1. Six stormwater culverts north of the site boundary that run from the north side of North Service Road to South of HWY 403,
2. A 375-675 mm storm sewer on Gallagher Road,
3. A 375-600 mm diameter storm sewer on Enfield Road,
4. Stormwater culvert that runs from the north to south side of Enfield Road,
5. A 2250 mm storm sewer on Willowbrook Road, and
6. A 250-675 mm diameter storm sewer on Cedarwood Place.

The locations of these sewers and culverts are shown on **Drawing STM-1**.

7.4. Storm Sewer Design

Storm sewers within the subdivision are sized to convey the 5-year storm in accordance with the City of Burlington standards. Local storm sewer pipes generally range in size from 300 mm diameter to 1200 x 1800 box culverts. Majority of stormwater within the development between Falcon Creek and King Road is conveyed to the stormwater ponds, which are situated in the southeast corner of the site on both sides of Indian Creek. For the flows being conveyed ultimately to Indian Creek, 100-year capture on Street D is proposed upstream of Pond 2.

The residential blocks located directly to the east of Falcon Creek as well as the road will outlet directly to Falcon Creek with appropriate controls.

The residential block and mixed-use employment block between Grindstone Creek and Falcon Creek will outlet to Grindstone Creek with appropriate controls.

For storm sewer sizing refer to 5-year design sheet in **Appendix B**.

7.5. Erosion Control

As identified in **Section 7.1** above, the stormwater designs are required to meet the erosion threshold target by providing a minimum 24 drawdown time of the 25 mm rainfall event in accordance with MECP criteria for the Indian Creek ponds. For Falcon Creek and Grindstone Creek where the catchments are very small, a best efforts approach will be implemented using the minimum allowable orifice size of 75 mm.

The erosion study conducted by GEO Morphix for all creeks concluded that for reaches IC-T3 and IC-T4 of Indian Creek, the erosion threshold is 0.103 m^3/s and 0.133 m^3/s respectively. For Falcon Creek reach FC-6b, 0.179 m^3/s was calculated to be the erosion threshold and for Grindstone Creek reach GC-T0, 0.023 m^3/s. Assuming a similar reach slope, roughness and boundary materials between GC-T0 and GC-T0A, a unitary erosion threshold was used to determine an erosion threshold of 0.065 m^3/s for GC-T. In the southern segment of Grindstone Creek, reach GC-T3 drains into an engineered ditch flowing westward. Using LiDAR data, the bankfull discharge of the ditch was determined to be 2.417 m^3/s, at which the shear stress in the bed and banks is 6.73 Pa and 5.05 Pa respectively. Based on these calculated parameters, the engineering channel is expected to be stable.

As part of the study, an erosion exceedance analysis was also completed for a select few reaches that were established to be to most erosion sensitive. The analysis concludes that the proposed stormwater management plan effectively mitigates erosions for reaches FC6a and GC-T0A. Reach IC-T4, is expected to experience an increase in erosion potential as the reach receives outflows from the two proposed SWM ponds. However, the exceedance is expected to be adequately addressed with optimization of the stormwater management plan and refinement of the calculations once field assessments are complete in Spring 2026.

Refer to **Appendix E** for the full report.

7.6. Quantity Control

7.6.1 Indian Creek

The SWMHYMO model has been updated to incorporate the proposed development, including the two stormwater management ponds and the 0.32 ha of proposed ROW that connects to King Road. Runoff from this ROW area is directed to the existing King Road due to grading constraints that preclude conveyance to the proposed ponds. No revisions were made to catchment delineations upstream of the highway or south of the subject lands. Drainage areas are illustrated on **Drawing STM-2**. **Table 7-8** presents the proposed catchment parameters.

Table 7-8: Indian Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
8	4.26	100.0
9	0.43	78.6
10	2.03	7.1
11	1.99	100.0
12	1.83	78.6
13	1.2	78.6
14	0.28	78.6
15	1.92	100.0

Catchment ID	Area (ha)	Impervious (%)
16	2.93	100.0
17	0.57	100.0
18	0.19	78.6
19	0.87	100.0
20	1.04	100.0
21	0.42	78.6
22	1.09	100.0
23	0.54	78.6
24	0.14	78.6
25	1.21	100.0
26	0.18	78.6
27	1.52	100.0
28	0.21	0.75
29	1.16	50.0
30	0.72	50.0

The ponds have been sized to control post-development discharges such that peak flows to the receiving creek do not exceed existing conditions for the 2- to 100-year events and the Regional storm at the property limit, at the King Road aqueduct, and at the location where King Road drainage enters the creek. **Tables 7-9** and **7-10** summarize the corresponding flow targets and storage requirements derived from the VO6 model. Detailed outlet structure design will be completed at the detailed design stage. Supporting VO6 model outputs are provided in **Appendix B**.

Table 7-9: Indian Creek Required Storage Volume Results – Pond 1

Pond Level	Elevation	Post Development Flows	Required Volume	Provided Volume
		m^3/s	(m^3)	(m^3)
ED	103.81	0.05	3,554	3,554
2	85.93	0.48	5,392	5,392
5	104.16	0.87	6,927	6,927
10	104.30	1.21	7,845	7,845
25	104.50	1.89	8,823	8,823
50	104.64	2.36	9,502	9,502
100	104.76	2.88	10,093	11,041
Regional	104.69	2.57	9,754	9,754
Emergency Spillway	105.25	-	-	13,265
Maintenance Rd	105.50	-	-	15,238

Table 7-10: Indian Creek Required Storage Volume Results – Pond 2

Pond Level	Elevation	Post Development Flows	Required Volume	Provided Volume
		m^3/s	(m^3)	(m^3)
ED	103.53	0.02	1,356	1,356
2	103.79	0.12	2,137	2,137
5	103.97	0.26	2,706	2,706
10	104.09	0.37	3,053	3,053
25	104.22	0.50	3,496	3,496
50	104.29	0.57	3,827	3,827
100	104.38	0.68	4,123	4,123
Regional	104.43	0.74	4,267	4,493
Emergency Spillway	104.80	-	-	5,706
Maintenance Rd	105.00	-	-	6,581

Flows at the three aforementioned downstream key locations are summarized in **Table 7-11** and demonstrate that there is no increase in peak discharge for any of the design storm events.

Table 7-11: Pre-Development SWMHYMO and VO6 Flow Comparison

NHYD	Scenario	Storm Distribution						
		2 (m^3/s)	5 (m^3/s)	10 (m^3/s)	25 (m^3/s)	50 (m^3/s)	100 (m^3/s)	Reg (m^3/s)
19	SWMHYMO	3.08	4.68	5.75	7.06	8.17	9.56	17.13
	Proposed	2.87	4.29	5.29	6.41	7.37	8.47	16.76
20	SWMHYMO	2.92	4.65	5.71	7.02	8.11	9.42	17.11
	Proposed	2.67	4.25	5.22	6.37	7.33	8.37	16.70
22	SWMHYMO	3.07	4.89	5.92	7.23	8.34	9.68	18.13
	Proposed	2.82	4.51	5.51	6.60	7.57	8.63	17.13

External Area Drainage

Consideration has been given to the potential future development of lands external to 1200 King Road that are also located south of Highway 403.

Approximately 2.8 ha of Hydro One–owned hydro corridor lands are located northeast of the subject property; development of these lands is not anticipated. Approximately 1.4 ha is occupied by two Indian Creek tributaries and the associated floodplain that enter the 1200 King Road property and is therefore not developable. After applying the 14 m MTO setback, the remaining potentially developable area is reduced to approximately 1.2 ha and may be further constrained by anticipated Highway 403 widening (six to eight lanes) and the associated replacement of the King Road bridge. In addition, the Hydro One Burlington transformer station immediately east of King Road is expected to remain in place, and the hydro corridor is therefore not expected to be removed or redeveloped.
In addition, an existing commercial development is located southeast of the site. This property is situated at the lowest point of the catchment west of King Road and currently drains south to the previously

realigned reach of Indian Creek. The proposed top-of-permanent-pool elevation for Pond 2 is approximately equal to the elevation of the southwest portion of this block; therefore, significant fill would be required for this property to redevelop and be graded to drain to Pond 2. While this property would have been a suitable location for a stormwater management pond, it has already been developed independently and is therefore not available for that purpose. Any future redevelopment of this property would be expected to provide stormwater controls consistent with those required for other catchments draining to Indian Creek.

7.6.2 Falcon Creek

The original ultimate condition GAWSER model for Falcon Creek assumed full development of Catchment 109, with all runoff conveyed to a proposed offline stormwater management facility located upstream of the railway. Subsequent refinement of the development limits, constrained by regulated features, has reduced the net developable area to approximately 3.24 ha, with the balance of the catchment remaining in its existing condition and continuing to drain directly to Falcon Creek via established drainage patterns. **Table 7-12** presents the proposed catchment parameters based on the established drainage patterns. Refer to **Drawing STM-2** for drainage areas.

Table 7-12: Falcon Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
5	1.06	78.6
6	1.53	100.0
7	0.62	100.0

Given the limited extent of new development and the fact that the majority of the catchment will remain undeveloped and naturally conveyed to the watercourse, a stormwater management pond is not warranted. City of Burlington guidelines indicate that end of pipe stormwater management facilities are required for drainage areas larger than 5 ha, and the Ministry of the Environment, Conservation and Parks guidelines indicate a preferred area of 10 ha. Storage will be provided through the use of a combination of surface, underground storage and LIDs, which will be specified during subsequent planning exercises. As there are known downstream flooding concerns, it is recommended that the proposed ROW have storage incorporated in it, which will be coordinated with the City as the design progresses. **Tables 7-13** summarizes the flow and storage values required based on the GAWSER model.

Table 7-13: Falcon Creek Flow Results

Storm Event	Target Flow (m^3/s)	Proposed Flow (m^3/s)	Required Storage* (m^3)
	Node 5		
2	2.22	2.22	849
5	6.60	6.60	1,094
10	9.89	9.89	1,240
25	13.56	13.56	1,427
50	16.17	16.16	1,569
100	19.11	19.10	1,681
Regional	23.25	23.22	-

*Regional storm has been modelled as uncontrolled

As demonstrated in the table above, Regional storm storage is not required, as the uncontrolled post-development peak flows remain below the applicable target discharge rate. Accordingly, no adverse downstream impact is anticipated under Regional storm conditions. Refer to the GAWSER model output provided in **Appendix B**.

7.6.3 Grindstone Creek

Topography for the subject lands indicated that approximately 6 ha is conveyed to Grindstone Creek compared to what was originally considered in the floodplain study. As this is a 24% increase in the area from the subject lands, primarily to the south tributary, the hydrology model has been revised to include this increase in the proposed condition. Time to peak for these revised catchments was calculated using the Airport Method, calculations are provided in **Appendix B**.

Table 7-14 presents the proposed catchment parameters. To see how the catchments are divided refer to **Drawing STM-2**.

Table 7-14: Grindstone Creek Post-Development Drainage Areas

Catchment ID	Area (ha)	Impervious (%)
1	0.8	100.0
2	0.8	78.6
3	0.72	78.6
4	3.06	100.0

Table 7-15 and **7-16** summarizes the flow and storage values required based on the VO6 model. Storage will be provided through the use of a combination of underground storage, surface and LIDs which will be specified during subsequent planning exercises. Refer to **Drawings STM-2** for drainage areas.

Table 7-15: Grindstone Creek Flow Results – Controlled Drainage to North

Storm Event	Target Flow (m^3/s)	Post Development Flow (m^3/s)	Required Storage Volume (m^3)
2	0.02	0.02	92
5	0.02	0.02	122
10	0.03	0.03	141
25	0.03	0.03	165
50	0.04	0.04	180
100	0.04	0.04	195
Regional	0.12	0.12	-

*The flows presented in the table above are the flows from the north tributaries as well as the flows from the development

Table 7-16: Grindstone Creek Flow Results – Controlled Drainage to South

Storm Event	Target Flow (m^3/s)	Post Development Flow (m^3/s)	Required Storage Volume (m^3)
2	0.05	0.05	671
5	0.06	0.06	882
10	0.08	0.08	1013
25	0.09	0.09	1173
50	0.10	0.10	1286
100	0.12	0.12	1390
Regional	0.35	0.45	-

*The flows presented in the table above are the flows from the north tributaries as well as the flows from the development

Regional stormwater quantity control is not proposed for the Grindstone Creek tributaries. The contributing development areas are approximately 1.6 ha and 3.78 ha, which are insufficient to practically support a standalone regional facility. Provision of such infrastructure would require a disproportionately large footprint relative to the drainage area and, in accordance with Conservation Halton requirements, would need to be located on publicly owned lands, making implementation neither practical nor efficient from a land use perspective. When the tributaries join the main creek, the incremental increase in peak discharge is minor (approximately 0.7% for the north tributary and 0.29% for the south tributary). **Table 7-17** presents the existing and post-development downstream creek flows. Further discussion of the downstream hydraulic conditions and confirmation that floodplain limits remain unchanged is provided in Section 8.3.

Table 7-17: Grindstone Downstream Creek Flow Results

NHYD	Source	Area (ha)	Return Period (Years)							Reg Storm
			25mm	2	5	10	25	50	100	
558	Existing	73.20	2.28	0.62	0.89	1.09	1.37	1.58	1.79	5.78
	Post Dev	73.38	2.38	0.65	0.92	1.11	1.39	1.60	1.81	5.82
	Difference		4.1%	3.7%	3.1%	1.8%	1.7%	1.4%	1.3%	0.6%
618	Existing	126.60	2.72	0.82	1.27	1.59	2.02	2.34	2.67	8.30
	Post Dev	126.78	2.82	0.84	1.29	1.61	2.04	2.37	2.70	8.33
	Difference		3.5%	2.0%	1.6%	1.3%	1.1%	1.0%	0.9%	0.3%
727	Existing	128.20	2.42	0.84	1.29	1.62	2.05	2.38	2.72	8.41
	Post Dev	128.38	2.51	0.86	1.31	1.64	2.08	2.41	2.74	8.44
	Difference		3.6%	2.0%	1.5%	1.3%	1.1%	1.0%	0.9%	0.3%
732	Existing	315.40	3.92	2.52	3.94	4.95	6.27	7.27	8.27	27.52
	Post Dev	315.58	4.00	2.54	3.96	4.97	6.29	7.30	8.29	27.55
	Difference		2.1%	0.7%	0.5%	0.4%	0.4%	0.4%	0.3%	0.1%
729	Existing	346.45	5.68	2.97	4.56	5.68	7.15	8.26	9.37	30.82
	Post Dev	347.08	5.84	3.02	4.62	5.75	7.22	8.34	9.45	30.91
	Difference		2.8%	1.7%	1.3%	1.1%	1.0%	0.9%	0.8%	0.3%

Refer to VO6 model output provided in **Appendix B**.

7.7. Quality Control

7.7.1 Indian Creek

Enhanced (Level 1) water quality protection through the removal of 80% of total suspended solids (TSS) will be provided.

Total drainage area to pond 1 is 19.66 ha. The required permanent pool volume was determined to be 198 m^3/ha for Pond 1 based on MOE Stormwater Management Planning and Design Manual Table 3.2 with an imperviousness of 78%. This equates to a required permanent pool volume of 3,486 m^3. As 4,543 m^3 is provided there is adequate storage to provide enhanced TSS removal.

Total drainage area to pond 2 is 6.75 ha. The required permanent pool volume was determined to be 260 m^3/ha for Pond 2 based on MOE Stormwater Management Planning and Design Manual Table 3.2 with an imperviousness of 91%. This equates to a required permanent pool volume of 1,485 m^3. As 2,082 m^3 is provided there is adequate storage to provide enhanced TSS removal.

Refer to **Appendix B** for the detailed calculations.

7.7.2 Falcon Creek

Enhanced (Level 1) water quality protection through the removal of >80% of total suspended solids (TSS) will be provided through the use of CB shields and OGS devices in street and jellyfish in building. The proposed OGS devices will be ETV accredited for 50% TSS removal and CB shield device provides 50% TSS removal. The total TSS removal for these BMPs in series can be calculated as 75% TSS removal for the Road areas. Jellyfish filter provides 87.6% TSS removal for the development areas. Opportunities to treat the road water with LIDs will be explored further in consultation with City of Burlington staff.

Based on the information from preliminary hydrologic modelling of Block 12 and 26 of **Drawing STM-2**, a manufacturer has provided recommendations regarding jellyfish sizing. The recommendations are a Jellyfish JF8-8-2 and a JF6-3-1 for Block 12 and 26 respectively. For filter sizing reports and performance specifications refer to **Appendix B**.

Sizing specifications are to be verified by the manufacturer during detailed design.

7.7.3 Grindstone Creek

Enhanced (Level 1) water quality protection through the removal of >80% of total suspended solids (TSS) will be provided through the use of CB shields and OGS devices in street and jellyfish in building. The proposed OGS devices will be ETV accredited for 50% TSS removal and CB shield device provides 50% TSS removal. The total TSS removal for these BMPs in series can be calculated as 75% TSS removal for the Road areas. Jellyfish filter provides 87.6% TSS removal for the development areas. Opportunities to treat the road water with LIDs will be explored further in consultation with City of Burlington staff.

Based on the information from preliminary hydrologic modelling of Block 8 and 14 of **Drawing STM-2**, a manufacturer has provided recommendations regarding jellyfish sizing. The recommendations are a Jellyfish JF10-14-4 and a Jellyfish JF6-4-1 for Block 8 and 14 respectively. For filter sizing reports and performance specifications refer to **Appendix B**.

Sizing specifications are to be confirmed and refined by the manufacturer during detailed design.

7.8. Water Balance

To achieve pre-development water balance infiltration rates, low impact development (LID) measures are proposed throughout the site. **Table 7-18** outlines potential measures to achieve water balance objectives on a best-efforts basis.

Table 7-18: Potential LID Measures

LID Measure	Notes
Rainwater Harvesting	A block level measure that can be used to irrigate landscape areas or mechanical reuse.
Bioretention	Improves stormwater quality, reduces runoff volumes and peak flows, and promotes infiltration through engineered soil and vegetation.
Grassed Swales	Conveyance LID to be located on continuous strips of green space, promotes TSS removal and infiltration.
Additional Topsoil	Minimum 200 mm of topsoil in landscaped areas enhances potential for groundwater recharge.
Green Roofs	The benefits of green roofs could be attenuation of flows, filtration and increased water available for evapotranspiration.
Tree Pits	Provides stormwater quality treatment, temporary storage, and infiltration while supporting healthy urban tree growth. Due to its compact and modular design, it is well suited for use in constrained spaces such as road right-of-way boulevards.
Permeable Pavement	Permeable pavement replaces conventional impervious surfaces by allowing stormwater to pass through the pavement into an underlying stone storage layer, where it can infiltrate into native soils or be held temporarily before outletting to storm infrastructure such as storm sewers.

LID measures will be designed in accordance with the TRCA/CVC *Low Impact Development Stormwater Management Planning and Design Guide* and the S.T.E.P. guidelines. During subsequent planning exercises, supporting calculations will be completed and coordination will be undertaken with the City of Burlington regarding LID placement within the proposed road right-of-way and park blocks.

The City of Burlington water balance criteria requires retention of the first 5 mm of runoff on site using acceptable LID practices. Based on a developable site area of 35.15 ha, this corresponds to a required retention volume of approximately 1,758 m³.

Terra-Dynamics Consulting Inc. completed preliminary water balance calculations in February 2025 indicating an annual infiltration rate of 100 mm/year for the areas composed of clayey silt and 125 mm/year for the areas composed of silt under existing conditions. Without post-development mitigation measures, the conversion of agricultural fields and forest into an 82% impervious area, results in an area change of 28.82 ha and a loss of infiltration of 28,823 to 36,029 m^3/year. Given the site conditions Terra-Dynamics Consulting Inc. concludes that reaching a 0% site water balance deficit may not be feasible. However, on-site infiltration of 5 mm daily storm events from 6.98 to 8.73 ha of clean roof area runoff

would significant help address the deficit. Furthermore, it is recommended a reduction in the deficit requirements be discussed with the City once site plans are finalized.

The feasibility of infiltration-based LIDs may be constrained where drawdown time criteria cannot be met due to low percolation rates, where a minimum 1.0 m vertical separation to the seasonal high groundwater table cannot be maintained from the base of the facility, or where adequate separation from building foundations cannot be achieved. In such cases, water balance performance would rely on passive measures, with filtration-based LIDs implemented on a best-efforts basis. Preliminary results from in-situ hydraulic conductivity tests has results in a mean value of 9×10^{-9} m/s, which is very low and indicates that soils will not be conducive to infiltration. Water balance calculations will be refined as the design advances and LID feasibility is further evaluated.

8. FLOODPLAIN

The watercourses within the subject property are Indian Creek, Falcon Creek and Grindstone Creek. Urbantech received the latest HEC-RAS models for Indian Creek, Falcon Creek, and Grindstone Creek from Conservation Halton in October 2025, which was used as the basis for determining the existing conditions within the subject property.

A hydraulic analysis for all three creeks was completed as part of this study in order to assess the hydraulic impacts of the proposed development, proposed crossings and proposed design of the Indian Creek Channel. All crossings have been designed to have no overtopping in the Regional event, in order to ensure safe access. The following detail the assumptions, methodology and design criteria used for this assessment to compare hydraulic impacts between pre and post-development conditions.

8.1. Indian Creek

The model was georeferenced based on the cross section shp file received and cross sections were updated based on detailed topography completed for the subject property.

The following parameters were kept unchanged from the HEC-RAS model received from CH in order to analyse the hydraulic conditions along Indian Creek:

- Manning's roughness coefficient, n, was used as follows:
  - 0.035 within the main channel;
  - 0.045 for the overbank;
- Contraction and expansion coefficients were set to 0.1 and 0.3, respectively, for smooth transitions between cross-sections. At abrupt transitions, such as upstream and downstream of culvert, the contraction and expansion coefficients were increased to 0.3 and 0.5, respectively.
- Downstream boundary condition was unchanged from the CH model of 94.8 m for the Regional Storms, respectively.

As part of the development, Indian Creek is proposed to be realigned and channelized under future conditions from south of the Highway 403 to the south edge of the property where Indian Creek becomes an engineered channel. The proposed realignment and channelization would improve the watercourses' hydraulic performance, as well as establish the watercourse within a defined creek block. As outlined in section 7.2.1 above, the SWMHYMO regional flows to Indian Creek are less than VO6 existing conditions so they have continued to be used in the HEC-RAS model.

The proposed channel realignment has the following characteristics:

- *Parallel to Highway 403* - Bottom width of 17 m, bank height of 1.1 m with 3:1 side slopes
- *South of Creek confluence* – Bottom width of 25 m, bank height of 1.1 m with 3:1 side slopes
- Two creek crossings at the proposed roadways with four 1.22 x 3.66 m CON/SPAN arch culverts at each location.

The existing and proposed conditions floodline for Indian Creek is presented on **Drawings FP-1** and **FP-2** respectively. **Table 8-1** below shows that the realigned channel is able to be contain the regional floodplain with no increases to water elevation downstream of the development, i.e. all flows are contained within the aqueduct crossing King Road**.** Modelling results for all reaches of Indian Creek are provided in **Appendix B**.

Table 8-1: Indian Creek – Existing to Post-Development Water Levels

River Station	Regional W.S. Elevation (m)		
	Existing	Proposed	Δ Water Level (+/-)
Reach: Indian-3			
753	103.67	103.66	-0.01
702	103.18	103.17	-0.01
675	103.12	103.3	0.18
648	103.11	103.01	-0.1
618	103.1	103.05	-0.05
600	103.07	103.02	-0.05
550	103.04	102.99	-0.05
520	103.03	102.98	-0.05
500	103.02	102.97	-0.05
460	103.01	102.96	-0.05
451	103.01	102.96	-0.05
450	Existing Culvert		N/A
449	102.99	102.93	-0.06
440	102.98	102.92	-0.06
400	102.92	102.85	-0.07
375	102.84	102.77	-0.07
373	102.38	102.33	-0.05

8.1.1 Flood Storage and Riparian Storage

As part of the development, the Indian Creek channel will be realigned to replace the existing channel, which is characterized by an expansive floodplain and spill. Under existing conditions there is significant out-of-bank spill from the channel that, at certain locations, spreads laterally in excess of 600 m and extends toward King Road. This behaviour is driven by the floodplain and flat landscape, such that a substantial portion of regulatory events conveys as broad overland sheet flow rather than remaining within a well-defined channel corridor. Consequently, the “available storage” under existing conditions is not contained within a single stable hydraulic system and cannot be practically or defensibly replicated through a constructed channel realignment. In addition, conventional HEC-RAS 1D modelling has inherent limitations in reliably representing spill that exits the defined system, and bypasses the modeled reach. For the purpose of this comparison and in absence of a 2D model, no spilled flow has been considered, with all flows assumed to being conveyed downstream.

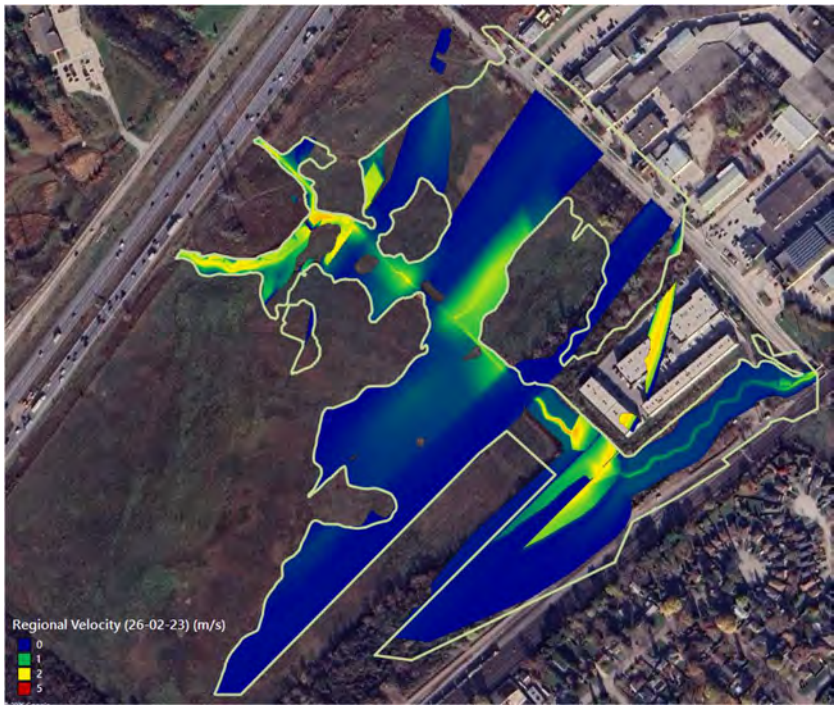
The riparian storage analysis in HEC-RAS was conducted by running the existing and proposed steady-state geometries with all culverts, ineffective flow areas and structures removed as per standard practice, and comparing the volume from the most upstream and downstream cross-sections within the development (XS 1061 and 753). The volume of water contained within the channel for the regional event (i.e., riparian storage) was extracted from the model and it was determined that the volume in the channel increases with the proposed changes. This result can be explained by the poorly defined existing channel that was previously discussed, and through the realignment the proposed channel will be more defined and have a larger amount of available storage.

Table 8-2 summarizes the existing and proposed riparian storage results from the HEC-RAS model.

Table 8-2: Indian Creek – Existing to Post-Development Water Levels

Channel Volume			Total Volume		
Existing	Proposed	% Difference	Existing	Proposed	% Difference
Volume (x1000 m^3)			Volume (x1000 m^3)		
0.73	8.19	91%	25.56	13.76	-46%

When comparing the total volume of water for the regional event, there is a decrease from the existing conditions to the proposed conditions. The decrease in total volume is largely due to the improved hydraulics in the proposed design. The decrease is unlikely to have any negative effect as 1D HEC-RAS flood mapping has shown that majority of the large existing floodplain has little to no velocity as shown in the below figure, which indicates that the storage is ineffective due to the lack of movement.



Additionally, the HEC-RAS geometry and VO6 regional flow hydrographs were incorporated into a PCSWMM model to provide an independent check of downstream hydrograph response and confirm that the revised channel geometry does not adversely affect flow timing or peak discharge. The results indicate that the peak flow at the downstream outfall is 19.26 m^3/s under existing conditions and 18.25 m^3/s under proposed conditions. The proposed condition therefore does not increase downstream peak flow and is not expected to increase downstream flood risk for the regional storm event, confirming that the proposed channel can adequately convey the regional storm flows. For more details regarding the model, refer to **Appendix B**.

Accordingly, the realignment has been assessed on the basis of demonstrated flood impacts—specifically flood levels and inundation extents—at downstream control points and key infrastructure. This includes review of conditions within the existing realigned channel and at the aqueduct crossing over King Road, to confirm these features continue to function as intended and that the proposed works do not introduce adverse downstream flooding effects associated with the 1200 King Road development.

8.2. Falcon Creek

The following parameters were kept unchanged from the HEC-RAS model received from CH in order to analyse the hydraulic conditions along Falcon Creek:

- Manning's roughness coefficient, n, was used as follows:
  - 0.035 within the main channel;
  - 0.08 for the overbank;
- Contraction and expansion coefficients were set to 0.1 and 0.3, respectively, for smooth transitions between cross-sections. At abrupt transitions, such as upstream and downstream of culvert, the contraction and expansion coefficients were increased to 0.3 and 0.5, respectively.
- Downstream boundary condition was unchanged from the CH model of 75.54 m for the Regional Storms, respectively.

Under the proposed conditions, Falcon Creek is to have a creek crossing associated with Street A. The crossing has been designed as twin 1.5 m x 3 m with ensuring that downstream water surface elevations were not exceeded. Additionally, the cross sections located immediately upstream and downstream of the crossing was revised to align with the of the proposed crossing.

The proposed conditions floodline for Falcon Creek is presented on **Drawing FP-2** and in **Table 8-3** which shows that downstream water levels are not impacted as a result of the crossing**.** Modelling results are provided in **Appendix B**.

Table 8-3: Falcon Creek - Existing to Post-Development Water Levels

River Station	Regional W.S. Elevation (m)			
	Existing	River Station	Proposed	Δ Water Level (+/-)
2098.346	104.49	2098.346	104.48	-0.01
2050	Existing Culvert	2050	Existing Culvert	N/A
2016.332	102.51	2016.332	102.48	-0.03
1968.483	101.93	1968.483	101.93	0
1920.542	101.31	1920.542	101.3	-0.01
1900	Existing Culvert	1900	Existing Culvert	N/A
1881.715	100.08	1881.715	100.06	-0.02

8.3. Grindstone Creek

As outlined in Section 7.6.3 it was determined under existing conditions a regional flow of 3.64 m^3/s is being directed from the subject property to a tributary of Grindstone Creek located to the north of site, and 4.40 m^3/s to a tributary located to the south. Under proposed conditions there is a flow of approximately 3.67 m^3/s being conveyed to the north tributary and 4.53 m^3/s being directed to the south. This results in an increase in flow of approximately 30 L/s and 130 L/s to the north and south tributaries, respectively.

To confirm that the minor increase in peak flow does not result in additional flood risk, the revised flows were incorporated into the Grindstone Creek hydraulic model. Comparison of existing and proposed conditions indicates no change in water surface elevations within the north tributary and a maximum increase of approximately 2 cm within the south tributary. In the south tributary, water levels are influenced by a downstream culvert near the GO station that creates a localized backwater condition; consequently, the incremental flow increase results in only a negligible change in depth and does not

alter the mapped floodplain limits. Furthermore, as there are no residential dwellings adjacent to the affected floodplain areas, the minor increase does not pose a risk.

The proposed conditions floodline for Grindstone Creek is presented on **Drawing FP-2** which shows that downstream floodplains are not significantly impacted from an aerial view**.** Hydraulic modelling results are provided in **Appendix B**.

Table 8-4: Grindstone Creek - Existing to Post-Development Water Levels

River Station	Regional W.S. Elevation (m)		
	River Station	Proposed	Δ Water Level (+/-)
River: S_141, Reach: S_141			
271	112.69	112.69	0.00
183	112.14	112.14	0.00
146	112.14	112.14	0.00
138	112.00	112.00	0.00
81.67761	Existing Culvert		N/A
31	107.15	107.15	0.00
River: S_140, Reach: S_140_B			
386	107.11	107.12	0.01
366	107.09	107.09	0.00
355	106.99	107.00	0.01
345.3622	Existing Culvert		N/A
328	106.84	106.84	0.00
320	106.81	106.81	0.00
River: S_140, Reach: S_140_A			
287	106.81	106.81	0.00
276	106.79	106.79	0.00
254	106.72	106.73	0.01
239	106.57	106.57	0.00
226	106.42	106.42	0.00
River: S_130, Reach: S_130			
733	104.63	104.65	0.02
581	104.63	104.65	0.02
383	104.63	104.64	0.01
188	104.63	104.64	0.01
River: S_100, Reach: S_100E			
1406	111.07	111.07	0.00
1316	111.06	111.06	0.00
1297	111.06	111.06	0.00
1212.377	Existing Culvert		N/A
1158	105.95	105.95	0.00

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9. EROSION AND SEDIMENT CONTROL

Erosion and sediment controls for the subject lands have been designed in conformance with the City of Burlington, and CH guidelines. Erosion and sediment controls will be implemented during all site construction works including but not limited to topsoil stripping, bulk earthworks, foundation excavation, site servicing and stockpiling of materials and will conform to ESC guidelines (2006). The following erosion and sediment control measures are proposed to be implemented during construction:

1. Installing heavy duty silt control fencing along the perimeter of the site at strategic locations. For areas adjacent to the creek double silt fence with staked straw bales will be installed between the silt fence for erosion control.
2. Installing a temporary mud mat at the construction site entrance.
3. Wrapping the tops of all inlet structures with filter fabric and installing silt sacks in all catchbasins.
4. Tree preservation fencing in accordance with the tree preservation plan.
5. Pumping of stormwater from the parking garage excavation will be discharged via a sediment filter bag.
6. Gravel mud mat, per Conservation Ontario detail at the construction vehicle access point to minimize off-site tracking of sediments. Entrances from a municipal road will be paved with hot mix asphalt between the city road and the property line.
7. Inspecting all sediment and erosion control controls to maintain them in good repair until such time as the Engineer or the City approves their removal.

Site-specific measures will be prepared during future site alteration as well as site plan applications.

10. DEVELOPMENT PHASING

The development of the 1200 King Road lands is closely tied to the evolution of the Halton Region's Integrated Master Plan pressure zone re-alignment strategy and the timing of planned capital upgrades.

Under existing conditions, the site is located within Pressure Zone B1A. However, due to local ground elevations, portions of the site, particularly toward the central and western areas, cannot reliably achieve the Region's minimum distribution pressure criteria without the planned transition to Zone B2. As a result, the phasing strategy has been structured to allow development to proceed in a logical manner that is consistent with both the hydraulic capability of the current system and the Region's approved long-term servicing plan.

Phase 1 is focused on the eastern portion of the property, generally corresponding to areas below approximately elevation 107 m. Hydraulic modelling completed by the Region confirms that these lands can be adequately serviced from the existing B1A system, with both domestic pressures and fire flow requirements meeting applicable standards. Importantly, the modelling demonstrates that Phase 1 can proceed immediately without the need for off-site municipal upgrades. To support both interim and ultimate conditions, the primary spine watermain along Street 'A' will be constructed with the Phase 1 road works, establishing a looped connection between King Road and Waterdown Road and forming the backbone of the future distribution system.

Development of the central and western portions of the site is identified as Phase 2 and is intended to align with the Region's capital program to eliminate Zone B1A and convert the area to Zone B2. These works, currently scheduled within the 2028–2031 horizon, include upgrades to the Kingsway Booster Pumping Station and strategic watermain extensions, including the funded 600 mm main on North Service Road. Once implemented, these projects will resolve the existing pressure limitations and allow the remainder of the lands to develop without the need for localized boosting or interim measures.

The proposed approach represents a coordinated and infrastructure-led phasing plan developed collaboratively with Regional staff through the master planning and in-house modelling process. As more information becomes available regarding construction timing, market priorities, or adjacent development, minor refinements to the limits between phases may be considered in consultation with the Region and the City. Nevertheless, the analysis completed to date confirms that the initial phase is fully supportable within the existing network and that ultimate build-out is directly enabled by infrastructure already identified and approved within the Regional capital framework.

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11. CONCLUSIONS

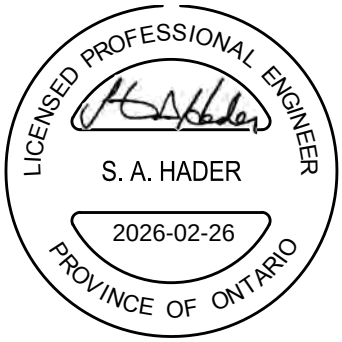
This report has demonstrated that:

- The proposed site will be graded to match to existing elevations at all property lines.
- Wastewater servicing to the site will be provided by a connection to the existing 450 mm sewer on King Road for the portion of the site east of Falcon Creek. Wastewater servicing for the west portion of the site will drain by gravity to the existing 450 mm sewer that runs under the railway line at the west boundary of the property and connects downstream at St. Matthews Avenue.
- Water servicing for the site will be provided by connecting to the existing 350 mm watermain on King Road and the existing 600 mm watermain on Waterdown Road. Development will commence on Zone B1A and will then transition to the higher Zone B2 as Master Plan infrastructure components are constructed.
- 2 to 100-year post to pre-development quantity control will be provided for all SWM Ponds and storage facilities. With Regional control also being provided within the proposed ponds.
- Water quality will be achieved by using two SWM Ponds for the Indian Creek and combination of OGS and CB shield devices, and Jellyfish filter for the proposed development areas for the Falcon Creek and Grindstone Creek. Opportunities to incorporate LIDs in the ROW for quality control will be explored as the design advances in consultation with the City of Burlington.
- Water balance objectives will be met by retaining the first 5 mm of rainfall events onsite which will be determined as part of detailed design.
- Erosion and sediment control measures will be determined at the design stage.
- The site is proposed to be developed in two (2) phases. Phase 1 (2026 timeframe) is reliant only on existing water (Zone B1A) and sanitary infrastructure. Both SWM Pond 1 and SWM Pond 2 would be constructed to service Phase 1. Phase 2 (2028 to 2030 timeframe) relies on the construction of future planned Regional services on North Service Road as well as Master Planning decisions regarding the extension of Pressure Zone B2 (for both 1200 King Road and the adjacent Aldershot MTSA).

Report Prepared by:



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APPENDIX A
Drawings and Figures

APPENDIX B
SWM Calculations

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APPENDIX C
Water Hydraulic Modelling Report (MES)

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APPENDIX D
Wastewater Design Sheets

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APPENDIX E
Fluvial Geomorphology Report (GEO Morphix)

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APPENDIX F
Background Report Excerpts