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Assessment of Adequacy of Public Service

Quinn's Pointe Phases 6-8



Assessment of Adequacy of Public Services Quinn's Pointe Phases 6-8

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Executive Summary

This Assessment of Adequacy of Public Services (AAPS) has been prepared with functional servicing level in support of a joint Draft Plan and Zoning By-Law application. This Report establishes a comprehensive servicing framework for Quinn's Pointe Phases 6–8, site at 3882 and 4000 Barnsdale Road. This AAPS builds upon the high-level servicing strategies outlined in the *Addendum to Extend the Master Servicing Study (2018)* (MSS Addendum - May 27, 2026), where the subject lands were formerly referred to as the S1 Area, and herein redesignated as Quinn's Pointe Phases 6–8.

This AAPS reflects the updated Concept Plan, identified as Concept Plan 54 (Rev. 17). As this plan differs from the Concept Plan included in the MSS Addendum (May 27, 2026), the servicing approaches and strategies have been refined to accommodate revisions to the road network and development configuration. Accordingly, this Report presents updated servicing strategies and expands upon the level of detail typically provided in an AAPS by documenting the supporting technical analyses and functional design work originally developed as part of the MSS Addendum.

Given the iterative nature of Concept Plans, this AAPS establishes a defensible yet adaptable servicing framework to support a future Draft Plan of Subdivision for Quinn's Pointe Phases 6–8., while allowing for refinement during detailed design of individual development phases.

The proposed servicing approach integrates stormwater, sanitary, and water infrastructure, with particular emphasis on stormwater management. The stormwater strategy is founded on a treatment train approach that incorporates Etobicoke Exfiltration Systems (EES) in areas with suitable subsurface conditions, complemented by a downstream filtration facility located within a dedicated Stormwater Management Block to service non-EES areas. This approach is consistent with the screening outlined in the MSS Addendum and has been further refined based on the latest Concept Plan and Draft Plan of Subdivision.

The recommended solution features a filtration-based dry facility with an upstream Manufactured Treatment Device (MTD), functioning solely sized as pre-treatment. A conservative design philosophy has been applied, whereby the filtration-based dry pond is independently sized to achieve the required 80% Total Suspended Solids (TSS) removal target, in accordance with accepted Ontario design practices and precedent applications within the City of Ottawa. An EES has been incorporated to the maximum feasible extent, subject to subsurface and CLI-ECA constraints, to support water balance objectives and reduce runoff volume.

This AAPS demonstrates that the subject lands can be adequately serviced with municipal water, sanitary sewer, storm sewer, and stormwater management infrastructure in accordance with City of Ottawa standards and applicable provincial requirements. The servicing strategy, initially identified in the MSS Addendum and further developed herein, confirms that sufficient system capacity exists—or can be provided through planned infrastructure improvements—to accommodate the proposed development without adverse impacts to existing municipal services.

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This AAPS documents the technical evaluations, analyses, and conclusions supporting the preferred servicing strategy for the Quinn's Pointe Phases 6-8. Given the level of detail presented, certain background components of the MSS Addendum—such as screening exercises—are not repeated herein; accordingly, this AAPS should be reviewed in conjunction with the MSS Addendum.

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1.0 Introduction

1.1 General

Minto Communities Inc. (Minto) has retained J.L. Richards & Associates Limited (JLR) to prepare an Assessment of Adequacy of Public Services (AAPS) outlining the municipal infrastructure required to support the proposed Quinn's Pointe Phases 6–8 (3882 and 4000 Barnsdale Road) within the City of Ottawa, consisting of both residential and industrial land uses. The AAPS will support a joint Draft Plan and Zoning By-Law application.

This AAPS establishes the servicing strategy for the Quinn's Pointe Phases 6–8 (3882 and 4000 Barnsdale Road) by building upon and consolidating the approaches developed in the Addendum to Extend the Master Servicing Study (2018), previously referred to as the MSS Addendum (May 27, 2026), and herein redesignated as Quinn's Pointe Phases 6–8.

As shown herein, this AAPS incorporates updates associated with the latest Concept Plan (Concept Plan 54, Rev. 17). Due to revisions to the road network and development layout relative to the MSS Addendum, this Report provides a higher level of detail than a typical AAPS, updating the underlying technical analyses and functional design framework to align with the revised Concept Plan noted above and the Draft Plan, both of which are included in Appendix A.

The MSS Addendum, together with this AAPS, confirms that the subject lands can be effectively serviced with municipal water, sanitary sewer, and stormwater management infrastructure in accordance with the City of Ottawa Sewer Design Guidelines and applicable provincial requirements. The analysis demonstrates that adequate capacity is available, or can be provided through planned infrastructure works, without causing adverse impacts to existing municipal systems.

This AAPS establishes a functional servicing framework for the proposed development by describing the overall infrastructure approach, identifying servicing requirements, and confirming compliance with applicable City of Ottawa and MECP standards. The report has been prepared in support of an Official Plan Amendment application.

Recognizing that the development concept for the Quinn's Pointe Phases 6–8 may continue to evolve—particularly in terms of roadway alignments and building typologies, as reflected in Concept Plan 54 (Rev. 16)—this AAPS is intended to provide a flexible and defensible framework to support subsequent stages of design and approval.

This AAPS summarizes the technical reviews, analyses, and conclusions that inform the preferred servicing strategy for Quinn's Pointe Phases 6–8. Given the level of detail provided, certain background components of the MSS Addendum are not repeated herein; accordingly, the MSS Addendum should be read in conjunction with this document.

Given the level of details included in the MSS Addendum, this AAPS includes summary discussions of key supporting studies typically required as part of an Official Plan Amendment application, including:

- Fluvial Geomorphological Assessment (Section 2.6.1)
- Stormwater Management Report (refer to Section 2.6.2)

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- Servicing Options Assessment (Section 9.0)

1.2 Site Description, Sub-Watersheds and Drainage Act

The subject lands encompass approximately 71 ha and are bounded by the Quinn's Pointe Subdivision to the east, the Ridge Subdivision to the north, Barnsdale Road to the south, and Borrisokane Road to the west (refer to Figure 1). The site forms part of the Barrhaven South community within the former City of Nepean and is legally described as portions of Lots 6 and 7, Concession 3 (Rideau Front), City of Ottawa.

The property consists primarily of undeveloped former agricultural lands, two woodlots, and a man-made pond constructed in the 1990s at the southwest corner of Borrisokane Road and Barnsdale Road. This pond area is currently designated for future industrial development. The lands are situated immediately adjacent to the existing urban boundary.

Topographically, the site is characterized by noticeable relief, generally trending from north to south, with elevations decreasing from approximately 112 m near the northern limit to 100 m near Barnsdale Road. From a hydrologic perspective, runoff from the site currently drains via sheet flow to two subwatersheds (see Figure 2):

- The majority of Quinn's Pointe Phases 6–8. (approximately 66.5 ha) ultimately drains to Mud Creek via the Thomas Baxter Municipal Drain;
- A smaller portion (approximately 3.4 ha) in the northeastern area drains toward the Jock River.

As a significant portion of Quinn's Pointe Phases 6–8. contributes to a Municipal Drain, Robinson Consultants Inc. (RCI) has been appointed by Council as Drainage Engineer to prepare an updated Engineer's Report under the Drainage Act (R.S.O. 1990, c. D.17), documenting proposed drainage modifications, including the partial redirection of flows currently tributary to the Thomas Baxter Municipal Drain.

1.3 Land Use, Concept Plan Revisions and Development Staging

Quinn's Pointe Phases 6–8. represents a logical and efficient extension of the Barrhaven South community, offering coordinated phasing opportunities and strong integration with adjacent developments and existing infrastructure, particularly the Quinn's Pointe Subdivision.

The proposed development aligns with the broader objectives of the City's Official Plan and contributes to the creation of a complete community, incorporating elements consistent with a 15-minute neighbourhood, although this is not a formal requirement for the area.

The lands are currently zoned Rural Countryside (RU), Agricultural (AG), and Mineral Aggregate Reserve (MR). Since completion of the MSS Addendum (May 27, 2026), the Concept Plan has been refined to reflect minor updates to the road network and building typologies within Stages 6 through 8. The updated Concept Plan and corresponding Draft Plan of Subdivision are provided in Appendix A.

Based on the revised plans, the proposed land uses include a mix of low- and medium-density residential development, vacant blocks to be developed later as industrial, retained woodlots,

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parkland, a stormwater management facility block, and servicing easements. The southern woodlot within the industrial/logistics area has been identified for potential future conveyance as an urban natural feature, consistent with the assumptions carried forward in both the MSS Addendum and this AAPS.

Development of Quinn's Pointe Phases 6–8. is anticipated to proceed in three primary phases, beginning with Stage 6, followed by Stages 7 and 8 (either concurrently or sequentially), with industrial lands developed subsequently.

Stage 6: Located adjacent to the existing Quinn's Pointe Subdivision and Caivan's Ridge development, Stage 6 is planned to include approximately 272 single-detached dwellings and 170 executive townhouses, along with a park block (Block 539), stormwater management block (Block 540), and retained woodlot (Block 538) as illustrated in the Concept Plan (Appendix A). Although block numbers are slightly different from those illustrated in the Draft Plan (Appendix A) those in the Concept Plan were used herein.

Stages 7 and 8: Situated south and west of Stage 6, extending toward Barnsdale Road, these stages will accommodate additional low-density residential development. Stage 7 is planned to include 24 single-detached units, 133 executive townhouses, and 42 avenue townhouses, while Stage 8 planned to include 152 single-detached units, 93 executive townhouses, and 56 avenue townhouses as illustrated in the Concept Plan.

Following, or potentially concurrent with these residential phases, vacant lands located in the northwest (13.22 ha) and southwest (3.19 ha) portion are intended to be developed later as industrial land usage.

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Quinn's Pointe Phases 6-8

2.0 Existing Conditions

2.1 General

Existing conditions within Quinn's Pointe Phases 6–8 were characterized in JLR's 2025 Existing Conditions Report (ECR), which provides a comprehensive overview of site-specific attributes including climate, geology, physiography, drainage patterns, surficial and bedrock conditions, aquifer systems, groundwater levels, and adjacent infrastructure. The most recent version of the ECR was included in the MSS Addendum for reference.

As part of the 2018 BSUEA Master Servicing Study (MSS), several background documents were reviewed and summarized in a Synopsis of Background Documents. The 2025 ECR supplements this information by incorporating more recent engineering studies related to Caivan's development activities within the former aggregate extraction lands. These additional documents were appended to the MSS Addendum.

2.2 Surface Water Characteristics, Watercourses and Site Hydrology

Quinn's Pointe Phases 6–8, which comprises approximately 71 ha, is located within the Barrhaven South community in the former City of Nepean and is bounded by the Quinn's Pointe Subdivision to the east, Caivan's Ridge Subdivision to the north, Barnsdale Road to the south, and Borrisokane Road to the west (refer to Figure 1 – Location Plan).

Analysis of LiDAR data and detailed topographic survey indicates that the site currently drains to two distinct subwatersheds, as illustrated on Figure 2 (Study Area and Drainage Divide). The majority of Quinn's Pointe Phases 6–8. drains toward Mud Creek via the Thomas Baxter Municipal Drain, while a smaller northeastern portion contributes to the Jock River watershed. Given that a large portion of the site is tributary to a Municipal Drain, Robinson Consultants Inc. (RCI) has been appointed by Council as the Drainage Engineer to prepare an updated Engineer's Report under the Drainage Act to address proposed drainage modifications, including partial disconnection from the Thomas Baxter Municipal Drain.

Subcatchment delineation across Quinn's Pointe Phases 6–8. has been developed based on LiDAR-derived topography and the influence of surface infrastructure, including roadways and culverts, which govern localized drainage patterns. The 2025 ECR (included as Appendix B in the MSS Addendum), particularly Sections 5.2 and 5.3 and Figures 5-2 through 5-4, presents detailed mapping of drainage areas to boundary culverts, as well as associated land use and soil characteristics for each contributing subcatchment.

Pre-development hydrologic conditions were originally assessed as part of the 2018 MSS using the SWMHYMO modelling platform, with flows evaluated at key boundary culverts under a range of storm events. Early MSS reporting (R0, October 2025) referenced flows derived from the 2016 ECR for the broader BSUEA study area.

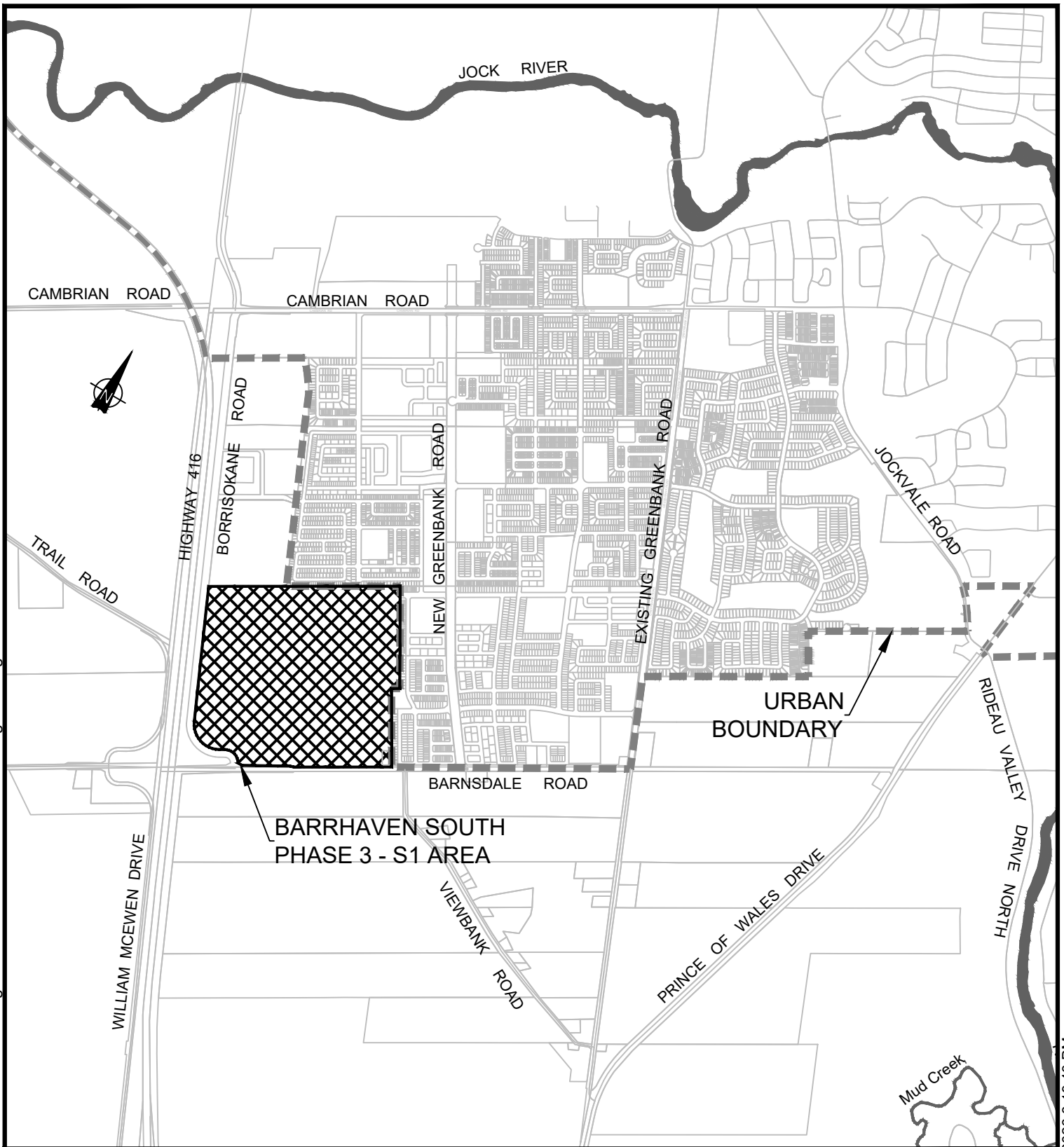
To enhance model resolution—particularly in relation to existing roadway culverts—the pre-development hydrologic model was subsequently transitioned to the PCSWMM platform. This approach ensures consistency with post-development modelling, which is also conducted in PCSWMM. Comparative results indicate that the original SWMHYMO simulations are consistent with the updated PCSWMM outputs. These models provide estimates of peak flows and runoff

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volumes under storm events ranging from the 2-year to 100-year return periods, in addition to the 4-hour 25 mm design storm.

Refined pre- and post-development modelling results are presented in Section 3.12.4 of this AAPS, including flow assessments at key locations such as roadside ditches along Borrisokane Road and Barnsdale Road, as well as at the headwaters of the Thomas Baxter Municipal Drain.

Model results under frequent storm conditions (4-hour, 25 mm) indicate relatively low flow rates at the site's boundary culverts, attributable to the high infiltration capacity of native soils and low existing imperviousness. Under more extreme storm events, runoff volumes increase as soil infiltration capacity is exceeded due to saturation.

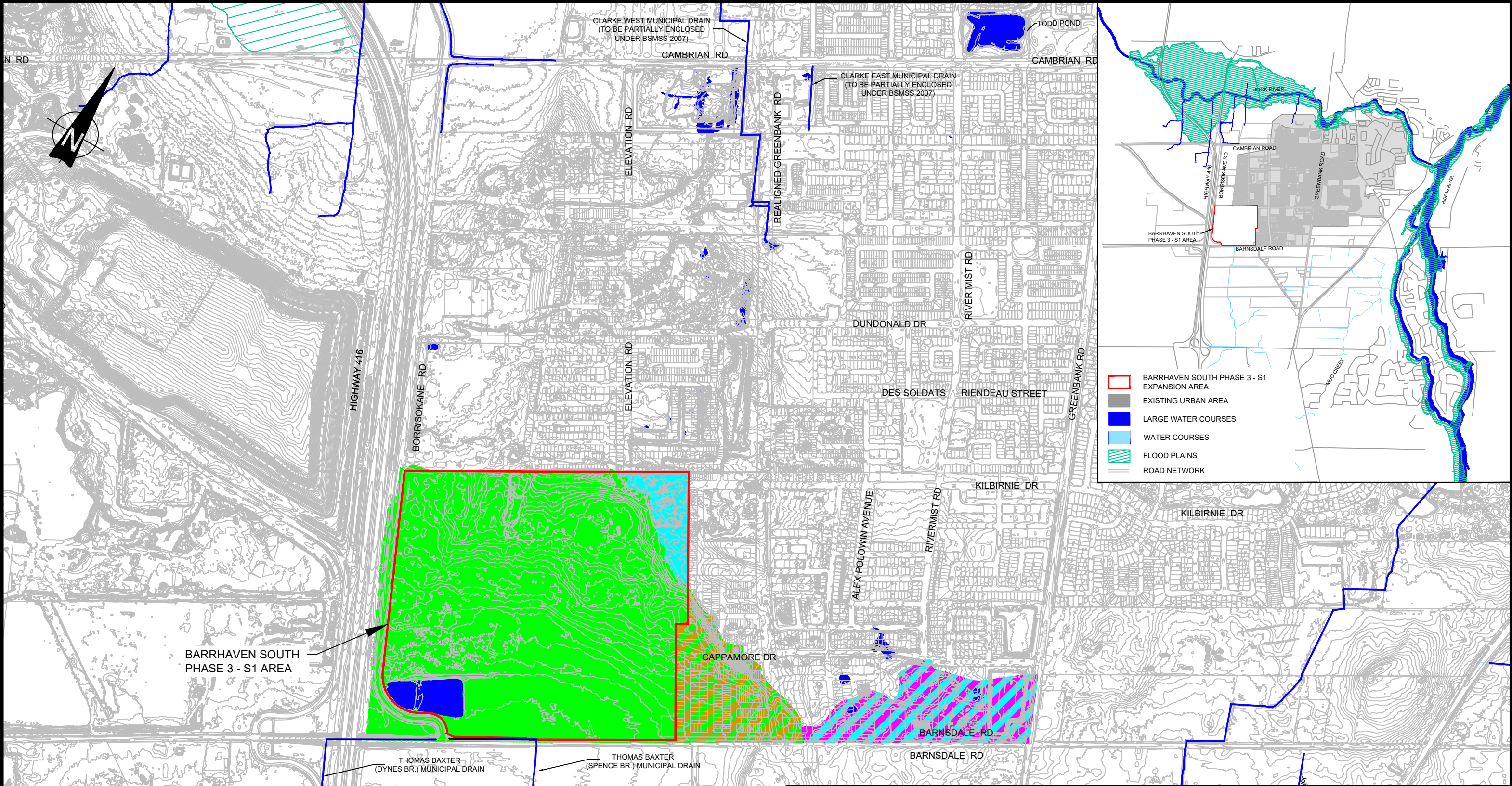


PROJECT: QUINN'S POINTE PHASES 6-8
3882 & 4000 BARNSDALE ROAD

DRAWING: LOCATION PLAN

 J.L. Richards ENGINEERS-ARCHITECTS-PLANNERS	This drawing is copyright protected and may not be reproduced or used for purposes other than execution of the described work without the express written consent of J.L. Richards & Associates Limited.	DESIGN: GF	DRAWING #: FIGURE 1
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File Location: P:\26000\26610-006 - Quinn's Pointe Stage 6\10-Production\01-Civil\Servicing Brief Figures\26610-006 C FIGURE 2.dwg



- BARRHAVEN SOUTH PHASE 3 - S1 EXPANSION AREA
- EXISTING URBAN AREA
- LARGE WATER COURSES
- WATER COURSES
- FLOOD PLAINS
- ROAD NETWORK

LEGEND

- S1 URBAN EXPANSION STUDY AREA
- SUBCATCHMENT AREA BOUNDARY
- WATERCOURSE/ MUNICIPAL DRAINS
- AGGREGATE EXTRACTION AREA
- TRIBUTARY AREA TO JOCK RIVER SUBWATERSHED
- TRIBUTARY AREA TO MUD CREEK SUBWATERSHED
- PRE-DEVELOPMENT EXTENTS TRIBUTARY TO RIDEAU RIVER REDIRECTED TO JOCK RIVER SUBWATERSHED WITH URBANIZATION
- CATCHMENT AREA TO HYDROLOGIC BOUNDARY CONTROL POINTS
- WATER BODIES
- FLOOD PLAIN
- PRE-DEVELOPMENT EXTENTS TRIBUTARY TO MUD CREEK REDIRECTED TO JOCK RIVER SUBWATERSHED WITH URBANIZATION

PROJECT:

QUINN'S POINTE PHASES 6-8
3882 & 4000 BARNSDALE ROAD

DRAWING:

STUDY AREA & DRAINAGE DIVIDE



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FIGURE 2

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Quinn's Pointe Phases 6-8

2.3 Site Features

Quinn's Pointe Phases 6–8. (Figure 2) comprises predominantly undeveloped former agricultural lands, two woodlots identified by the project biologist as being of ecological significance. As noted previously, the woodlot located within the proposed industrial and logistics lands has been identified as a suitable candidate for future conveyance as an urban natural feature through a subsequent planning process.

Accordingly, both this AAPS and the MSS Addendum assume the potential future protection and conveyance of this woodlot. The site also includes a man-made pond situated at the southwest corner of Borrisokane Road and Barnsdale Road. The lands are located immediately adjacent to the existing urban boundary.

Topographically, the site is characterized by notable variation in elevation, generally sloping from north to south. Elevations range from approximately 112 m at the northern extent to about 100 m near Barnsdale Road. The man-made pond, constructed in the 1990s, is visible at the intersection of Borrisokane Road and Barnsdale Road, as confirmed through available aerial imagery (e.g., geoOttawa).

2.4 Source Protection

The Mud Creek Subwatershed Study (2015) identifies Quinn's Pointe Phases 6–8. as being located within the Mississippi-Rideau Source Protection Region (MRSPR), as illustrated in Figure 8 of the source protection mapping. In addition, the entire site is designated as a Highly Vulnerable Aquifer based on mapping from the Ministry of the Environment, Conservation and Parks (MECP).

The conceptual storm servicing approach developed as part of this AAPS incorporates infiltration measures to the greatest feasible extent, while recognizing the constraints outlined in Table A2 of the Consolidated Linear Infrastructure Environmental Compliance Approval (CLI-ECA). These measures are intended to minimize potential impacts to the underlying shallow aquifer system, including the esker formation.

In regard to the screening process, sections 3.10 and 3.11 of the MSS Addendum be reviewed as it outlines the recommended servicing strategies that respond to site-specific constraints and opportunities. Section 3.10 of the MSS Addendum describes the proposed infiltration-based measures, which are implemented to the maximum practical extent in consideration of CLI-ECA limitations. These strategies are consistent with those initially established in the 2018 BSUEA MSS.

Section 3.11 of the MSS Addendum summarized the evaluation of filtration-based stormwater management strategies, including the screening process and identification of the preferred approach for areas where infiltration is not feasible or constrained.

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2.5 Geotechnical and Hydrogeological Investigations

2.5.1 General

A Geotechnical Investigation as well as a Hydrogeological Investigation were carried out by Paterson Group Inc. (Paterson) to assess general subsoil and groundwater conditions within the boundaries of Quinn's Pointe Phases 6–8. and to provide geotechnical recommendations for the design of municipal services associated as well as sub-surface information to assess infiltration measures with the proposed development.

The latest documents prepared by Paterson are as follows:

- Existing Conditions – Geotechnical Barrhaven South Phase 3 – S1 Area (June 27, 2025)
- Hydrogeological Existing Conditions Report – Barrhaven South Phase 3 S1 Area (March 25, 2025).

Both reports have been submitted to the City by Paterson under separate cover.

2.5.2 Geotechnical

Based on the Existing Conditions – Geotechnical Barrhaven South Phase 3 (S-1 Area) report prepared by Paterson, a total of 34 boreholes and 51 test pits were completed across the site and the immediate surrounding area with limited coverage in the northern wooded area and the southwest portion of the site near the existing stormwater pond. Borehole data indicate that the overburden thickness across the site is substantial, generally ranging from approximately 15 to 25 m, and bedrock was not encountered within the depths explored. The site is predominantly underlain by non-cohesive silty sand to sandy silt, which is considered suitable subgrade material for municipal servicing.

Localized silty clay deposits were encountered within the S-1 Lands, primarily in the northwest portion of the site and in isolated areas of the central-east. These clay deposits are not continuous and occur within an otherwise granular overburden profile dominated by silty sand and sandy silt. Where present, clay is typically encountered at shallow depths of approximately 0.5 to 3.5 m below ground surface, with thicknesses generally ranging from 0.5 to 3.0 m.

From a civil servicing and grading perspective, the presence of silty clay locally governs permissible grade raise, rather than limiting the feasibility of deep servicing. The report recommends a preliminary maximum grade raise of approximately 2.5 m in areas where silty clay remains beneath settlement-sensitive infrastructure, to control long-term total and differential settlement. These restrictions apply only within clay-affected zones and do not impact most of the site, which is underlain by granular soils suitable for conventional municipal servicing. In areas where higher grade raises are required, or where servicing alignments intersect clay-affected zones, supplemental investigation and mitigation measures may be required at later stages of design.

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2.5.3 Hydrogeological and Assessment of Infiltration Rates

2.5.3.1 Infiltration Characteristics and Soil Permeability

Paterson's Hydrogeological Existing Conditions Report for Quinn's Pointe Phases 6–8 (March 2025) states that the site is predominantly underlain by non-cohesive silty sand to sandy silt overburden associated with glaciofluvial and glaciomarine deposits. These materials form an unconfined overburden aquifer with generally favourable infiltration characteristics. Hydraulic conductivity testing of the saturated overburden yielded values ranging from 4.6×10^{-7} to 4.6×10^{-5} m/s, which are consistent with moderate to high permeability soils typically associated with groundwater recharge areas and was found to be consistent those measured as part of the BSUEA.

The in-situ infiltration testing results from the report show that typical infiltration rates generally range between 50 and 100 mm/hr, with higher infiltration rates exceeding 100 mm/hr primarily observed in the northwestern portion of the site. These elevated rates are attributed to coarser granular soils associated with the Kars Esker system. From a stormwater management and Low Impact Development (LID) perspective, these results confirm that the site is generally suitable for infiltration-based solutions, provided that adequate vertical separation to the groundwater table can be maintained. Simulated unfactored infiltration rates, for the areas intended to be serviced by an EES, is discussed in greater details in Section 3.10.3 of the MSS Addendum.

2.5.3.2 Groundwater Levels

From the borehole and monitoring well data presented in the Hydrogeological Report for Quinn's Pointe Phases 6–8 (March 2025), the overburden thickness across the site is substantial, generally ranging from 15 to 25 m. Bedrock was not encountered in boreholes advanced to depths of up to approximately 17 m below ground surface, indicating that bedrock depth does not present a constraint to deep servicing infrastructure. Instead, the depth to the groundwater table represents the primary hydrogeological constraint for servicing and infiltration design.

Groundwater elevations across the Quinn's Pointe Phases 6–8 are relatively consistent, generally ranging from approximately 98 m to 101 m in the northern portion of the site and approximately 96 m to 99 m in the southern portion. However, due to the natural site topography, which rises from approximately 99 m near Barnsdale Road to approximately 110 m toward Kilbirnie Drive, the depth to groundwater relative to surface elevation increases toward the northern portions of the site. In these areas, groundwater levels are locally observed at depths of approximately 6 m to 12 m below ground surface, providing sufficient vertical separation to support deeper sewer infrastructure and infiltration systems, including the proposed EES system.

Conversely, the southern portion of the site exhibits shallower groundwater levels due to the lower existing surface elevation and localized perched groundwater conditions, which may constrain the feasibility and extent of infiltration-based allowances in this area.

The Hydrogeological Report provided seasonal high groundwater levels for each borehole. This data was subsequently used by Paterson to generate seasonal high groundwater contours. The compiled information was subsequently adopted as the basis for design considerations

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presented in this Addendum. This information has been compiled on a Figure (Figure 9A) along with other information pertaining to the proposed configuration of the EES and included in Appendix D.

2.5.3.3 Groundwater Mounding Considerations

An Etobicoke Exfiltration System (EES) was the recommended servicing solution as part of the 2018 BSUEA MSS to achieve distributed infiltration and maintain the pre-development infiltration condition. The same screening process was carried out as part of this AAPS and MSS Addendum, which has been documented in the MSS Addendum (Section 3.10.3).

Similarly to the BSUEA, an EES is being proposed within areas underlain by coarse native materials, such as sand or sandy gravel, as was the case in the BSUEA and some of the areas within Quinn's Pointe Phases 6–8. There is a potential risk of groundwater mounding although more probable when native material are mostly fine-grained soils, which is not representative of the areas in Quinn's Pointe Phases 6–8. Nonetheless, groundwater mounding remains an important design consideration and must be evaluated to confirm system feasibility and long-term performance in accordance with the Sewer Design Guidelines (Appendix 10). Groundwater mounding was reviewed as part of the MSS Addendum (refer to Section 3.10.3) and should be consulted in conjunction with this AAPS.

2.6 Fluvial Geomorphological Report and Stormwater Management Report

2.6.1 Fluvial Geomorphological Assessment

As identified in the Environmental Impact Statement (prepared by Fotenn), there are no defined watercourses, engineered channels, or active fluvial systems present within or immediately adjacent to Quinn's Pointe Phases 6–8. This condition reflects limited surface runoff concentration and minimal fluvial activity across the site.

Based on these findings, a detailed fluvial geomorphological assessment was not required as part of either the Existing Conditions Report or the MSS Addendum. This conclusion is further supported by the proposed servicing strategy, which includes the partial redirection of approximately 59.8 ha of drainage from existing systems. Given the absence of identifiable fluvial features, corroborated by site investigations and LiDAR analysis, and the lack of downstream fluvial sensitivity, a comprehensive fluvial assessment was not warranted and was therefore not included within the project scope.

2.6.2 Stormwater Management Report

The Official Plan Amendment requirements identify the need for a Stormwater Management Report (SWMR). However, given the level of detail provided within the MSS Addendum and this AAPS, all core components typically required for an SWMR have been incorporated within these documents. As such, it is anticipated that the MSS Addendum, in conjunction with this AAPS, will satisfy the need for a stand alone SWMR.

Notwithstanding the above, a separate Stormwater Management Report was prepared and will be submitted to the Ministry of Transportation of Ontario (MTO) in support of a Highway Corridor Management (HCM) permit application. This stand-alone SWMR will focus specifically on off-site discharge conditions—both pre- and post-development—directed to the Borrisokane Road

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roadside ditch system, part of Highway's 416 corridor. Given that this information was extracted from this Report, the SWMR will not accompany the Official Plan Amendment.

3.0 Storm Drainage and Servicing

3.1 Background

A stormwater management strategy was developed for Quinn's Pointe Phases 6–8 to ensure that prior to any off-site discharge, runoff would meet both municipal and provincial requirements. In addition, the stormwater management strategy also requires that the integrity of the downstream systems be maintained from both a minor and major systems perspective.

The stormwater management strategy should include measures to promote infiltration, to the maximum extent, while recognizing constraints, supplemented by filtration solutions to the maximum extent. In addition to the above strategies, SWM needs is to be identified, including the facility type, size and location, along with major overland flow paths and proposed storm sewer trunks.

From a surface water perspective, Quinn's Pointe Phases 6–8 is tributary to the following two (2) sub-watersheds as depicted on Figure 2:

- **Jock River:** A small area located in the northeast portion of the Study Area.
- **Mud Creek:** The majority of the Study Area surrounding the northeast portion.

Quinn's Pointe Phases 6–8 is bounded by two (2) existing subdivisions and by the current urban boundary along both southern and western limits. Hence, linear infrastructure exists at the northern boundary with Caivan's Greenbank Development (The Ridge) and eastern boundary with Quinn's Pointe Subdivision (existing Stages 3 and 4 and future Stage 5).

There is an existing spill-over pond located within Quinn's Pointe Phases 6–8 (southeastern corner). This facility was designed as part of Quinn's Pointe Subdivision with the objective of intercepting major overland flow to prevent cascading onto the future Realigned Greenbank Road, an arterial.

There is no inlet sewer originating from Quinn's Pointe Subdivision to the spill-over pond as inflow to this feature is solely via cascading of major overland flow. The spill-over pond is currently connected to Quinn's Pointe Subdivision by means of an outlet structure complete with the 900 mm diameter outlet sewer which slowly releases the captured major overland flow. Figure 3 depicts existing surface drainage conditions, including existing ponds and culverts, in the vicinity of Quinn's Pointe Phases 6–8.

3.2 Storm Connections and Limitations

Quinn's Pointe Phases 6–8 is not directly connected to any defined receiving water course. As such, runoff from the study area will be captured and conveyed through a combination of proposed and existing storm sewers, end-of-pipe facilities, and/or roadside ditches prior to being discharged off-site, within the available capacity of downstream systems. Figure 3 illustrates the internal drainage within Quinn's Pointe Phases 6–8, as well as the perimeter culverts.

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Figure 4 presents the existing and planned storm servicing infrastructure surrounding Quinn's Pointe Phases 6–8. As outlined in the Existing Conditions Report (Appendix B of the MSS Addendum), several potential outlet systems were evaluated to determine their suitability for conveying flows from the site. This assessment concluded that a connection to the Elevation Road storm system is not feasible, as there is insufficient residual capacity to accommodate partial or full flows from Quinn's Pointe Phases 6–8.

With respect to the open ditch system along Borrisokane Road, which is part of MTO's 416 drainage corridor, the storm servicing strategy has been developed to minimize reliance (i.e., capacities) on this outlet. Only localized drainage is directed to the ditch system, given that it ultimately conveys flows to the Thomas Baxter Municipal Drain. Any modifications to the contributing drainage area or hydrologic regime of the Municipal Drain will require documentation through an updated Engineer's Report under the Drainage Act to facilitate a potential partial or full disconnection.

Under existing conditions, flows collected within the spillover pond located in the southeast portion of Quinn's Pointe Phases 6–8 are conveyed via the Quinn's Pointe storm sewer system, which includes pipes ranging from 900 mm to 1650 mm in diameter. These flows are ultimately directed to Dry Pond 2 before discharging into the Greenbank Road trunk storm system. The proposed storm servicing approach has been developed to conform to the allowable flow limits within the downstream 900 mm diameter storm sewer serving the Quinn's Pointe system.

3.3 Roadway Projects

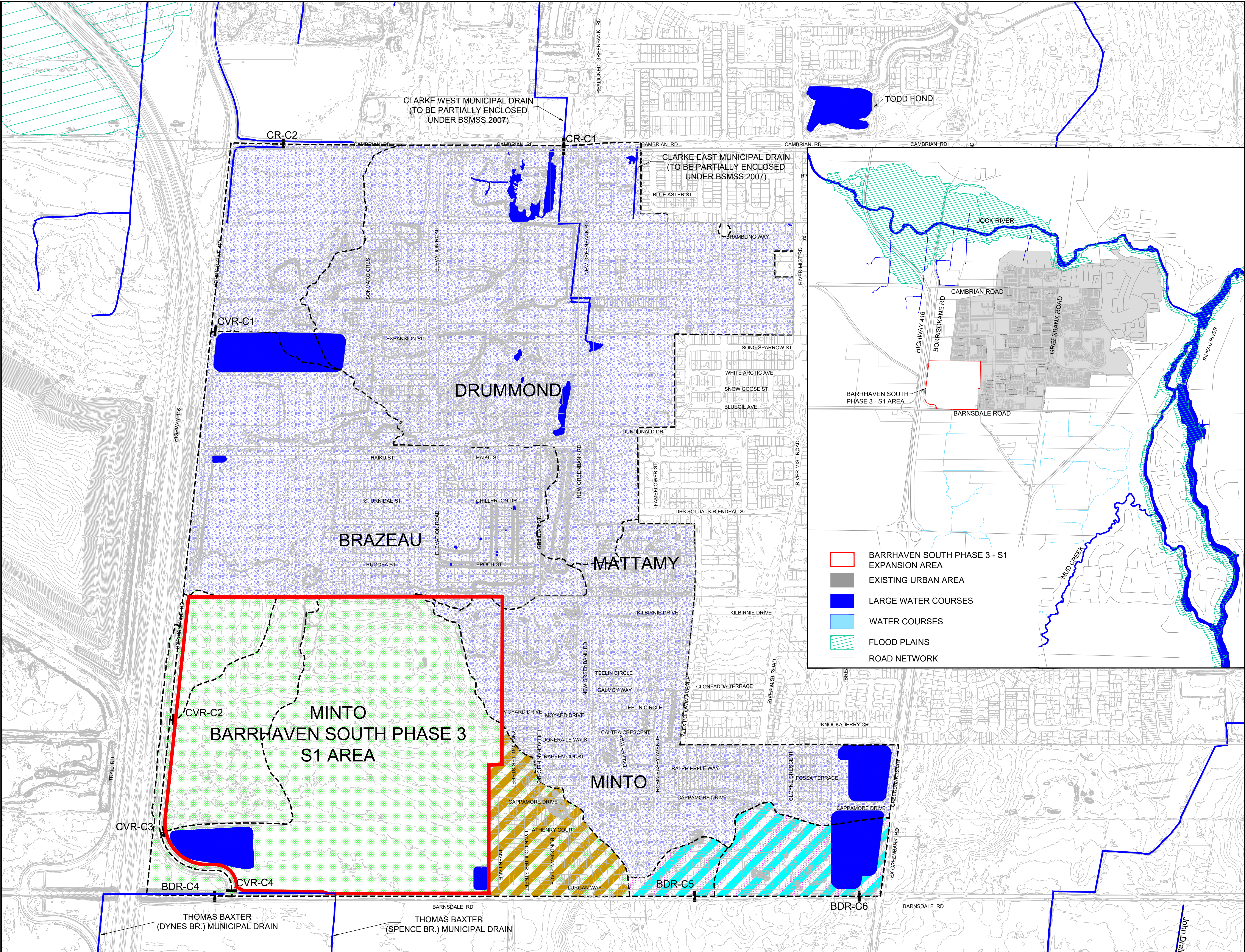
3.3.1 Barnsdale Road Widening

In 2025, the City of Ottawa initiated a Class Environmental Assessment for the Barnsdale Road widening project, from Prince of Wales Drive to Highway 416. It is important to note that there is a section of Barnsdale that bounds Quinn's Pointe Phases 6–8 and that will be part of the future Highway 416 interchange, an initiative led by the Ministry of Transportation (MTO), as discussed in Section 3.3.2 (below).

The above-noted undertaking will be carried out under a Schedule C Class Environmental Assessment and will examine widening options of Barnsdale Road, from a two (2) to four (4) lanes. A Transportation Environmental Study Report (ESR) will ultimately be prepared that would document outlet locations for the various sections of the roadway, along with high-level water quality and quantity measures to meet regulatory requirements. It should be noted that detailed design of the Barnsdale Road widening will need to be developed in accordance with the requirements of the City of Ottawa CLI-ECA (No 008-S701).

The servicing strategy developed and presented herein or in the MSS Addendum did not account for any drainage from the Barnsdale Road widening project in either the storm sewers or in the end-of-pipe facility within Quinn's Pointe Phases 6–8. It has been assumed that drainage for the widening project would be conveyed based on the pre-development flow pattern, which is from the roadway in a southerly direction, away from Quinn's Pointe Phases 6–8. It is expected the roadway project would culminate through the hierarchy of the CLI-ECA.

File Location: P:\26610\26610-006 - Quinns Pointe Stage 010-Production\01-Civil\Servicing Brief Figures\26610-006 C FIGURE 3 EX DRAINAGE.dwg



LEGEND

- URBAN EXPANSION STUDY AREA
- TRIBUTARY AREA TO JOCK RIVER SUBWATERSHED
- TRIBUTARY AREA TO MUD CREEK SUBWATERSHED
- TRIBUTARY AREA TO MUD CREEK REDIRECTED TO JOCK RIVER SUBWATERSHED WITH URBANIZATION
- TRIBUTARY AREA TO RIDEAU RIVER DIRECTED TO JOCK RIVER SUBWATERSHED WITH URBANIZATION
- CATCHMENT AREA TO HYDROLOGIC BOUNDARY CONTROL POINTS
- WATER COURSE / MUNICIPAL DRAINS
- WATER BODIES
- FLOOD PLAIN

NOTE: KILBIRNIE ROAD, ELEVATION ROAD AND A PORTION OF CAPPAMORE DRIVE CLASSIFIED COLLECTOR STREETS

No.	ISSUE / REVISION	DD/MM/YY

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SCALE: 1:4500

CLIENT:

minto
Communities

CONSULTANT:

J.L. Richards
ENGINEERS - ARCHITECTS - PLANNERS

CONSULTANT:

PROFESSIONAL STAMP	PROJECT NORTH

PROJECT:

QUINN'S POINTE PHASES 6-8

3882 & 4000 BARNSDALE ROAD

DRAWING:

EXISTING SURFACE DRAINAGE CONDITIONS

DESIGN: KF	DRAWING #:
DRAWN: KC	FIGURE 3
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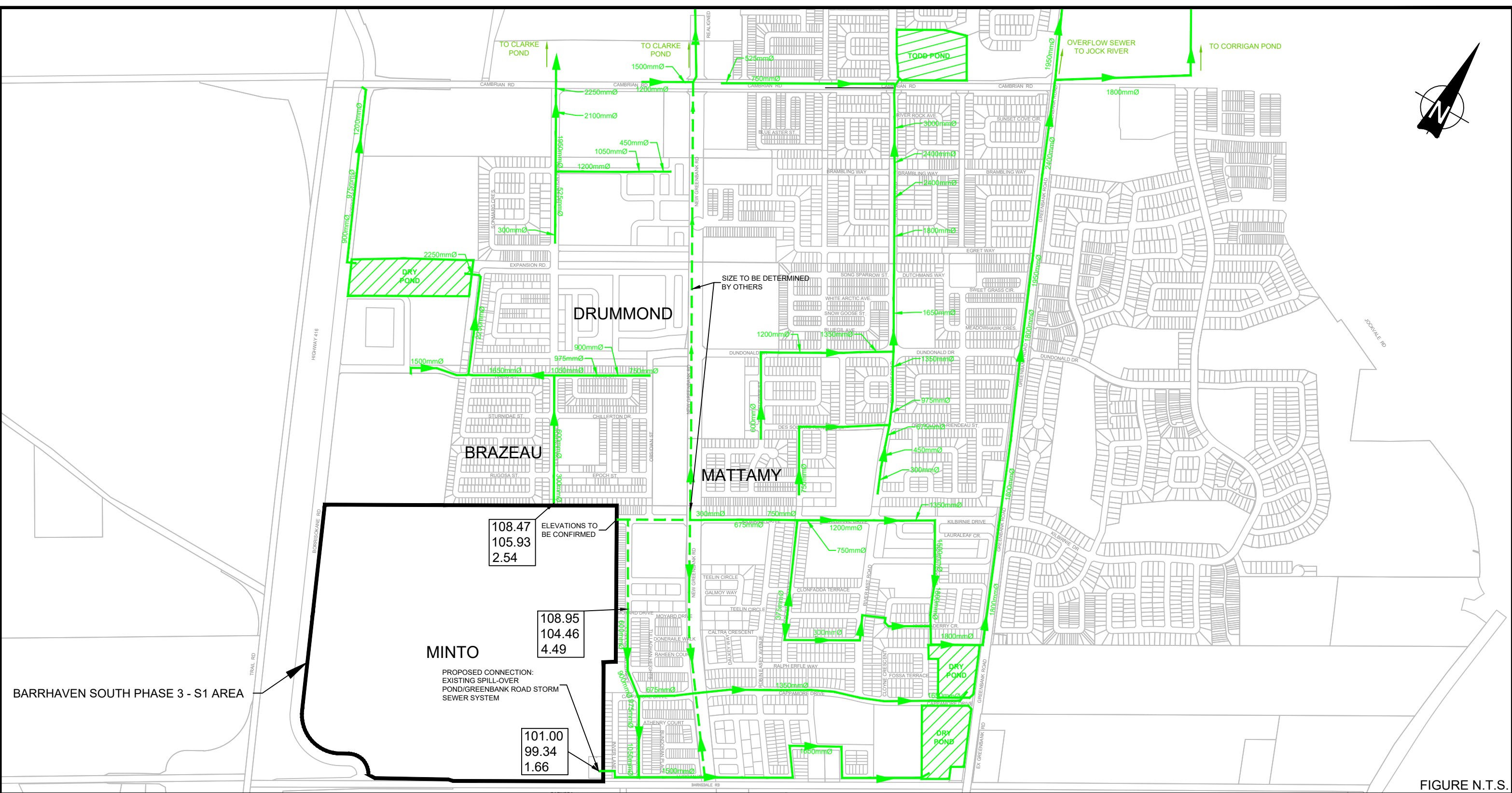


FIGURE N.T.S.

LEGEND

- EXISTING STORM SEWER
- PLANNED STORM SEWER

98.00	EXISTING GROUND ELEVATIONS (±)
94.07	STORM SEWER OBVERT
3.9	COVER DEPTH (m)

PROJECT:

QUINN'S POINTE PHASES 6-8
3882 & 4000 BARNSDALE ROAD

DRAWING:

EXISTING AND PLANNED STORM SERVICING INFRASTRUCTURE



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FIGURE 4

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From a quantity control perspective, simulation results summarized in Section 3.12.4 have shown a decrease in both peak flows and runoff volumes released to the headwater of the Thomas Baxter Municipal Drain. Hence, an opportunity may exist to outlet some of the runoff from the widening or interchange projects (refer to Section 3.3.2 below) to the Thomas Baxter Municipal Drain and take advantage of the partial disconnection of the majority of Quinn's Pointe Phases 6–8 as shown in Section 3.12.5.

3.3.2 Highway 416 at Barnsdale Road Interchange

A Transportation Environmental Study Report (ESR) was completed in 2023 by Morrison Hershfield. A series of alternatives were reviewed against six (6) separate criteria with Alternative 5 (Parclo AB/South Side Loop) selected as the technically preferred Alternative for the interchange configuration.

Section 5.1.4 of the Transportation ESR reads as follows:

“The recommended option implements a Parclo AB interchange with four (4) free flow ramps servicing movements to and from the north and two (2) inner loop ramps.”

The proposed configuration of the interchange will require the closure of roadways, including the southern portion of Borrisokane Road. Active transportation facilities are located on the north side of Barnsdale Road, with two (2) high risk potential conflict areas.

The latest Concept Plan (Appendix A) has considered land requirements for the Highway 416 / Borrisokane Road / Barnsdale Road interchange.

3.4 Master Grading Plan

Topography varies significantly throughout the Study Area, from elevation ± 109 m at the highest point near the northern boundary and drops to ± 99.5 m at the lowest point near Barnsdale Road. At the northern end of the Study Area, near the Ridge Subdivision, there is a vertical difference of 4 m within a horizontal length of 100 m. Overall, there is a grade differential of approximately 10 m from the northern to the southern portion of the Study Area.

A Conceptual Grading Plan (Drawing CG1 in Appendix H) was developed, at a macro-level, for the residential portion of Quinn's Pointe Phases 6–8. Centerline high points, with minimum 0.1% slopes from crests to crests, were established along ROWs and corridors, to identify locations and paths for major overland flow routes. Based on the high points and respective slopes between them, assumptions were made to assess where low points could be accommodated for future roadway sag storage. Overall, a target of 70 m³/ha of storage is anticipated to be achievable, which has been incorporated into the stormwater management strategy and dual drainage PCSWMM model as described in the following sections. It should be noted that the Conceptual Grading Plan was developed to target an overall balanced of cut and fill for the residential lands rather than on a phase-by-phase basis.

There are two (2) Blocks shown in the Draft Plan (Appendix A) as vacant. Both vacant lands form the western edge of the Study Limits and are bounding Borrisokane Road. A large vacant property located in the northwest (13.21 ha) and a smaller block (3.20 ha) in the southwest corner of the subject lands. Both vacant block will be developed in the future and could consist of

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industrial land uses. Given that both vacant blocks will be developed in the future, Conceptual Storm (Drawing CMST in Appendix H) and Conceptual Grading (Drawing CG1) were developed for these blocks as follows:

- **Northern Block:** Runoff from the entire northern Block will be conveyed to the subdivision's storm sewer system along Moyard Drive. It was assumed that this vacant block would be developed as industrial in the future. Runoff will be captured by the storm sewer identified as from STMH631 to STMH630A to STMH630 (refer to Drawings CS1 and CMST in Appendix H). This receiving storm sewer consists of an EES supplemented by a traditional storm sewer, as both contributions do not require any additional quality treatment. Treated stormwater from this Block combined to the areas with an EES will be conveyed easterly along Moyard Drive and then southerly along Street 13 and would connect to the Elevation Road EES system which at that point, would not receive any runoff from Elevation Road. From Manhole STM604, runoff would ultimately into the proposed stormwater management dry pond (Block 540), at STMH600, thereby by-passing the filtration dry pond.
- **Southern Block:** Runoff from the entire southern Block will be conveyed to the Subdivision's storm sewer system along Cappamore Drive. It was assumed that this vacant block would be developed as industrial in the future. Runoff will be captured by the storm sewer identified as from STMH680 to STMH647 to STMH646 (refer to Drawings CS1 and CMST in Appendix H). This receiving storm sewer consists of an EES supplemented by a traditional storm sewer, as both contributions do not require any additional quality treatment. Treated stormwater from this Block combined to the areas with an EES will be conveyed northerly along Street 14 and then easterly along Moyard Drive and would connect to the Elevation Road EES system which at that point, would not receive any runoff from Elevation Road. From Manhole STM604, runoff would ultimately into the proposed stormwater management dry pond (Block 540), at STMH600, thereby by-passing the filtration dry pond.

As a result of the above-noted sewer servicing strategies, the northern and southern vacant blocks developed in the future with industrial usage, are proposed to be entirely disconnected from both the Borrisokane Road ditch system (MTO corridor) and the Thomas Baxter Municipal Drain. In support of the urbanization of the southern vacant block, Minto had retained Paterson to provide guidance with this future development. A bathymetric survey and Memorandum were prepared by Paterson with geotechnical recommendations to infill the existing man-excavated pond. Refer to Appendix B for the Memorandum and Bathymetric Survey, prepared by Paterson.

3.5 Stormwater Drainage Criteria

3.5.1 General

A thorough review of relevant background reports was completed as part of the 2016 ECR supporting the BSUEA MSS. A synopsis summarizing that review was included in the BSUEA MSS (2018).

The stormwater management design target developed as part of the BSUEA MSS considered all relevant reports to the Study Area, the Existing Conditions Reports prepared by Minto's Consultant Team as well as the Ottawa Sewer Design Guidelines and MECP's Stormwater

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Management and Planning Manual (2003). Similarly, Quinn's Pointe Phases 6–8 is also bound by the recommendations of the Existing Conditions Reports prepared by all Minto's sub-consultants and the following background reports:

- The Jock River Reach 2 Subwatershed Study (2009)
- Corrigan Stormwater Management Facility and Design Brief (2010).
- The Lower Rideau River Subwatershed Report (2012)
- The Mud Creek Subwatershed Study (2015)
- The Barrhaven South Urban Expansion Area Master Servicing Study (BSUEA MSS) (2018).

Given that the BSUEA MSS (2018) was prepared while accounting for the recommendations from the other four (4) studies, Quinn's Pointe Phases 6–8 should also meet the same stormwater management criteria and design objectives along with the requirements of the latest Sewer Design Guidelines (December 2025).

The following sub-sections summarize the stormwater management targets as follows: Section 3.5.2.2 provides details on water balance, Section 3.5.2.3 on water quality, Section 3.5.2.4 on erosion while Section 3.5.2.5 on water quantity. Similarly, minor and major systems design criteria, is described in Section 3.5.3 and Section 3.5.4, respectively.

3.5.2 Stormwater Management Design Targets

3.5.2.1 General

The stormwater management design targets established as part of the 2018 BSUEA MSS were extended to Quinn's Pointe Phases 6–8 in accordance with the MSS Addendum and this AAPS. Those targets were combined with those included in the SDG (December 2025) and the city's CLI-ECA (particularly those under Appendix A), in establishing the overall design criteria for Quinn's Pointe Phases 6–8.

3.5.2.2 Water Balance

A water balance analysis was carried out as part of the BSUEA MSS (2018) which assessed the pre- and post-development infiltration regime while accounting for the proposed infiltration measures. As part of the 2018 BSUEA MSS, a statistical analysis was completed which demonstrated that infiltration measures along local roads, sized to capture and infiltrate the 22 mm event, was sufficient to maintain the pre-development infiltration condition when supplemented with rooftop infiltration within private blocks. This strategy was also found to meet the 80% TSS removal.

The Existing Conditions Report (JLR, March 2025) included the pre-development water balance for Quinn's Pointe Phases 6–8, which is summarized in Section 3.6 of this AAPS. Considering Table A2 of the CLI-ECA, the following constraints will prevent the use of the EES:

- ✓ Areas of high groundwater levels - Constraint (b)
- ✓ High Risk Activities – Constraint (e)
- ✓ Source protection Restrictions – Constraint (f)

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As a result, the implementation of the EES along local roads, to the maximum extent in areas not impacted by the above-noted constraints, is to be targeted to capture/infiltrate the 22 mm event which is to be supplemented by rooftop infiltration using sub-surface systems within both vacant Blocks of industrial land usage.

3.5.2.3 Water Quality

The constraints identified in Section 3.5.2.2 limit the feasibility of implementing infiltration measures throughout portions of Quinn's Pointe Phases 6–8. As a result, these constrained areas will require servicing through Low Impact Development (LID) filtration measures, implemented to the maximum practical extent.

The LID filtration systems are to be designed to manage the storm event defined through the water balance analysis originally completed as part of the 2018 BSUEA MSS and subsequently updated in the MSS Addendum and this AAPS. A design capture depth of 22 mm has been adopted as the target for both infiltration and filtration-based LID measures.

In addition, the proposed filtration approach is required to achieve a minimum of 80% Total Suspended Solids (TSS) removal, consistent with the water quality objectives for the Jock River watershed. This requirement may be achieved through a single facility or through a treatment train approach combining multiple measures. In accordance with CLI-ECA requirements, the filtration solution is to be designed to meet the 80% TSS removal objective.

3.5.2.4 Erosion Control

Potential erosion associated with off-site discharges must be evaluated for each receiving system, particularly where discharge occurs outside of enclosed storm sewer systems. Areas serviced by EES will inherently mitigate erosion concerns, as frequent, lower-magnitude events are captured and infiltrated on-site.

For areas serviced by the proposed LID filtration-based system, peak flows will be attenuated through detention and controlled release to the downstream dry pond, thereby reducing erosion risks. Erosion considerations at key discharge points are summarized as follows:

- Discharge to the existing Quinn's Pointe storm sewer system will not require additional mitigation measures, as flows will be limited to the allowable rates established under the original subdivision design, thereby maintaining system integrity.
- Discharge to roadside ditch systems along Borrisokane Road and Barnsdale Road must be evaluated based on changes in both peak flows and runoff volumes. Where post-development conditions demonstrate reductions relative to existing conditions—particularly for the 4-hour 25 mm and 2-year storm events—erosion risks are not expected to increase (refer to Section 3.12.4).

3.5.2.5 Water Quantity

Both minor and major drainage systems are to be designed in accordance with the criteria outlined in the following sections, including Sections 3.5.3 and 3.5.4. As this AAPS and MSS Addendum were prepared at the functional design level, assessments have been carried out to evaluate

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system performance under storm events up to the 100-year return period, including a climate change scenario corresponding to the 100-year event plus 20%, consistent with Sewer Design Guidelines (SDG 004).

From a flood control perspective, water quantity management measures will be implemented to restrict post-development peak flows to levels that do not exceed the capacity of the receiving infrastructure or pre-development flow conditions, whichever is more restrictive.

3.5.3 Minor System

Storm sewers servicing Quinn's Pointe Phases 6–8 are to be designed in accordance with the SDG as summarized below:

Design Capture and Level of Service

- Minimum 1:2-year and 1:5-year capture for local and collector roads, respectively.
- 1:5-year for both vacant blocks developed in the future with an assumed industrial land usage.
- 1:10-year capture for arterial roads, transitway or any depressed driveways, where applicable.

Design Flows

- Initial sizing of storm sewers via the Rational Method design sheets with final sizing to be confirmed via a hydraulic grade line (HGL) analysis.
- Rainfall intensity-duration-frequency (IDF) statistics as per SDG.
- Time of concentration (Tc) calculated based on an inlet time of 10 minutes and 15 minutes for the industrial lands.
- Runoff coefficients (Rc) to reflect proposed development. The Rc for Quinn's Pointe Phases 6–8 was based on recently approved developments in Quinn's Pointe and consistent with the SDG.

Design Criteria

Design Criteria as per the SDG:

- Minimum velocity 0.80 m/s
- Maximum velocity 6.0 m/s
- Manning roughness coefficient 0.013
- Minimum allowable slopes See Table 6.1 of the OSDG
- Minimum depth of cover 2.0 m
- Minor losses at manholes set in accordance with Appendix 6B of the OSDG
- Inlet control rate to be achieved with inlet control devices (ICDs) with minimum capture of 1:2 year for locals, 1:5 year for collectors and 1:10 year for arterials.

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3.5.4 Major System

Storm servicing was developed following the dual drainage concept with the minor system sized based on the criteria set out in Section 3.5.3, while the major system is configured based on the criteria outlined below and evaluated in accordance with the SDG.

Road Types and Allowable Flow Depths

- Local: 350 mm at edge of pavement
- Collector: 250 mm at edge of pavement
- Urban Arterial: No barrier curb overtopping; arterial should be designed to leave one lane free of water in each direction during the 1:100-year design storm.
- In the absence of barrier curbs, flow shall not encroach into adjacent private property.

Maximum Flow Velocity on Streets

The product of velocity (m/s) and depth (m) of overland flow on streets shall not exceed 0.6 during a 1:100-year peak flow.

Major System Flow Outlets

Major system flow should be directed towards either a watercourse/ditch or to a SWMF.

3.5.5 End-of-Pipe Facility

Any end-of-pipe control measures, either dry ponds, wet ponds, or filtration dry ponds recommended in this Addendum should be conceptually sized in accordance the SDG and MOE's publication entitled "SWM Planning and Design Manual, 2003" and other relevant source of data such as the Sustainable Technologies Evaluation Program (STEP).

Given that the 2008 BSUEA MSS recommended two (2) dry ponds due to significant presence of EES which results in the capture and infiltration of frequent storm events, this service option will also be extended to Quinn's Pointe Phases 6–8 with the addition of a filtration-based LID solution in accordance with the CLI-ECA.

3.6 Existing Conditions Water Budget

The water budget of a given area is often studied by hydrologists and hydrogeologists to better understand the movement of water through its various stages. Natural watershed systems maintain a balance between precipitation, runoff to water bodies, infiltration to the groundwater system, evaporation from water surfaces or transpiration from vegetation, completing the natural cycle back into atmospheric moisture and precipitation. This type of analysis establishes the baseline condition and would be used to assess any future changes in infiltration to subsurface water-bearing zones. It is also used to predict changes to the hydrological cycle that will result from urbanization with LID solutions.

A continuous hydrological model is often used to understand the water budget for a given area. As described in the ECR (refer to Appendix B of the MSS Addendum), the PCSWMM software platform was also used to assess the water budget as part of the BSUEA ECR and MSS

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completed in 2017 and 2018, respectively by JLR. Since the approach and methodology was adopted by both the City and RVCA, the PCSWMM platform and modelling approach was once again utilized for this analysis.

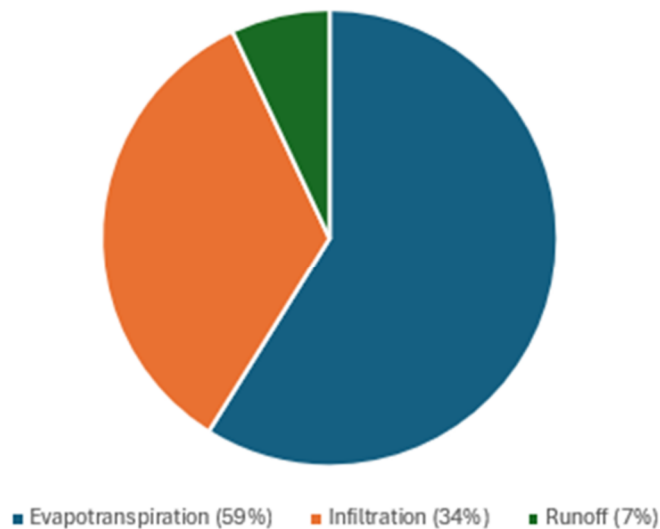
The water balance assessment completed for Quinn's Pointe Phases 6–8 was based on PCSWMM's simplified groundwater and snowmelt modules that allow continuous simulation of the water budget, including the elements of evapotranspiration, the groundwater table (lower saturated zone), snowfall and snowmelt.

Section 6.2 of the 2025 ECR summarized the continuous simulation results of the water budget. Specifically, Table 6-4 of the ECR (refer to Appendix B of the MSS Addendum) shows each component of the water budget which accounts for the annual precipitation depth of 864 mm. The components of the water budget were reported as followed (refer to the 2025 ECR, MSS Addendum, and AAPS for PCSWMM water budget modelling files):

- Evapotranspiration accounts for approximately 59% (513 mm).
- Infiltration accounts for approximately 34% (297 mm).
- Surface runoff accounts for 7% (61 mm).

Figure 5 below summarizes graphically the various components of the existing condition water budget while a more detailed breakdown is provided below as reported in the 2025 ECR.

Figure 5: Existing Conditions Water Budget



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Table 1: Water Budget Continuous Simulation Results

Annual Water Budget Component	Annual Average Depth to Jock River (mm)	Annual Average Depth to Mud Creek (mm)	Annual Average Depth to Rideau River (mm)	Area Weighted Total (mm)	Budget %
Precipitation	864	864	864	864	100%
Evapotranspiration	509	531	509	513	59%
Infiltration	296	314	286	297	34%
Surface Runoff	83	60	0	61	7%

It should be noted that the Rideau River does not receive surface runoff directly from Quinn's Pointe Phases 6–8. However, infiltrated runoff does contribute to the groundwater catchment of the Rideau River. This is reflected in the results in the above Table.

Based on the above summary Table 1, infiltration under existing condition accounts for approximately 34% of the total annual precipitation of 864 mm when the three (3) receivers are combined, while surface runoff accounts for approximately 7% on a yearly average.

3.7 EES Monitoring (On-Going)

3.7.1 Monitored Rainfall Events

Monitoring of the EES was made a condition of approval within the Quinn's Pointe subdivision to gather data and confirm the suitability of the design parameters adopted during the preliminary design of the EES as part of the 2017 BSUEA. To facilitate monitoring, the city advised that it should be completed using the SmartLevel™ technology supported by Link Utility Technology as it has been tested and adopted by the City's Operation Group. In parallel, the monitoring of rainfall events was carried out using the SmartRain™ system to collect real-time precipitation data. The objective of the monitoring was as follows:

- Determine the spill occurrences and identify under which rainfall events results in spills into the conventional storm sewer system occur; and
- Determine the drawdown time and identify the time it takes for the drawdown of the EES following a rainfall event.

Three (3) monitoring locations have been set up within Quinn's Pointe Stages 2 and 3. The locations were established to cover various sizes of serviced areas; a small area at MHST82176 (2.6 ha), a mid-size area at MHST82182 (7.6 ha) and a larger serviced area at MHST83698 (20.5 ha).

Level monitoring was initiated once a minimum occupancy of 80% of the serviced area was achieved. Level monitoring was initiated in June 2022 for one of the stations while monitoring for the other two (2) stations was initiated in November 2022. Given the 22 mm capture, the review of the monitored data mainly focused on events of 22 mm or greater magnitude.

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Of the 235 monitored events, twenty-seven (27) had total rainfall depths greater than 22 mm. The events greater than 22 mm depth are listed in the Table 2 below in descending order of total rainfall depth.

Table 2: Recorded Rainfall Events (>22 mm and 0°C)

Date	Event Start	Event Duration (hrs)	Peak Intensity (mm/hr)	Total Event Rainfall Depth (mm)
August 7, 2022	2:00 PM	46	19.81	92.07
April 29, 2023	9:00 AM	122	5.33	76.83
June 6, 2024	3:00 AM	60	16.51	61.15
August 10, 2023	12:00 PM	11	28.96	58.2
August 7, 2023	8:00 AM	28	14.73	54.28
August 8, 2024	7:00 PM	22	7.62	48.63
October 30, 2025	10:00 AM	37	4.32	47.07
September 18, 2022	10:00 AM	50	12.95	38.65
August 17, 2024	1:00 AM	63	8.64	38.21
June 21, 2024	9:00 AM	74	9.91	37.88
July 9, 2024	7:00 PM	49	8.38	37.1
May 27, 2024	9:00 AM	33	12.19	33.43
April 11, 2024	8:00 AM	57	4.32	31.8
October 7, 2025	10:00 AM	16	9.4	30.9
July 24, 2024	8:00 AM	13	9.4	30.2
October 6, 2023	10:00 AM	33	12.95	29.64
November 11, 2022	3:00 PM	14	5.84	29.12
July 15, 2023	9:00 PM	12	14.99	28.33
July 11, 2022	7:00 PM	24	17.78	28.27
August 28, 2025	3:00 AM	40	11.43	27.26
November 20, 2024	11:00 PM	47	2.54	27.14
July 21, 2023	12:00 AM	21	8.64	25.19
June 26, 2023	1:00 AM	22	15.24	24.98
July 18, 2022	7:00 AM	15	7.62	24.78
May 21, 2024	6:00 AM	15	14.73	24.33
July 24, 2022	12:00 PM	14	8.89	23.26

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Date	Event Start	Event Duration (hrs)	Peak Intensity (mm/hr)	Total Event Rainfall Depth (mm)
April 18, 2025	4:00 PM	18	9.65	22.26

Based on the above, the monitored events spanned, in general, between 22 mm to 92 mm with most of the events ranging between the 25 mm - 40 mm. The most significant event of 92 mm occurred over 46 hours and its 12-hour intensity was established to be comparable to a 1:10-year event when compared to the Ottawa IDF statistics.

3.7.2 EES Performance

Ten (10) out of the 27 events have resulted in flow conveyed by the traditional storm sewer, referred to as a spill event. These events are considered to have occurred when the depth recorded in the manhole is above the invert elevation of the conventional sewer. Monitoring details along the three (3) monitoring stations follows (refer to Table 3 below):

- MH309: A single spill occurred on August 7, 2022, which was triggered by a 92.07 mm event.
- MH327: No spills were recorded.
- MH109: Nine (9) spill events were monitored – MH located at the intersection of Edenderry Way and Oranmore Lane.

Table 3: Spill Event Summary

Event	Location	Rainfall up to Spill Event (mm)	Duration of Spill Event (hrs)	Total Event Rainfall Depth (mm)	Drawdown Duration
August 7, 2022	MH309	22.48	5mins	92.07	2hrs 56mins
April 29, 2023	MH109	37.3	2hrs 50mins	76.83	47hrs 12mins
August 7, 2023	MH109	39.13	5hrs 41mins	54.28	56hrs 18mins
August 10, 2023	MH109	12.09	5hrs 5mins	58.2	45hrs 41mins
May 27, 2024	MH109	24.21	4hrs 16mins	33.43	231hrs 15mins
June 6, 2024	MH109	41.33	2hrs 45mins	61.15	16hrs 40mins
June 26, 2024	MH109	17.12	2hrs 25mins	17.12	55hrs
July 9, 2024	MH109	34.83	2hrs 25 mins	37.1	10hrs 10 mins
March 16, 2025	MH109	9.39	3hrs 23 mins	12.88	40hrs 50 mins
October 7, 2025	MH109	21.47	4hrs 46 mins	30.9	23hrs 47 mins

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As shown above, the performance of the EES at both MH327 and MH309 by far exceed the design basis of the system, being the capture of a 22 mm rainfall event. The only recorded spill at MH309 was triggered by an event that exceeded 22 mm.

All the spills, apart from one event at MH309, occurred at MH109. The spills were triggered by storms exceeding 22 mm, except for three (3) events with rainfall of ± 12 mm (August 10, 2023), ± 17 mm (June 26, 2024) and ± 9.4 mm (March 16, 2025). Details on these events follow:

- The August 10, 2023, event (58.2 mm) occurred less than 48 hours after the end of the August 7, 2023, rainfall event which had totaled 54.28 mm. On August 10, 2023, the slow drawdown continued, indicating elevated groundwater at that station.
- The spill events that occurred on June 26, 2024 (9:45 PM), was found to occur immediately after three (3) consecutive storms occurring between June 21st, 2024, to June 25th, 2024, totalling 55.24 mm.
- The March 16, 2025, event was during high groundwater reflecting typical spring freshet.

Based on the monitoring completed from 2022-2025, the performance of the EES in general exceeds the design capture of the system at 22 mm. However, one of the three (3) monitoring locations (MH109), is subject to the most spills as shown in the above table. It should be noted that at that location, a separation of less than 1 m (actual separation was 0.28 m) exists between the recorded seasonally high groundwater table and the bottom of the granular envelope which explains why spills at that location are encountered more frequently.

3.7.3 Winter Performance

The annual EES monitoring Reports included graphical results showing monitoring results throughout the year including winter months, spanning from December until end of February. Although there are fewer precipitation events during this window, the data indicates that runoff from rain-on-snowmelt events, when applicable, is transmitted to the EES and infiltrated (refer to Annual EES Monitoring Reports), assuming that CB grates are free of ice. The Table 4 below summarizes a few winter events captured by the EES which were infiltrated.

Table 4: Winter Events Captured by EES

Date	Event Start	Event Duration (hrs)	Peak Intensity (mm/hr)	Total Event Rainfall Depth (mm)
January 9, 2024	5:00 AM	29	6.35	29.3
January 4, 2023	2:00 PM	9	4.32	13.51
December 16, 2024	8:50 PM	13	2.79	8.83

3.7.3.1 Drawdown Time During Winter Months

Based on the review of the monitoring data, drawdown of the EES—down to the invert of the inlet—continues to occur during winter months either within the duration of individual rainfall events or shortly thereafter, as illustrated in Figures 6, 7 and 8 below.

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Winter precipitation events are typically lower in intensity and depth, as expected for this time of year. Nevertheless, the data presented (refer to the accompanying table and figures) demonstrate that infiltration through the EES remains active during winter conditions. Although the observed average infiltration rate of approximately 18 mm/hr is slightly below the design rates adopted in the MSS modelling, the infrequent nature of winter events limits any material impact on the overall design performance objectives.

Design infiltration rates ranged from 19.96 mm/hr to 28.42 mm/hr depending on location. These values are generally consistent with, or slightly exceed, the rates applied in the EES modelling once the 2.5 safety factor was incorporated. Accordingly, simulated infiltration rates remain representative of expected winter performance. While calculated field rates are marginally lower than the design assumptions, the reduced frequency and magnitude of precipitation during winter result in fewer successive events and a diminished reliance on exfiltration capacity.

Table 5: Winter Drawdown Rate Calculations (MH309)

Date	Rainfall Event depth up to Dry Period (mm)	Start Recorded Level (mm)	End Recorded Level (mm)	Dry Period Duration (hrs)	Total Event Rainfall Depth (mm/hr)
January 10, 2024, 11:00am	29.3	573	400	6.92	25
December 17, 2024, 10:10am	8.83	511	400	12.33	9
January 4, 2023, 11:00pm	13.51	585	400	9.3	20
Average Rate					18

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Figure 6: Drawdown Rate Chart (January 2023 Event)

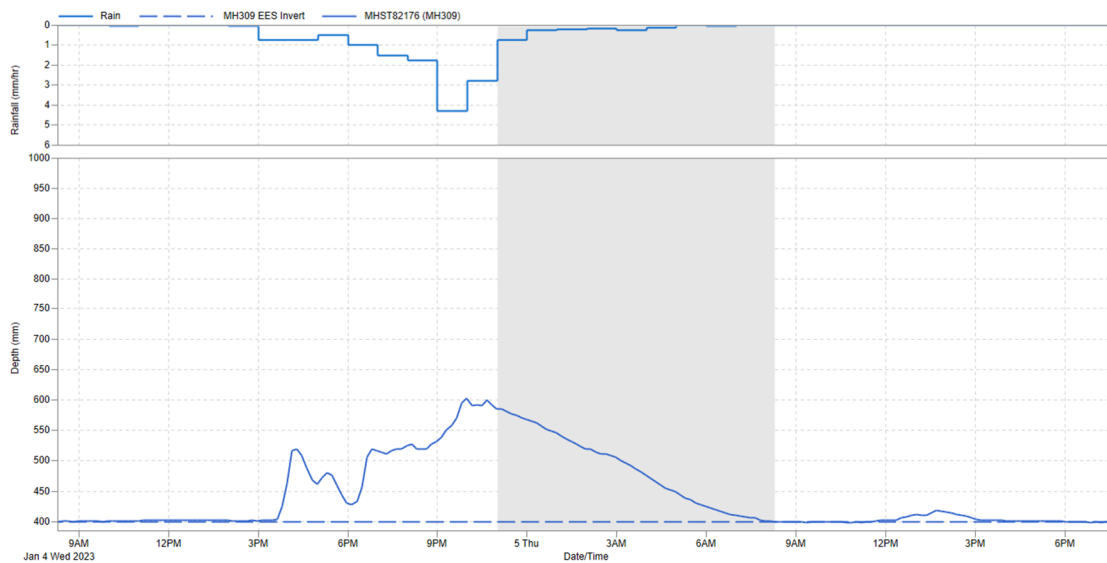
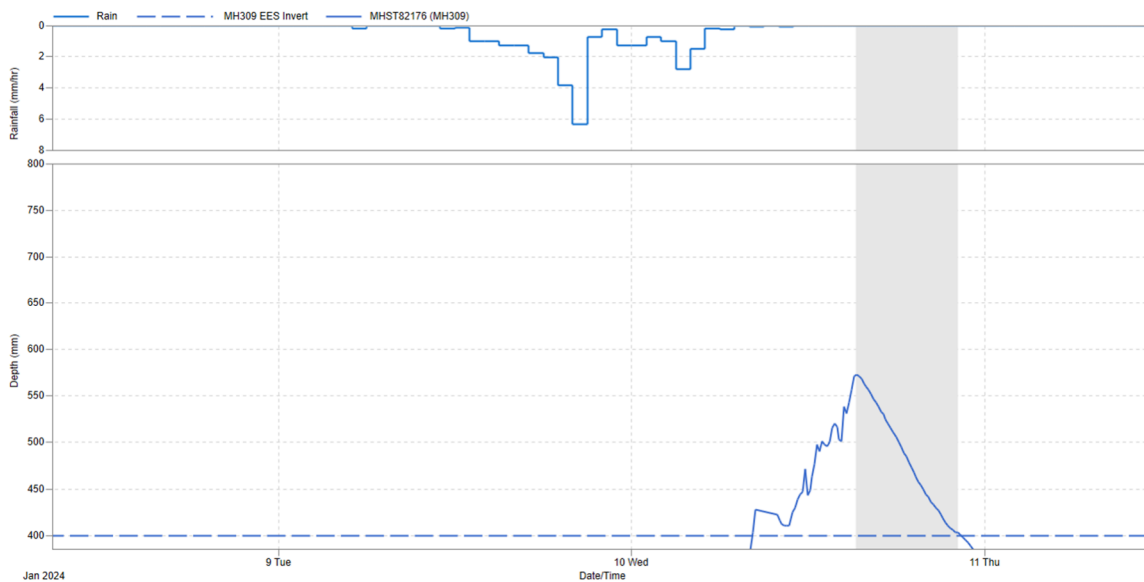


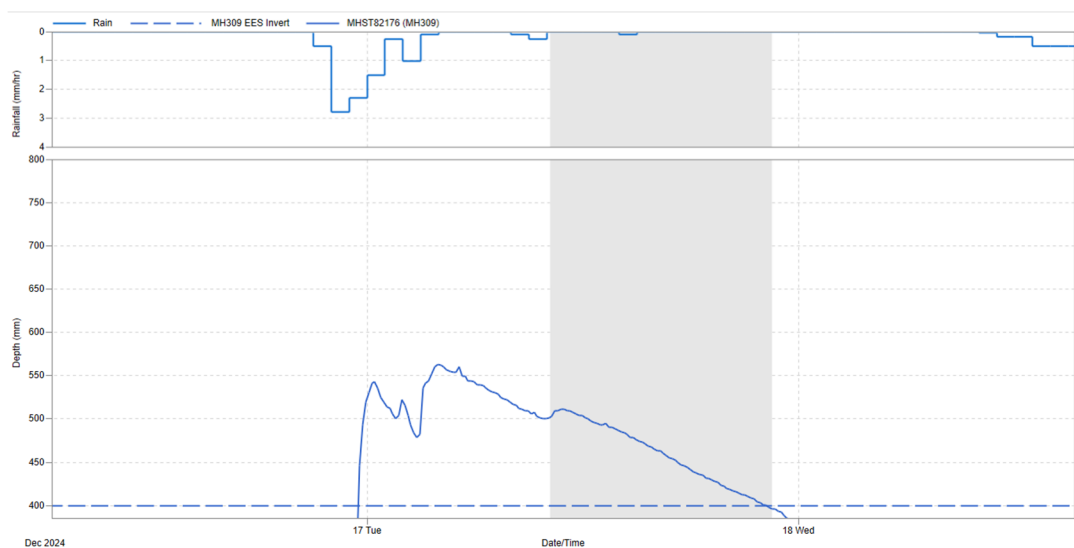
Figure 7: Drawdown Rate Chart (January 2024 Event)



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Figure 8: Drawdown Rate Chart (December 2024 Event)



3.7.4 Operation and Maintenance Activities - EES

Operation and Maintenance (O&M) activities are essential to ensure the long-term functionality and performance of a newly installed EES. Once ownership is transferred, the EES becomes part of the city's wastewater collection system. Similarly, the ownership of the proposed EES in Quinn's Pointe Phases 6–8 will be transitioned from the developer to the city.

An initial version of a Draft OMM was included in the MSS Addendum. Based on comments received via email, a second draft version of a generic Operation and Maintenance Manual (OMM) was prepared and included in Appendix I. This Manual is intended in assisting the city with typical procedures to reduce the risk of blockages and other events which could trigger costly emergency interventions and would extend the system's service life (refer to Appendix I).

It should also be noted that an OMM has been prepared specifically for existing Stages 2-4 and will be issued to the city under separate cover. However, it is intended that this generic second Draft version of the OMM be included as part of detailed design of both Stage 5 and Stage 6 and as part of the upcoming 2026 EES Monitoring Report (2025 data) and shared to the group which attended the June 17, 2025, meeting that discussed EES monitoring.

3.8 Consolidated Linear Infrastructure Environmental Compliance Approval

In 2022, the Ministry of the Environment, Conservation and Parks (MECP) launched a new approval process with the goal of modernizing the approach to authorize construction of linear infrastructure and stormwater management facilities. The intent of the framework was to reduce efforts on both the Ministry and with Municipalities by setting up a consistent set of conditions to improve environmental protection. By doing so, the old system which consisted of the issuance of individual Environmental Compliance Approval (ECA) would be replaced by a system that relies on pre-authorization of works with conditions. The new system adopted by MECP is referred to as the Consolidated Linear Infrastructure Environmental Compliance Approval (CLI-ECA).

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On December 10, 2025, and January 27, 2026, the City of Ottawa received their CLI-ECA for storm and sanitary, respectively. The transition period for the storm CLI-ECA is identified as December 26, 2026. However, the transition does not apply to Quinn's Pointe Phases 6–8 as the infrastructure is proposed and will be constructed in 2027. The said infrastructure will need to be designed in accordance with MECP's document entitled "Design Criteria for Sanitary Sewers, Storm Sewers and Forcemains for Alterations Authorized under an Environmental Compliance Approval (May 31, 2023)".

3.9 Evaluation of Servicing Strategies

Servicing strategies were evaluated as part of a screening process that was documented in great details in the MSS Addendum. Infiltration strategies were reviewed in Section 3.10 while those for filtration as part of Section 3.11. Those sections from the MSS Addendum are to be reviewed in conjunction with this AAPS.

A stormwater management strategy has been developed on a functional level basis based on the following constraints/targets:

- Residual capacities of existing storm outlet sewers.
- Off-site post-development peak flow and runoff volume discharging to MTO's Borrisokane Road open ditch system will be limited to those of the pre-development condition.
- Off-site post-development peak flow and runoff volume discharging to the Barnsdale Road open ditch system will be limited to those of the pre-development condition.
- Off-site post-development peak flow and runoff volume discharging to the Thomas Baxter Municipal Drain, when the above contributions are accounted and will be limited to those of the pre-development condition.
- Runoff volume control target hierarchy, as presented in the screening process of the CLI-ECA and documented in the MSS Addendum as follows.
 - Hierarchy Priority No. 1: LID Infiltration/retention
 - Hierarchy Priority No. 2: LID Filtration
 - Hierarchy Priority No. 3: Conventional treatment

Based on the infiltration LID Evaluation Impact Table (refer to Table 6 of the MSS Addendum), the following conclusions were drawn:

- An EES system is to be implemented to capture and infiltrate the 22 mm event in Areas 1A & 1B (Figure 9) when separation of 1 m exists between the bottom of granular and seasonal high groundwater table.
- Within the Industrial lands (Areas 3A and 3B – Figure 9), infiltration of rooftop flows is to be implemented using a sub-surface system to capture and infiltrate the 22 mm event.
- Filtration LID solutions to be developed to the maximum extent for Areas 1C & 2 (Figure 9).

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- Given that Areas 3A and 3B (Figure 9) when developed, will include on-site stormwater management controls which will meet the applicable quality control criterion at the time of approval, filtration-type solution will not be accounted for these areas.

3.10 Recommended Servicing Solution

As per the screening process for infiltration measures, Table 6 of the MSS Addendum summarized the measures within Quinn's Pointe Phases 6–8. Similarly, several filtration measures were evaluated in the MSS Addendum and summarized in the Evaluation Matrix (Table 7 of the MSS Addendum). Both Table 6 and Table 7 as well as Section 3.10.3.2 and Section 3.11.2 from the MSS Addendum should be reviewed in conjunction with this AAPS.

As shown in Table 7 of the MSS Addendum, Alternative 5 was found to score the highest at 36. Consequently, this Option was recommended as the preferred filtration LID Solution to supplement the EES along local roads shown in Figure 9. Functional level information is provided below in this AAPS under Appendix C while Drawings are included in Appendix H.

The preferred strategy, Alternative 5, was developed based on a treatment train approach where an upstream MTD has been conceptually sized for pre-treatment followed by a filtration-based dry pond. Both MTD and filtration dry pond was sized to capture, pre-treat and provide filtration of the 22 event prior in being released into the adjacent dry pond. The recommended solution identified in the MSS Addendum consisted of the following components:

- Defender Model FD-8HC (ETV particle size) as pre-treatment
- Flow-splitter manhole forcing the 22 mm to the filtration dry pond while flows in excess, would by-pass the filtration dry pond and would be conveyed to the downstream dry pond.

The simulated filtration rate of the facility was assumed to be consistent with the measured unfactored infiltration rates reported by Paterson in their Existing Conditions Report at BH2-22, the closest borehole, as amended soil performance would be similar. An average infiltration rate of 126 mm/hr was, therefore, calculated based on two (2) measurements of unfactored infiltration rates at BH2-22.

Similar to the EES, a safety factor of 2.5 was then applied to the average unfactored infiltration rate of 126 /mm/hr for an overall effective assumed filtration rate of 50.4 mm/hr. The assumed filtration rate of 50.4 mm/hr is consistent with the amended soil filtration rate of 50 mm/hr recommended by Sustainable Technology Evaluation Program (STEP).

Configuration of Filtration-Based Dry Pond consisting of:

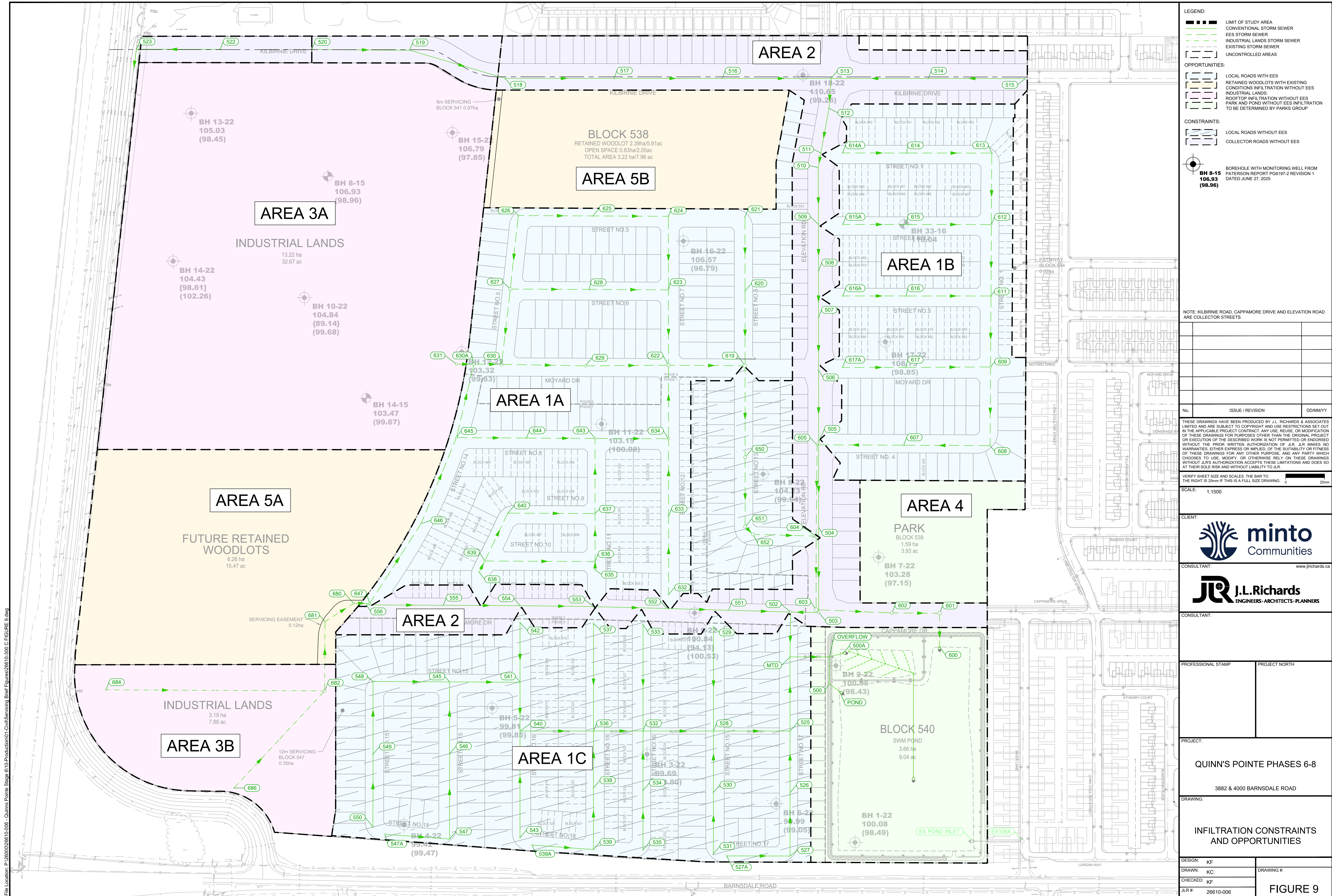
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|---|----------------------|
| • Operating Levels: | 100.10m – 100.80 m |
| • 3:1 side slope | |
| • Filtration rate: | 50.4 mm/hr |
| • Maximum Ponding Depth prior to Spill: | 700 mm |
| • Spillway (15 m wide): | 100.80 m |
| • Footprint at elevation 100.10 m: | 1,822 m ² |
| • Surface Area at Spill of 100.80 m: | 2,235 m ² |
| • Volume of Filtration Dry Pond: | 1,418 m ³ |

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- Hydro-seeded footprint & side slopes
- Amended soil (bio-retention soil media) 500-700 mm filter media
- Choker Layer 100 mm
- Clear stone 300-350 mm
- subdrains (spurs & header) 150 mm and 200 mm
- Inspection/clean-out ports

The proposed servicing within Block 540 along with pre-treatment is shown on Drawing CMST (Appendix H) with information included in Appendix C. The above-noted configuration of the filtration pond as well as the filtration rate will be reviewed at the Stage 6 detailed design.

File Location: P:\26610\006 - Quinns Pointe Stage 010-Production\01-Civil\Servicing Brief Figures\26610-006 C FIGURE 9.dwg



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3.10.1 Water Quality Control Assessment - Infiltration/Filtration Practices

3.10.1.1 General

An 80% TSS removal has been historically used for any development tributary to the Jock River, Rideau River and/or Thomas Baxter Municipal Drain (Mud Creek). Storage volume requirements for different types of SWM practices to achieve an 80% TSS removal are identified in Table 3.2 of MECP's publication entitled "Stormwater Management Planning and Design Manual, March 2003".

Quality control requirements for municipal works have since evolved with the implementation of MECP's framework which is based on pre-authorized works defined in the CLI-ECA, particularly those described in Appendix A.

Section 3.11.5 of the MSS Addendum identified the preferred filtration alternative as Alternative 5. It consists of a pre-treatment MTD combined to a filtration dry pond. Given that this Addendum extended the analyses and conclusions from the 2018 BSUEA MSS, the design capture of the 22 mm event was maintained. Hence, the capture of the 22 mm event was extended to the sizing of the filtration solution given that it also relies on infiltration/filtration through a bio-retention media.

As a parallel verification, the proposed infiltration and filtration LIDs is being reviewed against Table 3.2 of the SWMPDM to ensure that it meets its recommendations. Sub-sections 3.10.1.2 and 3.10.1.3 summarize the verification against Table 3.2.

3.10.1.2 Infiltration Practices

An EES is proposed within two (2) residential blocks located north of Cappamore Drive, and east and west of Elevation Road. These Blocks have been identified as Area 1A and Area 1B on Figure 9.

Based on the average imperviousness of $\pm 64\%$ for each residential areas serviced by an EES, a water quality storage of $\pm 33 \text{ m}^3/\text{ha}$ was calculated to be necessary to achieve 80% TSS removal based on interpolation between imperviousness of 55% ($25 \text{ m}^3/\text{ha}$) and 70% ($35 \text{ m}^3/\text{ha}$) for an infiltration SWM Practice. Based on an EES along local roads sized for the 22 mm event (per 2018 BSUEA MSS), the storage volume provided is equivalent, on average, to $\pm 150 \text{ m}^3/\text{ha}$. Therefore, the proposed EES along local roads ($\pm 150 \text{ m}^3/\text{ha}$) by far exceeds the infiltration requirements of $33 \text{ m}^3/\text{ha}$, the enhanced level of protection per Table 3.2 of the SWMPDM. Consequently, no further downstream water quality controls are necessary for Areas 1A and 1B apart from the EES as it meets Table 3.2 and the CLI-ECA.

3.10.1.3 Filtration-Type Practices

The screening process presented in the MSS Addendum concluded that the preferred filtration LID solution consisted of Alternative 5. It includes a pre-treatment MTD with 37% TSS removal (ETV particle size) combined to a filtration dry pond.

The filtration dry pond was sized to capture and filtrate the 22 mm event using a bio-retention soil media with an underlain sub-drain system and clear stone envelope. Although the effluent is

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not infiltrated, the filtration mechanism proposed remains equivalent to infiltration; however, the filtered runoff will be conveyed in the dry pond portion of Block 540 for quantity control.

The filtration dry pond would service Area 1C (high groundwater table) as well as Area 2 consisting of the three (3) collector roads due to roadway salts. When both Area 1C and Area 2 are combined, an overall area of ± 20.4 ha and average imperviousness of $\pm 67\%$ was calculated. Given that the proposed filtration dry pond operates as an infiltration SWM practice with an underlain sub-drain system, a storage volume of $34 \text{ m}^3/\text{ha}$ was estimated by means of interpolation between imperviousness of 55% ($25 \text{ m}^3/\text{ha}$) and 70% ($35 \text{ m}^3/\text{ha}$) for this SWM practice. Based on an area of ± 20.4 ha and the storage volume of $34 \text{ m}^3/\text{ha}$, the filtration dry pond should be sized with $\pm 700 \text{ m}^3$. As shown in Section 3.11.5 of the MSS Addendum, the volume provided is $1,629 \text{ m}^3$ thereby exceeding the volume of Table 3.2. Consequently, the filtration dry pond is found to meet both Table 3.2 and CLI-ECA (Appendix A).

The proposed servicing of Block 540 along with pre-treatment is shown on Drawings CS1 and CMST as well as on Drawings POND and D2 in Appendix H.

3.11 Storm and Stormwater Servicing Assumptions and Methodology

The design and evaluation of the proposed storm servicing strategy was undertaken with the PCSWMM platform. Several design parameters and assumptions have been made to support both minor and major system designs as described in the sub-sections that follow.

3.11.1 Runoff Coefficient

Surface runoff is generated when precipitation exceeds the infiltration capacity of the landscaped surface and/or the initial abstraction of hard surfaces. To assess runoff rates from the various surfaces, hydrological parameters representing different drainage characteristics were calculated. The type of land cover and associated hydrological parameters are based on area weighting of each catchment's imperviousness using the various breakdown shown in Table 6.

Table 6: Land Cover Drainage Parameters

Land Cover	Impervious (%)	Runoff Coefficient	Draining to Pervious (%)
Collector Roads	71	0.70	0
Local Roads	64	0.65	0
Parks	36	0.45	100
Woodlots	0	0.20	100
Single Family Residential	64	0.65	70
Townhouse Residential	64	0.70	70
Back-to-Back Residential	71	0.70	70
Industrial Lands	86	0.8	0

Based on the guidance of the SDG, Runoff Coefficients (C-Factors) associated with the various types of residential usages were based on actual impervious areas for the adjoining Quinn's

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Pointe subdivision. Similar housing product will be proposed as part of Quinn's Pointe Phases 6–8.

3.11.2 Storm Sewer Sizing

Preliminary sizing of storm sewers is included in Appendix D and was carried out based on the design criteria listed in Section 3.5.3 for the proposed road patterns shown on the Concept Plan. The storm servicing strategy was also developed while respecting the filtration and infiltration servicing solutions presented in Section 3.10 (infiltration) and filtration (Section 3.11.2). As such, the storm sewer strategy was developed with two (2) independent storm systems; one that collects and convey runoff from areas serviced by an EES and a second independent storm sewer system collecting runoff from collector roads and areas of high groundwater.

This functional level exercise is intended to demonstrate the feasibility of the overall storm servicing strategy. The preliminary storm sewer layout and associated drainage areas are shown in the following drawings (Appendix H):

- CS1: Conceptual Servicing
- CG1: Conceptual Grading
- CMST: Conceptual Storm

Based on the above Drawings, storm sewer design sheets have been prepared and included in Appendix G. The traditional storm sewers along the various road types have been sized in accordance with the level of service identified in Section 3.5.3, being 1:2 year and 1:5-year capture for local and collector roads, respectively.

3.11.3 Roadway Sag Storage

The Conceptual Grading Plan (Drawing CG1 in Appendix H) was developed to provide a route to safely convey major overland flow while minimizing earth works. To fulfill these objectives, grading along the major overland system will be carried out following a saw-tooth profile with low points introduced along the major overland system. Since detailed design will dictate the location and extent of the roadway sag depths, modelling was carried out assuming an on-site storage of 70 m³/ha which reflects the storage achieved in the Quinn's Pointe Subdivisions. Roadways with flatter slopes will generate greater storage capacity, while those that are steeper will have limited storage capacity.

The design of the major overland system will allow runoff to be stored in the roadway depressions prior to its capture by the catch basins and inlet control devices (ICD). During infrequent events, runoff exceeding the rate of capture of the minor system and roadway sag storage will cascade along the major overland system.

3.11.4 Dual Drainage Modelling – Assumption and Parameters

The PCSWMM software platform was used to simulate hydrology with the given land use and corresponding imperviousness shown in Table 6 as well as confirming the preliminary minor and major systems and the assessment of the post-development water budget by modelling the infiltration opportunities documented in Section 3.10.3 of the MSS Addendum.

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The software used combines the EPA SWMM engine within a powerful GIS environment. The PCSWMM dual drainage model will provide preliminary confirmation of the overland flow depths along the local/collector roads, feasibility of the proposed sewer sizing in terms of hydraulic grade line (HGL) elevations, confirm block sizing of Block 540, the assessment of the proposed infiltration measures and the off-site peak flow discharges. Model inputs and results are contained in Appendix E.

The dual drainage analysis uses a dynamic simulation to calculate the hydrology and hydraulics of the major and minor systems. The model has been developed based on the following:

- The Chicago and SCS design storms derived from the Ottawa Intensity-Duration-Frequency (IDF) statistics which were applied across catchments with characteristics area weighted imperviousness.
- Sag storage of 70 m³/ha applied as a storage node below the weir elevation which represents the overland spill for each residential sub-catchment.
- Both vacant Blocks have been modelled assuming an industrial usage with sufficient storage to self-contain their own runoff up to and including the 1:100-year storm in accordance with the SDG for the given land usage.
- A 1:2 year and 1:5-year design capture for the local and collector roads, respectively.
- Exit losses are applied at the downstream end of each section of sewer based on the SDG.
- Storm Sewers have a minimum cover of 1.3 meters below the proposed centre line grades shown on Drawing CG1. With this minimum freeboard to the HGL elevation, underside of footings (USFs) will meet the prescribed minimum freeboard of 300 mm.
- Several unfactored infiltration rates were used to represent the soil characteristics in areas where an EES is being contemplated. Paterson's unfactored infiltration rates were reduced by 2.5 to account for a safety factor. Simulated unfactored infiltration rates are presented in Section 3.12.1.1.
- The storage and filtration were simulated as a storage node along the minor system. This configuration within residential areas has proven to demonstrate the capture of the 22 mm storm and maintain pre-development infiltration conditions (refer to the BSUEA MSS, Appendix C).
- Roof areas in the Industrial Lands were assumed to reflect 30% of the overall area for the water budget purposes. The same assumption was used as part of the BSUEA MSS for any of the ICI Blocks. It was assumed that infiltration measures would be implemented to capture and infiltrate by means of sub-surface systems the 22 mm storm.

3.12 Simulation Results

Several types of simulations have been carried out to assess the performance of proposed servicing, including the water budget, sizing of linear infrastructure, major overland flow route, as well as confirming sizing of features in Block 540. The following sub-sections provides these assessments.

3.12.1 Post-Development Water Budget

The existing condition water budget revealed that existing infiltration across the overall Quinn's Pointe Phases 6–8 accounted for 34% of the total annual precipitation based on long-term simulations (refer to Section 3.6 and ECR in Appendix B of the MSS Addendum).

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A similar analysis was carried out to assess the post-development water budget while accounting for constraints within Quinn's Pointe Phases 6–8 as well as the opportunities within Quinn's Pointe Phases 6–8 to incorporate an EES. As noted under Section 3.10.3 of the MSS Addendum, all EES being proposed provide the minimum 1 m separation between the seasonal high groundwater and the bottom of granular (refer to Figure 9A in Appendix D for figure displaying the separation between EES and groundwater). All EES lengths have been accounted for as part of the water budget and during detailed design best efforts will be made to utilize all EES lengths for the purpose of achieving the pre-development water budget. This is achieved by placing the plugs for the first reach at the downstream end. The sub-sections that follow provides the approach and assumptions made as part of the post-development water budget.

3.12.1.1 Simulated Infiltration Rates

The Existing Conditions Hydrogeological Report prepared by Paterson documented the hydrogeological condition within Quinn's Pointe Phases 6–8 (refer to Section 2.5.3). As part of this undertaking, Table 3 documented the unfactored infiltration rates measured in the field.

The information summarized in Paterson's Table 3 along with other historical unfactored infiltration rate measurements were used to simulate the performance of the EES in various sub-areas (refer to Figure 9 for details).

As shown in the table below, the largest area, namely Area 1A, was broken down into three (3) smaller areas; the northern part which consists of the area north of Moyard Drive, with two (2) smaller areas south of Moyard Drive consisting of an area to the west and one to the east. It should be noted that the unfactored infiltration rates noted in Paterson's Table 3 were derived based on the Site Evaluation and Soil Testing Protocol (Appendix C) of the CVC/TRCA LID Manual. The simulated unfactored infiltration rates shown in Table 7 below, includes the 2.5 safety factor.

Table 7: Simulated Unfactored Infiltration Rates

Area No.	Borehole No	Average Unfactored Infiltration(mm/hr)	Effective ¹ Unfactored Infiltration (mm/hr)
1A (northern) Area north of Moyard	BH11-22 & BH16-22	109.5	43.8
1A (southwest) Area south of Moyard	BH11-22 & BH18-15	88.5	35.4
1B	BH17-22 & BH18-22	84.0	33.6

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Note¹: Simulated unfactored infiltration was calculated by applying a safety factor of 2.5 on the average unfactored infiltration rate.

3.12.1.2 Post-Development Water Budget Findings

The post-development water budget analysis was carried out while accounting for EES reaches being implemented in the following areas (refer to Figure 9):

- All of Area 1A.
- All of Area 1B.

In terms of simulation of the EES in the above-noted areas, the same configuration and approach used in all stages of the Quinn's Pointe Subdivisions has been extended for Quinn's Pointe Phases 6–8. The configuration is based on the following:

- 1000 mm thick clear stone envelope, surrounded by geotextile
- 2-300 mm diameter perforated pipes, surrounded by geotextile
- Width of clear stone envelope is based on the following:
 - ✓ Sewer diameter plus 450 mm clearance on either sides for 900 mm diameter sewer or smaller
 - ✓ Sewer diameter plus 500 mm clearance on either side for sewers over 900 mm diameter.

The analysis was completed using Paterson's unfactored infiltration rates (refer to Section 3.12.1.1). A comparison was made between the existing and post-development water budgets with the modelling approach and assumptions summarized above.

Table 8: Post-Development Water Budget

Annual Water Budget Component	Annual Average Depth to Jock River (mm)	Annual Average Depth to Mud Creek (mm)	Annual Average Depth to Rideau River (mm)	Area Weighted Total (mm)	Budget %
Precipitation	864 [864]	864 [864]	864 [864]	864 [864]	100% [100%]
Evapotranspiration	248 [509]	262 [531]	252 [509]	249 [513]	29% [59%]
Infiltration	301 [296]	214 [314]	274 [286]	297 [297]	34% [34%]
Surface Runoff	316 [83]	389 [60]	339 [0]	319 [61]	37% [7%]

*Values in brackets ([]) represent pre-development values for purpose of comparison with post-development water budget values.

The above simulation results (Table 8) show that the infiltration strategy, as depicted on both MST in Appendix H capable in maintaining the existing condition infiltration rate of 34%.

Given that the existing water budget is maintained under post-development conditions with the proposed infiltration measures, the infiltration requirements of the CLI-ECA (Appendix A) will be achieved.

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3.12.2 Dual Drainage

3.12.2.1 Minor System

It was generally assumed that the USF elevations are approximately two (2) meters below the road centerline grades. However, in cases where the storm sewers are relatively shallow, such as to prevent the EES from being located within the groundwater, risers and appropriate grading will be required to ensure a minimum freeboard of 300 mm is maintained between the sewer HGL and the USF elevations of the units.

3.12.2.2 Major System

The dual-drainage analysis was reviewed in relation to the performance of the major overland system. Elevations above the major system nodes have been extracted and shown in Table 9. As shown below, there is no ponding in the 1:2-year event and there is no ponding in the 1:5-year in the industrial sites or along collector roads. Moreover, the depth of flow along the major system is below 350 mm during the 1:100-year event. Simulation results for other storm events, including climate change and historical storms, have been summarized in Appendix E.

Table 9: Major Overland Flow Results

Major System Node	1:2 yr - 3 hr Chi Ponding Depth (mm)	1:2 yr - 24 hr Ponding Depth (mm)	1:5 yr - 3 hr Chi Ponding Depth (mm)	1:5 yr - 24 hr Ponding Depth (mm)	1:100 yr 3 hr Chi Ponding Depth (mm)	1:100 yr - 24 hr SCS Ponding Depth (mm)
S_504-503	10	10	30	30	130	120
S_525-MTD	10	10	50	50	170	160
S_528-525	10	10	50	50	180	160
S_532-528	10	10	50	50	160	150
S_536-532	10	20	80	80	240	230
S_540-536	10	10	70	60	210	200
S_545-541	10	20	60	60	200	190
S_607-605	10	20	60	60	220	200
S_620-619	10	10	50	50	230	220
S_622-619	10	10	30	30	170	160
S_623-622	10	10	60	70	240	230
S_627-630	10	10	30	30	150	140
S_633-634	10	10	50	40	160	150
S_637-643	10	10	50	50	200	180
S_652-604	10	10	40	40	180	170

As part of the MSS/functional level modelling, only the critical sewers and trunk sewers are modelled to ensure that the site is serviceable and to which further details are typically refined during conceptual/detailed designs. However, to ensure accuracy and conservativeness within such a model, the time of concentration calculated in the Storm Sewer Design Sheet for each downstream sewer reach within a tributary area was used to calculate the Rational Method

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minor system inflow from major system node to the minor system. As a result, in some instances Table 9 above will show that there is ponding in some areas during the 1:2-year event and in the 1:5-year in the industrial sites or along collector roads. However, once the model is further refined during the detailed design stage, drainage areas will be further delineated and will be tributary to individual storm sewer reaches in which case the rational inflow for each area will be calculated based on a 10 minutes time of concentration and will in turn eliminate any ponding during the 1:2-year event and in the 1:5-year in the industrial sites or along collector roads. Moreover, the depth of flow along the major system is below 350 mm during the 1:100-year event. Simulation results for other storm events, including climate change and historical storms, have been summarized in Appendix E.

3.12.3 Proposed Dry Pond Expansion

An existing spill-over area exists within Quinn's Pointe Phases 6–8, in the southeastern portion of Block 540. This facility was constructed as part of Quinn's Pointe Subdivision to capture major overland flow being directed westerly given that it could not cross future Greenbank Road. The existing spill-over area is serviced by an existing outlet sewer (900 mm diameter storm sewer) discharging to the Inver Lane 1500 mm diameter storm sewer system. The 900 mm diameter outlet sewer was sized to empty the spill-over pond during infrequent storm events.

Drawings CS1, CMST, POND and D2 provide the configuration of the proposed end-of-pipe facility consisting of the following elements (refer to Section 3.11.5 of the MSS Addendum for additional details and Appendix C for information):

- Pre-treatment device - Defender Model FD-8
- By-pass manhole (flow splitter) forcing the 22 mm to the filtration dry pond while excess flows to by-pass the filtration dry pond and would be conveyed to the downstream dry pond.

Filtration Dry Pond:

- Surface Area at Spill of 100.8m: 2,235 m²
- Volume of Infiltration Dry Pond: 1,418 m³
- 700 mm thick of amended soil (bio-retention soil media)
- 350 mm thick clear stone
- 150 mm diameter sub-brains
- Geosynthetic membrane

Dry pond:

- Volume prior to spill: 28.790 m³
- Outlet elevation: 98.45 m
- Internal sloping: 1.0-1.3%
- 3 m wide perimeter access road
- Major overland flow route: Barnsdale Road (events > 100-year)

Once storm flows have been filtered by the filtration dry pond, they will merge with the treated flows from the EES and Industrial Lands and will be controlled by the existing outlet consisting of the 900 mm diameter storm sewer system. Table 10 and Table 11 (below) summarize key parameters relating to the configuration of these facilities.

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Table 10: Filtration Dry Pond Configuration and Results

Pond Configuration	Filtration Dry Pond
Pond Invert (m)	100.10
Footprint Area (m ²) @ 100.10 m	1,822
Top of Spillway Area (m ²) @ 100.80 m	2,235
Top of Bank (m) at Spill	100.80
Active Storage Depth (m)	0.70
Elevation of 15 m wide Spillway (m)	100.80
Outlet Diameter (m)	N/A
Drawdown Time (hrs) - 4-hour 22mm Design Storm	16.75
Overall effective filtration rate (mm/hr)	50.4

Table 11: Expanded Dry Pond Configuration and Results

Pond Configuration	Expanded Dry Pond
Pond Invert (m)	98.45
100-year Modelled Release Rate (m ³ /s) - 3-hour Chicago Storm	0.349
Pond Top of Bank (m)	100.8
Active Storage Depth (m)	2.35
Freeboard to adjacent building envelopes(m)	>0.3
Outlet Bottom Orifice Elevation (m)	98.45
Outlet Bottom Orifice Diameter (m)	0.335
Outlet Upper Orifice Elevation (m)	100.8
Outlet Upper Orifice Dimensions (m)	250 mm x 900 mm
Drawdown Time (hrs) - 4-hour 22mm Design Storm	17.5
Surface Area (ha)	3.66

The operation of the filtration pond is provided in Table 12 below for the 3-hour Chicago storm distribution and Table 13 for the 24-hour SCS storm Distribution.

Table 12: Filtration Pond Operation (3-hour Chicago Storm)

Event	Max HGL (m)	Max Depth (m)	Total Inflow (L/s)	Peak Filtration Rate (L/s)	Max Volume (m ³)
4-hour, 22 mm	100.67	0.57	1,066	26	1,134
1:2 year	100.90	0.80	1,832	26	1,649
1:5 year	100.95	0.85	2,095	26	1,754
1:100 year	101.05	0.95	5,591	26	1,998
1:100 year + 20%	101.10	1.00	7,804	26	2,111

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Table 13: Filtration Pond Operation (24-hour SCS Type II Storm)

Event	Max HGL (m)	Max Depth (m)	Total Inflow (L/s)	Peak Filtration Rate (L/s)	Max Volume (m3)
1:2 year	100.92	0.82	1,863	26	1,681
1:5 year	100.95	0.85	2,196	26	1,753
1:100 year	101.03	0.93	4,712	26	1,945
1:100 year + 20%	101.08	0.98	6,697	26	2,058

The operation of the dry pond is provided in Table 14 below for the 3-hour Chicago storm distribution and Table 15 for the 24-hour SCS storm Distribution.

Table 14: Dry Pond Operation (3-hour Chicago Storm)

Event	Max HGL (m)	Max Depth (m)	Total Inflow (L/s)	Peak Minor System Release Rate (L/s)	Max Volume (m3)
4-hour, 22 mm	99.40	0.95	1,767	200	2,717
1:2 year	99.77	1.32	3,915	248	7,128
1:5 year	100.08	1.63	6,729	282	12,390
1:100 year	100.80	2.35	11,860	349	28,780
1:100 year + 20%	100.98	2.53	14,400	495	33,240

Table 15: Dry Pond Operation (24-hour SCS Type II Storm)

Event	Max HGL (m)	Max Depth (m)	Total Inflow (L/s)	Peak Minor System Release Rate (L/s)	Max Volume (m3)
1:2 year	99.79	1.34	4,209	250	7,374
1:5 year	100.06	1.61	6,651	280	11,920
1:100 year	100.69	2.24	10,670	340	26,070
1:100 year + 20%	100.91	2.46	12,960	424	31,590

The modelling results demonstrate that the pond contains flow in up to the 1:100-year event. The maximum water level in the pond is 100.8 m, which provides a minimum 300 mm freeboard to adjacent properties (Quinn's Pointe) and a controlled spill occurs in events greater than the 1:100-year storm event onto Barnsdale Road.

3.12.4 Off-Site Peak Flow Discharges

3.12.4.1 General

The proposed storm servicing strategy outlined herein was developed to ensure that there are no net increases in off-site peak flows at any of the discharge points. Due to the constraints of the Thomas Baxter Municipal Drain—the natural receiving outlet with limited conveyance capacity and shallow elevation—the storm servicing approach for most of Quinn's Pointe

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Phases 6–8 involves capturing post-development drainage within a storm sewer system and redirecting it to a proposed on-site end-of-pipe facility in Block 540. Outflows from this facility would then discharge into the existing Quinn's Pointe storm sewer system at a rate equal to the flow rate identified for the Quinn's Pointe Subdivision under buildout conditions.

This storm servicing approach, referred to as a “disconnection,” aligns with the secondary objective of facilitating approval timelines under the Ontario Drainage Act by minimizing the release of post-development drainage to the Thomas Baxter Municipal Drain. This approach also aligns with a previously completed disconnection of an area (refer to 2018 BSUEA MSS) of which was tributary to the Thomas Baxter Municipal Drain and later formalized by RCI in its report entitled “Engineer's Report for the Subsequent Disconnection Thomas R. Baxter Municipal Drain (October 2021)”.

Section 3.12.5.3 (below) discusses in greater detail the area being contemplated for disconnection from the Thomas Baxter Municipal Drain. An area of 10.7 ha was initially disconnected as reported in the 2018 BSUEA MSS while the current proposal is to extend an additional disconnection of a 59.8 ha area. This value represents a reduction when compared to the previous MSS Addendum (October 2025) shown as 65.8 ha, as the flow pattern of the southern woodlot (identified as a good candidate for conveyance as natural feature) unchanged under post-development conditions as it will continue to sheet flow towards the Borrisokane Road open ditch system.

3.12.4.2 Hydrological Analysis and Simulation Results

In addition to the comparison of peak flows at the boundary of the Quinn's Pointe Subdivision, this hydrological analysis was refined to assess peak flows and runoff volumes at two (2) locations along the Borrisokane Road/Barnsdale Road open ditch systems, as well as at the boundary culvert leading to the headwater of the Thomas Baxter Municipal Drain.

The refined hydrological analysis considered a range of storm events, including the 4-hour Chicago 25 mm storm (the industry-standard water quality event) and typical events ranging from the 1:2-year to the 1:100-year return periods. The analysis also accounted for half of the Barnsdale Road ROW and the eastern half of the Borrisokane Road ROW, as detailed in Figure 5-2 of JLR's Existing Conditions Report (Appendix B of the MSS Addendum).

Given that the SWMHYMO platform used in 2017 for the Existing Conditions for the BSUEA (as reported in JLR's ECR - S1 Area) is no longer compatible with current computer and operating systems, the pre-development analysis was revised using the PCSWMM platform, which aligns with the Addendum post-development condition hydrologic analysis.

The refined analysis was conducted under both pre-development and post-development conditions, with peak flows and runoff volumes extracted at several discharge points along the boundary. Figures entitled *Pre-Development Model Schematic* and *Post-Development Model Schematic* are included in Appendix H.

Peak flows and runoff volumes were generated at the following discharge points:

1. **Existing Quinn's Pointe 900 mm diameter storm sewer:** Peak flows are summarized in Table 16.

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2. **Borrisokane open ditch/culvert system – Node “EXD_9”** (refer to Table 17 and Table 18)
3. **Borrisokane/Barnsdale Road open ditch/culvert system and Node “EXD_12”** (refer to Table 19 and Table 20)
4. **Thomas Baxter Municipal Drain** immediately downstream of the merging contributions between the Barnsdale Road and Borrisokane Road open ditch systems, at **Node “Municipal Drain”/“OF3”** (Table 21 and Table 22), refer to Appendix H for Schematics.

The *Pre-Development Model Schematic* and *Post-Development Model Schematic* is contained in Appendix E.

Table 16: Off-Site Peak Flow - Quinn's Pointe 900 mm diameter Sewer

Development Condition	1:100-year Peak Flow (L/s) 3-hour Chicago	1:100-year +20% Peak Flow (L/s) 3-hour Chicago
Quinn's Pointe 900 mm diameter Sewer		
Existing Condition	349	499
Post-Development Condition	349	495

Peak flows and runoff volumes have been summarized under the 4-hour 25 mm storm as well as under the SCS Type II storm, during events ranging between 1:2-year to 1:100-year as follows:

- Table 17 and Table 18 at Node “EXD_9”
- Table 19 and Table 20 at Node “EXD_12”
- Table 21 and Table 22 at Node “Municipal Drain”

Refer to Appendix E for pre- and post-development PCSWMM Schematics.

Table 17: Peak Flow Comparison at Node EXD_9

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Peak Flow (m³/s)	Off-Site Peak Flow (m³/s)					
	Return Period	Return Period (years)					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	0.162	0.260	0.297	0.332	0.370	0.399	0.427
POST DEVELOPMENT	0.070 (57%)	0.129 (50%)	0.235 (21%)	0.266 (20%)	0.323 (13%)	0.363 (9%)	0.400 (6%)

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Note: The values in parentheses are the reduction expressed in percentages

Table 18: Runoff Volume Comparison at Node EXD_9

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Runoff Volume (m ³)	Off-Site Runoff Volume (m ³)					
	Return Period	Return Period					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	518	742	1,148	1,523	2,059	2,544	3,082
POST DEVELOPMENT	347 (33%)	549 (26%)	854 (26%)	1,162 (24%)	1,620 (21%)	2,030 (20%)	2,462 (20%)

Note: The values in parentheses are the reduction expressed in percentages

Table 19: Peak Flow Comparison at Node EXD_12

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Peak Flow (m ³ /s)	Off-Site Peak Flow (m ³ /s)					
	Return Period	Return Period (years)					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	0.41	0.636	0.894	1.057	1.255	1.408	1.560
POST DEVELOPMENT	0.173 (58%)	0.296 (53%)	0.459 (49%)	0.581 (45%)	0.699 (44%)	0.795 (44%)	0.892 (43%)

Note: The values in parentheses are the reduction expressed in percentages

Table 20: Runoff Volume Comparison at Node EXD_12

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Runoff Volume (m ³)	Off-Site Runoff Volume (m ³)					
	Return Period	Return Period					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	1310	1,886	2,766	3,516	4,570	5,508	6,550
POST DEVELOPMENT	974 (26%)	1,503 (20%)	2,182 (21%)	2,748 (22%)	3,528 (23%)	4,196 (24%)	4,903 (25%)

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Note: The values in parentheses are the reduction expressed in percentages

Peak flows and runoff volumes have been summarized in Table 21 and Table 22 at the headwater of the Thomas Baxter Municipal Drain Node as reference by Node "Municipal Drain" / "OF3" on the pre- and post-development PCSWMM Schematics, respectively (Appendix E).

Table 21: Peak Flow Comparison at Node Municipal Drain

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Peak Flow (m ³ /s)	Off-Site Peak Flow (m ³ /s)					
	Return Period	Return Period (years)					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	0.443	0.709	1.021	1.223	1.471	1.668	1.866
POST DEVELOPMENT	0.213 (52%)	0.373 (47%)	0.612 (40%)	0.774 (37%)	0.948 (36%)	1.077 (35%)	1.220 (35%)

Note: The values in parentheses are the reduction expressed in percentages

Table 22: Runoff Volume Comparison at Node Municipal Drain

	Design Storm						
	Water Quality Event	6 hr SCS Type II					
	Off-Site Runoff Volume (m ³)	Off-Site Runoff Volume (m ³)					
	Return Period	Return Period					
	4 hr 25 MM	1:2	1:5	1:10	1:25	1:50	1:100
EXISTING CONDITIONS	1628	2,334	3,481	4,468	5,849	7,062	8,394
POST DEVELOPMENT	1289 (21%)	1,990 (15%)	2,981 (14%)	3,831 (14%)	4,990 (15%)	5,973 (15%)	7,009 (16%)

Note: The values in parentheses are the reduction expressed in percentages

The following conclusions can be drawn from the review of the above summary Tables:

- The 1:100-year post-development peak flows released to the existing Quinn's Pointe storm sewer are maintained at levels identified for the Quinn's Pointe Subdivision at the build-out conditions (Table 16)
- The lower peak flows and runoff volumes at Node EXD_9 (Table 17 and Table 18) are attributable to a reduction in contributing drainage area from 22.0 ha under pre-development conditions to 9.0 ha under post-development conditions.

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- The lower peak flows and runoff volumes at Node EXD_12 (Table 19 and Table 20) are attributed to a reduction in contributing drainage area from 73.2 ha under pre-development conditions to 13.4 ha under post-development conditions.
- As a result of the reductions in contributing drainage area noted above, post-development peak flows and runoff volumes discharging to the Borrisokane Road open ditch system and to the Borrisokane Road/Barnsdale Road at Nodes EXD_9 and EXD_12, respectively are lower than pre-development values for all storm events, as summarized in the above Tables.
- Peak flows and runoff volumes at the Municipal Drain (Table 21 and Table 22) reflect a reduction in the contributing drainage area from 80.3 ha under pre-development conditions to 20.5 ha under post-development conditions. Accordingly, Table 21 and Table 22 indicate decreases in both peak flows and runoff volumes for all storm events.

3.12.5 Impact of Post-Development Drainage Modifications

3.12.5.1 General

The proposed drainage limit modifications will alter the hydrological regime as highlighted in Section 3.12.4.2. In all cases, the proposed disconnection reduces both peak flows and runoff volumes at the analyzed off-site discharge points.

The following sections provide an impact assessment of the modified hydrological regime at key locations:

- Borrisokane Road open ditch system
- Barnsdale Road open ditch system
- Thomas Baxter Municipal Drain
- Downstream Fisheries

3.12.5.2 Borrisokane Road Open Ditch System

The Borrisokane Road open ditch system is part of Highway 416's right-of-way (ROW), which is owned and operated by the Ministry of Transportation (MTO). Therefore, any modifications to the hydrological regime of the Borrisokane Road open ditch system should be reviewed by MTO.

As reported in Section 3.12.4.2, peak flows and runoff volumes were evaluated at the downstream end of the Borrisokane Road open ditch system at Node EXD_9 (Appendix E for Schematics). Simulation results indicated the following:

- Post-development peak flows are reduced by $\pm 57\%$ and 6% at Node EXD_9 during the 4-hour 25 mm storm and 1:100-year storm, respectively. Similarly, runoff volumes are reduced by $\pm 33\%$ and $\pm 20\%$ during the 4-hour 25 mm storm and 1:100-year event, respectively.
- The simulated 1:2-year peak flows at Node EXD_9 were found to be $0.260 \text{ m}^3/\text{s}$ and $0.129 \text{ m}^3/\text{s}$ under pre- and post-development conditions, respectively as the lowered peak flows under post-development conditions are due to the reduction in area from 22.0 ha to 9.0 ha. Based on the length of the matured roadside ditch along Borrisokane Road of 640 m,

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from Kilbirnie Road to Node EXD_9, a travel time (contact time) of 35 minutes was estimated for the 1:2-year post-development velocities.

Based on literature in Ontario (e.g. MECP SWMPDM and STEP), well-vegetated roadside ditch systems function as grassed swales or conveyance control low impact development (LID) measures, where runoff is conveyed under non-erosive velocities (velocities ≤ 0.25 m/s), promoting extended contact time (i.e., 35 minutes) with vegetation and underlying soils. As documented in MECP guidance, STEP and U.S. EPA/FHWA literature, these conditions facilitate sedimentation and filtration processes responsible for total suspended solids (TSS) removal. Given that extended flow path length of 640 m and residence time of 35 minutes, such systems can reasonably be considered to provide a level of treatment consistent with enhanced protection objectives.

Based on the above simulation results, the proposed drainage modifications (i.e., disconnection) will provide relief to the Borrisokane Road ditch system as peak flows, runoff volumes and velocities will all be reduced. Therefore, there are no anticipated adverse impacts along this drainage system owned and operated by MTO. As a result, no water quality measures are being recommended for the Kilbirnie Drive extension (0.435 ha).

3.12.5.3 Barnsdale Road Open Ditch - Headwater Features

An Environmental Impact Statement (EIS) was prepared by Arcadis, with the latest version of the Report issued by Fotenn (March 3rd, 2026). As part of this EIS, a Headwater Drainage Feature (HDF) Assessment identified three (3) reaches along Barnsdale Road—BN-1-A, BN-1-1B, and BN-2-A—which received a management recommendation of mitigation requiring the following:

The reaches identified should have their functions replicated or enhanced through improved conveyance measures, with flows directed to the appropriate downstream receiver.

A review of topography shows that the invert of the Barnsdale Road open ditch system varies from approximately 99.6 m at BN-1-A to 98.7 m at BN-2-A, matching the measured edge-of-water elevation of the adjacent man-made pond. Most reaches identified in the EIS would appear to be groundwater-fed rather than being fed by the northern areas, based on measured water elevations shown on the topographical map. Additionally, as a secondary source of inflow, the alignment of these HDFs coincide with the Barnsdale Road open ditch system, which receives runoff from half of the Barnsdale Road ROW, including about 4.5 m wide asphalt strip and a gravel shoulder at a typical 3:1 side slope. While part of Quinn's Pointe Phases 6–8 is proposed to be disconnected from BN-1-A, BN-1-1B, and BN-2-A, runoff from the Barnsdale ROW, including the shoulder and groundwater contribution will be maintained, allowing these reaches to function as they do under existing conditions.

Simulation of runoff volume at BN-2-A (EXD_12), as indicated in the above Table, shows a 26% reduction during the 4-hour 25 mm storm, which should not adversely impact the HDFs given the above-noted sources of inflow will be maintained. However, the EIS provided mitigation measures (Section 8.2.2) should any construction activities be planned to take place below the high-water mark of the HDFs. These are the following:

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- Due to their classification as indirect fish habitat, any direct construction below the high-water mark in Features BN-1, BN-2, BN-3, and Feature 416 will require authorization from DFO through the submission of a Request for Review.
- Direct construction in BN-1, BN-2, BN-3, and Feature 416 and at elevations below the high-water mark should incorporate shallow groundwater measures as well as LID construction practices.
- No work within the high-water mark outside of the fish timing window.

No direct construction activities are envisioned to impact directly any of the above-noted HDFs. The following activities are however anticipated:

- Construction of the residential subdivision and infrastructure within Quinn's Pointe Phases 6–8, located outside of the HDFs.
- Construction of the Barnsdale Road 375 mm diameter sanitary sewer extension, planned from the existing stub (at Bundoran Place) to Street 17, and northerly. The alignment of this sanitary sewer extension is away from BN-1-A.

Upcoming Infrastructure Work

It is important to note that the MTO and the City of Ottawa plan to widen this section of Barnsdale Road in the future, as an ongoing Class Environmental Assessment was initiated in 2025 by the city. It is unknown at this time whether a rural or urban cross-section will be selected. However, widening will increase the paved surface and encroach on HDF reaches BN-1-A, BN-1-1B, and BN-2-A. Consequently, the function of these reaches will significantly be altered or completely removed as widening occurs over the next 5 to 10 years. Until such time, the function of these HDFs will remain due to groundwater inflow combined with surface runoff from Barnsdale Road.

3.12.5.4 Thomas Baxter Municipal Drain

The partial disconnection of approximately 59.8 ha results in reductions to both peak flows and runoff volumes at the headwaters of the Thomas Baxter Municipal Drain (Node "Municipal Drain"), as illustrated on the Pre-Development and Post-Development Model Schematics provided in Appendix E.

Peak flows and runoff volumes at the Municipal Drain (Table 21 and Table 22) similarly reflect a reduction in the contributing drainage area from 80.3 ha under pre-development conditions to 20.5 ha under post-development conditions. Accordingly, Table 21 and Table 22 indicate decreases in both peak flows and runoff volumes for all storm events. Accordingly, the partial disconnection is not anticipated to adversely affect the Municipal Drain. Subject to approval, the proposed modifications to the hydrologic regime will be documented in an updated Engineer's Report prepared in accordance with the Drainage Act (Section 78, Chapter D.17, R.S.O. 1990).

As shown on Figure 10, the total study area of approximately 69.9 ha is comprised of two different watersheds: Mud Creek Subwatershed and the Jock River Subwatershed. Under existing condition, most of the study area is tributary to the Thomas Baxter Municipal Drain/Mud Creek Subwatershed and a small section in the Northeast corner is tributary to the Jock River. Under post-development condition a partial disconnection of approximately 59.8ha from the

Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

Mud Creek Subwatershed is captured tributary to the proposed dry pond which in turn tributary to the Jock River. A breakdown of the tributary areas under pre- and post-development conditions are summarized in the Table 23 below:

Table 23: Study Area Breakdown by Watershed

Subwatershed	Pre-Development Area (ha)	Post-Development Area (ha)
Thomas Baxter Municipal Drain/Mud Creek	66.5	6.7
Jock River	3.4	63.2

3.12.5.5 Downstream Fisheries

Simulation results summarized in Section 3.13 demonstrate reductions in both peak flows and runoff volumes at the headwaters of the Thomas Baxter Municipal Drain, immediately downstream of the southern limit of the Barnsdale Road right-of-way. From a fisheries perspective, the 4-hour, 25-mm storm event is considered more influential than higher return period events, as it occurs approximately four to five times annually, compared to the less frequent 2-year and 100-year events.

Under post-development conditions, the runoff volume associated with the 4-hour, 25-mm rainfall event was estimated at approximately 1,289 m³, representing a reduction of about 339 m³ (approximately 21%) relative to pre-development conditions. As no fish habitat is present at the headwaters of the Thomas Baxter Municipal Drain near Barnsdale Road, this reduction in runoff volume is not expected to adversely affect downstream fish communities, which are primarily located near the confluence with Mud Creek.

LiDAR analysis indicates that the total drainage area of the Thomas Baxter Municipal Drain at its confluence with Mud Creek is approximately 883 ha, whereas the study area reflects a tributary area of approximately 80.3 ha—representing less than 10% of the total contributing area at Mud Creek. As additional runoff contributions occur along the reach between Barnsdale Road and Mud Creek, the relative influence of the reduced runoff volume diminishes as flow from the remaining ±803 ha of largely unchanged drainage area is incorporated. Consequently, hydrologic conditions in proximity to the Mud Creek fish communities are expected to remain effectively unchanged.

Consistent with this assessment, Section 8.2.2 of the EIS concludes that conditions near the Mud Creek fish communities are anticipated to remain virtually unchanged and align with the HDF Assessment mitigation objective to maintain or replicate existing conveyance functions toward appropriate receivers. Accordingly, the proposed disconnection is not expected to result in adverse impacts to fish communities near Mud Creek.

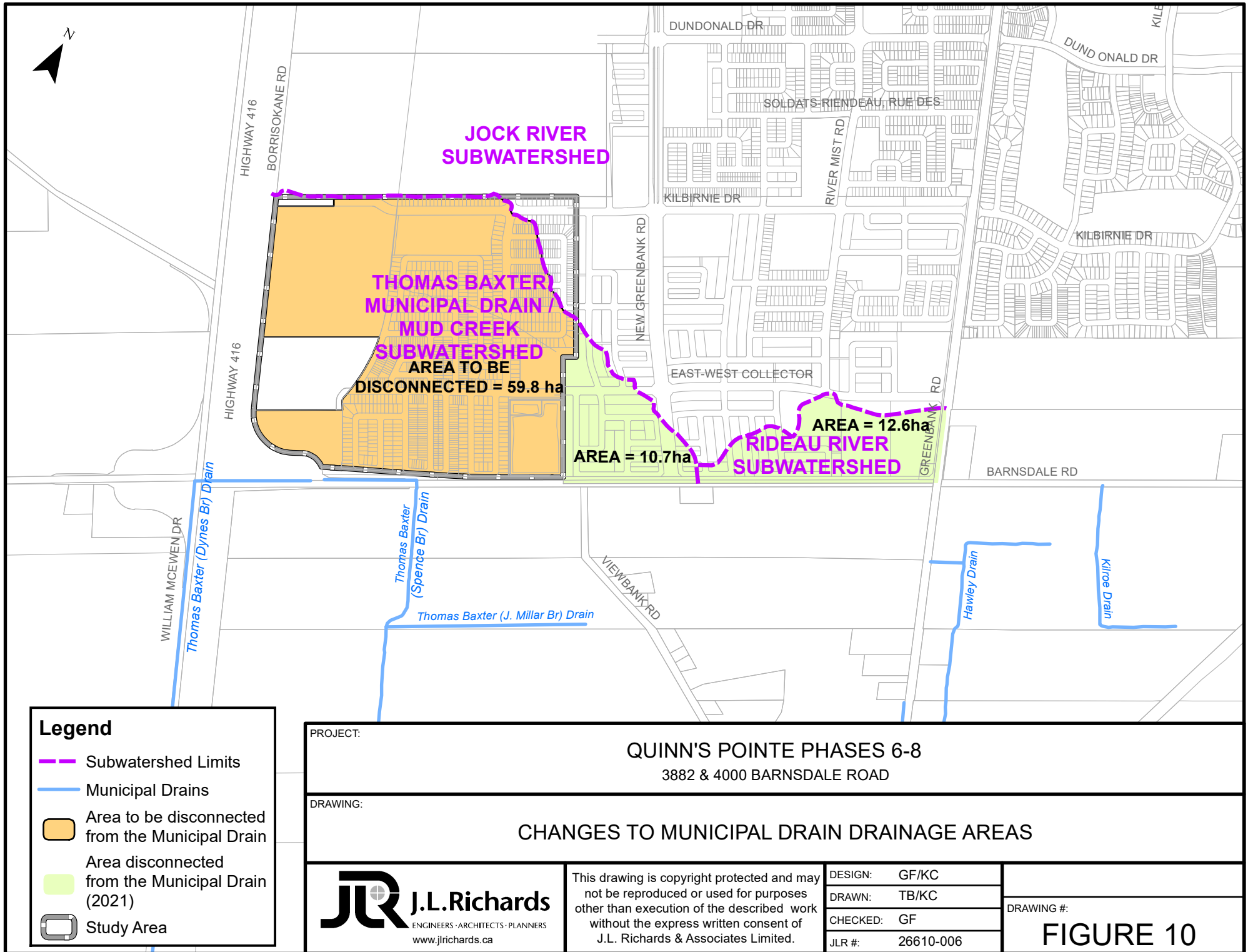
3.12.6 Water Quality for the Constraint Areas

Calculations presented in Section 3.10.1 demonstrate that water quality control requirements are satisfied for all areas serviced by an EES, as this system exceeds the infiltration criteria outlined in Table 3.2 of the Stormwater Management Planning and Design Manual (SWMPDM). Furthermore, the infiltration strategy detailed in Section 3.12.1 is shown to be comparable to the

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existing condition water budget. Accordingly, the proposed stormwater servicing approach satisfies the requirements of the CLI-ECA.

Water quality treatment for areas identified as constraints on Figure 9 will be provided at both storm sewer inlets (STMH501 and STMH554) to the expanded dry pond (refer to Drawing CMST in Appendix H). As infiltration measures have been implemented to the maximum practical extent and post-development water balance conditions are comparable to existing conditions, the requirements of the CLI-ECA are met. For the areas noted as constraints, water quality will be achieved via a treatment train approach; a pre-treatment MTD (ETV sized) and filtration dry pond sized to provide on its own an 80% TSS removal.



Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

3.13 Stormwater Summary and Recommendations

This AAPS and MSS Addendum presented a storm servicing strategy, which was based on the extension of the approved solution described in the 2018 BSUEA MSS. This solution consisted of a traditional storm sewer system, sized based on the SDG, combined with infiltration measures that were developed to the maximum extent possible while recognizing a series of constraints described in Section 3.7.1 and depicted on Figure 9.

. To supplement this system, the areas referenced as constraints will be serviced by a treatment train approach combining a pre-treatment MTD with a filtration dry pond.

The conceptual servicing design was found to fulfill the following:

- Achieves the prescribed flow rate along the existing 900 mm diameter storm sewer within Quinn's Pointe.
- Achieves the minimum 300 mm freeboard across Quinn's Pointe Phases 6–8 (residential).
- Achieves flow depths of 350 mm or less along the major overland system of Quinn's Pointe Phases 6–8.
Achieves the infiltration requirements as specified in Appendix A of the CLI-ECA as the existing condition water budget will be maintained.
- Achieves the filtration requirements as specified in Appendix A of the CLI-ECA as the water quality control will be met.
- Achieves a comparable water budget to existing levels.
- Peak flow and runoff volume reductions to the Borrisokane Road open ditch system, to the Thomas Baxter Municipal Drain and Barnsdale Road open ditch system.

It is recommended that storm servicing in Quinn's Pointe Phases 6–8 be carried out based on:

- ✓ Conventional storm sewers along with an EES on local roads, when permitting
- ✓ Lot-level sub-surface infiltration measures for each building within the Industrial lands.
- ✓ Areas defined as constraints to be serviced by a treatment train approach, combining a pre-treatment MTD with a filtration dry pond.

Simulation results indicate that the EES along local roads is sufficient to achieve an enhanced level of protection when sized to capture 22 mm of runoff. In contrast, roadway areas not serviced by an EES require a filtration alternative prior to discharging off-site. The recommended filtration solution as identified by the screening process is to provide a filtration dry pond supplemented by pre-treatment MTD.

Overall outflows from both trunk storm sewers are then controlled by the end-of-pipe facility dry pond discharging to the existing sewers in Quinn's Pointe.

Simulation results have revealed that the EES along local roads is sufficient when sized for 22 mm in meeting an enhanced protection level. Similarly, the filtration dry pond which uses the same infiltration principle is sufficient to meet the CLI-ECA.

Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

4.0 Wastewater Servicing

4.1 Background

The wastewater collection system within Quinn's Pointe Phases 6–8 has been conceptually sized to convey flows to existing gravity sanitary sewers within the Barrhaven South Community. These trunk sanitary sewers ultimately discharge the captured flows to the 900 mm diameter trunk sanitary sewer located along Greenbank Road. From that point, wastewater is then conveyed to the South Nepean Collector, followed by the West Rideau Collector, which ultimately outlets to the Robert O. Pickard Environmental Centre (ROPEC), where it is treated prior to being discharged into the Ottawa River.

In JLR's Serviceability Study (2019), it was demonstrated that the parcels adjacent to the BSUEA, which includes Quinn's Pointe Phases 6–8, could be serviced through extensions of the existing and proposed infrastructure in the area. From a wastewater servicing perspective, it was confirmed that Quinn's Pointe Phases 6–8 could be serviced via a connection to the existing 375 mm diameter sanitary sewer located on Barnsdale Road, south of the existing Quinn's Pointe Stage 4 (refer to Figure 11 and Figure 12).

More recently, the 2025 ECR (Appendix B of the MSS Addendum) identified several critical sewers where residual capacities of the existing trunk sanitary sewer network were to be reviewed to confirm that they could accommodate the increase in peak wastewater flows from Quinn's Pointe Phases 6–8. These critical pipe reaches are summarized in Table 24 below.

Table 24: Critical Residual Capacity in Existing Trunk Sanitary Sewers

Trunk Sanitary Sewer	Limiting Pipe Reach
Barnsdale Road	MH203 to MH202
Greenbank Road	MH200C to MH45
Greenbank Road	MH435A to MH501A

The wastewater servicing strategy for Quinn's Pointe Phases 6–8 was developed in accordance with the design criteria outlined in the OSDG and subsequent technical bulletins, while also respecting the capacity limitations of the existing downstream infrastructure.

File Location: P:\26000\26610-006 - Quinns Pointe Stage 6\10-Production\01-Civil\Servicing Brief Figures\26610-006 C FIGURE 11 - EX SAN.dwg

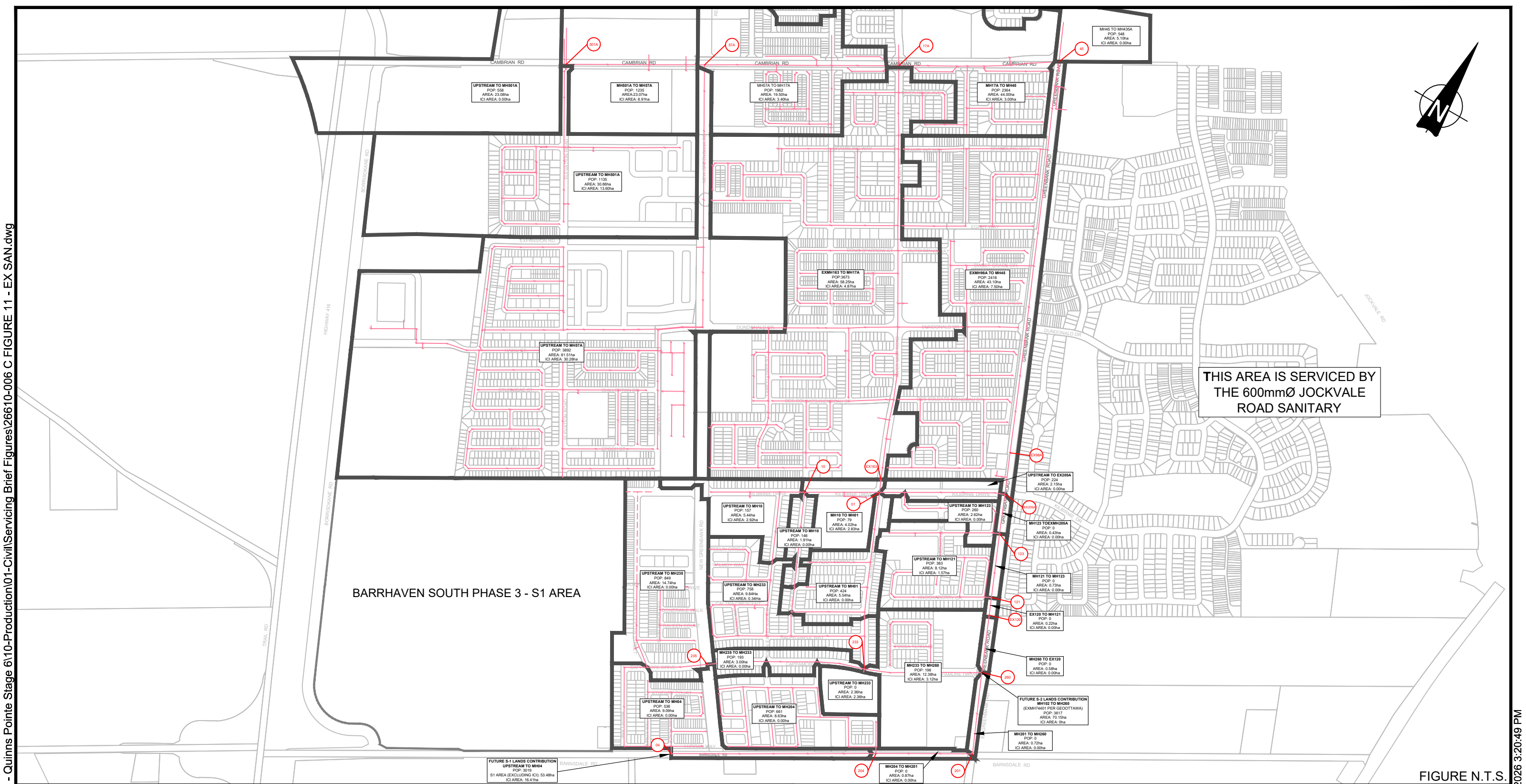


FIGURE N.T.S.

LEGEND

- S1 URBAN EXPANSION STUDY AREA
- EXISTING SANITARY DRAINAGE BOUNDARIES
- EXISTING SANITARY SEWER AND MANHOLE
- PLANNED SANITARY SEWER

PROJECT:

QUINN'S POINTE PHASES 6-8
3882 & 4000 BARNSDALE ROAD

DRAWING:

EXISTING OVERALL SANITARY DRAINAGE PLAN

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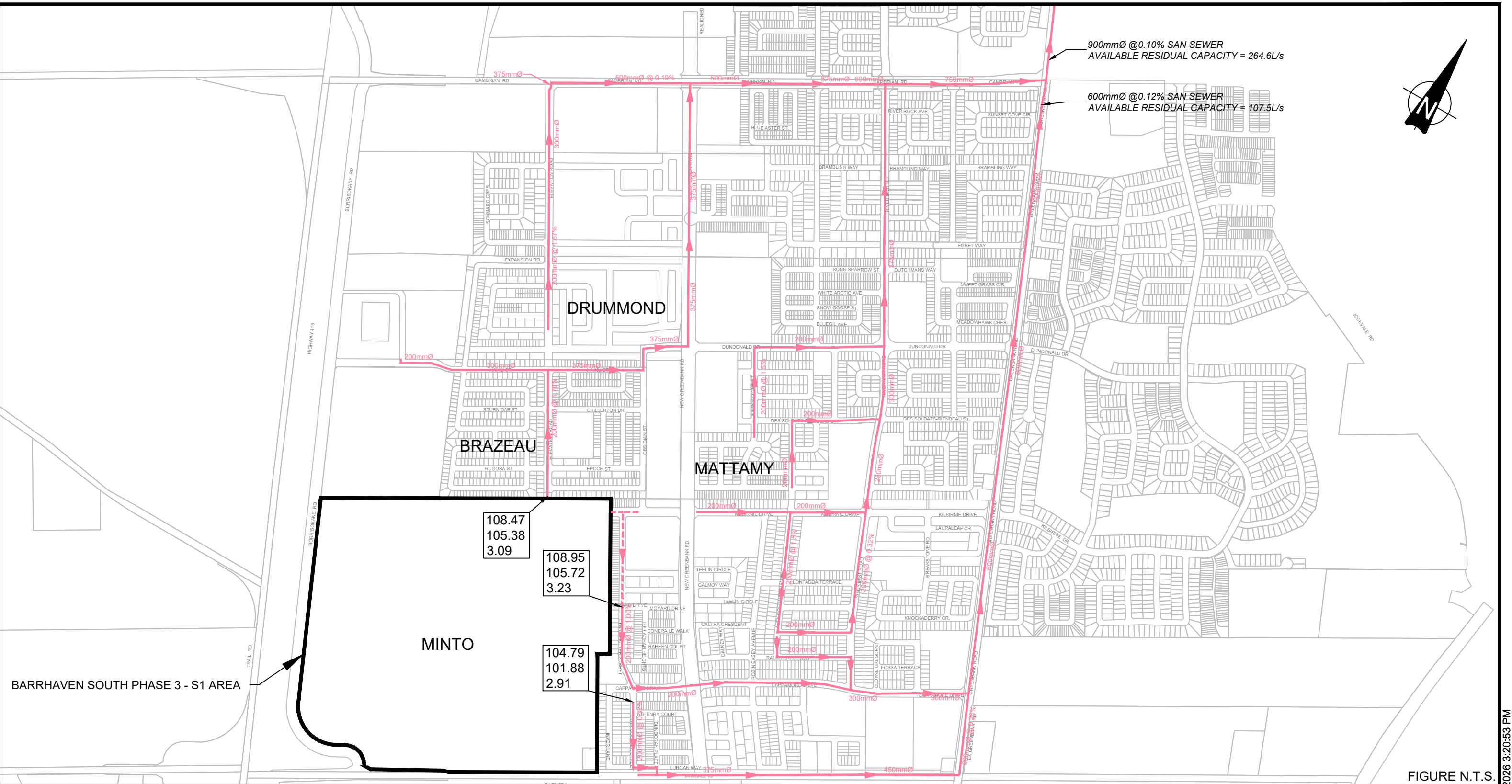
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FIGURE 11

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LEGEND

EXISTING SANITARY SEWER

PLANNED SANITARY SEWER

98.45	EXISTING GROUND (±)
91.77	SANITARY SEWER OBVERT
6.7	COVER DEPTH (m)

PROJECT:

QUINN'S POINTE PHASES 6-8
3882 & 4000 BARNSDALE ROAD

DRAWING:

EXISTING AND PLANNED SANITARY SERVICING INFRASTRUCTURE



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FIGURE 12

Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

4.2 Wastewater System Servicing

4.2.1 Design Criteria

Trunk sanitary sewers servicing the Quinn's Pointe Phases 6–8 were designed in accordance with the following criteria from the ODSG and subsequent Technical Bulletins:

Table 25: Sanitary Design Criteria

Design Criteria	Design Value
Residential Average Flow	280 L/cap/day
Residential Peaking Factor	Harmon Formula x 0.8
Light Industrial Average Flow	28,000 L/gross ha/day
ICI Peaking Factor ⁽¹⁾	1.0/1.5
Total Infiltration	0.33 L/s/ha
Minimum Velocity	0.6 m/s
Maximum Velocity	3.0 m/s
Manning Roughness Coefficient (for smooth wall pipes)	0.013
Minimum Allowable Slopes	Varies
Population Density	Single Family: 3.4 ppu Townhouse: 2.7 ppu

(1) 1.5 if ICI Contribution > 20%, 1.0 otherwise.

4.2.2 Residual Capacities in the Existing Sanitary Sewer Network

As part of the BSUEA MSS (2018), a Master Sanitary Design Sheet was developed to assess the available capacity of the existing sanitary sewer infrastructure within the BSUEA, and further downstream in the existing areas. This design sheet was subsequently refined during the 2019 Serviceability Study and further updated in the ECR (2025) to incorporate the most recent design flow data for Quinn's Pointe Stages 2 through 5, as well as the future S1 and S2 Areas. The S2 Area represents a planned expansion located east of Greenbank Road and north of Barnsdale Road which is also in the MSS preparation stage.

The Serviceability Study (2019) identified segments of the existing network with potential capacity limitations. This analysis was further refined in the ECR (2025), with a specific focus on Quinn's Pointe Phases 6–8. The sanitary trunk sewers evaluated for this area were located along Barnsdale Road and Greenbank Road. The limiting pipe reaches identified in this report are summarized in Table 26.

At the time of the preparation of the ECR (2025), the Concept Plan for Quinn's Pointe Phases 6–8 was at a preliminary stage thus the available servicing capacities of the existing system have been re-evaluated to reflect the most recent iteration of the Concept Plan and Draft Plan (Appendix D).

4.3 Wastewater Collection System Strategy

4.3.1 Phasing

As shown in the preferred Concept Plan (Appendix D), development of Quinn's Pointe Phases 6–8 is proposed to proceed in three sequential residential phases: Phases 6, Phase 7, and

Assessment of Adequacy of Public Services Quinn's Pointe Phases 6-8

Phase 8, including two industrial land parcels totaling 13.22 ha to the north and 3.19 ha to the south. The ultimate sanitary outlet for the entire Quinn's Pointe Phases 6–8 is located within Phase 6 at Barnsdale Road at Street No. 15. Accordingly, the downstream sewer infrastructure as part of Phase 6 will be designed to accommodate ultimate buildout conditions, including all residential phases and both industrial lands. By sizing the initial phase infrastructure to meet full future capacity requirements, the system will inherently satisfy the servicing needs of all interim development phases.

4.3.2 Proposed Sewer System Layout and Sizing

As shown in the Concept Plan (Appendix D), the wastewater collection system for Quinn's Pointe Phases 6–8 is intended to service single family units, townhouse blocks, the industrial lands and a park block.

The proposed sanitary sewers within Quinn's Pointe Phases 6–8 have been conceptually sized based on the design criteria listed in Table 27 and the drainage areas depicted on Drawing CMSAN in Appendix H. A trunk sanitary sewer system layout has been developed in accordance with the latest Concept Plan (Appendix D). Within Quinn's Pointe Phases 6–8, there are two (2) designated collector roads which are:

- Elevation Road: This roadway serves as the north-south collector from Kilbirnie Drive to Cappamore Drive; and
- Cappamore Drive: This roadway serves as the east-west collector from the woodlot, potentially conveyed as natural feature, which is adjacent to the Industrial Lands to the existing Quinn's Pointe Stage 4.

Since Elevation Road and Cappamore Drive are the designated collector roads within Quinn's Pointe Phases 6–8, the trunk sanitary sewers are proposed to follow their alignments within the Study Area.

The remainder of the sanitary sewer network is to follow the configuration of the local road system, with careful consideration given to maintaining minimum flow velocities, ensuring adequate cover, and avoiding infrastructure conflicts. The sanitary sewer layout is to be developed in coordination with the stormwater and watermain networks.

The Master Sanitary Sewer Design Sheet, prepared as part of the ECR, included the population and corresponding flows for Quinn's Pointe Phases 6–8 (refer to Appendix F). It also shows trunk sanitary sewer sizing. Based on this assessment, wastewater from Quinn's Pointe Phases 6–8 is expected to be conveyed primarily through an internal sanitary network consisting of sanitary sewers ranging from 200 mm to 375 mm in diameter. Final pipe sizing will be confirmed during detailed design.

4.3.3 Barnsdale Road Outlet

The 2019 Serviceability Study evaluated potential sanitary sewer connections for Quinn's Pointe Phases 6–8 and concluded that servicing could be achieved via a connection to the existing 375 mm diameter sanitary sewer located on Barnsdale Road, south of the Quinn's Pointe Stage 4 development. This recommendation was based on the available residual capacity within the Barnsdale Road sewer infrastructure. Additionally, the Master Sanitary Sewer Design Sheet

Assessment of Adequacy of Public Services Quinn's Pointe Phases 6-8

(refer to Appendix F) included in the report confirmed that the existing sanitary sewers on Cappamore Drive do not have sufficient capacity to accommodate additional flows.

4.4 Impact on Downstream System

The ECR (2025) included the refinement of the BSUEA Master Sanitary Design Sheet to assess the available capacity of the existing sanitary sewer infrastructure. This analysis incorporated the most up-to-date design flow data for Quinn's Pointe Stages 2 through 5, along with projected flows for the future S1 and S2 Areas.

This Master Sewer Design Sheet has been refined further as part of this assessment to include design flows from the most recent iteration of the Concept Plan (Appendix D). With the addition of the S1 and S2 Areas, the calculated theoretical peak wastewater flows were found to be within the available residual capacities of the existing downstream sanitary sewer (refer to Master Sanitary Design Sheet in Appendix F). Consequently, no further infrastructure improvements are required along the existing downstream system. Table 26 provides a summary of the impact of the S1 and S2 areas to the critical pipe reaches in Table 26.

Table 26: Residual Capacity in Existing Trunk Sanitary Sewers

Trunk Sanitary Sewer	Limiting Pipe Reach	Residual Capacity
Barnsdale Road	MH203 to MH202	122.0 L/s
Greenbank Road	MH200C to MH45	12.3 L/s
Greenbank Road	MH435A to MH501A	99.4 L/s

4.5 Wastewater Summary and Recommendations

The wastewater collection system illustrated in Drawing CMSAN in Appendix H and the accompanying Master Sanitary Sewer Design sheet (Appendix F) has been conceptually sized based on the design criteria outlined in Table 25, while accounting for the residual capacities of the pipe segments identified in Table 26.

As noted in Section 4.3, the layout of the trunk sanitary sewer system has been developed in accordance with the most recent Concept Plan, with Elevation Road and Cappamore Drive designated as the primary north-south and east-west trunk sewers, respectively.

It is expected that wastewater generated by Quinn's Pointe Phases 6–8 can be accommodated by a series of 200 mm diameter sanitary sewers as shown in the Master Sanitary Sewer Design Sheet (Appendix F). Furthermore, the calculated theoretical peak wastewater flows for Quinn's Pointe Phases 6–8 were found to be within the available residual capacities of the existing downstream sanitary sewer while accounting for the projected flows from the S2 Area (refer to Appendix F). Consequently, no further infrastructure improvements are required along the existing downstream system.

Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

5.0 Water Distribution

5.1 Background

The 2018 BSUEA MSS reviewed and evaluated servicing options with the identification of preferred trunk watermain, which were subsequently constructed. This Addendum, prepared as an extension to the 2018 BSUEA MSS, identifies feeder mains to support the urbanization of Quinn's Pointe Phases 6–8.

In 2019, the Serviceability Study water supply assessment confirmed that extensions of existing watermain would provide potable water that met the design criteria and objectives listed in Section 4.2.2 of the Ottawa Design Guidelines for Water Distribution (December 2025).

The review of the existing and future planned watermain was carried out as part of the 2025 ECR (refer to Appendix B of the MSS Addendum). Figure 13 depicted the existing and planned watermain along the boundary of Quinn's Pointe Phases 6–8. This review has shown that the current urban development to the east of the Study Area (i.e., Quinn's Pointe Subdivision) and to the north of the Study Area (i.e., The Ridge Subdivision) are both serviced by the municipal distribution system, which draws its water from the Ottawa River at both the Britannia and Lemieux Water Purification Plants. From those purification plants, potable water is distributed via a series of feeder mains and water booster stations. The City of Ottawa water distribution system is planned based on achieving the following design objectives:

- Quantity: sufficient water at sufficient pressure (Section 4.2.2 of the Ottawa Design Guidelines for Water Distribution)
- Quality: safe drinking water to the required standards (Section 4.1 of the Ottawa Design Guidelines for Water Distribution).
- Reliability: to ensure water supply under emergency conditions.
- Effectiveness and affordability: to reduce lifecycle costs being sustainable in the long term.

Three (3) feeder mains service the Barrhaven South Community from the Barrhaven reservoir, which are as follows:

- a 406 mm main along Greenbank Road
- a 406 mm main along Cambrian Road; and
- a 305 mm feeder main along River Mist Road

Two (2) sections of the planned feeder mains were also recently constructed as follows:

- 305 mm feeder main along New Greenbank Road, from Barnsdale Road to Kilbirnie Road.
- 406 mm feeder main on Cambrian Road, from Elevation Road to existing Greenbank Road.

Future planned feeder mains are as follows:

- 305 mm feeder main on Kilbirnie Drive, from New Greenbank Road to the eastern perimeter of Quinn's Pointe Phases 6–8, including a connection to Obsidian Street.

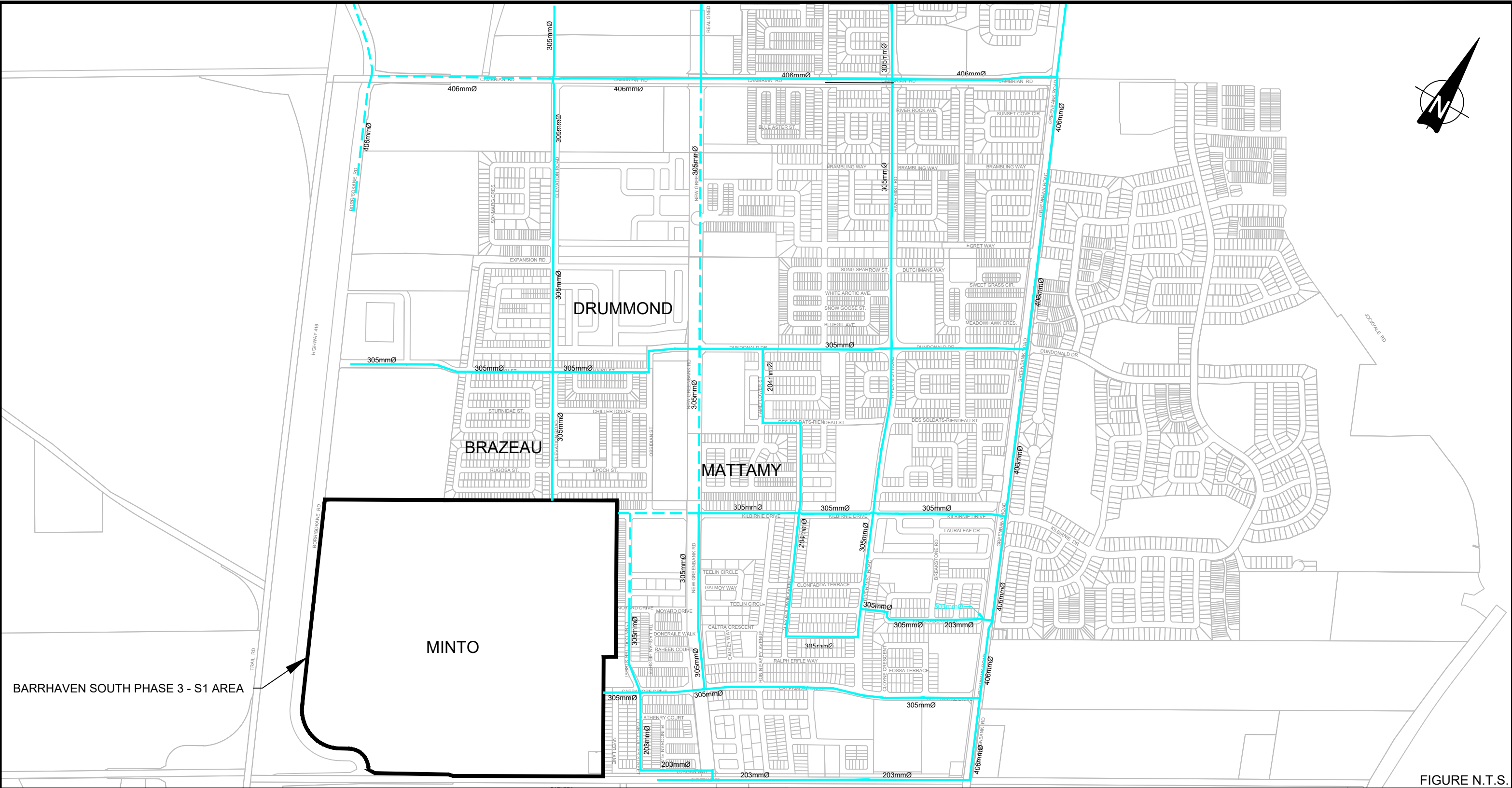


FIGURE N.T.S.

LEGEND

- EXISTING WATERMAIN
- PLANNED WATERMAIN

PROJECT:

QUINN'S POINTE PHASES 6-8
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EXISTING AND PLANNED WATER MAIN SERVICING INFRASTRUCTURE



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FIGURE 13

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Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

5.2 Water Servicing

5.2.1 Design Criteria

The Ottawa Design Guidelines for Water Distribution (December 2025) require that all new development additions to the public water distribution system be designed to conform to the following:

- In general conformance with the MOE Design Guidelines for Drinking-Water Systems the desired range of system pressures should be approximately 350 to 480 kPa (50 to 70 psi) and not less than 275 kPa (40 psi) during normal operating conditions.
- Where fire flow has been provided; during periods of maximum day and fire flow demand the residual pressure at any point in the distribution system shall not be less than 140 kPa (20 psi).
- In accordance with the Ontario Code & Guide for Plumbing, the maximum pressure at any point in the distribution system in occupied areas outside of the public right-of-way shall not exceed 552 kPa (80 psi.)
- The maximum pressure at any point in the distribution system in unoccupied areas shall not exceed 689 kPa (100 psi.)
- Feeder mains, which have been provided primarily for the purpose of redundancy, shall meet, at a minimum, the basic day plus fire flow demand.

In addition to the above targets, servicing should be carried out to minimize the use of dead-end water mains.

5.2.2 Phasing

As shown in the preferred Concept Plan (Appendix D), development of Quinn's Pointe Phases 6–8 is proposed to proceed through three sequential residential phases: Phase 6 including an industrial land parcel of 13.22 ha to the north, Phase 7, and Phase 8 including an industrial land parcel of 3.19 ha to the south. All watermain connections from Quinn's Pointe Phases 6–8 to the existing municipal network will be constructed as part of Phase 6, with the subsequent phases and future industrial lands extending from Phase 6.

Accordingly, the analysis and results in this Section are structured to reflect the cumulative staging of development:

- Phase 6 + Northern Industrial Parcel
- Phase 6 + Northern Industrial Parcel + Phase 7
- Phase 6 + Northern Industrial Parcel + Phase 7 + Phase 8 + Southern Industrial Parcel

The proposed connection at Kilbirnie Drive will extend from Quinn's Pointe Stage 5, which has not yet been constructed but is planned to be completed prior to advancement of Quinn's Pointe Phases 6–8. Accordingly, the analysis and results incorporate the units from Quinn's Pointe Stage 5 (herein referred to as "Stage 5").

Assessment of Adequacy of Public Services

Quinn's Pointe Phases 6-8

5.2.3 Domestic Water Demands

The water demands calculated for Quinn's Pointe Phases 6–8 reflect the unit count and land-use areas shown on the Concept Plan (Appendix A).

System level parameters (Table 27) were provided by the City of Ottawa at the onset of this project given that the population of Quinn's Pointe Phases 6–8 exceeds 3,000 (refer to Appendix G).

The maximum day demand shown in the table below is the sum of the average day demand, the water loss per connection and the outdoor water demand. The peak hour peaking factors shown below were provided by the City.

Table 27: Theoretical Water Consumption Rate

Land Use Type	Consumption Rate (L/cap/day)	Avg. Demand (L/unit/day)	Water Loss per Connection (L/unit/day)	Outdoor Water Demand (L/unit/day)	Max Day Demand (L/unit/day)	Peak Hour Demand
Single Family Homes @ 3.4 ppu	180	612	80	700	1,392	2.1 x Max
Executive and Avenue Towns @ 2.7 ppu	198	535	80	350	965	2.1 x Max
EMP (L/ha/day)	17,000	N/A	N/A	N/A	1.5 x Avg	1.8 x Max

When all the above parameters are applied to Quinn's Pointe Phases 6–8 along with an assumed constant demand of 4 L/s for the park block, the following water demands were calculated as shown in the detailed water demand spreadsheet in Appendix G.

Table 28: Quinn's pointe Phases 6-8 Water Demands

Phasing Scenario	Average Day Demand (L/s)	Maximum Day Demand (L/s)	Peak Hour Demand (L/s)
Phase 6	13.27	20.48	32.20
Phase 6, 7	14.70	22.82	36.91
Phase 6, 7, 8	17.61	27.88	46.94

*Note all scenarios include Stage 5.

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5.2.4 Required Fire Flow

Based on the City's Design Guidelines, municipal watermain shall be designed to meet fire flow requirements estimated based on the Fire Underwriters Survey (FUS) method. To evaluate the proposed water distribution system, a maximum fire flow of 13,000 L/min (217 L/s), based on the previous MSS Addendum, was targeted in the system for each phasing scenario. A lesser fire flow of 10,000 L/min (167 L/s) was also assessed based on the City's capped fire flow requirement for single family homes and row townhouses that achieve adequate rear yard spacing.

5.2.5 Internal Water Servicing

Several 300 mm diameter feeder mains are being proposed within Quinn's Pointe Phases 6–8. These trunk watermain locations have been determined based on the existing watermain surrounding Quinn's Pointe Phases 6–8 as well as the Concept Plan and hydraulic requirements. The following trunk watermain are proposed, as illustrated on the Conceptual Watermain Drawing (CMWM in Appendix H):

- 300 mm diameter watermain westerly extension on Kilbirnie Drive.
- 300 mm diameter watermain westerly extension on Moyard Drive with 300 mm diameter watermain upgrade (±40m) on Moyard Drive west of Lynn Coulter Street.
- 300 mm diameter watermain westerly extension on Cappamore Drive.
- 300 mm diameter watermain southerly extension on Elevation Road.
- 300 mm diameter watermain westerly extension on Barnsdale Road.
- 200 mm or 300 mm diameter watermain at the west side of Quinn's Pointe Phases 6–8 to provide looping between the extended watermain, such to avoid the creation of dead-end feeder mains.

5.2.6 Boundary Conditions

At the onset of this Study, boundary conditions were requested from the City for the existing pressure zone configuration and for the future South Urban Community (SUC) pressure zone reconfiguration. As the future SUC pressure zone reconfiguration is not expected to be completed until 2027, this Study analyzes results for both the existing and future pressure zones.

The boundary conditions received from the City (Appendix G) include connections to the existing watermain at Moyard Drive, Kilbirnie Drive, Obsidian Street, Elevation Road, Cappamore Drive, and Barnsdale Road. Boundary conditions were requested with and without a connection to Barnsdale Road as well as for the different phasing scenarios shown in Table 28. Although the boundary conditions indicate a marginal pressure increase with the Barnsdale Road connection, it is recommended that this looped connection be constructed for redundancy purposes given that the trunk sanitary sewer will also be extended westerly along this ROW. Thus, this Study only assesses the water servicing with the Barnsdale Road connection.

Boundary conditions received on November 24, 2025, were used for Phase 6 and Phase 6-7 scenarios and boundary conditions received on August 25, 2025, were used for the Phase 6-8 buildout scenario. It is noted that the current boundary conditions (received November 24, 2025) are generally reduced in pressure as compared to those presented in the previous studies

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carried out for these lands. This is attributed to the City's recent updates to their hydraulic water model which included updates to the pump controls at Barrhaven Pumping Station and updates to all water demands throughout the model, especially in neighbouring areas.

5.3 Water Servicing Analysis

The Hydraulic Network Analysis (HNA) was conducted using a model in WaterCAD® with a series of trunk watermains with a nominal diameter of 300 mm (modelled with an internal diameter of 297 mm) and 200 mm (modelled with an internal diameter of 204 mm). The 300 mm and 200 mm diameter watermains use a Hazen-Williams roughness coefficient (C-factor) of 120 and 110, respectively, per City Design Guidelines. A model schematic is presented in Appendix G that shows the proposed street centreline elevations and watermain sizing and layout within Quinn's Pointe Phases 6–8. The modelled watermain configuration follows the Master Watermain Drawing (CMWM in Appendix H). The HNA encompassed system performance under three key scenarios: maximum hydraulic grade line (HGL) conditions, peak hour demand conditions, and maximum day demand plus fire flow conditions. These scenarios were assessed under both existing and future conditions (i.e., without and with the pressure zone reconfiguration). The model result schematics are presented in Appendix G.

The model results are presented for the phasing as follows:

- **Phase 6** = Phase 6 + Northern Industrial Parcel
- **Phase 7** = Phase 6 + Northern Industrial Parcel + Phase 7
- **Phase 8** = Phase 6 + Northern Industrial Parcel + Phase 7 + Phase 8 + Southern Industrial Parcel

5.3.1 Maximum HGL

The maximum HGL simulation was assessed to confirm that system pressures are expected to adhere to the maximum pressure constraint of 552 kPa (80 psi). This simulation was carried out assuming no demands, which is a conservative approach.

5.3.1.1 Existing Conditions

Table 29 below shows the pressure ranges for each phase scenario.

Table 29: Model Results – Maximum HGL Existing Conditions

Phase	Minimum Pressure	Maximum Pressure
Phase 6	443 kPa (59.9 psi)	533 kPa (77.3 psi)
Phase 7	436 kPa (61.5 psi)	527 kPa (76.4 psi)
Phase 8	424 kPa (59.8 psi)	515 kPa (74.7 psi)

Simulations for Phases 6, 7 and 8 result in pressures that are below the maximum allowable system pressure of 552 kPa (80 psi). Accordingly, each phase scenario satisfies the applicable pressure requirements.

5.3.1.2 Future Conditions

Table 30 below shows the pressure ranges for each phase scenario.

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Table 30: Model Results – Maximum HGL Future Conditions

Phase	Minimum Pressure	Maximum Pressure
Phase 6	366 kPa (53.1 psi)	456 kPa (66.1psi)
Phase 7	366 kPa (53.1 psi)	456 kPa (66.1 psi)
Phase 8	362 kPa (52.5 psi)	452 kPa (65.6 psi)

Simulations for Phases 6, 7 and 8 result in pressures that are below the maximum allowable system pressure of 552 kPa (80 psi). Accordingly, each phase scenario satisfies the applicable pressure requirements.

5.3.2 Peak Hour

The peak hour simulation was assessed to confirm that system pressures generally comply with the normal operating conditions' minimum pressure constraint of 275 kPa (40 psi). This simulation was carried out by applying the peak hour demands in the model as summarized in Section 5.2.3.

5.3.2.1 Existing Conditions

Table 31 below shows the pressure ranges for each phase scenario.

Table 31: Model Results – Peak Hour Existing Conditions

Phase	Minimum Pressure	Maximum Pressure (kPa)
Phase 6	289 kPa (41.9 psi)	380 kPa (55.1 psi)
Phase 7	260 kPa (37.7 psi)	350 kPa (50.8 psi)
Phase 8	282 kPa (40.9 psi)	373 kPa (54.1 psi)

Under Phase 6 and Phase 8, the pressures under peak hour conditions exceed the minimum pressure requirements.

Under Phase 7, the lowest system pressures occur at the east end of the study area (Appendix G) and along Kilbirnie Drive. Although these pressures fall below the minimum recommended pressure of 275 kPa for normal operating conditions, they are primarily attributable to local topographic high points rather than excessive headlosses within the watermain network. Under the future pressure zone reconfiguration, pressures improve along both the east end of the study area and Kilbirnie Drive, as presented next in Section 5.3.2.2. Furthermore, the pressure zone modification will quite possibly occur prior to the construction of Phase 7. While pressures along Kilbirnie Drive during Phase 7 are marginally below guideline values for normal operating conditions, the latest City of Ottawa Design Guidelines do not explicitly define minimum pressures under peak hour conditions. This assessment will be refined at the detailed design stage when more definitive centreline elevation data becomes available.

5.3.2.2 Future Conditions

Table 32 below shows the pressure ranges for each phase scenario.

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Table 32: Model Results – Peak Hour Future Conditions

Phase	Minimum Pressure	Maximum Pressure
Phase 6	285 kPa (41.3psi)	376 kPa (54.5 psi)
Phase 7	282 kPa (40.9 psi)	373 kPa (54.1 psi)
Phase 8	274 kPa (39.7 psi)	365 kPa (52.9 psi)

All modeled phase scenarios result in pressures that exceed the maximum allowable system pressure of 275 kPa (40 psi). Accordingly, each phase scenario satisfies the applicable pressure requirements.

5.3.3 Maximum Day plus Fire Flow

To ensure adequate fire protection, a maximum day demand was simulated with a fire flow along the proposed watermain within Quinn's Pointe Phases 6–8. As per Section 5.2.4, a fire flow of 10,000 L/min (167 L/s) and 13,000 L/min (217 L/s) was used in this analysis and assessed under existing and future pressure zone configurations.

5.3.3.1 Existing Conditions Fire Flow (167 L/s)

Under existing pressure zone conditions, for Phases 6, 7 and 8, the proposed watermain network is expected to be capable of delivering fire flows of 167 L/s during a maximum day demand.

5.3.3.2 Future Conditions Fire Flow (167 L/s)

Under future pressure zone conditions, for Phases 6, 7 and 8, the proposed watermain network is expected to be capable of delivering fire flows of 167 L/s during a maximum day demand.

5.3.3.3 Existing Conditions Fire Flow (217 L/s)

Under existing pressure zone conditions, for the proposed watermain network for Phase 6, 7 and 8, the network is generally expected to be capable of delivering fire flows of 217 L/s during maximum day demand. Under existing conditions simulations, nodes J-90 and J-34 along the woodlot easement and node J-3 on Kilbirnie Drive are unable to provide 217 L/s.

The lower available fire flow at this node can be attributed to the higher topography in this localized area and the headlosses along the watermain. Currently, on Street 5 there are no residential units along this section of watermain which require 217 L/s. Along Kilbirnie Drive, specifically at J-3, there are single family units along this section of watermain which are expected to require only 167 L/s of fire flow. The required fire flow for the industrial block will be assessed at the detailed design stage for its site plan. It is noted that the available fire flow at these nodes improve with the future pressure zone reconfiguration as discussed in the next section.

5.3.3.4 Future Conditions Fire Flow (217 L/s)

Under future pressure zone conditions, for the proposed watermain network for Phase 6, 7 and 8, the network is generally expected to be capable of delivering fire flows of 217 L/s during

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maximum day demand. Under future pressure zone conditions simulations, nodes J-90 and J-34 along the woodlot easement are unable to provide 217 L/s.

Currently, there are no residential units requiring 217 L/s along these sections of watermain. The lower available fire flow at these nodes can be attributed to the higher topography in this localized area and the headlosses along the watermains.

5.4 Water Summary and Recommendations

The proposed watermain trunk servicing within Quinn's Pointe Phases 6–8 is shown on the Master Watermain Drawing CMWM in Appendix H. Water demands were calculated for Quinn's Pointe Phases 6–8 using system level parameters provided by the City of Ottawa. A maximum fire flow target of 10,000 L/min (167 L/s) and 13,000 L/min (217 L/s) was used in this Addendum. Based on the boundary conditions provided by the City and the HNA presented herein, the proposed trunk watermains are generally expected to provide adequate water supply to Quinn's Pointe Phases 6–8.

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6.0 Impact and Mitigation

6.1 Design, Construction and Operation

The analyses summarized in Sections 3.0 to 5.0 confirm that the preferred servicing strategy for Quinn's Pointe Phases 6–8 is the extension of existing municipal services from the Quinn's Pointe and Ridge Subdivisions. The recommended servicing framework is illustrated on Drawings CS1, CG1, CMST, CMSAN, CMWM, CPOND, CD1, and CD2.

The evaluation process of servicing strategies followed as part of the BSUEA MSS was extended to Quinn's Pointe Phases 6–8 which had included an assessment of several indicators related to design, construction and operation, social, natural, and economic factors. The MSS Addendum reviewed both infiltration and filtration-type LID solutions as part of a screening process which identified the preferred LID solutions and was moved forward in this AAPS. The impacts of the preferred servicing strategy on Quinn's Pointe Phases 6–8 and the mitigation required are outlined in this section.

6.1.1 Storm and Stormwater Management Servicing

The preferred storm sewer and stormwater management servicing strategy for Quinn's Pointe Phases 6–8 is based on a two-part approach, consisting of both infiltration and filtration measures:

- The infiltration-based solution builds upon the preferred servicing strategy identified in the 2018 BSUEA Master Servicing Study and includes: (i) the implementation of an Etobicoke Exfiltration System (EES) to promote recharge of the overburden aquifer, and (ii) infiltration of rooftop runoff through subsurface infiltration systems located on private properties.
- The filtration-based solution adopts a treatment train approach comprising a filtration dry pond supplemented with a mechanically treated device (MTD) to provide pre-treatment. This solution is proposed to service areas characterized by high groundwater conditions and areas where runoff quality may be influenced by roadway de-icing salts, such as collector roads.

With respect to the EES, one of the earliest known applications in Ontario was implemented in the former City of Etobicoke in 1993 as part of a retrofit project. While EES installations require enhanced inspection relative to conventional storm sewers, over approximately three decades of operation they have demonstrated reliable performance and maintenance requirements comparable to traditional systems. As part of the 2018 BSUEA MSS, several technical deliverables were prepared to support the acceptance of the EES; these documents are included in Appendix C of the MSS Addendum and should be reviewed in conjunction of this AAPS.

Prior to completion of the 2018 BSUEA MSS, the City confirmed that multiple EES installations form part of its existing municipally owned infrastructure. Since that time, EES has also been implemented within Quinn's Pointe Stages 2 through 4. Monitoring of the existing system indicates that it is operating in accordance with its design intent; however, elevated groundwater levels have the potential to limit system performance, as discussed in Section 3.7.

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The preferred filtration-based servicing strategy consists of a filtration-based dry pond supplemented by an MTD for pre-treatment. This approach has been implemented elsewhere in Ontario and has been accepted by the City as a future municipally owned stormwater management facility.

The storm sewer and stormwater management servicing strategy detailed in this AAPS and MSS Addendum provide the following benefits:

- Preservation of the overall pre-development infiltration regime (water budget).
- Preservation of the baseflow characteristics.
- Mitigate adverse impacts (water quality) on the esker by means of implementing an infiltration LID solution for the high-risk areas and areas where runoff will be impacted by chlorides.
- Reduce post-development peak flows, which in turn, will prevent increases in flood potential downstream.
- Protect downstream water quality by infiltrating or filtrating the first flush (22 mm storm) of the frequent storms, for parts of the Study Area where proper separation exists between the invert of the EES and groundwater levels.
- The servicing strategies meet the requirements of the CLI-ECA where the proposed solutions were developed based on the hierarchy described in Appendix A as follows:
 - ✓ Prioritize infiltration servicing solutions to the maximum extent while accounting for constraints.
 - ✓ Prioritize filtration servicing solutions to the maximum extent.

Since both types of solutions have been maximized, traditional stormwater management works such as a wet pond are not required nor recommended in this AAPS and MSS Addendum.

6.1.2 Sanitary Servicing

Sanitary servicing of Quinn's Pointe Phases 6–8 has been described in greater detail in Section 4.0. Wastewater servicing will be achieved via extensions of the existing wastewater collection system. Consequently, there is no need for additional infrastructure to increase downstream conveyance capacity or to implement pumping to achieve gravity connections. In terms of operation, the extension of the gravity systems within Quinn's Pointe Phases 6–8 will require standard operation and maintenance as for similar systems within the City.

6.1.3 Water Servicing

Water servicing of Quinn's Pointe Phases 6–8 has been described in greater detail in Section 5.0. Watermain servicing will be achieved via extensions of the existing and planned trunk watermain and supplemented by local watermain. Consequently, there is no need for additional infrastructure to increase capacity or to implement pumping systems for servicing higher elevations.

There is an imminent pressure zone reconfiguration that is envisioned about the same time as the initial stage of Quinn's Pointe Phases 6–8 will be made operational. The pressure characteristics were simulated and discussed in greater details in Section 5.0. In terms of

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operation, the extension of trunk watermain within Quinn's Pointe Phases 6–8 will require standard operation and maintenance as for similar systems within the City.

7.0 Trunk Storm and Sanitary Sewers and Watermains

Sections 3.0, 4.0 and 5.0 documented the technical analyses that were carried out in support of trunk infrastructure sizing within Quinn's Pointe Phases 6–8. None of the trunk infrastructure identified will be subject to any additional Class EA review nor Approval.

Given that the BSUEA MSS reviewed several servicing alternatives in accordance with the Class EA, these solutions were extended to this undertaking. Screening of infiltration and filtration servicing solutions were investigated and evaluated as part of a screening process, consistent with municipal Class EA. Linear infrastructure proposed as part of this Addendum is as follows:

7.1 Stormwater Management and Servicing Projects

Several trunk sewers of various diameters are proposed to service Quinn's Pointe Phases 6–8, as illustrated on (Drawing CMST). The key stormwater management components include the following:

- Trunk storm sewers along future Cappamore Drive, extending from the western limit near the southern Industrial Lands at Manhole STMH556 to the storm inlet at the dry pond (Manhole STMH600).
- Trunk storm sewers along Elevation Road, extending from Kilbirnie Drive at Manhole STMH513 through the pre-treatment device located along Street No. 17 to a flow-splitting manhole (labelled "Overflow") within Block 540, discharging to either Manhole STMH500A or STMH500.
- A pre-treatment mechanically treated device (MTD; Defender Model FD-8HC), located south of Manhole STMH502 near the western boundary of Block 540.
- A filtration dry pond with inlet at Manhole STMH500A, situated near the future intersection of Cappamore Drive and Elevation Road.
- A flow-splitter manhole labelled "Overflow", located east of the proposed MTD.
- An end-of-pipe dry pond encompassing the filtration dry pond, with dual inlets at Manholes STMH500 and STMH600.
- A network of local storm sewers of varying sizes servicing residential and industrial lands within Quinn's Pointe Phases 6–8.
- The existing 900 mm diameter storm sewer outlet and outlet control structure near the southeast corner of Block 540 will be retained. The structure will be retrofitted with low- and high-level orifices sized in accordance with the criteria summarized in Table 11 (above).
- On-site storage facilities within both vacant Blocks assumed as industrial usage, designed to detain the 100-year design storm onsite while releasing flows in accordance with the available minor system capacity.

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- Stormwater quality control measures within both vacant blocks (industrial usage), to be designed at the Site Plan stage to meet applicable water quality control requirements in effect at the time of approval.
- Implementation of an Etobicoke Exfiltration System (EES) along local roads, with a minimum 1.0 m separation from seasonally high groundwater levels. The EES is proposed within Areas 1A and 1B, as shown on Figure 9.

7.2 Sanitary Servicing Projects

Several trunk sanitary sewers are proposed to service Quinn's Pointe Phases 6–8, as illustrated on Drawing CMSAN). The proposed trunk sanitary sewer alignments include the following:

- Along Street No. 17, extending from the future Cappamore Drive extension at Manhole SAMH703 to Barnsdale Road at Manhole SAMH701.
- Along the Barnsdale Road right-of-way, extending from Manhole SAMH701 to the existing stub connection along Barnsdale Road near Sanitary Sewer Manhole SAEX04 within the Quinn's Pointe Subdivision.

7.3 Water Servicing Projects

Several 305 mm diameter watermain extensions are being proposed, along the following roadways:

- Future Kilbirnie Drive extension, from the eastern boundary of the existing CDP limit to the western limit of the retained woodlot.
- Future Cappamore Drive extension, from the eastern boundary of the existing CDP limit to a point adjacent to the southern portion of the Industrial Lands.
- Future Elevation Road extension, from the northern boundary of the existing CDP limit to future Cappamore Drive extension, and southerly along Street No. 17 to Barnsdale Road.
- Future Moyard Drive extension from the eastern limit of the CDP to Street No. 5, and southerly along Street No. 14 to the Cappamore Drive extension.

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8.0 Regulatory Approval Requirements and Development Applications

City Of Ottawa

This Addendum was prepared as an extension to the 2018 BSUEA MSS which was prepared as an Environmental Assessment Master Plan (i.e., Master Servicing Study). As such, this Addendum will require a technical review by several groups at the city of Ottawa. Ultimately, this Environmental Assessment Master Plan will require approvals under the *Planning Act*.

Given that this Addendum was prepared as an extension to the BSUEA MSS (2018) where several servicing strategies were analyzed under the Municipal Class Environmental Assessment, the proposed servicing strategies presented herein were developed in accordance with the planning principles of the Class Environmental Assessment. As previously noted, a screening process was carried out to assess various LID solutions with the preferred strategy scored the highest.

With Quinn's Pointe Phases 6–8 being the last growth area in this quadrant bound by Highway 416 and Barnsdale Road, the servicing strategies for both wastewater and water are limited, and consequently, servicing is based on the capacity of linear infrastructure bounding the Site.

Ministry of the Environment, Conservation and Parks

Servicing strategies evaluated herein and in the 2018 BSUEA MSS were analyzed under the Municipal Class Environmental Assessment and are in accordance with the planning principles of the Class Environmental Assessment. Given that pre-authorized conditions of the Municipal CLI-ECA have been issued under the Environmental Protection Act, infrastructure meeting these conditions will fulfill the approval requirements of section 53 of the Ontario Water Resources Act for sewage works (sanitary and stormwater).

Section 7.0 documented trunk infrastructure supporting the urbanization of Quinn's Pointe Phases 6–8 Concept Plan. All infrastructure listed in Section 7.1, 0 and 7.3 has been found to meet MECP's pre-authorized criteria under the CLI-ECA framework. Therefore, no additional approval from MECP is required.

Ontario Drainage Act

Any modifications to the hydrological regime or alteration to the serviced area of a municipal drain within the Mud Creek subwatershed, will require that it be documented in an Engineers Report, which is to be prepared by the Engineer of Record in accordance with the Ontario Drainage Act (Section 78, Chapter D.17, R.S.O. 1990).

RCI, the appointed Drainage Engineers for this project, will be coordinating this effort upon the approval of the MSS Addendum by council. Given that a partial disconnection is being proposed, the process will be on-going in parallel and will not interfere with the completion of this MSS Addendum nor with the approval of the initial phase of Quinn's Pointe Phases 6–8.

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Ministry of Transportation of Ontario

Although no formal approval is required at the MSS level, acceptance by the Ministry of Transportation of Ontario (MTO) of the proposed drainage alteration was sought. A meeting was held on January 26, 2026, with representatives from MTO, to discuss the project as Borrisokane Road is part of MTO's Highway 416 right-of-way, and future interchange. Comments were issued by MTO on May 15, 2026, and a response to those comments submitted on May 22, 2026, for MTO's review. Shortly after the above, the last version of the MSS Addendum was submitted on May 27, 2026.

As noted in Section 0, there is a planned interchange in the southwest quadrant of the Barnsdale Road and Highway 416/Borrisokane Road intersection. A partial disconnection is being proposed which will alter the hydrological regime of the Borrisokane Road open ditch system. As previously noted, the net impact of the drainage modification results in a reduction in both peak flows and runoff volumes released to this open ditch system.

Notwithstanding the above, a separate Stormwater Management Report was prepared and submitted under separate cover to the Ministry of Transportation of Ontario (MTO) in support of a Highway Corridor Management (HCM) permit application. This stand-alone SWMR focused specifically on off-site discharge conditions—both pre- and post-development—directed to the Borrisokane Road roadside ditch system, part of Highway's 416 corridor, based on information extracted from this Report. This SWMR accompanies the Application under an Official Plan Amendment.

Rideau Valley Conservation

There was no formal approval of the MSS Addendum by the RVCA. However, the RVCA may review future development applications within Quinn's Pointe Phases 6–8 to ensure that it is consistent with the conservation, development and management of natural resources. As part of this process, the RVCA will ensure that property and public are protected from hazards such as flooding and erosion. However, given the storm servicing strategy presented herein, flooding and erosion are not of concern since off-site discharges are less than those under existing conditions. Similar to the Existing Conditions Report (JLR, 2025), the City of Ottawa, to their discretion, will circulate the final version of this MSS Addendum (May 27, 2026) to the RVCA as information. Similarly, this AAPS may be shared to the RVCA as information, at the discretion of the city.

Future Development Applications

With the upcoming council approval of the MSS Addendum, the lifting of the neighborhood overlay will occur. This is in the process as the document will be posted to the Planning Committee.

The alignment and sizing of infrastructure, as shown herein, is subject to change as part of *Planning Act* approvals and would be treated as minor changes as part of the Draft Plan approvals as the Concept Plan included in Appendix A of this AAPS has been slightly adjusted.

Subsequent development applications will be made in accordance with the policies outlined in the city of Ottawa Official Plan. The specific studies required under the development approval

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process are determined at the commencement of the process and provided at the pre-application consultation meeting with City staff. Specific studies may include:

- Geotechnical studies
- Community transportation and traffic studies
- Noise studies
- Site Servicing Study, including stormwater management
- Environmental Impact Studies.

Several of the above-noted Studies have already been completed as part of the Concept Plan Approval process and are unlikely require an update. A planning rationale outlining how the development meets the intent of the overarching plans for the area will also be required to support the Application under Official Plan Amendment.

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9.0 Servicing Options Assessment

9.1 General

It is standard practice that a Servicing Options Report be made a condition of approval under an Official Plan Amendment. As documented in the MSS Addendum and this AAPS, Quinn's Pointe Phases 6–8 represents the final developable lands within the southeast quadrant of the Barnsdale Road / Highway 416 / Borrisokane Road area.

Under the Municipal Class Environmental Assessment framework, the assessment of alternative Servicing Options is, therefore, inherently limited. While a comprehensive range of alternatives was evaluated as part of the 2018 BSUEA Master Servicing Study, the MSS Addendum (May 27, 2026) and updated in this AAPS, it was confirmed that servicing for Quinn's Pointe Phases 6–8 must rely on the extension and optimization of existing municipal infrastructure, with minimal flexibility for alternative approaches.

9.2 Storm and Stormwater Management Servicing

Storm servicing options are constrained by the absence of a direct receiving watercourse and reliance on downstream municipal systems. The AAPS and MSS Addendum confirms that Quinn's Pointe Phases 6–8 is not directly connected to a natural watercourse and must discharge via existing storm sewer networks. Evaluation of potential outlet options determined that alternative systems, including the Elevation Road network, do not have sufficient residual capacity, effectively limiting storm discharge to the Quinn's Pointe system.

Within this constrained framework, servicing options are limited to a hybrid approach consisting of a dual-drainage system combined with Low Impact Development (LID) measures. Infiltration-based servicing, primarily through Etobicoke Exfiltration Systems (EES), is implemented to the maximum extent feasible, consistent with the CLI-ECA hierarchy. However, site-specific constraints—including groundwater levels, source protection restrictions, and high-risk land uses—limit the extent of infiltration and necessitate filtration-based alternatives. Accordingly, stormwater servicing is restricted to a defined treatment train approach combining pre-treatment MTDs and a filtration dry pond for constraint areas as it was shown in the MSS Addendum and AAPS.

9.3 Wastewater Servicing

Wastewater servicing alternatives are similarly limited. The MSS Addendum and AAPS confirms that extension of the existing 375 mm sanitary sewer along Barnsdale Road represents the only viable servicing outlet with sufficient residual capacity and appropriate hydraulic grade. Other potential connections were assessed and determined to be either capacity constrained or hydraulically infeasible. As a result, wastewater servicing for Quinn's Pointe Phases 6–8 is dependent on a single downstream outlet, with no practical alternative servicing routes available.

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9.4 Water Servicing

Water servicing is also constrained to extensions of the existing municipal water distribution system from the Quinn's Pointe and Ridge Subdivisions. As identified in the MSS Addendum and AAPS, no alternative feedermain corridors are available within or adjacent to the Study Area. Servicing must therefore rely on existing and planned trunk watermain infrastructure and established pressure zone configurations. While overall capacity is adequate, the absence of alternative supply routes limits redundancy and flexibility in system configuration.

9.5 Summary

Overall, this AAPS and MSS Addendum demonstrated that servicing alternatives for Quinn's Pointe Phases 6–8 are highly constrained across all disciplines. The recommended servicing approach relied on strategic extensions of existing infrastructure, maximization of LID measures where feasible, and implementation of a treatment train approach where required.

These strategies represent the only viable and supportable servicing solutions based on confirmed system capacity, hydraulic feasibility, and compliance with City of Ottawa standards and the CLI-ECA framework. As a result, a stand-alone Servicing Options Report was not prepared to support the Official Plan Amendment.

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10.0 Infrastructure Phasing and Implementation

10.1 Adaptability

The MSS Addendum allowed lifting of overlay. In tandem with the MSS Addendum, this AAPS will support the Draft Plan of Subdivision process. It is, however, acknowledged that there may be changes prior to the construction being initiated/completed which would be documented as part of the subdivision approval process.

The alignment and sizing of infrastructure, as depicted on Drawings CS1, CMST, CMSAN and CMWM and SWM projects depicted on Drawings CPOND, CD1 and CD2, is subject to change as part of *Planning Act* approvals and would be treated as minor changes to this AAPS.

10.2 Detailed Design

The objective of the MSS Addendum and this AAPS is to provide a high-level servicing solution, which can demonstrate that development is serviceable/feasible in accordance with municipal and provincial standards. Street-by-street detailed design and grading would still be required at the Plan of Subdivision.

10.3 Staging of Infrastructure

The development of Quinn's Pointe Phases 6–8 will proceed from the eastern boundary of the Study Area, adjacent to the Quinn's Pointe Subdivision where infrastructure currently exists. Given that the Quinn's Pointe Stage 5 has yet been constructed, it could potentially, to the discretion of Minto, be combined with the initial phase of development within Quinn's Pointe Phases 6–8, referred to as Stage 6.

The intended phasing limits for Quinn's Pointe Phases 6–8 is shown on the Concept Plan (Appendix A). As illustrated, the implementation of Quinn's Pointe Phases 6–8 will proceed into three (3) successive phases; the initial phase noted as Stage 6, with two (2) subsequent phases noted as Stages 7 and 8. At this time, Minto plans to proceed with the development of the Industrial lands once Stages 6-8 have been completed.

10.4 Construction of Infrastructure

Infrastructure, including storm, water and wastewater for Stage 6 will consist of the following:

- **Etobicoke Exfiltration System:** Street No 1, Street No. 2, Street No. 3, and Street No.4 as well as along Moyard Drive and Street No. 5, Street No. 6, Street No. 7.
- **Local and Trunk Storm sewers:** All storm sewers will be constructed along all roadways within Stage 6, with the addition of the east leg of Street 17.
- **Watermains:** All local and trunk watermains will be constructed along the roadways and along the east leg of Street 17, as well as along the easement Blocks 541 and 542.
- **Sanitary Sewers:** All local and trunk sanitary sewers will be constructed along the roadways and along the east leg of Street 17 and Barnsdale Road.
- **End-of-Pipe Facility:** All the drainage components and associated infrastructure identified within or in the vicinity of Block 540, including the pre-treatment MTD, filtration dry pond and underlain subdrain system, and the dry pond will be constructed as part of the Stage

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6 infrastructure work while the existing outlet structure and storm sewer will be maintained.

10.5 Monitoring

As the implementation of infrastructure proceeds with each of the phases of development, it is recommended that the on-going EES monitoring being conducted in Quinn's Pointe Subdivision be extended into Quinn's Pointe Phases 6–8 once it has been completed in Quinn's Pointe Subdivision. At such time, monitors would be relocated initially in Stage 6 to gather additional data.

10.6 Implementation and Capacity Analysis

The initial phases of development can represent the most vulnerable period from a servicing and operational perspective, as only a portion of the ultimate infrastructure network is in place. During these early stages, temporary conditions such as reduced system redundancy, interim drainage patterns, and incomplete downstream works may exist. Accordingly, the proposed implementation strategy is intended to ensure that each phase of development functions independently and safely, with adequate servicing capacity and controls always provided until the full build-out of the infrastructure is achieved.

10.6.1.1 Storm Sewers and End-of-Pipe Facility

Storm sewers and ultimate end-of-pipe facility, including the MTD, by-pass manhole, filtration dry pond and dry pond including its existing outlet, are proposed to be constructed as part of Stage 6. Given that all drainage works in Stages 6 is intended to serve additional land, to account for future Stage 7, Stage 8, and Industria Lands, there are no capacity limitations for any of the storm sewers within Stage 6.

In terms of interim drainage conditions, Quinn's Pointe Phases 6–8 is bounded by two (2) existing subdivisions, and by Barnsdale Road and Borrisokane Road. Therefore, there is no interim drainage conditions apart from the installation of temporary ditch inlets at the western and southern boundaries of Stage 6. The interim drainage condition and temporary infrastructure, including ICDs, will be developed as part of detailed design of Stage 6.

10.6.1.2 Sanitary Sewers

Trunk sanitary sewers that are planned to be constructed as part of Stage 6, are intended to serve additional land, to account for future Stage 7, Stage 8 and Industrial Lands, when constructed. Hence, there are no capacity limitations for any of the sanitary sewers in Stage 6.

Under the interim condition, temporary ICDs will be installed along any sanitary sewer that will service additional lands to limit wastewater flows that could be intercepted. Temporary flow restriction and details will be reviewed as part of detailed design of Stage 6.

10.6.1.3 Water Distribution Analysis

Water distribution systems are often vulnerable during the initial phase of a given development. As a result, the performance of each additional interim phase was evaluated under maximum HGL, peak hour and maximum day plus fire flow. It should be noted that as part of Phase 6, all

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connection points to the existing municipal distribution system will be utilized via trunk watermain extensions.

The Water HNA and modelling assessment presented in Section 5.0 demonstrates the following:

- Under Maximum HGL existing conditions, each phase is expected to adhere to the maximum pressure constraint of 552 kPa (80 psi).
- Under Maximum HGL future conditions, each phase is expected to adhere to the maximum pressure constraint of 552 kPa (80 psi).
- Under Peak Hour existing conditions, Phase 6 and Phase 8 are expected to adhere to the minimum pressure constraint of 275 kPa (40 psi). Phase 7 expects a few localized low-pressure areas, but these results are anticipated to be temporary if not trivial based on the pressure zone modification.
- Under Peak Hour future conditions, each phase is generally expected to adhere to the minimum pressure constraint of 275 kPa (40 psi).
- Under Maximum Day plus Fire Flow existing and future conditions, each phase is expected to provide 167 L/s at all nodes, and 217 L/s at most nodes. All required fire flows can be met with the proposed trunk watermains for each phase.

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11.0 Conclusions

This AAPS in conjunction with the MSS Addendum were prepared to support an Official Plan Amendment. The 2018 BSUEA MSS had reviewed a series of servicing strategies in accordance with the Class Environment Assessment (Class EA) and was completed in accordance with the Integrated Planning Process.

A screening process was carried out as documented in the MSS Addendum and shall be consulted for further details. Infiltration LID and filtration LID alternatives were reviewed and documented in the MSS Addendum. The infiltration solution adopted as part of the 2018 BSUEA MSS was extended to Quinn's Pointe Phases 6–8 while the proposed filtration LID solution that scored the highest has been recommended.

The MSS Addendum have identified a recommended high-level servicing strategy for storm, wastewater and water servicing which were moved forward as part of this AAPS. These strategies were revised to suit the revised Concept Plan (Appendix D).

The high-level servicing strategy developed herein demonstrates the feasibility of servicing the proposed Quinn's Pointe Phases 6–8. The design criteria utilized to develop the servicing strategies shall be used to develop detailed design. It is anticipated that the detailed design to be carried out in accordance with this AAPS and could be adapted for any alterations to the road and layout configurations. The AAPS has been developed with this flexibility in the design.

Principal findings of this AAPS and MSS Addendum are set out below:

Storm Drainage and Stormwater Management

- The design of the storm drainage system has been undertaken using the dual-drainage approach. This AAPS has set out the design criteria for future draft plan and/or site plan applications for Quinn's Pointe Phases 6–8. The design criteria are consistent with current design standards.
- A Master Grading Plan was developed based on existing and proposed grades and utilized to develop the Master Storm Drainage Plan.
- The storm servicing solution developed for Quinn's Pointe Phases 6–8 is being shown on the Key Servicing Plan and reflects the adopted servicing strategy.
- The infiltration servicing strategy was developed as part of MSS Addendum to the maximum extent while recognizing opportunities and constraints listed in Table A2 of the CLI-ECA. The strategy incorporates the Etobicoke Exfiltration System (EES), the recommended solution identified as part of the 2018 BSUEA MSS and supplemented by rooftop infiltration using sub-surface infiltration measures within the Industrial Lands.
- Both infiltration measures were sized to capture and infiltrate the 22 mm design storm event.
- Five (5) filtration-type servicing solutions were developed/evaluated as part of the MSS Addendum for the areas impacted by the constraints identified in the CLI-ECA. This information can be reviewed for further details.
- The recommended filtration solution that scored the highest, Alternative 5, is being proposed and moved forward in this AAPS. For other details, the MSS Addendum should be reviewed in conjunction with this AAPS.

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- This LID solution is based on a treatment train approach where pre-treatment is being proposed by means of a MTD that is supplemented by a filtration dry pond, with bio-retention media equipped with an underlain subdrain system surrounded by clear stone envelope, geotextile. Due to roadway salts, the filtration dry pond will be lined with a choked later to minimize migration of roadway salts to the aquifer.
- Given the same principle as an EES, the filtration dry pond was also sized to capture and filtrate the 4-hour 22 mm storm.
- Both proposed infiltration and filtration solutions were found to exceed the requirements of Table 3.2 of MECP's SWMPDM for an 80% TSS removal.
- The post-development water budget with proposed infiltration measures showed comparable infiltration identified under the pre-development water budget.
- The downstream storm boundary conditions or targeted flow allowances developed in the MSS Addendum and moved forward in this AAPS.
- A pre- and post-development analysis was completed to assess peak flows and runoff volumes along both Borrisokane Road (system owned by MTO) and Barnsdale Road open ditch systems as well as at the upper reach of the Thomas Baxter Municipal Drain. The analysis showed a reduction in both peak flows and runoff volumes along the open ditches noted above for all storm events. Hence, no adverse impacts were found along both open ditch systems.
- A Stand-alone Stormwater Management Report is being submitted under separate cover to the MTO to support a Highway Corridor Management (HCM) permit application. This Report focuses specifically on off-site discharge conditions—both pre- and post-development—directed to Borrisokane Road for the purpose of an HCM permit.
- Simulation results along both Barnsdale Road and Thomas Baxter Municipal Drain have been assessed to not cause adverse impacts by the Team's biologist as reported in the Environmental Impact Statement for the S1 Lands.
- The trunk sewer network was conceptually sized based on the revised Concept Plan and modelled to ensure that the hydraulic grade line (HGL) elevations along the system maintained a 300 mm freeboard, which corresponds to 2 meters below proposed centreline elevations.
- The major overland system was simulated to assess ponding depths (static and dynamic) against the requirements of the SDG (December 2025) being that it shall not exceed 350 mm depth.
- Allowable minor system release rates were set at the required storm event, and future design should maintain the same release rate criteria.
- The stormwater management solution contained in Block 540 or proximity includes a pre-treatment MTD, a filtration dry pond which will be supplemented by an expanded dry pond surrounding this facility, replacing the existing spill-over area constructed as part of the Quinn's Pointe Subdivision.
- Post-development peak flows discharged in the existing 900 mm diameter storm outlet are maintained below those reported in the Site Servicing Report of the Quinn's Pointe Subdivisions.
- A significant area of Quinn's Pointe Phases 6–8 is proposed to be disconnected from the Thomas Baxter Municipal Drain, which will result in a reduction in both peak flows and runoff volumes for all storm events. An update to the Engineers Report will be prepared by the appointed Drainage Engineer.

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Wastewater Servicing

- The residual capacities in the surrounding wastewater services were assessed to determine viable servicing outlets to accommodate wastewater flows from Quinn's Pointe Phases 6–8.
- Out of the potential connections, the extension of the existing Barnsdale Road 375 mm sanitary sewer was the only viable outlet having sufficient residual capacity and grade to accommodate wastewater flows from Quinn's Pointe Phases 6–8.
- The existing constructed wastewater system network is capable to accommodate servicing for Quinn's Pointe Phases 6–8.
- The MSS Addendum contains design criteria which are to be used for Quinn's Pointe Phases 6–8 in future draft plan and utilized for this AAPS. The design criteria are consistent with current design standards.
- The proposed wastewater system layout has been functionally sized based on the Draft Plan of Subdivision and latest Concept Plan.

Water Distribution

- Quinn's Pointe Phases 6–8 can be serviced under current conditions with watermain extensions from the existing surrounding watermains within both the Quinn's Pointe and the Ridge Subdivisions. It can also be serviced from the reconfigured pressure zone anticipated for implementation in 2027.
- The analysis confirmed that there are no upgrades to the existing nor to the planned distribution network identified in the latest Infrastructure Master Plan.
- Demand allocation for water supply servicing has been derived based on the Concept Plan and associated densities. The allocations were based on system level water consumption rates, and peaking factors provided by the City as the population for Quinn's Pointe Phases 6–8 exceeds 3,000.
- Trunk watermain sizing has been carried out based on the Concept Plan/Draft Plan and reflects trunk watermain extensions at the boundary of Quinn's Pointe Phases 6–8.
- The MSS Addendum has set design criteria which was used for the Draft Plan for lands included in Quinn's Pointe Phases 6–8. The design criteria are consistent with current design standards.
- Individual potable water assessments will be carried out as part of the detailed design of the various phases of the subdivision as well as the Site Plans for the Industrial Lands using parameters consistent with the Water Distribution Design Guidelines (December 2025).

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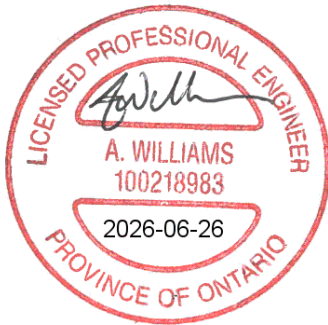
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Assessment of Adequacy of Public Services Quinn's Pointe Phases 6-8

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Appendix A

Concept Plan 54 (Rev 17) and Draft Plan of Subdivision

Appendix B

Paterson Memorandum and Bathymetric Survey

Appendix C

Functional Level Information
of Preferred Solution

Appendix D

Figure (EES separation) and
Storm Sewer Design Sheets

Appendix E

PCSWMM Model Inputs and Results and Schematics

Appendix F

Sanitary Sewer Design Sheets and Calculations

Appendix G

Water Distribution

Appendix H

Drawings

Appendix I

Operation and Maintenance Manual (EES) and Modelling Files

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