



665 Eliza Street, Arthur

**Functional Servicing and Stormwater
Management Report**

March 2025

Submitted by:

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Submission History

Submission	Date	In Support Of	Distributed To
1 st	February, 2025	Draft Plan of Subdivision	Municipality of Wellington North, Grand River Conservation Authority



1.0 Introduction

SCS Consulting Group Ltd. has been retained by Tribute/Sorbara Arthur Holdings Inc. to prepare a Functional Servicing and Stormwater Management Report (FSSR) for a proposed development at 665 Eliza Street and the property bounded by Wells Street, Maccauley Street and Eliza Street in Arthur with no municipal address, located in the Township of Wellington North.

1.1 Purpose

This FSSR has been prepared in support of a Plan of Subdivision for the proposed development. The Draft Plan of Subdivision is provided in **Appendix A**. The proposed development consists of the following land uses:

- low density residential,
- medium density residential,
- parks,
- pumping station,
- stormwater management (SWM) pond blocks, and
- proposed roads.

The proposed development consists of an east parcel, 665 Eliza Street, and west parcel, property bounded by Wells Street, Maccauley Street and Eliza Street in Arthur, with no municipal address. The proposed development is separated by Eliza Street / County Road 14 running north-south.

The purpose of this report is to demonstrate that the development can be graded and serviced in accordance with the Township of Wellington North, Grand River Conservation Authority (GRCA), and the Ministry of Environment, Conservation and Parks (MECP) design criteria.

1.2 Study Area

The study area (see **Figure 1.1**) is approximately 55.34 ha in size and is bound by:

- Existing agricultural lands and single dwelling farm houses to the north;
- Existing agricultural lands and Wells Street extension to the west;
- Existing agricultural lands to the east;
- Existing agricultural, existing residential development, future Maccauley Street, and future Cachet development area to the south; and
- Eliza Street running north-south which separates the study area into an east and west parcel.



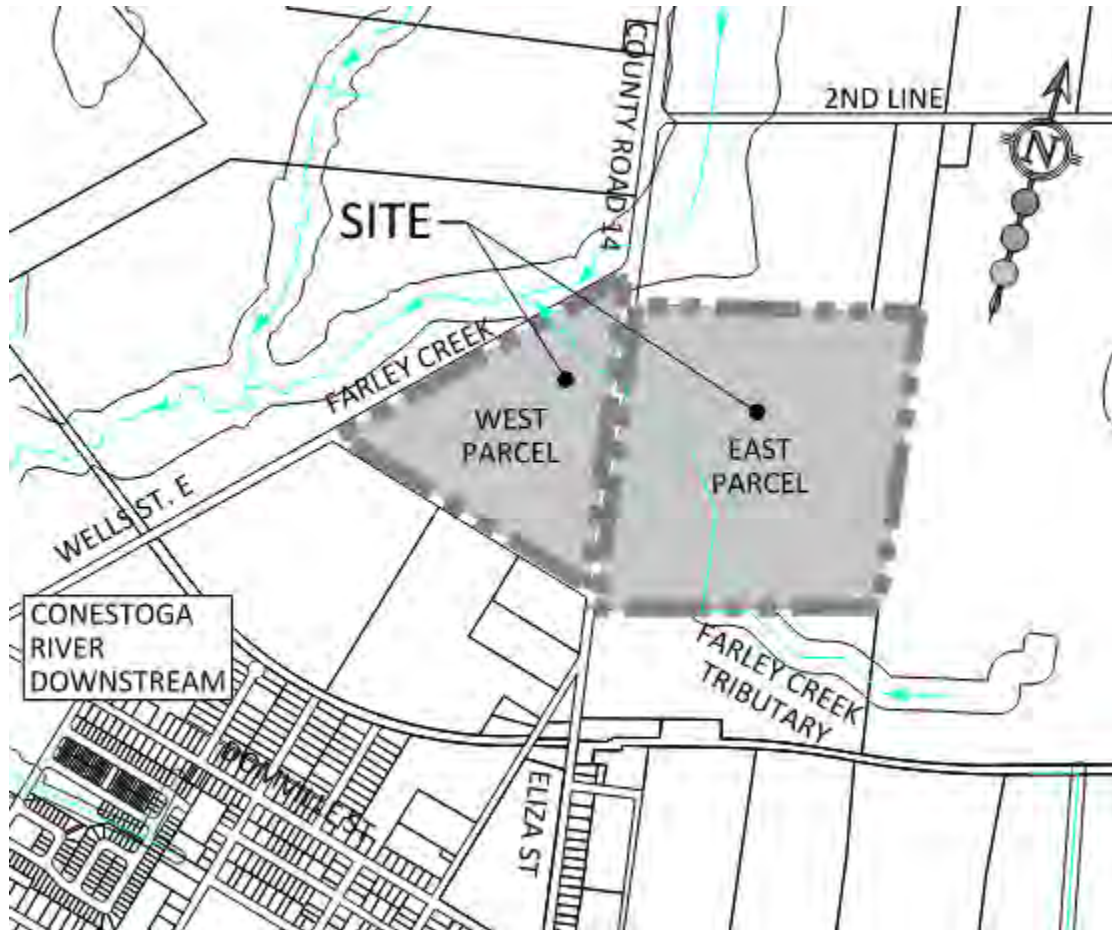


Figure 1.1: Site Location Plan

The existing subject lands are comprised of agricultural land. The proposed development is located within the Grand River Watershed in the Township of Wellington North. A tributary of Farley Creek runs south to north-west through the study area.

1.3 Background Information

In preparation of the servicing and stormwater management (SWM) strategies, the following design guidelines and standards were used:

- Ministry of Environment, Conservation and Parks (MECP) Municipal Consolidated Linear Infrastructure Environmental Compliance Approvals, June 2023;
- Municipal Servicing Standards, Township of Wellington North, March 2023;
- Ministry of Environment (MOE) Stormwater Management Planning and Design Manual, March 2003; and,



- Ministry of Transportation (MTO) Drainage Management Manual, 1997.

The servicing and SWM strategies in this report are based on the following reports:

- Land Use Compatibility Study, prepared by HGC Acoustics, dated February 2025;
- Arthur Water and Wastewater Servicing Summary Report, prepared by DLW Engineering Services, dated January 2025;
- Meander Belt Width Assessment – 665 Eiza Street, prepared by GeoProcess Research Associates, dated January, 2025;
- Schedule 'B' Municipal Class Environmental Assessment Arthur Water System Supply Redundancy & Storage, prepared by Triton Engineering, dated November 2024;
- Draft Hydrogeological Investigation – Proposed Residential Development 665 Eliza Street, prepared by GEMTEC Consulting Engineers and Scientists Limited, dated October 2024;
- Geotechnical Site Investigation – Proposed Residential Development 665 Eliza Street, prepared by GEMTEC Consulting Engineers and Scientists Limited, dated September 2024;
- Geotechnical Site Investigation – Proposed Residential Development Southwest Quadrant of Eliza Street and Wells Street East, prepared by GEMTEC Consulting Engineers and Scientists Limited, dated September 2024; and

The servicing and SWM strategies in this report are based on the following approved reports and as-recorded drawings:

- Arthur Water and Sanitary Systems Technical Study, prepared by Triton Engineering Services Limited, dated September 2020;
- Village of Arthur Domville Street Drawing 1 of 6, File No. 98038, Plan and Profile of Reconstruction and Watermain from Wells Street to STA. 1+340, dated July 1998;
- Village of Arthur Domville Street Drawing 2 of 6, File No. 98038, Plan and Profile of Reconstruction and Sanitary Sewer from STA. 1+340 to STA. Conestoga Street, dated July 1998;
- Village of Arthur Preston Street Drawing 3 of 6, File No. 98038, Plan and Profile of Reconstruction watermain and Sanitary Sewer from Domville to STA. 3+220, dated July 1998;
- Village of Arthur Easement Drawing 4 of 6, File No. 98038, Plan and Profile of Sanitary Sewer from Domville St. to STA. 4+220, dated July 1998;
- Village of Arthur Easement Drawing 5 of 6, File No. 98038, Plan and Profile of Sanitary Sewer from STA. 4+220 to STA. 4+390, dated July 1998; and



- Village of Arthur Wells Street Drawing 6 of 6, File No. 98038, Plan and Profile of Grading and Paving from STA. 2+020 to STA. 2+300, dated July 1998.

Refer to excerpts in **Appendix B**.



2.0 Geotechnical and Hydrogeological Conditions

2.1 Soils

The soil classifications for the subject site were identified using the Ontario Soil Survey Complex from OMAFRA and land uses visible in recent aerial photography. The mapping identifies that the soils within the study limits are Perth Silt Loam. According to the Design Flood Estimation Design Chart H2-6A, the soils are considered as Hydrologic Soil Group C. This is consistent with the Geotechnical Investigation prepared by GEMTEC, 2024 that notes the predominant soil type is Silt Loam, which is a Hydrologic Soil Group C according to the MTO Drainage Management Manual (1997) Design Chart 1.08.

The Soil Conservation Service Curve Numbers (CN) have been determined for the subject site based on this information. Refer to **Appendix B1** for the soils mapping, excerpts from the Geotechnical Investigation (GEMTEC, 2024) and the calculations for the CN.

GEMTEC have also completed hydraulic conductivity estimates which found a range of estimated hydraulic conductivities of 5×10^{-9} to 6×10^{-8} m/s.

2.2 Groundwater

Groundwater levels were measured by GEMTEC in monitoring wells across the subject site with readings taken from July 2024 to August 2024. The groundwater levels ranged from 1.2 to 3.1 m below existing ground.

Groundwater monitoring is ongoing and will continue to ensure the spring high groundwater level is observed. Based on the groundwater levels collected to date, the seasonally high groundwater elevations are approximately 1.2 m below existing ground. Refer to **Appendix B2** for excerpts from the Draft Hydrogeological Investigation (GEMTEC, 2024) including the groundwater monitoring results.



3.0 Topography and Grading

3.1 Existing Conditions

3.1.1 Topography

The existing topography has slopes in the range of 0.40% to 20%. The ground surface elevations through the west parcel of the study area range from approximately 463.0 m in the southeast corner sloping northwest towards the tributary of Farley Creek to an elevation of approximately 457.8 m. The ground surface elevations through the east parcel of the study area range from approximately 466.0 m in the northeast corner sloping west towards the tributary of Farley Creek to an elevation of approximately 460.0 m. Refer to **Figure 3.1** for the existing topography for the subject site.

3.1.2 Floodplain

The existing subject lands contain a tributary of Farley Creek running generally southeast-northwest through the subject site which crosses under Eliza Street via an existing 2750 mm diameter CSP culvert and then crosses under Wells Street via an existing 2000 mm diameter CSP culvert. Downstream of Wells Street, the tributary outlets to Farley Creek, which ultimately connects into the Conestogo River approximately 3 km downstream. The tributary is regulated by Grand River Conservation Authority (GRCA) under Ontario Regulation (O.Reg.) 41/24 and as such a permit will be required for site development.

Farley Creek and the tributary through the development do not have engineered regulatory (Regional Storm) floodplain mapping. The total drainage area to the tributary is approximately 199.26 ha (**Figure 3.2**); therefore, a detailed floodplain analysis is required to establish regulatory (Regional Storm) flood elevations and delineate the regulatory floodline in order to set the limits of development. SCS Consulting Group has prepared existing conditions hydrology and hydraulic models to define the existing Regional Storm floodplain. This information was previously submitted to GRCA in a letter dated October 4, 2024, and included in **Appendix C1**.

To determine the Regional Storm peak flow through the tributary, a Visual Otthymo (VO6.2) model was prepared. The total upstream drainage area to the connection to Farley Creek was determined utilizing detailed topographic information for the proposed development as well as GeoHub LIDAR external topographic information. As noted above, the total drainage area at the connection of the tributary to Farley Creek is 199.26 ha (**Figure 3.2**). The Regional Storm event (Hurricane Hazel) produced a Regional flow of approximately 13.55 m³/s for the total drainage area at the connection to Farley



Creek. To be conservative in estimated flood elevations, this Regional flow was utilized as a single flow node to assess the floodline throughout the development and the Eliza Street and Wells Street crossings. In the future, the hydrology model may be evaluated further to discretize the drainage area upstream of Eliza Street and introduce a flow node change to further refine the floodline. Refer to the hydrology model within **Appendix C2**.

To determine the existing Regional Storm floodplain, a hydraulic model (HEC-RAS version 6.4.1) was created. J.D. Barnes completed detailed topographic survey dated August 2024, which was utilized to develop cross-sections for the hydraulic model within the confines of the proposed development. As noted above, there are two (2) culvert crossings of the tributary, Eliza Street and Wells Street. The culvert sizes and inverts are based on the detailed J.D. Barnes topographical survey dated August 2024. As mentioned above, Geohub LIDAR topographic survey information was utilized for areas external to the subject site. The detailed topographic survey information was completed in CGVD 28-1978 which is consistent with the GRCA floodplain mapping criteria. The Geohub LIDAR topographic information was completed in CGVD 2013; however, as outlined in the Existing Floodplain Modelling letter to the GRCA, **Appendix C1**, the difference in vertical datum is considered insignificant.

Conestoga River has an estimated Regional Storm floodline per the GRCA regulatory floodline information available to date. As outlined in the Existing Floodplain Modelling letter to the GRCA, the downstream boundary condition was set based on of utilizing the GRCA provided Conestoga River hydraulic model, the Regional Storm floodline elevation at the junction of Farley Creek with the tributary was estimated at 457.09 m. However, since this letter was submitted, SCS obtained the latest Conestoga River hydraulic model and has updated the approach to set the downstream boundary condition to be more accurate. The downstream boundary condition was set based on the flow depth within Farley Creek at the junction of Farley Creek and the tributary. The flows at the junction of Farley Creek and the tributary were determined utilizing based on the MTO Equation 8.31 transposition of flood discharges and utilized the Conestoga River approved hydraulic modelling. Refer to calculations in **Appendix C3** for the flows. The flow depth at the junction was determined utilizing Flowmaster based on a cross-section of the junction from GeoHub LIDAR topographic survey information. The flow depths for the 2 – 100 year and Regional Storm event were utilized as the downstream boundary condition. Refer to calculations in **Appendix C3**.

The hydraulic model was simulated for the Regional storm event and the existing conditions floodline delineated for the subject site. Refer to **Figure 3.3** for the floodline and **Appendix C4** for the hydraulic model. As shown on **Figure 3.3**, there is a backwater



condition created by the existing Eliza Street culvert. Due to the backwater condition and the existing site topography, a portion of the Regional Storm floodplain spills north, through the adjacent property to Farley Creek. The floodplain modelling has not taken into account the spill condition, and thus the downstream flood elevations on the tributary are conservative. With the proposed tributary modifications, as outlined further in *Section 3.2*, it is proposed for the spill condition to be eliminated.

3.1.3 Riparian Storage

Based on the wide-shallow nature of the existing floodplain due to existing topography and constrained road crossing infrastructure, a tributary realignment is proposed to facilitate the proposed development, which requires floodplain modifications as outlined further in *Section 3.2*. The existing riparian storage volume will need to be maintained in the proposed tributary configuration.

As noted above, the floodline upstream of Eliza Street is in a backwater condition as a result of the existing Eliza Street culvert. To determine the required existing riparian storage volume that is to be maintained within the proposed tributary corridor, the existing riparian storage volume has been calculated based on an existing floodplain assessment without manmade structures causing a backwater effect, to reflect a natural riparian storage condition. A separate hydraulic model (HEC-RAS version 6.4.1) has been prepared for this assessment, refer to **Appendix C4**.

The Regional storm even was utilize to evaluated the riparian storage at HEC-RAS cross-sections 242 and 771 to identify cumulative storage in the channel east of Eliza Street. Existing conditions riparian storage volumes are presented below in **Table 3.1**:

Table 3.1: Existing Riparian Storage

Flow Scenario ID	Existing Riparian Storage (m ³)		
	Cross-Section 771	Cross-Section 242	Cumulative (East of Eliza Street)
Regional Storm Event	14,450	3,750	10,700



3.2 Proposed Conditions

3.2.1 Site Grading

In general, the proposed development will be graded in a manner which will satisfy the following goals:

- Satisfy the Township of Wellington North lot and road grading criteria including:
 - Minimum Road Grade: 0.5%
 - Maximum Road Grade: 5.0%
 - Minimum Lot Grade: 2%
 - Maximum Lot Grade: 6%
- Provide continuous road grades for overland flow conveyance;
- Minimize the need for retaining walls;
- Minimize the volume of earth to be moved and minimize cut/fill differential;
- Minimize the need for rear lot catchbasins;
- Incorporate noise barriers and berms where required;
- Limit the height of noise barriers and berms; and,
- Achieve the stormwater management objectives required for the proposed development.

A preliminary grading plan is provided on **Figure 3.4**. As shown on **Figure 3.4**, a 2.5 m high noise barrier is required on the northeast corner as identified within the report Land Use Compatibility Study (Noise) (2025) as the option 1 recommendation, refer to the excerpt in **Appendix B4**. Per the preliminary grading plan, provided **Figure 3.4**, an equivalent 2.5 m barrier is proposed via a combined flat shelf berm and acoustic fence approach.

At the detailed design stage, the preliminary grading shown on **Figure 3.4** will be subject to a more in-depth analysis in an attempt to balance the cut and fill volumes and minimize slopes and walls.

3.2.2 Floodplain Modifications

To address the backwater condition at Eliza Street and limit the extent of the regulatory floodplain associated with the Farley Creek tributary as discussed in *Section 3.1.2*, a channel realignment is proposed. The proposed channel realignment will be located east of the Eliza Street. The proposed channel realignment will also include proposed culvert upgrades at the Eliza Street crossing from an existing 2750 mm CSP culvert to twinned 2750 mm CSP culverts. A proposed 3.0 m x 6.0 m box culvert at the future internal tributary crossing at Street G (refer to **Appendix A**) is required to accommodate



the proposed development east of the proposed channel alignment. Per the Meander Belt Width Assessment (GeoProcess, 2025), the proposed channel will consist of a 24.6 m wide meander belt. With an 8.0 m toe erosion setback, the total bottom width of the channel is 40.6m. In accordance with O.Reg. 41/24, a 6.0 m erosion allowance setback is also proposed to separate the development limit from the channel. SCS Consulting Group has prepared proposed conditions hydrology and hydraulic models to define the proposed Regional Storm floodplain based on the proposed channel design.

To determine the proposed Regional Storm peak flow through the tributary, a Visual Otthymo (VO6.2) model was prepared. The total upstream drainage area to the connection to Farley Creek was determined based on the preliminary grading plan (refer to **Figure 3.4**), as well as GeoHub LIDAR external topographic information. The total drainage area at the connection to Farley Creek (HEC-RAS cross-section 5) is 211.83 ha. The Regional Storm event (Hurricane Hazel) produced Regional flows of approximately 15.26 m³/s for the total drainage area at the connection to Farley Creek based on the proposed land use shown on the Draft Plan, refer to **Appendix A**. To assess the floodline throughout the development and the Eliza Street and Wells Street crossings, regional flows were utilized as flow nodes of 7.14 m³/s and 13.21 m³/s, respectively. These Regional flows were utilized to evaluate the floodline elevations of the upstream most section of the Farley Creek tributary (HEC-RAS cross-section 874) and the upstream side of the Eliza Street crossing (HEC-RAS cross-section 238). Refer to the hydrology model within **Appendix C4**.

To determine the proposed Regional Storm floodplain, a hydraulic model (HEC-RAS version 6.4.1) was created. The preliminary grading plan (refer to **Figure 3.4**) was utilized to develop the HEC-RAS cross-sections within the confines of the proposed development. As mentioned above, Geohub LIDAR topographic survey information was utilized for areas external to the site. As with the existing model and per the GRCA provided Conestogo River hydraulic model, the flow depths of each storm event within Farley Creek at the junction with the tributary was utilized as the downstream water surface boundary condition. The upstream boundary condition was set to critical depth. As noted above, there are three (3) culvert crossings of the tributary, Eliza Street, Wells Street, and proposed Street G. The Wells Street crossing will remain identical to that of the existing condition; the culvert sizes and inverts of the Eliza Street and proposed Street G crossings have been sized to mitigate backwater conditions in the proposed condition and eliminate the spill condition as discussed in *Section 3.1.2*.

The hydraulic model was simulated for the Regional storm event and the proposed conditions floodline delineated for the subject site. Refer to **Figure 3.3** for the proposed floodline and **Appendix C4** for the hydraulic model. As shown on **Figure 3.3**, the



proposed channel realignment has been designed to safely convey Regional storm flows with a minimum vertical freeboard of 0.70 m from residential lots and a minimum horizontal clearance of 6 m from residential lots.

3.2.3 Riparian Storage

To determine riparian storage volumes within the proposed channel realignment, a separate hydraulic model (HEC-RAS version 6.4.1) was prepared. This model was based on the removal of manmade structures from the proposed hydraulic model discussed in **Section 3.2.2** to reflect a natural riparian storage condition.

As with the existing riparian storage analysis, the Regional storm event was utilized to evaluate the proposed riparian storage at equivalent HEC-RAS cross-sections 279 and 874 to identify cumulative storage in the channel east of Eliza Street. Riparian storage volumes below in **Table 3.2**:

Table 3.2: Proposed Riparian Storage

Flow Scenario ID	Proposed Riparian Storage (m ³)		
	Cross-Section 874	Cross-Section 279	Cumulative (East of Eliza Street)
Regional Storm	17,560	3,670	13,890

The cumulative riparian storage in the channel east of Eliza Street is greater in the proposed condition than in the existing condition by approximately 3,190 m³ within the Regional storm event. As such, the existing riparian storage volumes have been maintained.



4.0 Rights-of-way and Sidewalks

Standard 20.0 m right-of-way (ROW) local street cross-sections are proposed per Township of Wellington North standard STD.R1.

The proposed sidewalk location plan is provided on **Figure 4.1**. For the areas where sidewalk will be provided along one side of the street, sidewalks will typically be located on the north or east side of the boulevard or the boulevard side where the larger number of frontages can be serviced.

Future Macauley Street ROW, outlined within the Transportation Impact Study (2025), is a standard 20 m urbanized ROW per Township of Wellington North standard STD.R1. As such, a future sidewalk has been identified on **Figure 4.1**. Refer to the cross-section in **Appendix B5**.



5.0 Storm Drainage and Servicing

5.1 Existing Conditions

5.1.1 Storm Drainage

The existing catchments, runoff coefficients, and drainage conditions are shown on **Figure 3.1**. The existing drainage boundaries were delineated using a combination of topographic mapping surveyed by J.D. Barnes dated August 2024 and Geohub LIDAR topographic data.

Storm runoff from Catchment 101 (29.43 ha) and external Catchment 201 (157.60 ha) drains southwest via overland flow toward the Farley Creek tributary that runs through the subject lands east of Eliza Street.

Storm runoff from Catchment 102 (8.01 ha) drains north via overland flow and channelized flow via ditches along the boulevard of Eliza Street toward the Farley Creek tributary that runs through the subject lands east of Eliza Street.

Storm runoff from Catchment 103 (13.30 ha) drains west via overland flow toward Farley Creek. Storm runoff from Catchment 104 (1.03 ha) drains north via channelized ditches along the Eliza Street Boulevard and drains west across Eliza Street to Farley Creek via the Eliza Street existing 2750 mm diameter CSP culvert.

Storm runoff from Catchment 105 (3.84 ha) and external Catchment 202 (0.63 ha) drains west via overland flow toward the Farley Creek tributary that runs through the subject lands east of Wells Street.

5.1.2 Storm Servicing

The existing site does not contain any stormwater management facilities, structures, or controls, with the exception of an existing 2750 mm dia. CSP culvert which crosses under Eliza Street and an existing 2000 mm dia. CSP culvert which crosses underneath Wells Street to convey on-site flows to Farley Creek (refer to **Figure 3.1**).

5.2 Proposed Conditions

5.2.1 Storm Drainage

The proposed catchments, runoff coefficients, and drainage conditions are shown on **Figures 5.1** and **Figure 5.2**. Catchments were delineated based on the preliminary grading plan (refer to **Figure 3.4**). Impervious coverage percentage was determined based on proposed land uses per the Draft Plan (refer to **Appendix A**) and typical impervious coverage values per the Town Engineering Standards and Design Criteria.



Proposed major and minor system flows from Catchment 312 (2.45 ha), Catchment 313 (0.60 ha), Catchment 315 (22.70 ha), and external Catchment 401 (64.15 ha) will be captured and conveyed toward the proposed East SWM Pond via a combination of the proposed internal storm sewer system and overland flow. External Catchment 401 will be captured via front draining lots and proposed rear lot catchbasins along the eastern property boundary.

Proposed major and minor flows from Catchment 302 (1.92 ha) and Catchment 304 (11.12 ha) will be captured and conveyed toward the proposed west SWM Pond via a combination of the proposed internal storm sewer system and overland flow. Proposed major and minor flows from Catchment 309 (4.83 ha) will be captured and conveyed toward the proposed west SWM Pond via the proposed 100-year capture point and internal storm sewer system.

Uncontrolled flows from Catchment 308 (0.76 ha), Catchment 310 (0.35 ha), Catchment 311 (0.5 ha), Catchment 314 (1.38 ha), Catchment 316 (1.97 ha), Catchment 317 (2.62 ha), and external Catchment 402 (93.38 ha) will drain northwest via overland flow and channelized flow via ditches along the boulevard of Eliza Street toward the Farley Creek tributary that runs through the subject lands east of Eliza Street.

Uncontrolled flows from Catchment 306 (0.32 ha), Catchment 307 (0.24 ha), Catchment 318 (1.32 ha), and external Catchment 403 (0.63 ha) will drain northwest via overland flow toward the Farley Creek tributary that runs through the subject lands east of Wells Street.

Uncontrolled flows from Catchment 301 (0.27 ha) and Catchment 303 (0.50 ha) drain northwest via overland flow toward Farley Creek. Uncontrolled flows from Catchment 305 (1.50 ha) will drain to future Macauley Street via overland flow and ultimately to Farley Creek.

5.2.2 Storm Servicing

5.2.2.1 Minor System

The preliminary layout for the proposed storm sewer within the subject lands is provided on the Preliminary Servicing Plan, **Figure 5.3**. The storm sewers are proposed to convey flows to the three outlets, two (2) proposed SWM facilities and to the future Macauley Street ditch. The proposed SWM facilities discharge to the Farley Creek tributary which runs southeast-northwest through the subject lands. A portion of the proposed development does not drain into the two proposed SWM facilities due to grading constraints, therefore, a separate outlet to the future Macauley Street is required which discharges into the Wells Street ditch and ultimately discharges into Farley Creek.



The storm sewer system will typically be designed with grades between 0.5% and 2%. Throughout the proposed development, the storm sewer will be constructed at a minimum depth of 1.2 m to provide frost protection.

It will not be possible to provide deep enough storm sewers to service foundation drains by gravity; therefore, sump pumps will be required throughout the proposed development on every lot.

The storm drainage system will be designed in accordance with the Township of Wellington North and MECP guidelines, including the following:

- Pipes to be sized to accommodate runoff from a 5 year storm event,
- Minimum Pipe Size: 300 mm diameter,
- Maximum Flow Velocity: 4.5 m/s,
- Minimum Flow Velocity: 0.75 m/s,
- Minimum Pipe Depth: 2.4 m to obvert for gravity service connections,
- Minimum Pipe Depth: 1.2 m to obvert.

5.2.2.2 Major System

Major system flows (greater than the 5 year up to the 100 year storm event) will be conveyed within the road rights-of-way to the east or west SWM facility or the Wells Street outlet via future Macauley Street. Standard 20.0 m ROW local street cross-sections are proposed per Township of Wellington North standard STD.R1, as outlined in *Section 4.0*.

A 100-year capture location is proposed at the eastern connection to Eliza Street. Refer to **Figure 5.4** for the 100-year capture location.

The proposed right-of-way cross-sections and capacity calculations are provided in **Appendix D** which show that the major system flows can be safely conveyed within the proposed road rights-of-way. At the detailed design stage, design of the 100 year capture and rear lot catchbasins to capture the external flow will be completed.



6.0 Stormwater Management

6.1 Stormwater Management Criteria

The following stormwater management criteria have been established based on the greatest requirements of each of the design guidelines and standards listed in **Section 1.3**. The stormwater management criteria are summarized below in **Table 6.1**:

Table 6.1: Stormwater Management Criteria

Criteria	Control Measure
Quality Control	Provide an “Enhanced” level of protection in accordance with Ministry of Environment (MOE) Stormwater Management Planning and Design Manual (2003) with a minimum 80% Total Suspended Solids (TSS) removal.
Erosion Control	Retention of the 25 mm rainfall runoff for a minimum of 24 hours.
Quantity Control	Post-to-Pre-Control for the 2 through 100 year storm events.

6.2 Stormwater Management Targets

6.2.1 Water Quality

An “Enhanced” level of quality control is required for the proposed development. The quality control parameters as presented in Table 3.2 of the Stormwater Management Planning and Design Manual, (MOE, March 2003) were used in the design of SWM facilities. The East and West SWM facilities will be sized to service a drainage area of 90.49 ha and 17.87 ha, respectively. For the East and West SWM facilities, and based on Table 3.2, respective imperviousness values of 20% and 60% and respective total storages of 92.0 m³/ha and 202.8 m³/ha are required for wet ponds (refer to **Appendix E**). Of this total storage, 56.0 m³/ha and 163.0 m³/ha represent the respective required volumes of the permanent pools for the East and West SWM facilities. For the East and West SWM facilities, and based on respective drainage areas of 90.49 ha and 17.87 ha, approximately 5,078 m³ and 2,909 m³ of respective permanent pool storages are therefore required to obtain an “Enhanced” level of protection.

6.2.2 Erosion Control

The SWMP Manual states that the extended detention storage is based on the greater of 40 m³/ha or the storage volume required to retain a 25mm storm for 24 to 48 hours. As outlined in **Table 6.1** above, a detention time of 24 hours is required for the subject site. The Visual Otthymo 6.1 hydrologic model was used to establish the post-



development runoff from the proposed development during a 25mm, 4 hour Chicago rainfall event. The required extended detention storage based on the runoff volume detained for 24 hours was determined to be approximately 7,303 m³ and 2,865 m³ for the East and West SWM facilities, respectively (refer to **Appendix E**). The modelled runoff yielded larger storage volumes than the extended detention volumes based on 40 m³/ha for both SWM facilities, therefore the erosion control storage volumes of 7,303 m³ and 2,865 m³ will be used in facility design. A copy of the model schematic and the output file are included as **Appendix F** with digital modelling files included via a file sharing link on the **Appendix F** cover page.

6.2.3 Quantity Control

As outlined in **Table 6.1**, control of post-development flows to pre-development flows is required for the 2 through 100 year storm events. The pre-development flow rates for the 2 through 100 year storm events have been established utilizing the Visual Otthymo Version 6.2 software (VO6) based on the 3-hour Chicago Distribution method per GRCA and Wellington North municipal standards. The study area is located within the Township of Wellington North, therefore, the IDF rainfall information was obtained from the MTO website to determine the existing peak flows to outlet locations.

The existing total flows from the study area to Farley Creek, and the proposed uncontrolled areas draining to Farley Creek, Wells Street, and Eliza Street are summarized in **Table 6.2**.

Table 6.2: Summary of Existing Flows

Return Period Storm	Uncontrolled Direct to Farley Creek (m ³ /s)	Uncontrolled to Wells Street (m ³ /s)	Uncontrolled to Eliza Street (m ³ /s)	Total to Farley Creek (m ³ /s)
2 Year	0.316	0.130	0.465	1.016
5 Year	0.436	0.207	0.818	1.756
10 Year	0.521	0.266	1.084	2.306
25 Year	0.629	0.347	1.444	3.049
50 Year	0.714	0.409	1.723	3.620
100 Year	0.798	0.475	2.011	4.209

A summary of modelling parameters and an existing VO6 schematic are provided in **Appendix F1** with digital modelling files included with the submission of this report.



6.3 Proposed Stormwater Management Plan

The following study area characteristics and constraints were taken into consideration in developing the SWM Plan for the site:

- The existing topography generally slopes to the northwest in the range of 0.40% to 20%.
- The study area contains split drainage to a tributary of Farley Creek which runs southeast-northwest through the study area and Farley Creek to the west.
- Based on the Geotechnical investigation (GEMTEC, 2024), study area soils consist of silt loam;
- Within the installed site wells, groundwater was observed at depths ranging from 1.2 to 3.1 m below existing ground; and
- The proposed subdivision development is approximately 55.34 ha and consists of residential developments, Natural Heritage Systems, park blocks, SWM facilities, and proposed roads.

Taking these constraints into consideration, and the SWM criteria listed in *Section 6.1*, the proposed SWM Plan includes two (2) end-of-pipe SWM facilities.

Refer to **Sections 6.3.1** below, for additional information on end-of-pipe SWM facilities.

6.3.1 End-of-Pipe Stormwater Management Facilities

Two (2) SWM facilities are proposed to provide quality, erosion and quantity control. The East SWM pond will be located along Eliza Street, on the east parcel, as shown on **Figure 6.1**, and will outlet to the Eliza Street Outlet and ultimately to Farley Creek. The SWM facility will be sized for a total drainage area of 90.49 ha assuming an imperviousness of 20%. Refer to **Appendix D1** for the impervious calculations.

The West SWM pond will be located along Wells Street, on the west parcel, as shown on **Figure 6.2**, and will outlet to the Wells Street Outlet and ultimately to Farley Creek. The SWM facility will be sized for a total drainage area of 17.87 ha assuming an imperviousness of 60%. Refer to **Appendix D1** for the impervious calculations.

6.3.1.1 Quality Control

The function of the permanent pool is to provide quality control by removing sediment from the storm runoff conveyed to the SWM facility. The West and East SWM Facilities will be designed to provide permanent pool storage based on MOE (2003) Enhanced Level Protection for a wet pond based on the impervious drainage area to the pond (see Table 3.2, 2003 MOE Guidelines). The required and available permanent pool volumes



for each SWM facility are summarized in **Table 6.4**. See **Figures 6.1 and 6.2** and calculations in **Appendix E** for details.

Table 6.3: Summary of Proposed Quality Control

SWM Facility	Total Area Draining to Pond (ha)	Impervious Drainage Area (%)	Permanent Pool Storage Requirement ¹ (m ³ /ha)	Required Permanent Pool Volume (m ³)	Provided Permanent Pool Storage (m ³)
East Pond	90.49	20	56	5,078	19,978
West Pond	17.87	60	163	2,909	15,846

¹Per MOE (2003) Enhanced Level Protection for a wet pond.

For the small portion of the development that cannot drain to the SWM facilities and discharges south to future Macauley Street, an oil grit separator will be provided within Street C to provide a level of quality control.

6.3.1.2 Erosion Control

The attenuation of the extended detention volume in the pond will provide erosion protection for the downstream watercourse as well as promote sediment removal for water quality. The extended detention volume for the proposed stormwater management facilities will be sized based on the detention of the 25 mm - 4 hour Chicago rainfall event. The volume calculated for the extended detention will be attenuated for a minimum of 24 hours.

The required extended detention volume for the East SWM Pond is 7,303 m³ (see **Appendix E**). This volume is greater than the 2003 MOE guidelines minimum extended detention volume of 40 m³/ha or 3,620 m³ based on the 90.49 ha drainage area. The required extended detention volume for the West SWM Pond is 2,865 m³ (see **Appendix E**). This volume is greater than the 2003 MOE guidelines minimum extended detention volume of 40 m³/ha or 715 m³ based on the 17.87 ha drainage area.

The calculations for the extended detention component of the proposed stormwater management facilities are provided in **Appendix E**, with hydrologic modelling provided in **Appendix F**. Orifice sizing calculations and details will be provided at detailed design.

6.3.1.3 Quantity Control

The proposed SWM facilities will control future flows from the proposed development to the existing flow rates to Farley Creek provided in **Table 6.2** for the 2 to 100 year storm events. Proposed hydrologic modelling was completed using the Visual Otthymo model (Version 6.2) to determine the required volumes of the East and West SWM



facilities. Refer to the File Safe Cloud Link provided in **Appendix F2** to download the VO hydrology model files.

The 3-hour Chicago Storm distribution per the GRCA and Township of Wellington North requirements were modelled for the proposed conditions hydrology model. A summary of the resulting storage requirements for the SWM pond is provided in **Table 6.5**.

Table 6.4: SWM Facility Storage Requirements

SWM Facility	East Pond		West Pond	
	Discharge (m ³ /s)	Required Storage (m ³)	Discharge (m ³ /s)	Required Storage (m ³)
2 Year	0.299	6,355	0.091	3,169
5 Year	0.527	8,557	0.128	4,323
10 Year	0.665	10,371	0.154	5,143
25 Year	0.875	12,766	0.201	6,163
50 Year	1.045	14,384	0.241	6,868
100 Year	1.194	16,115	0.283	7,566

6.3.1.4 General SWM Facility Design Criteria

Preliminary SWM facility grading is provided on **Figure 5.2**. The pond block size was established based on the following general criteria:

- A 4 m wide maintenance access road will be provided from a proposed municipal road with a maximum longitudinal slope of 10% and a crossfall of 2% (max). It will be used to facilitate machinery to access the forebay during scheduled maintenance as well as to access the outlet structure for maintenance purposes;
- A maximum slope of 3:1 from the pond bottom to 0.5 m below the normal water level will be provided;
- A maximum slope of 5:1 from 0.5 m below and above the normal water level will be provided; and,
- A maximum slope of 3:1 will be provided from 0.5 m above the normal water level to the pond grading limits.



6.4 Stormwater Management Plan Performance

6.4.1 Water Quality

An “Enhanced” protection level for quality control will be achieved through the implementation of the end-of-pipe SWM facilities, as outlined in **Section 6.3.1.1**.

6.4.2 Erosion Control

The erosion control criterion is to provide a minimum of 24 hour extended detention of the runoff from a 25 mm rainfall event which will be achieved in both of the wet ponds. The preliminary design requirements of the end-of-pipe SWM facility are discussed above in **Section 6.3.1.2**.

6.4.3 Quantity Control

The proposed SWM facilities will control proposed flows to allowable release rates provided in **Table 6.3** for the 2 through 100 year storm events. To ensure that the quantity control criterion of providing post to pre control for the 2 through 100 year storm events was achieved through the proposed SWM Plan as outlined above, the total flow to Farley Creek was reviewed. **Table 6.6** provides a comparison of the total existing and proposed flows to Farley Creek downstream of the subject site.

Table 6.5: Comparison of Total Existing and Proposed Flows

Node	Total Peak Flow to Farley Creek (m ³ /s)	
	Existing	Proposed
Return Period Storm		
2 Year	1.016	0.879
5 Year	1.756	1.525
10 Year	2.306	1.965
25 Year	3.049	2.552
50 Year	3.620	3.047
100 Year	4.209	3.544

As shown in **Table 6.6**, the proposed flows are less than or equal to the existing flows for the 2 through 100 year storm events at all target locations and thus the quantity control criterion is achieved.



7.0 Sanitary Servicing

7.1 Existing Sanitary Sewer System

There are no existing sanitary sewers within the immediate vicinity of the subject lands. The existing sanitary sewers did not account for the subject site lands.

Per the Triton Water and Sanitary Systems Technical Study (2020), there is an existing sanitary sewer system which services the town located south of the Canadian Pacific Railway. The existing sanitary sewer system consists of a network of sanitary sewers, the Arthur Waste Water Treatment Plant (WWTP), the Preston Street trunk sewer, and two sewage pumping stations (SPS) located on Wells Street and Frederick Street. Refer to the existing sanitary system within the Triton Figure Sanitary System in **Appendix B3**.

The Preston Street trunk sewer services Preston Street, the western portion of Domville Street, and the Wells Street SPS discharge via gravity. The Wells Street SPS receives industrial flows from the west side of the town and pumps flows via a 1 km long, 150 mm PVC/AC forcemain. The Frederick Street SPS receives flows from the central, southern and eastern portions of the system and pumps flows via a 750 m long, 250 mm forcemain. The Arthur WWTP has a rated average day flow (ADF) capacity of 1,465 m³/day and discharges to the Conestogo River.

7.2 Proposed External Sanitary System

The 2016 Arthur WWTP Class EA proposes a staged capacity increase to the sanitary servicing system. Phase 1 consists of a design average daily flow of 1,860 m³/d based on additional servicing capacity for minor upgrades to the WWTP. Phase 2 consists of a design average daily flow of 2,300 m³/d based on projected servicing needs for 2031.

The total proposed peak sanitary flow for the proposed development is 46.26 L/s and the total proposed ADF for the proposed development is 908.28 m³/day. The proposed developments east and west of Eliza Street are represented in the Triton Water and Sanitary Systems Technical Study (2020) as areas FD4-2 and FD4-3, as shown in Figure Stage 4 Development Scenario in **Appendix B3**. Areas FD4-2, comprised of 966 Equivalent Residential Units (ERU), and FD4-3 (433 ERU) represent a total ERU of 1,399. Using an average flow rate of 400 L/cap/day and 2.7 persons/unit, the projected 2045 ADF for the proposed development is 1,510.92 m³/day.

A summary of the proposed and projected ADFs are shown in **Table 7.1**.



Table 7.1: Summary of Proposed and Projected ADF

Proposed ADF (m ³ /day)	Projected 2045 ADF (Triton, 2020) (m ³ /day)
908.28	1,510.92

Therefore, the proposed ADF is within the projected 2045 ADF estimated by Triton (2020). Proposed ADF calculations are provided in **Appendix H**.

As discussed within the Arthur Water and Wastewater Servicing Summary Report (2025), further modifications to the WWTP are required in addition to the WWTP Phase 2 expansions to service the proposed development. DLW Engineering recommends potential solutions to integrate the proposed Phase 2 WWTP expansion with the proposed development. Refer to the Arthur Water and Wastewater Servicing Summary Report (2025), prepared by DLW for further details.

7.2.1 Proposed External Sanitary Sewer System

Per the Triton Water and Sanitary Systems Technical Study (2020), sanitary sewer upgrades will only be required along Francis Street, George Street, and Frederick Street through Phase 4 of the staged capacity increase plan, refer to Triton Figure 5.7 in **Appendix B3**. As the proposed development does not drain into the sanitary sewers that require updates, the downstream existing sanitary network is sufficient to service the proposed development.

The proposed development will connect into the future sanitary sewer on the future Macauley Street, which ultimately connects to the Arthur WWTP. The future Macauley Street sanitary sewer connects into an extension of the existing gravity sanitary sewer which currently terminates at the existing industrial property and Wells Street, via Wells Street. Wells Street plan and profiles as built drawings, provided by the Township of Wellington North, were utilized to detail the Wells Street and future Macauley Street connection, refer to **Appendix B3**.

7.3 Proposed Internal Sanitary Sewer System

The preliminary layout for the proposed sanitary sewer within the subject lands is provided on **Figure 5.3** and **Figure 7.1**.

The proposed development will connect into the future gravity sanitary sewer on the future Macauley Street and connect into the existing Wells Street sanitary sewer, which ultimately connects to the Arthur WWTP.



The proposed sanitary sewer system of the proposed development located east of the tributary of Farley Creek, will flow northwest towards the proposed sanitary pumping station. Refer to the Arthur Water and Wastewater Servicing Summary Report (2025), prepared by DLW Engineering, excerpts in **Appendix B3** for details on the proposed sanitary pumping station. The proposed sanitary pumping station is located immediately north of the east stormwater management pond block and will pump through a forcemain at an estimated capacity of 20 L/s. The proposed peak sanitary flow for the east parcel is 22.71 L/s and the proposed ADF is 438.48 m³/day. The proposed sanitary pumping station will connect into the proposed gravity sanitary system west of the tributary via a forcemain within Eliza Street.

The sanitary sewers within the proposed development will have slopes ranging between 0.5% and 2% (typically) and will be provided at 3.0 m to 7 m deep.

The sanitary sewer system will be designed in accordance with the Township of Wellington North and MECP criteria, including but not limited to:

- Residential Sanitary Generation Rate: 400 L/cap/d,
- Population Density: 2.7 people/unit,
- Peaking Factor: Harmon (Max. 4.0),
- Infiltration Rate: 0.15 L/s/ha,
- Minimum Pipe Size: 200 mm diameter,
- Minimum Pipe Cover: 2.4 m,
- Minimum Actual Velocity: 0.60 m/s, and
- Maximum Velocity: 3.0 m/s.

8.0 Water Supply and Distribution

8.1 Existing Water Distribution

There are no existing watermains within the immediate vicinity of the subject lands.

Per the Arthur Water and Sanitary Servicing Summary Report (DLW, 2025), there is an existing, single-pressure zone watermain system which services the town. The existing watermain system consists of a network of watermains, the Charles Street Water Tower, the Freud Water Tower, and three bedrock wells.

Well No. 7B is located on Wells Street near the Conestogo River and has a rated capacity of 22.7 L/s or 1,961 m³/day. Wells No. 8A/8B, which may not operate concurrently, are located on Part of Lots 20 and 21, Concession A and have rated capacities of 26.1 L/s or 2,255 m³/day.

The Charles Street Tower is located in the southeast portion of the system and is a multi-legged steel tank with a volume of 227 m³ and operating range of 494.2 – 499.6 m. The Freud Tower is located in the northwest part of the system and is a steel spheroid tank with a volume of 1,137 m³ and operating range of 494.0 – 499.2 m. The total existing system storage volume is 1,364 m³.

8.2 Proposed Water System

Per the Schedule 'B' Municipal Class Environmental Assessment Arthur Water System Supply Redundancy & Storage (Arthur Water System Class EA, 2024), the existing water system including the existing wells and elevated water reservoir do not have sufficient capacity for the 2051 demands within Arthur, therefore the Arthur Water System Class EA (2024) has proposed a new 900 m³ elevated water storage tank.

A block has been provided within the proposed development for the new well and elevated water reservoir, refer to **Figure 8.1**. Block 67 has been proposed at the northeast corner of Wells Street and future Macauley Street, which is located in the similar location as the test well per the Arthur Water System Class EA (2024). The new tank is required to support the projected 2051 demands; however, it will be insufficient to accommodate the full proposed Arthur development as outlined within the Arthur Water and Sanitary Servicing Summary Report (DLW, 2025).

Per the water servicing analysis prepared by DLW Engineering (2025), additional water storage demands will be required for the proposed development. Depending on whether the storage will be based on existing and proposed populations or based on the population of strictly the proposed development, the required water storage tank will be the following:



- 1,232 to 1,359 m³ for a 823-lot residential development; or
- 1,271 to 1,406 m³ for a 872-lot residential development.

The additional water storage demands may be satisfied by either an inground concrete reservoir with a booster pumping station, or an elevated storage tank. DLW Engineering recommends that the Arthur Water System Class EA (2024) proposed tank of 900 m³ should be increased to a minimum of 1,800 m³ to sufficiently accommodate the existing and proposed development. Refer to the Arthur Water and Sanitary Servicing Summary Report (DLW, 2025) for further details.

The preliminary layout for the proposed watermain system is provided on **Figure 8.1**. As illustrated, the proposed development will be serviced via a new well and elevated water reservoir.

The proposed internal water distribution system will connect into the future watermain within the future Macauley Street and connect to the new well and elevated water reservoir. The future Macauley Street watermain will ultimately connect to the existing watermain within Wells Street.

The watermain system will be designed in accordance with the Town of Wellington North and MECP criteria including:

- Average day water usage rate: 350 L/cap/d,
- Population Density: 2.7 people/unit,
- Minimum Pipe Size: 150 mm diameter,
- Minimum Pipe Depth: 2.0 m, and
- Maximum Hydrant Spacing: 150 m.



9.0 Servicing Allocation

Servicing capacity is assigned by Wellington County to the Township of Wellington North for allocation to individual developments. As part of the Draft Plan of Subdivision approval, servicing capacity allocation is provided by the Township of Wellington North Council.



10.0 Utility Considerations

The utility companies (hydro, natural gas, and telecommunications) have been contacted to circulate the proposed draft plan of subdivision to confirm whether there is sufficient servicing capacity.



11.0 Erosion and Sediment Control During Construction

During the detailed design stage, erosion and sediment control measures will be designed with a focus on erosion control practices (such as stabilization, track walking, staged earthworks, etc.) as well as sediment controls (such as fencing, mud mats, catchbasin sediment control devices, rock check dams and temporary sediment control ponds). These measures will be designed and constructed as per the Municipal Servicing Standards (Township of Wellington North, 2023). A detailed erosion and sediment control plan will be prepared for review and approval by the Township of Wellington North and the GRCA prior to any proposed grading being undertaken. This plan will address phasing, inspection and monitoring aspects of erosion and sediment control. All reasonable measures will be taken to ensure sediment loading to the adjacent watercourses and properties are minimized both during and following construction.



12.0 Summary

This Functional Servicing and Stormwater Management Report has been prepared in support of the Draft Plan of Subdivision applications for the proposed development in the community of Arthur, in the Township of Wellington North. This report outlines the means by which the proposed development can be graded and serviced in accordance with the Township of Wellington North, Grand River Conservation Authority, MTO, and the Ministry of Environment, Conservation and Parks design criteria and policies.

General Information

- The existing subject lands are comprised of agricultural land, low-density residential lands, and a tributary of Farley Creek which runs south to north-west;
- The proposed development is located within the Grand River Watershed;
- The proposed development land uses consist of low and medium residential, parks, SWM pond blocks, proposed roads, and a pumping station; and
- The proposed development consists of an east and west parcel, which is separated by Eliza Street / County Road 14 running north-south.

Stormwater Management and Storm Servicing

- Quality Control: MECP Enhanced (Level 1) water quality protection can be provided through the use of two end-of-pipe SWM facilities.
- Erosion Control: The runoff volume from a 25 mm rainfall event will be detained over 24 hours by the wet SWM ponds;
- Quantity Control: Quantity control will be provided via two wet SWM ponds to control proposed runoff rates in the 2 through 100 year storm events;
- Storm Servicing:
 - Storm runoff will be conveyed by storm sewers designed in accordance with Municipality and MECP criteria;
 - Storm sewers will generally be designed for the 5 year storm event; and
 - Adequate 100 year overland flow routes and 100 year capture locations will be provided.

Sanitary Servicing

- There are no existing sanitary sewers within the immediate vicinity of the subject lands;
- The proposed development will connect into the future sanitary sewer on the future Macauley Street and connect into the existing sanitary sewer within Wells Street which ultimately connects to the Arthur WWTP;
- The existing sanitary sewer network does not require upgrades to accommodate the proposed development;



- A proposed sanitary pumping station is required for the lands east of the tributary and a forcemain is proposed within Eliza Street to connect to the proposed gravity sanitary sewer system; and
- Additional upgrades to the Arthur WWTP are required as outlined within the Arthur Water and Wastewater Servicing Summary Report prepared by DLW Engineering.

Water Supply and Distribution

- There are no existing watermains within the immediate vicinity of the subject lands;
- The proposed development is to be serviced with the new well and elevated water reservoir;
- The proposed water distribution system will connect to a future watermain within future Macauley Street and future watermain extension along Wells Street that ultimately connects to the existing watermain within Wells Street; and
- DLW Engineering has completed a water servicing which demonstrates that additional water storage demands will be required for the proposed development, which may be satisfied by either an inground concrete reservoir with a booster pumping station, or an elevated storage tank.

Grading

- The proposed development grading has been developed to match to the existing surrounding grades, and provide conveyance of stormwater runoff, including external drainage; and
- The site grading will be subject to further grading design at the detailed design stage.

Rights-of-Way and Sidewalks

- Standard 20.0 m ROW local street cross-sections are proposed per Township of Wellington North standards.

Erosion and Sediment Control during Construction

- An erosion and sediment control plan will be prepared at the detailed engineering stage, in accordance with the Township of Wellington North Municipal Servicing Standards (2023).

Utility Considerations



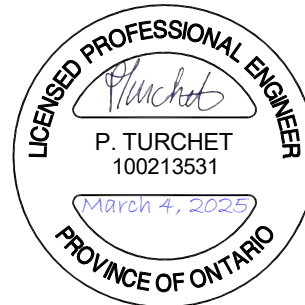
- ➔ The utility companies (hydro, natural gas, and telecommunications) have been contacted to circulate the proposed draft plan of subdivision to confirm whether there is sufficient servicing capacity.

Respectfully Submitted:

SCS Consulting Group Ltd.



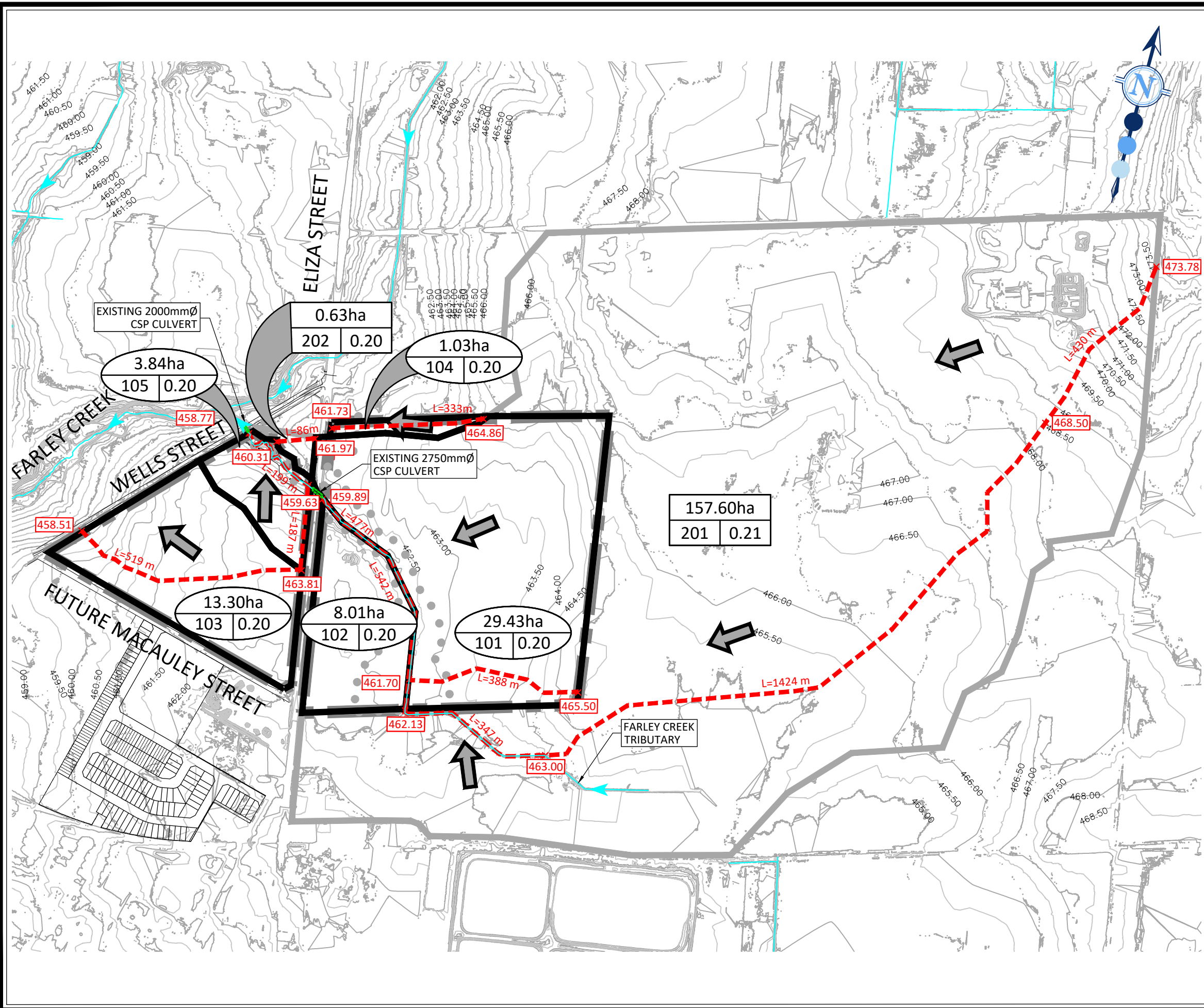
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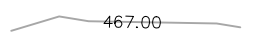

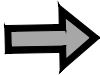
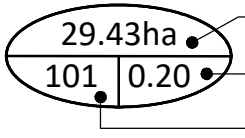
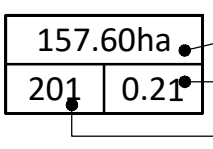
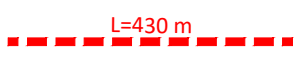


Paige Turchet, P.Eng.
pturchet@scsconsultinggroup.com

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Figures



LEGEND:

-  LIMIT OF PARTICIPATING PROPERTY
-  EXISTING DRAINAGE BOUNDARY
-  EXTERNAL DRAINAGE BOUNDARY
-  EXISTING REGULATORY FLOODPLAIN
-  EXISTING CONTOUR AND ELEVATION
-  WATERCOURSE
-  OVERLAND FLOW
-  DRAINAGE AREA (HECTARES)
RUNOFF COEFFICIENT
CATCHMENT ID
-  EXTERNAL STORM DRAINAGE AREA (HECTARES)
RUNOFF COEFFICIENT
CATCHMENT ID
-  TIME OF PEAK FLOW PATH AND LENGTH
-  TIME TO PEAK ELEVATION
-  EXISTING CULVERT

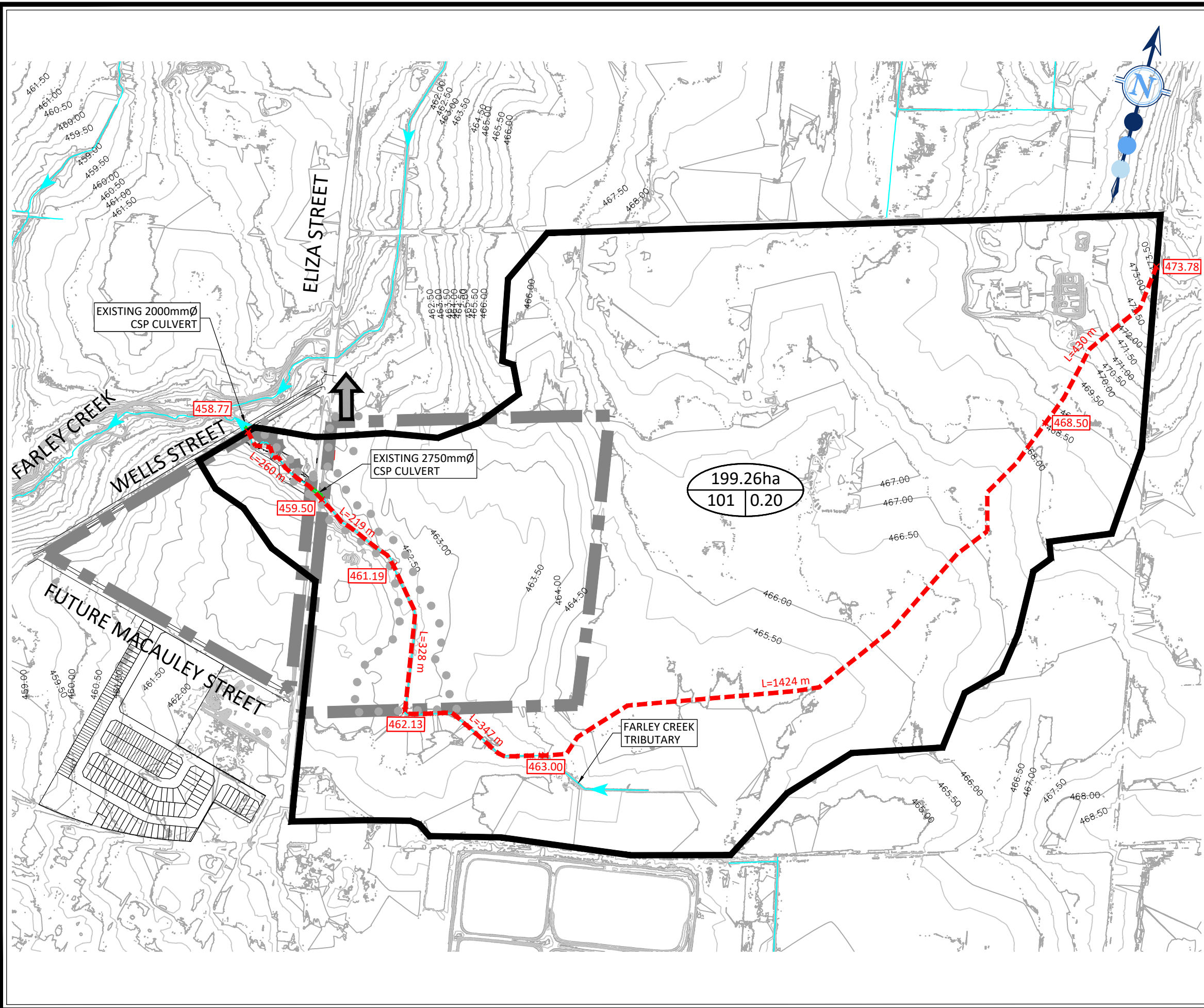
 30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

EXISTING DRAINAGE PLAN

DESIGNED BY:	L.W.	CHECKED BY:	P.A.T.
SCALE:	1:8500	DATE:	FEBRUARY 2025
PROJECT No:	2504	FIGURE No:	3.1



LEGEND:

- LIMIT OF PARTICIPATING PROPERTY
- EXISTING DRAINAGE BOUNDARY
- EXISTING REGULATORY FLOODPLAIN
- EXISTING CONTOUR AND ELEVATION
- WATERCOURSE
- OVERLAND FLOW
- DRAINAGE AREA (HECTARES)
199.26ha
101 | 0.20
- TIME OF PEAK FLOW PATH AND LENGTH
L=430 m
- TIME TO PEAK ELEVATION
x 468.50
- EXISTING CULVERT

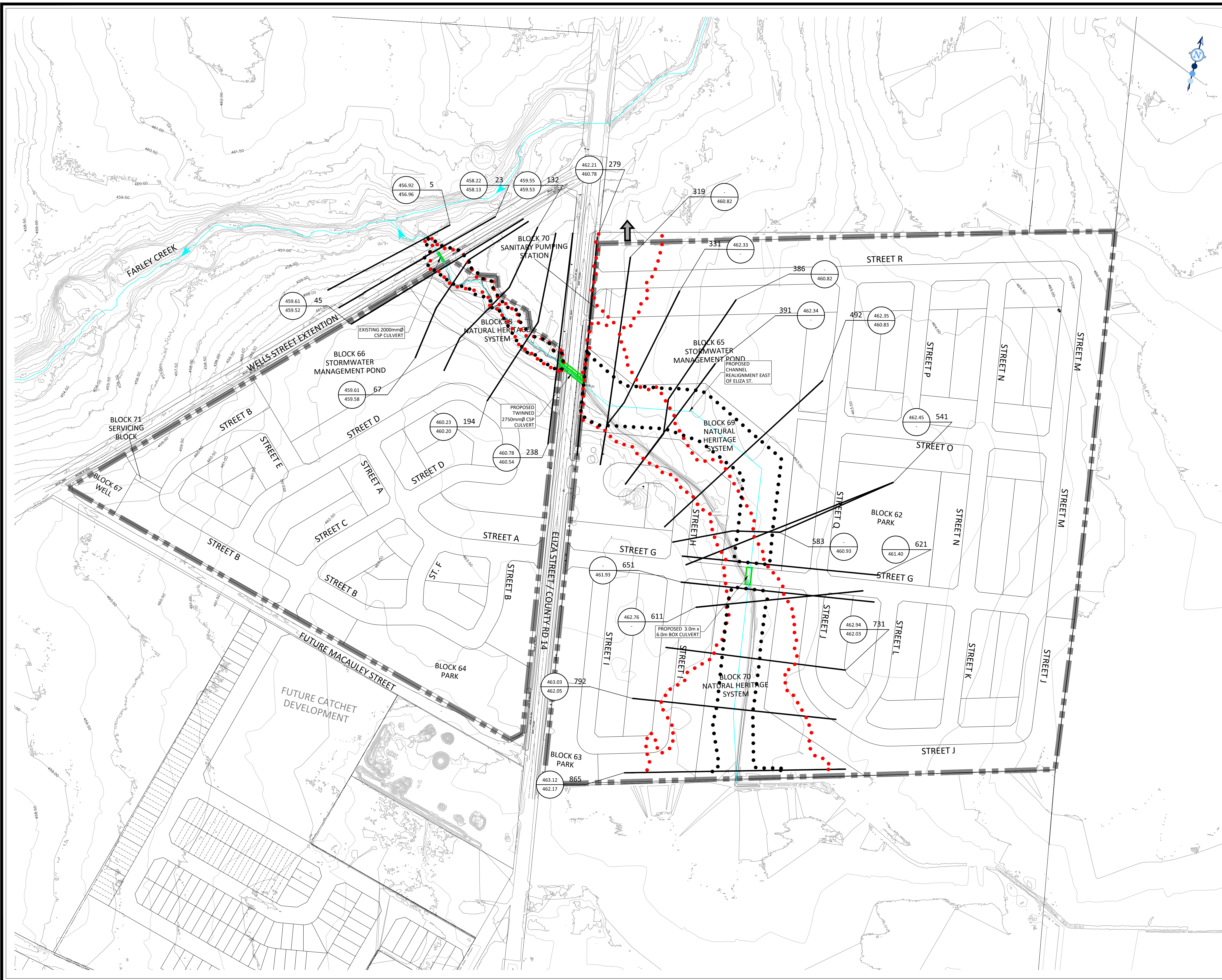
30 CENTURIAN DRIVE, SUITE 100
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TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

EXISTING DRAINAGE PLAN - HYDRAULIC MODEL

DESIGNED BY:	L.W.	CHECKED BY:	P.A.T.
SCALE:	1:8500	DATE:	FEBRUARY 2025
PROJECT No:	2504	FIGURE No:	3.2



LEGEND:

- LIMIT OF PARTICIPATING PROPERTY
- EXISTING REGULATORY (REGIONAL STORM) FLOOD ELEVATION
- HYDRAULIC SECTION ID
- PROPOSED REGULATORY (REGIONAL STORM) FLOOD ELEVATION
- FLOODLINE ELEVATION LEADER
- HEC CROSS-SECTION
- EXISTING REGULATORY (REGIONAL STORM) FLOODPLAIN (SCS, 2025)
- PROPOSED REGULATORY (REGIONAL STORM) FLOODPLAIN (SCS, 2025)
- WATERCOURSE
- EXISTING CONTOUR AND ELEVATION
- EXISTING REGULATORY FLOODPLAIN SPILL

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

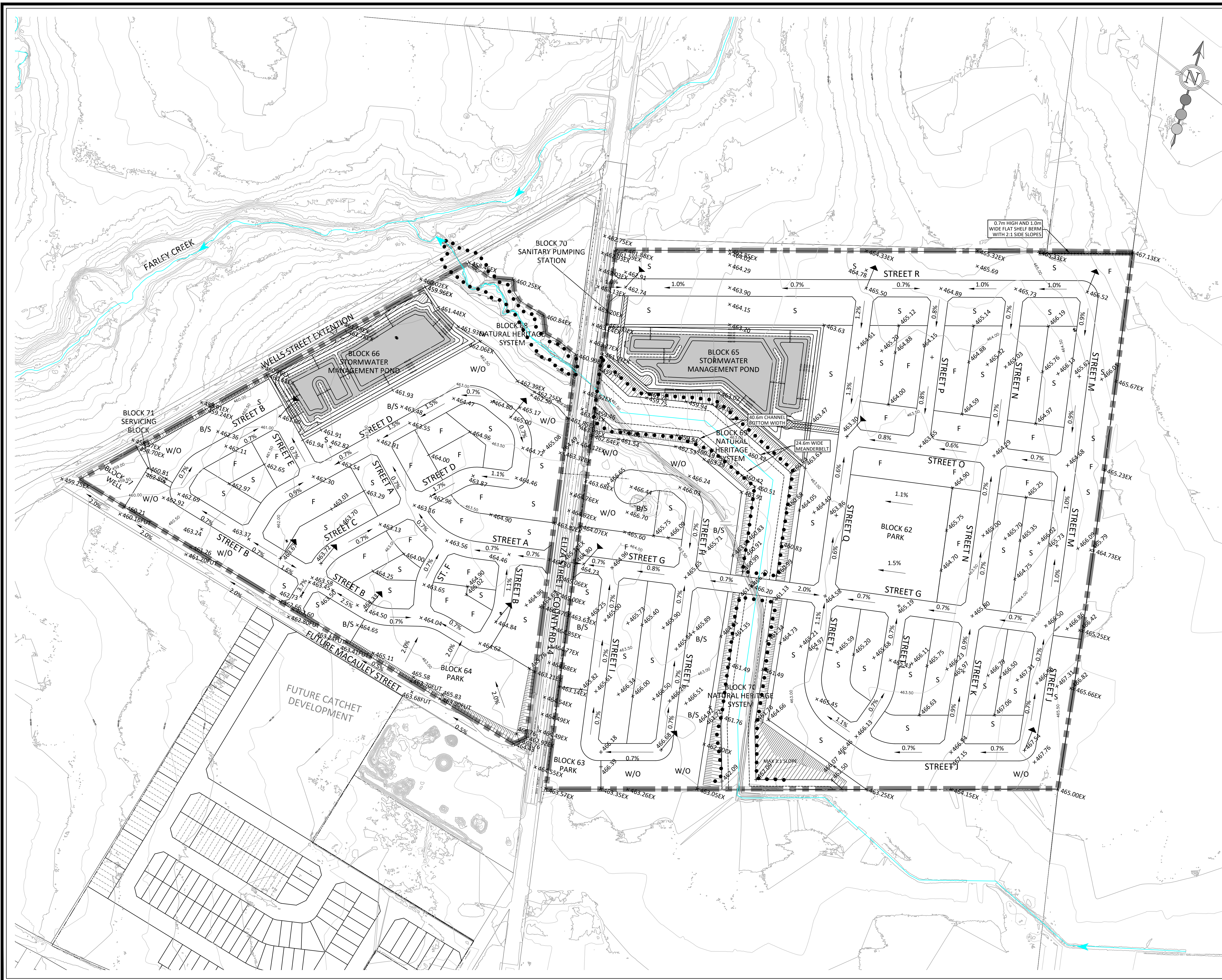
SGS consulting group Ltd
 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

EXISTING AND PROPOSED FLOODPLAIN

DESIGNED BY: L.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 3.3



LEGEND:

- LIMIT OF PROPOSED DEVELOPMENT
- EXISTING CONTOUR AND ELEVATION
- MAX 3:1 SLOPE
- ROAD SLOPE
- FRONT LOT
- FRONT SPLIT LOT
- SPLIT LOT
- BACK SPLIT LOT
- WALKOUT LOT
- PROPOSED ELEVATION
- FUTURE ELEVATION
- EXISTING ELEVATION
- PROPOSED HIGH POINT
- PROPOSED LOW POINT
- PROPOSED REGULATORY (REGIONAL STORM) FLOODPLAIN (SCS, 2025)
- WATERCOURSE
- BOREHOLE
- MEANDERBELT
- 1.8m HIGH ACOUSTIC FENCE AND 1.0m WIDE FLAT SHELF BERM

*NOTE: LAYOUT IS SCHEMATIC ONLY. DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

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 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.





ARTHUR WELLINGTON NORTH DEVELOPMENT

GRADING PLAN


DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 3.4



LEGEND:

-  LIMIT OF PARTICIPATING PROPERTY
-  PROPOSED SIDEWALK
-  FUTURE SIDEWALK
-  WATERCOURSE

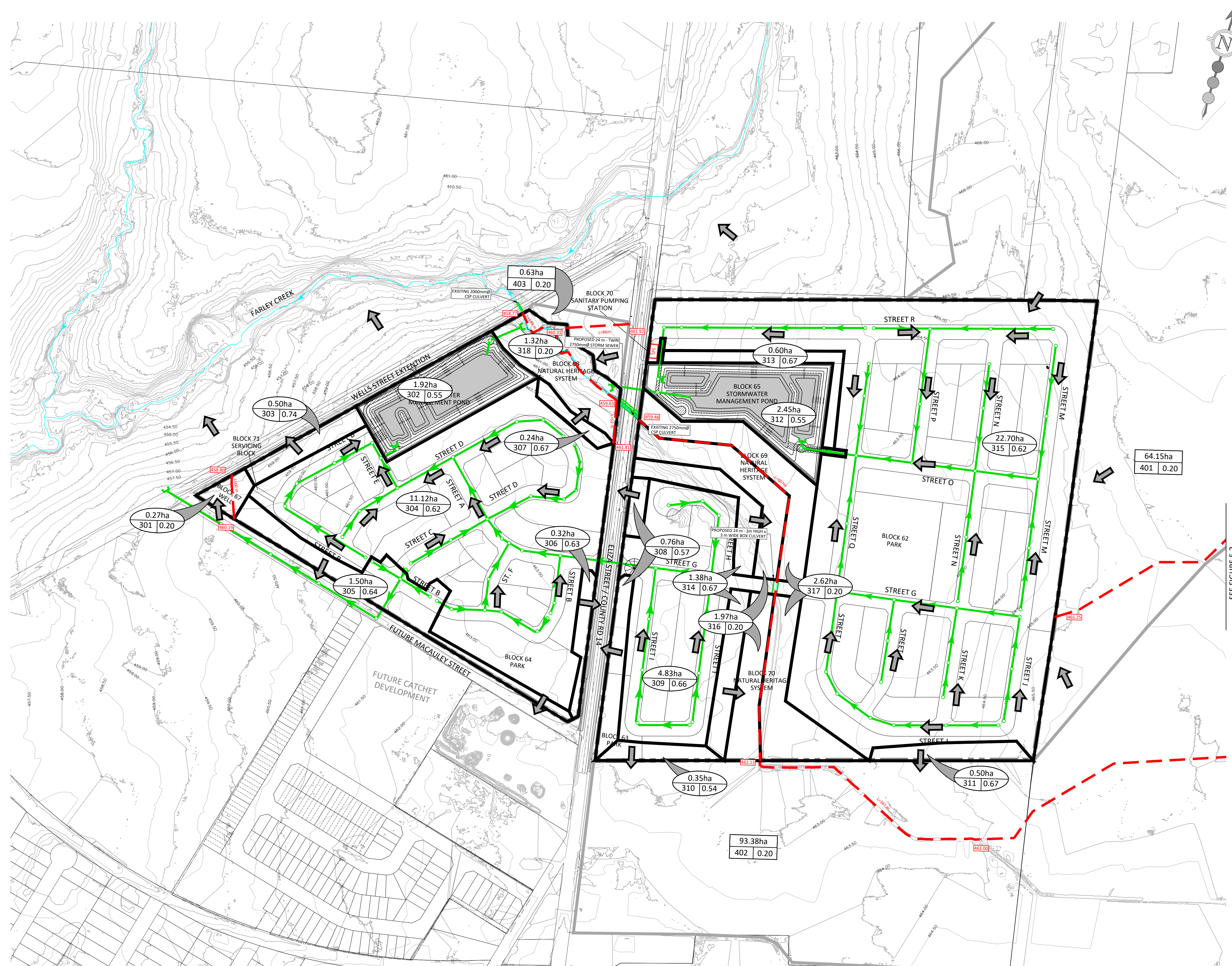
*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.
ARTHUR WELLINGTON NORTH DEVELOPMENT

SIDEWALK LOCATION PLAN

DESIGNED BY: L.W.	CHECKED BY: P.T.
SCALE: 1:5000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 4.1



- LEGEND:**
- LIMIT OF PARTICIPATING PROPERTY
 - PROPOSED STORM DRAINAGE BOUNDARY
 - EXTERNAL STORM DRAINAGE BOUNDARY
 - EXISTING CONTOUR AND ELEVATION
 - MAJOR SYSTEM, OVERLAND FLOW
 - DRAINAGE AREA (HECTARES)
 - RUNOFF COEFFICIENT
 - CATCHMENT ID
 - EXTERNAL STORM DRAINAGE AREA (HECTARES)
 - RUNOFF COEFFICIENT
 - CATCHMENT ID
 - TIME OF PEAK FLOW PATH AND LENGTH
 - TIME TO PEAK ELEVATION
 - PROPOSED STORM SEWER AND MAINTENANCE HOLE
 - PROPOSED HEADWALL
 - 100 YEAR CAPTURE LOCATION

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SGS consulting group ltd
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 TEL: (905) 475-1900
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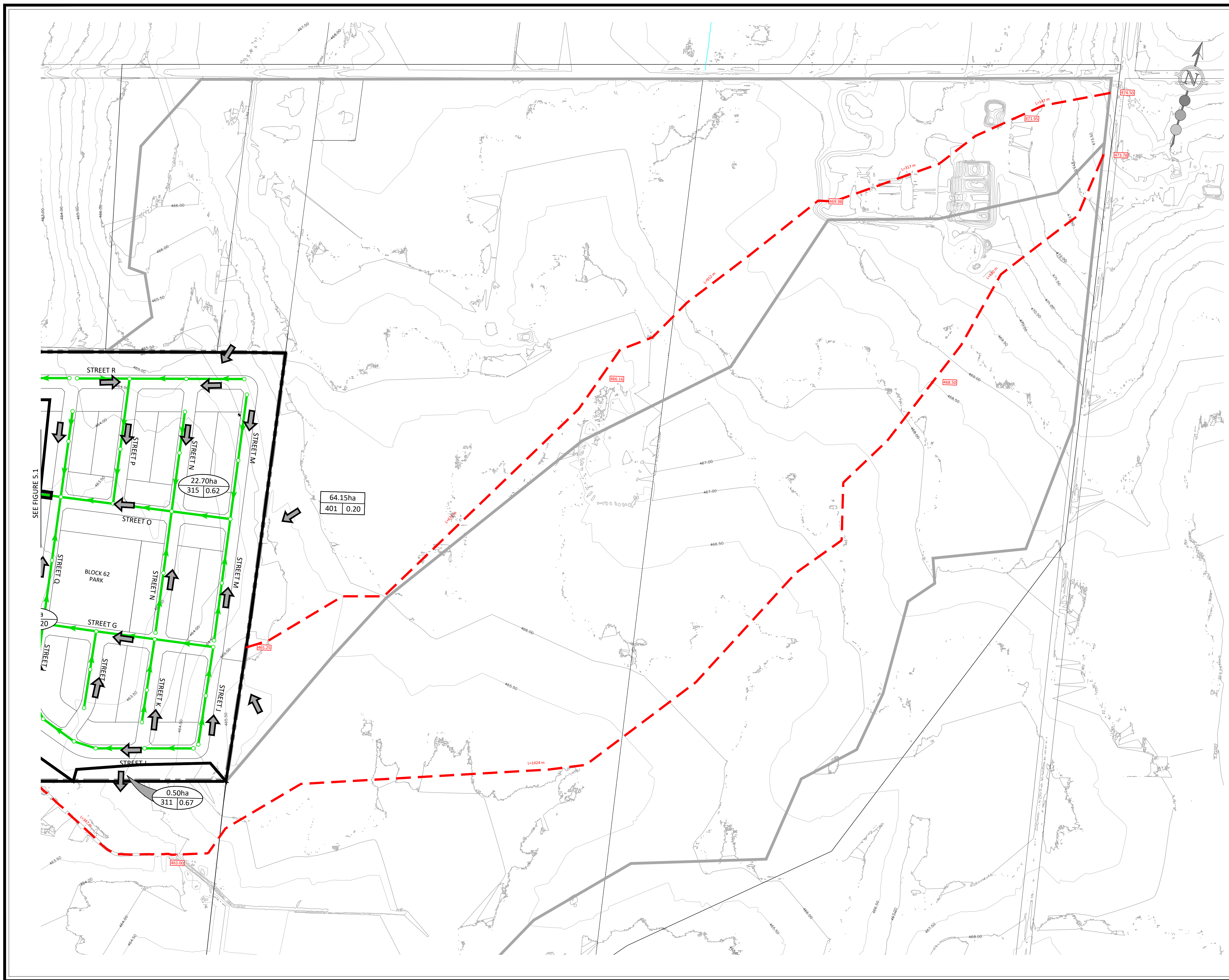
TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

PROPOSED STORM DRAINAGE PLAN

DESIGNED BY: L.J.W. CHECKED BY: P.A.T.
 SCALE: 1:2500 DATE: FEBRUARY 2025

PROJECT No: 2504 FIGURE No: 5.1



- LEGEND:**
- LIMIT OF PARTICIPATING PROPERTY
 - PROPOSED STORM DRAINAGE BOUNDARY
 - EXTERNAL STORM DRAINAGE BOUNDARY
 - EXISTING CONTOUR AND ELEVATION
 - MAJOR SYSTEM, OVERLAND FLOW
 - DRAINAGE AREA (HECTARES)
 - RUNOFF COEFFICIENT
 - CATCHMENT ID
 - EXTERNAL STORM DRAINAGE AREA (HECTARES)
 - RUNOFF COEFFICIENT
 - CATCHMENT ID
 - TIME OF PEAK FLOW PATH AND LENGTH
 - TIME TO PEAK ELEVATION
 - PROPOSED STORM SEWER AND MAINTENANCE HOLE
 - PROPOSED HEADWALL
 - 100 YEAR CAPTURE LOCATION

SEE FIGURE 5.1

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

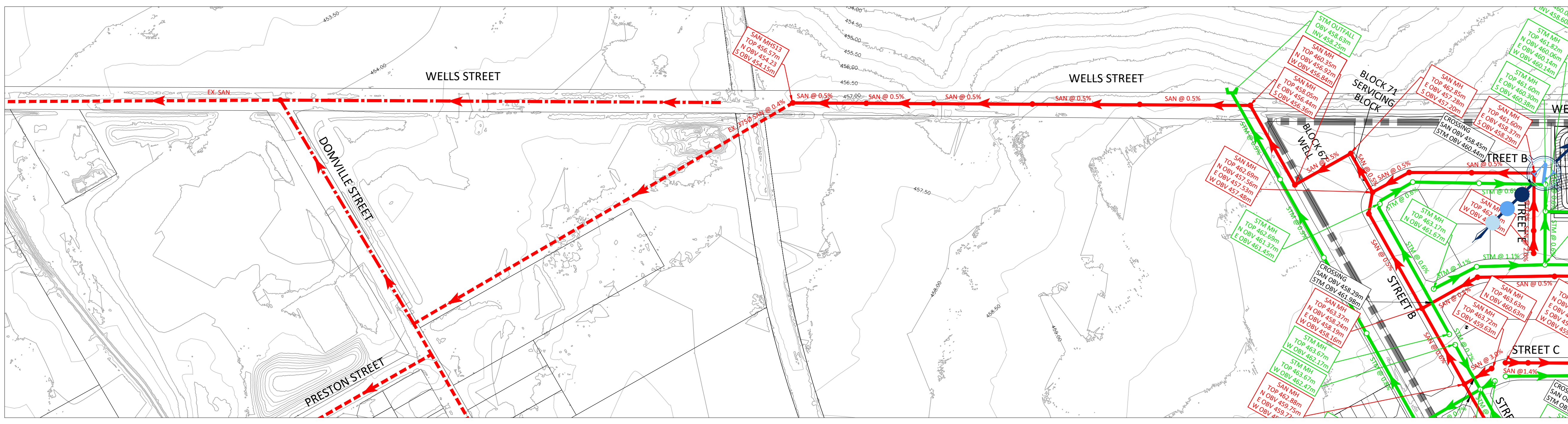
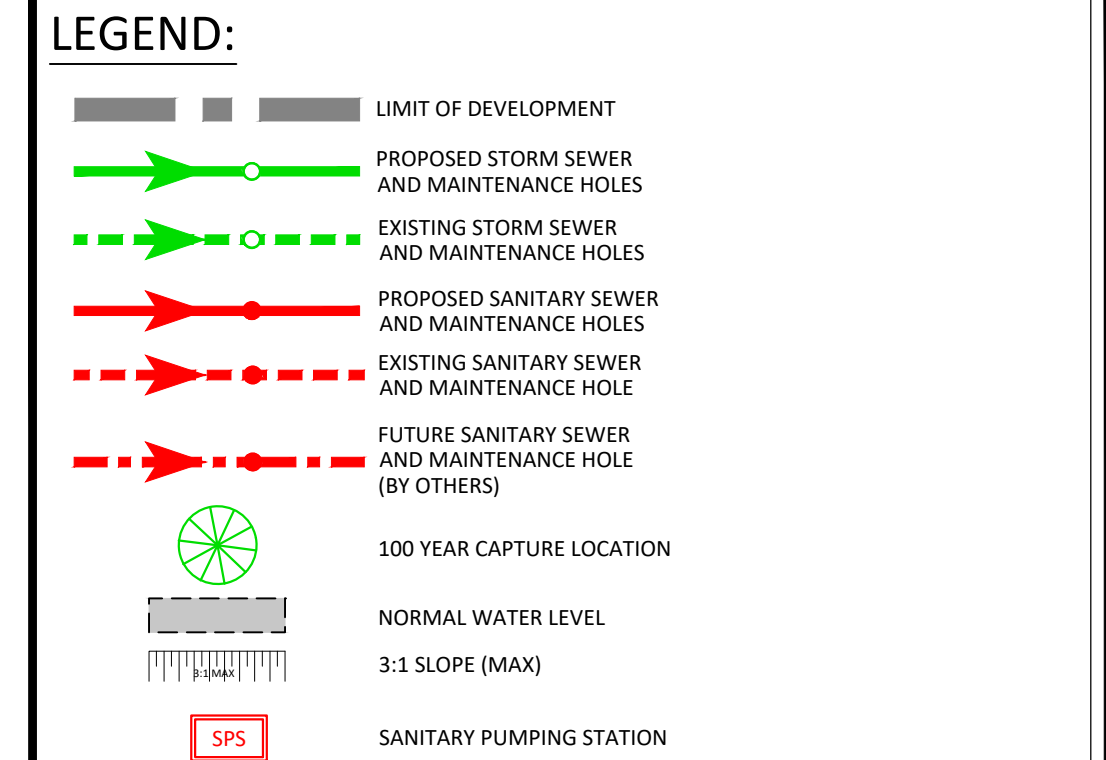
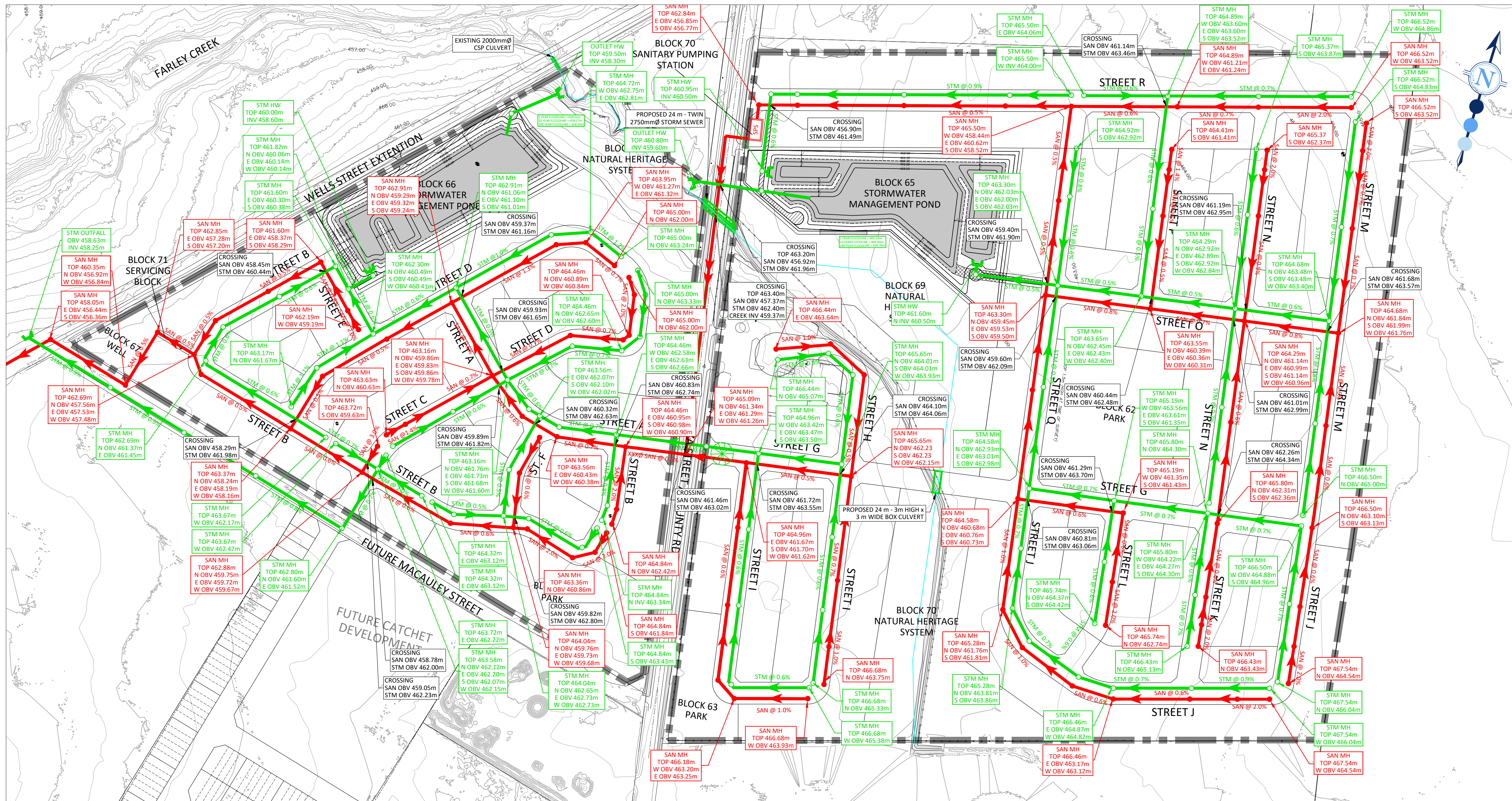
TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

PROPOSED STORM DRAINAGE PLAN

DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2500	DATE: FEBRUARY 2025
PROJECT No:	FIGURE No:

2504 5.2

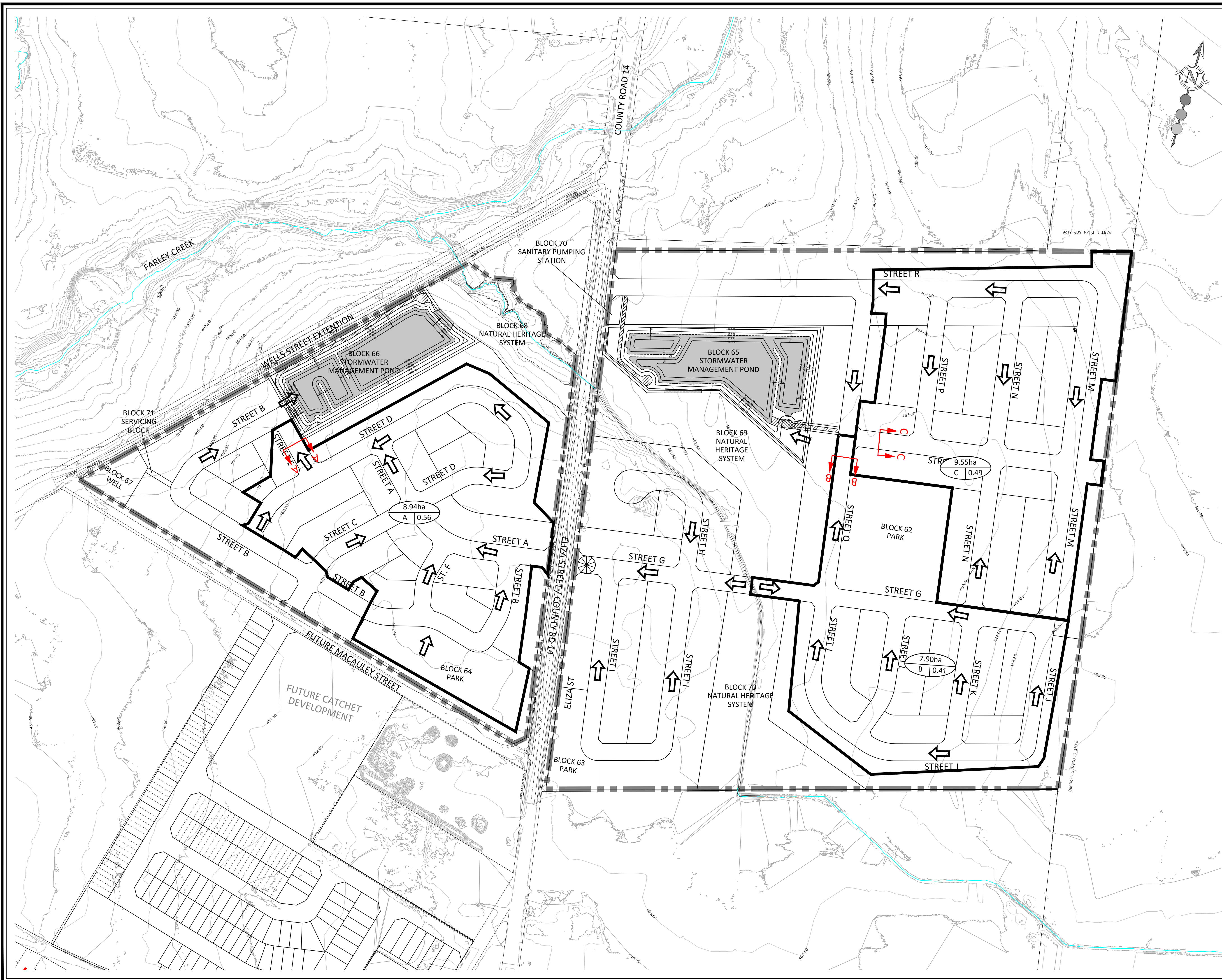


NOTE: LAYOUT IS SCHEMATIC ONLY. DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SGS consulting group ltd
 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.
 ARTHUR WELLINGTON NORTH DEVELOPMENT
 PROPOSED STORM AND
 SANITARY SERVICING PLAN

DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 5.3



- LEGEND:**
- LIMIT OF PARTICIPATING PROPERTY
 - PROPOSED STORM DRAINAGE BOUNDARY
 - EXISTING CONTOUR AND ELEVATION
 - MAJOR SYSTEM - OVERLAND FLOW
 - 100 YEAR CAPTURE LOCATION
 - DRAINAGE AREA (HECTARES)
 - RUNOFF COEFFICIENT
 - CATCHMENT ID
 - CROSS SECTION

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

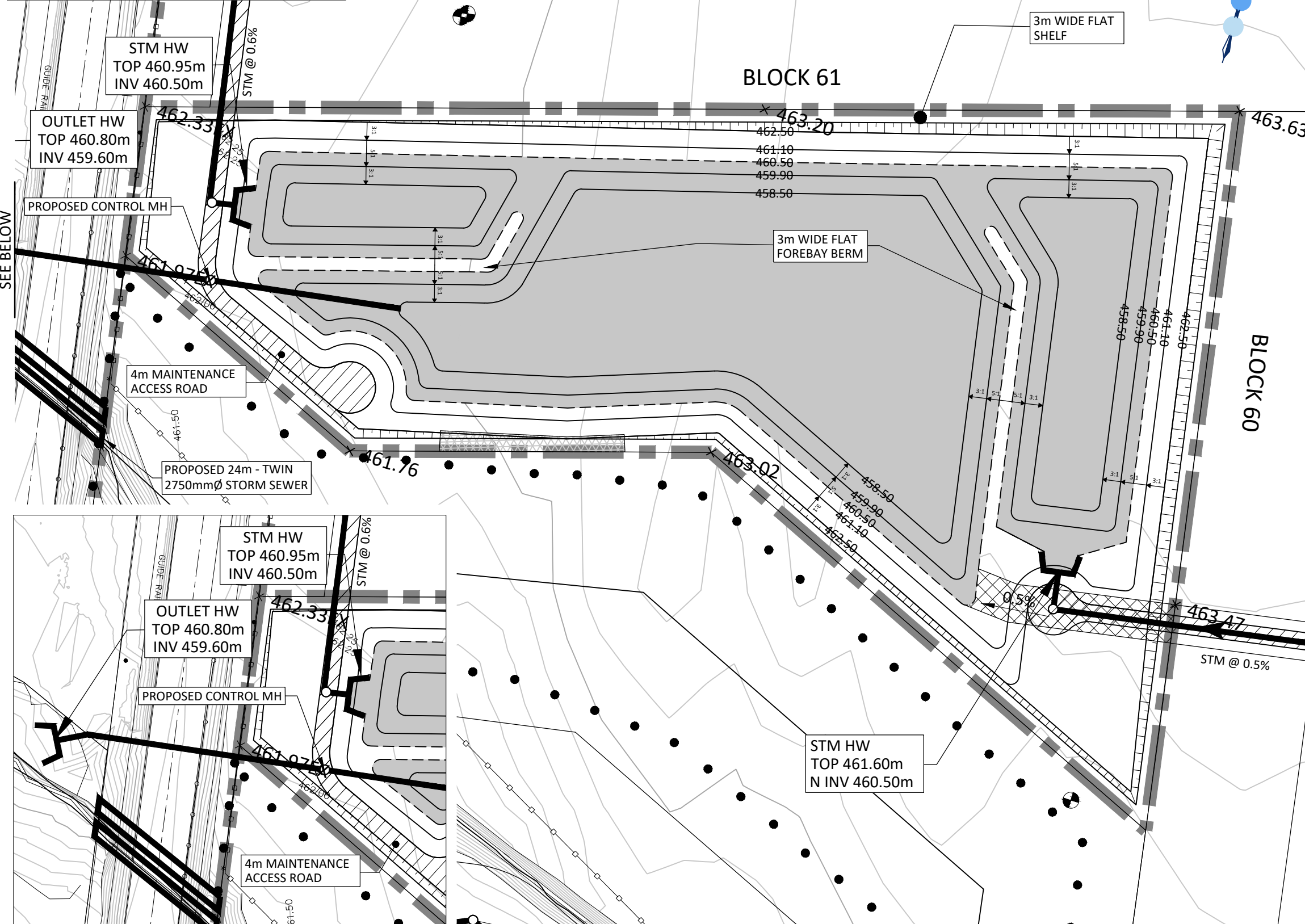
SGS consulting group ltd
 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.
ARTHUR WELLINGTON NORTH DEVELOPMENT
PROPOSED OVERLAND FLOW ROUTE

DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 5.4

EAST SWM POND SUMMARY:

POND BLOCK AREA = 2.40 ha
 PERMANENT POOL REQUIRED = 5,083 m³
 PERMANENT POOL PROVIDED = 19,978 m³
 EXTENDED DETENTION STORAGE REQUIRED = 7,767 m³
 EXTENDED DETENTION MAXIMUM WATER LEVEL = 460.99 m
 100yr ACTIVE STORAGE REQUIRED = 16,115 m³
 100yr MAXIMUM WATER LEVEL = 461.54 m
 FREEBOARD = 0.7 m



LEGEND:

- LIMIT OF SWM POND BLOCK
- EXISTING CONTOUR AND ELEVATION
- STORM SEWER AND MANHOLE
- NORMAL WATER LEVEL CONTOUR AND ELEVATION
- PROPOSED CONTOUR AND ELEVATION
- PROPOSED POND SLOPE
- PROPOSED ELEVATION
- EXISTING ELEVATION
- PROPOSED REGULATORY (REGIONAL STORM) FLOODPLAIN (SCS, 2025)
- MAINTENANCE ACCESS ROAD
- EMERGENCY SPILLWAY
- NORMAL WATER LEVEL
- OVERLAND FLOW ROUTE
- MAX 3:1 SLOPE
- FLOW ROUTE ARROW

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SCS consulting group ltd
 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.
ARTHUR WELLINGTON NORTH DEVELOPMENT


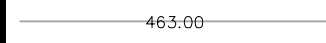
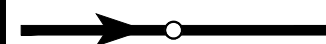
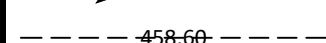
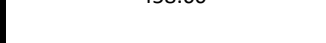
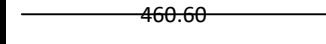
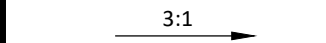

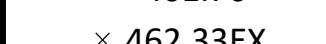
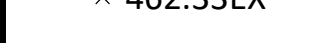


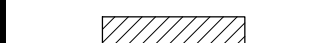






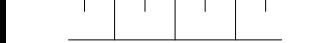

EAST STORMWATER MANAGEMENT POND

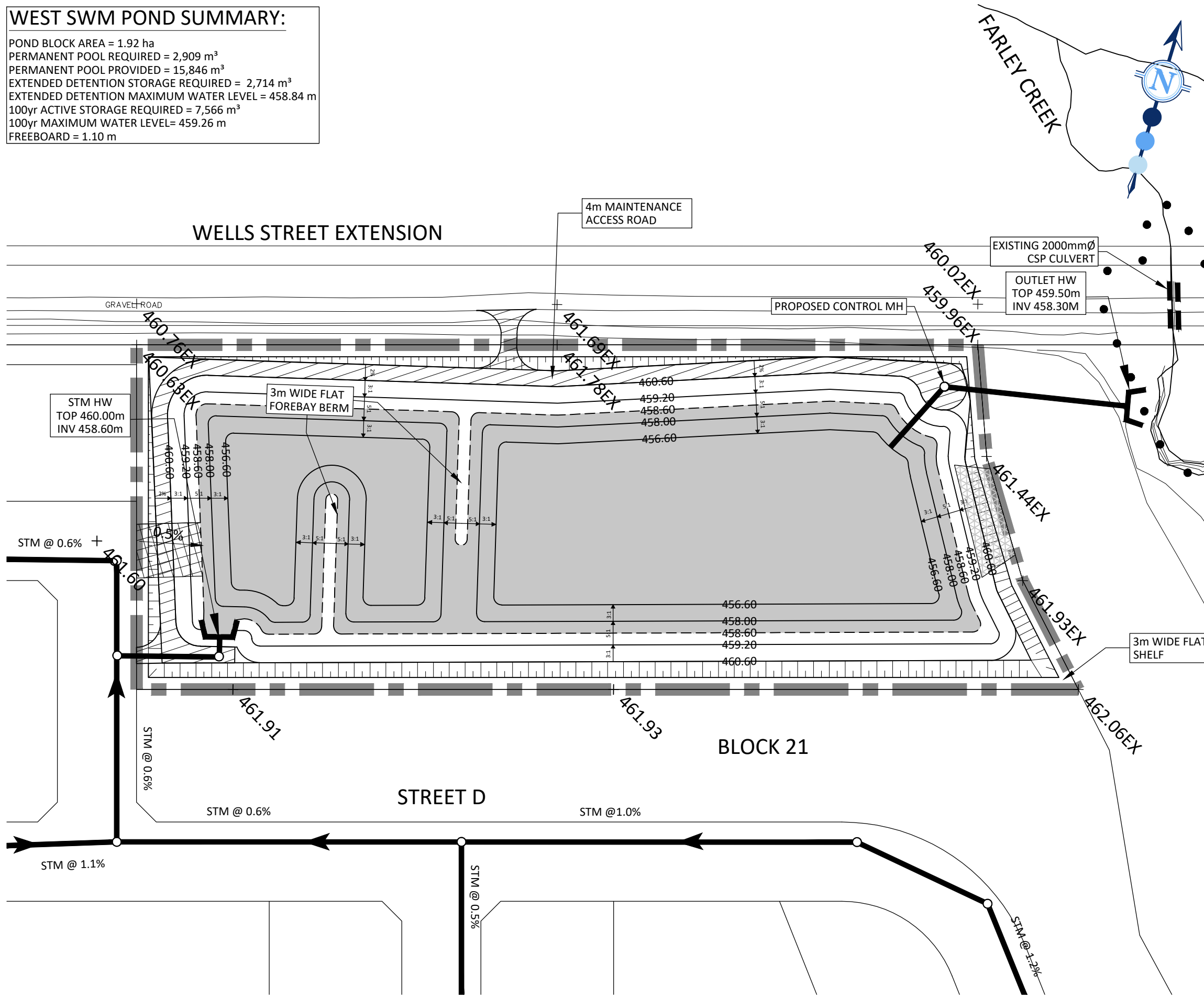
DESIGNED BY: J.H.	CHECKED BY: P.A.T.
SCALE: 1:1000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 6.1

WEST SWM POND SUMMARY:

POND BLOCK AREA = 1.92 ha
 PERMANENT POOL REQUIRED = 2,909 m³
 PERMANENT POOL PROVIDED = 15,846 m³
 EXTENDED DETENTION STORAGE REQUIRED = 2,714 m³
 EXTENDED DETENTION MAXIMUM WATER LEVEL = 458.84 m
 100yr ACTIVE STORAGE REQUIRED = 7,566 m³
 100yr MAXIMUM WATER LEVEL = 459.26 m
 FREEBOARD = 1.10 m

LEGEND:

-  LIMIT OF SWM POND BLOCK
-  EXISTING CONTOUR AND ELEVATION
-  463.00
-  458.60
-  460.60
-  PROPOSED CONTOUR AND ELEVATION
-  3:1
-  PROPOSED POND SLOPE
-  × 461.76
-  × 462.33EX
-  PROPOSED ELEVATION
-  × 462.33EX
-  EXISTING ELEVATION
-  PROPOSED REGULATORY (REGIONAL STORM) FLOODPLAIN (SCS, 2025)
-  MAINTENANCE ACCESS ROAD
-  EMERGENCY SPILLWAY
-  NORMAL WATER LEVEL
-  OVERLAND FLOW ROUTE
-  MAX 3:1 SLOPE
-  0.5%
-  FLOW ROUTE ARROW



*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

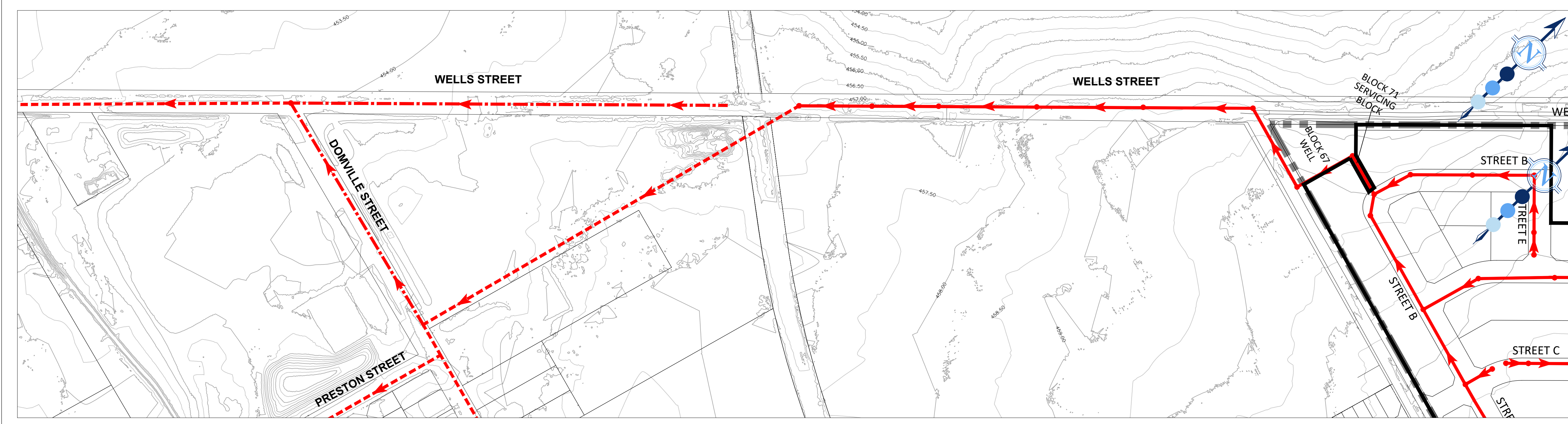
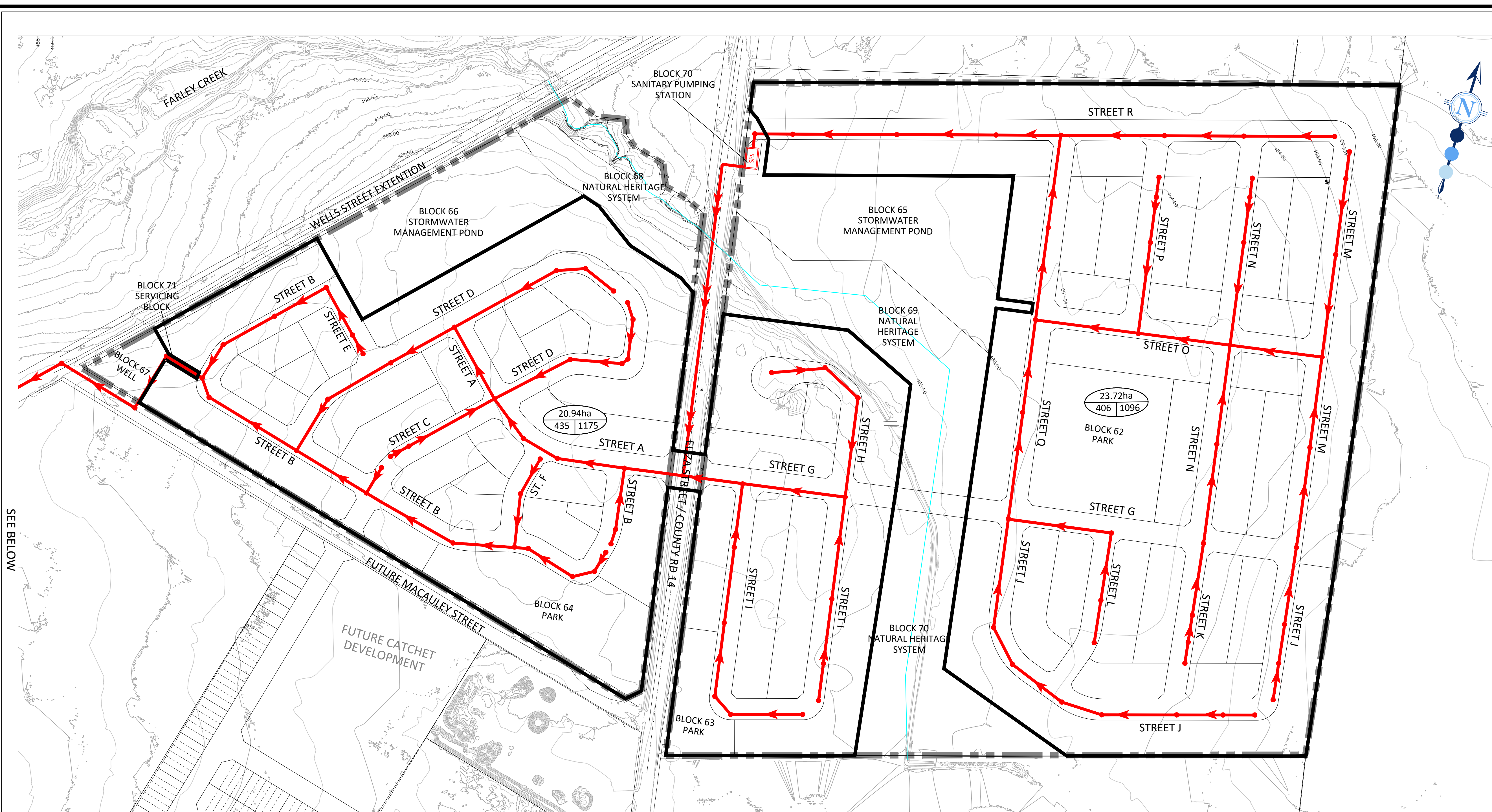
SCS consulting group ltd
 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR WELLINGTON NORTH DEVELOPMENT

WEST STORMWATER MANAGEMENT POND

DESIGNED BY: J.H.	CHECKED BY: P.A.T.
SCALE: 1:1000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 6.2

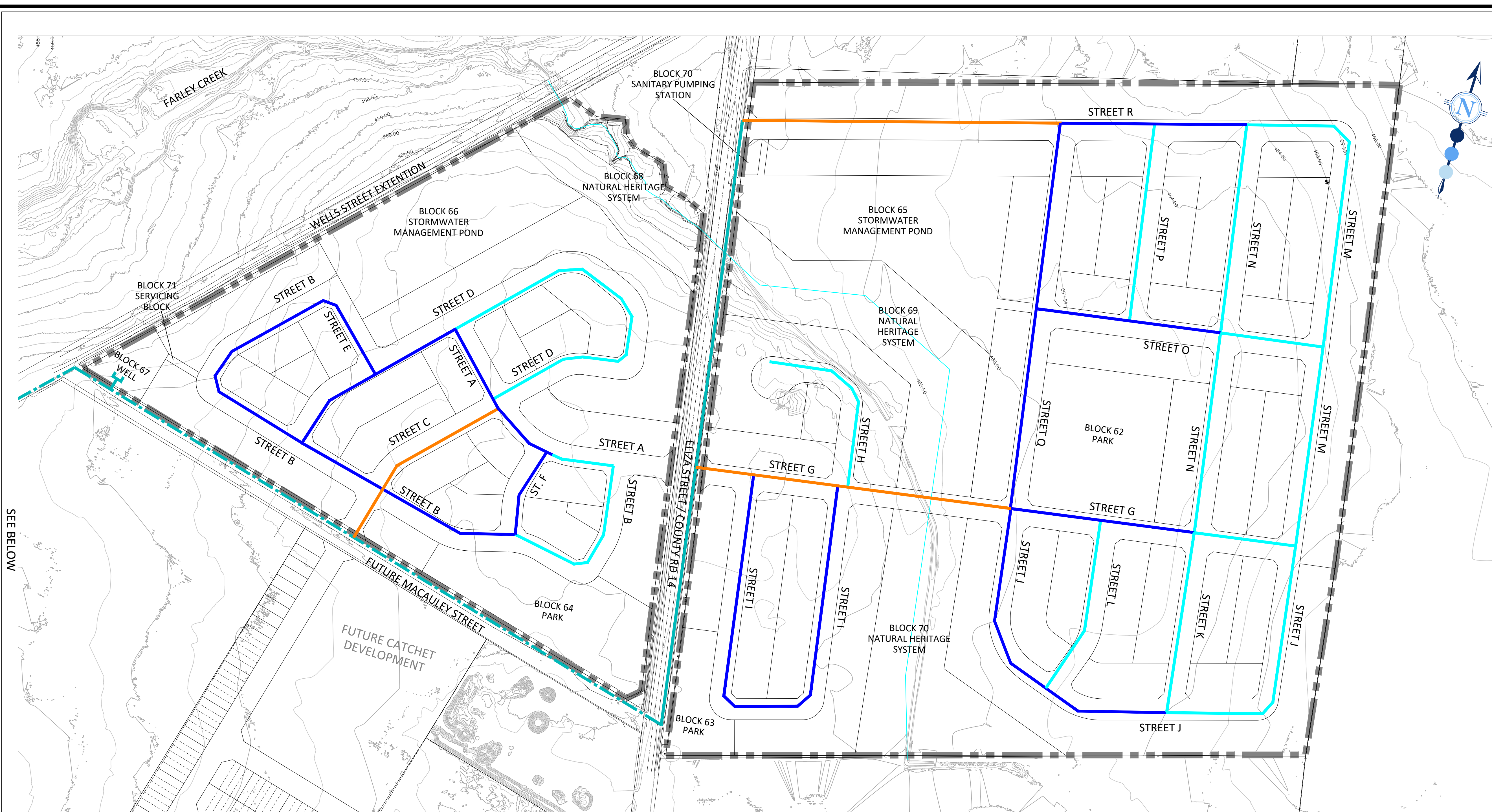


- LEGEND:**
- LIMIT OF PARTICIPATING PROPERTY
 - EXISTING CONTOUR AND ELEVATION
 - PROPOSED SANITARY SEWER AND MAINTENANCE HOLE
 - FUTURE SANITARY SEWER AND MAINTENANCE HOLE (BY OTHERS)
 - EXISTING SANITARY SEWER
 - PROPOSED SANITARY SEWER PLUG
 - SANITARY CATCHMENT BOUNDARY
 - DRAINAGE AREA (HECTARES)
EQUIVALENT SANITARY POPULATION
 - NUMBER OF UNITS

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

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 FAX: (905) 475-8335

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.	
ARTHUR WELLINGTON NORTH DEVELOPMENT	
PROPOSED SANITARY DRAINAGE PLAN	
DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 7.1



LEGEND:

	LIMIT OF PARTICIPATING PROPERTY
	EXISTING CONTOUR AND ELEVATION
	PROPOSED 1500 WATERMAIN
	PROPOSED 2000 WATERMAIN
	PROPOSED 2500 WATERMAIN
	PROPOSED 3000 WATERMAIN
	3000 FUTURE WATERMAIN
	1500 EXISTING WATERMAIN
	2500 EXISTING WATERMAIN
	3000 WATERMAIN PLUG

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SGS consulting group Ltd

30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

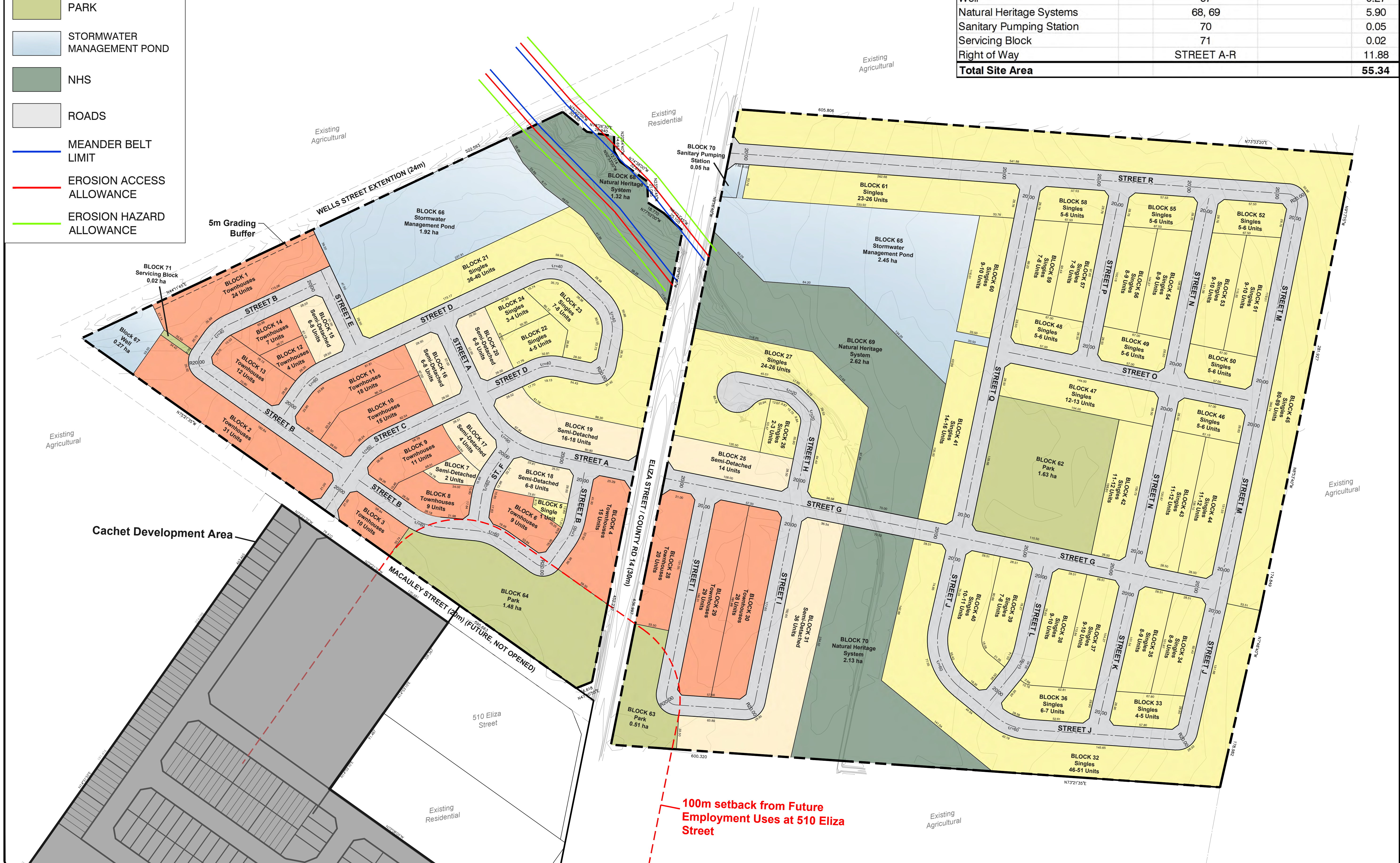
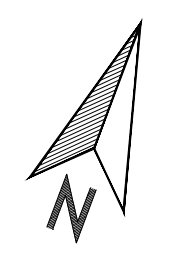
TRIBUTE/SORBARA ARTHUR HOLDINGS INC.	
ARTHUR WELLINGTON NORTH DEVELOPMENT	
PROPOSED WATER DISTRIBUTION PLAN	
DESIGNED BY: L.J.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: FEBRUARY 2025
PROJECT No: 2504	FIGURE No: 8.1

Appendix A Draft Plan



LEGEND

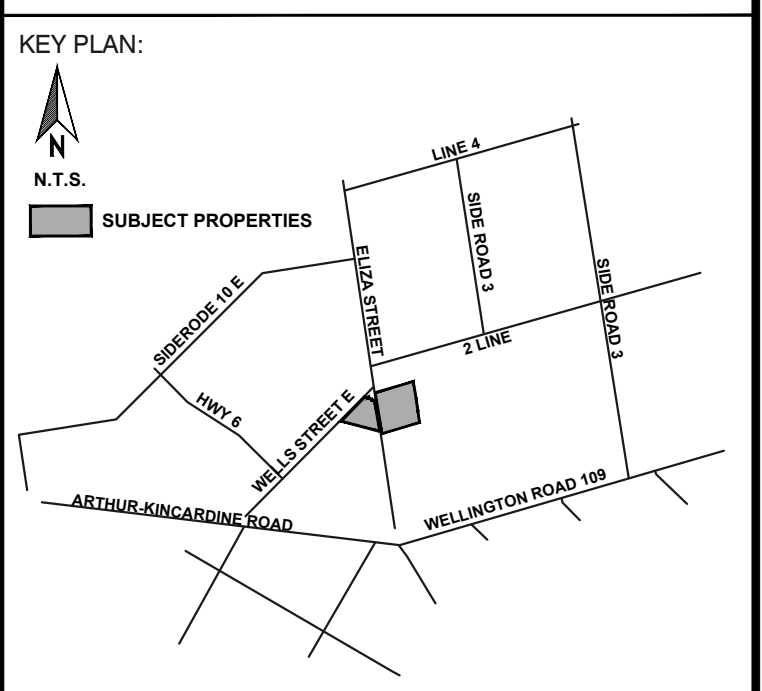
- 36' AND 40' SINGLES
- 20' FREEHOLD TH
- 25' SEMIS
- PARK
- STORMWATER MANAGEMENT POND
- NHS
- ROADS
- MEANDER BELT LIMIT
- EROSION ACCESS ALLOWANCE
- EROSION HAZARD ALLOWANCE



Schedule of Land Use			
Description	Lot / Block No.	Residential Units	Area (ha)
Single Detached Residential	5, 21-24, 26, 27, 32-61	454-504	19.96
Semi-Detached	7, 15-20, 25, 31	112-113	3.21
Street Townhouse	1-4, 6, 8-14, 28-30	249	6.05
Net Developable Total		815-866	29.22
Park	62-64		3.62
Stormwater Management Pond	65, 66		4.38
Well	67		0.27
Natural Heritage Systems	68, 69		5.90
Sanitary Pumping Station	70		0.05
Servicing Block	71		0.02
Right of Way	STREET A-R		11.88
Total Site Area			55.34

TITLE:
DRAFT PLAN OF SUBDIVISION

LEGAL DESCRIPTION:
PART OF PARK LOTS 1 AND 2
NORTH OF MACAULEY STREET
CROWN SURVEY
AND
PART LOT 1 CONCESSION 2
WEST LUTHER AS IN R0N174408
TOWNSHIP OF WELLINGTON NORTH
COUNTY OF WELLINGTON



REQUIRED INFORMATION:
AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT R.S.O. 1990.

(a) SEE PLAN (g) SEE PLAN (i) SEE PLAN (m) SEE PLAN (o) SEE PLAN (s) SEE PLAN (u) SEE PLAN (w) SEE PLAN (y) SEE PLAN

(b) SEE PLAN (h) PIPED WATER TO BE PROVIDED (j) SEE PLAN (n) SEE PLAN (p) SEE PLAN (r) SEE PLAN (t) SEE PLAN (v) SEE PLAN (x) SEE PLAN (z) SEE PLAN

(c) SEE KEY MAP (f) SEE PLAN (l) SEE PLAN (q) SEE PLAN (s) SANITARY & STORM SEWERS TO BE PROVIDED (u) SEE PLAN (w) SEE PLAN (y) SEE PLAN

NOTE: CONTOURS RELATE TO CANADIAN GEODETIC DATUM

SURVEYOR'S CERTIFICATE:
I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATE AND CORRECTLY SHOWN IN ACCORDANCE WITH A PLAN OF SURVEY PREPARED BY J.D. BARNES LIMITED

RAYMOND J. SIBTHORP O.L.S.

DATE

OWNER'S CERTIFICATE:
I HEREBY AUTHORIZE THE BIGLIERI GROUP LTD. TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE COUNTY OF WELLINGTON

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

DATE

ARTHUR, WELLINGTON NORTH DEVELOPMENT

APPROVAL STAMP:

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

REVISIONS

No.	Description	Date	Int.
3			
2			
1			

PROJECT No.: 22853
DATE: January 14, 2025
SCALE: 1:1750
DRAFTED BY: EC CHECKED BY: MP
DRAWING No.: **DP-01**

BIGLIERI GROUP

2472 Kingston Road, Toronto
21 King Street W. Suite 1100, Hamilton
(416) 693-6165
thebiglierigroup.com

100m setback from Future Employment Uses at 510 Eliza Street

Appendix B Background Information



Appendix B1 Geotechnical Site Investigation





GEMTEC

www.gemtec.ca

Submitted to:

Tribute/Sorbara Arthur Holdings Inc.
1815 Ironstone Manor, Unit 1
Pickering, Ontario
L1W 3W9

**Geotechnical Site Investigation
Proposed Residential Development
665 Eliza Street
Arthur, Ontario**

September 26, 2024
GEMTEC Project: 101764.038

2.2 Site Geology

According to published physiographic mapping (*L.J. Chapman and Putnam, 1984; The Physiography of Southern Ontario, Third Edition*) the site is within a physiographic region known as the Stratford Till Plain.

The Stratford Till Plain refers to an area of broad clay plain extending from London in the south of Blyth and Listowel in the north with a projection toward Arthur and Grand Valley. The area is featured by moraines and interrupted by terminal moraines mostly at the southwestern portion and flat areas at the northern half of the region (modified by one or two moraines).

The soils encountered in this region consist of silty clay (heavy textured calcareous) till, silt, and clay mostly from previously deposited varved clays of the Lake Huron basin. The silt and clays are shallow deposits overlying the silty clay till. Sand or gravel is sometimes present in the intermorainal valleys south of St. Marys. The site is likely within the Brookston series which consist of an upper silt and clay loam, greyish brown/brownish grey mottled with olive/yellowish brown clay loam overlying a clay calcareous till.

3.0 PREVIOUS SUBSURFACE INVESTIGATION

A preliminary geotechnical investigation was previously carried out at the property by HLV2K Engineering Ltd. (HLV2K) titled “*Preliminary Geotechnical Investigation Report, 665 Eliza Street, Arthur, Ontario*” dated May 5, 2022.

During the previous geotechnical investigation, six boreholes, noted as Boreholes BH1 to BH6 were advanced in April 2022 within the site, and ranged in depth between about 5.8 m and 6.5 m below ground surface (Elev. 457.4 m and Elev. 459.8 m).

The subsurface condition consisted of compact to dense sandy silt and compact to very dense sandy clayey silt till. The cohesive deposits have been described like a non-cohesive deposit. However, it is our opinion that the cohesive soils encountered in HLV2K’s boreholes are generally of a stiff to hard consistency based on standard practice. Further, the sandy silt described in these boreholes may be of a cohesive texture.

A total of three single monitoring wells were installed in Boreholes BH1 to BH3 as part of the previous geotechnical investigation. Groundwater levels were manually measured in the monitoring wells on April 12, 2022, by HLV2K and ranged from about 1.1 m to 1.8 m below ground surface (between approximately Elev. 462.4 m to Elev. 465.0 m).

During the previous geotechnical investigation, the approximate elevations at the borehole locations were surveyed by HLV2K personnel and the geodetic datum was not recorded in the report. During GEMTEC’s site visit, the monitoring wells installed in Boreholes BH2 and BH3 were not present and may have been damaged due to agricultural activities.

The approximate borehole locations for the previous geotechnical investigation are shown in Appendix C. Descriptions of the subsurface conditions observed in these boreholes are provided on the Record of Borehole Sheets in Appendix D for completeness. A total of four grain size analysis were conducted during the previous investigation, and have been provided in Appendix E.

4.0 CURRENT SITE INVESTIGATION METHODOLOGY

The field work for the current geotechnical site investigation was carried out between June 25 and 27, and on July 25, 2024. Eleven boreholes, noted as Boreholes BH24-1 to BH24-11 were advanced between approximately 6.0 m and 9.0 m below ground surface.

The boreholes were advanced using a track mounted drill rig operated by TCI Field Services of Whitby, and Aardvark Drilling Inc. of Guelph Ontario, who are a MECP-licensed Water Well Contractor. The field work was observed throughout by a member of our geotechnical engineering staff who directed the drilling operations and logged the samples and boreholes.

Standard penetration tests (SPT) were carried out in the boreholes and samples of the soils encountered were recovered using a 50-millimetre diameter split spoon sampler. The SPT N values were measured directly in the field and are unfactored.

Single monitoring wells were installed at Boreholes BH24-6, BH24-8, BH24-10 and BH24-11. A bi-level monitoring well was installed at Borehole BH24-1. The monitoring wells were constructed using nominal 50 mm diameter, Schedule 40 polyvinyl chloride (PVC) pipe with a No. 10 machine slotted screen (0.01-inch slot). The annular space between the monitoring well screen and surrounding soils was backfilled with a silica sand filter to a maximum of 0.3 m above the top of the screen, and the remainder of the annular space was sealed with bentonite. All monitoring wells were completed with flush-mounted protective steel casings at ground surface. The monitoring well installation details are provided in the Record of Borehole Sheets (Appendix D).

Co-ordination for clearances of underground utilities was provided by GEMTEC. The borehole locations were selected by GEMTEC and positioned on site relative to existing features, and underground and above ground utility constraints. Upon completion of drilling, the ground surface and top of monitoring well standpipe elevations and coordinates at the borehole and monitoring well locations completed by J.D. Barnes Ltd. on September 23, 2024, and provided to GEMTEC. The borehole elevations are geodetic and referenced to local datums (CGVD-1928:1978). The coordinates at the borehole and monitoring well locations were referenced to the Universal Transverse Mercator (UTM) Zone 17, NAD 1983).

Following completion of the drilling, the soil samples were returned to our Oshawa laboratory for examination by a geotechnical engineer. Selected samples were submitted for grain size distribution, Atterberg limits, and moisture content testing.

The borehole and monitoring well locations are shown on the Borehole Location Plan in Appendix C. Descriptions of the subsurface conditions observed in the boreholes are provided on the Record of Borehole Sheets in Appendix D. The results of the geotechnical laboratory tests are provided on the Record of Boreholes and in Appendix E.

5.0 SUBSURFACE CONDITIONS

As previously indicated, the soil and groundwater conditions identified in the boreholes are shown on the Record of Borehole Sheets in Appendix D. The Record of Boreholes indicate the subsurface conditions at the specific borehole locations only. Boundaries between the different soils on the Records are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at locations other than the boreholes may vary from the conditions encountered in the boreholes, both laterally and with depth. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

In general, the subsurface conditions at the site consist of silty clay deposit underlain by silty clay till deposit and silty clay to clayey silt till deposit. Silt and sand deposit were observed in some borehole locations.

5.1 Topsoil

A veneer of topsoil was encountered at ground surface in all boreholes, with thickness ranging from approximately 80 mm to 130 mm.

5.2 Silty Clay (CL)

A cohesive silty clay deposit consisting of trace sand and trace gravel was encountered underlying topsoil in Boreholes BH24-1(S/D) to BH24-5 and BH24-7 to BH24-11. The silty clay deposit extended to depths ranging from 1.4 m to 6.6 m below ground surface. Oxidation staining, rootlets, and rusty mottling were observed within the silty clay deposit.

Standard penetration tests carried out in the silty clay deposit gave SPT N-values ranging from about 4 blows to 65 blows per 0.3 m of penetration, which generally indicates a soft to hard consistency, but generally a firm to stiff consistency.

Grain size distribution testing was undertaken on one sample of the cohesive silty clay from Borehole BH24-1. The results are provided in Appendix E and are summarized in Table 5.1.

Table 5.1 – Summary of Grain Size Distribution Test (Silty Clay Deposit)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt / Clay (%)
BH24-1	5	3.1 – 3.5	1.6	13.1	85.3

Atterberg limit testing was carried out on two samples of the silty clay deposit obtained from Boreholes BH24-1, and the results gave a plastic limit of about 16 percent, a liquid limit of about 29 percent, and a plasticity indices of about 14; and indicates a silty clay of low plasticity. The results are shown on the Record of Boreholes and in Appendix E.

The water content measured on samples of the cohesive silty clay deposit ranged from about 7 percent to 23 percent.

5.3 Silty Clay Till (CL)

A cohesive silty clay glacial till, sandy to some sand, trace to some gravel, was encountered in all boreholes, underlying the topsoil or silty clay deposit. Oxidation staining, sand pockets, and silty sand and silty clay lens were observed within the cohesive till deposit at various borehole locations. The boreholes were terminated within the silty clay till deposit.

It is anticipated that cobbles and / or boulders may be present within the glacially derived deposits which can be inferred from auger grinding and split spoon sampling not fully penetrating the tills during drilling operations.

Standard penetration tests carried out in the cohesive till deposit gave SPT N-values ranging from about 8 blows to 75 blows per 0.3 m of penetration and 50 blows per 0.13 m of penetration, which indicates a stiff to hard consistency, but generally of a very stiff to hard consistency.

The water contents measured on samples of the cohesive till deposit ranged from 9 percent to 18 percent.

5.4 Silt to Silt and Sand (ML)

A non-cohesive silt and sand deposit was encountered in Borehole BH24-7 only, interlayered within the cohesive till deposit. A non-cohesive silt of slight plasticity was encountered in Borehole BH24-8 underlying the silty clay deposit and overlying the silty clay till deposit.

One standard penetration test carried out in the silt and sand deposit gave an SPT N-value of 12 blows per 0.3 m of penetration, which indicates a compact compactness condition. One standard penetration test carried out in the silt deposit gave an SPT N-value of 85 blows per 0.3 m of penetration, which indicates a very dense compactness condition.

One grain size distribution test was undertaken on each deposit, the silt and sand deposit, and silt of slight plasticity from Boreholes BH24-7 and BH24-8, respectively. The results are provided in Appendix E and are summarized in Table 5.2.

Table 5.2 – Summary of Grain Size Distribution Test (Silt to Silt and Sand Deposits)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt /Clay (%)
BH24-7	6	4.6 – 5.0	0.9	45.7	53.4
BH24-8	5	3.0 – 3.4	14.5	4.4	81.1

The water content measured on two samples of the non-cohesive deposits were about 13 percent and 15 percent.

Atterberg limits test was carried out on one sample of the silt deposit in Borehole BH24-8 and gave a plastic limit of 15, a liquid limit of 18 and a plasticity index of 3 which indicates a silt with slight plasticity. The results are shown on the Record of Boreholes and in Appendix E.

5.5 Groundwater Levels

5.5.1 Groundwater Levels in GEMTEC’s Monitoring Wells

The unstabilized groundwater level was measured in the open borehole upon completion of drilling and ranged from 3.0 m to 5.3 m below ground surface. Boreholes BH24-1(S/D) to BH24-4, and Boreholes BH24-8 to BH24-11, were observed to be dry upon completion of drilling.

Groundwater levels were measured in the monitoring wells installed in Boreholes BH24-1(S/D), BH24-6, BH24-8, BH24-10 and BH24-11, between July and August 2024. The groundwater levels and elevations have been provided in Table 5.3.

Table 5.3 – Approximate Groundwater Depths and Elevations (Wells Installed During Current Investigation)

Monitoring Wells	Groundwater Below Existing Ground Surface (metres)				Groundwater Elevation (metres)			
	09-Jul-24	18-Jul-24	02-Aug-24	16-Aug-24	09-Jul-24	18-Jul-24	02-Aug-24	16-Aug-24
BH24-1S	1.1	0.7	1.0	1.2	461.3	461.7	461.4	461.2
BH24-1D	8.7	5.2	4.4	3.1	453.6	457.1	457.9	459.2
BH24-6	3.5	1.2	1.3	1.3	459.0	461.2	461.2	461.1
BH24-8	1.6	1.3	1.4	1.6	461.6	461.9	461.8	461.6
BH24-10	N/A	N/A	4.2	3.0	N/A	N/A	460.8	462.1
BH24-11	N/A	N/A	1.3	1.6	N/A	N/A	463.7	463.4

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

5.5.2 Groundwater Levels in HLV2K’s Monitoring Well

During the fieldwork, the groundwater levels were measured in the existing monitoring well installed in Borehole BH1 advanced by HLV2K and are summarized in Table 5.4.

Table 5.4 – Approximate Groundwater Depths and Elevations (Well Installed During Previous Investigation)

Monitoring Wells	Groundwater Below Existing Ground Surface (metres)			Groundwater Elevation (metres)		
	July 9, 2024	July 18, 2024	August 2, 2024	July 9, 2024	July 18, 2024	August 2, 2024
BH1	N/A	1.2	2.0	N/A	461.8	461.0

6.0 GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

This section of the report provides guidance on the geotechnical engineering design aspects of the project based on our interpretation of the boreholes advanced as part of the preliminary site investigation. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Additional geotechnical and

hydrogeological investigation will be required as the development details become more available. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety, and equipment capabilities.

The professional services retained for this project include only the geotechnical engineering and hydrogeological aspects of the subsurface and groundwater conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from offsite sources are outside the terms of reference for this report and have not been investigated or addressed.

Based on the result of the investigations, the subsurface soil conditions encountered at the site are considered to be generally suitable for the proposed development, utilizing conventional shallow strip and spread footings, with slab-on-grade or basement construction, serviced underground utilities, and overall site development. Consideration will need to be given to groundwater control and management during both construction and, depending on the final grading, long-term operations (i.e., stormwater ponds, basement foundations).

6.1 Site Preparation and Site Grading

Existing structures and associated foundations, septic system and water well, weeping tiles, vegetation, topsoil, and any soil containing organics should be stripped / removed / decommissioned from the site prior to placement of engineered fill. Outside of road allowances, utility corridors, and building envelopes and the like, the topsoil may be reused for general grading in areas where the fill is not required to support settlement sensitive structures. Any oversized cobbles, boulders and other deleterious materials should be excluded from the reuse as site grading fill.

Where the topsoil is used as general lot fill (i.e., non-structural fill), its thickness should be limited to about 1.5 m. The topsoil fill should be placed in maximum 300 mm loose lifts and uniformly compacted to 95 percent of Standard Proctor Maximum Dry Density (SPMDD). To have any success in placing topsoil as lot grading fill, it must be placed at or very close (+/- 2%) to its optimum water content to achieve workability and adequate compaction, in order to minimize post-construction settlements and/or lateral movements (e.g., of fences, etc.).

Upon completion of topsoil stripping, the near surface silty clay deposit should be heavily proofrolled and compacted under the direction of qualified geotechnical personnel.

Preliminary grading plans were not available at the time of this report, however, due to the topography of the site, cut and fill operations is anticipated to be required in order to achieve final grades within the site.

Depending on the grade raise and based on the existing subsurface conditions, long-term settlements of the firm silty clay within 0.5 m and 1.2 m below existing grades (depending on the borehole location) should be anticipated. However, based on our local experience and understanding of the subsurface conditions, this is anticipated to take 2 to 3 months to achieve 90 percent of the consolidation settlement assuming a grade raise of up to 2 m. Once the design is advanced, GEMTEC should be given the opportunity to review the available information and provide updated commentary on the impacts to underlying materials.

6.2 Engineered Fill

At the time of this report preparation, preliminary grading plans were not available. However, significant grade raise is anticipated to achieve final grades within the site. Any significant fills (greater than 1 m) should be reviewed by GEMTEC to assess the anticipated short and long-term settlements of the underlying soils and their potential impact on the overall site development.

Based on the soil classification and frost group described in Table 13.1 of the Canadian Foundation Engineering Manual (CFEM, 2006), the cohesive deposit and cohesive glacial till encountered on the site are regarded as being high frost susceptibility. This should be considered for any design elements exposed to freezing temperatures (concrete flatworks, exterior concrete slabs, and the like). The silty clay deposit are not suitable for reuse as engineered fill due to the potential to induce long-term and differential settlement below the residential building foundations.

The approved engineered fill should consist of imported material which meets the requirements for OPSS.PROV 1010 Select Subgrade Material (SSM), Granular A and placed in maximum 300 mm loose lifts and compacted to a minimum 98 percent SPMDD or as noted otherwise. The water contents at the time of placement should be within +/- 2 percent of its optimum moisture content to achieve the required compaction. Further to this, soft / loose / deleterious materials, and cobbles and boulders should be removed during fill placement.

After stripping and sub-excavating to the founding elevations (or sub-excavation depth, where required), the prepared subgrade should be proof rolled (where possible) and inspected by qualified geotechnical personnel to confirm that the foundation soils are uniform and consistent with those encountered in the boreholes and are free of any softened / loosened or deleterious materials. Locations where less competent subgrade conditions (i.e., soft / loose soil, organic soils, or other deleterious materials) are identified during subgrade inspection should be sub-excavated and replaced with engineered fill.

It is strongly recommended that construction should be carried out under dry conditions. If construction is required during freezing temperatures, the subgrade should be protected immediately from freezing by placing additional soil cover above the final subgrade, to be removed prior to continuing construction, to provide temporary frost protection.

6.3 Shallow Foundations

6.3.1 Residential Buildings

Shallow spread footings and strip foundations are considered feasible foundation elements for the proposed structures and buildings within engineered fill and competent native soils consisting of firm to hard silty clay and stiff to hard cohesive till deposit.

Spread footings founded on or within the engineered fill and the native undisturbed deposit consisting of firm silty clay deposit may be designed using a factored geotechnical resistance at Ultimate Limit State (ULS) of 225 kPa and an unfactored geotechnical reaction at Serviceability Limit State (SLS) of 150 kPa.

Spread footings founded on or within the native stiff to hard silty clay deposit may be designed using a factored geotechnical resistance at ULS of 300 kPa and an unfactored geotechnical reaction at SLS of 200 kPa.

The ULS factored geotechnical resistance values are based on resistance factor of 0.5, while the SLS unfactored reaction values are based on 25 mm and 19 mm of total and differential settlement, respectively; both for footing widths ranging between 900 mm and 2500 mm.

The geotechnical bearing resistance / reaction should be confirmed at the time of construction and assumes that all loose / soft, or otherwise deleterious materials are removed from the bearing surface. All footings should be provided with a minimum of 1.5 m of earth cover, otherwise frost protection measures should be included and can be provided upon request.

Depending on the final grading plan and assuming a basement level at approximately 3 m below final grade, the proposed basement elevations will be founded on the silty clay or cohesive till which may be below or above the groundwater table. Where basements are located below the long term groundwater table, the basement should be designed as a fully drained structure with perimeter and under slab drain at foundation level leading to a permanent frost-free outlet, such as a continuously pumped sump or a direct outlet to a sewer line, should be provided. For basement foundations located above the ground water table, perimeter subdrains will be sufficient.

If stepped spread footings are constructed at different founding levels, the difference in elevation between individual footings should not be greater than one half the clear distance between the footings. Should this not be possible, GEMTEC should be consulted to provide field inspection to ensure that the footings exceeding the above requirement are stable and the bearing for the upper (i.e., higher) footing is not compromised. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly. Stepped strip footings, if required, should be constructed in accordance with the 2012 Ontario Building Code (2012 OBC), Section 9.15.3.9.

The founding materials are susceptible to disturbance by construction activity especially during wet weather and care should be taken to preserve the integrity of the bearing strata, including engineered fill. Prior to pouring concrete for the footings, the foundation excavations must be inspected to confirm that the footings are located on a competent bearing stratum, which has been cleaned of ponded water and loosened or softened material and addressed the potential need to provide remedial work. Sub-excavated soils can be replaced with engineered fill or unshrinkable fill with a minimum compressive strength of 0.7 MPa or as directed by geotechnical personnel.

6.3.2 Slabs-on-Grade (Heated Areas Only)

For slab-on-grade foundations founded on a prepared subgrade, a modulus of subgrade reaction, k_{vb} , is typically used to represent the soil stiffness. The modulus of vertical subgrade reaction (k_{vb}) is not a fundamental soil property, and the value changes with footing size and / or the size of the loaded area(s). The current state of practise uses a standard reference vertical subgrade reaction, k_{v1} , associated with a 0.3 m x 0.3 m (i.e., 1 ft x 1 ft) plate. The modulus of vertical subgrade reaction can be estimated from the equations given below for foundations and / or slabs on non-cohesive or cohesive soils (CFEM, 2006). However, it should be noted that these methods are approximate only and it is generally considered that carrying out a detailed settlement analysis is the more rational approach (once design details are known) to obtaining more realistic values of k_{vb} .

For slab-on-grade foundations, the modulus of subgrade reaction is defined as:

$$k = \frac{q}{\delta}$$

Where:

k is the modulus of vertical subgrade reaction for actual foundation width, b (MPa/m);

q is the applied bearing or contact pressure on the foundation; and,

δ is the settlement of the foundation under the applied pressure q .

For cohesive soils:

$$k_{vb} = \frac{0.3 k_{v1}}{b} \left[\frac{m + 0.5}{1.5m} \right]$$

Where:

k_{vb} is the modulus of vertical subgrade reaction for actual foundation dimension, b (MPa/m);

k_{v1} is the modulus of vertical subgrade reaction for a 1 ft x 1 ft plate (MPa/m);

b is the foundation width (m); and,

m is the ratio of foundation length to width (i.e., $m = L / b$).

For non-cohesive soils:

$$k_{vb} = k_{v1} \left[\frac{3.3b + 1}{6.6b} \right]^2$$

Where:

k_{vb} is the modulus of vertical subgrade reaction for actual foundation dimension, b (MPa/m);

k_{v1} is the modulus of vertical subgrade reaction for a 1 ft x 1 ft plate (MPa/m); and,

b is the foundation width (m).

The base for all floor slabs should consist of at least 200 mm of Ontario Provincial Standard Specification (OPSS) Granular A compacted to at least 100 percent SPMDD in suitable lift thicknesses (typically maximum 200 mm).

For the design of interior floor slabs using a spring constant, a modulus of vertical subgrade reaction, k_{v1} , of 30 MPa/m may be used for design for the design of slabs placed directly on granular fill. The design modulus of vertical subgrade reaction is derived based on the assumption that the subgrade is not disturbed during construction.

Again, the modulus of subgrade reaction is not a fundamental nor intrinsic soil property and will vary depending on the rigidity of the slab and the thickness of the granular bedding. Additional analysis and input may be required for the structural design to refine the range of k_{vb} values. Where designs are sensitive to the specific modulus value(s), a more rigorous method of analysis (i.e., settlement analysis) should be undertaken to obtain modulus value(s) that are more representative of the site conditions.

To assist in preventing potential long-term settlement of the floor slab, any soft / loose / disturbed, wet, or otherwise deleterious materials should be removed from below the floor slab. An underfloor drainage system is not considered necessary provided that the floor slab level is above the finished exterior ground surface.

A polyethylene vapour barrier / retarder is recommended below the floor slabs where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The ACI 302.1R-04 "*Guide for Concrete Floor and Slab Construction*" should be referenced for design purposes.

The floor slabs should be structurally separate from the foundation walls and columns and sawcut control joints should be provided at regular intervals and along column lines to reduce shrinkage cracking and allow for any differential settlement of the floor slabs.

If any areas of the building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

6.3.3 Foundation Walls and Isolated Piers

To avoid ad-freeze and possible jacking (heaving) of the foundations, the interior and exterior of the foundation walls should be backfilled with free draining, non-frost susceptible material that meets OPSS requirements for Granular B Type I or II. The backfill should be compacted in maximum 300 mm thick lifts to at least 95 percent SPMDD using suitable vibratory compaction equipment. Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks, or other similar surfaces), the backfill should be placed in maximum 200 mm thick lifts and compacted to 95 percent SPMDD using suitable compaction equipment.

Backfilling against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material meeting OPSS Granular B Type I or II requirements. Other measures to prevent frost jacking of foundation elements can be provided if required.

Where areas of hard surfacing (pavement etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.5 m below finished grade to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical (1H:1V), or flatter.

6.3.4 Seismic Site Class

Based on the results of the investigation, it is anticipated that the proposed structures will be founded on engineered fill or the native firm to very stiff cohesive till deposits.

The seismic design provisions of Table 4.1.8.4-A of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. Based on the subsurface conditions encountered and our experience with similar developments in the area, the foundations at the site may be designed using a Site Class D designation.

It is likely that the site class could be improved by in situ testing. If recommended by the structural engineer, in situ geophysical testing can be carried out at the site.

6.4 Temporary Excavation

Excavations for the proposed structures and underground utilities are anticipated to be through the engineered fill, and native firm to hard silty clay and stiff to hard cohesive till deposits. Due to the nature of the glacial till, which is anticipated to contain cobbles and / or boulders, excavation equipment should be chosen that can handle removal of any cobbles / boulders.

The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act (OHSA). According to the Act, the native soils can be generally classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1H:1V extending upwards from the base of the excavation. Please note that if the excavation extends below the groundwater table without adequate dewatering, the soil at the face of the excavation would be classified as Type 4. The soil type classifications indicated above are provisional and are subject to change based on field observations of the actual conditions at the time of exposure.

Excavations should be left open for as short a duration as possible and completely backfilled at the end of each working day. All excavated material should be stockpiled well away (i.e., minimum 2 m) from the sides / crest of the excavation.

Where side slopes of excavations are required to be steepened to limit the extent of the excavation, then some form of trench support system may be required. It is emphasized that 'trench box' support systems provide protection for construction personnel, but do not provide any lateral support for the adjacent excavation walls, underground services, or existing structures. Any voids between the excavation walls and the exterior of the trench box should be filled immediately to restore lateral support.

If there is insufficient space to excavate temporary open cuts, it is recommended that a shoring system consisting of braced steel sheet piles or potentially a slide rail system designed by a Professional Engineer, including assessment of the potential for basal heave be utilized. If shoring is implemented at the site, the requirements of OPSS.PROV 539 should be followed. The design of temporary works is (entirely) the responsibility of the contractor.

6.5 Lateral Earth Pressure (Temporary Shoring System)

The static active thrust (P_a) force acting (linearly) on the side of the temporary shoring should be calculated using the following formula:

$$P_a = 0.5 K_a \gamma H^2$$

Where:

- P_a : Static active thrust component (kN per linear metre of wall);
- γ : Bulk material unit weight (kN/m³);
- K_a : Coefficient of "Active" earth pressure (anticipated movement of the wall); and,
- H: Wall height (m).

The static thrust component (P_a) acts at a point located H/3 above the base of the wall. For design purposes, the soil parameters provided in Table 6.1 can be used to calculate the active thrust components acting on the wall.

Table 6.1 – Summary of Coefficient of Lateral Earth Pressures

Parameter	Engineered Fill	Cohesive Deposit ¹	Cohesive Till
Material Unit Weight, γ (kN/m ³)	21	20	21
Estimated Friction Angle (degrees)	32	-	33
Coefficient of At-Rest Earth Pressure, K_o	0.47	0.5	0.46
Coefficient of Active Earth Pressure, K_a (assuming horizontal backfill behind the structure)	0.31	1.0	0.29

Notes:

- 1) Assumes short-term / total stress conditions and that the excavation is temporary and of short duration (i.e., 4 week).

Shoring system may include braced soldier pile and lagging, braced sheet piles or an engineered slide rail system. The shoring elements should extend into the competent native soils to provide adequate wall / soil stiffness at the embedment toe. Potential lateral pressures from the groundwater table may need to be considered in the shoring system design depending on the groundwater control measures selected by the contractor.

6.6 Temporary Groundwater Control

The groundwater levels measured in the monitoring wells ranged from about 0.7 m to 8.7 m below ground surface (between Elev. 453.6 m and Elev. 461.7 m).

The cohesive till and silty clay generally have a low hydraulic conductivity and low specific yield and as such are not expected to generate large quantities of groundwater for excavations open for a relatively short period of time. The groundwater was observed to have a slow recovery with signs of gradual increase in groundwater levels between June and August 2024 which is indicative of the low hydraulic conductivity of the glacial tills and silty clay. Where excavations are carried out within these low hydraulic conductive soils, groundwater can likely be controlled by pumping from properly constructed and filtered sumps located within the excavations, assuming the excavation will be open for a short period of time.

The rate and volume required for dewatering will be dependent on the depth of the required excavations, the groundwater levels at the time of construction and the construction methods and staging chosen by the Contractor. Dewatering should be undertaken by a specialist dewatering contractor who is an MECP-licensed Water Well Contractor. The design of the dewatering array should be based on the dewatering contractor's independent assessment of site conditions, and on their experience and equipment.

Details regarding dewatering needs and anticipated volumes have been provided in the Hydrogeological assessment report provided in a separate cover.

6.7 Site Servicing

It should be noted that details of the underground utilities (i.e., watermain, storm and sanitary sewers etc.) have not been provided at the time of preparing this report. As such, we have assumed that underground infrastructure within the site will be located within 4 m from the existing site grades. Based on this, the founding soils for the proposed site services will likely consist of native firm to hard silty clay and stiff to hard cohesive till which are suitable for supporting the proposed underground service.

In general, specifications for pipe bedding, cover, and trench backfill should be in accordance with the Township of Wellington North Municipal Servicing Standards dated March 2023.

6.7.1 Pipe Bedding and Cover

The pipe bedding material should consist of well graded crushed stone meeting OPSS gradation requirements for Granular A. The minimum bedding thickness should be 200 mm for pipes within the competent native soils. The use of clear crushed stone or 'high-performance bedding' as bedding material should not be permitted.

Cover material, from pipe spring line to at least 300 mm above the top of the pipe, should consist of granular material, such as OPSS Granular B. The use of clear crushed stone or 'high-performance bedding' as the cover material should not be permitted.

The bedding and cover materials should be compacted in maximum 200 mm loose lift thickness to at least 98 percent SPMDD.

6.7.2 Trench Backfill

Most of the excavated materials will consist of native silty clay and cohesive till. The soils within the anticipated excavated depths will mostly be below or at their estimated optimum water contents for compaction. Based on this, the excavated cohesive till materials at water contents of +/- 2 percent of their optimum water contents may be re-used as trench backfill, provided they are free of topsoil, organic or other deleterious materials. Some moisture conditioning may be required prior to placement as backfill. Contractors should make appropriate effort to promote positive surficial drainage of backfilled materials in order to minimize ponding and weakening of the backfilled materials to aid in the successful preparation of the pavement structure.

The silty clay deposit is not recommended to be used as trench backfill due to difficulty in breaking it down for compaction, high water contents which are above the anticipated optimum water contents, sensitivity upon contact with moisture, and potential long-term settlements.

Alternatively, the trench may be backfilled with imported free draining, non-frost susceptible granular or sandy material. The material should meet OPSS gradation requirements for SSM or Granular B Type II. To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, driveways, etc., the backfill material should be compacted in

maximum 300 mm loose lift thickness to a minimum 95 percent SPMDD from the top of the cover material to 1.0 m below the subgrade elevation. The upper 1 m of the backfill should consist of granular material and compacted to at least 98 percent SPMDD to support the pavement structure. Oversized cobbles and boulders (i.e., greater than 150 mm in size) should be removed from the backfill.

Normal post-construction settlement of the compacted trench backfill should be expected. As such, consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction; and / or,
- Make provision to defer final paving of the surface course asphalt in the roadway for 3 months, or longer, to allow the trench backfill settlement to occur and thereby improve the final roadway appearance.

Trench plugs (i.e., clay) may be required at a limited number of locations where the trenches are along steep grades to prevent preferential water flow through the granular bedding and trench backfill. These clay plugs could be constructed using the excavated silty clay (at least 2 percent above its optimum water content) or manufactured clay plugs. The need for and frequency of trench plugs should be evaluated during the detailed design phase, generally confirm to OPSD 802.095, and should be considered at the construction limits, and equally spaced at a maximum 100 m spacing (depending on pipe length, may be less).

6.8 Stormwater Management Pond

Based on the Concept Site Plan provided, the proposed SWM pond will be located at the northwest quadrant of the site in the vicinity of Boreholes BH24-1, BH1, and BH4. As noted earlier, the sandy silt described in HLV2K's report is in our opinion a cohesive deposit and has been treated as such for the purpose of this report. It is recommended that additional investigation be carried out within the SWM pond footprint once design has been advanced.

The following comments and recommendations are general in nature, should be considered to be preliminary and are provided to assist in the preliminary design of the SWM pond. Once the pond design is more advanced, the recommendations should be revised and updated as appropriate along with additional investigatory field work carried out, if required.

The proposed SWM pond base has been assumed to be located within about 4 m below ground surface depending on the final grade.

- Based on the subsurface conditions encountered within the site, the subsurface soil conditions below the topsoil are expected to generally consist of firm to hard silty clay and firm to hard cohesive till deposits.

- The maximum groundwater level measured in the monitoring well installed in Borehole BH24-1 (S/D) was about 0.7 m below ground surface (Elev. 461.7 m) in July.
- The tills and cohesive soils are characterized by moderate to low hydraulic conductivity and low specific yield. Groundwater contributions from these units are not expected to be significant.
- The predominant soil types encountered within the site are generally considered to be of low hydraulic conductivity. Therefore, no clay liner (i.e., natural clay or geosynthetic) or ballasted geomembrane combined with perimeter and an underdrain system will be required in these soils.
- Any constructed berms around the pond should have a top width of at least 3 m to allow access by maintenance vehicles. The material used to construct the berms should be approved by the geotechnical engineer prior to placement. In this regard, the silty clay and cohesive till materials would generally be suitable for reuse provided that the compaction water content is at, or up to 3 percent below, its optimum water content. The approved material used to construct berms should be placed in maximum 200 mm thick loose lifts and uniformly compacted to at least 98 percent of the material's SPMDD.
- Further to the above, care should be taken to ensure homogeneity of the constructed berm (i.e., no erodible layers). The prepared foundation for the berm should be inspected by the geotechnical engineer prior to placement of berm fill material. A key trench, a minimum of 0.6 m deep and 2 m wide, keyed into the till deposit, should be provided along the full length of the constructed berm.
- Pond side slopes above the permanent water level in the pond should be inclined no steeper than 3H:1V; side slopes below the permanent water table should be inclined at 4H:1V or flatter.
- Cut side slopes of the pond should be inspected by the geotechnical engineer during construction. Where erodible seams (e.g., sand or silt seams) are encountered, some form of blanketing, flattening of the slope angles or the like would be required. The need for and the design of any blanketing or other remedial measures should be determined during construction by the geotechnical engineer.
- During construction, water accumulated in the pond should be removed and all soft, loosened, or disturbed soils at the base and slopes of the pond should be sub-excavated as recommended on site by GEMTEC. The native, undisturbed, competent soils should be exposed and heavily proofrolled in conjunction with an inspection by GEMTEC to confirm the base and slopes of the pond are cleaned of ponded water, loosened/softened soils, or other deleterious material. Any soft spots identified during proofrolling should be sub-excavated to expose competent soils prior to backfilling.
- The pond should be equipped with an emergency spillway or similar structure(s) designed to eliminate the possibility of over-topping of the berms.
- Where pipes enter or exit the pond, they should be provided with a concrete collar and be backfilled with a relatively impermeable material (e.g., native silty clay) to reduce preferential flow through the pipe bedding and backfill and possible loss of ground. Pipes

entering or exiting the pond should be sized and designed to allow for cleaning. The exposed end of the riser portion should be provided with a protective wire mesh or the like to prevent unauthorized access (e.g., by children).

- Regular inspections by the geotechnical engineer should be carried out during the pond construction. The final pond side slopes should be sodded or otherwise similarly treated to reduce erosion. Maintenance will be required over the first several years until the vegetative mat has taken root.

Further comments and revisions on the above recommendation and construction of the pond will be provided once the design details and drawings are provided.

6.9 Asphalt Pavement Construction

6.9.1 Pavement Structure

Based on the results of the geotechnical investigation and on review of the Township of Wellington North Municipal Servicing Standards dated March 2023, the following minimum pavement structure is suggested for internal (local) roadways at this site:

Table 6.2 – Flexible Pavement Design Recommendations

Material	Thickness and Type of Pavement Elements
Surface Course	40 mm HL 3
Base Course	50 mm HL 4
Granular Base	150 mm Granular A
Granular Subbase	450 mm Granular B, Type I or II

6.9.2 Effects of Subgrade Disturbance

If the road subgrade surface becomes disturbed or wetted due to construction operations or precipitation, or the granular pavement materials are to be used by construction traffic, the Granular B thicknesses provided above may not be adequate and it may be necessary to increase the thickness of the Granular B subbase or exclusively use Granular B, Type II subbase. The contractor should be responsible for providing suitable access for construction equipment.

The required thickness of the subbase materials will depend on a number of factors, including contractor workmanship and schedule, contractor methodology, soil types, and weather conditions, and should be assessed by geotechnical personnel at the time of construction. In our opinion, the recommended approach for subgrade preparation from a geotechnical point of view is to:

- Proofroll the subgrade conditions at the time of construction under the supervision of experienced geotechnical personnel; and,
- Adjust the thickness or type of the subbase material and include a woven geotextile separator, as required. Unit rate allowances should be made in the contract for sub-excavation and replacement with OPSS Granular B, Type II (as required).

6.9.3 Granular Material Placement

The pavement granular materials should be compacted in maximum 300 mm thick lifts to at least 98 percent SPMDD using suitable vibratory compaction equipment.

6.9.4 Asphaltic Cement

Performance graded PG 58-28 asphaltic cement is recommended for the internal road construction.

6.9.5 Transition Treatments

In areas where the new pavement structure will abut existing pavements, the depths of the granular materials should taper up or down at 5H:1V, or flatter, to match the depths of the granular material(s) exposed in the existing pavement. Any undermining or broken edges resulting from the construction activities should be removed by saw cut. All milled surfaces and butt joints should be properly tack coated prior to asphalt placement.

6.9.6 Pavement Drainage

In order to provide drainage of the granular base and subbase, the granular material should extend to ditches or drainage outlets. The bottom of the granular subbase layer should be at least 0.3 m above the bottom of the ditch or drainage outlet and should have positive drainage away from the site.

If storm sewers and catch basins are installed, it is suggested that continuous subdrains connected to catch basins be provided along the perimeter of both sides of all roadways to assist with drainage of the pavement structure. These drains should be installed at the bottom of the subbase layer.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Monitoring Well Abandonment

All monitoring wells installed as part of this investigation should be decommissioned when no longer required by a licensed Water Well Contractor in accordance with applicable legislation. A licensed Water Well Contractor should also decommission any on-site water supply wells in accordance with applicable legislation. The well abandonment could be carried out in advance of or during construction.

7.2 Management of Excess Soil

It is noted that the professional services retained for this project include only the geotechnical and hydrogeological aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination, including naturally occurring sources of contamination, are outside the terms of reference for this report. This report does not constitute a Phase II Environmental Site Assessment (ESA), nor does it constitute a contaminated material management plan.

It is recommended that soil samples be collected prior to and / or during construction to support the disposal or re-use of excess soil generated from the site.

7.3 Corrosion Considerations

The potential for the subsurface soil and groundwater conditions to corrode concrete and steel elements, or the like, should be considered in the final design. Additional sampling and / or testing may be required, or suitable protection measures (i.e., sulphate resistance concrete, sacrificial thickness, cathodic protection, etc.) should be considered by the designer.

7.4 Design Review and Construction Observation

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed site development and excavations do not materially differ from those given in the report, and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for foundations, services, pavement construction should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

Appendix B2 Draft Hydrogeological Investigation





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DRAFT

**Hydrogeological Investigation
Proposed Residential Development
665 Eliza Street
Arthur, Ontario**

GEMTEC Project: 101764.038



GEMTEC

www.gemtec.ca

Submitted to:

Tribute/Sorbara Arthur Holdings Inc.
1815 Ironstone Manor, Unit 1
Pickering, Ontario
L1W 3W9

**Hydrogeological Investigation
Proposed Residential Development
665 Eliza Street
Arthur, Ontario**

October 11, 2024
GEMTEC Project: 101764.038

GEMTEC Consulting Engineers and Scientists Limited
850 Champlain Avenue, Unit 101
Oshawa, ON, Canada
L1J 8C3

October 11, 2024

File: 101764.038 – RevA

Tribute/Sorbara Arthur Holdings Inc.
1815 Ironstone Manor, Unit 1
Pickering, Ontario
L1W 3W9

Attention: Frank Zadorozniak, C.E.T.

**Re: Hydrogeological Investigation Report
Proposed Residential Development, 665 Eliza Street, Arthur, Ontario**

Please find enclosed our hydrogeological investigation report for the proposed residential subdivision located at 665 Eliza Street, Arthur, Ontario. This report was prepared by Andy Weatherson, M.Env.Sc., P.Geo., and reviewed by Dale Edwards, C.Tech. Please provide any comments on the draft report. Finalization of the report is planned following receipt of any consolidated comments on the draft report.

DRAFT

DRAFT

Andy Weatherson, M.Env.Sc., P.Geo.
Hydrogeologist

Dale Edwards, C.Tech.
Branch Manager - Oshawa

AW/CH/DBE/sv

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1.0 INTRODUCTION

GEMTEC Consulting Engineers and Scientists Limited (GEMTEC) has been retained by Tribute/Sorbara Arthur Holdings Inc. to carry out a hydrogeological investigation in support of a Draft Plan of Subdivision application for the proposed residential subdivision located at 665 Eliza Street, Arthur, Ontario, herein referred to as the 'site'. This report should be read in conjunction with the associated geotechnical investigation report (issued under separate cover; GEMTEC, 2024).

The purpose of the hydrogeological investigation was to characterize the general subsurface and groundwater conditions at the site by means of a limited number of boreholes and monitoring wells and, based on the information obtained, to provide hydrogeological recommendations for construction dewatering permitting needs and water disposal options, which could influence both design and construction of the development.

This report is subject to the *Conditions and Limitations of This Report*, which follows the text of the report, and are considered an integral part of the report (see Appendix A).

2.0 PROJECT DESCRIPTION AND SETTING

2.1 Project Description

The site is located approximately 400 metres (m) to 1,000 m south of Line 2 and east of Eliza Street in Arthur, Ontario, with an approximate area of 38.4 hectares (ha). The land use at the site is predominantly agricultural with a rural residence (and barn) occupying a portion of the west side of the site and a watercourse (tributary to Farleys Creek) running through the western portion of the site. The approximate site location and boundaries are shown on Figure 1, Site Plan, attached.

Based on the conceptual site plan provided, it is understood that the planned residential development will consist of single and semi-detached homes, townhouses, parklands, a stormwater management (SWM) pond, and internal roadways.

At the time of preparation of this report, details of the proposed development (e.g., site grading, building structures, servicing depths, SWM pond etc.) were not provided due to the preliminary stage of the project. Therefore, assumptions regarding excavation areas and depths were made for preliminary assessment of temporary construction dewatering permitting requirements.

2.2 Existing Reports

The following report and project documents have been provided to and considered by GEMTEC in the preparation of this report:

- Geotechnical Report titled "*Preliminary Geotechnical Investigation Report, 665 Eliza Street, Arthur, Ontario*" prepared by HLV2K Engineering Limited, dated May 5, 2022.

2.3 Topography, Drainage, and Natural Heritage

According to topographic mapping provided in the on-line Ministry of Natural Resources and Forestry (MNR), *Make a Map: Natural Heritage Areas* tool (MNR, 2023), existing site grades vary by approximately 3 m from approximately Elev. 464 m above mean sea level (amsl) (on the west side of the site) to Elev. 467 m amsl (on the east side of the site). A tributary to Farleys Creek is present along the west side of the site and drains to the northwest into Farleys Creek (MNR, 2023). No wetlands are mapped on or in the vicinity of the site (see Figure 2, Topography and Natural Heritage, attached).

2.4 Physiography and Geology

The site is located within the physiographic region known as the Stratford Till Plain. At this location, undrumlined till plains are the dominant physiographic landform (Chapman and Putnam, 1984).

Published surficial geology mapping (Ontario Geological Survey, 2010) indicates that the surficial soils at the site consist of clay to silt-textured till (Halton Till). Paleozoic bedrock geology mapping (Armstrong and Dodge, 2007) indicates that the overburden is underlain by dolostone, shale, and evaporites of the Salina Formation. Bedrock was not encountered within the boreholes advanced at the site as part of this investigation; however, based on the review of the Ministry of the Environment, Conservation and Parks (MECP) Water Well Records, the depth to bedrock ranges from approximately 42 m to 48 m below ground surface (bgs).

2.5 MECP Water Well Records

A review of available MECP Water Well Records was carried out for the area within approximately 500 m of the Site (see Figure 4, MECP Well Records within 500 Metres, attached). The database indicated 18 records within this search area. Six of the records contained no information. Three are identified as monitoring wells and three wells are listed as not used. These wells are not considered further. Of the six remaining records, all six are identified as domestic or domestic/stock watering supply wells. A summary of the information provided on the records is presented in Table 2.1 below.

Table 2.1 – Well Records Review, Supply Well Summary

Well Type	Depth (m)			Overburden Source	Bedrock Source	Well Use	
	min	max	avg			DO, DO/ST	Other
Drilled	52.4	131.7	103.2	0	6	6	0
Totals	-	-	-	0	6	6	0

Notes:

- min = minimum
- max = maximum
- avg = arithmetic mean
- DO = Domestic, DO/ST = Domestic and stock watering.

One well record (#7425145) is reportedly located on the site (location not confirmed by GEMTEC). This well record did not contain any information other than it was installed in 2022. It is possible this well services the existing residential dwelling on the site.

The subsurface conditions reported in the remaining water well records within the search area were in general agreement with the published geological mapping. The subsurface conditions reported were predominantly comprised of deposits of clay and/or inferred glacial till with some silt and/or sand. The recorded depth to bedrock (limestone) ranged from 42.1 m to 47.9 m below ground surface (bgs), with an average depth of 44.8 m bgs (n = 6). The reported water source used by the water supply wells was limestone bedrock (all six wells). The dates of construction for the supply wells ranged from 1978 to 2022.

Typically, shallow dug and bored wells are the most susceptible to water level fluctuations and surficial sources of contamination. There are no shallow dug or bored supply wells within 500 m of the site, and the bedrock aquifer is hydraulically isolated from the ground surface by a vertically extensive aquitard composed of low permeability clay/till deposits.

2.6 Source Water Protection

The MECP Source Protection Information Atlas (MECP, 2023a) was reviewed to assess the presence of source water protection areas on the site including Wellhead Protection Areas (WHPA) associated with municipal groundwater supplies, Intake Protection Zones (IPZ) associated with municipal surface water supplies, Significant Groundwater Recharge Areas (SGRA) and Highly Vulnerable Aquifers (HVA).

The nearest WHPA is located about 550 m west of the site in Arthur, Ontario. Part of the site is mapped as an IPZ-3 associated with the tributary to Farleys Creek. There are no HVAs or SGRAs mapped within 500 m of the site.

2.7 Registered Water Takings

The Environmental Approvals and Registrations database (MECP, 2023b) was reviewed for nearby registered water takings. No active Permit to Take Water (PTTW) or Environmental Activity Sector Registry (EASR) entries for water takings were identified within 500 m of the site at the time of preparation of this report.

3.0 PREVIOUS INVESTIGATION

As noted above, a geotechnical investigation (HLV2K, 2022) was previously carried out at the site by others in 2022. As part of this investigation, six boreholes were advanced across the site to depths between about 5.8 m and 6.5 m bgs.

The described subsurface conditions consisted of topsoil, underlain by sandy silt to sandy clayey silt.

Three of the boreholes (Boreholes BH1 to BH3) were instrumented with shallow monitoring wells. Groundwater levels were manually monitored in the monitoring wells on April 12, 2022. The depth to groundwater at the monitoring wells ranged from about 1.1 m to 1.8 m bgs; approximately Elev. 462.4 m to Elev. 465.0 m amsl.

The approximate borehole locations from the previous geotechnical investigation are shown in Figure 1, attached.

The monitoring wells in Boreholes BH2 and BH3 could not be located by GEMTEC in 2024 and were assumed to be destroyed/decommissioned sometime after the HLV2K site visit on April 12, 2022.

4.0 CURRENT SITE INVESTIGATION METHODOLOGY

4.1 Geotechnical Investigation

The geotechnical investigation (GEMTEC, 2024) was carried out between June 25 and 27, 2024. Nine boreholes (Boreholes BH24-1 to BH24-9) were advanced between approximately 6.6 m and 9.6 m bgs (from Elev. 452.7 m to Elev. 458.8 m amsl).

Two additional boreholes (Boreholes BH24-10 and BH24-11) were drilled on July 25, 2024 in the general areas of HLV2K Monitoring Wells BH2 and BH3.

The Record of Borehole Sheets are provided in Appendix B; refer to the geotechnical investigation (GEMTEC, 2024) for details on the borehole drilling program.

4.2 Site Instrumentation

Monitoring wells were installed at three borehole locations (Boreholes BH24-1, BH24-6 and BH24-8). A bi-level monitoring well (i.e., shallow (S) and deep (D) monitoring well pairs installed in separate and adjacent boreholes) was installed at Borehole BH24-1 for a total of four monitoring wells.

Two additional monitoring wells were installed in Boreholes BH24-10 and BH24-11 to replace Boreholes BH2 and BH3.

The monitoring wells were constructed using nominal 50 mm diameter, Schedule 40 polyvinyl chloride (PVC) pipe with a No. 10 machine slotted screen (0.01-inch slot). The annular space between the monitoring well screen and surrounding soils was backfilled with a silica sand filter pack to a maximum of 0.3 m above the top of the screen, and the remainder of the annular space was sealed with bentonite. All monitoring wells were completed with above-ground protective steel casings. The monitoring well installation details are tabulated in Table C-1, and the measured groundwater depths and elevations are tabulated in Table C-2, Appendix C.

Following installation, the monitoring wells were developed to remove drilling fluids, solids or other particles that may have been introduced during drilling / installation. All monitoring wells were purged using dedicated 16 mm inside diameter low density polyethylene (LDPE) tubing and a D-25 Waterra™ foot valve. The monitoring wells were developed by removing three casing volumes or until dry.

In addition, a shallow piezometer (PZ) and staff gauge (SG) pair (PZ24-2/SG24-2) was installed at the site on July 9, 2024, in the on-site tributary to evaluate the vertical gradient at the south end of the tributary. The shallow piezometer was installed to an approximate depth of 0.9 m bgs.

J.D. Barnes Ltd. surveyed the GEMTEC monitoring wells and PZ/SG pair on September 4 and 20, 2024. The ground surface and top of pipe / post elevations were collected during the surveying. The elevations are geodetic and derived from GNSS observations using Natural Resources Canada's Geoid Model HTv2.0 (CGVD-1928:1978). The coordinates at the borehole locations were referenced to the Universal Transverse Mercator (UTM) Zone 17, NAD 1983.

4.3 Hydraulic Response Testing

In situ hydraulic response testing was carried out in four monitoring wells (Monitoring Wells BH1, BH24-1S, BH24-1D, and BH24-8) to estimate the bulk horizontal hydraulic conductivity (K_b) of the overburden materials adjacent to the screened intervals. The testing consisted of creating an instantaneous change through rapid well purging of the well by removing a known volume of water, followed by recording the time taken for the water level to return to static conditions (i.e., rising head test).

The data was analyzed using the Bouwer and Rice (1976) solution. A summary of the test results is provided in Table C-3, Appendix C. A sheet summarizing the test data, analysis interval, input parameters and estimated bulk hydraulic conductivity for each test is provided in Appendix D.

4.4 Groundwater and Surface Water Sampling

To evaluate disposal options for pumped groundwater from potential future construction dewatering activities, a groundwater sample was collected from the monitoring well installed in Borehole BH24-1S on July 11, 2024. In addition, a surface water grab sample (SW) was collected from the tributary to Farleys Creek to assess background surface water quality conditions in the on-site watercourse. The locations of the monitoring well and surface water sampling station are shown on Figure 1, attached.

Prior to collecting the groundwater sample, the monitoring well was purged in accordance with industry standards to obtain a representative sample. Following purging, the groundwater was sampled with the use of a dedicated bailer and poured directly into laboratory-supplied sample bottles. The groundwater was not field filtered to allow for comparison of the analytical results to the Wellington North Sewer Use By-Law # 095-16, Schedule "B" Restricted Wastes Sanitary and Combined Sewer Discharges (Sanitary Limits) and Section 3 – Storm Sewer Requirements

(Storm Limits). In addition and for discussion purposes, the results were compared to the *Water Management Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of the Environment and Energy* (now the MECP), *Table 2 – Table of PWQOs and Interim PWQOs* (PWQO) (reprinted February 1999).

The samples were packed into coolers with ice for transit to the analytical laboratory. The samples were taken to Bureau Veritas Laboratories (BV) in Mississauga, Ontario on the day of sampling.

5.0 SUBSURFACE CONDITIONS

As previously indicated, the soil and groundwater conditions identified in the boreholes are shown on the Record of Borehole Sheets in Appendix B.

The Record of Boreholes indicate the subsurface conditions at the specific borehole locations only. Boundaries between the different soils on the Records are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at locations other than the boreholes may vary from the conditions encountered in the boreholes, both laterally and with depth. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

Generally, the subsurface conditions encountered over the site consist of the following:

- Surficial topsoil ranging in thickness from about 0.08 m to 0.13 m. The surficial topsoil was underlain by;
- A silty clay deposit in Boreholes BH24-1(S/D), BH24-2 to BH24-5, and BH24-7 to BH24-11 ranging in thickness from about 0.6 m to 6.4 m, underlain by;
- A cohesive silty clay till comprised silty clay to clayey silt (cohesive) was encountered in all boreholes underlying either the topsoil or silty clay deposit. The till extended to the termination depths of the boreholes, between about 6.2 m and 9.6 m bgs (El. 452.7 m to 458.8 m amsl);
- A non-cohesive silt and sand deposit (0.5 m thick) was encountered in Borehole BH24-7 interlayered with the cohesive till deposit; and,
- A non-cohesive deposit of silt (1.2 m thick) was encountered in Borehole BH24-8 underlying the silty clay deposit and overlying the silty clay till deposit.

5.1 Groundwater and Surface Water Levels

Groundwater and surface water levels were measured in all monitoring wells and the piezometer / surface gauge pair (installed by HLV2K and GEMTEC) on four events: July 9, July 18, August 2, and August 16, 2024. The groundwater levels at the monitoring wells ranged from 1.2 m bgs (Borehole BH24-1S) to 3.1 m bgs (Borehole BH24-1D) and from Elev. 459.2 m (Borehole BH24-1D) to Elev. 463.4 m amsl (Borehole BH24-11) during the last event on August 16, 2024. It is interpreted that some of the groundwater levels may not have stabilized during the monitoring period as a result of slow water level recovery in the cohesive glacial till. Active flow was observed at SG23-1 in the tributary to Farleys Creek during each monitoring event, with an approximate surface water elevation of Elev. 461.8 m amsl on August 16, 2024. The groundwater and surface water measurements for all monitoring locations and monitoring events completed by HLV2K and GEMTEC are provided in Table D-2, Appendix D.

It is important to note the groundwater levels represent small amounts of water trapped within the low-permeability till soils beneath the property and are not indicative of an aquifer that would be expected to yield significant volumes of groundwater.

The groundwater and surface water conditions described in this report refer only to those measured at the place and time of observation. Seasonal and annual fluctuations should be anticipated.

The groundwater elevation data on August 16, 2024, is presented on Figure 5, Groundwater Flow (August 16, 2024), attached. Groundwater flow was inferred to be southwest towards the tributary to Farleys Creek, generally following surface topography.

The vertical hydraulic gradient was assessed from the groundwater levels measured on August 16, 2024, at the location of the bi-level monitoring well pair (i.e., Borehole BH24-1S/D). The approximate vertical hydraulic gradient for the bi-level monitoring wells was -0.691 m/m (downward).

The vertical gradient at PZ23-1/SG23-1 appeared to be downward (indicating recharging conditions) on August 16, 2024.

5.2 Hydraulic Response Test Results

The data and results of the hydraulic response testing carried out in the monitoring wells are presented in Table C-3, Appendix C. Borehole BH23-1D was interpreted to be partially recovered prior to the hydraulic response test and is not discussed further in this report. A summary of the test results is provided in Table 5.1 below.

Table 5.1 – Summary Hydraulic Conductivity Estimates

Monitoring Well ID	Unit	No. of Tests	Hydraulic Conductivity K_b [m/s]
BH24-1S	(CL) Silty Clay	1	6×10^{-8}
BH24-8	(CL) Silty Clay Till	1	5×10^{-9}
BH1	Sandy Silt	1	5×10^{-9}

Notes: 1. K_b = bulk hydraulic conductivity; m/s = metres per second

The estimated hydraulic conductivity of the silty clay deposit at the tested location (Borehole BH24-1S) was approximately 6×10^{-8} m/s, which is slightly above the literature range for clay of 10^{-11} m/s to 10^{-8} m/s (Fetter, 1994).

The estimated hydraulic conductivity of the glacial till deposit at the tested location (Borehole BH24-8) was approximately 5×10^{-9} m/s, which is slightly below the literature range for till of 10^{-8} m/s to 10^{-6} m/s (Fetter, 1994).

The estimated hydraulic conductivity of the sandy silt deposit at the tested location (Borehole BH1) was approximately 5×10^{-9} m/s, which is slightly below the literature range for sandy silt of 10^{-8} m/s to 10^{-6} m/s (Fetter, 1994). However, the sandy silt deposit described in the HLV2K (2022) report may be a cohesive deposit as discussed in the GEMTEC (2024) geotechnical investigation report.

5.3 Groundwater and Surface Water Quality Results

A summary of the analytical results for the groundwater sample from Borehole BH24-1S as well as the surface water sample from the tributary to Farleys Creek at Station SW (SG24-1 location) collected July 11, 2024, with comparison to the Wellington North Sanitary and Storm Limits and/or PWQO, is presented in Table E-1 in Appendix E. The Laboratory Certificate of Analysis is also provided in Appendix E.

There were no exceedances of the Sanitary and Storm Limits in the groundwater sample. For discussion purposes, parameters with concentrations exceeding the PWQO are summarized in Table 5.2 below.

Table 5.2 – Summary of Water Quality Exceedances

Parameter	Water Quality Result	Sanitary Limits	Storm Limits	PWQO
Total Cobalt	0.0014 mg/L (BH24-1S)	5 mg/L	-	0.0009 mg/L
Total Copper	0.0057 mg/L (SW)	2 mg/L	0.01 mg/L	0.005 mg/L

Parameter	Water Quality Result	Sanitary Limits	Storm Limits	PWQO
Phenols-4AAP	0.0027 mg/L (SW)	0.1 mg/L	-	0.001 mg/L
Phosphorus	0.036 mg/L (BH24-1S) 0.82 mg/L (SW)	10 mg/L	0.4 mg/L	0.03 mg/L
Toluene	0.0029 mg/L (SW)	0.02 mg/L	-	0.0008 mg/L
Total Oil & Grease, Mineral/Synthetic	0.7 mg/L (BH24-1S) 0.6 mg/L (SW)	15 mg/L	-	0.5 mg/L

Notes: 1. = No Limit or PWQO for the accompanying parameter.

The concentrations of total cobalt, phosphorus, and total mineral/synthetic oil and grease in groundwater (Borehole BH24-1S) exceeded the PWQO. Total copper, phenols-4AAP, phosphorus, toluene, and total mineral/synthetic oil and grease in surface water (SW) also exceeded the PWQO.

It is expected that the elevated cobalt concentration is related to the dissolution of suspended clay minerals during sample acidification. The measured concentrations of toluene and oil and grease appear anomalous as the property is an agricultural field.

It is important to consider that the 4AAP laboratory analysis for Phenols detects a wide variety of naturally occurring organic substances, along with the chemical Phenol (C₆H₅OH). Measured exceedances of “Phenols” may not be indicative of a contamination issue, but rather groundwater that is influenced by natural environmental factors.

Treatment of pumped water from the excavations for sediment removal to meet the Storm Limit for TSS of 15 mg/L at the point(s) of discharge should be anticipated. Treatment measures may include sediment or weir tanks, filter bags or cannisters, and the like. Additional treatment may be required if discharge is to the natural environment.

6.0 PRELIMINARY ASSESSMENT OF WATER TAKING REQUIREMENTS

6.1 Temporary Construction Dewatering Permitting

Construction of the proposed development is anticipated to result in excavations below the water levels measured in the low-permeability till soils underlying the subject property. Excavations of variable dimensions and in variable soil conditions, including excavations for linear servicing and a SWM pond, will likely require some form of positive groundwater control.

As discussed in Section 2.1, details of the proposed development (e.g., site grading, building structures, servicing depths, SWM pond etc.) were not provided. As such, it has been preliminarily assumed that the proposed underground servicing excavation depths will be up to approximately

3 m bgs and the SWM pond will be excavated up to approximately 4 m bgs. A typical length of open servicing trench using a daily cut and cover method, and assumptions of seasonal high groundwater levels and soil conditions were used to assess temporary dewatering rates. The daily water taking rates presented in this section are conservative for permitting purposes; however, it is important to note detailed construction dewatering calculations to evaluate dewatering requirements must be prepared once detailed site information becomes available. Assessment of dewatering requirements, discharge treatment requirements, and discharge disposal options can only be provided once detailed site information becomes available.

The site is generally underlain by relatively low permeability cohesive glacial till, although a localized deposit of silty sand was also encountered (Borehole BH24-7). The steady state groundwater inflow rates for a typical length of linear servicing trench (i.e. 30 m) and for the SWM pond excavation are not likely to exceed 50,000 L/day. Accounting for higher dewatering rates when groundwater is first removed from storage, groundwater inflow rates may be greater than 400,000 L/day, and then decline toward the steady state rate. Further evaluation of dewatering requirements based on site design details will be necessary to accurately evaluate whether an EASR will suffice or whether a PTTW will be necessary.

If combined dewatering rates for the entire site exceed 400,000 L/day, this would result in the need to carry out a detailed investigation and prepare a Hydrogeological Investigation Report supporting an application for a Category 3 Permit to Take Water (PTTW) from the MECP. This would also require the preparation of a discharge plan. The report must generally include an assessment of potential impacts from construction dewatering on existing groundwater, surface water and natural heritage resources as well as potential geotechnical impacts to existing structures from dewatering-induced soil settlement. The discharge plan for the discharge of groundwater pumped during construction should consider source water protection areas, as well as potential impacts to existing surface water and natural heritage resources. A monitoring, mitigation and contingency program should be included in the reporting. It is noted that the MECP review of PTTW applications typically takes at least three months, which should be factored into the construction schedule. These findings should be re-evaluated as site designs progress, the final excavation dimensions are available, and construction methods and schedules are developed.

7.0 CLOSURE

We trust that this report meets your immediate requirements. If conditions that differ from those assumed in this geotechnical and hydrogeological investigation report are encountered during construction, GEMTEC should be given the opportunity to review the recommendations presented herein.

If you have any questions or require additional information, please contact the undersigned.

Regards,

GEMTEC Consulting Engineers and Scientists Limited

DRAFT

Andy Weatherson, M.Env.Sc., P.Geo.
Hydrogeologist

DRAFT

Dale Edwards, C.Tech.
Branch Manager – Oshawa

8.0 REFERENCES

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ATTACHMENTS

Figures

Figure 1 – Site Plan

Figure 2 – Topography and Natural Heritage

Figure 3 – Surficial Geology

Figure 4 – MECP Water Well Records Within 500 m

Figure 5 – Groundwater Flow (August 16, 2024)

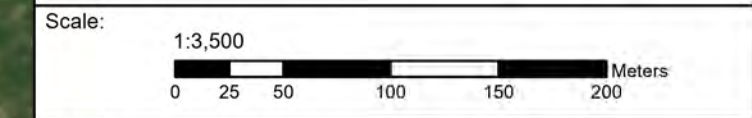


Legend

BH #	BOREHOLE ID		
	BOREHOLE LOCATION (GEMTEC, CURRENT INVESTIGATION)		MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
	MONITORING WELL LOCATION (GEMTEC, CURRENT INVESTIGATION)		DESTROYED MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
	WELL NEST LOCATION (GEMTEC, CURRENT INVESTIGATION)		PIEZOMETER/ STAFF GAUGE (GEMTEC, CURRENT INVESTIGATION)
	BOREHOLE LOCATION (HLV2K, 2022 INVESTIGATION)		WATERCOURSE
			SITE BOUNDARY
			WEST AREA

NOTES:

- All locations approximate
- Coordinate system: NAD 1983 UTM Zone 17N
- Geographic dataset source: Ontario GeoHub.
- Contains information licensed under the Open Government Licence – Ontario.
- Service Layer Credits: World Imagery: Maxar, Microsoft World Street Map: Province of Ontario, Esri Canada, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, USDA, NRCan, Parks Canada



Drawing: **SITE PLAN**

Client: **TRIBUTE/SORBARA ARTHUR HOLDINGS INC.**

Project: **HYDROGEOLOGICAL INVESTIGATION
PROPOSED ARTHUR DEVELOPMENT (EAST)
665 ELIZA STREET, ARTHUR, ONTARIO**

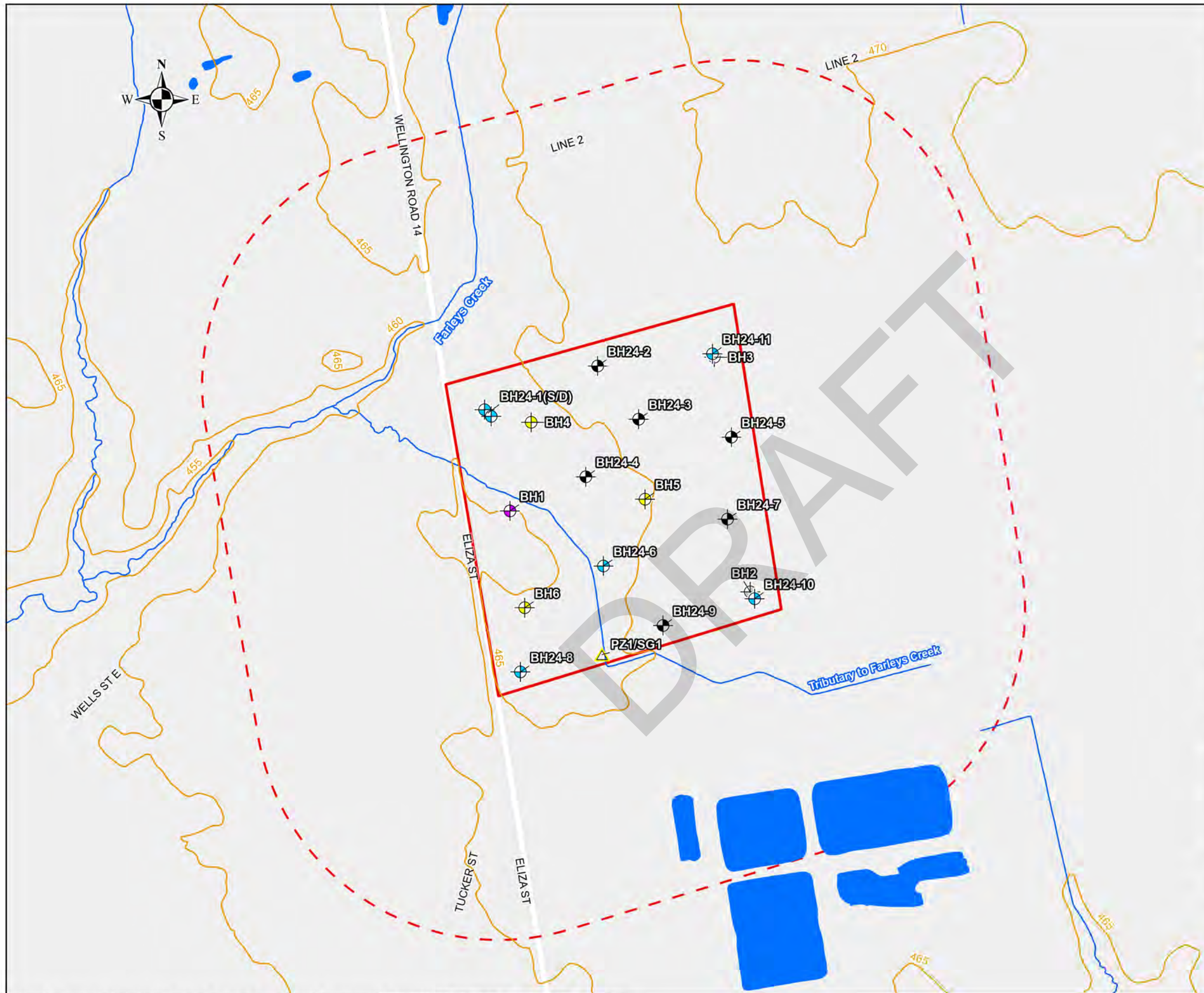
Drwn By: **K.C.** Chkd By: **A.W.**

Project No. **101764.038** Revision No. **0**

Date: **SEPTEMBER 2024** **FIGURE 1**

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

6695 Millcreek DR #7,
Mississauga, ON L5N 5M4
T: (416) 347-7427
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graeme.skinner@gemtec.ca

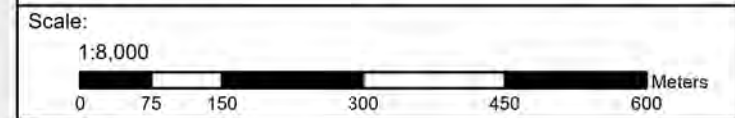


Legend

- BH # BOREHOLE ID
- BOREHOLE LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - MONITORING WELL LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - WELL NEST LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - BOREHOLE LOCATION (HLV2K, 2022 INVESTIGATION)
 - MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
 - DESTROYED MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
 - PIEZOMETER/ STAFF GAUGE (GEMTEC, CURRENT INVESTIGATION)
 - ELEVATION CONTOURS (m amsl)
 - WATERCOURSE
 - WATERBODY
 - 500 METRE RADIUS FROM SITE
 - SITE BOUNDARY

NOTES:

1. All locations approximate
2. Coordinate system: NAD 1983 UTM Zone 17N
3. Geographic dataset source: Ontario GeoHub,
4. Contains information licensed under the Open Government Licence – Ontario.
5. m amsl = meters above mean sea level.
6. Service Layer Credits: Light Grey Canvas Background: Esri Community Maps Contributors, Province of Ontario, Esri Canada, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, NRCan, Parks Canada



Drawing
TOPOGRAPHY AND NATURAL HERITAGE

Client:
TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

Project
**HYDROGEOLOGICAL INVESTIGATION
PROPOSED ARTHUR DEVELOPMENT (EAST)
665 ELIZA STREET, ARTHUR, ONTARIO**

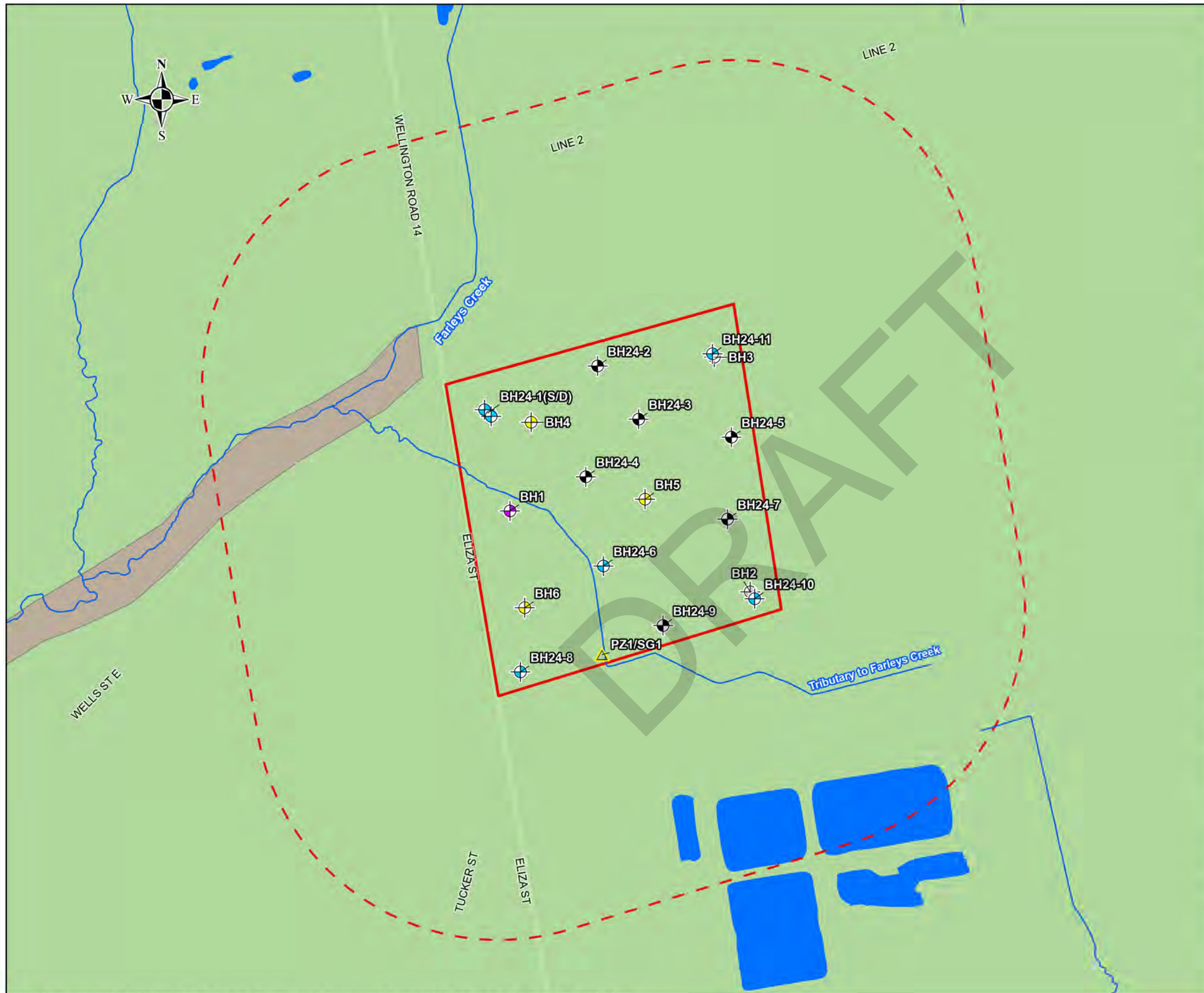
Drwn By: **K.C.** Chkd By: **A.W.**

Project No. **101764.038** Revision No. **0**

Date **SEPTEMBER 2024** **FIGURE 2**

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graeme.skinner@gemtec.ca



Legend

- BH # BOREHOLE ID**
- BOREHOLE LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - MONITORING WELL LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - WELL NEST LOCATION (GEMTEC, CURRENT INVESTIGATION)
 - BOREHOLE LOCATION (HLV2K, 2022 INVESTIGATION)
 - MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
 - DESTROYED MONITORING WELL LOCATION (HLV2K, 2022 INVESTIGATION)
 - PIEZOMETER/ STAFF GAUGE (GEMTEC, CURRENT INVESTIGATION)
 - WATERCOURSE
 - WATERBODY
 - 500 METRE RADIUS FROM SITE
 - SITE BOUNDARY

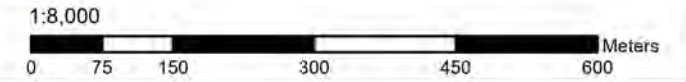
SURFICIAL GEOLOGY

- MODERN ALLUVIAL DEPOSITS (CLAY, SILT, SAND, GRAVEL, MAY CONTAIN ORGANIC DEPOSITS)
- HALTON TILL (CLAY TO SILT-TEXTURED TILL)

NOTES:

1. All locations approximate
2. Coordinate system: NAD 1983 UTM Zone 17N
3. Geographic dataset source: Ontario GeoHub.
4. Contains information licensed under the Open Government Licence – Ontario.
5. Service Layer Credits: Light Grey Canvas Background: Esri Community Maps Contributors, Province of Ontario, Esri Canada, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, NRCan, Parks Canada

Scale:



Drawing **SURFICIAL GEOLOGY**

Client: **TRIBUTE/SORBARA ARTHUR HOLDINGS INC.**

Project **HYDROGEOLOGICAL INVESTIGATION
PROPOSED ARTHUR DEVELOPMENT (EAST)
665 ELIZA STREET, ARTHUR, ONTARIO**

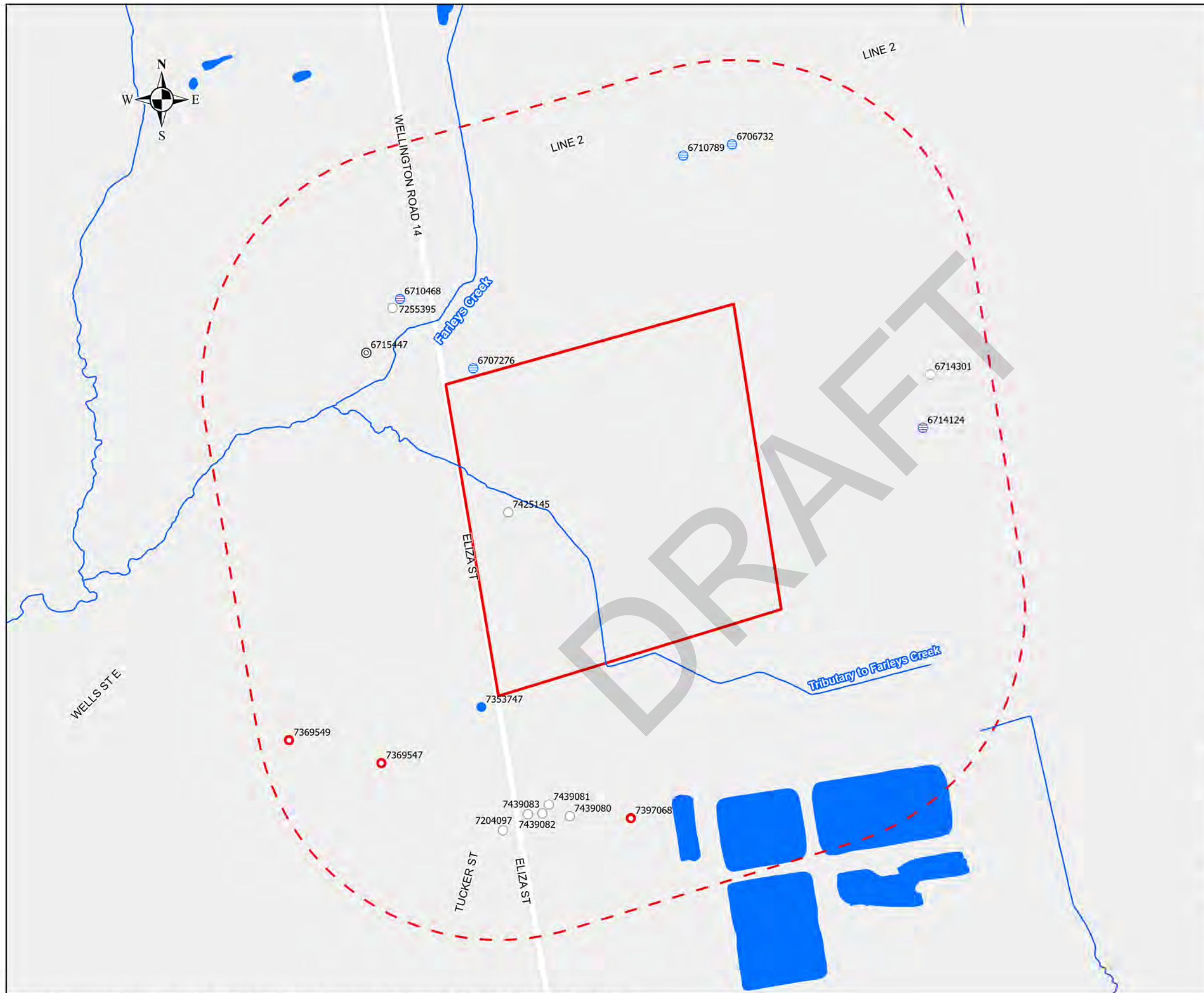
Drwn By: **K.C.** Chkd By: **A.W.**

Project No. **101764.038** Revision No. **0**

Date **SEPTEMBER 2024** **FIGURE 3**

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Legend

- WELL # WELL ID
- WATERCOURSE
 - WATERBODY
 - - - 500 METRE RADIUS FROM SITE
 - ▭ SITE BOUNDARY

WELL TYPE

- ⊙ SHALLOW (<10 M) DUG/BORED
- SHALLOW (<10M) DRILLED OVERBURDEN
- DEEP (>10M) DRILLED OVERBURDEN
- ⊕ DRILLED BEDROCK
- NO INFORMATION/ALTERATION

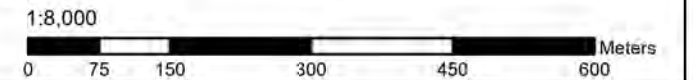
WATER USE

- DOMESTIC SUPPLY
- LIVESTOCK AND DOMESTIC SUPPLY
- MONITORING AND TEST HOLE
- ABANDONED
- OTHER (NOT USED, UNKNOWN USE)

NOTES:

1. All locations approximate
2. Coordinate system: NAD 1983 UTM Zone 17N
3. Geographic dataset source: Ontario GeoHub.
4. Contains information licensed under the Open Government Licence – Ontario.
5. Service Layer Credits: Light Grey Canvas Background: Esri Community Maps Contributors, Province of Ontario, Esri Canada, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, NRCan, Parks Canada
World Street Map: Esri, FAO, NOAA, USGS

Scale:



Drawing
MECP WATER WELL RECORDS WITHIN 500 M

Client:
TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

Project
HYDROGEOLOGICAL INVESTIGATION
PROPOSED ARTHUR DEVELOPMENT (EAST)
665 ELIZA STREET, ARTHUR, ONTARIO

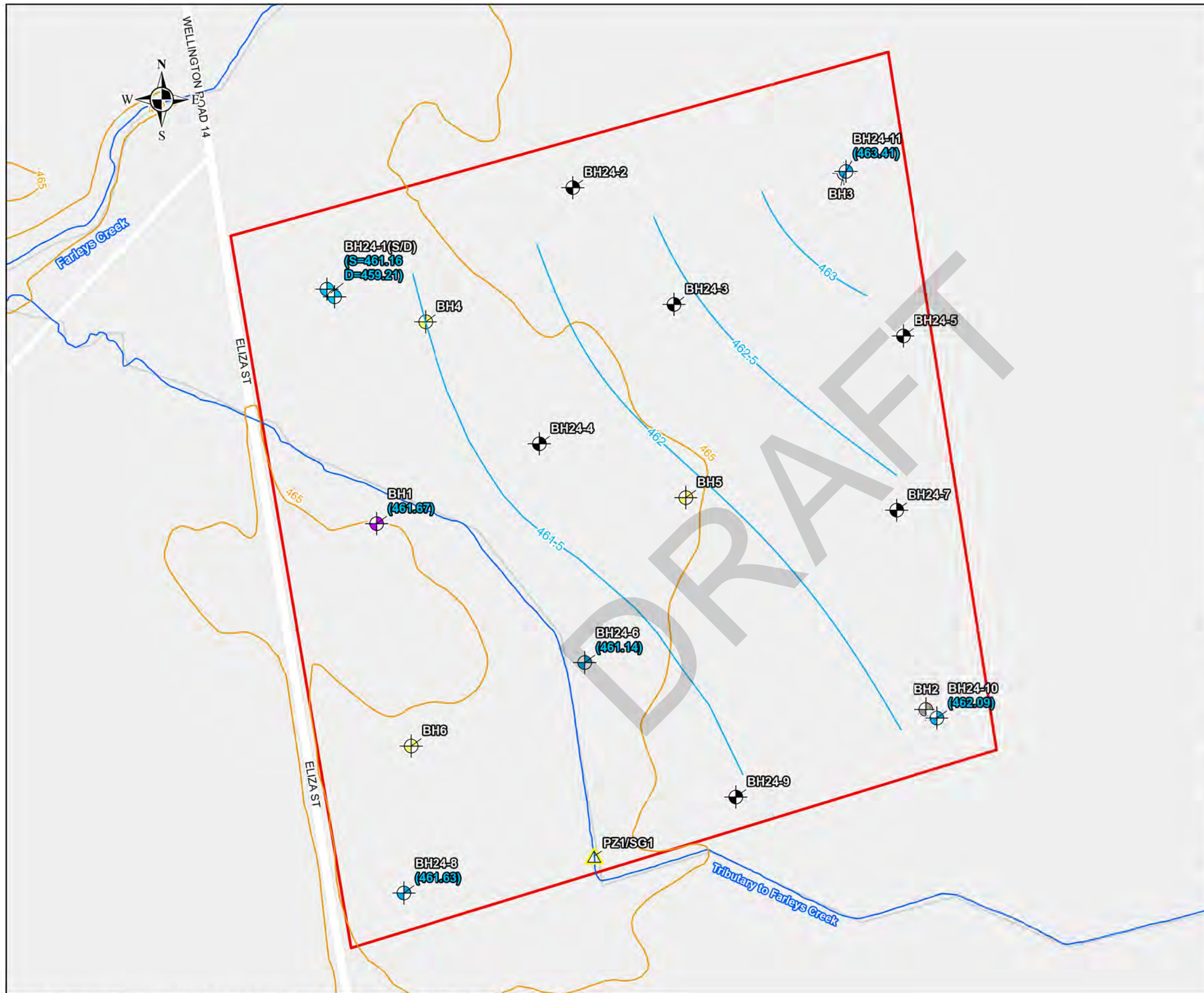
Drwn By: K.C. Chkd By: A.W.

Project No. 101764.038 Revision No. 0

Date SEPTEMBER 2024 **FIGURE 4**

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AND SCIENTISTS

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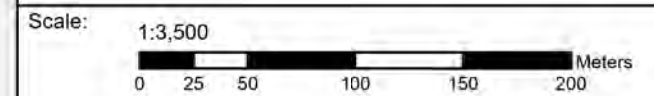


Legend

- BH # BOREHOLE ID
- 460.95** GROUNDWATER ELEVATIONS, (m amsl)
(AUGUST 16, 2024)
- BOREHOLE LOCATION
(GEMTEC, CURRENT INVESTIGATION)
- MONITORING WELL LOCATION
(GEMTEC, CURRENT INVESTIGATION)
- WELL NEST LOCATION
(GEMTEC, CURRENT INVESTIGATION)
- BOREHOLE LOCATION
(HLV2K, 2022 INVESTIGATION)
- MONITORING WELL LOCATION
(HLV2K, 2022 INVESTIGATION)
- DESTROYED MONITORING WELL LOCATION
(HLV2K, 2022 INVESTIGATION)
- PIEZOMETER/ STAFF GAUGE
(GEMTEC, CURRENT INVESTIGATION)
- GROUNDWATER CONTOUR (m amsl)
- ELEVATION CONTOURS (m amsl)
- WATERCOURSE
- WATERBODY
- SITE BOUNDARY

NOTES:

1. All locations approximate
2. Coordinate system: NAD 1983 UTM Zone 17N
3. Geographic dataset source: Ontario GeoHub.
4. Contains information licensed under the Open Government Licence – Ontario.
5. m amsl=metres above mean sea level
5. Service Layer Credits: Light Grey Canvas Background: Esri Community Maps Contributors, Province of Ontario, Esri Canada, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, NRCan, Parks Canada



Drawing		GROUNDWATER FLOW (AUGUST 16, 2024)	
Client:		TRIBUTE/SORBARA ARTHUR HOLDINGS INC.	
Project		HYDROGEOLOGICAL INVESTIGATION PROPOSED ARTHUR DEVELOPMENT (EAST) 665 ELIZA STREET, ARTHUR, ONTARIO	
Drwn By:	K.C.	Chkd By:	A.W.
Project No.	101764.038	Revision No.	0
Date	SEPTEMBER 2024	FIGURE 5	
		6695 Millcreek DR #7, Mississauga, ON L5N 5M4 T: (416) 347-7427 www.gemtec.ca graeme.skinner@gemtec.ca	

DRAFT

APPENDIX C

Tables

Table C-1: Monitoring Well Construction Information - Proposed Arthur Development (East), 665 Eliza Street, Arthur, Ontario

Well Name	UTM Coordinates (approximate)		Installation Date	Ground Surface Elevation	Top of Casing Elevation	Measured Stick up	Top of Screen	Bottom of Screen	Top of Screen	Bottom of Screen	Screened Lithology
	Easting	Northing		(m amsl)	(m amsl)	(m)	(m bgs)	(m bgs)	(m amsl)	(m amsl)	
BH24-1S	537414	4855122	25-Jun-24	462.37	463.44	1.07	4.88	6.40	457.49	455.97	(CL) Silty Clay
BH24-1D	537414	4855122	25-Jun-24	462.29	463.26	0.97	7.62	9.14	454.67	453.15	(CL) Silty Clay Till
BH24-6	537651	4854809	26-Jun-24	462.47	463.36	0.89	4.57	6.10	457.90	456.37	(CL) Silty Clay Till
BH24-8	537481	4854592	26-Jun-24	463.20	464.02	0.82	6.10	7.62	457.10	455.58	(CL) Silty Clay Till
BH24-10	537961	4854742	25-Jul-24	465.04	465.84	0.80	4.57	6.10	460.47	458.94	(CL) Silty Clay Till
BH24-11	537874	4855244	25-Jul-24	464.99	465.85	0.86	4.57	6.10	460.42	459.75	(CL) Silty Clay Till

Notes:

- m - metre
- m amsl - metres above mean sea level
- m bgs - metres below ground surface
- UTM - Universal Transverse Mercator, Zone 17T

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Table C-2: Groundwater and Surface Water Depths and Elevations - Proposed Arthur Development (East), 665 Eliza Street, Arthur, Ontario

Well Name	Ground Surface Elevation (m amsl)	Top of Casing Elevation (m amsl)	Measured Stick up (m)	Top of Screen (m amsl)	Bottom of Screen (m amsl)	Screened Lithology	April 12, 2022		July 9-11, 2024		July 18, 2024		August 2, 2024		August 16, 2024	
							WL Below Ground (m bgs)	Approximate WL Elev. (m amsl)	WL Below Ground (m bgs)	Approximate WL Elev. (m amsl)	WL Below Ground (m bgs)	Approximate WL Elev. (m amsl)	WL Below Ground (m bgs)	Approximate WL Elev. (m amsl)	WL Below Ground (m bgs)	Approximate WL Elev. (m amsl)
							GEMTEC Monitoring Wells, Piezometers, and Staff Gauges									
BH24-1S	462.37	463.44	1.07	457.49	455.97	(CL) Silty Clay	N/I	N/I	1.12	461.25	0.71	461.66	1.00	461.37	1.21	461.16
BH24-1D	462.29	463.26	0.97	454.67	453.15	(CL) Silty Clay Till	N/I	N/I	8.70	453.59	5.23	457.06	4.44	457.85	3.08	459.21
BH24-6	462.47	463.36	0.89	457.90	456.37	(CL) Silty Clay Till	N/I	N/I	3.51	458.96	1.24	461.23	1.31	461.16	1.33	461.14
BH24-8	463.20	464.02	0.82	457.10	455.58	(CL) Silty Clay Till	N/I	N/I	1.60	461.60	1.30	461.90	1.37	461.83	1.57	461.63
BH24-10	465.04	465.84	0.74	460.47	458.94	(CL) Silty Clay Till	N/I	N/I	N/I	N/I	N/I	N/I	4.20	460.84	2.95	462.09
BH24-11	464.99	465.85	0.91	460.42	459.75	(CL) Silty Clay Till	N/I	N/I	N/I	N/I	N/I	N/I	1.30	463.69	1.58	463.41
PZ24-1	461.83	463.48	1.65	459.58	459.28	N/A	N/I	N/I	Dry	<459.28	0.45	461.38	0.37	461.46	0.34	461.49
SG24-1	461.73	463.22	1.49	N/A	N/A	N/A	N/I	N/I	-0.07	461.80	-0.18	461.91	-0.10	461.83	-0.04	461.77
HLV2K Monitoring Wells																
BH1	463.03	463.89	0.86	204.33	202.81	Sandy Silt	1.82	461.21	2.45	460.58	1.28	461.75	2.11	460.92	1.36	461.67
BH2*	N/A	N/A	N/A	N/A	N/A	Sandy Clayey Silt Till	1.12	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BH3*	N/A	N/A	N/A	N/A	N/A	Sandy Clayey Silt Till	1.28	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

- Not Measured
 - Negative values indicate that water levels are above the ground surface.
 - Elev. - Elevation
 - m - metre
 - m amsl - metres above mean sea level
 - m bgs - metres below ground surface
 - m toc - metres below top of casing
 - WL - Water Level
 - N/A - Not available/applicable
 - N/I - Not installed
 - * - BH2 and BH3 were assumed to have been destroyed after the April 12, 2022 site visit by HLV2K.
- Water levels measured on April 12, 2022 were obtained from the HLV2K report titled "Preliminary Geotechnical Investigation"

DRAFT

Table C-3: Summary of Hydraulic Conductivity Values - Single Well Response Tests - Proposed Arthur Development (East), 665 Eliza Street, Arthur, Ontario

Well Name	Date of Test	Ground Surface Elevation	Top of Screen	Bottom of Screen	Top of Screen	Bottom of Screen	Screened Lithology	Type of Test	Hydraulic Conductivity Estimate
		(m amsl)	(m bgs)	(m bgs)	(m amsl)	(m amsl)			(m/s)
GEMTEC Monitoring Wells									
BH24-1S	9-Jul-24	462.37	4.88	6.40	457.49	455.97	(CL) Silty Clay	Rising Head	6E-08
BH24-1D	9-Jul-24	462.29	7.62	9.14	454.67	453.15	(CL) Silty Clay Till	Rising Head	--
BH24-8	11-Jul-24	463.20	6.10	7.62	457.10	455.58	(CL) Silty Clay Till	Rising Head	5E-09
HLV2K Monitoring Wells									
BH1	11-Jul-24	463.03	3.25	6.30	459.78	456.73	Sandy Silt	Rising Head	5E-09

Notes:

All test were analysed using Bouwer and Rice (1976)

-- Monitoring well interpreted to be partially recovered prior to testing and as such, hydraulic conductivity has been omitted from results.

m amsl - metres above mean sea level

m bgs - meters below ground surface

m/s - meters per second

Bouwer, H. and R.C. Rice, 1976. A slug test method for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells, Water Resources Research, vol. 12, no. 3, pp. 423-428.

DRAFT

experience • knowledge • integrity



civil	civil
geotechnical	géotechnique
environmental	environnement
structural	structures
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

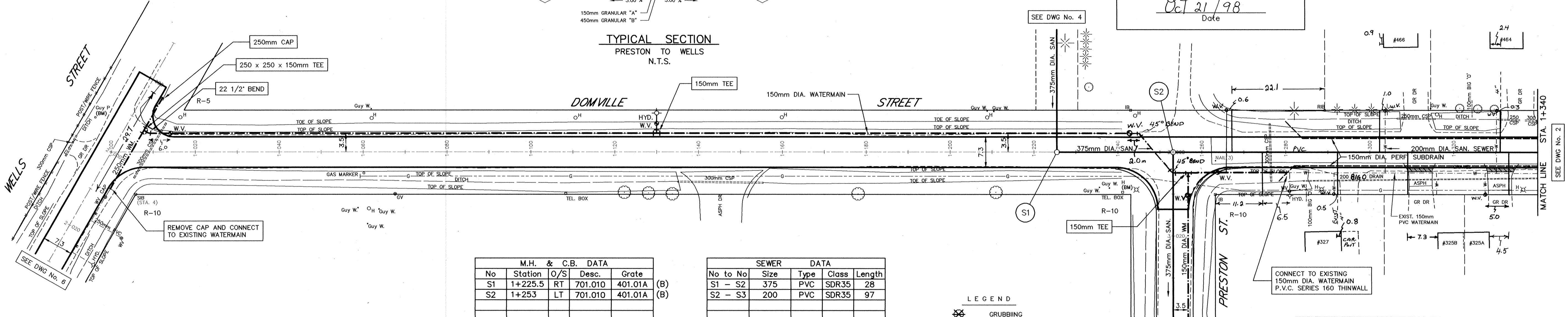
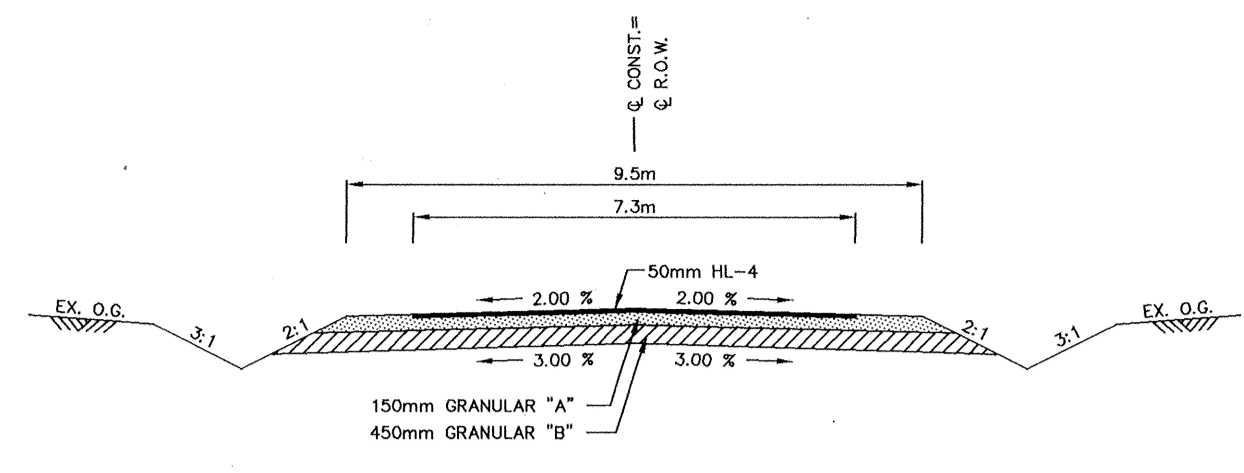
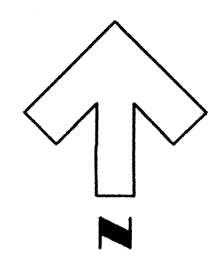
expérience • connaissance • intégrité



Appendix B3 Wasterwater Background Information



AS RECORDED
 The information on this record drawing is believed to be reliable, however it may be based, in part, upon information provided by others, which can not be verified by B.M. Ross and Associates Limited. Those relying on this record drawing are advised to obtain additional verification of its accuracy before applying it for any purpose.
 Date Oct 21/98



M.H. & C.B. DATA

No	Station	O/S	Desc.	Grate
S1	1+225.5	RT	701.010	401.01A (B)
S2	1+253	LT	701.010	401.01A (B)

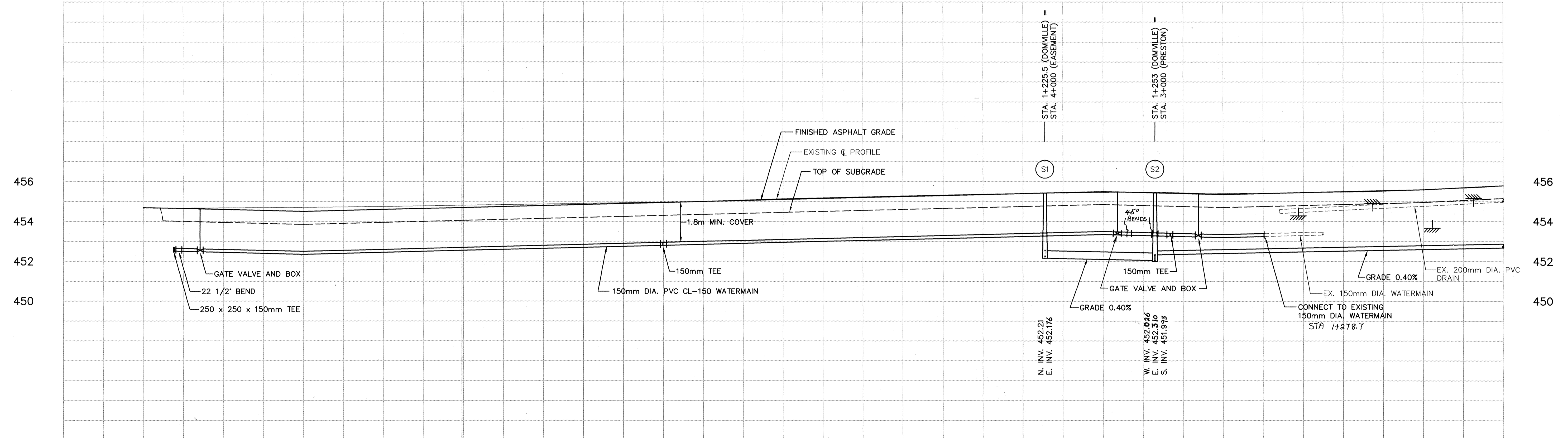
SEWER DATA

No to No	Size	Type	Class	Length
S1 - S2	375	PVC	SDR35	28
S2 - S3	200	PVC	SDR35	97

- LEGEND**
- GRUBBING
 - REMOVE EXISTING CONC. SIDEWALK
 - PLACE CONC. SIDEWALK AND DRIVES
 - REMOVE EXISTING ASPHALT PAVT
 - PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
 - REMOVE EXISTING CONC. CURB

NOTE:
 PLACE INLINE DRAIN CATCHBASINS No.1 TO 3 ON EXISTING DRAINS AS PER STANDARD DRAWING AND AS DIRECTED BY THE ENGINEER AT THE TIME OF CONSTRUCTION

NOTE:
 THE LOCATIONS OF EXISTING UNDERGROUND UTILITIES ARE SHOWN IN AN APPROXIMATE WAY ONLY AND HAVE NOT BEEN INDEPENDENTLY VERIFIED BY THE OWNER OR ITS REPRESENTATIVE. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL EXISTING UTILITIES BEFORE COMMENCING WORK, AND AGREES TO BE FULLY RESPONSIBLE FOR ANY AND ALL DAMAGES WHICH MIGHT BE OCCASIONED BY THE CONTRACTOR'S FAILURE TO EXACTLY LOCATE AND PRESERVE ANY AND ALL UNDERGROUND UTILITIES.



1+000	454.70	454.677	454.60	454.655	454.60	454.70	454.80	454.90	455.00	455.10	455.20	455.30	455.40	455.50	455.55	455.50	455.45	455.40	455.35	455.40	455.50	455.60	455.80
20		454.676																					
40		454.688																					
60		454.705																					
80		454.747																					
1+100		454.872																					
120		454.936																					
140		455.011																					
160		455.134																					
180		455.229																					
200		455.325																					
220		455.403																					
240		455.430																					
260		455.455																					
280		455.469																					
300		455.433																					
320		455.413																					
340		455.412																					
1+300		455.475																					
320		455.598																					
340		455.795																					

LEGEND

- SAN. or STM. EXISTING SEWERS, SANITARY or STORM
- MANHOLE and CATCHBASIN
- WATERMAIN
- FIRE HYDRANT
- GASMAIN
- UNDERGROUND TELEPHONE
- UNDERGROUND HYDRO
- UNDERGROUND T.V. CABLE
- UTILITY POLES

No.	DATE	REVISION

B.M. Elev. 455.706
 PK NAIL AND FLAG IN NE FACE OF H. POLE AT SW COR. DOMVILLE AND PRESTON

B.M. Elev. 454.598
 PK NAIL AND FLAG IN SE FACE OF GUY POLE AT NW COR. DOMVILLE AND WELLS

B.M.ROSS AND ASSOCIATES LIMITED
 CONSULTING ENGINEERS
 GODERICH AND MOUNT FOREST

DATE: JULY 30/98
 DRWN: P.J.L.
 CHKD: D.A.H.

SCALE: HORZ. 1 : 500
 VERT. 1 : 100

VILLAGE OF ARTHUR
DOMVILLE STREET

PLAN AND PROFILE OF STREET RECONSTRUCTION AND WATERMAIN FROM WELLS STREET TO STA. 1+340

FILE No. **98038**
 DWG No. **1 of 6**

LEGEND

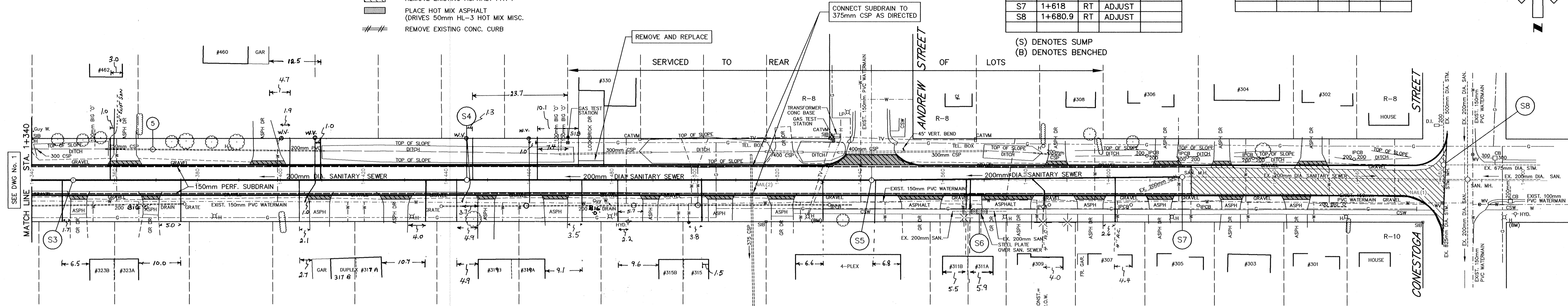
- GRUBBING
- REMOVE EXISTING CONC. SIDEWALK
- PLACE CONC. SIDEWALK AND DRIVES
- REMOVE EXISTING ASPHALT PAV'T
- PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
- REMOVE EXISTING CONC. CURB

M.H. & C.B. DATA

No	Station	O/S	Desc.	Grate
S3	1+350	©	701.010	401.01A (B)
S4	1+445	©	701.010	401.01A (B)
S5	1+543	©	701.010	401.01A (B)
S6	1+568.8	LT		REMOVE
S7	1+618	RT		ADJUST
S8	1+680.9	RT		ADJUST

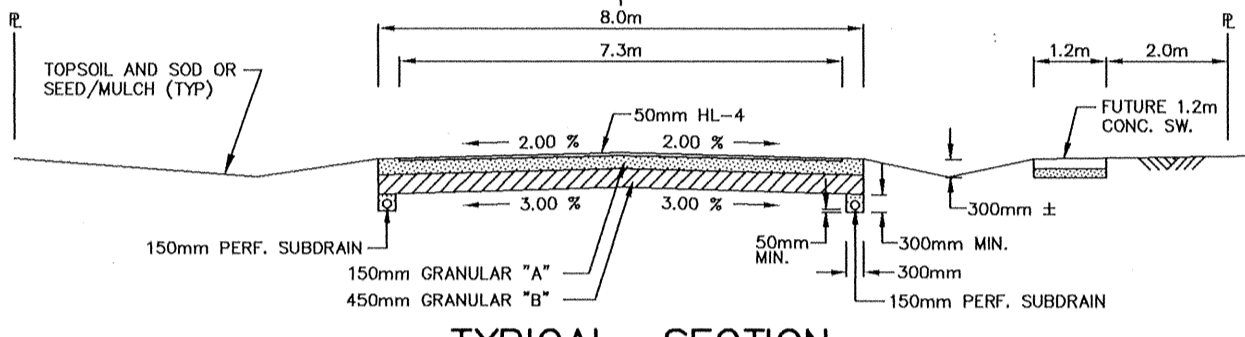
SEWER DATA

No to No	Size	Type	Class	Length
S3 - S4	200	PVC	SDR35	95
S4 - S5	200	PVC	SDR35	98
S5 - S7	200	PVC	SDR35	75

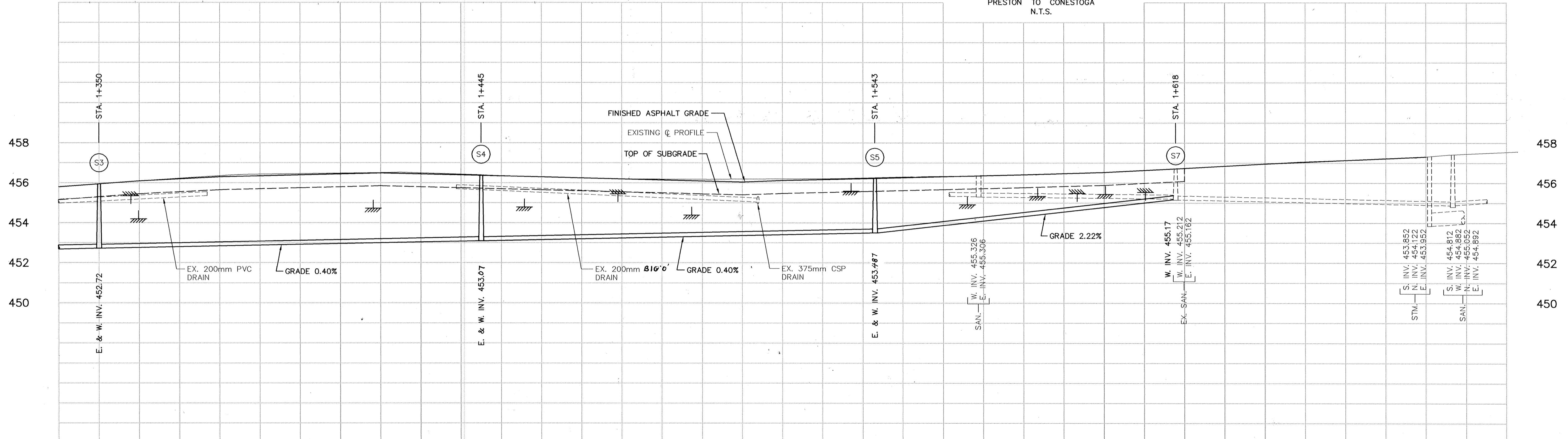


AS RECORDED
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 Oct 21 / 98
 Date

NOTE:
 PLACE INLINE DRAIN CATCHBASINS No.4 to 8 ON EXISTING DRAINS AS PER STANDARD DRAWING AND AS DIRECTED BY THE ENGINEER AT THE TIME OF CONSTRUCTION



NOTE:
 THE LOCATIONS OF EXISTING UNDERGROUND UTILITIES ARE SHOWN IN AN APPROXIMATE WAY ONLY AND HAVE NOT BEEN INDEPENDENTLY VERIFIED BY THE OWNER OR ITS REPRESENTATIVE. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL EXISTING UTILITIES BEFORE COMMENCING WORK, AND AGREES TO BE FULLY RESPONSIBLE FOR ANY AND ALL DAMAGES WHICH MIGHT BE OCCASIONED BY THE CONTRACTOR'S FAILURE TO EXACTLY LOCATE AND PRESERVE ANY AND ALL UNDERGROUND UTILITIES.



340	455.80	455.795	455.80	FINISHED ASPHALT GRADE
360	456.10	456.099	456.10	EXISTING C GRADE
380	456.30	456.356	456.30	STATION
400	456.40	456.460	456.40	
420	456.50	456.507	456.50	FILE No.
440	456.40	456.429	456.40	
460	456.30	456.303	456.30	DWG No.
480	456.20	456.191	456.20	
500	456.10	456.181	456.10	B.M. Elev. 457.629 NAIL IN W. FACE OF HLP AT SE COR. DOMVILLE AND CONESTOGA
520	456.12	456.200	456.12	
540	456.22	456.246	456.22	B.M. ROSS AND ASSOCIATES LIMITED CONSULTING ENGINEERS GODERICH AND MOUNT FOREST
560	456.32	456.334	456.32	
580	456.42	456.410	456.42	SCALE: HORZ. 1 : 500 VERT. 1 : 100
600	456.54	456.544	456.54	
620	456.74	456.752	456.74	PLAN AND PROFILE OF STREET RECONSTRUCTION AND SANITARY SEWER FROM STA. 1+340 TO CONESTOGA STREET
640	456.96	456.967	456.96	
660	457.14	457.140	457.14	
680	457.32	457.315	457.32	
700	457.80	457.550	457.80	

LEGEND

- SAN. or STM. EXISTING SEWERS, SANITARY or STORM
- M.H. or C.B. MANHOLE and CATCHBASIN
- WATERMAIN, FIRE HYDRANT
- CASMAIN
- UNDERGROUND TELEPHONE
- UNDERGROUND HYDRO
- UNDERGROUND T.V. CABLE
- UTILITY POLES

No.	DATE	REVISION

B.M. ROSS AND ASSOCIATES LIMITED
 CONSULTING ENGINEERS
 GODERICH AND MOUNT FOREST

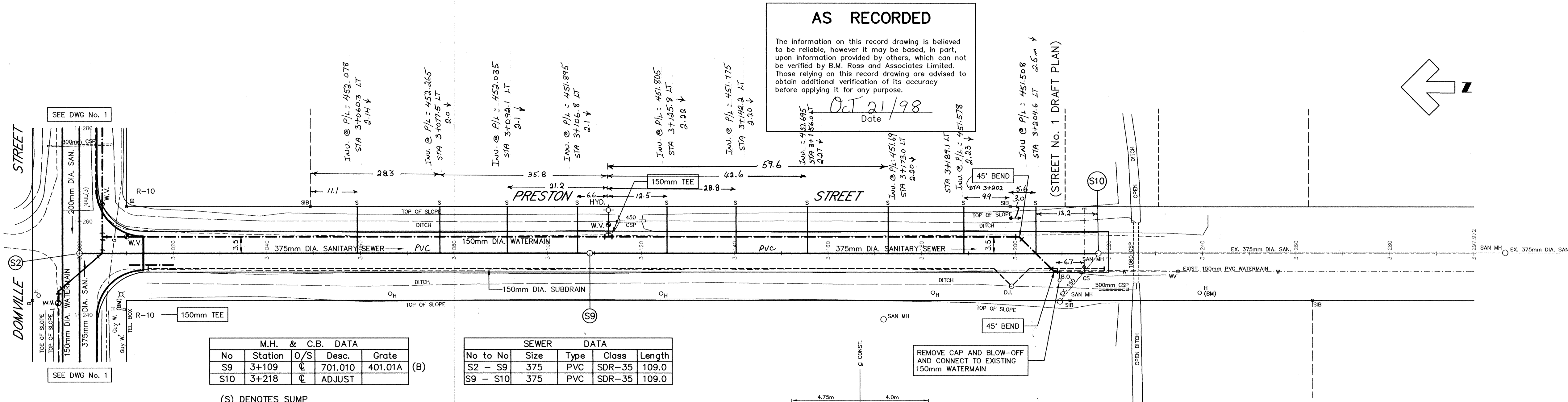
VILLAGE OF ARTHUR
 DOMVILLE STREET

PLAN AND PROFILE OF STREET RECONSTRUCTION AND SANITARY SEWER FROM STA. 1+340 TO CONESTOGA STREET

DATE: JULY 31/98
 DRWN - P.J.L.
 CHKD - D.A.H.

SCALE: HORZ. 1 : 500
 VERT. 1 : 100

FILE No. 98038
 DWG No. 2 of 6



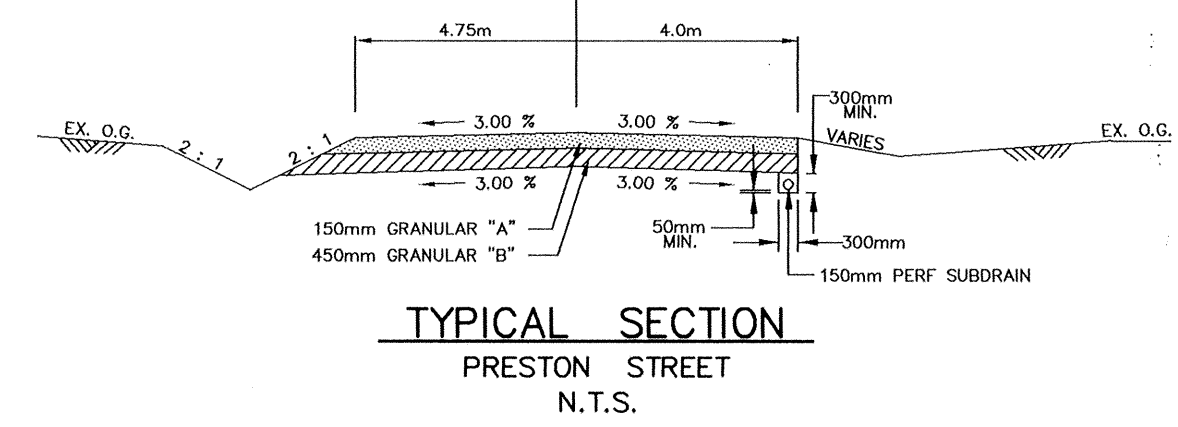
M.H. & C.B. DATA

No	Station	O/S	Desc.	Grate
S9	3+109	☐	701.010	401.01A (B)
S10	3+218	☐	ADJUST	

(S) DENOTES SUMP
(B) DENOTES BENCHED

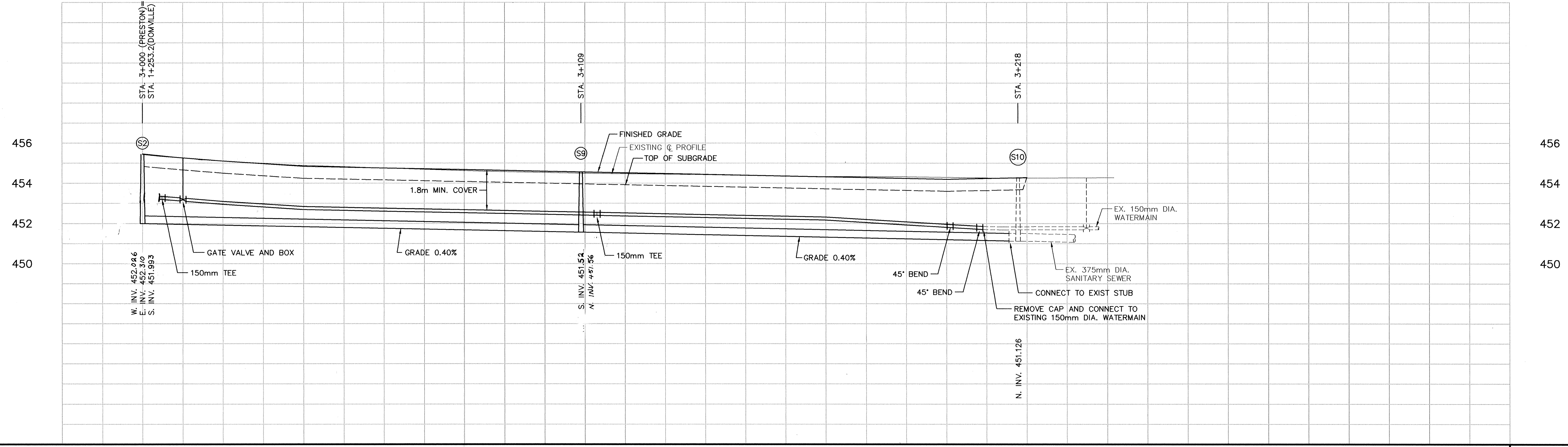
SEWER DATA

No to No	Size	Type	Class	Length
S2 - S9	375	PVC	SDR-35	109.0
S9 - S10	375	PVC	SDR-35	109.0



- LEGEND**
- GRUBBING
 - REMOVE EXISTING CONC. SIDEWALK
 - PLACE CONC. SIDEWALK AND DRIVES
 - REMOVE EXISTING ASPHALT PAVT
 - PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
 - REMOVE EXISTING CONC. CURB

NOTE:
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3+000	455.10	454.84	454.76	454.68	454.60	454.52	454.44	454.36	454.28	454.20	454.28	FINISHED GRADE
20	455.105	454.853	454.751	454.622	454.527	454.465	454.421	454.346	454.329	454.291	454.255	EXISTING ☐ GRADE
40												STATION
60												
80												
100												
120												
140												
160												
180												
200												
220												

LEGEND

- SAN. or STM. EXISTING SEWERS, SANITARY or STORM
- M.H. or C.B. MANHOLE and CATCHBASIN
- W. WATERMAIN, F. FIRE HYDRANT
- G. GASMAIN
- T. UNDERGROUND TELEPHONE
- H. UNDERGROUND HYDRO
- TV. UNDERGROUND T.V. CABLE
- U. UTILITY POLES

No.	DATE	REVISION

B.M. Elev. 455.706
 PK NAIL NE FACE HYDRO POLE AT
 SW. COR. DOMVILLE AND PRESTON

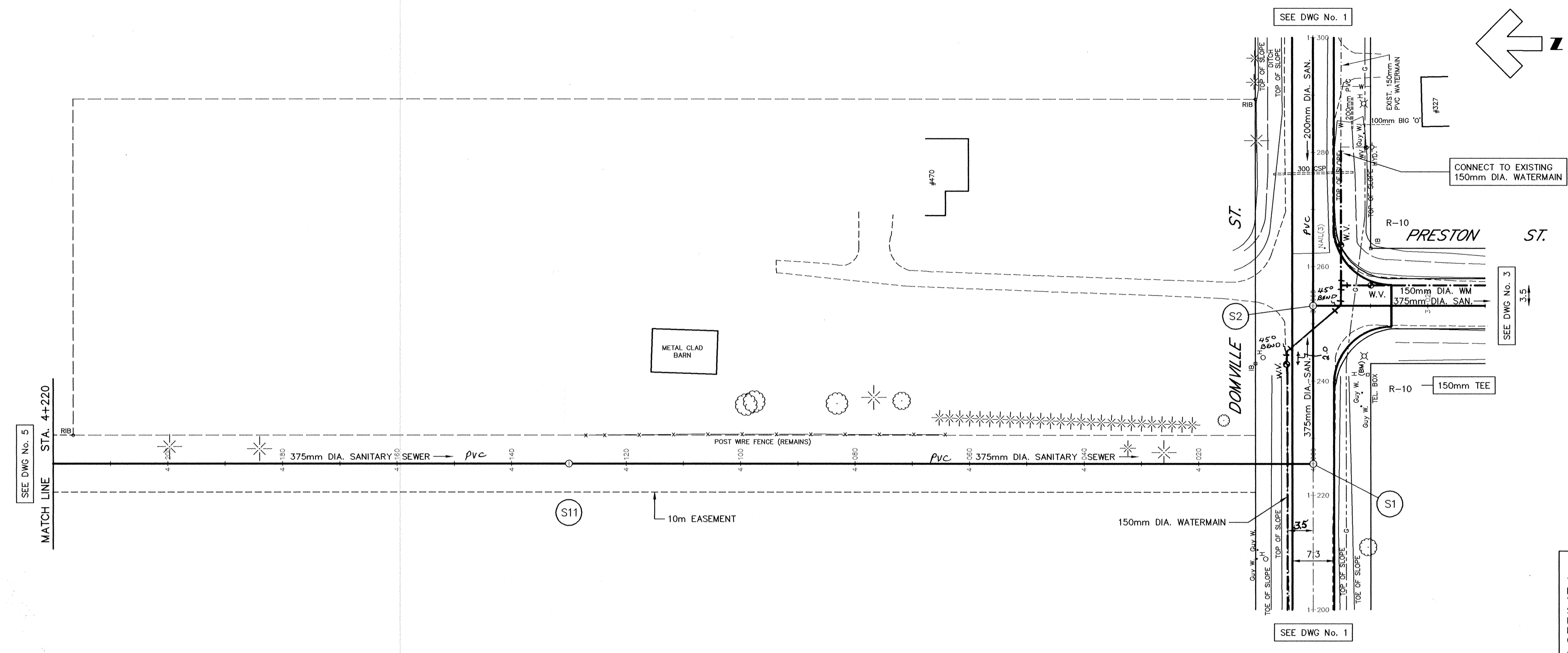
B.M.ROSS AND ASSOCIATES LIMITED
 CONSULTING ENGINEERS
 GORDERICH AND MOUNT FOREST

DATE: JULY 31/98
 DRWN: P.J.L.
 CHKD: D.A.H.

SCALE:
 HORZ. 1 : 500
 VERT. 1 : 100

VILLAGE OF ARTHUR
 PRESTON STREET
 PLAN AND PROFILE OF STREET RECONSTRUCTION
 WATERMAIN AND SANITARY SEWER FROM DOMVILLE
 TO STA. 3+220

FILE No. 98038
 DWG No. 3 of 6



M.H. & C.B. DATA				
No	Station	O/S	Desc.	Grate
S11	4+130	0	701.010	401.01A (B)

(S) DENOTES SUMP
(B) DENOTES BENCHED

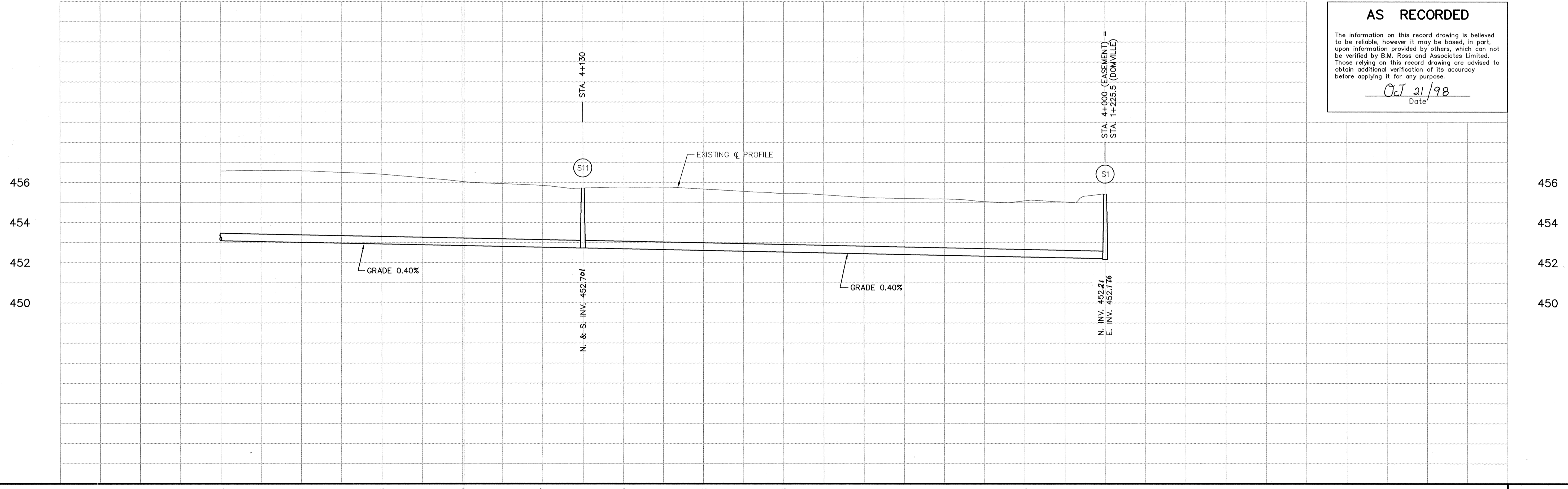
SEWER DATA				
No to No	Size	Type	Class	Length
S1 - S11	375	PVC	SDR35	1.30
S11 - S12	375	PVC	SDR35	1.30

LEGEND

- GRUBBING
- REMOVE EXISTING CONC. SIDEWALK
- PLACE CONC. SIDEWALK AND DRIVES
- REMOVE EXISTING ASPHALT PAVT
- PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
- REMOVE EXISTING CONC. CURB

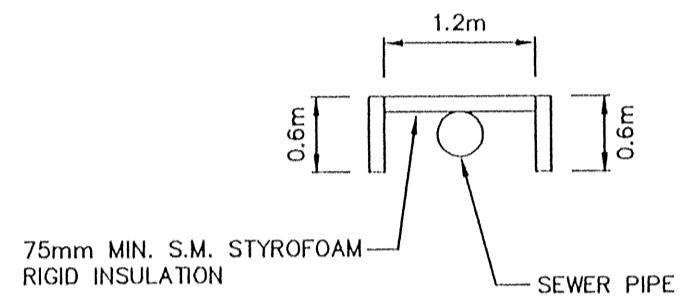
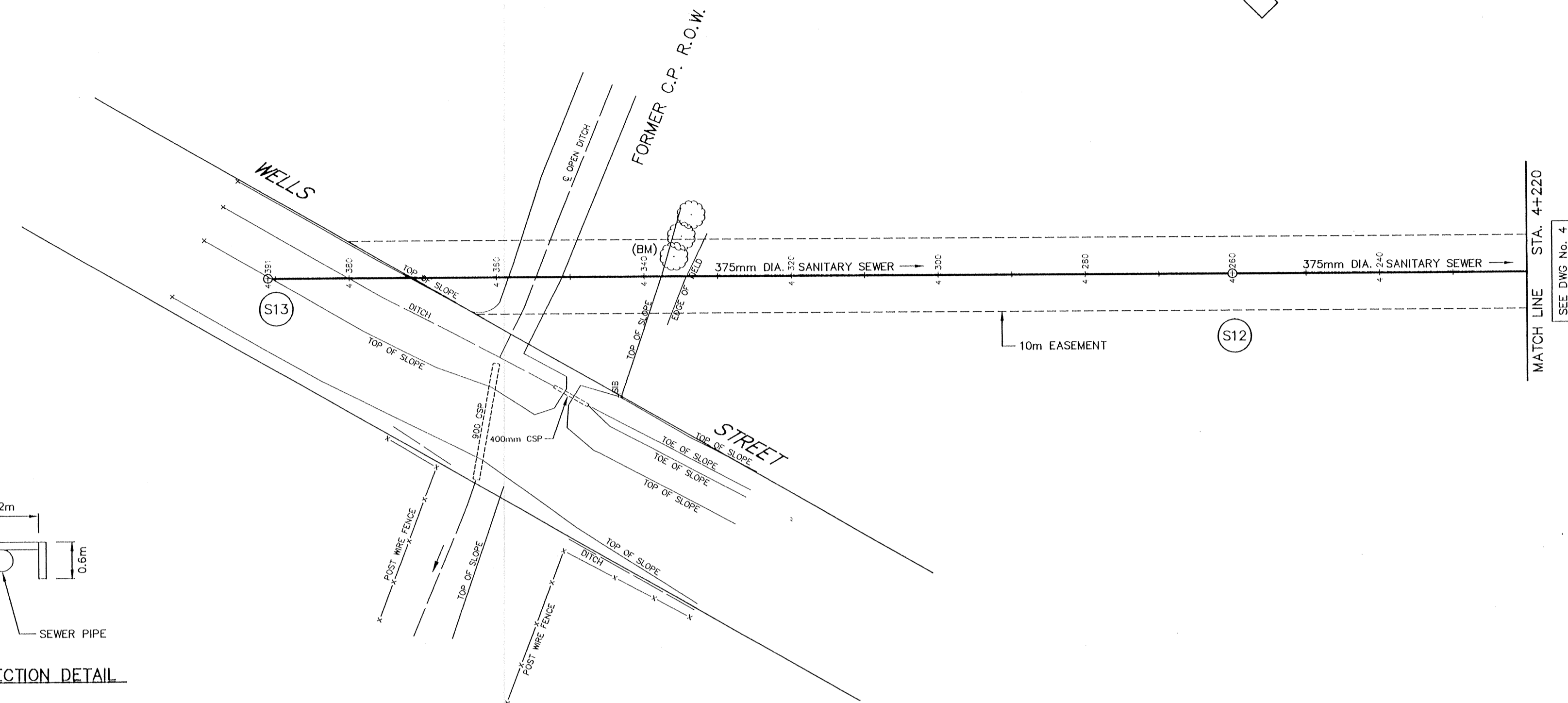
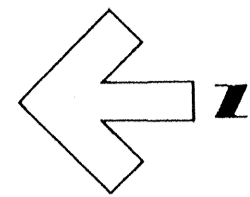
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Oct 21/98
Date



220	456.572	EXISTING G GRADE
4+200	456.576	STATION
180	456.408	
160	456.045	
140	455.847	
120	455.765	
4+100	455.672	
80	455.438	
60	455.251	
40	455.175	
20	455.088	
4+000	455.417	

<p>LEGEND</p> <p>SAN. or STM. EXISTING SEWERS, SANITARY or STORM</p> <p>M.H. OR C.B. MANHOLE and CATCHBASIN</p> <p>WATERMAIN, FIRE HYDRANT</p> <p>GASMAIN</p> <p>UNDERGROUND TELEPHONE</p> <p>UNDERGROUND HYDRO</p> <p>UNDERGROUND T.V. CABLE</p> <p>UTILITY POLES</p>	<p>B.M. Elev. 455.706</p> <p>PK NAIL AND FLAG IN NE FACE OF H. POLE AT SW COR DOMVILLE AND PRESTON</p>	<p>B.M.ROSS AND ASSOCIATES LIMITED</p> <p>CONSULTING ENGINEERS</p> <p>GODERICH AND MOUNT FOREST</p>	<p>VILLAGE OF ARTHUR</p> <p><u>EASEMENT</u></p> <p>PLAN AND PROFILE OF SANITARY SEWER FROM DOMVILLE ST. TO STA. 4+220</p>	<p>FILE No.</p> <p>98038</p>
				<p>DWG No.</p> <p>4 of 6</p>
<p>No.</p> <p>DATE</p> <p>REVISION</p>	<p>DATE</p> <p>JULY 31/98</p>	<p>DRWN - P.J.L.</p> <p>CHKD - D.A.H.</p>	<p>SCALE</p> <p>HORZ. 1 : 500</p> <p>VERT. 1 : 100</p>	



FROST PROTECTION DETAIL

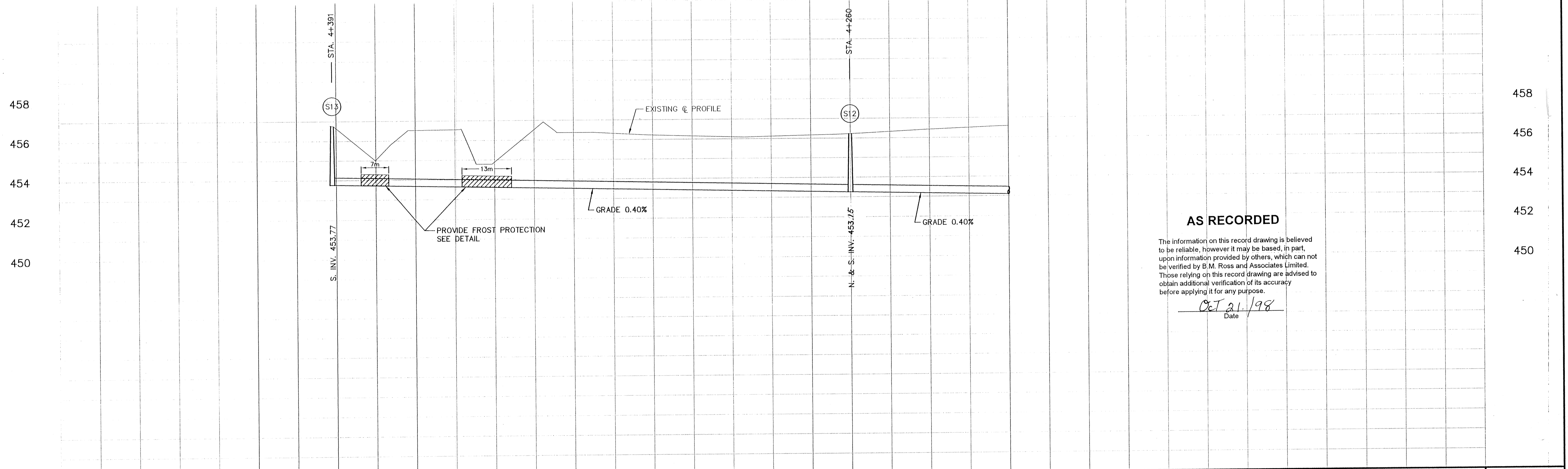
M.H. & C.B. DATA				
No	Station	O/S	Desc.	Grate
S12	4+260	Q	701.010	401.01A (B)
S13	4+391	Q	701.010	401.01A (B)

SEWER DATA				
No to No	Size	Type	Class	Length
S12 - S13	375	PVC	SDR35	131

(S) DENOTES SUMP
(B) DENOTES BENCHED

- LEGEND
- GRUBBING
 - REMOVE EXISTING CONC. SIDEWALK
 - PLACE CONC. SIDEWALK AND DRIVES
 - REMOVE EXISTING ASPHALT PAVT
 - PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
 - REMOVE EXISTING CONC. CURB

NOTE:
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Oct 21 / 98
Date

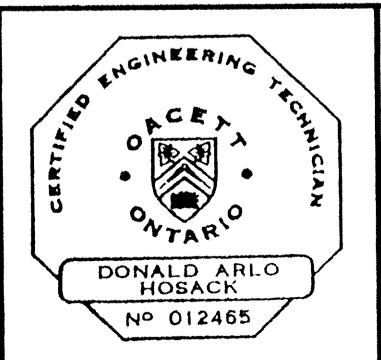
4+400	390	380	360	340	320	4+300	280	260	240	220	EXISTING Q. GRADE
456.650	455.026	456.550	456.650	456.317	456.160	456.121	456.224	456.424	456.572		STATION

LEGEND

- S.H. or S.M. EXISTING SEWERS, SANITARY or STORM
- M.H. or C.B. MANHOLE and CATCHBASIN
- WATERMAIN, FIRE HYDRANT
- GASMAIN
- UNDERGROUND TELEPHONE
- UNDERGROUND HYDRO
- UNDERGROUND T.V. CABLE
- UTILITY POLES

No.	DATE	REVISION

B.M. Elev. 457.301
PK NAIL AND FLAG IN W. FACE OF
600# ELM ON S. SIDE OF OLD RAIL
PROPERTY 30m E. OF WELLS



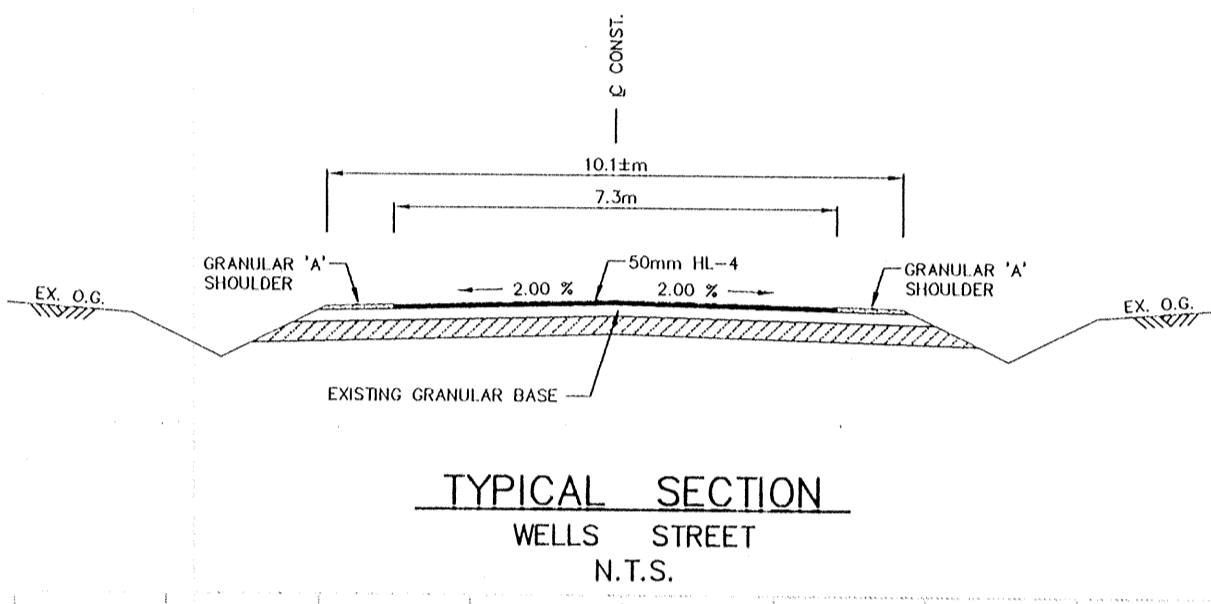
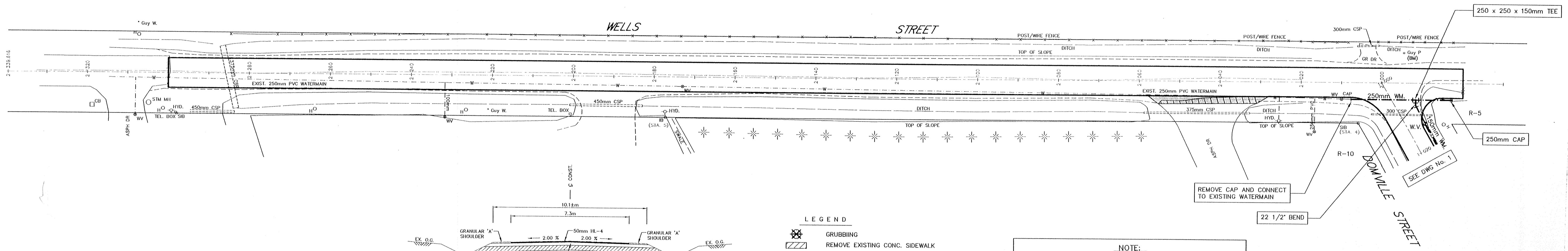
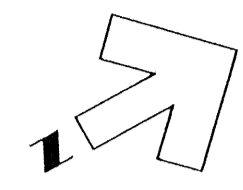
B.M.ROSS AND ASSOCIATES LIMITED
CONSULTING ENGINEERS
GODERICH AND MOUNT FOREST

DATE JULY 31/98	DRWN-P.J.L. CHKD-D.A.H.	SCALE HORZ. 1 : 500 VERT. 1 : 100
--------------------	----------------------------	---

VILLAGE OF ARTHUR
EASEMENT

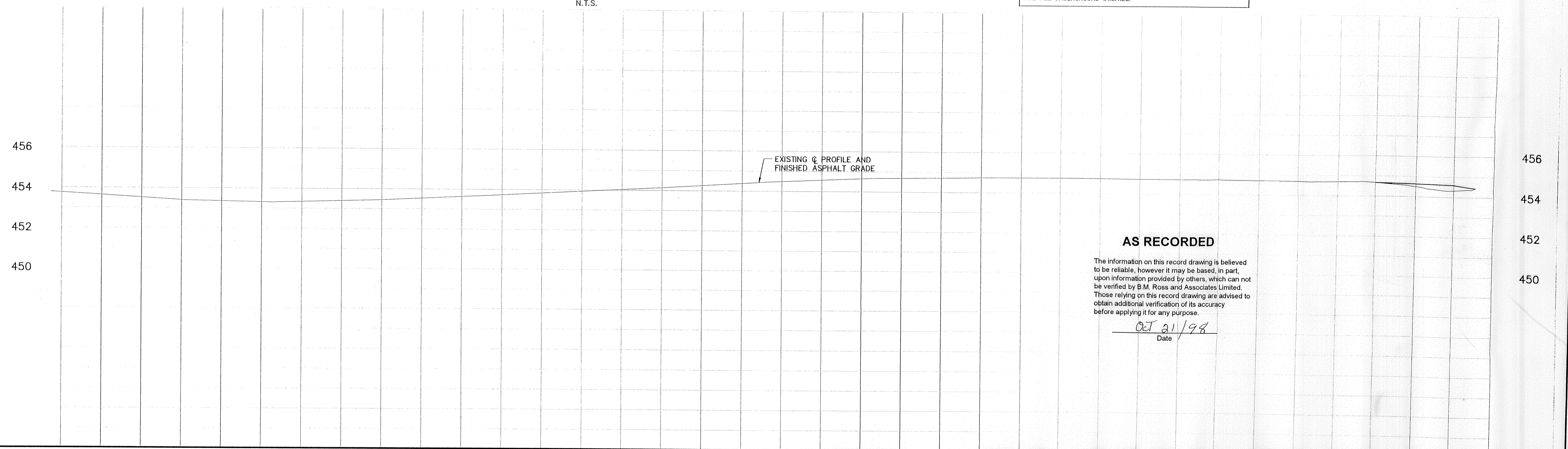
PLAN AND PROFILE OF SANITARY SEWER
FROM STA. 4+220 TO STA. 4+390

FILE No. 98038
DWG No. 5 of 6



- LEGEND**
- GRUBBING
 - REMOVE EXISTING CONC. SIDEWALK
 - PLACE CONC. SIDEWALK AND DRIVES
 - REMOVE EXISTING ASPHALT PAVT
 - PLACE HOT MIX ASPHALT (DRIVES 50mm HL-3 HOT MIX MISC.)
 - REMOVE EXISTING CONC. CURB

NOTE:
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 O.J. 21/98
 Date

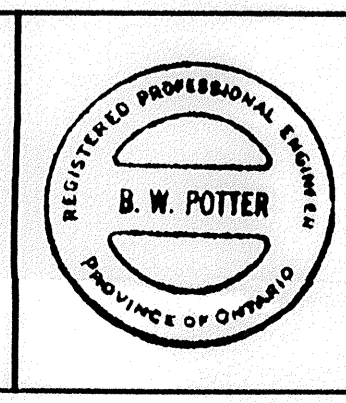
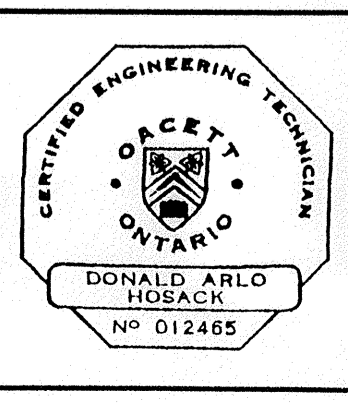
340	453.649	453.403	453.37	453.45	453.60	453.79	453.99	454.20	454.43	454.61	454.74	454.79	454.80	454.79	454.81	454.75	454.70	454.60	FINISHED ASPHALT GRADE
																			EXISTING G. GRADE
																			STATION

LEGEND

- S&I or STM --- EXISTING SEWERS, SANITARY or STORM
- W --- WATERMAIN
- G --- GASMAIN
- T --- UNDERGROUND TELEPHONE
- H --- UNDERGROUND HYDRO
- V --- UNDERGROUND T.V. CABLE
- U --- UTILITY POLES
- M --- MANHOLE and CATCHBASIN
- F --- FIRE HYDRANT

No.	DATE	REVISION

B.M. Elev. 454.598
 PK. NAIL AND FLAG IN SE. FACE OF GUY POLE AT NW. COR. DOMVILLE AND WELLS



B.M.ROSS AND ASSOCIATES LIMITED
 CONSULTING ENGINEERS
 GODERICH AND MOUNT FOREST

DATE	DRWN - P.J.L.	SCALE
JULY 29/98	CHKD - D.A.H.	HORZ. 1 : 500 VERT. 1 : 100

VILLAGE OF ARTHUR
WELLS STREET
 PLAN AND PROFILE OF GRADING AND PAVING
 FROM STA. 2+020 TO STA. 2+300

FILE No.
98038
 DWG No.
6 of 6

**Arthur Water and
Wastewater Servicing**
In the Township of Wellington North

Summary Report
FEBRUARY 2025

Report Prepared For:
TRIBUTE COMMUNITIES

Report Prepared By:



Engineering Services

DLW Engineering Services Ltd

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DLW Project Number: TRB001

1.1 INTRODUCTION

DLW Engineering Services Limited was retained by Tribute Communities to provide consulting engineering services related to the conceptual design of the water and wastewater servicing of the Tribute Arthur Development in the Township of Wellington North. This memorandum has been prepared to establish the viability of expanding the existing Arthur Water Supply and Wastewater Treatment Plant (WWTP) to accommodate the Tribute Arthur Development. This memorandum will show that there are viable water and wastewater servicing options for the proposed Tribute Arthur Development.

1.2 WATER SERVICING

According to the September 2020 Water and Sanitary Systems Technical Study – Arthur, completed by Triton Engineering Services Limited for the Township of Wellington North, the Arthur water system is a single-pressure zone water main distribution network that is pressurized by two elevated towers. Water is supplied to the system from three bedrock wells. The system currently provides service to 939 homes and 109 Industrial/Commercial/Institutional (ICI) properties, according to Township records (2020). The system also provides fire protection to the entire service area.

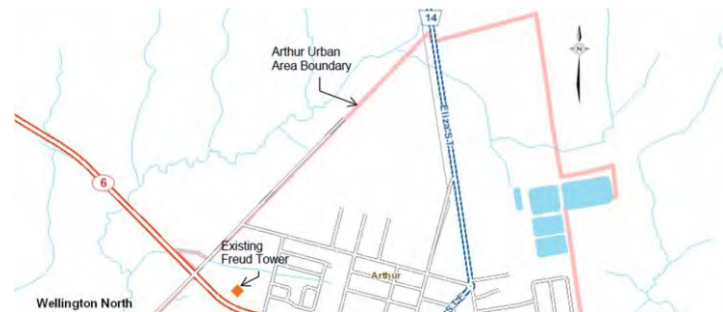




Figure 1: Existing water works location plan

1.2.1 Water Demands

A review of the historical water usage rates within Arthur indicates that the maximum day demand for the existing system is 652 Lpcd as presented in the Reserve Capacity Calculation (RCC), and this per person RCC value includes the ICI demands. The Triton 2020 study states that a maximum day-per-person flow rate of 500 Lpcd will be applied for future demand calculations. Based on Table 3.1 in that report, the total population (equivalent residential units) is multiplied by 500 Lpcd to determine the future flows.

At this preliminary stage, the proposed Tribute Arthur development will be based on either 823 or 872 residential units. Based on a residential density of 2.7 persons per unit this will equate to 2,222 persons for 823 units, and 2,354 persons for 872 units (Table 1).

As noted in the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, the existing 2023 Arthur Water supply system services an equivalent population of approximately 3,195 persons. With the addition of the Tribute Arthur development, the water supply service population will increase the total population to the following:

Table 1: Water Service Population

Development	Total Existing and Proposed Population
Existing 2023 Development (Nov 2024 PIC Boards)	3,195
Existing plus 823 Unit Tribute Development	5,417
Existing plus 872 Unit Tribute Development	5,549

1.2.2 Water Wells

Water supply is provided by three bedrock wells, named Well No. 7B, Well No. 8A, and Well No. 8B as shown in the figure above and which are described as follows:

1.2.2.1 Well No. 7B

- Located at 109 Wells Street West near the Conestoga River.
- Commissioned in 1998.
- 46 m deep drilled.
- Well pump is a submersible type complete with a 30kW (40 hp) motor that discharges directly to the distribution system (i.e., no high lift pumps).
- Rated capacity is 22.7 L/s (1,961 m³/day).
- Disinfection using sodium hypochlorite. Contact time is provided by the oversized discharge main.
- Iron sequestering treatment provided.

1.2.2.2 Well No. 8A/8B

- Located on Part of Lots 20 and 21, Concession A, approximately 1.15 km south of County Road 109 and 235 m east of Highway 6.
- Commissioned in 2005.
- Depth of wells is 61.9 m and 62.2 m for 8A and 8B, respectively.
- Well pumps are submersible type complete with a 30 kW (40hp) motor which discharges directly to the distribution system (i.e., no high lift pumps).
- Rated capacities are 26.1 L/s (2,255 m³/day) each; however, the Permit to Take Water (PTTW) allows for the operator to pump either Well 8A or Well 8B, but not both wells concurrently. Therefore, total production from this facility is limited to 2,255 m³/day.
- Disinfection using sodium hypochlorite. Contact time is provided by the oversized discharge main.
- Treatment for high manganese is provided.
- Standby power provided by diesel generator.

MECP lists the firm capacity of infrastructure as the capacity with the largest unit out of operation. Thus, based on this definition and the above, the total production capability (i.e. source capacity) of the Arthur water system is 4,216 m³/day. Because of the duty/standby nature of Wells 8A and 8B which is mandated by MECP, this is also the firm production capacity for the well system.

Notwithstanding this, in the 4 April 2022 Triton reserve capacity analysis report for Arthur, Triton states that the Arthur firm capacity is 2,255 m³/day.

1.2.3 Reserve Capacity Analysis

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the Maximum Day Demand flow of the 3 years (2019, 2020, & 2021) was 1,531 m³/day. Given Triton’s assertion that the firm capacity of the water supply system is 2,255 m³/day this results in a reserve capacity of 724 m³/day. This translates to 489 uncommitted reserve capacity equivalent residential units (Table 2).

Table 2: Reserve Capacity Analysis – Arthur Water Supply

Item	Description	Based on Firm Capacity in 2022 Triton Report
1	Firm Capacity (m ³ /day)	2,255
2	Three-Year Maximum Day Demand (m ³ /day)	1,531
3	Three-Year Average Day Demand (m ³ /day)	1,036
4	Available reserve capacity (1-2) (m ³ /day)	724
5	Three-Year Max/Avg Day Peaking Factor (2/3)	1.48
6	Three-Year Average Residential Demand (m ³ /d)	604
7	Peaked Max Day Residential Flow (5x6) (m ³ /day)	893
8	Occupied Serviced households	968
9	PPU (2016 census data)	2.4
10	Population Served (8x9)	2,323
11	Maximum Residential Day Demand per Capita (7/10) (m ³ /day)	0.384
12	Additional Population that can be served (4/11)	1,885
13	Persons per new Equivalent Residential Unit (ERU) (2018 Growth Management Plan)	2.7
14	Additional Equivalent Residential Units that can be served (12/13)	701
15	Committed Equivalent Residential Units	212
16	Uncommitted Reserve Capacity Equivalent Residential Units (14-15)	489

It is worth noting that in the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards the current water system now has an uncommitted reserve capacity of 710 m³/d. This translates to 480 uncommitted reserve capacity equivalent residential units.

The November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards also states that a new well will be provided that will supply at least 864 m³/d but more likely that a new 2,332 m³/d well will be provided. This well was shown to be located on the Tribute Arthur lands. We presume the larger capacity well is typical of the well capacities seen at Wells 7B, 8A and 8B. When this well is constructed the uncommitted reserve capacity will increase to 1,574 m³/d to 3,042 m³/d, which even on the lower end is more than enough flow for the proposed Tribute Arthur development.

1.2.4 Water Treatment

As noted above, each well is treated with iron or manganese removal plus disinfected with chlorine to control bacteria, algae, and viruses by the provincial Procedures for Disinfection for Drinking Water in Ontario guidelines. We assume that any new water supply well will provide water of similar quality as the existing supply wells and will require either iron or manganese treatment as well as chlorine disinfection. The treatment plant for the proposed well(s) will require a small building structure to house the discharge piping, valves, and flowmeter, iron or manganese treatment system, chlorination system, MCC's, instrumentation system to monitor water quality, a chlorine contact tank to ensure that the water receives a sufficient amount of chlorine contact before reaching the first consumer. The treatment plant will also include a generator to provide uninterrupted power for the pumps to ensure uninterrupted water supply to the distribution system.

While we believe that there is sufficient well and associated water treatment capacity for the proposed Tribute Arthur development, if additional well supplies are required, the appropriate treatment system can be provided to treat the new well supplies and will likely include iron and manganese treatment plus disinfection.

1.2.5 Water Storage

According to the September 2020 Triton Water and Sanitary Systems Technical Study, the storage for the Arthur water system is provided by two elevated facilities. These facilities are as follows:

1.2.5.1 Charles Street Tower

- Located near the intersection of Charles Street East and Isabella Street, in the southeast part of the system (195 Isabella Street East).

- Multi-legged steel tank
- Commissioned in 1932
- Volume is 227 m³
- Operation range: 494.2 m – 499.6 m

The November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards states that this tower has reached the end of its useful life and that the Township will be decommissioning this tower when the new elevated water storage tank is constructed.

1.2.5.2 Freud (Spheroid) Tower

- Located just north of Smith Street between Preston and Wells Streets in the northwest
- part of the system (460 Smith Street).
- All steel spheroid tank
- Commissioned in 1967
- Volume is 1,137 m³
- Operation range: 494.0 m – 499.2 m

According to Triton discussions with Township staff, these facilities have been recently inspected and no significant deficiencies were noted. The total system storage volume currently available is 1,364 m³.

1.2.5.3 New Elevated Water Storage Tank proposed in 2024 Class EA

According to the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, a new 900m³ elevated water storage tank is proposed to address proposed development to 2051. The additional elevated water storage tank was identified as being located adjacent to the proposed new well on the Tribute Arthur lands. The Class EA states that this additional capacity is required to accommodate a 2051 population of 4,800 persons. This new storage tank was proposed to accommodate an additional population of 1,605 persons (or 639 additional units based on 2.6 persons per unit). It is worth noting that this amount of storage will not accommodate the full Tribute Arthur development which on the top end will result in a total population of 5,549 persons.

1.2.5.4 Proposed Storage Requirement

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the existing system storage in Arthur is 1,137m³ and the required storage in 2021 was 994 m³ based on MECP design guidelines. Thus, the existing available surplus storage in the system was 143 m³. Triton's existing storage assumption is based on the fact that the Charles Street Tower is reaching the end of its useful life and thus excluded from their existing storage calculation. According to the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards Triton now projects a storage deficit of 118m³.

We have assumed the MECP formulation for the development of system storage and used the Triton April 2022 recommendation for maximum day flows based on 500 Lpcd. Based on the existing serviced and committed population in combination with the proposed Tribute Arthur development we estimate the total storage requirement for Arthur will increase to the following:

- 2,875 m³ for 823 units (Tribute development requires 1,359 m³)
- 2,922 m³ for 872 units, and (Tribute development requires 1,406 m³)

The above Tribute storage requirements are based on the fact that the total system fire flows will grow from the existing 100 L/s for 2 hours (based on Triton 2020 report) to the proposed 150 L/s for 3 hours with the addition of 823 units, or 152 L/s for 3 hours for 872 units, based on the increase in service population.

The Tribute Arthur development will require a new reservoir to meet their storage demands. Assuming the MECP formulation for the development of system storage and using the Triton April 2022 recommendations, if we assume that the Tribute Arthur development storage is sized on the basis of just its own population, the water storage tank for the Tribute Arthur development will be as follows:

- 1,232 m³ for 823 units and
- 1,271 m³ for 872 units.

Thus, the storage for the Tribute development will be based on the whether the storage is based on servicing strictly the Tribute Arthur development or on the impact to the full Arthur development will be as follows:

- For 823 development units, storage will be between 1,232 to 1,359 m³, and
- For 872 development units 1,271 to 1,406 m³.

Based on the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, the proposed elevated water storage tank should increase from its currently proposed 900 m³ to at least 1,800 m³ to accommodate the existing development and the full Tribute Arthur development.

If the proposed elevated water storage tank in the Class EA is not expanded to accommodate the Tribute Arthur development, the reservoir configuration at the Tribute Arthur development could be either of the following:

- Inground Concrete Reservoir with associated Booster Pumping Station, all located on the Tribute Arthur development site with jockey, duty, and fire pumps.
- Elevated Storage Tank located on the Tribute Arthur development site fed directly from the wells

1.2.6 Proposed Water Supply and Storage System Layout

Based on the preliminary concept plan, the proposed well, pumphouse, and elevated water storage tank will be located on proposed Block 67 at the intersection of Wells Street and Macaulay Street. Figure 2 below illustrates a plan showing the proposed groundwater well, well pumphouse/water treatment plant and elevated water storage tank. This plan shows that an area of approximate size 0.2 ha is required to house these facilities.

1.3 WASTEWATER SERVICING

1.3.1 Project Background

Further to our preliminary site servicing investigation dated March 18, 2022, SCS and DLW held a conference call on April 25, 2022, with Matthew Aston, who is the Director of Operations at the Township of Wellington North.

Mr. Aston provided us with a Memorandum Re: Review of the 2016 Arthur WWTP Class EA – Summary of the Basis of Development of the Design ADF of 2,300 m³/d (EA Review). According to the EA Review “*Existing (base) 2012 service population was reported to be 2,596 persons and residential growth projections were based on 103 lots from the planned Phase 3 Eastridge contribution (284 persons) and other anticipated growth within the service area (714 persons), for a total residential growth of 998 persons to 2031. The residential growth forecast for the Village of Arthur (excluding Eastridge) was taken from the Wellington County Official Plan 1999, revised in February 2011 (Part 3, Wellington Growth Strategy, Table 2, p. 10). The total projected residential population to 2031 was, therefore, estimated to be 3,594 persons.*” The Class EA used a design per capita sewage generation flow rate (for residential growth) of 370 L/cap/d (dry weather) plus an Inflow/Infiltration allowance of 90 L/cap/d, for an overall value of 460 L/cap/d.

The 2016 Arthur WWTP Class EA identified a Phase 1 expansion of the plant from 1,465 m³/d to 1,860 m³/d and a Phase 2 expansion from 1,860 m³/d to 2,300 m³/d. Mr. Aston also provided a Staff Report on water and wastewater technical updates for the Mayor and Members of Council Special for a Meeting held on June 2, 2021. In this staff report, the shovel-ready Phase 2 wastewater treatment plant expansion was identified to cost approximately \$8.3 million based on 2018 pricing.

The EA Review memorandum stated that the Conestoga River was determined to be a Policy 2 receiver for total phosphorus (TP), E. coli, and dissolved oxygen (July and August only). Further, the MOE (now MECP) approved an effluent Total Phosphorus limit of 0.25 mg/L, equivalent to an annual TP loading limit of 210 kg/year at an ADF of 2,300 m³/d. This is critical because with different technology that provides a higher level of treatment, phosphorus can be reliably reduced to as low as 0.05mg/L. The WWTP appears to have sufficient capacity to accommodate this expansion, but this will be addressed in design.

1.3.2 Existing WWTP Information and Sewerage Pumping Stations

1.3.2.1 Existing WWTP

The Arthur Wastewater Treatment Plant (WWTP), constructed in approximately 1990, services the community of Arthur in the Township of Wellington North (the Township). The WWTP is an extended aeration plant with tertiary filtration, with offsite effluent

storage lagoons, and seasonal discharge of final effluent to the Conestoga River. Per the ECA, discharge is permitted between September 16 to April 30 and limited to no more than 6,500 m³/d. It is important to note that no discharge is allowed into the river from June to September due to the low flow conditions in the river. The allowable effluent flow rate is dependent on the effluent TAN concentration and the flow rate in the Conestoga River. In 2016 when the ESR was prepared, the rated average day flow (ADF) capacity of the works was 1,465 m³/d. Flow to the WWTP is conveyed via two sewage pumping stations (Wells St. Sewage Pumping Station (SPS) and Frederick St. SPS) and gravity sewers. The unit process unit details of the 2016 WWTP are given in Table 3.

Table 3: Unit Sizing and Process Details of the existing WWTP based on ADF of 1,465 m³/day

Parameter	Process Design
Grit Removal Type Number Dimensions Capacity	Manually cleaned Grit Channels 2 5.4 m x 0.75 m x 0.5 m SWD 5,045 m ³ /d
Comminution Capacity	5,045 m ³ /d
Screening Type Capacity	Manually cleaned 5,045 m ³ /d
Aeration Tank Number Dimensions Volume (each cell) Volume (total) Diffuser Type	2 cell annular ring-type aeration tank 27.95 m Equivalent Length x 4.65 m x 4.18 m SWD - Cell 1 27.26 m Equivalent Length x 4.65 m x 4.18 m SWD - Cell 2 543 m ³ - Cell 1 530 m ³ - Cell 2 1,073 m ³ Coarse Bubble
Blowers Number Capacity	2 (1 duty, 1 standby) 486 L/s, each

Parameter	Process Design
RAS/WAS Pumps Number Capacity Storage Volume	2 34 L/s, each 50 m ³
Secondary Clarifier Type Number Dimensions Surface Area	Circular inlet clarifier 1 13.5 m diameter x 3.8 m SWD 143 m ²
Chemical Pumps (Alum) Number Capacity Chemical Storage Volume	2 (1 duty, 1 standby) 250 L/d, each 23 m ³ storage tank & 450 L/day tank
Tertiary Filtration Type Number of Modules Total Filtration Area Backwash pumps	Continuous backwash, upflow, deep bed granular media (1 m depth) 6 27.9 m ³ 2 wash water reject pumps (1 duty, 1 standby) Each rated at 6.1 L/s at 3.5 m TDH
UV Disinfection No. of bank Modules Per bank Lamps per module Channel dimension Capacity	2 banks in a series 4 7.9 m long x 0.5 m wide x 0.9 m SWD 6,500 m ³ /d

1.3.2.2 Pumping stations

The Arthur wastewater collection system consists of a gravity sewer network with two SPSs:

1. The Wells St. SPS.

- Two submersible pumps (one duty, one standby), each with a rated capacity of 16L/s (1,382 m³/d).

- One wet well with a liquid volume of approximately 120 m³.
- One standby diesel generator.

2. The Frederick St. SPS.

- Two submersible pumps (one duty, one standby), each with a rated capacity of 58.4 L/s (approximately 5,045 m³/d).
- One reinforced wet well, measuring approximately 5.3 m x 5.3 m x 6.2 m (deep), providing a total storage volume of approximately 174 m³.
- One 60 kW standby diesel generator with 450 L fuel tank.

Each pumping station has a force main that discharges near the treatment plant. Over the review period (2007 – 2015), bypasses of the Frederick St. SPS have occurred during peakflow periods. As such, the Township wishes to evaluate the existing capacity of both pumping stations and determine if any upgrades and/or expansions are required for the study's design year of 2031.

1.3.2.3 Population growth and flow projections for Phase-II expansion

The future servicing needs for the Town of Arthur are based on the historic flows from the existing service area, plus the projected flows from the future residential and industrial/commercial/institutional (ICI) development.

The Existing (base) 2012 service population was reported to be 2,596 persons and residential growth projections were based on 103 lots from the planned Phase 3 Eastridge contribution (284 persons) and other anticipated growth within the service area (714 persons), for a total residential growth of 998 persons to 2031 (Table 4). The total projected residential population in 2031 was estimated to be 3,594 persons including future Eastridge contribution (Arthur WWTP Class EA, 2016). Industrial / commercial / institutional (ICI) flows were also considered. Based on the 2012 Master Plan Study for Water Supply and Sanitary Sewage (Triton Engineering, 2012), a total of 28.7 ha of developable non-residential land would be serviced by 2031, including a vacant designated industrial parcel with 25.1 ha of developable land, and 3.6 ha of developable Highway Commercial parcels. Future growth contribution associated with Golden Valley Farms was based on an equivalent of 9 residential units which was reported to “reflect the remaining unused portion of the Golden Valley Farms allocation” as included in the 2012 Reserve Capacity Calculations for the Arthur WWTP (Triton Engineering, 2012).

Table 4: Population growth and flow projections

Parameter	Value
Residential Flow Projections	
2012 Service Population	2,596
Historical ADF	1,171 m ³ /d
Future Population Contribution from Eastridge	284
Other anticipated population growth	714
2031 Projected Service Population	3,594
Design Per Capita Flow	370 Lpcd
Design Per Capita Average Inflow and Infiltration	90 Lpcd
2031 Residential Flow	1,653 m ³ /d
ICI Flow Projections	
Industrial - Golden Valley Historic Flows	171 m ³ /d
Industrial - Golden Valley Growth Flows to 2031	11 m ³ /d
Commercial/Industrial Land Flows	488 m ³ /d
Total ICI Flow Projections	670 m ³ /d
TOTAL 2031 FLOW PROJECTION	2,300 m³/d

It is worth noting that the 2019-2020 Annual Performance Report for the Arthur WWTP states that the annual design flow was 1,329 m³/d and the maximum monthly average flow occurred in March 2020 at an ADF of 2,088 m³/d. Interestingly, the annual report lists the plant capacity at 1,465 m³/d and the Total Phosphorus effluent concentration limit at 1.0 mg/L which we understand should have been 0.25 mg/L in both Phases 1 and 2 as identified in the Class EA.

The biosolids management strategy identified in the Class EA for the plant is as follows:

Phase 1 – No biosolids work is required. Liquid biosolids are shipped to the Lystek regional processing facility located in Dundalk, Ontario.

Phase 2 - Onsite aerobic digestion followed by dewatering of digested sludge using geotextile tubes. The 2 existing (total 4 nos) sludge storage tanks, each of 150 m³ volume will be converted into anaerobic digester which will reduce the sludge storage capacity to 300 m³. Hence, an additional sludge storage volume of 120 m³ is required (In phase-II expansion, a total of 420 m³ of sludge storage volume is required based on 240 days of onsite storage capacity, Table 3.14 of Class EA). Upgrades include increased blower capacity and increased sludge transfer pump size. The dewatered cake will be land-applied seasonally.

1.3.2.5 Projected cost estimates identified in the Class EA

The project liquid stream costs identified in the Class EA were as follows:

Table 7: Project Cost Estimates for WWTP Phase-I and Phase-II Liquid Stream Expansions

Treatment Process	Phase-I	Phase-II
General/Miscellaneous	\$340,000	\$569,000
Headworks	\$0	\$3,020,000
Storage Lagoon Conveyance Upgrades	\$695,000	\$1,275,000
Blowers, Standby Power, and Other Common Upgrades	\$681,000	\$0 (2)
Equalization Tank	\$1,674,000	\$0
Secondary Treatment	\$0	\$809,000
Sub Total	\$3,390,000	\$5,673,000

1.3.3 Progress to Date on Wastewater Treatment Plant (Completed)

The proposed Phase-I components in the Class EA report are already completed, resulting in an updated capacity of 1,860 m³/day from 1,465 m³/day (Annual Performance Report 2022). The key components covered under the phase-I expansion as identified in the Class EA are given below:

- a. Primary treatment:** A new 2,100m³ equalization tank is added to restrict the flow at 6,450 m³/d (design capacity of the headworks building). Hence, no changes in the headworks building are required. *This is confusing as we understand the existing treated capacity of the headworks system before the Phase I expansion was listed at 5,045 m³/d.*
- b. Secondary treatment:** It is important to mention that based on the influent BOD concentrations reported in the 2021 annual performance report, the required aeration tank volume for the Phase-I rated capacity of 1869 m³/day at F/M ratio, MLSS concentrations of 0.1 and 4000 mg/l is 1612 m³. Notably, the existing aeration tank volume is only 1,073 m³ and needs an additional volume of 612 m³. In contrast, there is no provision for additional volume considered for phase-1 expansion. Consequently, WWTP is operated at a higher volumetric loading of 0.378 kg/m³/day and F/M ratio of 0.15 than that specified in the MECP guidelines i.e., 0.17 to 0.24 kg/m³/day and 0.05 to 0.15. Moreover, the existing surface area for clarification is only 143 m² whereas the minimum required additional surface area for clarification for Phase-I expansion i.e., 1860 m³/day is 143 m² based on a SOR of 37 m³/m²/day at a design peak flow of 7,069 m³/d plus 10% (10% of ADF) recycled flow from the tertiary filters (Table 4.2, Class EA). Thus, there seems to be insufficient clarification tankage available in Phase-I, which could be the reason for the current poor plant performance. It is also worth noting that the historical data from the class EA suggests that the peak SOR was 59 m³/m²/day rather than the MECP specified maximum value of ≤ 37 m³/m²/day which would exacerbate this issue for Phase-I.
- c. Tertiary Filtration / Chemical Addition / UV Disinfection:** The MDF to existing tertiary filters will be 7,068 m³/day, resulting in a need for a surface area of 25.32 m² which is less than the existing surface area of 27.2 m². Hence, existing systems have a sufficient capacity for Phase-I expansion.
- d. Effluent Storage and Conveyance:** The existing conveyance system consists of a 2.56 km forcemain and transfer pumps. The existing forcemain consists of 860 m of 200 mm diameter pipe, 1,100 m of 250 mm diameter pipe, and 600 m of 300 mm diameter pipe. There are 2 pumps each rated at 58.5 L/s (5,054 m³/d) *which closely matches the peak flow of the existing headworks facility.* Phase-I peak flows would be restricted to approximately 6,450 m³/d through the operation of an equalization tank. There is insufficient capacity in the conveyance system to transfer projected equalized peak flows. Additional conveyance capacity can be provided by replacing the entire 860 m length of the existing 200 mm diameter pipe with 350 mm diameter pipe. It is anticipated that the existing transfer pumps may have sufficient capacity required via minor modifications (i.e. new impellers). The projected storage requirement at the Phase-I ADF capacity is approximately 246,000 m³,

and the existing capacity of the treated effluent holding ponds is approximately 340,000 m³ which is sufficient for Phase-I.

- e. **Sludge Management:** No upgrade is required. Liquid biosolids will be hauled to the Lystek regional processing facility located in Dundalk, Ontario.



Figure 4: Google Earth Aerial View of Arthur WWTP December 2023

1.3.4 Reserve Capacity Analysis Post Phase-I Expansion

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the ADF of the 3 years (2019, 2020, & 2021) was 1,293 m³/day, resulting in a reserve capacity of 567 m³/day (Table 9). For the reserve capacity analysis, the adopted per capita flow in the Triton report was 350 Lpcd. On the contrary, the ESR report recommended a per capita flow of 460 Lpcd (370 Lpcd + I/I of 90 Lpcd). Therefore, in our assessment, we adapted the ESR recommended value and revised the reserve capacity assessment. Considering the per capita flow of 460 Lpcd, an additional population of 1,232 can be further served using this available reserve capacity. Considering a residential

density of 2.7 ppu (based on the 2018 growth management plan), 456 additional new units can be serviced. Per Table 3 of the Triton report, there are 212 committed development residential units, resulting in 244 uncommitted development residential units. These available uncommitted development residential units are equivalent to a population of 658 and a flow capacity of 303 m³/day.

Table 9: Reserve Capacity Analysis Post Phase-I Execution

Item	Description	2021
1	Updated capacity of the WWTP (m ³ /day)	1,860
2	Average ADF (m ³ /day)	1,293
3	Available reserve capacity (1-2) (m ³ /day)	567
4	Future development per capita flow (m ³ /day)	0.460
5	The additional population that can be served (3/4)	1,232
6	PPU	2.7
7	New additional residential units that can be served (5/6)	456
8	Committed residential developmental units	212
9	Available uncommitted residential developmental units (7-8)	244
10	Available uncommitted flow capacity (9 x 6 x4) (m ³ /day)	303

1.3.5 Future Development Scenarios

1.3.5.1 Phase-II expansion: rated capacity of 2,300 m³/day

We believe that the Township is now focusing on expanding the WWTP plant capacity to 2,300 m³/day (Phase-II development identified in the class EA). The components proposed under the Phase-II expansion are given below:

- a. Primary treatment:** A new headworks building is required because the existing system has reached the end of its useful life, hence, the old headworks system will be decommissioned. The headworks components should be designed at a Phase-II PIF i.e., 12,887 m³/day. However, this proposed capacity of the equipment is not going to be sufficient for the future growth. Hence, it is vital to add the provisions for the future growth. *Moreover, it is typical to provide equalization after the headworks system to mitigate the potential for solids deposition in the equalization tankage.*
- b. Secondary treatment:** The proposed solution to achieve Phase-II plant secondary treatment capacity is to twin the existing extended aeration package plant. In this context, it is proposed that the equalization tanks constructed in Phase-I will be going to be converted into an extended aeration tank and secondary clarifier. *However, this proposed approach results in directing the Peak Instantaneous Flows to the downstream processes such as secondary treatment system, clarifiers, tertiary filters, and UV disinfection system, consequently, disturbing the biological process and need to design secondary clarification and tertiary filtration process units to accept the peak instantaneous flows.*

- c. Tertiary Filtration / Chemical Addition / UV Disinfection System:** The capacity of existing tertiary filtration, UV disinfection system, and chemical addition processes is sufficient to treat the Phase-II ADF capacity of 2,300 m³/d. *While the chemical addition processes may be sufficient, removing the equalization tankage from the process train as proposed in the Phase-II expansion plan (Class EA) requires additional filters and UV disinfection system capacity. The existing tertiary filters have an ECA-rated capacity of 6,500 m³/d at a filtration rate of 2.7 L/m²/sec. They could potentially treat a maximum inflow of 7,955 m³/day (based on the MECP design criteria of 3.3 L/m²/sec and an existing total filter area of 27.9 m²). In contrast, considering design based on the MDF of 8828 m³/day resulted in a need for additional filters having a minimum surface area of 4.65 m². Likewise, the UV disinfection system has an ECA-rated capacity of 6,500 m³/d and needs an expansion at MDF flow. However, without the equalization tankage, the filters and UV disinfection system will see peak flows of as high as 12,887 m³/d. Thus, this strategy would require either adding additional filtration and UV disinfection capacity or adding new equalization tankage to buffer out the peak flows.*
- d. Effluent Storage and Conveyance:** For Phase-II capacity, additional conveyance capacity would be added by upgrading the remaining 1,100 m of 250 mm diameter pipe to 350 mm diameter pipe, and through the installation of new conveyance pumps. For Phase-II, approximately 304,000 m³ of storage will be required which is less than the existing lagoon storage of 340,000 m³. Thus, there is no additional storage capacity required for the Phase-II expansion.
- e. Sludge Management:** The sludge management options offered in the Class EA, onsite aerobic digestion, with onsite storage of biosolids using geotextile tubes are proposed. This involves the conversion of two of the four existing sludge holding tanks (each is sized for 150 m³) to aerobic digesters. However, based on our calculations the sludge production will range between 50 to 55 m³/day. This resulted in a need for a total aerobic digestion volume of 1573 m³. Considering the existing digestion volume of 468 m³ and converting all sludge storage tanks into aerobic digesters, still 505 m³ of additional volume is required. Upgrades include increased blower capacity and increased sludge transfer pump size. Moreover, an additional sludge storage tank having a total volume of 400 m³ is required for the dewatered sludge storage. The dewatered cake will be land-applied seasonally.

1.3.5.2 Tribute Communities Developments

Tribute Communities is assessing the potential for further expanding the Arthur WWTP to service an additional 823 or 872 residential units. Consequently, the plant capacity will be further developed to 3,346 m³/day or 3,406 m³/d serving the range of development considered by Tribute Communities. In this report, the flow projected with 823 and 872 lots is denoted as Scenario-I and II expansions as presented in Table 10 below:

Table 10: Design flows for Arthur WWTP including Phase-II expansion flow plus proposed Tribute Communities Developments

		Scenario-1 (Phase-II Flow + 823 Lots)	Scenario-2 (Phase-II Flow + 872 Lots)
Population		7,379	7,379
Peaking Factor ¹		5.6	5.6
Maximum Day Factor ²		3.8	3.8
Average Dry Weather Flow	m ³ /d	2,680	2,730
	L/s	31.03	31.60
Average Wet Weather Flow or Average Day Flow (ADF)	m ³ /d	3,333	3,395
	L/s	38.60	39.30
Maximum Day Flow	m ³ /d	12,665	12,899
	L/s	146.60	149.30
Peak Dry Weather Flow	m ³ /d	15,014	15,289
	L/s	174	177
Peak Wet Weather Flow	m ³ /d	16,658	16,965
	L/s	192.8	196.35
Peak Instantaneous Flow	m ³ /d	18,509	18,849
	L/s	214.23	218
¹ Peaking Factor and Maximum day Factor ² are based on the class EA report (page 349)			

1.3.5.3 Sewage Pumping Station

Based on the current Tribute concept plan, it has been determined that a new sewage pumping station will be required in the Tribute Arthur development for the eastern portion of the proposed development. The pumping station will be located on a 0.05 ha parcel of land on Block 70 at the intersection of Street R and Eliza Street. The pumping station will pump through a forcemain to the sanitary collection system at the intersection of Street A and Eliza Street. The station is estimated to have a capacity of approximately 20 L/s.

Figure 5 below illustrates a plan showing the proposed sewage pumping station. This plan shows that an area of approximate size 0.05 ha is required to house these facilities.

1.4.2 Wastewater System

The existing capacity of the Arthur wastewater treatment plant after the Phase-I expansion (as identified in the APR) is 1,860 m³/day with an uncommitted reserve capacity of 303 m³/day (Refer to Table 7). Under the phase-II expansion (also as identified in the Class EA), it is proposed to expand the plant to the capacity of 2,300 m³/day to cover the demand for the year 2031. The recently planned Tribute Communities development will be based on 823 or 872 additional residential lots. This proposed development will require a new sewage pumping station located on the Tribute Arthur lands. The pumping station will be located on a 0.05 ha parcel of land on Block 70 at the intersection of Street R and Eliza Street. The pumping station will pump through a forcemain to the sanitary collection system at the intersection of Street A and Eliza Street. The proposed Tribute Arthur development will further need WWTP expansion to cater to future flows. This report discusses the possible solutions to integrate the proposed Phase-II WWTP expansion with the planned new developments that are being planned by Tribute Communities. The key recommendations and conclusions are as follows:

1. The proposed headworks capacity of 12,887 m³/day for the phase-II PIF is not sufficient for the anticipated Tribute developments. Hence, either capacity increase or provision for additional units must be arranged for future Tribute developments (Scenario I and II).
2. For the Phase-II expansion, it is proposed to convert the equalization tank (constructed in Phase-I expansion) into a secondary treatment system (aeration tank + clarifier). This conversion will eliminate the WWTP's capacity to shave the future peak flow of 12,887 m³/day for phase-II expansion (Table 3). Consequently, it will result in the downstream process units being undersized and will work at rates that far exceed the MECF design guidelines for the aeration tankage, clarifiers, tertiary filters, and UV disinfection system. Therefore, it is imperative to provide sufficiently sized (>12 hours at ADF) equalization tankage in Phase-II to mitigate the potential for having to significantly upsize the secondary and tertiary treatment systems at substantial additional cost to the project. The equalization tankage would eliminate the carryover of the excess MLSS to the clarifiers at peak events, and the need for a large surface area for clarification, filtration, and ultraviolet disinfection units. The existing storage lagoons are sufficiently sized for the Phase-II expansion, but the effluent pumps will need to be upsized.
3. The design concentrations for the recently completed Phase-I (1,860 m³/day) and proposed Phase-II (2,300 m³/day) expansions were based on the typical organic loading specified in the MECF guidelines (Figure 5 and Table 11). However, the influent concentrations given in the annual performance reports indicate that the incoming BOD₅ concentrations are 35 to 40% higher. Hence, it is important to revise the proposed Phase-II designs accordingly and the same should be adopted for future expansions.
4. The total phosphorus (TP) and total ammonia nitrogen concentrations of 0.34 mg/l and 2.80 mg/l in the effluent (year 2019-2020) were higher than the approved concentration of 0.25 mg/l and 8.12 listed in the Class EA report (Table 6). Notably, the addition of the equalization tank, increased blower capacity, and high alum dosing in the Phase-I expansion resulted in an improved effluent quality (refer to 2021-22 APR) meeting the

ECA limits. However, converting the equalization tanks into a secondary treatment system may deteriorate the effluent quality.

5. Based on the available historical data, the WAS rate to the digester was 60.4 m³/day, which seems very high. This is possibly due to the low WAS consistency of less than 1% compared to the 2% assumed in the class EA and/or higher influent BOD₅ concentration compared to the design value. Even the maximum monthly sludge generation calculated in the class EA was only 27.4 m³/day which is still lower than the quantity observed in the historical data. Hence, it is important to evaluate the sludge treatment system and storage capacity considering the actual WAS consistency. The WAS range as per the MECP is 0.8 to 1.0% for extended aeration processes but class EA is considered 2%, assuming substantial thickening in the sludge hopper. This assumed WAS consistency of 2% may result in an under sizing of the sludge treatment system and storage units.
6. The requirement for holding the effluent in storage lagoons is based on the Total Ammonia Nitrogen concentrations in the effluent and the typically low flows in the Conestoga River from June to September. It may be possible to convince the MECP that with an enhanced level of treatment, the flow restriction to the Conestoga River may be eliminated. However, there is some risk associated with this approach and we have assumed that the storage and flow restriction to the river will continue for all future expansion phases. Thus, for all future flow conditions additional storage capacity is required for the non-discharge period (June to September).



WATER AND SANITARY SYSTEMS

TECHNICAL STUDY – ARTHUR

SEPTEMBER 2020

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systems.” additionally, “...development that optimizes the efficient use of this infrastructure should be prioritized and balanced with the construction of new infrastructure. Future infrastructure planning is required to be undertaken on a watershed- and asset management basis, through servicing master plans and environmental assessments...”.

1.3 Township of Wellington North, 2018 Development Charges Background Study & By-Law Final Report (DFA Infrastructure International Inc., 2018)

DFA Infrastructure International Inc. (DFA) was retained by the Township to complete a development charges (DC) background study, calculate new rates and prepare a new DC By-law to meet the requirements of the Development Charges Act and replace the existing By-law 51-13. Development charges collected in the Township are for municipal services, which include: administration, fire protection, parks and recreation, water and wastewater, and roads and related services. Associated costs for the extent and delivery of these services considers the existing population and businesses and expected future growth, which was determined for the 10-year period of 2018 through 2027 and 2028 through 2041 build-out period and considered servicing studies completed for the Township, which identified growth related capital needs. New growth areas and population growth for the community are in accordance with the GMP. Historical growth (last 10 years) was considered to determine historical service levels. A copy of the 2018 DC Report is included in Appendix A2.

2 FUTURE DEVELOPMENT PROJECTIONS

2.1 General

Per the GMP, “the 2036 and 2041 population, housing and employment growth forecasts for the Township of Wellington North, as established in the County Official Plan, should continue to be used for planning purposes to determine urban land requirements.” And “The growth forecasts for Wellington North and the distribution of the population and housing forecasts within the Township should be revised and updated through future reviews of the County Official Plan, to align the forecasts with local growth patterns and infrastructure plans”.

The population growth forecast in the GMP for Arthur reflects the increase in available capacity resulting from the Phase 2 expansion of the Arthur Wastewater Treatment Facility and is focused within the existing built-boundary that was delineated by the Province as part of the 2006 Growth Plan for the Greater Golden Horseshoe. Additionally, consistent with the County Official Plan and 2006 Growth Plan, development within the existing built-up areas should intensify by 20%, although, this target may be increased as a result of the next Municipal Comprehensive Review (to be issued on or before July 1, 2022). “The target is intended to focus growth and development within the existing built-up area of the urban centres, and to ensure outward growth in greenfields is compact”, which can be achieved through infilling and development of existing vacant land in the built-up area, building expansions or conversion, or redevelopment.

Development outside of the delineated built-up area (i.e. “intensification area”), but within the urban boundary that is designated for urban land uses (i.e., designated greenfield areas [DGA] or “urban expansion area”) is called greenfield development. The current designated greenfield area target for the Township is 40 residents and jobs per hectare, consistent with the County Official Plan.

The GMP provided the expected growth to 2041, which is summarized in Table 2.1. As per Table 2.1, the 2016 population is greater than the existing serviced population (based on 2020 Reserve

Capacity Calculations (RCC); however, this is likely caused by a difference in total population versus serviced population. Therefore, for the purposes of this study, the 2016 serviced population is used as the existing population with growth added to this. It is anticipated that all future population will be connected to both water and sanitary systems.

Table 2.1 - Arthur Growth (as per GMP)

Arthur Growth (as per GMP)				
Year	Population (Capita)	Households (Equivalent Residential Units, ERUs)	Capita per ERU	Growth (Capita/Year)
2016	2,725	1,005	2.7	-
2036	4,115	1,525	2.7	69.5
2041	4,460	1,665	2.7	69.0

As per the Growth Plan for the Greater Golden Horseshoe, the Township can plan for infrastructure beyond the 2041 planning horizon presented in the current County Official Plan. Four future land development stages were identified in the GMP (refer to “Development Stages – Arthur” figure in the GMP in Appendix A1). Details for each development scenario are provided in Sections 2.2 through 2.5. The intensification and greenfield growth targets are applied to each development scenario to evaluate the adequacy of the systems for the existing and future population beyond calendar year 2045. Assuming that the 2041 population noted in the GMP (refer to Table 2.1) is achieved, growth to 2045 can be extrapolated and associated water demands and sanitary loading within Arthur can be estimated. The interpolated growth for Arthur 2020 to 2045 study period is provided in Table 2.2. Refer to Appendix A3 for additional information regarding interpolated growth projections.

Table 2.2 - Arthur Growth (Interpolated)

Arthur Growth (Interpolated)				
Year	Population (Capita)	Households (ERUs)	Capita per ERU	Growth (Capita/Year)
2020	2,410	970	2.5	-
2025	3,351	1,242	2.7	69.5
2030	3,698	1,370	2.7	69.5
2035	4,046	1,499	2.7	69.5
2040	4,391	1,639	2.7	69.0
2045	4,736	1,768	2.7	69.0

2.2 Development Scenarios

As presented in the GMP, the four Development Scenarios have been assessed to determine development potential based on land availability and reasonable/expected growth densities described herein. The expectation is that not all of the areas identified within the stages will fully develop, however when assessing linear infrastructures ability to service the future lands, full development of viable lands will be assumed. The expectation is that modelling the future serviceability this way will flag certain infrastructure as having insufficient capacity to service the full buildout of that Development Stage. This information will then help the municipality make informed decisions regarding development areas based on infrastructure expansion/remediation costs. Further, this information will be used to size future infrastructure upgrades required to support development.

Vertical infrastructure, such as area wide water supply/storage and sewage treatment, will be assessed based on the overall growth targets set out in the GMP, not on specific development areas/parcels.

2.3 Stage 1 Development Scenario

The Stage 1 Development Scenario includes an additional 8.7 hectares (ha) of land within Arthur's urban area boundary, with lands zoned as low-density residential (5.5 ha with 212 ERUs as per the latest Draft Plans provided in support of these development areas), high-density residential (0.6 ha with 36 ERUs as per the latest Draft Plan), future development (0.5 ha with 15 ERUs as per zoning mapping) and highway commercial (2.1 ha with 55 ERUs assumed based on MECP Equivalents and Developable Land). The Stage 1 Development Area is presented on Figure 2.1 and the Future Land Use Table for the Stage 1 Development Area is presented in Appendix B1.

As per the County Official Plan and 2006 Growth Plan, development within the existing built-up areas should intensify by 20% to 2041; however, since these growth targets were developed in 2016 and this technical study is being completed in 2020, the intensification targets have been revised such that the 20% intensification targets will be reached by 2045. This intensification will be applied to the "Downtown Intensification Area" (refer Figure 2.1) and results in an additional 231 units added by 2045 in the Stage 1 Scenario.

2.4 Stage 2 Development Scenario

The Stage 2 Development Scenario includes an additional 15.2 ha of land within Arthur's urban area boundary including lands zoned as low-density residential and future development (9.8 ha with 250 ERUs), mixed high and low density residential (5.1 ha with 287 ERUs) and mixed highway commercial and future development (0.3 ha with 8 ERUs). The Stage 2 Development Area is presented on Figure 2.2 and the Future Land Use Table for the Stage 2 Development Area is presented in Appendix B1.

2.5 Stage 3 Development Scenario

The Stage 3 Development Scenario includes an additional 33.5 ha of land within Arthur's DGA (all adjacent to the delineated built-area boundary), with lands zoned as low-density residential (0.8 ha with 12 ERUs based on the residential growth target of 14.82 ERAs per ha), high density residential (1.7 ha with 51 ERUs based on the residential growth target of 14.82 ERUs per ha), future development (1.8 ha with 27 ERUs based on the future residential growth target of 14.82 units per ha) and industrial (29.2 ha with 771 ERUs). The Stage 3 Development Area is presented on Figure 2.3 and the Future Land Use Table for the Stage 3 Development Area is presented in Appendix B1.

2.6 Stage 4 Development Scenario

The Stage 4 Development Scenario includes an additional 66.6 ha of land within Arthur's DGA, with lands zoned as future development (66 ha with 1,742 ERUs) and highway commercial (0.6 ha with 16 ERUs). The ERUs are assumed based on MECP expected maximum day demands for Future Industrial/Commercial/Institutional (ICI) lands. The Stage 4 Development Area is presented on Figure 2.4 and the Future Land Use Table for the Stage 4 Development Area is presented in Appendix B1.

4 WATER DISTRIBUTION NETWORK

4.1 Existing

The distribution network presently services all existing developed areas within Arthur's urban area boundary. The network includes approximately 19.1 km of watermain ranging in size from 50 mm to 600 mm with 1048 services. Type of watermain used to construct the network has varied throughout the years and includes cast iron, ductile iron and PVC. Within the last 10-15 years, any upgrades and extensions to the existing distribution system have been with PVC watermain. The existing distribution network by size and type is illustrated on Figure 4.1. The network includes a trunk main consisting of 250 mm and 300 mm pipe which runs from the Wells 8A/8B along Jones Baseline, Highway 6, George Street and Smith Street, past the Freud Tower to Wells Street and along Wells Street to Well 7B.

The information presented in Tables 2.1 and 2.2 can be used to determine the expected demands of the water system's vertical infrastructure (i.e., wells and water towers); however, the expected demand on horizontal infrastructure (i.e. watermains) and the associated capacities needs to consider the expected demand as a result of complete build out of the development scenarios (i.e. Stages 1 through 4 development scenarios as per the GMP).

The updated WaterCAD model was used to review the fire flow capabilities throughout the water distribution network for the current community and for each of the development scenarios. A minimum residual pressure of 20 psi was used to establish water taking capability, consistent with the normal accepted industry standard for firefighting. This review also assumes that the system is operating during maximum day demand, with the municipal wells running to support the fire effort. Results of this analysis, which reflects the available flow from the mains at a location in the system, rather than a specific hydrant, is provided in the following sections for each of the development scenarios.

Generally, higher fire flows are required in industrial and high-density commercial area (i.e., downtown core), with actual fire flow requirements being site specific. For evaluation of the existing system, the MOE population based recommended figure of 103 l/s was used. Further, the MOE recommends a minimum 30 l/s capability throughout a system that provides firefighting service. Based on these criteria, the Arthur system provides adequate fire flows throughout the network. The lower flow areas are generally restricted to dead-end areas. Fire flow scenarios can be simulated using the model to assess system performance for specific properties/developments or system conditions, as required.

There has been a number of network improvements completed in Arthur since the Master Plan Study was completed in 2012, however a number of watermains remain that are recommended for upgrade and/or replacement. A summary of these watermains is provided in Table 4.1 below.

Table 4.1 – Recommended Water System Replacements

Location	Size (mm)	Length (m)
Edward Street	150	260
Frederick Street West (Edward Street to George Street)	150	140
Walton Street (Clarke Street to Tucker Street)	150	175

The watermain extensions required to service the proposed Development Stages are detailed in

5 WASTEWATER COLLECTION AND TREATMENT

5.1 Existing Infrastructure

The Arthur wastewater system includes a dedicated sanitary sewer/forcemain collection network, two sewage pumping stations (SPS), a wastewater treatment plant (WWTP) and an effluent storage lagoon facility.

5.1.1 Collection System

The network services the entire developed area of Arthur (i.e. within the urban boundary) and currently provides 1032 service connections, according to Township records (2020). The network includes approximately 19.1 km of sewer, ranging in size (diameter) from 150 mm to 450 mm, approximately 4 km of forcemain that is 150 mm to 250 mm in diameter. The type of sanitary sewer pipes varies within the network and includes asbestos cement, concrete and PVC. Upgrades and extensions to the sanitary sewer network within the last 10-15 years have been PVC pipe. The existing sanitary sewer collection system is presented on Figure 5.1 and Figure 5.2.

A computer simulation model (i.e. SewerCAD V8i) of the Arthur sanitary collection system was created as part of the Master Plan and has been updated to support this technical study to estimate peak flows throughout the network and compare them to the hydraulic capacity of the various sewers and forcemain under the various development scenarios and expected population growth.

5.1.2 Treatment Plant

The Arthur WWTP is located at the south end of Preston Street near the Conestogo River. The plan provides tertiary treatment utilizing the extended aeration process. The treatment process components include:

- *Preliminary Treatment:* grit channels, comminutor, manual bar screen.
- *Secondary Treatment:* aeration, secondary clarifier.
- *Tertiary Treatment:* filtration, ultra-violet disinfection.
- *Biosolids Management:* aerobic digestion, biosolids storage.
- *Effluent Pumping Station:* two effluent pumps to transfer treated effluent from the WWTP to the lagoons during the non-discharge period.
- *Discharge:* to the Conestogo River during the discharge period, effluent is stored in the lagoons during the non-discharge period.

The rated average day flow (ADF) capacity of the WWTP is 1,465 m³/day, and discharges to the Conestogo River (River); however, due to assimilative capacity limitations of the River, discharging to the River is restricted to between September 16 and April 30. Between May 1 and September 15, effluent from the WWTP must be pumped to the effluent storage lagoons for holding until discharging to the River is permitted.

A Schedule C Class Environmental Assessment (XCG, 2016) was completed for the WWTP, which recommended the plant and associated Fredrick St. SPS be expanded in a phased approach from the existing rated capacity a 1,465 m³/d to 1,860 m³/d (Phase 1) and 2,300 m³/d (Phase 2). Since WWTP information is available from this EA document, this Technical update will not provide information or comment on this infrastructure in detail.

5.1.3 Sewage Pumping Stations

The collection network for the Arthur wastewater system is divided into three service areas, and is collected by the following trunk sewer and sewage pumping stations:

Wells Street SPS:

The Wells Street SPS pumps via a 1 km - 150 mm diameter PVC/AC forcemain to a manhole at the intersection of Preston and Smith Streets. This SPS receives primarily industrial flows from the industry located in the west side of the town.

Preston Street Trunk Sewer:

Preston Street trunk sewer services Preston Street and the western portion of Domville Street along with the Wells SPS discharge. This area flows by gravity directly into the Arthur WWTP and services a mix of residential and industrial users.

Frederick Street SPS:

The Frederick Street SPS receives the majority of the flows in the community including the central, southern and eastern portions of the system. It pumps directly into the WWTP through a 750 m long, 250 mm diameter forcemain. This SPS services primarily commercial and residential flows.

5.1.4 Reserve Capacity

Triton completed a review of the reserve capacity for the Arthur WWTP for 2020, in accordance with the requirements outlined in the MECP guidelines. The ADF based on flows recorded at the WWTP in calendar years 2017, 2018, and 2019 is 1,400 m³/day, which is in compliance with the Certificate of Approval for the WWTP. The WWTP reserve capacity of 65 m³/day corresponds to an additional 45 equivalent residential units that can be served; however, given that 31 equivalent residential units are committed to Golden Valley Farm's, the reserve capacity corresponds to 14 uncommitted equivalent residential units. Refer to Appendix D1 for the 2020 Reserve Capacity Calculations for the Arthur WWTP.

5.1.5 Per Person Flow Rate

The existing average per person flow rate in Arthur is estimated at 581L./day, as presented in the Arthur 2020 RCC, however this value includes ICI flows which are significant within the community. Our expectation is that as the community continues to grow with residential development, the per person flow rate will begin to decrease to more typical values. Therefore, for future planning purposes, the MECP recommended per person flow rate of 400L/day was used. This rate should be reviewed as part of the annual RCC and adjusted accordingly if actual per person flows decrease significantly. As with the water RCC, it is recommended that any significant future ICI developments be required to apply for sewage treatment allocation to ensure that the RCC reflects future usage requirements.

5.1.6 Future Reserve Capacity

Based on the growth as described in section 2, the information presented in the Arthur Wastewater Treatment Plant Expansion Design Brief, and the calculations shown in Table 5.1.6 below, the proposed plant expansions will have sufficient capacity to treat the sewage until the year 2045, with a minor deficit of 25 ERUs remaining at that time. Further, in order to keep pace with the development, it is expected that the Phase 2 expansion should be implemented by 2030.

Table 5.1.6 – Future Sanitary Reserve Capacity

Year	Population (Capita)	Households (ERU)	ADF (m ³ /day)	Phase 1 Reserve Capacity		Phase 2 Reserve Capacity	
				m ³	ERU	m ³	ERU
Rated Capacity				1,860m ³		2,300m ³	
2020	2,410	949	1400	460	402		
2025	3,351	1,242	1777	83	69		
2030	3,698	1,370	1915	-55		385	317
2035	4,046	1,499	2055	-195		245	202
2040	4,391	1,639	2193	-333		107	89
2045	4,736	1,768	2331	-471		-31	-25

5.2 Stage 1 Development Scenario

The following Developments have been included in the Stage 1 Development scenario.

Table 5.2 – Stage 1 Developments

Map ID	Development Name	Area (ha)	Units (ERU)
R1-1	Eastridge Subdivision	4.5	162
R1-2	Gordon Street Condo	0.6	36
R1-3	Forest View Estates (WN-50)	1.0	50
C1-1	Existing Residential on 6	0.1	3
C1-2	Existing Residential on 109	0.1	3
C1-3	Vacant Land on 6	1.0	26
C1-4	Vacant Land on 6	0.4	11
C1-5	Vacant Land on 6	0.5	13

To service the developments within Stage 1 there are no sanitary sewer extensions or upgrades required. However, the expectation is that in the event that a land area is not serviceable by gravity (i.e. C1-3) a sewage pumping station or low-pressure sanitary system will be installed on site to provide connection to the nearest municipal infrastructure serviced by gravity. The configuration of such infrastructure would need to be reviewed based on the specific development needs.

5.3 Stage 2 Development Scenario

In addition to the Developments included in Stage 1, the following Developments have been included in the Stage 2 modelling.

Table 5.3a – Stage 2 Developments

Map ID	Development Name	Area (ha)	Units (ERU)
R2-1	Burnside Subdivision	9.8	250
R2-2	Cachet Subdivision	5.1	287
C2-1	Smith & Wells	0.3	8
C2-2	Highway 6 Vacant Commercial	5.1	135

Under the Stage 2 development scenario the following sanitary sewers are over capacity as indicated in Table 5.3b.

Table 5.3b – Stage 2 Sanitary Sewers Over Capacity

Label	Start MH	End MH	Street	Length (m)	Diameter (mm)	Slope (%)	Percent Full (%)
CO-21	MH-179	MH-178	Francis Street	76.5	200	0.36	164.2
CO-22	MH-178	MH-177	Francis Street	45.1	200	0.41	159.5
CO-23	MH-177	MH-176	Francis Street	36	200	0.42	157.7
CO-24	MH-176	MH-175	Francis Street	22.9	200	0.53	140.7
CO-50	MH-78	MH-77	Intersection of George & Fredrick	20.9	300	0.13	115.2

5.4 Stage 3 Development Scenario

Table 5.4a – Stage 3 Developments

Map ID	Development Name	Area (ha)	Units (ERU)
R3-1	West Anderson	0.8	12
R3-2	East Anderson	1.8	27
R3-3	Vacant Residential	1.7	25
M3-1	North Arthur Industrial Lands	29.2	771

Under the Stage 3 conditions the following sanitary sewers are over capacity as indicated in Table 5.4b. Note, this assumes full development of lands within Stage 1, Stage 2 and Stage 3.

Table 5.4b – Stage 3 Sanitary Sewers Over Capacity

Label	Start MH	End MH	Street	Length (m)	Diameter (mm)	Slope (%)	Percent Full (%)
CO-21	MH-179	MH-178	Francis Street	76.5	200	0.36	164.2
CO-22	MH-178	MH-177	Francis Street	45.1	200	0.41	159.5
CO-16	MH-190	MH-189	Francis Street	87.5	200	0.32	159
CO-23	MH-177	MH-176	Francis Street	36	200	0.42	157.7
CO-24	MH-176	MH-175	Francis Street	22.9	200	0.53	140.7
CO-15	MH-191	MH-190	Francis Street	37.8	200	0.11	115.3
CO-50	MH-78	MH-77	Intersection of George & Fredrick	20.9	300	0.13	121.3

5.5 Stage 4 Development Scenario

Table 5.5a – Stage 4 Developments

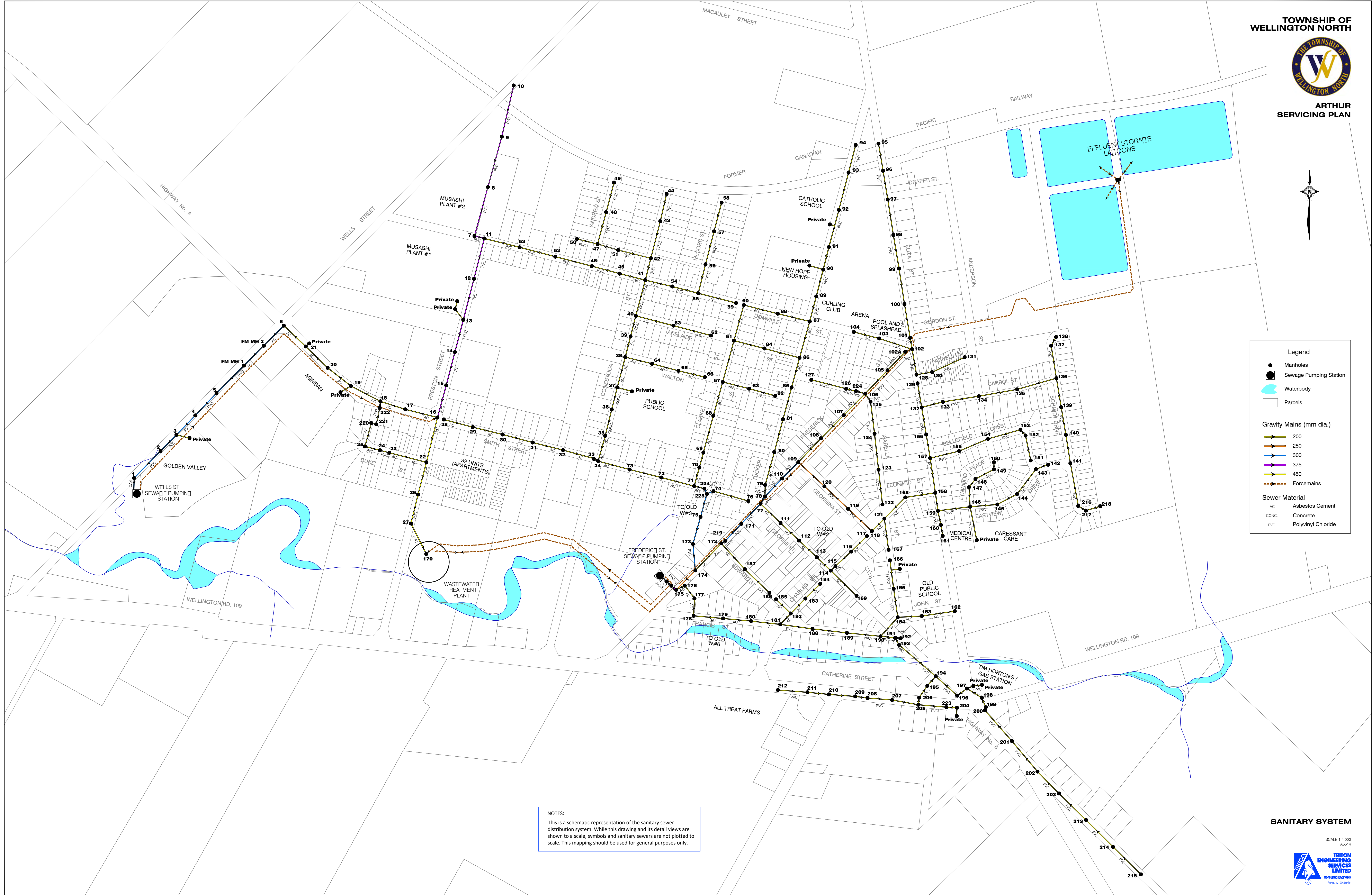
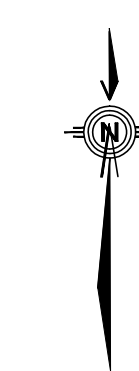
Map ID	Development Name	Area (ha)	Units (ERU)
FD4-1	North of Lagoons	12.1	319
FD4-2	East of Eliza Street	36.6	966
FD4-3	West of Eliza	16.4	433

Under the Stage 4 conditions the following sanitary sewers are over capacity as indicated in Table 5.5b. Note, this assumes full development of lands within Stage 1, Stage 2, Stage 3 and Stage 4.

Table 5.5b – Stage 4 Sanitary Sewers Over Capacity

Label	Start MH	End MH	Street	Length (m)	Diameter (mm)	Slope (%)	Percent Full (%)
CO-21	MH-179	MH-178	Francis Street	76.5	200	0.36	164.2
CO-22	MH-178	MH-177	Francis Street	45.1	200	0.41	159.5
CO-16	MH-190	MH-189	Francis Street	87.5	200	0.32	159
CO-23	MH-177	MH-176	Francis Street	36	200	0.42	157.7
CO-24	MH-176	MH-175	Francis Street	22.9	200	0.53	140.7
CO-15	MH-191	MH-190	Francis Street	37.8	200	0.11	115.3
CO-50	MH-78	MH-77	Intersection of George & Fredrick	20.9	300	0.13	144.8

The expectation is that FD4-2 and FD4-3 will be serviced by the Preston Street Trunk.



Legend

- Manholes
- Sewage Pumping Station
- Waterbody
- Parcels

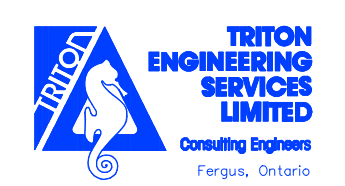
Gravity Mains (mm dia.)

- 200
- 250
- 300
- 375
- 450
- Force mains

Sewer Material

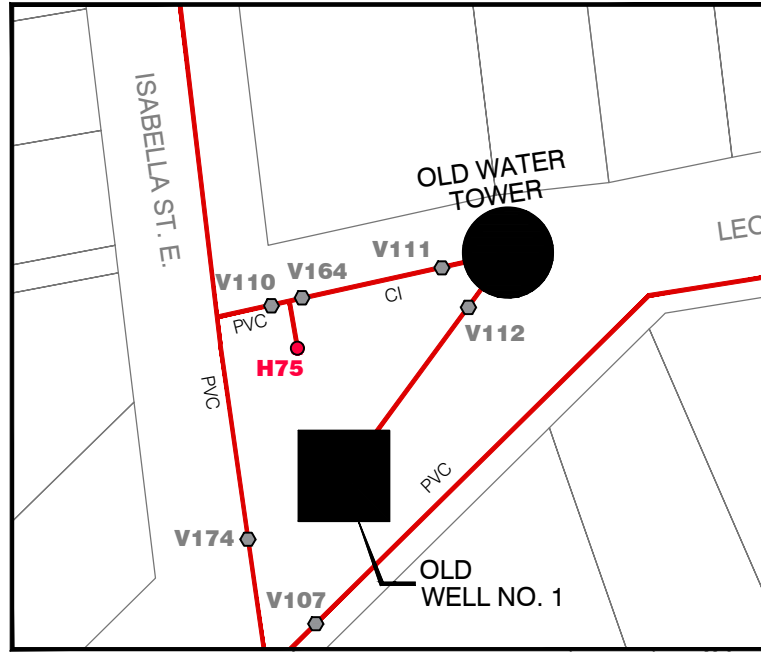
- AC Asbestos Cement
- CONC Concrete
- PVC Polyvinyl Chloride

NOTES:
 This is a schematic representation of the sanitary sewer distribution system. While this drawing and its detail views are shown to a scale, symbols and sanitary sewers are not plotted to scale. This mapping should be used for general purposes only.

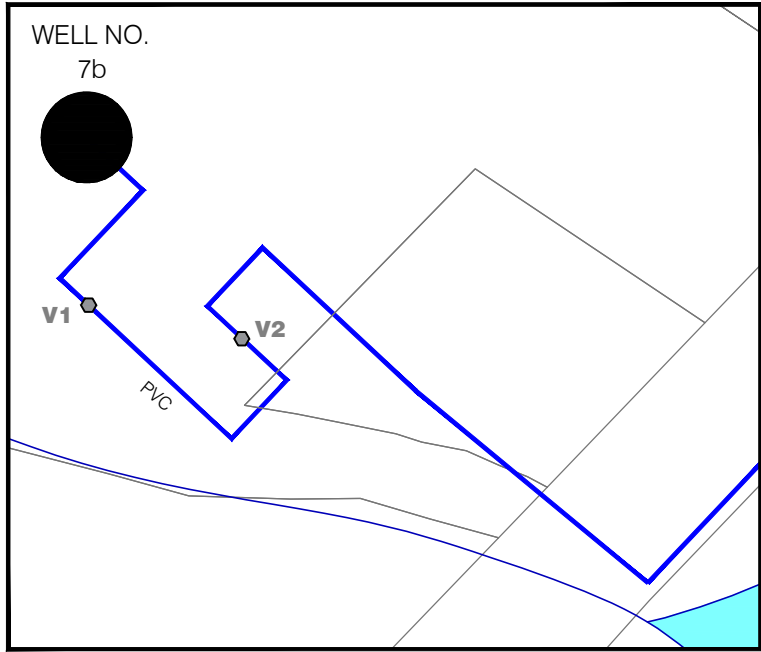




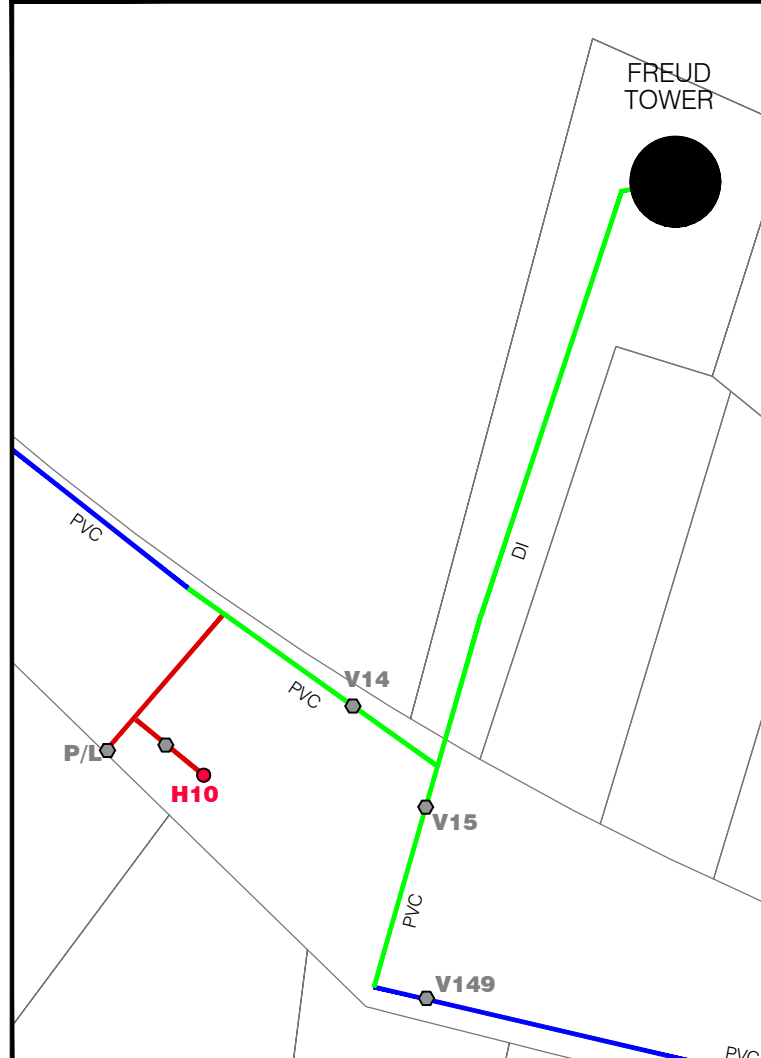
INSET-1



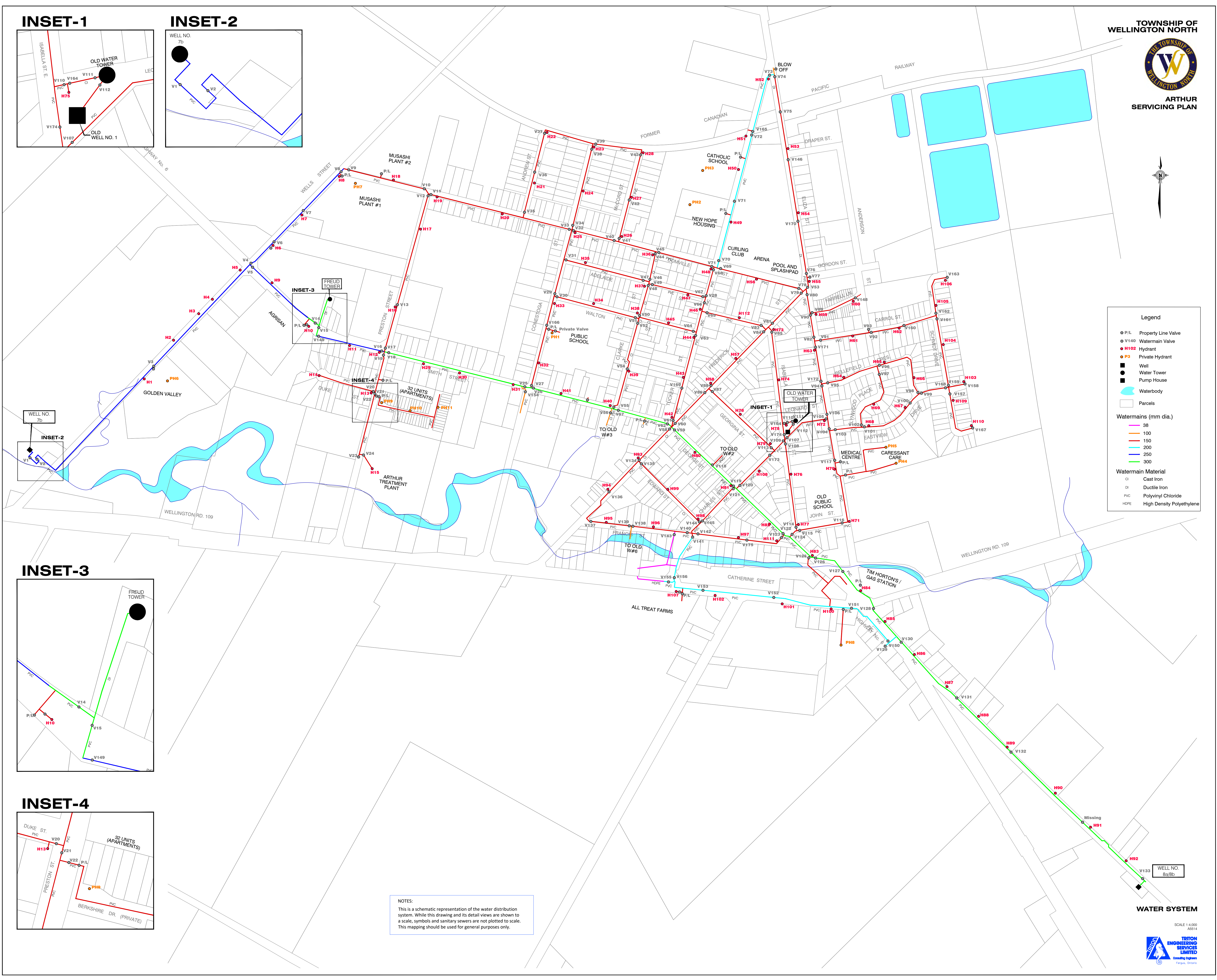
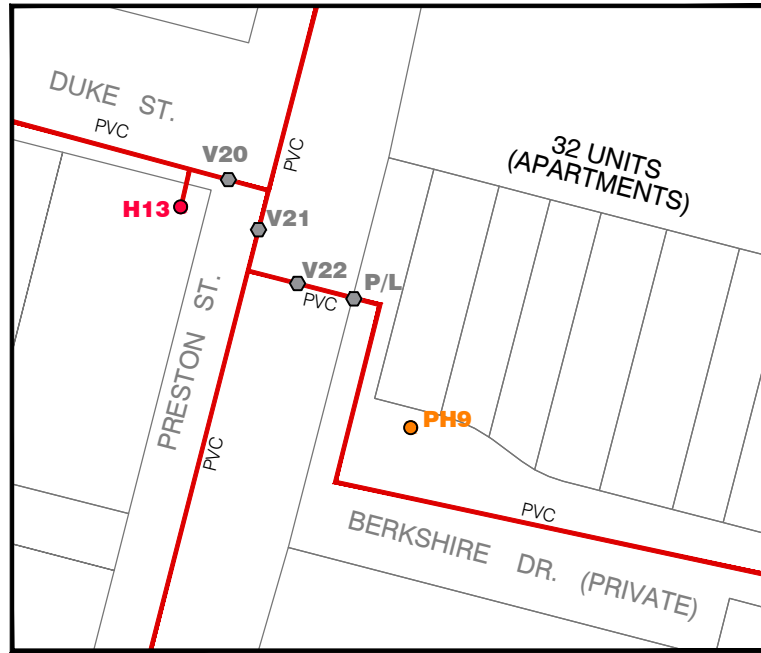
INSET-2



INSET-3



INSET-4



Legend

- P/L Property Line Valve
- V140 Watermain Valve
- H102 Hydrant
- P3 Private Hydrant
- Well
- Water Tower
- Pump House
- Waterbody
- Parcels

Watermains (mm dia.)

- 38
- 100
- 150
- 200
- 250
- 300

Watermain Material

- CI Cast Iron
- DI Ductile Iron
- PVC Polyvinyl Chloride
- HDPE High Density Polyethylene

NOTES:
 This is a schematic representation of the water distribution system. While this drawing and its detail views are shown to a scale, symbols and sanitary sewers are not plotted to scale. This mapping should be used for general purposes only.





ARTHUR WATER AND SANITARY SERVICING STUDY



- LEGEND
- EXISTING SEWER TO REMAIN
 - UPGRADES
 - EXTENSIONS

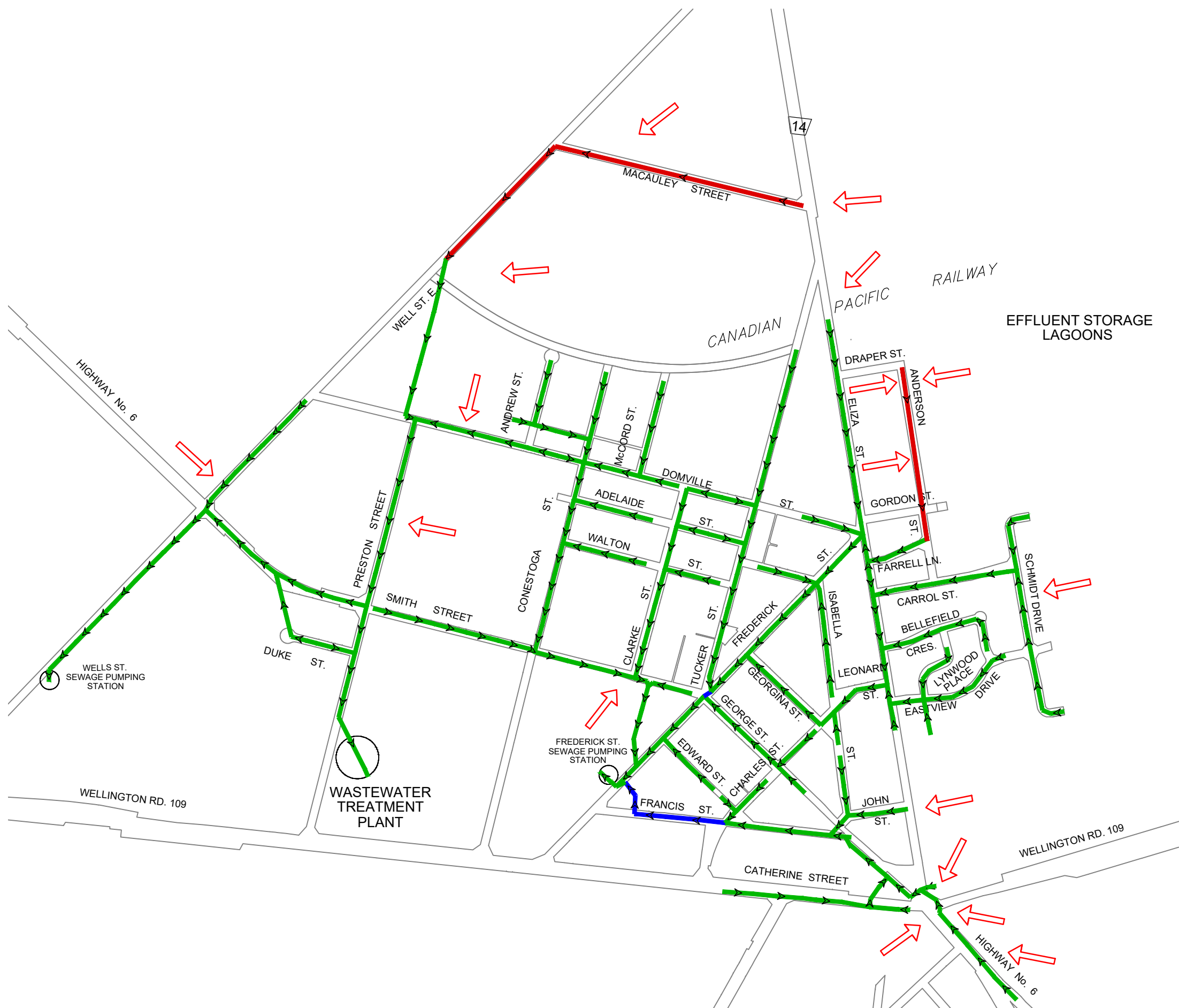


FIGURE 5.8

RECOMMENDED SANITARY UPGRADES AND EXTENSIONS

SCALE 1:10,000
A5514



TOWNSHIP OF WELLINGTON NORTH



ARTHUR WATER AND SANITARY SERVICING STUDY



- LEGEND:
- APPLICATION UNDER REVIEW
 - INDUSTRIAL
 - HIGHWAY COMMERCIAL
 - CENTRAL BUSINESS DISTRICT
 - INTENSIFICATION CORRIDOR
 - FUTURE RESIDENTIAL COMMERCIAL
 - STAGE 4 BOUNDARY
 - POLICY AREA
 - R2-1 9.8 ha DEVELOPMENT ID ASSUMED DEVELOPABLE LAND

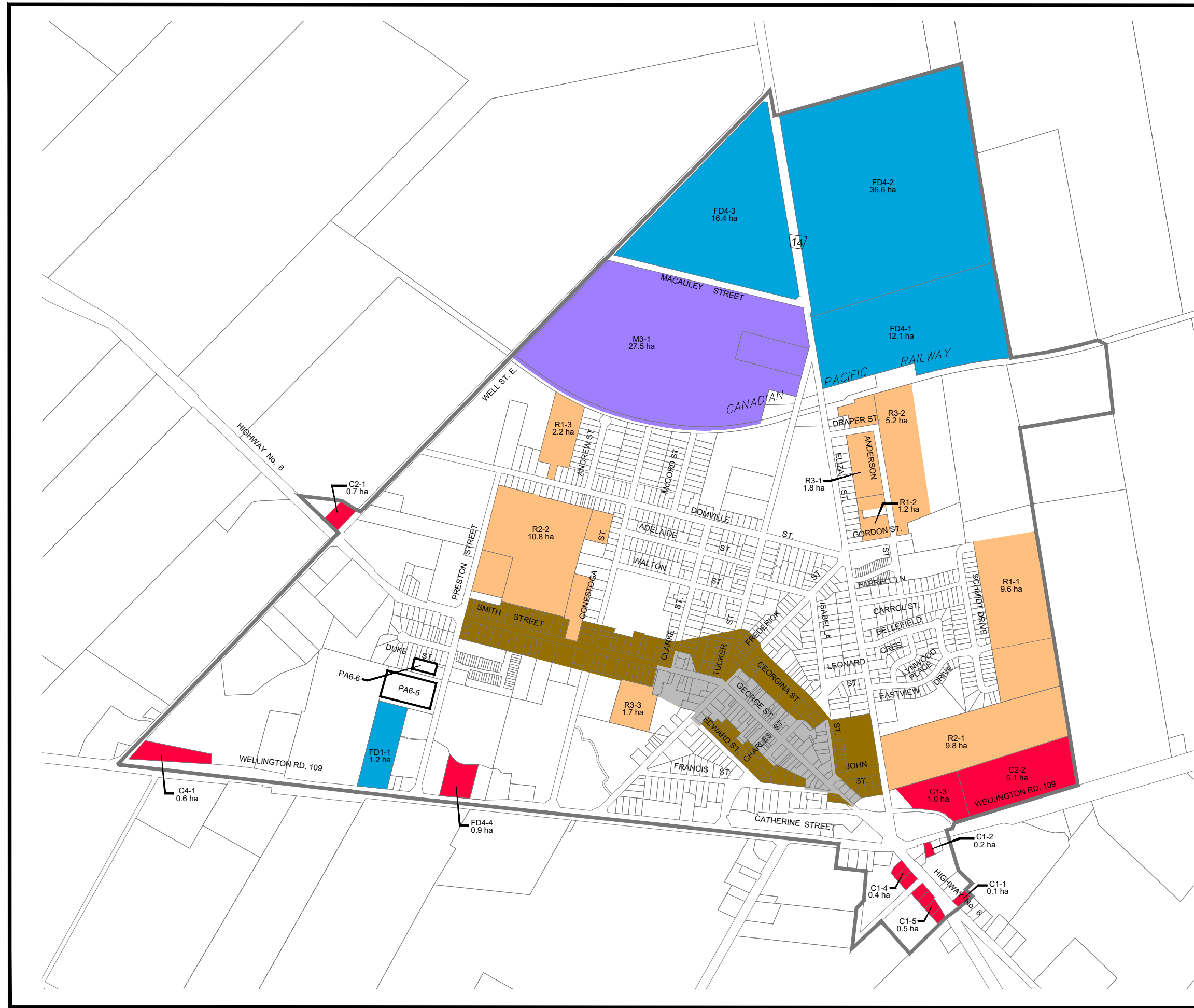


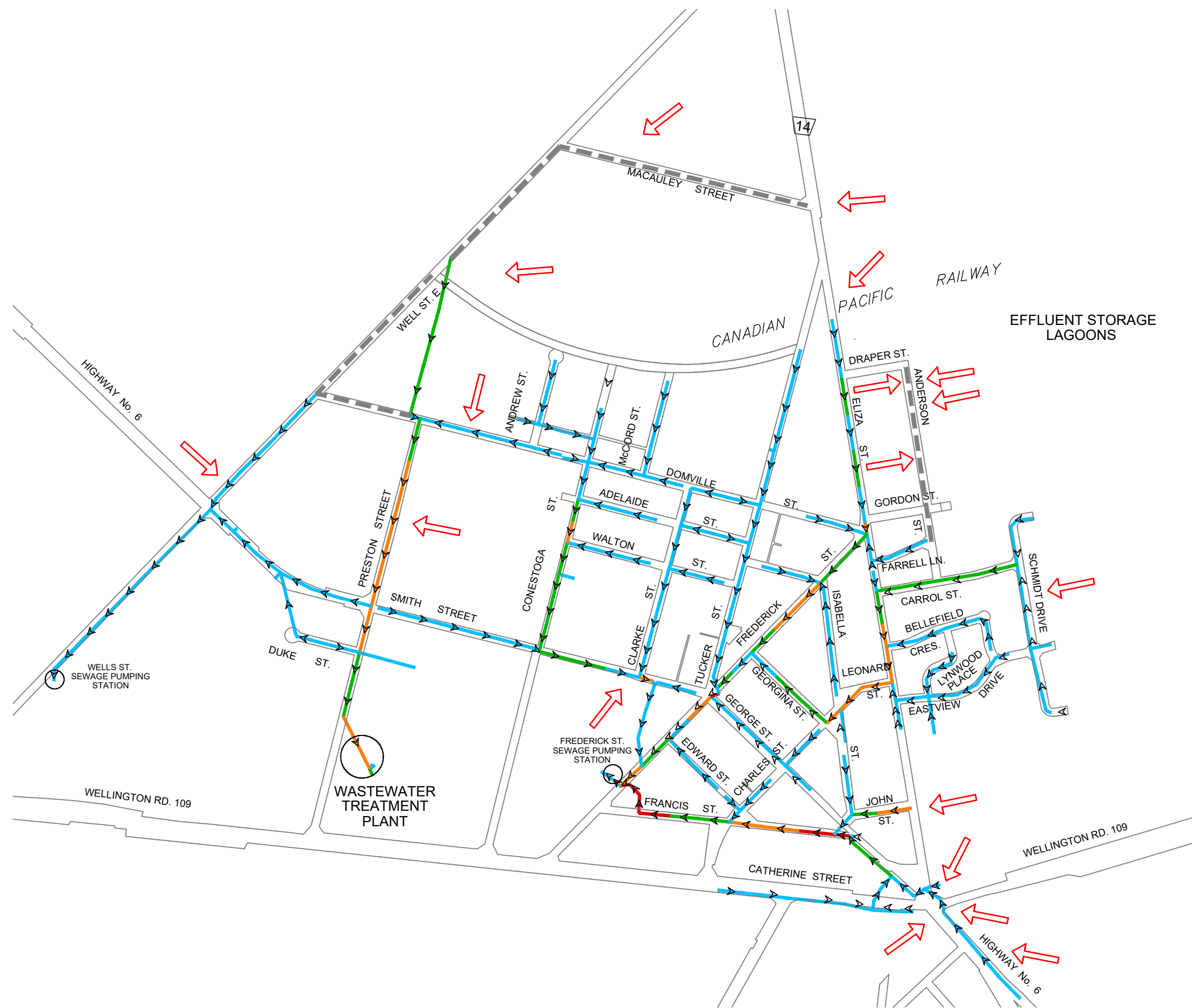
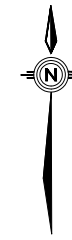
FIGURE 4.4
STAGE 4 DEVELOPMENT SCENARIO

SCALE 1:1,500
A5514





ARTHUR WATER AND SANITARY SERVICING STUDY



- PEAF FLOW / CAPACITY
- 50
 - 75
 - 100
 - 100
 - FUTURE SEWER EXTENSIONS
 - ↖ FUTURE DEVELOPMENT CONTRIBUTION

FIGURE 5.7
PEAF SEWAGE FLOWS [STAGE 4]

SCALE 1:10,000
A5514

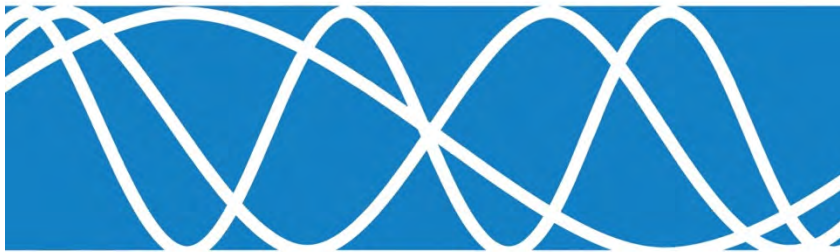
Appendix B4 Acoustic Background Information



Land Use Compatibility Study (Noise)

Proposed Residential Development Eliza Street Arthur, Ontario

March 3, 2025
HGC Project #: 02000939



Prepared for:

Tribute/Sorbara Arthur Holdings Inc.
Unit 1, 1815 Ironstone Manor,
Pickering, Ontario

Table 9: Predicted Impulsive Source Sound Levels at the Proposed Development [dBAI]

Receptor	Sound Levels		Criteria		Meet Criteria (Y/N)
	Façade (Day/Eve/Night)	OLA (Day/Eve)	Façade (Day/Eve/Night)	OLA (Day/Eve)	
R1	55 / 55 / 55	56	50 / 50 / 45	50 / 45	N
R2	50 / 50 / 50	50	50 / 50 / 45	50 / 45	N
R3	45 / 45 / 45	44	50 / 50 / 45	50 / 45	Y
R4	47 / 47 / 47	49	50 / 50 / 45	50 / 45	N

These results indicate that sound levels under a worst-case operational scenario, noise from the surrounding facilities may exceed the criteria at the closest residential receptors due to activities at the RMC, contractor’s yard and trucking facility to the northeast of the site. Recommendations are provided in the following section.

5.5 Recommendations

In order to reduce sound levels to within the applicable Class 2 criteria for the entire site, primarily from noise from the proposed concrete batch plant and contractor’s yard, significant mitigation would be required at the development site.

The MECP does not accept mechanical ventilation as a mitigation measure for stationary noise sources since the criteria apply outside the residential windows. On-site mitigation is thereby generally implemented through the provision of property line noise barriers, and in this case such barriers would be ineffective at upstairs windows due to the height of the receptors, height of the sources and the frontage onto the roadways.

Two options are presented. The recommended Option 1 requests a Class 4 designation from the municipality for the subject lands. Higher sound level criteria apply for lands designated as Class 4. If Class 4 is not granted, Option 2 provides mitigation measures to meet Class 2 limits.

5.5.1 Option 1 Mitigation – Class 4

Request the Town to designate the subject lands as a Class 4 area. This designation provides relaxed (higher) daytime and nighttime sound level limits from that otherwise permitted in an urban area, for both indoor and outdoor areas. A Class 4 Area permits receptor-based noise control measures (noise walls, specific construction techniques and materials, etc.) to be used within a proposed new sensitive land use within the vicinity of industrial uses. Class 4 Areas require formal recognition of the classification by the land use planning authority.

A Class 4 designation for the proposed site is recommended since the mitigation measures required to achieve Class 1 criteria would generally be considered onerous for a stacked townhouse units. Class 4 would allow 60 dBA during the day and 55 dBA at night at the facades and 55 dBA in rear yards.

A Class 4 area is defined in NPC-300 as:

- is an area intended for development with new noise sensitive land use(s) that are not yet built;
- is in proximity to existing, lawfully established stationary source(s); and
- has formal confirmation from the land use planning authority with the Class 4 area classification which is determined during the land use planning process.

This designation provides relaxed (higher) daytime and nighttime sound level limits from that otherwise permitted in an urban area, for both indoor and outdoor areas. The sound level limits for a Class 4 area is 60 dBA during the day in an OLA and at the façade, and 55 dBA during the night at the façade. A Class 4 Area permits receptor-based noise control measures (noise walls, specific construction techniques and materials, etc) to be used within a proposed new sensitive land use within the vicinity of an industrial use. Class 4 Areas require formal recognition of the classification by the land use planning authority.

With a Class 4 designation, the following mitigation is required:

1. The buildings are required to include air conditioning.
2. An additional clause is required to be included in the property and tenancy agreements and offers of purchase and sale for all dwelling units with a Class 4 designation.

Type F:

Purchasers/tenants are advised that sound levels due to the adjacent facility are required to comply with sound level limits that are protective of indoor areas and are based on the assumption that windows and exterior doors are closed. This dwelling unit has been supplied with a ventilation/air conditioning system which will allow windows and exterior doors to remain closed.

3. Additionally, upgraded building and glazing constructions are recommended for all dwellings with a Class 4 designation, for example windows with a minimum rating of STC-33, for all windows into sensitive spaces to further protect the interior spaces with exposure to the industrial uses to the south and northeast with a Class 4 designation.

All of the predicted sound levels would be within the Class 4 sound level limits at most of the building facades if it is granted. A 2.5 m high noise barrier will be required at the northeast corner adjacent to Ivan Armstrong Trucking facility to protect the dwellings to closest to the truck yard as shown on Figure 9.

5.5.2 Option 2 Mitigation

If a Class 4 designation is not granted by the municipality, the preliminary mitigation measures to meet Class 2 criteria are described here for Option 2, and are as follows.

4. A 4.5m to 6.0m high noise barrier at the northeast boundary, adjacent to Ivan Armstrong Trucking as shown on Figure 10.
5. A 2.5 m high noise wall on top of the proposed parkland berm, near the future RMC facility.
6. A 5.5 m high noise barrier along the park lands, east of Eliza Street.

7. For dwelling facades with remaining sound level excesses, they could be designed such that there are no windows to noise sensitive spaces. Facades requiring mitigation are indicated on Figure 10. Note that dwelling locations will affect the number of facades with excesses and design consideration can be incorporated to reduce the number of impacted facades. Further input can be provided as design progresses.

These measures to meet Class 2 limits are generally considered to be relatively onerous in the context of residential townhouse and single detached construction.

6 IMPACT OF DEVELOPMENT ON ITSELF & ENVIRONMENT

As part of the development, a sanitary pumping station and a well pumphouse are proposed, noted as Blocks 67 and 70. There are two existing residences near Block 70.

Preliminary design for these blocks by DLW Engineering Services were reviewed. It is understood that there will be emergency backup generators located at both blocks. The testing of the emergency generators are subjected to the requirements of NPC-300 and the criteria are 5 dBA above the applicable limits. The limits do not apply during emergency use. Typically, a noise enclosure for the generator will provide adequate noise attenuation. When detailed design of the pumping station and well pumphouse is available, the noise sources associated with these two blocks shall be reviewed and confirmed that there are no offsite noise impact at the proposed and existing residential uses.

7 RECOMMENDATIONS

The results of the study indicate that the proposed residential development is feasible on this site with respect to noise. We have the following recommendations.

Traffic Noise

1. A Forced air ventilation with ductwork sized for the future installation of central air conditioning by the occupant is required for the dwelling units adjacent to Eliza Street as shown on Figure 4.
2. Warning clauses should be included in the property and tenancy agreements and offers of purchase and sale to inform the future owners/residents of the road traffic noise impact.

Stationary Noise

3. Option 1: Mitigation Measures with a Class 4 designation is recommended.
 - a. Class 4 designation for the subject lands.
 - b. Air conditioning is required for all buildings.
 - c. Brick exterior façade constructions and upgraded glazing is required for specific facades with direct exposure to the nearby industries.
 - d. A 2.5 m high noise barrier at the northeast corner, adjacent to Ivan Armstrong Trucking facility, refer to Figure 9.
 - e. A warning clause should be included in the property and tenancy agreements and offers of purchase and sale for the dwelling units to inform the future/occupants that the lot has been designated as Class 4.



4. Option 2: Mitigation Measures to achieve Class 1 Criteria if Class 4 designation is not granted.
 - a. A 4.5m to 6.0m high noise barrier at the northeast boundary, adjacent to Ivan Armstrong Trucking shown on Figure 10.
 - b. A 2.5 m high noise wall on top of the proposed parkland berm, near the future RMC facility.
 - c. A 5.5 m high noise barrier along the park lands, east of Eliza Street.
 - d. Design the buildings with no windows into sensitive spaces for facades with exposure to neighbouring facilities. Refer to Figure 10.
 - e. Warning clauses should be included in the property and tenancy agreements and offers of purchase and sale to inform the future owners/residents of the presence of the nearby industrial uses.

These measures to meet Class 2 limits are generally considered to be relatively onerous in the context of residential townhouse construction.

5. A detailed noise study shall be conducted when siting, floor plans and building elevations are available for the dwellings proposed to be designated Class 4, and/or requiring mitigation, the drawings should be reviewed to refine acoustic mitigation requirements.

The reader is referred to the previous sections of the report where these recommendations are discussed in more detail.

LEGEND

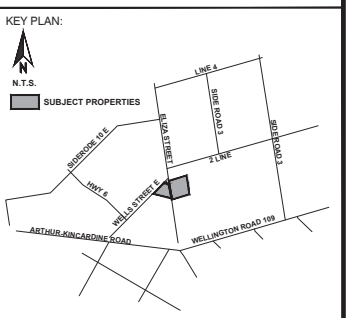
- 36' AND 40' SINGLES
- 20' FREEHOLD TH
- 25' SEMIS
- PARK
- STORMWATER MANAGEMENT POND
- NHS
- ROADS
- MEANDER BELT LIMIT
- EROSION ACCESS ALLOWANCE
- EROSION HAZARD ALLOWANCE



Schedule of Land Use			
Description	Lot / Block No.	Residential Units	Area (ha)
Single Detached Residential	5, 21-24, 26, 27, 32-61	454-504	19.96
Semi-Detached	7, 15-20, 25, 31	112-113	3.21
Street Townhouse	1-4, 6, 8-14, 28-30	249	6.05
Net Developable Total		815-866	29.22
Park	62-64		3.62
Stormwater Management Pond	65, 66		4.38
Well	67		0.27
Natural Heritage Systems	68, 69		5.90
Sanitary Pumping Station	70		0.05
Servicing Block	71		0.02
Right of Way	STREET A-R		11.88
Total Site Area			55.34

DRAFT PLAN OF SUBDIVISION

LEGAL DESCRIPTION:
PART OF PARK LOTS 1 AND 2
NORTH OF MACAULEY STREET
CROWN SURVEY
AND
PART LOT 1 CONCESSION 2
WEST LUTHER AS IN RON74408
TOWNSHIP OF WELLINGTON NORTH
COUNTY OF WELLINGTON



REQUIRED INFORMATION:
AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT R.S.O. 1990.

(a) SEE PLAN (b) SEE PLAN (c) SEE PLAN (d) SEE PLAN (e) SEE PLAN (f) SEE PLAN (g) SEE PLAN (h) SEE PLAN (i) SEE PLAN (j) SEE PLAN (k) SEE PLAN (l) SEE PLAN (m) SEE PLAN (n) SEE PLAN (o) SEE PLAN (p) SEE PLAN (q) SEE PLAN (r) SEE PLAN (s) SEE PLAN (t) SEE PLAN (u) SEE PLAN (v) SEE PLAN (w) SEE PLAN (x) SEE PLAN (y) SEE PLAN (z) SEE PLAN (aa) SEE PLAN (ab) SEE PLAN (ac) SEE PLAN (ad) SEE PLAN (ae) SEE PLAN (af) SEE PLAN (ag) SEE PLAN (ah) SEE PLAN (ai) SEE PLAN (aj) SEE PLAN (ak) SEE PLAN (al) SEE PLAN (am) SEE PLAN (an) SEE PLAN (ao) SEE PLAN (ap) SEE PLAN (aq) SEE PLAN (ar) SEE PLAN (as) SEE PLAN (at) SEE PLAN (au) SEE PLAN (av) SEE PLAN (aw) SEE PLAN (ax) SEE PLAN (ay) SEE PLAN (az) SEE PLAN (ba) SEE PLAN (bb) SEE PLAN (bc) SEE PLAN (bd) SEE PLAN (be) SEE PLAN (bf) SEE PLAN (bg) SEE PLAN (bh) SEE PLAN (bi) SEE PLAN (bj) SEE PLAN (bk) SEE PLAN (bl) SEE PLAN (bm) SEE PLAN (bn) SEE PLAN (bo) SEE PLAN (bp) SEE PLAN (bq) SEE PLAN (br) SEE PLAN (bs) SEE PLAN (bt) SEE PLAN (bu) SEE PLAN (bv) 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SURVEYORS CERTIFICATE:
I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATE AND CORRECTLY SHOWN IN ACCORDANCE WITH A PLAN OF SURVEY PREPARED BY J.D. BARNES LIMITED

RAYMOND J. SIBTHORP O.L.S.

OWNERS CERTIFICATE:
I HEREBY AUTHORIZE THE BIGLIERI GROUP LTD. TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE COUNTY OF WELLINGTON

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

ARTHUR, WELLINGTON NORTH DEVELOPMENT

APPROVAL STAMP:

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

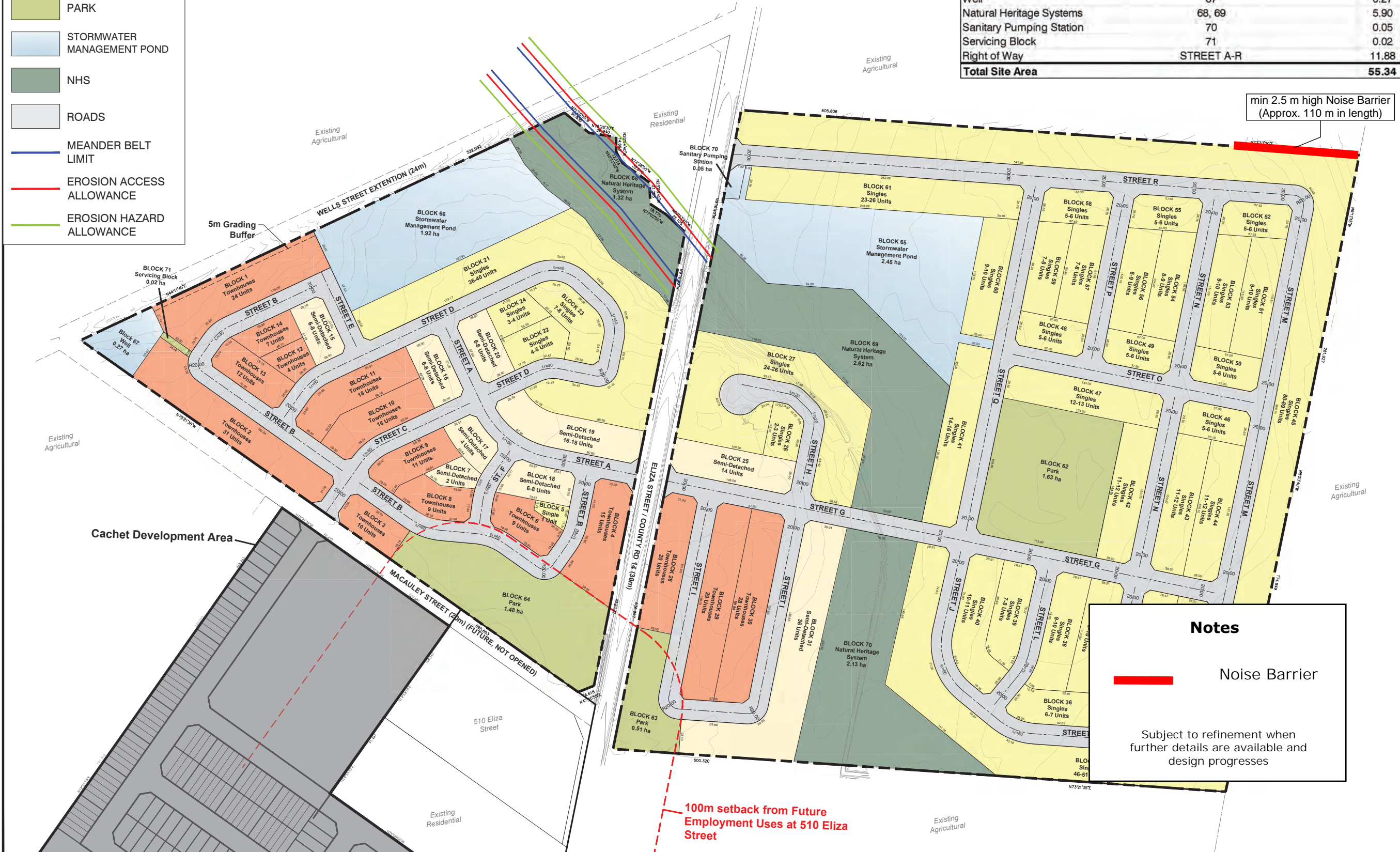
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PROJECT No.: 22853
DATE: January 14, 2025
SCALE: 1:1750
DRAFTED BY: EC CHECKED BY: MP
DRAWING No.: **DP-01**

BIGLIERI GROUP

2472 Kingston Road, Toronto
21 King Street W., Suite 1102, Hamilton
(416) 693-8155
biglierigroup.com



min 2.5 m high Noise Barrier (Approx. 110 m in length)

Notes

Noise Barrier

Subject to refinement when further details are available and design progresses

100m setback from Future Employment Uses at 510 Eliza Street

Figure 9: Plan Showing Noise Barrier Requirements to meet Class 4 Limits

LEGEND

- 36' AND 40' SINGLES
- 20' FREEHOLD TH
- 25' SEMIS
- PARK
- STORMWATER MANAGEMENT POND
- NHS
- ROADS
- MEANDER BELT LIMIT
- EROSION ACCESS ALLOWANCE
- EROSION HAZARD ALLOWANCE

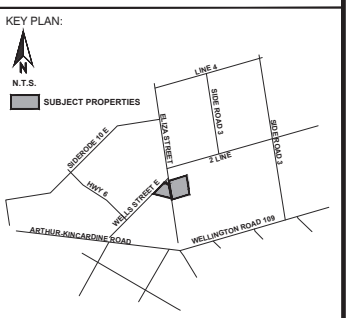


Schedule of Land Use

Description	Lot / Block No.	Residential Units	Area (ha)
Single Detached Residential	5, 21-24, 26, 27, 32-61	454-504	19.96
Semi-Detached	7, 15-20, 25, 31	112-113	3.21
Street Townhouse	1-4, 6, 8-14, 28-30	249	6.05
Net Developable Total		815-866	29.22
Park	62-64		3.62
Stormwater Management Pond	65, 66		4.38
Well	67		0.27
Natural Heritage Systems	68, 69		5.90
Sanitary Pumping Station	70		0.05
Servicing Block	71		0.02
Right of Way	STREET A-R		11.88
Total Site Area			55.34

DRAFT PLAN OF SUBDIVISION

LEGAL DESCRIPTION:
PART OF PARK LOTS 1 AND 2 NORTH OF MACAULEY STREET CROWN SURVEY AND PART LOT 1 CONCESSION 2 WEST LUTHER AS IN RON74408 TOWNSHIP OF WELLINGTON NORTH COUNTY OF WELLINGTON



REQUIRED INFORMATION:
AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT R.S.O. 1990.

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SURVEYORS CERTIFICATE:
I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATE AND CORRECTLY SHOWN IN ACCORDANCE WITH A PLAN OF SURVEY PREPARED BY J.D. BARNES LIMITED

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TRIBUTE/SORBARA ARTHUR HOLDINGS INC.
DATE

ARTHUR, WELLINGTON NORTH DEVELOPMENT

APPROVAL STAMP:

TRIBUTE/SORBARA ARTHUR HOLDINGS INC.

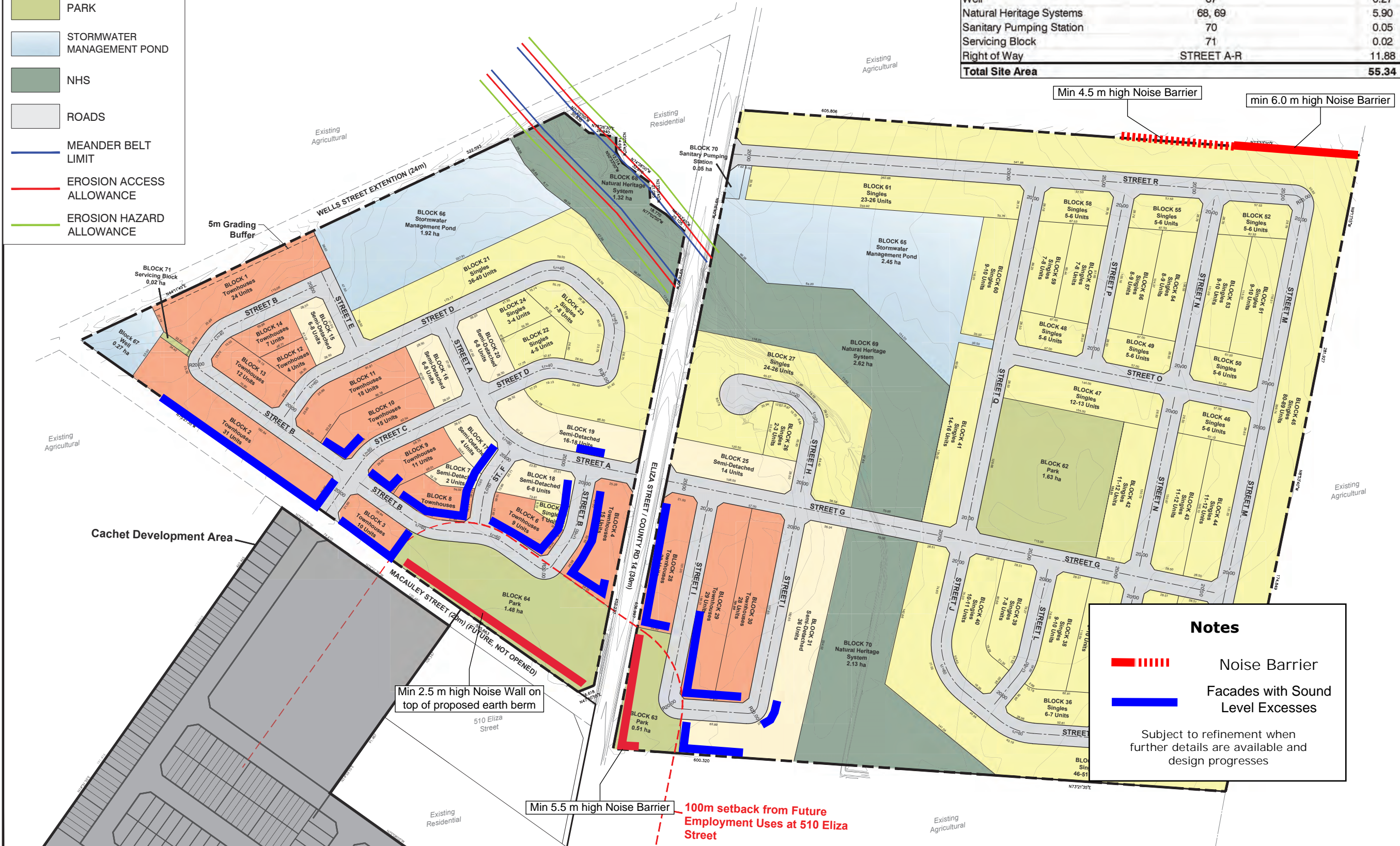
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No.	Description	Date	Int.
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PROJECT No.: 22853
DATE: January 14, 2025
SCALE: 1:1750
DRAFTED BY: EC CHECKED BY: MP
DRAWING No.: **DP-01**

BIGLIERI GROUP

2472 Kingston Road, Toronto
21 King Street W., Suite 1002, Hamilton
(416) 899-8155
biglierigroup.com



Notes

- Noise Barrier
- Facades with Sound Level Excesses

Subject to refinement when further details are available and design progresses

Figure 10: Plan Showing Facades with Stationary Noise Excesses Above Class 2 Limits and Noise Barrier Requirements

Appendix C Floodplain Modification – Hydraulic Modelling



**Appendix C1 Existing Floodplain Modelling Letter prepared by
SCS Consulting Group Ltd. dated October 4, 2024**



Ms. Jessica Conroy
Grand River Conservation Authority
400 Clyde Road, PO Box 729
Cambridge, ON N1R 5W6

Dear Ms. Conroy:

**Re: Existing Floodplain Modelling
665 Eliza Street, Arthur, ON
Tribute/Sorbara Arthur Holdings Inc.**

The purpose of this letter is to provide a summary of the hydrologic & hydraulic analyses and associated modelling to date for GRCA’s review and comment related to establishing the existing floodplain limits within the proposed development located at 665 Eliza Street in Arthur, prior to making a formal Functional Servicing Report submission in support of rezoning. The purpose of this letter is to ensure that GRCA is satisfied with the existing floodlines and the existing riparian floodplain storage calculation approach to be used when investigating proposed floodplain and channel routing modifications.

1. Background Information

The proposed development is located at 665 Eliza Street in Arthur. The proposed development is separated into two parcels, east and west of Eliza Street and generally located south-east of Wells Street and north of Domville Street. Refer to the site location plan below.



Figure 1.0 – Site Location Plan

The proposed development has a creek running generally southeast-northwest through the site and crosses under Eliza Street via an existing 2750 mm dia. CSP culvert and then crosses under Wells Street via an existing 2000 mm dia. CSP culvert from where it outlets to Farley Creek, which ultimately connects into the Conestogo Creek. Farley Creek and the creek through the development do not have engineered regulatory (Regional Storm) floodplain mapping, however, estimated Regional Storm floodplain mapping is available. The creek through the property is identified as being regulated by GRCA based on their regulated mapping and has an estimated Regional Storm floodline associated with the creek. The total drainage area to the creek is approximately 199.26 ha. Therefore, A detailed floodplain analysis is required to set the limits of development. SCS Consulting Group has prepared existing conditions hydrology and hydraulic models to define the existing Regional Storm floodplain.

2. Hydrology Modelling

A Visual Otthymo (VO6.2) model was prepared to determine the Regional Storm peak flow through the creek.

2.1. Drainage Area

The total upstream drainage area to the connection to Farley Creek was determined utilizing detailed topographic information for the proposed development as well as GeoHub LIDAR external topographic information. The total drainage area at the connection to Farley Creek is 199.26 ha.

2.2. Hydrology Parameters

The uplands methodology was used to calculate the time to peak (refer to the existing floodplain drainage area figure, **Figure 2.1** and **Attachment A**). The external soil types were determined using the online AgSoils mapping and the internal soils were confirmed with the draft geotechnical report prepared by Gemtec dated September 2024.

2.3. Hydrology Model

The Regional Storm event (Hurricane Hazel) produced a Regional flow of approximately 13.55 m³/s for the total drainage area at the connection to Farley Creek. This Regional flow was utilized as a single flow node to assess the floodline throughout the development and the Eliza Street and Wells Street crossings. In the future, the hydrology model may be evaluated to discretize the drainage area upstream of Eliza Street and introduce a flow node change. Refer to the hydrology model within **Attachment A**.

3. Hydraulic Modelling

To determine the existing Regional Storm floodplain, a hydraulic model (HEC-RAS version 6.4.1) was created.

3.1. Topographic Information

Topographic survey was utilized to develop the HEC-RAS cross-sections. J.D. Barnes completed detailed topographic survey dated August 2024, which was utilized to develop the cross-sections within the confines of the proposed development. As mentioned above, Geohub LIDAR topographic survey information was utilized for areas external to the site. The detailed topographic survey information was completed in CGVD 28-1978 which is consistent with the GRCA floodplain mapping criteria. The Geohub LIDAR topographic information was completed in CGVD 2013.

The detailed topographic survey and the LIDAR utilize different vertical datums. When comparing the detailed topographic survey and LIDAR along the western limit (Wells Street), they have similar elevations and the discrepancy is within approximately 0.10 m. When comparing the detailed topographic survey and LIDAR along the eastern limits, the discrepancy is within approximately 0.3 m. When comparing the detailed topographic survey and LIDAR along the northern limits, the discrepancy is within approximately 0.5 m. When comparing the detailed topographic survey and LIDAR along the southern limits, the discrepancy is within approximately 0.8 m. It is noted that while the discrepancy is the largest at the southern limits, the topography within the southern limits is relatively flat. The southern limit of the property is utilized for determining the overall drainage area to the tributary and the discrepancy would not affect the delineation of the drainage boundary.

3.2. Limits of Floodplain Assessment

The limits of the floodplain assessment were determined based on the updated definition of a watercourse per Ontario Regulation 41/24. The downstream limit of the creek connects into Farley Creek therefore, the western limits of the assessment will terminate at Farley Creek. The headwater portion of the creek terminates at the south/southeast limits of the property as per the updated definition of a watercourse; therefore, the floodplain assessment will begin at the southern property limit.

3.3. Downstream Boundary Condition

Farley Creek has an estimated Regional Storm floodline per the GRCA regulatory floodline information available to date. This estimated Regional Storm floodline elevation at the junction with the creek is

approximately 459 m and was utilized as the downstream water surface boundary condition. The upstream boundary condition was set to critical depth.

3.4. Manning's n Values

Manning's n values for the creek and overbanks were delineated based on site aerial imagery. The creek manning's n value utilized was 0.035 which is consistent with the GRCA floodplain mapping criteria. As the existing site is cultivated, the manning's n values for the overbanks within the cultivated lands utilized was 0.05 which falls within the standard range within the GRCA floodplain mapping criteria. For the manning's n value for the overbanks within the developed areas, a "low intensity" value of 0.08 was utilized which is consistent with the GRCA floodplain mapping criteria.

3.5. Culvert Crossings

As noted above, there are two (2) culvert crossings of the creek, Eliza Street and Wells Street. The culvert sizes and inverts are based on the detailed J.D. Barnes topographical data dated August 2024. The ground surface elevations at the upstream and downstream cross-sections of the crossings were manually aligned to the culvert inverts to avoid blockage as the culvert inverts are coincident with the creek inverts.

The ineffective flow areas were set at the midpoint between the culvert obvert and road low point elevation at the downstream side of each crossing; otherwise set at the road low point elevation. The stationing for the ineffective flow areas was set to 0.3 m from the edge of culvert(s). The contraction and expansion coefficients of 0.3 and 0.5, respectively were utilized for the two cross-sections upstream of the crossings and the one cross-section downstream of the crossings.

The resulting existing conditions Regional Storm floodplain is illustrated on **Figure 2.2**.

4. Existing Regional Storm Floodplain Spill

As shown on **Figure 2.2**, there is an existing backwater condition created by the existing Eliza Street culvert. Due to the backwater condition and the existing site topography, a portion of the Regional Storm floodplain spills north, through the adjacent property to Farley Creek. The floodplain modelling has not taken into account the spill condition. With the proposed creek modifications, it is intended for the spill condition to be eliminated.

5. Riparian Storage Methodology

Based on the wide-shallow nature of the existing floodplain due to existing topography and constrained road crossing infrastructure, a creek realignment is proposed to facilitate the proposed development, which requires floodplain modifications. Proposed creek modifications and alignment are still under review, however could include culvert upgrades. The existing riparian storage volume will need to be maintained in the proposed creek configuration.

As shown on **Figure 2.1**, the floodline upstream of Eliza Street is in a backwater condition as a result of the existing Eliza Street culvert. Therefore, to determine the required existing riparian storage volume that is to be maintained with the proposed creek realignment, the required existing riparian storage volume will be based on an existing floodplain assessment without manmade structures causing a backwater effect, to reflect a natural riparian storage condition. A separate floodplain model will be prepared for this assessment and approach moving forward which will follow in the formal re-zoning application. In addition to maintaining the existing riparian storage volume requirements, an analysis will be completed to assess the conveyance capabilities of the proposed creek modifications which may or may not include culvert enhancements.

6. Conclusion

The above information outlines the methods used to determine the existing Regional Storm floodline and the approach that will be used to calculate the existing required riparian storage volume with potential proposed creek modifications. We request that GRCA provide their feedback on the hydrology and hydraulic models and associated Regional Storm floodline mapping, and the approach to calculate the existing required riparian storage.

Re: Existing Floodplain Modelling
665 Eliza Street, Arthur, ON
Tribute/Sorbara Arthur Holdings Inc.

File #: 2504
October 4, 2024
Page 6 of 6

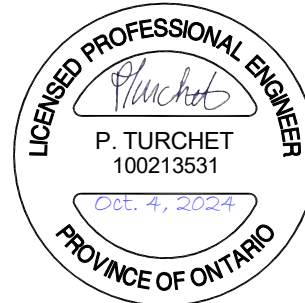
Please contact the undersigned if you have any questions or require any additional information.

Sincerely,

SCS Consulting Group Ltd.



Logan Wintemute
lwintemute@scsconsultinggroup.com



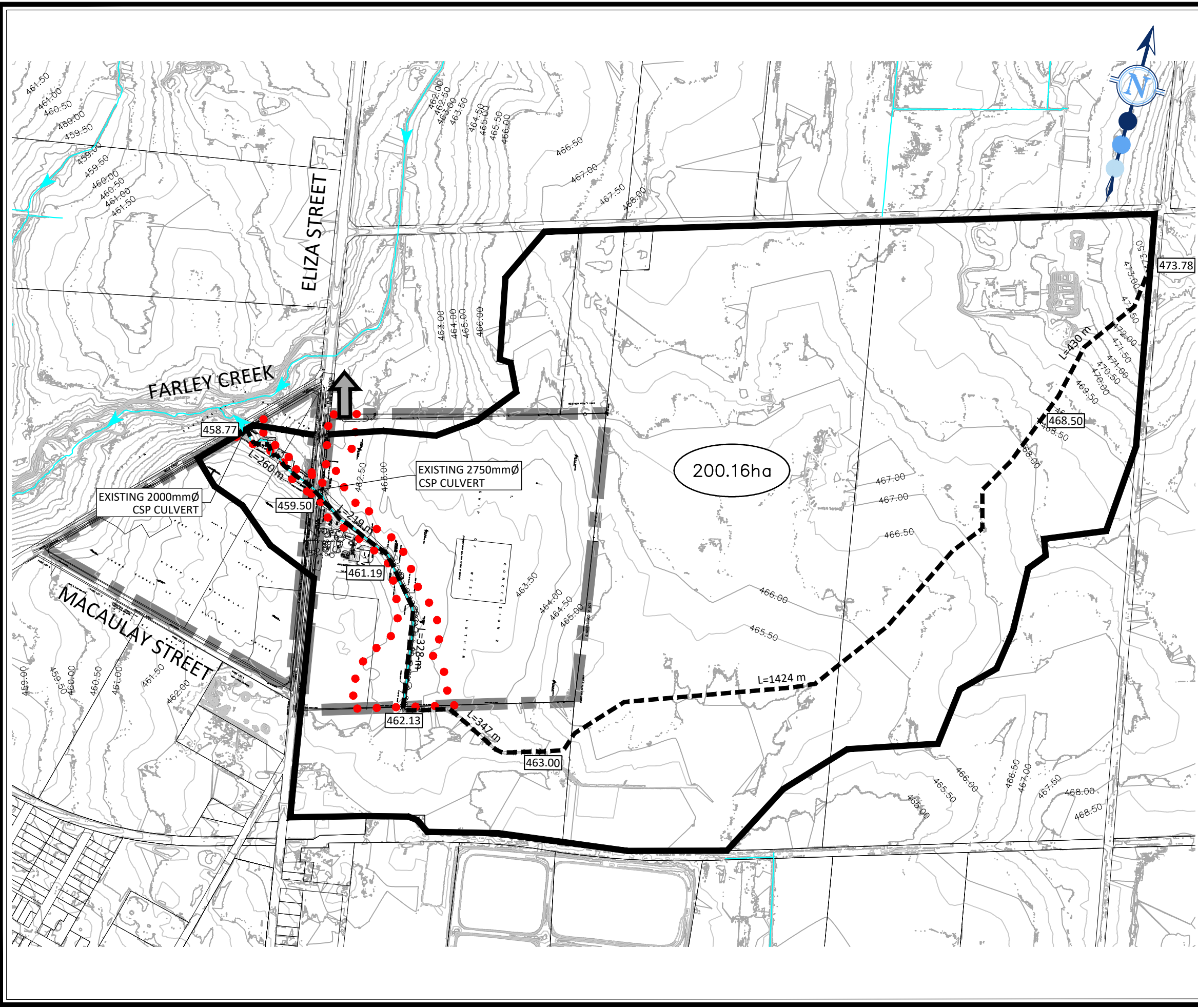
Paige Turchet, P. Eng.
pturchet@scsconsultinggroup.com

Attachments




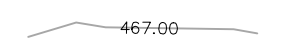

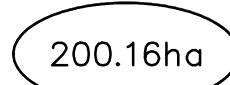
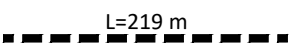
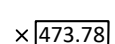
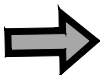
Figure 2.1 – Existing Floodplain Drainage Area
Figure 2.2 – Existing Floodplain Mapping
Attachment A – Hydrology Model
Attachment B – Hydraulic Model

c. Ms. Susan Zuccherro, Tribute Communities

P:\2504 665 Eliza St, Arthur - Tribute\Correspondence\Letters\2024 10(Oct) 04 - Existing Floodplain Model Letter.docx



LEGEND:

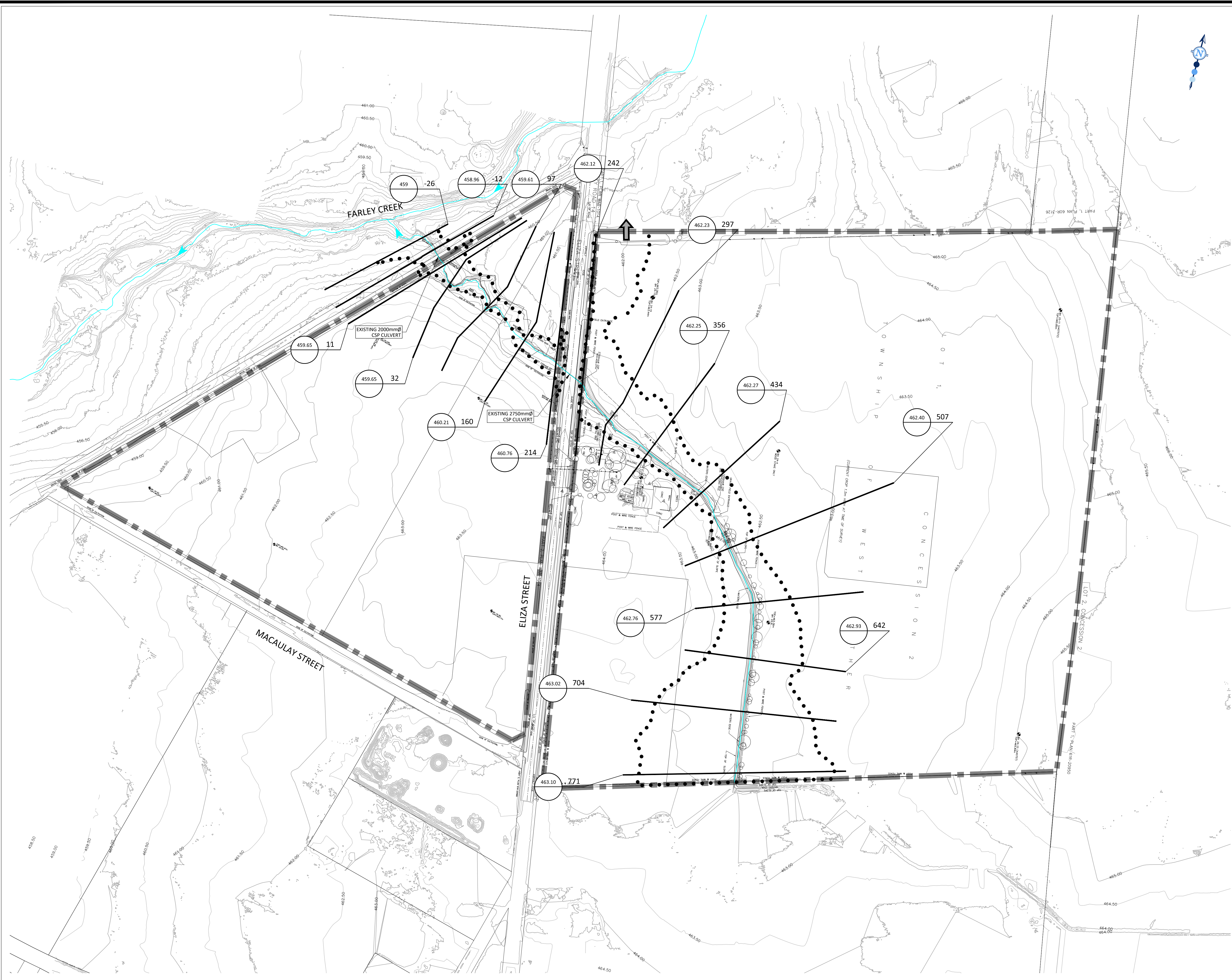
-  LIMIT OF PARTICIPATING PROPERTY
-  EXISTING DRAINAGE BOUNDARY
-  EXISTING REGULATORY FLOODPLAIN
-  EXISTING CONTOUR AND ELEVATION
-  WATERCOURSE
-  200.16ha DRAINAGE AREA
-  L=219 m TIME OF PEAK FLOW PATH AND LENGTH
-  x 473.78 TIME TO PEAK ELEVATION
-  EXISTING REGULATORY FLOODPLAIN SPILL

 30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

665 ELIZA ST, ARTHUR - TRIBUTE

EXISTING DRAINAGE PLAN

DESIGNED BY:	L.W.	CHECKED BY:	P.A.T.
SCALE:	1:8500	DATE:	OCTOBER 2024
PROJECT No:	2504	FIGURE No:	2.1



- LEGEND:**
- LIMIT OF PARTICIPATING PROPERTY
 - EXISTING REGULATORY FLOOD ELEVATION
 - HYDRAULIC SECTION ID
 - FLOODLINE ELEVATION LEADER
 - HEC CROSS-SECTION
 - EXISTING REGULATORY FLOODPLAIN
 - WATERCOURSE
 - EXISTING CONTOUR AND ELEVATION
 - EXISTING REGULATORY FLOODPLAIN SPILL

*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

SGS consulting group Ltd

30 CENTURIAN DRIVE, SUITE 100
 MARKHAM, ONTARIO L3R 8B8
 TEL: (905) 475-1900
 FAX: (905) 475-8335

665 ELIZA ST, ARTHUR - TRIBUTE

EXISTING FLOODPLAIN

DESIGNED BY: L.W.	CHECKED BY: P.A.T.
SCALE: 1:2000	DATE: SEPTEMBER 2024
PROJECT No: 2504	FIGURE No: 2.2

Appendix C2 Floodplain Hydrology Model



Digital Report and Modelling Files

The following FileSafe Cloud link includes:

665 Eliza Street, Arthur Visual Otthymo 6.2 Floodplain Modelling – Functional Servicing and Stormwater Management Report (February 2025)

Contents:

<https://filesafecloud.scsconsultinggroup.com/url/hrkry2zcdvrpapwr>



Proposed Conditions VO Parameter Summary

NASHYD

Number	301	316	317	318	401	402	403
Description							
DT(min)	1	1	1	1	1	1	1
Area (ha)	0.27	1.97	2.62	1.32	64.15	93.38	0.63
CN*	73.0	72.0	72.0	82.0	88.0	86.0	86.0
CN (AMC III)	88.0	86.0	86.0	90.0	93.0	92.0	92.0
IA(mm)	5.0	8.0	8.0	9.0	7.1	7.9	8.0
TP Method	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands
TP (hr)	0.03	0.36	0.36	0.14	2.48	2.24	0.04

STANDHYD

Number	302	303	304	305	306	307	308	309	310	311	312	313	314	315
Description														
DT(min)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Area (ha)	1.92	0.5	11.12	1.5	0.32	0.24	0.76	4.83	0.35	0.5	2.45	0.6	1.38	22.70
TIMP ²	0.50	0.78	0.60	0.63	0.62	0.67	0.52	0.65	0.48	0.67	0.50	0.67	0.67	0.60
XIMP ^{1,2}	0.50	0.78	0.60	0.62	0.61	0.67	0.51	0.65	0.47	0.67	0.50	0.67	0.67	0.59
CN*	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0
CN (AMC III)	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0
IA(mm)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
SLPP(%)	2	2	2	2	2	2	2	2	2	2	2	2	2	2
LGPM(m)	40	40	40	40	40	40	40	40	40	40	40	40	40	40
MNP	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
DPSI (mm)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
SLPI(%)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
LGI(m)	113.14	57.74	272.27	100.00	46.19	40.00	71.18	179.44	48.30	57.74	127.80	63.25	95.92	389.01
MNI	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013

¹Note that where there is NO directly connected area (ie: roof runoff to grassed areas), the hydrology program does not accept XIMP=0%, therefore, XIMP = 1% has been used

²Note that where there is NO pervious area, the hydrology program does not accept TIMP and XIMP=100%, therefore, TIMP and XIMP = 99% has been use

Total Area = 129.4 ha

Proposed Conditions CN Calculations

Site Soils: (per Ag Maps)

Soil Type

Perth Silt Loam

Hydrologic Soil Group

C

TABLE OF CURVE NUMBERS (CN's)**										
Land Use	Hydrologic Soil Type								Manning's 'n'	Source
	A	AB	B	BC	C	CD	D			
Meadow "Good"	30	44	58	64.5	71	74.5	78	0.40	MTO	
Woodlot "Fair"	36	48	60	66.5	73	76	79	0.40	MTO	
Gravel	76	80.5	85	87	89	90	91	0.30	USDA	
Lawns "Good"	39	50	61	67.5	74	77	80	0.25	USDA	
Pasture/Range	58	61.5	65	70.5	76	78.5	81	0.17	MTO	
Crop	66	70	74	78	82	84	86	0.13	MTO	
Fallow (Bare)	77	82	86	89	91	93	94	0.05	MTO	
Low Density Residences	57	64.5	72	76.5	81	83.5	86	0.25	USDA	
Streets, paved	98	98	98	98	98	98	98	0.01	USDA	

1. MTO Drainage Manual (1997), Design Chart 1.09-Soil/Land Use Curve Numbers
2. USDA (1986), Urban Hydrology for Small Watersheds, Table 2.2-Runoff Curve Numbers for Urban Areas

HYDROLOGIC SOIL TYPE (%) - Proposed Conditions								
Catchment	Hydrologic Soil Type							TOTAL
	A	AB	B	BC	C	CD	D	
301					100			100
316					100			100
317					100			100
318					100			100
401					100			100
402					100			100
403					100			100
302					100			100
303					100			100
304					100			100
305					100			100
306					100			100
307					100			100
308					100			100
309					100			100
310					100			100
311					100			100
312					100			100
313					100			100
314					100			100
315					100			100

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
301				100.0						100.0
316	100.0									100.0
317	100.0									100.0
318		49.2				50.8				100.0
401		2.1	8.9	3.8		80.6		4.6		100.0
402		1.4	1.1			97.1		0.4		100.0
403						100.0				100.0
302				100.0						100.0
303				100.0						100.0
304				100.0						100.0
305				100.0						100.0
306				100.0						100.0
307				100.0						100.0
308				100.0						100.0
309				100.0						100.0
310				100.0						100.0
311				100.0						100.0
312				100.0						100.0
313				100.0						100.0
314				100.0						100.0
315				100.0						100.0

Note: Where STANDHYD command used (shaded), impervious fraction is not considered in CN determination, since %Imp directly input in STANDHYD command



Proposed Conditions CN Calculations

Arthur Wellington North Development
Project Number: 2504
Date: February 2025
Designer Initials: J.H.

CURVE NUMBER (CN) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
301	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
316	71.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	71
317	71.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	71
318	0.0	35.9	0.0	0.0	0.0	41.7	0.0	0.0	0.0	78
401	0.0	1.5	7.9	2.8	0.0	66.1	0.0	0.0	4.5	83
402	0.0	1.0	1.0	0.0	0.0	79.6	0.0	0.0	0.4	82
403	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82
302	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
303	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
304	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
305	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
306	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
307	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
308	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
309	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
310	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
311	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
312	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
313	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
314	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
315	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74

** AMC II assumed



Proposed Conditions CN Calculations

Arthur Wellington North Development
 Project Number: 2504
 Date: February 2025
 Designer Initials: J.H.

Input Values									
Subcatchment:	301		316	317	318	401	402	403	
CN (AMC II):	74		71	71	78	83	82	82	
CN (AMC III) =	88		86	86	90	93	92	92	
100 Year Precipitation, P =	71.5	mm	71.5	71.5	71.5	71.5	71.5	71.5	

Proposed Conditions IA Calculations

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
301				100.0						100.0
316	100.0									100.0
317	100.0									100.0
318		49.2				50.8				100.0
401		2.1	8.9	3.8		80.6			4.6	100.0
402		1.4	1.1			97.1			0.4	100.0
403						100.0				100.0
302				100.0						100.0
303				100.0						100.0
304				100.0						100.0
305				100.0						100.0
306				100.0						100.0
307				100.0						100.0
308				100.0						100.0
309				100.0						100.0
310				100.0						100.0
311				100.0						100.0
312				100.0						100.0
313				100.0						100.0
314				100.0						100.0
315				100.0						100.0

IA VALUES (mm) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
IA (mm)	8	10	2	5	8	8	3	2	2	
301				5.0						5.0
316	8.0									8.0
317	8.0									8.0
318		4.9				4.1				9.0
401		0.2	0.2	0.2		6.5			0.1	7.1
402		0.1	0.0			7.8			0.0	7.9
403						8.0				8.0
302				5.0						5.0
303				5.0						5.0
304				5.0						5.0
305				5.0						5.0
306				5.0						5.0
307				5.0						5.0
308				5.0						5.0
309				5.0						5.0
310				5.0						5.0
311				5.0						5.0
312				5.0						5.0
313				5.0						5.0
314				5.0						5.0
315				5.0						5.0

* IA values based on TRCA guidelines

Proposed Conditions Time to Peak Calculations

Uplands Method:

Catchment ID	High Elevation	Low Elevation	Length (m)	Slope (%)	Land Cover Type	Velocity (m/s)	Time of Concentration (s)	Time of Concentration (hr)	Time to Peak (hr)
301a	460.19	458.80	60	2.32	Pasture	0.33	180.6	0.05	0.03
301									0.03
316a	462.13	459.48	607	0.44	Waterway	0.32	1921.5	0.53	0.36
316									0.36
317a	462.13	459.48	607	0.44	Waterway	0.32	1921.5	0.53	0.36
317									0.36
318a	461.81	459.63	59	3.69	Pasture	0.42	140.3	0.04	0.03
318b	459.63	458.77	199	0.43	Waterway	0.31	633.1	0.18	0.12
318									0.14
401a	474.50	471.95	147	1.73	Cultivated Straight Row	0.37	399.8	0.11	0.07
401b	471.95	469.98	317	0.62	Cultivated Straight Row	0.22	1436.3	0.40	0.27
401c	469.98	466.16	912	0.42	Cultivated Straight Row	0.18	5027.7	1.40	0.94
401d	466.16	465.25	671	0.14	Cultivated Straight Row	0.10	6480.3	1.80	1.21
401									2.48
402a	473.78	468.50	430	1.23	Cultivated Straight Row	0.31	1388.7	0.39	0.26
402b	468.50	463.00	1424	0.39	Cultivated Straight Row	0.17	8173.2	2.27	1.52
402c	463.00	462.13	347	0.25	Cultivated Straight Row	0.14	2469.0	0.69	0.46
402									2.24
403a	461.97	460.31	86	1.93	Cultivated Straight Row	0.39	221.8	0.06	0.04
403									0.04

Proposed Conditions Percent Impervious Calculations

			StandHyd IDs														
			302	303	304	305	306	307	308	309	310	311	312	313	314		315
Catchment Area (ha)			1.92	0.5	11.12	1.50	0.32	0.24	0.76	4.83	0.35	0.5	2.45	0.6	1.38	22.70	
Land Use Areas	Timp	Ximp	Land Use Areas													Total	
Parks	17%	13%			1.16	0.23	0.08		0.28		0.13					1.63	3.51
SWM Pond	50%	50%	1.92										2.45				4.37
Single Houses	67%	67%			2.37			0.24	0.20	0.80		0.50		0.60	0.75	14.65	
20m ROW	54%	54%			3.32	0.35				1.87					6.42	11.96	
Semi-detached	67%	67%			1.63					0.73	0.22				0.63		3.21
Townhouses	78%	78%		0.50	2.64	0.92	0.24		0.28	1.43						6.01	
Total Land Use =			1.92	0.50	11.12	1.50	0.32	0.24	0.76	4.83	0.35	0.50	2.45	0.60	1.38	22.70	49.17
Timp =			50%	78%	60%	63%	62%	67%	52%	65%	48%	67%	50%	67%	67%	60%	33%
Ximp =			50%	78%	60%	62%	61%	67%	51%	65%	47%	67%	50%	67%	67%	59%	32%

Appendix C3 Downstream Boundary Condition





Conestoga River - Arthur Cross Section Map

Legend

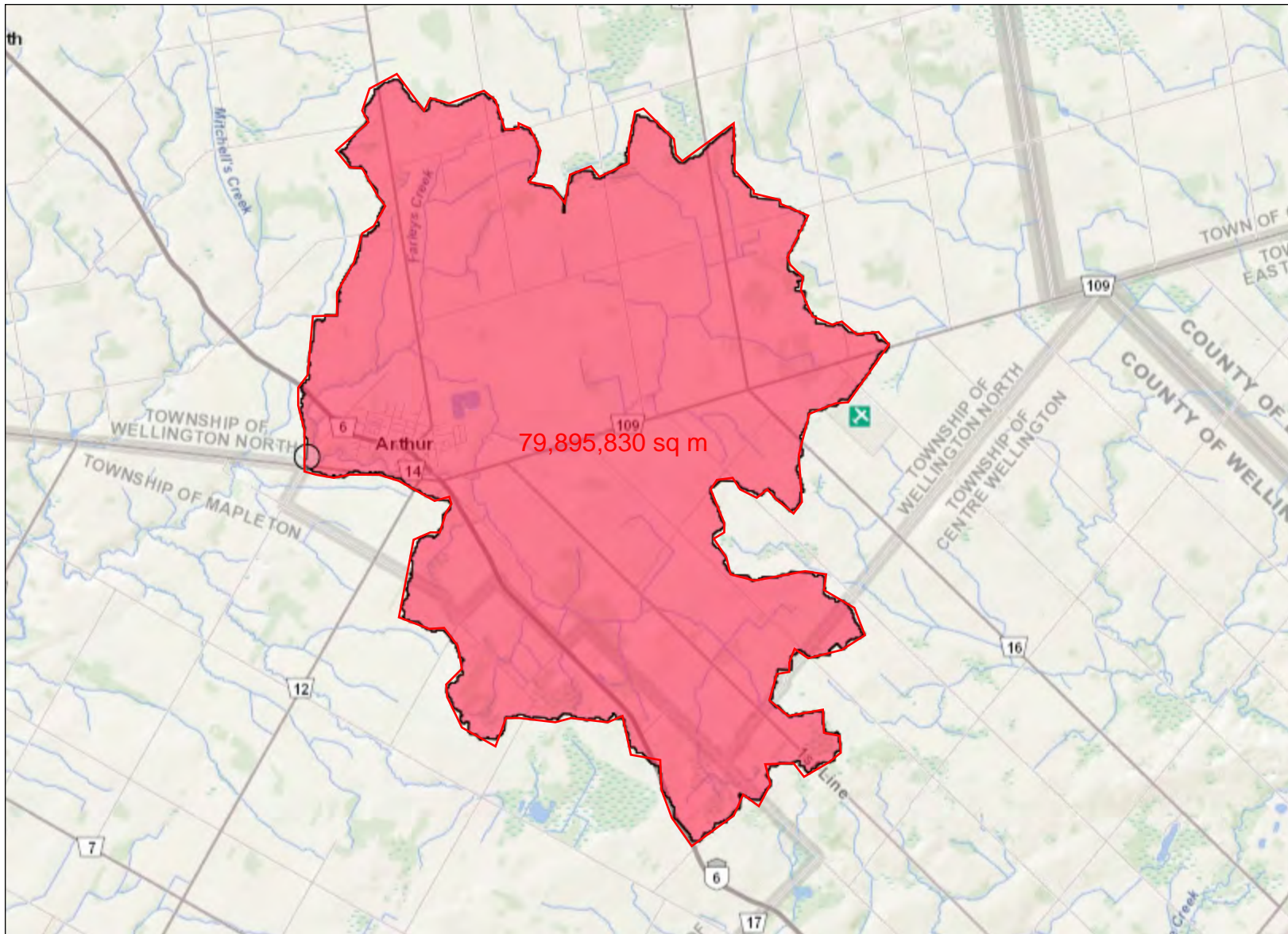
- Regulation Limit (GRCA)
- Floodplain (GRCA)**
 - Engineered
 - Estimated
 - Approximate
- Floodplain - Special Policy Area (GRCA)
- Slope Erosion (GRCA)**
 - Steep
 - Oversteep
 - Toe
- Slope Valley (GRCA)**
 - Steep
 - Oversteep
- Regulated Watercourse (GRCA)
- Regulated Waterbody (GRCA)
- Wetland (GRCA)
- Lake Erie Flood (GRCA)
- Lake Erie Shoreline Reach (GRCA)
- Lake Erie Dynamic Beach (GRCA)
- Lake Erie Erosion (GRCA)
- Parcel - Assessment (MPAC/MNRF)
- Cross Section (GRCA)



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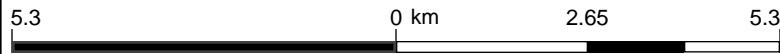


Legend

- Assessment Parcel
- Secondary Watershed
- Tertiary Watershed
- Quaternary Watershed
- Great Lakes - St. Lawrence Basin
- Hudson - James Bay Basin
- Nelson River Basin
- Hydrometric Monitoring Station
- Diversions
- Waterbody Outlet
- Conservation Authority Dam
- Provincial Dam
- Federal Dam
- OPG Dam
- Other Dam
- Virtual Flow Segment

Land Cover Compilation

- Other
- Cloud/Shadow
- Clear Open Water
- Turbid Water
- Shoreline
- Mudflats
- Marsh
- Swamp
- Fen
- Bog
- Heath
- Sparse Treed
- Treed Upland
- Deciduous Treed
- Mixed Treed
- Coniferous Treed
- Plantations - Treed Cultivated
- Hedge Rows
- Disturbance
- Open Cliff and Talus
- Alvar
- Sand Barren and Dune
- Open Tallgrass Prairie
- Tallgrass Savannah
- Tallgrass Woodland
- Sand/Gravel/Mine
- Tallings/Extraction
- Bedrock
- Community/Infrastructure
- Agriculture and Undifferentiated Rural Land Use



Scale: 1 : 104,206

Projection: Web Mercator

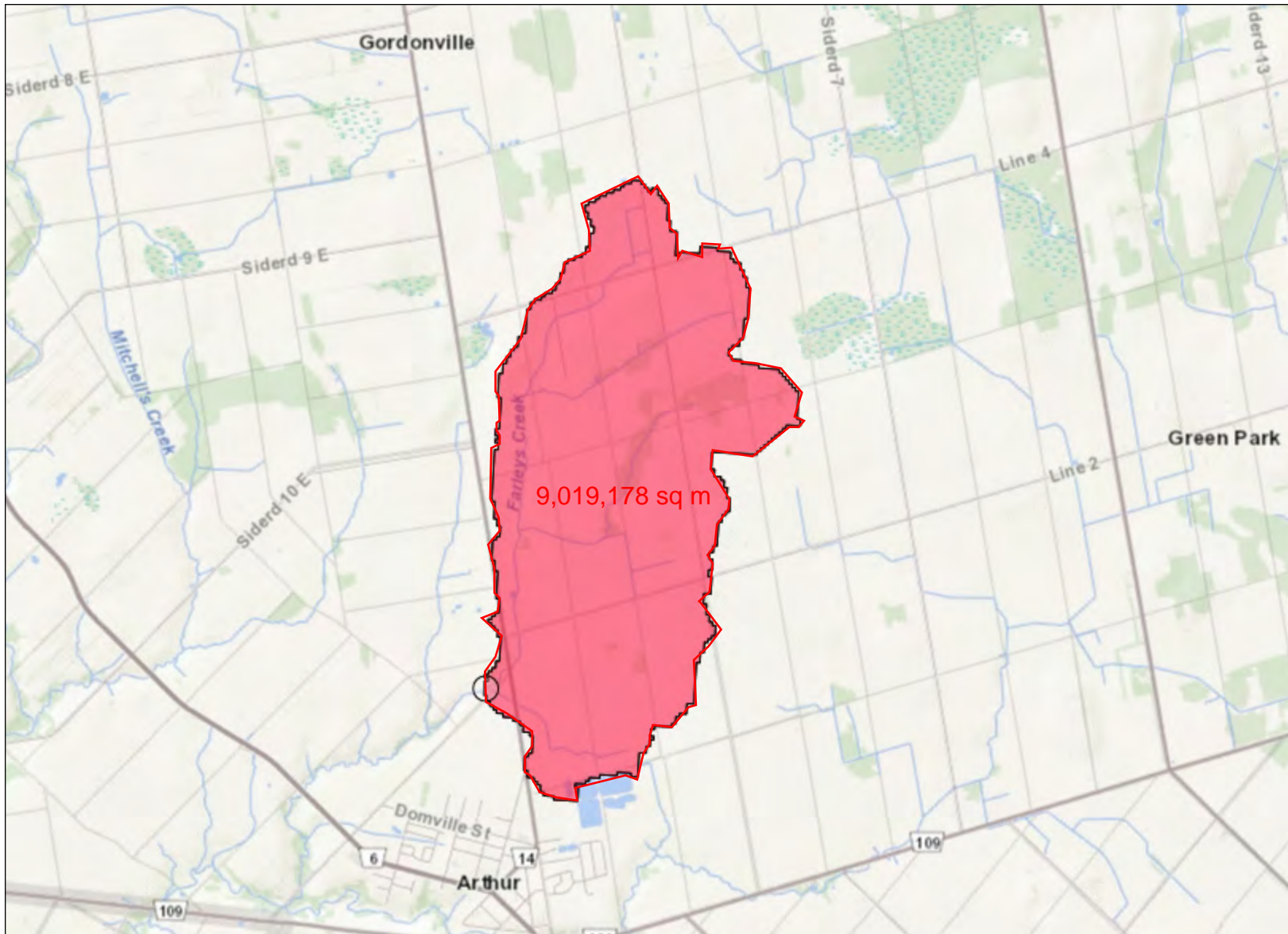


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















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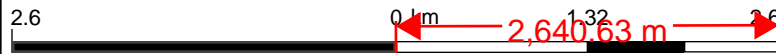


Legend

-  Assessment Parcel
-  Secondary Watershed
-  Tertiary Watershed
-  Quaternary Watershed
-  Great Lakes - St. Lawrence Basin
-  Hudson - James Bay Basin
-  Nelson River Basin
-  Hydrometric Monitoring Station
-  Diversions
-  Waterbody Outlet
-  Conservation Authority Dam
-  Provincial Dam
-  Federal Dam
-  OPG Dam
-  Other Dam
-  Virtual Flow Segment

Land Cover Compilation

-  Other
-  Cloud/Shadow
-  Clear Open Water
-  Turbid Water
-  Shoreline
-  Mudflats
-  Marsh
-  Swamp
-  Fen
-  Bog
-  Heath
-  Sparse Treed
-  Treed Upland
-  Deciduous Treed
-  Mixed Treed
-  Coniferous Treed
-  Plantations - Treed Cultivated
-  Hedge Rows
-  Disturbance
-  Open Cliff and Talus
-  Alvar
-  Sand Barren and Dune
-  Open Tallgrass Prairie
-  Tallgrass Savannah
-  Tallgrass Woodland
-  Sand/Gravel/Mine
-  Tailings/Extraction
-  Bedrock
-  Community/Infrastructure
-  Agriculture and Undifferentiated Rural Land Use



Scale: 1 : 52,071

Projection: Web Mercator



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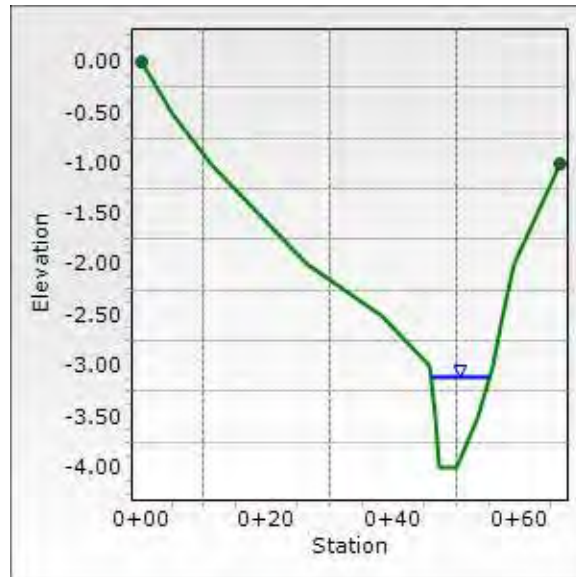
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Cross Section for Irregular Section - 2 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	886.8 mm
Discharge	8.71 m ³ /s



Worksheet for Irregular Section - 2 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	8.71 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	886.8 mm
Roughness Coefficient	0.030
Elevation	-3.11 m
Elevation Range	-4.0 to 0.0 m
Flow Area	5.4 m ²
Wetted Perimeter	9.5 m

Worksheet for Irregular Section - 2 Year

Results

Hydraulic Radius	566.8 mm
Top Width	9.18 m
Normal Depth	886.8 mm
Critical Depth	722.8 mm
Critical Slope	1.171 %
Velocity	1.61 m/s
Velocity Head	0.13 m
Specific Energy	1.02 m
Froude Number	0.672
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

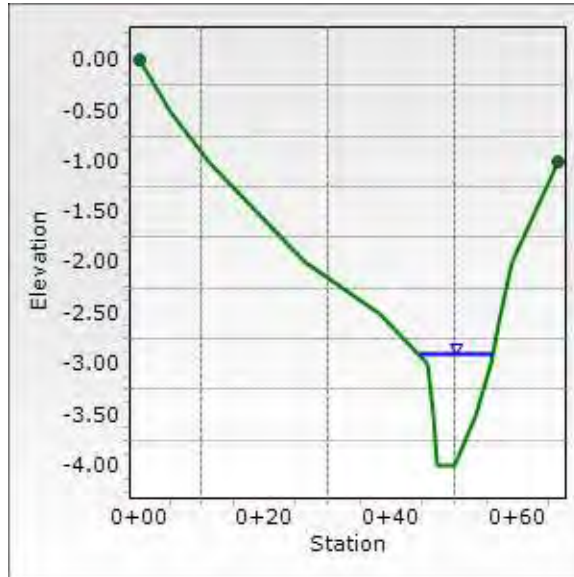
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	886.8 mm
Critical Depth	722.8 mm
Channel Slope	0.500 %
Critical Slope	1.171 %

Cross Section for Irregular Section - 5 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	1,093.1 mm
Discharge	12.83 m ³ /s



Worksheet for Irregular Section - 5 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	12.83 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1,093.1 mm
Roughness Coefficient	0.030
Elevation	-2.91 m
Elevation Range	-4.0 to 0.0 m
Flow Area	7.5 m ²
Wetted Perimeter	12.0 m

Worksheet for Irregular Section - 5 Year

Results

Hydraulic Radius	621.1 mm
Top Width	11.63 m
Normal Depth	1,093.1 mm
Critical Depth	882.4 mm
Critical Slope	1.108 %
Velocity	1.72 m/s
Velocity Head	0.15 m
Specific Energy	1.24 m
Froude Number	0.683
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

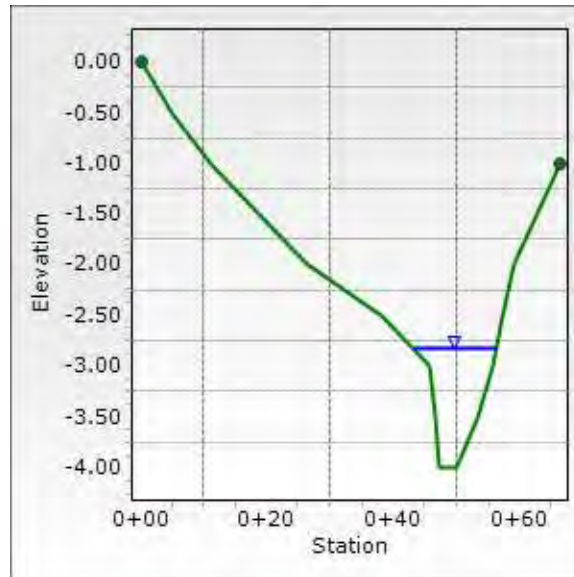
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	1,093.1 mm
Critical Depth	882.4 mm
Channel Slope	0.500 %
Critical Slope	1.108 %

Cross Section for Irregular Section - 10 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	1,176.7 mm
Discharge	14.68 m ³ /s



Worksheet for Irregular Section - 10 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	14.68 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1,176.7 mm
Roughness Coefficient	0.030
Elevation	-2.82 m
Elevation Range	-4.0 to 0.0 m
Flow Area	8.5 m ²
Wetted Perimeter	13.6 m

Worksheet for Irregular Section - 10 Year

Results

Hydraulic Radius	625.7 mm
Top Width	13.19 m
Normal Depth	1,176.7 mm
Critical Depth	945.2 mm
Critical Slope	1.087 %
Velocity	1.72 m/s
Velocity Head	0.15 m
Specific Energy	1.33 m
Froude Number	0.685
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

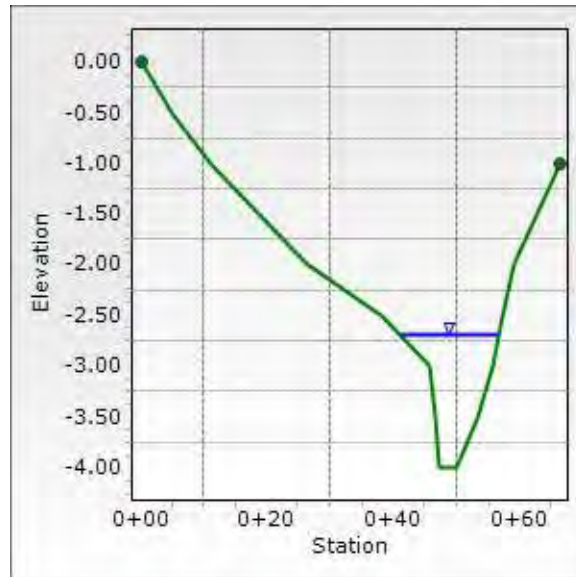
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	1,176.7 mm
Critical Depth	945.2 mm
Channel Slope	0.500 %
Critical Slope	1.087 %

Cross Section for Irregular Section - 25 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	1,312.5 mm
Discharge	18.50 m ³ /s



Worksheet for Irregular Section - 25 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	18.50 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1,312.5 mm
Roughness Coefficient	0.030
Elevation	-2.69 m
Elevation Range	-4.0 to 0.0 m
Flow Area	10.5 m ²
Wetted Perimeter	16.2 m

Worksheet for Irregular Section - 25 Year

Results

Hydraulic Radius	648.4 mm
Top Width	15.71 m
Normal Depth	1,312.5 mm
Critical Depth	1,083.5 mm
Critical Slope	1.071 %
Velocity	1.77 m/s
Velocity Head	0.16 m
Specific Energy	1.47 m
Froude Number	0.691
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

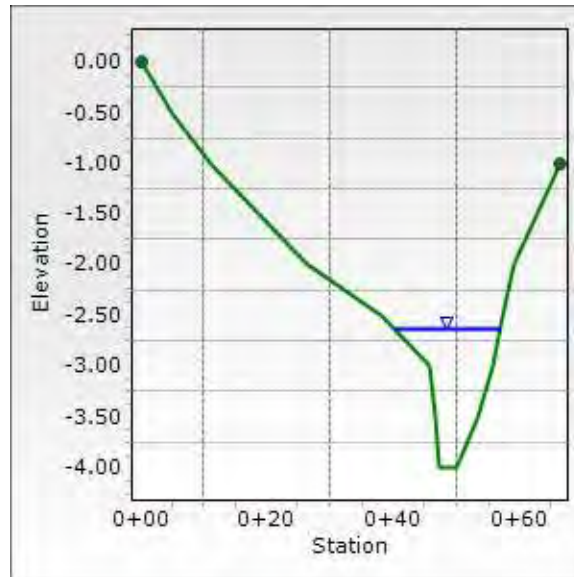
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	1,312.5 mm
Critical Depth	1,083.5 mm
Channel Slope	0.500 %
Critical Slope	1.071 %

Cross Section for Irregular Section - 50 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	1,371.1 mm
Discharge	20.47 m ³ /s



Worksheet for Irregular Section - 50 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	20.47 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1,371.1 mm
Roughness Coefficient	0.030
Elevation	-2.63 m
Elevation Range	-4.0 to 0.0 m
Flow Area	11.4 m ²
Wetted Perimeter	17.3 m

Worksheet for Irregular Section - 50 Year

Results

Hydraulic Radius	662.3 mm
Top Width	16.80 m
Normal Depth	1,371.1 mm
Critical Depth	1,148.3 mm
Critical Slope	1.067 %
Velocity	1.79 m/s
Velocity Head	0.16 m
Specific Energy	1.53 m
Froude Number	0.693
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

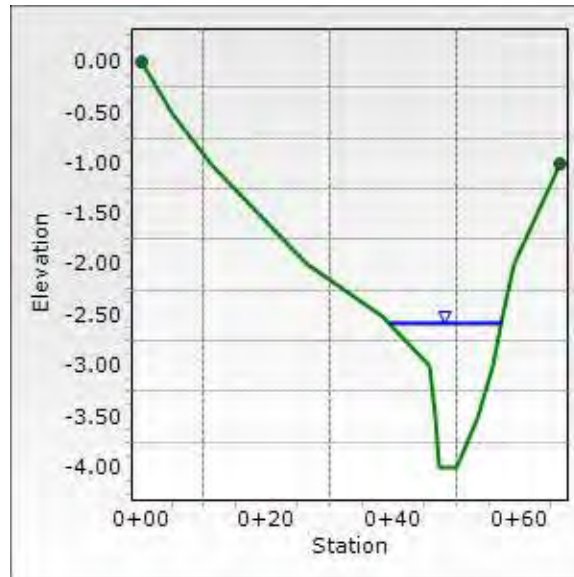
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	1,371.1 mm
Critical Depth	1,148.3 mm
Channel Slope	0.500 %
Critical Slope	1.067 %

Cross Section for Irregular Section - 100 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	1,425.3 mm
Discharge	22.47 m ³ /s



Worksheet for Irregular Section - 100 Year

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	22.47 m ³ /s

Section Definitions

Station (m)		Elevation (m)
	0+00	0.00
	0+05	-0.50
	0+11	-1.00
	0+19	-1.50
	0+26	-2.00
	0+38	-2.50
	0+46	-3.00
	0+47	-3.50
	0+47	-4.00
	0+50	-4.00
	0+53	-3.50
	0+56	-3.00
	0+58	-2.50
	0+59	-2.00
	0+63	-1.50
	0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1,425.3 mm
Roughness Coefficient	0.030
Elevation	-2.57 m
Elevation Range	-4.0 to 0.0 m
Flow Area	12.4 m ²
Wetted Perimeter	18.3 m

Worksheet for Irregular Section - 100 Year

Results

Hydraulic Radius	676.7 mm
Top Width	17.81 m
Normal Depth	1,425.3 mm
Critical Depth	1,205.9 mm
Critical Slope	1.062 %
Velocity	1.82 m/s
Velocity Head	0.17 m
Specific Energy	1.59 m
Froude Number	0.696
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

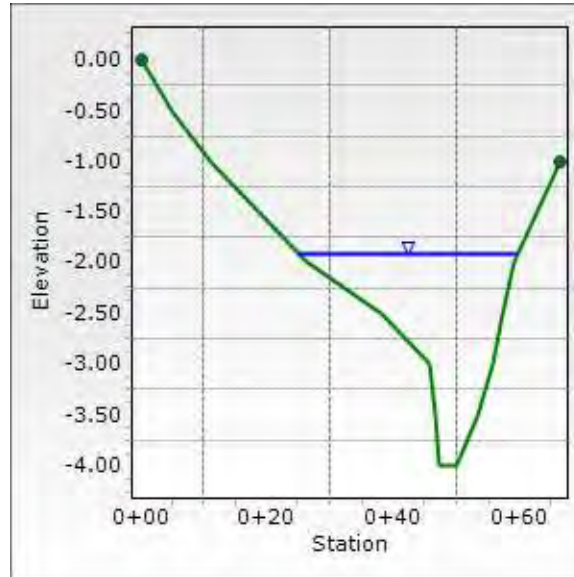
GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	1,425.3 mm
Critical Depth	1,205.9 mm
Channel Slope	0.500 %
Critical Slope	1.062 %

Cross Section for Irregular Section - Regulatory

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Channel Slope	0.500 %
Normal Depth	2,084.9 mm
Discharge	62.05 m ³ /s



Worksheet for Irregular Section - Regulatory

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.500 %
Discharge	62.05 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.00
0+05	-0.50
0+11	-1.00
0+19	-1.50
0+26	-2.00
0+38	-2.50
0+46	-3.00
0+47	-3.50
0+47	-4.00
0+50	-4.00
0+53	-3.50
0+56	-3.00
0+58	-2.50
0+59	-2.00
0+63	-1.50
0+67	-1.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.00)	(0+67, -1.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	2,084.9 mm
Roughness Coefficient	0.030
Elevation	-1.92 m
Elevation Range	-4.0 to 0.0 m
Flow Area	29.6 m ²
Wetted Perimeter	35.2 m

Worksheet for Irregular Section - Regulatory

Results

Hydraulic Radius	839.5 mm
Top Width	34.67 m
Normal Depth	2,084.9 mm
Critical Depth	1,862.9 mm
Critical Slope	0.985 %
Velocity	2.10 m/s
Velocity Head	0.22 m
Specific Energy	2.31 m
Froude Number	0.725
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 mm
Length	0.0 m
Number Of Steps	0

GVF Output Data

Upstream Depth	0.0 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	0.00 m/s
Upstream Velocity	0.00 m/s
Normal Depth	2,084.9 mm
Critical Depth	1,862.9 mm
Channel Slope	0.500 %
Critical Slope	0.985 %

Appendix C4 Hydraulic Model



Digital Report and Modelling Files

The following FileSafe Cloud link includes:

665 Eliza Street, Arthur Visual Otthymo 6.2 Floodplain Modelling – Functional Servicing and Stormwater Management Report (February 2025)

Contents:

<https://filesafecloud.scsconsultinggroup.com/url/hrkry2zcdvrpapwr>



Appendix D Overland Flow Calculations



Catchment A		100 Year Return Period Factor = 1.25		
	Runoff Coefficient	Area (ha)	Weighted Runoff Coefficient (5 Year)	Weighted Runoff Coefficient (100 Year)
Townhouse	0.70	1.77	0.14	0.17
Semi-Detached	0.60	1.63	0.11	0.14
Singles	0.45	1.67	0.08	0.11
ROW	0.60	2.70	0.18	0.23
Park	0.35	1.17	0.05	0.06
TOTAL		8.94	0.56	0.70

Catchment A	Return Period
	5 Year
Area (ha)=	8.94
Runoff Coeff. =	0.56
T _c (min)=	10.00
a =	31
b =	0.00
c =	0.699
Intensity (mm/hr) =	108.46
Runoff (m³/s) =	1.506

Catchment A	Return Period
	100 Year
Area (ha)=	8.94
Runoff Coeff. =	0.70
T _c (min)=	10.00
a =	52
b =	0.0
c =	0.699
Intensity (mm/hr) =	180.19
Runoff (m³/s) =	3.128

*Area and Runoff coefficient per **Figure 5.3**

*IDF parameters per Township of Wellington North

5 Year Flow (Catchment A)

Q_{5yr} (m³/s) = 1.506

100 Year Flow(Catchment A)

Q_{100yr} (m³/s) = 3.128

Required 100 Year Capture Capacity

Q_{100-5yr} (m³/s) = 1.622

This section of road is at a minimum slope of 0.7%. As per the ROW conveyence sheets attached, the 20m ROW has capacity to convey flow from Catchment B

Catchment B		100 Year Return Period Factor = 1.25		
	Runoff Coefficient	Area (ha)	Weighted Runoff Coefficient (5 Year)	Weighted Runoff Coefficient (100 Year)
Singles	0.45	3.43	0.20	0.24
ROW	0.40	2.84	0.14	0.18
Park	0.35	1.63	0.07	0.09
TOTAL		7.90	0.41	0.51

Catchment B	Return Period
	5 Year
Area (ha)=	7.90
Runoff Coeff. =	0.41
T _c (min)=	10.00
a =	31
b =	0.00
c =	0.699
Intensity (mm/hr) =	108.46
Runoff (m³/s) =	0.980

Catchment B	Return Period
	100 Year
Area (ha)=	7.90
Runoff Coeff. =	0.51
T _c (min)=	10.00
a =	52
b =	0.0
c =	0.699
Intensity (mm/hr) =	180.19
Runoff (m³/s) =	2.035

*Area and Runoff coefficient per **Figure 5.3**

*IDF parameters per Township of Wellington North

5 Year Flow (Catchment B)

$Q_{5yr} (m^3/s) = 0.980$

100 Year Flow(Catchment B)

$Q_{100yr} (m^3/s) = 2.035$

Required 100 Year Capture Capacity

$Q_{100-5yr} (m^3/s) = 1.055$

This section of road is at a minimum slope of 0.6%. As per the ROW conveyence sheets attached, the 20m ROW has capacity to convey flow from Catchment D

Catchment C		100 Year Return Period Factor = 1.25		
	Runoff Coefficient	Area (ha)	Weighted Runoff Coefficient (5 Year)	Weighted Runoff Coefficient (100 Year)
Singles	0.45	6.77	0.32	0.40
ROW	0.60	2.78	0.17	0.22
TOTAL		9.55	0.49	0.62

Catchment C	Return Period
	5 Year
Area (ha)=	9.55
Runoff Coeff. =	0.49
T _c (min)=	10.00
a =	31
b =	0.00
c =	0.699
Intensity (mm/hr) =	108.46
Runoff (m³/s) =	1.420

Catchment C	Return Period
	100 Year
Area (ha)=	9.55
Runoff Coeff. =	0.62
T _c (min)=	10.00
a =	52
b =	0.0
c =	0.699
Intensity (mm/hr) =	180.19
Runoff (m³/s) =	2.950

*Area and Runoff coefficient per **Figure 5.3**

*IDF parameters per Township of Wellington North

5 Year Flow (Catchment C)

Q_{5yr} (m³/s) = 1.420

100 Year Flow(Catchment C)

Q_{100yr} (m³/s) = 2.950

Required 100 Year Capture Capacity

Q_{100-5yr} (m³/s) = 1.529

This section of road is at a minimum slope of 0.6%. As per the ROW conveyence sheets attached, the 20m ROW has capacity to convey flow from Catchment C

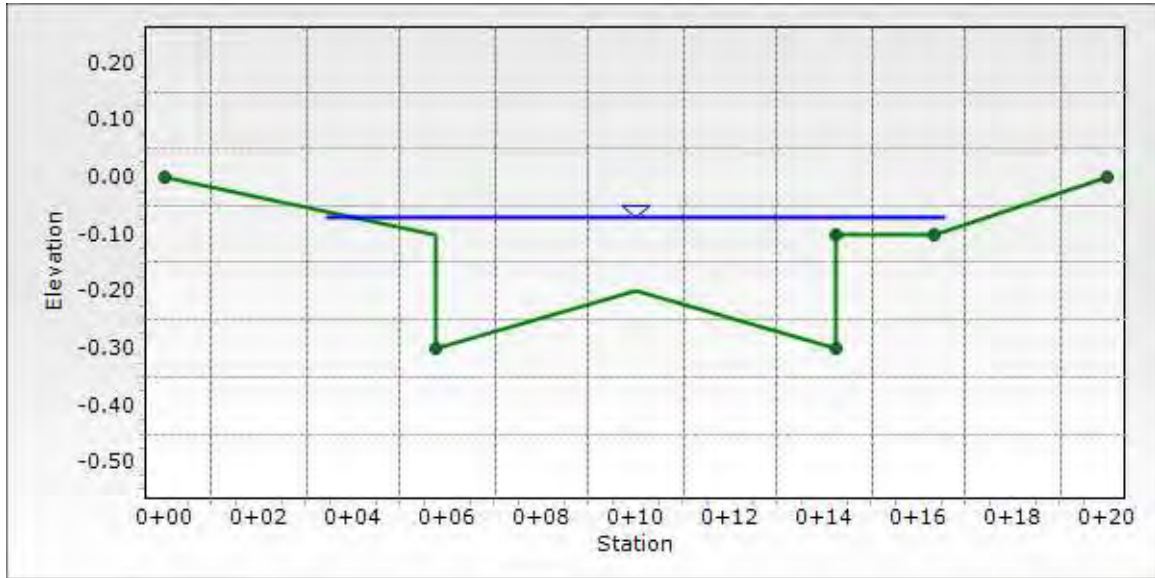
Cross Section for Catchment A

Project Description

Friction Method	Manning
	Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.700 %
Normal Depth	196.48 mm
Discharge	1.62 m ³ /s



Worksheet for Catchment A

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.700 %
Discharge	1.62 m ³ /s

Section Definitions

Station (m)		Elevation (m)
	0+00	0.000
	0+06	-0.115
	0+06	-0.265
	0+10	-0.180
	0+14	-0.265
	0+14	-0.115
	0+16	-0.073
	0+20	0.000

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.000)	(0+06, -0.265)	0.025
(0+06, -0.265)	(0+14, -0.265)	0.013
(0+14, -0.265)	(0+14, -0.115)	0.025
(0+14, -0.115)	(0+16, -0.073)	0.013
(0+16, -0.073)	(0+20, 0.000)	0.025

Options	
Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results	
Normal Depth	196.48 mm
Roughness Coefficient	0.016
Elevation	-0.069 m
Elevation Range	-0.3 to 0.0 m
Flow Area	1.4 m ²
Wetted Perimeter	13.5 m
Hydraulic Radius	105.34 mm
Top Width	13.148 m
Normal Depth	196.48 mm
Critical Depth	207.11 mm
Critical Slope	0.559 %
Velocity	1.14 m/s
Velocity Head	0.067 m
Specific Energy	0.263 m

Worksheet for Catchment A

Results

Froude Number	1.114
Flow Type	Supercritical

GVF Input Data

Downstream Depth	0.00 mm
Length	0.0 m
Number Of Steps	0

GVF Output Data

Upstream Depth	0.00 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	Infinity m/s
Upstream Velocity	Infinity m/s
Normal Depth	196.48 mm
Critical Depth	207.11 mm
Channel Slope	0.700 %
Critical Slope	0.559 %

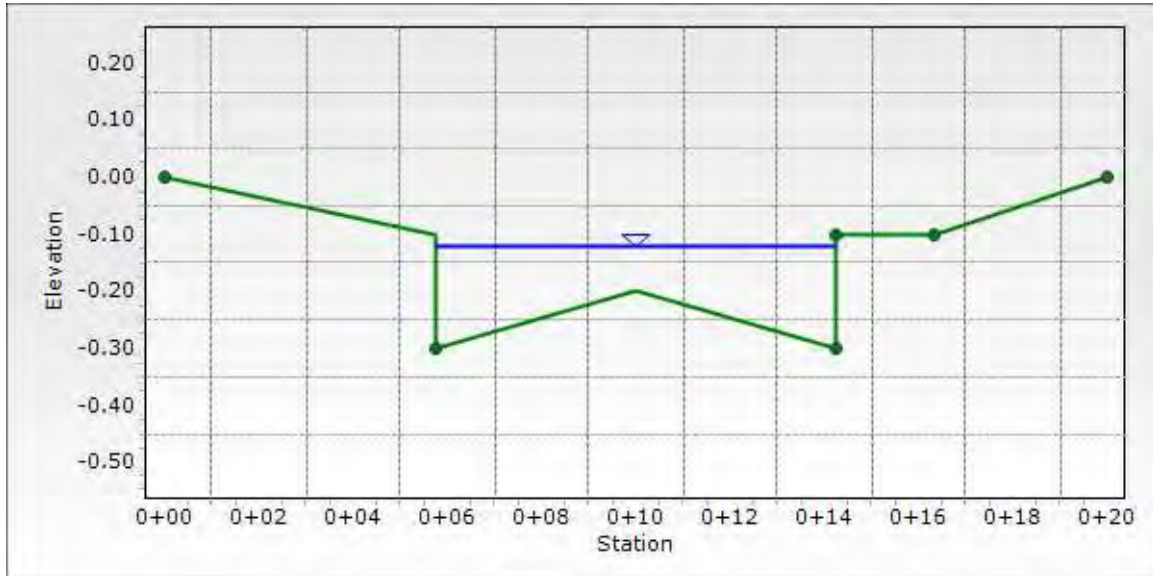
Cross Section for Catchment B

Project Description

Friction Method	Manning
	Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.600 %
Normal Depth	144.35 mm
Discharge	1.06 m ³ /s



Worksheet for Catchment B

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.600 %
Discharge	1.06 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.000
0+06	-0.115
0+06	-0.265
0+10	-0.180
0+14	-0.265
0+14	-0.115
0+16	-0.073
0+20	0.000

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.000)	(0+06, -0.265)	0.025
(0+06, -0.265)	(0+14, -0.265)	0.013
(0+14, -0.265)	(0+14, -0.115)	0.025
(0+14, -0.115)	(0+16, -0.073)	0.013
(0+16, -0.073)	(0+20, 0.000)	0.025

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	144.35 mm
Roughness Coefficient	0.014
Elevation	-0.121 m
Elevation Range	-0.3 to 0.0 m
Flow Area	0.9 m ²
Wetted Perimeter	8.8 m
Hydraulic Radius	98.49 mm
Top Width	8.500 m
Normal Depth	144.35 mm
Critical Depth	163.54 mm
Critical Slope	0.398 %
Velocity	1.22 m/s
Velocity Head	0.076 m
Specific Energy	0.220 m

Worksheet for Catchment B

Results

Froude Number	1.219
Flow Type	Supercritical

GVF Input Data

Downstream Depth	0.00 mm
Length	0.0 m
Number Of Steps	0

GVF Output Data

Upstream Depth	0.00 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	Infinity m/s
Upstream Velocity	Infinity m/s
Normal Depth	144.35 mm
Critical Depth	163.54 mm
Channel Slope	0.600 %
Critical Slope	0.398 %

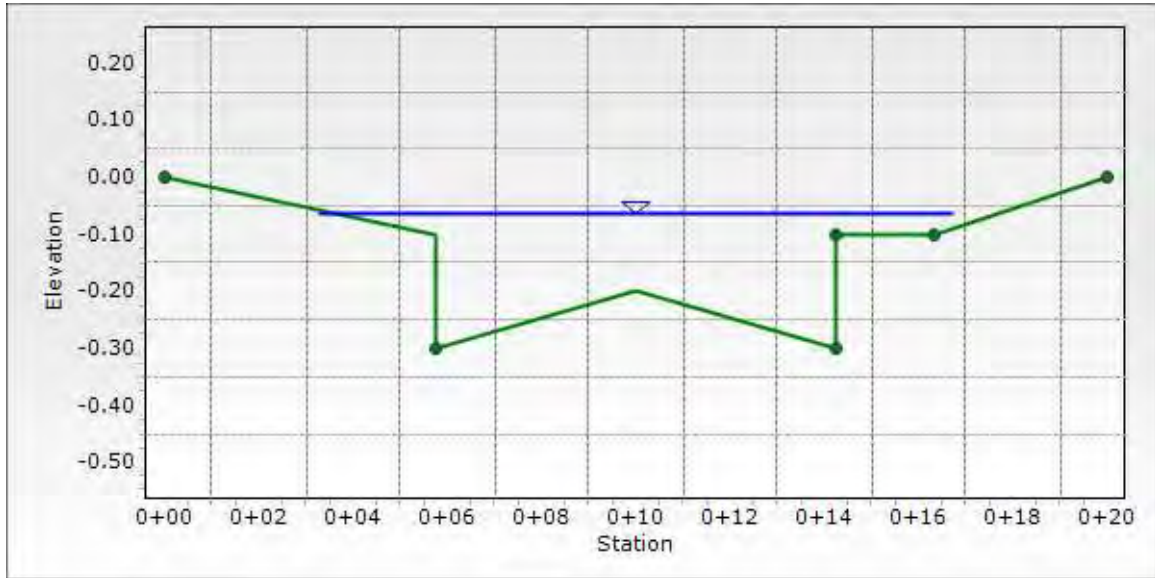
Cross Section for Catchment C

Project Description

Friction Method	Manning
Solve For	Normal Depth

Input Data

Channel Slope	0.600 %
Normal Depth	199.68 mm
Discharge	1.53 m ³ /s



Worksheet for Catchment C

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth
Input Data	
Channel Slope	0.600 %
Discharge	1.53 m ³ /s

Section Definitions

Station (m)	Elevation (m)
0+00	0.000
0+06	-0.115
0+06	-0.265
0+10	-0.180
0+14	-0.265
0+14	-0.115
0+16	-0.073
0+20	0.000

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.000)	(0+06, -0.265)	0.025
(0+06, -0.265)	(0+14, -0.265)	0.013
(0+14, -0.265)	(0+14, -0.115)	0.025
(0+14, -0.115)	(0+16, -0.073)	0.013
(0+16, -0.073)	(0+20, 0.000)	0.025

Options	
Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results	
Normal Depth	199.68 mm
Roughness Coefficient	0.017
Elevation	-0.065 m
Elevation Range	-0.3 to 0.0 m
Flow Area	1.5 m ²
Wetted Perimeter	13.8 m
Hydraulic Radius	105.98 mm
Top Width	13.468 m
Normal Depth	199.68 mm
Critical Depth	201.28 mm
Critical Slope	0.580 %
Velocity	1.05 m/s
Velocity Head	0.056 m
Specific Energy	0.256 m

Worksheet for Catchment C

Results

Froude Number	1.016
Flow Type	Supercritical

GVF Input Data

Downstream Depth	0.00 mm
Length	0.0 m
Number Of Steps	0

GVF Output Data

Upstream Depth	0.00 mm
Profile Description	N/A
Profile Headloss	0.00 m
Downstream Velocity	Infinity m/s
Upstream Velocity	Infinity m/s
Normal Depth	199.68 mm
Critical Depth	201.28 mm
Channel Slope	0.600 %
Critical Slope	0.580 %

Appendix E Stormwater Management



Permanent Pool and Extended Detention Sizing East Pond

Weighted Impervious Calculation

Catchment ID	Total Area (ha)	Imperviousness (%)	Impervious Area (ha)
312	2.45	67	1.63
313	0.60	67	0.40
315	22.70	60	13.51
401	64.15	5	2.97
Total	89.90	21	18.51

Allowable Release Rates

Storm	Total to Farley Creek (Pre-Dev NHYD 3)
	(m ³ /s)
2 Year	1.182
5 Year	1.948
10 Year	2.511
25 Year	3.267
50 Year	3.845
100 Year	4.441

Per existing conditions VO Model in **Appendix F**.

PERMANENT POOL

Level of Protection = **Enhanced** (Level 1)

Weighted Impervious = 21 %

Drainage Area = 89.90 ha

SWMP Type = **4. Wet Pond**

Required Permanent Pool (including 40m³/ha for extended detention)= 96.5 m³/ha

Required Permanent Pool (minus 40m³/ha for extended detention)= 57 m³/ha

Required Permanent Pool = 5083 m³

**TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MECP SWM PLANNING AND DESIGN MANUAL - 2003)**

Protection Level	SWMP Type	Storage Volume (m ³ /ha) for Impervious Level			
		35%	55%	70%	85%
Enhanced (Level 1)	1. Infiltration	25	30	35	40
	2. Wetlands	80	105	120	140
	3. Hybrid Wet Pond/Wetland	110	150	175	195
	4. Wet Pond	140	190	225	250
Normal (Level 2)	1. Infiltration	20	20	25	30
	2. Wetlands	60	70	80	90
	3. Hybrid Wet Pond/Wetland	75	90	105	120
	4. Wet Pond	90	110	130	150
Basic (Level 3)	1. Infiltration	20	20	20	20
	2. Wetlands	60	60	60	60
	3. Hybrid Wet Pond/Wetland	60	70	75	80
	4. Wet Pond	60	75	85	95
	5. Dry Pond (Continuous Flow)	90	150	200	240

EXTENDED DETENTION

Using the 25mm - 4 hour Chicago Storm

Erosion Control Volume (V) = Runoff Depth (mm) x Drainage Area (ha) x 10 (m³) / (mm)(ha)

Erosion Control Volume (V) = **8.64** mm x 89.90 ha x 10 m³ / mm·ha

Erosion Control Volume (V) = 7767 m³

Using 40m³/ha

Extended Detention Volume (V) = 40m³/ha x Drainage Area (ha)

Extended Detention Volume (V) = 40 m³/ha 89.90 ha

Extended Detention Volume (V) = 3596 m³

Governing Volume (V) = 7767 m³

Permanent Pool and Extended Detention Sizing East Pond

Elevation (m)	Area (m ²)	Area (m ²)	H (m)	Vol (m ³)	Volume (m ³)	Storage (m ³)	Depth (m)	
458.50	7443						0	
		9072	1.4	12701				
459.90	10701				12701		1.4	
		12129	0.6	7278				
460.50	13558				19978	0	2	N.W.L.
		14774	0.6	8864				
461.10	15990				28843	8864	2.6	
		17344	1.4	24282				
462.50	18699				53124	33146	4	

Permanent Pool Volume Required = 5083 m³
Permanent Pool Volume Provided = 19978 m³

Extended Detention volume required = 7767 m³
Extended Detention waterlevel = 461.02 m

2 year control volume required = 6355 m³
2 year control waterlevel = 460.94 m
2 year active fluctuation = 1.56 m

5 year control volume required = 8557 m³
5 year control waterlevel = 461.08 m
5 year active fluctuation = 1.42 m

10 year control volume required = 10371 m³
10 year control waterlevel = 461.19 m
10 year active fluctuation = 1.31 m

25 year control volume required = 12766 m³
25 year control waterlevel = 461.34 m
25 year active fluctuation = 1.16 m

50 year control volume required = 14384 m³
50 year control waterlevel = 461.44 m
50 year active fluctuation = 1.06 m

100 year control volume required = 16115 m³
100 year waterlevel = 461.54 m
Freeboard = 0.96 m

FOREBAY SIZING CALCULATIONS EAST POND

Forebay

Elevation (m)	Area (m ²)	Average Area (m ²)	Height (m)	Volume (m ³)	Cumulative Volume (m ³)	Depth (m)
458.50	1281	1728	1.4	2,420	0	0
459.90	2176				2,420	1.4
460.50	3021	2598	0.6	1,559	3,979	2

Total Permanent Pool

Elevation (m)	Area (m ²)	Average Area (m ²)	Height (m)	Volume (m ³)	Cumulative Volume (m ³)	Depth (m)
458.50	7443	9072	1.4	12,701	0	0
459.90	10701				12,701	1.4
460.50	13558	12129	0.6	7,278	19,978	2

Minimum Criteria (per MECP guidelines)

Forebay area is **20** % of total Permanent Pool area

Maximum Forebay area is **33** % of total Permanent Pool area

Therefore the minimum criteria per MECP guidelines is satisfied.

2. Forebay Settling Length

$$\text{Dist} = (r \times Q_p / V_s)^{0.5}$$

where: Dist = forebay length (m)

$$\text{Dist} = (3.1 \times 0.28 / 0.0003)^{0.5}$$

r = length to width ratio

$$= 3.10$$

Q_p = peak flow rate from pond during design quality storm (m^3/s) per VO Model

$$= 0.282$$

V_s = settling velocity (m/s)*

$$= 0.0003$$

$$\text{Dist} = 54.0$$

Minimum forebay length is (m) **54.0**

Actual forebay length is (m) **94.6**

CRITERIA SATISFIED

3. Forebay Dispersion Length

$$\text{Dist} = (8 \times Q) / (d \times V_f)$$

where: Dist = forebay length (m)

$$\text{Dist} = (8 \times 3.816) / (1.4 \times 0.5)$$

Q = inlet flow rate (m^3/s) per VO Model - 5 yr

$$= 3.816$$

d = depth of permanent pool in forebay (m)

$$= 1.4$$

V_f = desired velocity in forebay (m/s)*

$$0.5$$

$$\text{Dist} = 43.6$$

Minimum forebay length is (m) **43.6**

Actual forebay length is (m) **94.6**

CRITERIA SATISFIED

4. Minimum Forebay Bottom Width

$$\text{Width} = \text{Dist} / 8$$

where: Width = minimum forebay bottom width (m)

$$\text{Width} = 54 / 8$$

Dist = minimum forebay length (m)

$$= 54.0$$

$$\text{Width} = 6.8$$

Minimum bottom width is (m) **6.8**

Actual bottom width is (m) **40.0**

CRITERIA SATISFIED

5. Maximum Velocity Check

$$V = Q/A$$

where: V = velocity (m/s)

$$V = 1.508 / 44.86000000000002$$

Q = inlet flow rate (m^3/s) - 25mm 4 hr Chicago

$$= 1.508$$

A = average cross-sectional area of entire forebay (m^2) (see **Page 3**)

$$= 44.9$$

$$V = 0.03$$

Maximum velocity permitted is (m/s) **0.15**

Actual velocity is (m/s) **0.03**

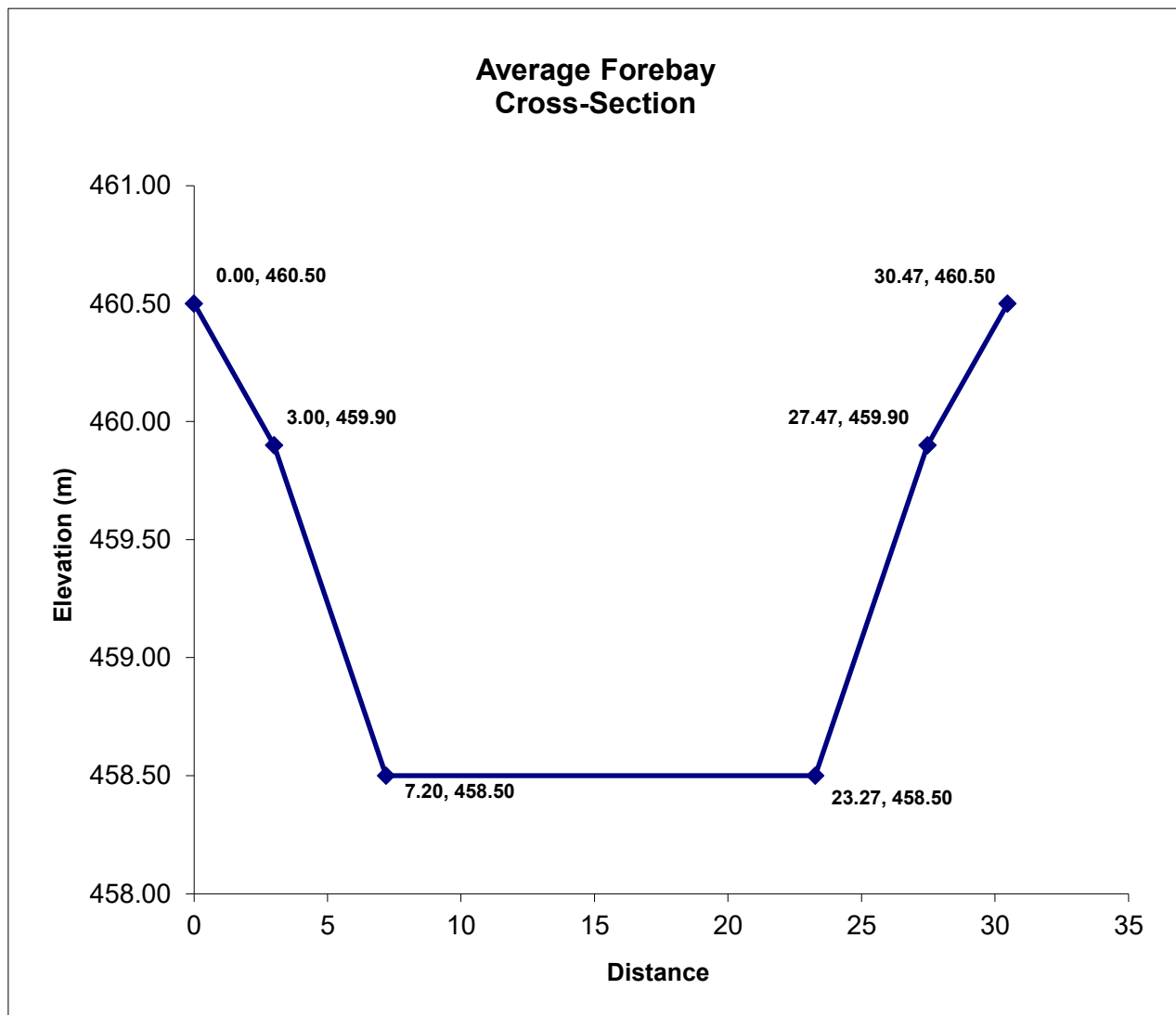
CRITERIA SATISFIED

*Value recommended by the MECP Stormwater Management Planning & Design Manual

FOREBAY SIZING CALCULATIONS EAST POND

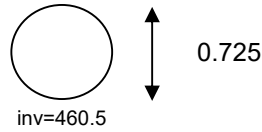
Distance (m)	Elevation (m)	Depth (m)	Incremental Area (m ²)
0.00	460.50	0.00	--
3.00	459.90	0.60	0.90
7.20	458.50	2.00	5.46
23.27	458.50	2.00	32.14
27.47	459.90	0.60	5.46
30.47	460.50	0.00	0.90

Area (m²) = 44.86



Orifice 1

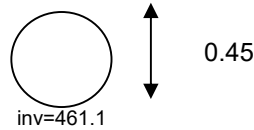
Invert = 460.50 m
 Size = 0.725 m
 Orifice Coefficient, C = 0.62
 Obvert = 461.225 m



Orifice 2

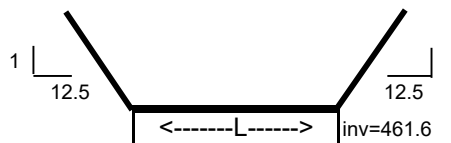
(Round Only)

Invert = 461.10 m
 Size = 0.450 m
 C = 0.80
 Obvert = 461.55 m



Broad Crested Weir (Weir 1)

Length = 60.00 m
 Elevation = 461.60 m
 Crest Breadth = 3.0 m
 Side Slope = 12.5
 (0 = vertical, 1 = 1H to 1V, 3 = 3H to 1 v)



Ditch Inlet Grate - Type A,B,C (per OPSD 403.010)

Top Elevation = 461.85 m
 Grate Slope = 6
 Grate Size Length = 0.762 m
 (1 = 1H to 1V, 2 = 2H to 1 v, 6 = 6H to 1v)

OUTFLOW SUMMARY EAST POND

Starting Water Level (m) = 460.50
Elevation Increment (m) = 0.02

Shading represents Storage-Discharge pairings used in VO modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Orifice 2 Outflow (cms)	Weir 1 Outflow (cms)	Ditch Inlet Outflow (cms)	Backwater Elevation (m)	Stage (m)	Total Flow (cms)	Storage (m ³)	Detention Time (hrs)
460.50	0.000	0.000	0.000	0.00	0.00	460.50	0.000	0	0.0
460.52	0.001	0.000	0.000	0.00	0.00	460.52	0.001	272	0.0
460.54	0.003	0.000	0.000	0.00	0.00	460.54	0.003	546	0.0
460.56	0.007	0.000	0.000	0.00	0.00	460.56	0.007	821	16.1
460.58	0.012	0.000	0.000	0.00	0.00	460.58	0.012	1098	24.5
460.60	0.018	0.000	0.000	0.00	0.00	460.60	0.018	1376	29.8
460.62	0.026	0.000	0.000	0.00	0.00	460.62	0.026	1656	33.3
460.64	0.035	0.000	0.000	0.00	0.00	460.64	0.035	1938	35.9
460.66	0.045	0.000	0.000	0.00	0.00	460.66	0.045	2221	37.9
460.68	0.056	0.000	0.000	0.00	0.00	460.68	0.056	2506	39.5
460.70	0.068	0.000	0.000	0.00	0.00	460.70	0.068	2793	40.8
460.72	0.082	0.000	0.000	0.00	0.00	460.72	0.082	3081	41.9
460.74	0.096	0.000	0.000	0.00	0.00	460.74	0.096	3371	42.8
460.76	0.112	0.000	0.000	0.00	0.00	460.76	0.112	3662	43.5
460.78	0.128	0.000	0.000	0.00	0.00	460.78	0.128	3955	44.2
460.80	0.146	0.000	0.000	0.00	0.00	460.80	0.146	4250	44.8
460.82	0.164	0.000	0.000	0.00	0.00	460.82	0.164	4546	45.3
460.84	0.183	0.000	0.000	0.00	0.00	460.84	0.183	4844	45.8
460.86	0.203	0.000	0.000	0.00	0.00	460.86	0.203	5143	46.2
460.88	0.203	0.000	0.000	0.00	0.00	460.88	0.203	5445	46.7
460.90	0.220	0.000	0.000	0.00	0.00	460.90	0.220	5747	47.1
460.92	0.272	0.000	0.000	0.00	0.00	460.92	0.272	6052	47.4
460.94	0.316	0.000	0.000	0.00	0.00	460.94	0.316	6358	47.7
460.96	0.354	0.000	0.000	0.00	0.00	460.96	0.354	6665	47.9
460.98	0.389	0.000	0.000	0.00	0.00	460.98	0.389	6975	48.2
461.00	0.420	0.000	0.000	0.00	0.00	461.00	0.420	7286	48.4
461.02	0.450	0.000	0.000	0.00	0.00	461.02	0.450	7598	48.6
461.04	0.478	0.000	0.000	0.00	0.00	461.04	0.478	7912	48.8
461.06	0.504	0.000	0.000	0.00	0.00	461.06	0.504	8228	49.0
461.08	0.529	0.000	0.000	0.00	0.00	461.08	0.529	8545	49.1
461.10	0.553	0.000	0.000	0.00	0.00	461.10	0.553	8864	49.3
461.12	0.575	0.001	0.000	0.00	0.00	461.12	0.576	9184	49.5
461.14	0.597	0.002	0.000	0.00	0.00	461.14	0.600	9505	49.6
461.16	0.618	0.005	0.000	0.00	0.00	461.16	0.623	9827	49.8
461.18	0.639	0.009	0.000	0.00	0.00	461.18	0.648	10150	49.9
461.20	0.659	0.014	0.000	0.00	0.00	461.20	0.672	10473	50.0
461.22	0.678	0.019	0.000	0.00	0.00	461.22	0.697	10797	50.2
461.24	0.697	0.026	0.000	0.00	0.00	461.24	0.723	11122	50.3
461.26	0.715	0.033	0.000	0.00	0.00	461.26	0.748	11447	50.4
461.28	0.733	0.042	0.000	0.00	0.00	461.28	0.774	11774	50.5
461.30	0.750	0.050	0.000	0.00	0.00	461.30	0.800	12101	50.6
461.32	0.767	0.060	0.000	0.00	0.00	461.32	0.827	12429	50.8
461.34	0.783	0.069	0.000	0.00	0.00	461.34	0.852	12757	50.9
461.36	0.800	0.105	0.000	0.00	0.00	461.36	0.905	13087	51.0
461.38	0.816	0.132	0.000	0.00	0.00	461.38	0.948	13417	51.1
461.40	0.831	0.154	0.000	0.00	0.00	461.40	0.986	13748	51.2
461.42	0.847	0.174	0.000	0.00	0.00	461.42	1.020	14080	51.3
461.44	0.862	0.191	0.000	0.00	0.00	461.44	1.053	14412	51.3
461.46	0.876	0.207	0.000	0.00	0.00	461.46	1.083	14746	51.4
461.48	0.891	0.222	0.000	0.00	0.00	461.48	1.113	15080	51.5
461.50	0.905	0.236	0.000	0.00	0.00	461.50	1.141	15415	51.6
461.52	0.919	0.249	0.000	0.00	0.00	461.52	1.168	15750	51.7
461.54	0.933	0.261	0.000	0.00	0.00	461.54	1.194	16087	51.8
461.56	0.947	0.273	0.000	0.00	0.00	461.56	1.220	16424	51.8
461.58	0.960	0.285	0.000	0.00	0.00	461.58	1.245	16762	51.9
461.60	0.974	0.296	0.000	0.00	0.00	461.60	1.269	17101	52.0
461.62	0.987	0.306	0.240	0.00	0.00	461.62	1.533	17440	52.1
461.64	1.000	0.316	0.682	0.00	0.00	461.64	1.998	17781	52.1
461.66	1.012	0.326	1.259	0.00	0.00	461.66	2.598	18122	52.1
461.68	1.025	0.336	1.946	0.00	0.00	461.68	3.307	18464	52.2
461.70	1.038	0.345	2.731	0.00	0.00	461.70	4.114	18806	52.2
461.72	1.050	0.354	3.605	0.00	0.00	461.72	5.009	19150	52.2
461.74	1.062	0.363	4.561	0.00	0.00	461.74	5.986	19494	52.2
461.76	1.074	0.372	5.595	0.00	0.00	461.76	7.041	19839	52.3
461.78	1.086	0.380	6.703	0.00	0.00	461.78	8.169	20184	52.3
461.80	1.098	0.388	7.882	0.00	0.00	461.80	9.368	20531	52.3
461.82	1.109	0.397	9.648	0.00	0.00	461.82	11.154	20878	52.3
461.84	1.121	0.404	11.037	0.00	0.00	461.84	12.562	21226	52.3
461.86	1.132	0.412	12.494	0.01	0.00	461.86	14.046	21575	52.3

OUTFLOW SUMMARY EAST POND

Starting Water Level (m) = 460.50
Elevation Increment (m) = 0.02

Shading represents Storage-Discharge pairings used in VO modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Orifice 2 Outflow (cms)	Weir 1 Outflow (cms)	Ditch Inlet Outflow (cms)	Backwater Elevation (m)	Stage (m)	Total Flow (cms)	Storage (m ³)	Detention Time (hrs)
461.88	1.144	0.420	14.018	0.02	0.00	461.88	15.602	21925	52.3
461.90	1.155	0.427	15.608	0.04	0.00	461.90	17.228	22275	52.3
461.92	1.166	0.435	17.146	0.06	0.00	461.92	18.806	22626	52.3
461.94	1.177	0.442	18.852	0.09	0.00	461.94	20.557	22978	52.3
461.96	1.188	0.449	20.619	0.12	0.00	461.96	22.374	23331	52.3
461.98	1.198	0.456	22.448	0.15	0.00	461.98	24.256	23684	52.3
462.00	1.209	0.463	24.337	0.19	0.00	462.00	26.203	24038	52.3
462.02	1.220	0.470	26.285	0.24	0.00	462.02	28.215	24393	52.3
462.04	1.230	0.477	28.293	0.29	0.00	462.04	30.290	24749	52.4
462.06	1.241	0.483	30.359	0.34	0.00	462.06	32.428	25106	52.4
462.08	1.251	0.490	32.484	0.40	0.00	462.08	34.629	25463	52.4
462.10	1.261	0.496	34.197	0.47	0.00	462.10	36.424	25821	52.4
462.12	1.271	0.503	36.407	0.54	0.00	462.12	38.718	26180	52.4
462.14	1.281	0.509	38.672	0.61	0.00	462.14	41.073	26540	52.4
462.16	1.291	0.515	40.993	0.69	0.00	462.16	43.489	26900	52.4
462.18	1.301	0.521	43.370	0.77	0.00	462.18	45.965	27261	52.4
462.20	1.311	0.527	45.802	0.86	0.00	462.20	48.501	27623	52.4
462.22	1.321	0.533	48.289	0.95	0.00	462.22	51.096	27986	52.4
462.24	1.331	0.539	50.831	1.05	0.00	462.24	53.751	28350	52.4
462.26	1.340	0.545	53.428	1.15	0.00	462.26	56.466	28714	52.4
462.28	1.350	0.551	56.080	1.26	0.00	462.28	59.239	29079	52.4
462.30	1.359	0.556	58.786	1.37	0.00	462.30	62.072	29445	52.4
462.32	1.369	0.562	61.546	1.49	0.00	462.32	64.963	29811	52.4
462.34	1.378	0.568	64.361	1.61	0.00	462.34	67.913	30179	52.4
462.36	1.387	0.573	67.229	1.73	0.00	462.36	70.922	30547	52.4
462.38	1.397	0.579	70.152	1.86	0.00	462.38	73.989	30916	52.4
462.40	1.406	0.584	73.128	2.00	0.00	462.40	77.115	31286	52.4
462.42	1.415	0.590	76.159	2.14	0.00	462.42	80.300	31656	52.4
462.44	1.424	0.595	79.243	2.28	0.00	462.44	83.542	32027	52.4
462.46	1.433	0.600	82.381	2.43	0.00	462.46	86.843	32399	52.4
462.48	1.442	0.606	85.573	2.58	0.00	462.48	90.203	32772	52.4
462.50	1.451	0.611	88.818	2.74	0.00	462.50	93.621	33146	52.4

Permanent Pool and Extended Detention Sizing West Pond

Weighted Impervious Calculation

Catchment ID	Total Area (ha)	Imperviousness (%)	Impervious Area (ha)
302	1.92	50	0.96
304	11.12	60	6.71
309	4.83	65	3.14
Total	17.87	60	10.81

Allowable Release Rates

Storm	Total to Farley Creek (NHVD 3)
	(m ³ /s)
2 Year	1.182
5 Year	1.948
10 Year	2.511
25 Year	3.267
50 Year	3.845
100 Year	4.441

Per existing conditions VO Model in **Appendix F**.

PERMANENT POOL

Level of Protection = **Enhanced** (Level 1)

Weighted Impervious = 60 %

Drainage Area = 17.87 ha

SWMP Type = **4. Wet Pond**

Required Permanent Pool (including 40m³/ha for extended detention)= 202.8 m³/ha

Required Permanent Pool (minus 40m³/ha for extended detention)= 163 m³/ha

Required Permanent Pool = 2909 m³

**TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MECP SWM PLANNING AND DESIGN MANUAL - 2003)**

Protection Level	3.845	Storage Volume (m ³ /ha) for Impervious Level			
		35%	55%	70%	85%
Enhanced (Level 1)	1. Infiltration	25	30	35	40
	2. Wetlands	80	105	120	140
	3. Hybrid Wet Pond/Wetland	110	150	175	195
	4. Wet Pond	140	190	225	250
Normal (Level 2)	1. Infiltration	20	20	25	30
	2. Wetlands	60	70	80	90
	3. Hybrid Wet Pond/Wetland	75	90	105	120
	4. Wet Pond	90	110	130	150
Basic (Level 3)	1. Infiltration	20	20	20	20
	2. Wetlands	60	60	60	60
	3. Hybrid Wet Pond/Wetland	60	70	75	80
	4. Wet Pond	60	75	2909	95
	5. Dry Pond (Continuous Flow)	90	150	200	240

EXTENDED DETENTION

Using the 25mm - 4 hour Chicago Storm

Erosion Control Volume (V) = Runoff Depth (mm) x Drainage Area (ha) x 10 (m³) / (mm)(ha)

Erosion Control Volume (V) = **15.19** 17.87 ha x 10 m³ / mm·ha

Erosion Control Volume (V) = 2714 m³

Using 40m³/ha

Extended Detention Volume (V) = 40m³/ha x Drainage Area (ha)

Extended Detention Volume (V) = 40 m³/ha 17.87 ha

Extended Detention Volume (V) = 715 m³

Governing Volume (V) = 2714 m³

Permanent Pool and Extended Detention Sizing West Pond

Elevation (m)	Area (m ²)	Area (m ²)	H (m)	Vol (m ³)	Volume (m ³)	Storage (m ³)	Depth (m)	
456.60	6033						0	
		7252	1.40	10153				
458.00	8472				10153		1.4	
		9488	0.6	5693				
458.60	10505				15846	0	2	N.W.L.
		11353	0.60	6812				
459.20	12202				22658	6812	2.6	
		13311	1.4	18636				
460.60	14421				41294	25448	4	

Permanent Pool Volume Required = 2909 m³

Permanent Pool Volume Provided = 15846 m³

Extended Detention volume required = 2714 m³

Extended Detention waterlevel = 458.84 m

2 year control volume required = 3169 m³

2 year control waterlevel = 458.89 m

2 year active fluctuation = 1.71 m

5 year control volume required = 4323 m³

5 year control waterlevel = 458.99 m

5 year active fluctuation = 1.61 m

10 year control volume required = 5143 m³

10 year control waterlevel = 459.06 m

10 year active fluctuation = 1.54 m

25 year control volume required = 6163 m³

25 year control waterlevel = 459.15 m

25 year active fluctuation = 1.45 m

50 year control volume required = 6868 m³

50 year control waterlevel = 459.20 m

50 year active fluctuation = 1.40 m

100 year control volume required = 7566 m³

100 year waterlevel = 459.26 m

Freeboard = 1.34 m

FOREBAY SIZING CALCULATIONS WEST POND

Forebay

Elevation (m)	Area (m ²)	Average Area (m ²)	Height (m)	Volume (m ³)	Cumulative Volume (m ³)	Depth (m)
456.60	1518	2075	1.4	2,905	0	0
458.00	2632				2,905	1.4
458.60	3601	3116	0.6	1,870	4,775	2

Total Permanent Pool

Elevation (m)	Area (m ²)	Average Area (m ²)	Height (m)	Volume (m ³)	Cumulative Volume (m ³)	Depth (m)
456.60	6033	7252	1.4	10,153	0	0
458.00	8472				10,153	1.4
458.60	10505	9488	0.6	5,693	15,846	2

Minimum Criteria (per MECP guidelines)

Forebay area is **31** % of total Permanent Pool area

Maximum Forebay area is **33** % of total Permanent Pool area

Therefore the minimum criteria per MECP guidelines is satisfied.

2. Forebay Settling Length

$$\text{Dist} = (r \times Q_p / V_s)^{0.5}$$

where: Dist = forebay length (m)

$$\text{Dist} = (1.88 \times 0.06 / 0.0003)^{0.5}$$

r = length to width ratio

$$= 1.88$$

Q_p = peak flow rate from pond during design quality storm (m^3/s) per VO model Node 2

$$= 0.063$$

$$\text{Dist} = 19.9$$

V_s = settling velocity (m/s)*

$$= 0.0003$$

Minimum forebay length is (m) **19.9**

Actual forebay length is (m) **56.5**

CRITERIA SATISFIED

3. Forebay Dispersion Length

$$\text{Dist} = (8 \times Q) / (d \times V_f)$$

where: Dist = forebay length (m)

$$\text{Dist} = (8 \times 2.977) / (1.4 \times 0.5)$$

Q = inlet flow rate (m^3/s) per VO model Node 2 - 5 yr

$$= 2.977$$

d = depth of permanent pool in forebay (m)

$$= 1.4$$

$$\text{Dist} = 34.0$$

V_f = desired velocity in forebay (m/s)*

$$0.5$$

Minimum forebay length is (m) **34.0**

Actual forebay length is (m) **56.5**

CRITERIA SATISFIED

4. Minimum Forebay Bottom Width

$$\text{Width} = \text{Dist} / 8$$

where: Width = minimum forebay bottom width (m)

$$\text{Width} = 34 / 8$$

Dist = minimum forebay length (m)

$$= 34.0$$

$$\text{Width} = 4.3$$

Minimum bottom width is (m) **4.3**

Actual bottom width is (m) **15.6**

CRITERIA SATISFIED

5. Maximum Velocity Check

$$V = Q/A$$

where: V = velocity (m/s)

$$V = 1.218 / 43.92000000000002$$

Q = inlet flow rate (m^3/s) - 25mm 4 hr Chicago VO Model Node :

$$= 1.218$$

A = average cross-sectional area of entire forebay (m^2)

(see **Page 3**)

$$= 43.9$$

$$V = 0.03$$

Maximum velocity permitted is (m/s) **0.15**

Actual velocity is (m/s) **0.03**

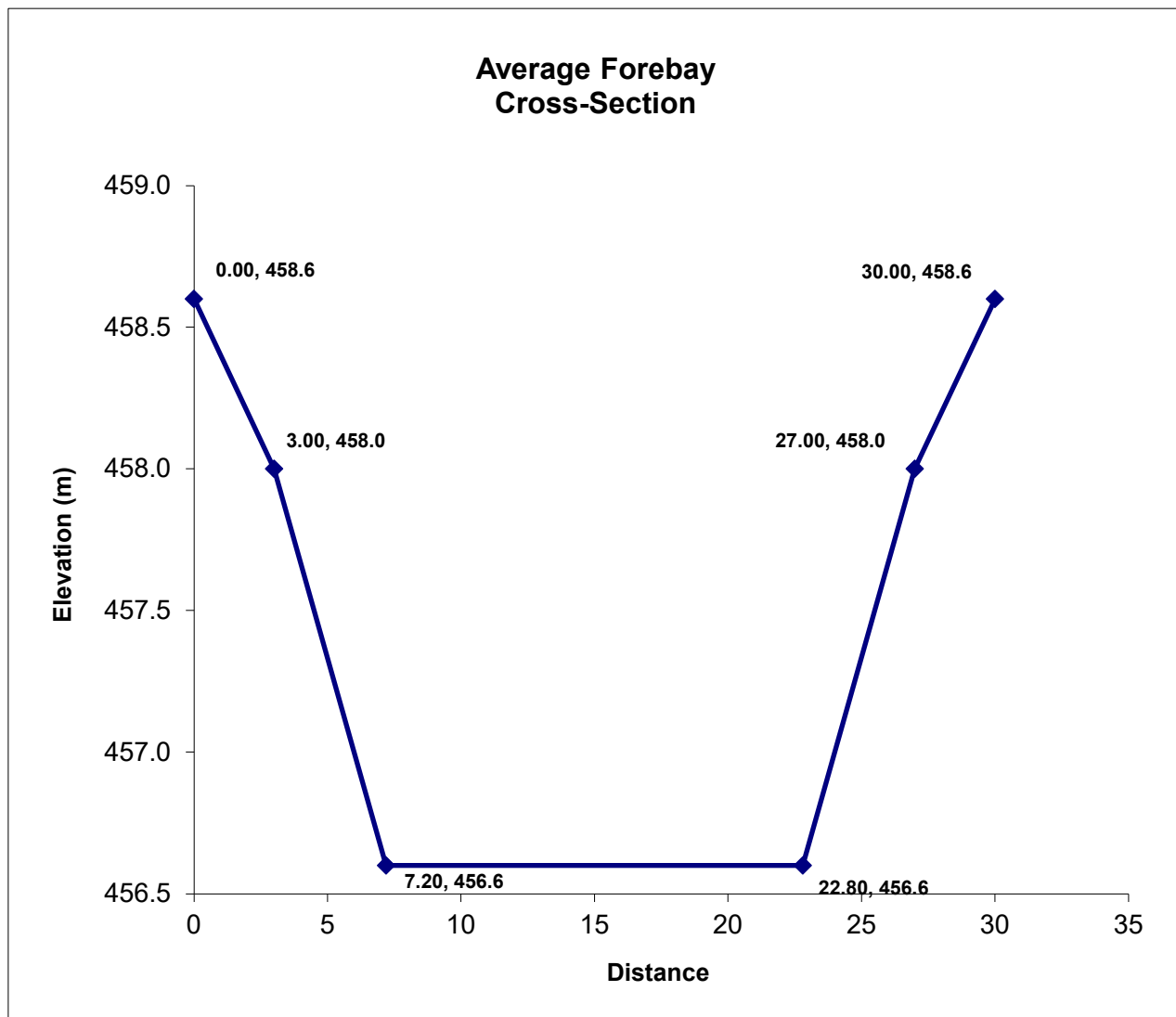
CRITERIA SATISFIED

*Value recommended by the MECP Stormwater Management Planning & Design Manual

FOREBAY SIZING CALCULATIONS WEST POND

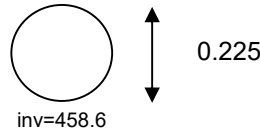
Distance (m)	Elevation (m)	Depth (m)	Incremental Area (m ²)
0.00	458.6	0.00	--
3.00	458.0	0.60	0.90
7.20	456.6	2.00	5.46
22.80	456.6	2.00	31.20
27.00	458.0	0.60	5.46
30.00	458.6	0.00	0.90

Area (m²) = 43.92



Orifice 1

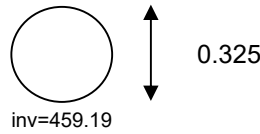
Invert = 458.60 m
 Size = 0.225 m
 Orifice Coefficient, C = 0.62
 Obvert = 458.825 m



Orifice 2

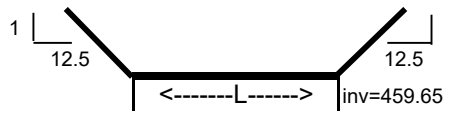
(Round Only)

Invert = 459.19 m
 Size = 0.325 m
 C = 0.80
 Obvert = 459.515 m



Broad Crested Weir (Weir 1)

Length = 75.00 m
 Elevation = 459.65 m
 Crest Breadth = 3.0 m
 Side Slope = 12.5
 (0 = vertical, 1 = 1H to 1V, 3 = 3H to 1 v)



OUTFLOW SUMMARY WEST POND

Starting Water Level (m) = 458.60
Elevation Increment (m) = 0.01

Shading represents Storage-Discharge pairings used in VO modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Orifice 2 Outflow (cms)	Weir 1 Outflow (cms)	Ditch Inlet Outflow (cms)	Backwater Elevation (m)	Stage (m)	Total Flow (cms)	Storage Below Extended Detention (m ³)	Storage (m ³)	Detention Time (hrs)	Design Storm 1
458.60	0.000	0.000	0.000	0.00	0.00	458.60	0.000	0	0	0.0	2 Year
458.61	0.000	0.000	0.000	0.00	0.00	458.61	0.000	105	0	0.0	2 Year
458.62	0.000	0.000	0.000	0.00	0.00	458.62	0.000	211	0	0.0	2 Year
458.63	0.001	0.000	0.000	0.00	0.00	458.63	0.001	316	0	0.0	2 Year
458.64	0.002	0.000	0.000	0.00	0.00	458.64	0.002	422	0	0.0	2 Year
458.65	0.002	0.000	0.000	0.00	0.00	458.65	0.002	529	0	0.0	2 Year
458.66	0.003	0.000	0.000	0.00	0.00	458.66	0.003	635	0	10.1	2 Year
458.67	0.005	0.000	0.000	0.00	0.00	458.67	0.005	742	0	17.5	2 Year
458.68	0.006	0.000	0.000	0.00	0.00	458.68	0.006	849	0	23.1	2 Year
458.69	0.007	0.000	0.000	0.00	0.00	458.69	0.007	957	0	27.6	2 Year
458.70	0.009	0.000	0.000	0.00	0.00	458.70	0.009	1065	0	31.3	2 Year
458.71	0.011	0.000	0.000	0.00	0.00	458.71	0.011	1173	0	34.4	2 Year
458.72	0.011	0.000	0.000	0.00	0.00	458.72	0.011	1281	0	37.2	2 Year
458.73	0.014	0.000	0.000	0.00	0.00	458.73	0.014	1390	0	39.7	2 Year
458.74	0.018	0.000	0.000	0.00	0.00	458.74	0.018	1498	0	41.5	2 Year
458.75	0.021	0.000	0.000	0.00	0.00	458.75	0.021	1608	0	43.1	2 Year
458.76	0.024	0.000	0.000	0.00	0.00	458.76	0.024	1717	0	44.4	2 Year
458.77	0.026	0.000	0.000	0.00	0.00	458.77	0.026	1827	0	45.6	2 Year
458.78	0.028	0.000	0.000	0.00	0.00	458.78	0.028	1937	0	46.7	2 Year
458.79	0.030	0.000	0.000	0.00	0.00	458.79	0.030	2047	0	47.8	2 Year
458.80	0.032	0.000	0.000	0.00	0.00	458.80	0.032	2158	0	48.8	2 Year
458.81	0.034	0.000	0.000	0.00	0.00	458.81	0.034	2268	0	49.7	2 Year
458.82	0.036	0.000	0.000	0.00	0.00	458.82	0.036	2380	0	50.6	2 Year
458.83	0.037	0.000	0.000	0.00	0.00	458.83	0.037	2491	0	51.4	2 Year
458.84	0.039	0.000	0.000	0.00	0.00	458.84	0.039	2603	0	52.2	Extended Detention
458.85	0.040	0.000	0.000	0.00	0.00	458.85	0.040	0	0	52.2	
458.86	0.042	0.000	0.000	0.00	0.00	458.86	0.042	0	112	53.0	
458.87	0.043	0.000	0.000	0.00	0.00	458.87	0.043	0	225	53.7	
458.88	0.045	0.000	0.000	0.00	0.00	458.88	0.045	0	338	54.4	
458.89	0.046	0.000	0.000	0.00	0.00	458.89	0.046	0	451	55.1	
458.90	0.047	0.000	0.000	0.00	0.00	458.90	0.047	0	564	55.8	
458.91	0.049	0.000	0.000	0.00	0.00	458.91	0.049	0	678	56.5	
458.92	0.050	0.000	0.000	0.00	0.00	458.92	0.050	0	792	57.1	
458.93	0.051	0.000	0.000	0.00	0.00	458.93	0.051	0	906	57.7	
458.94	0.052	0.000	0.000	0.00	0.00	458.94	0.052	0	1021	58.4	
458.95	0.053	0.000	0.000	0.00	0.00	458.95	0.053	0	1136	59.0	
458.96	0.054	0.000	0.000	0.00	0.00	458.96	0.054	0	1251	59.6	
458.97	0.055	0.000	0.000	0.00	0.00	458.97	0.055	0	1366	60.1	
458.98	0.056	0.000	0.000	0.00	0.00	458.98	0.056	0	1482	60.7	
458.99	0.058	0.000	0.000	0.00	0.00	458.99	0.058	0	1598	61.3	
459.00	0.059	0.000	0.000	0.00	0.00	459.00	0.059	0	1714	61.8	
459.01	0.060	0.000	0.000	0.00	0.00	459.01	0.060	0	1830	62.4	
459.02	0.061	0.000	0.000	0.00	0.00	459.02	0.061	0	1947	62.9	
459.03	0.062	0.000	0.000	0.00	0.00	459.03	0.062	0	2064	63.5	
459.04	0.062	0.000	0.000	0.00	0.00	459.04	0.062	0	2182	64.0	
459.05	0.063	0.000	0.000	0.00	0.00	459.05	0.063	0	2299	64.5	
459.06	0.064	0.000	0.000	0.00	0.00	459.06	0.064	0	2417	65.0	
459.07	0.065	0.000	0.000	0.00	0.00	459.07	0.065	0	2535	65.5	
459.08	0.066	0.000	0.000	0.00	0.00	459.08	0.066	0	2654	66.0	
459.09	0.067	0.000	0.000	0.00	0.00	459.09	0.067	0	2773	66.5	
459.10	0.068	0.000	0.000	0.00	0.00	459.10	0.068	0	2892	67.0	
459.11	0.069	0.000	0.000	0.00	0.00	459.11	0.069	0	3011	67.5	
459.12	0.070	0.000	0.000	0.00	0.00	459.12	0.070	0	3131	68.0	
459.13	0.071	0.000	0.000	0.00	0.00	459.13	0.071	0	3250	68.5	
459.14	0.071	0.000	0.000	0.00	0.00	459.14	0.071	0	3371	68.9	
459.15	0.072	0.000	0.000	0.00	0.00	459.15	0.072	0	3491	69.4	
459.16	0.073	0.000	0.000	0.00	0.00	459.16	0.073	0	3612	69.8	
459.17	0.074	0.000	0.000	0.00	0.00	459.17	0.074	0	3733	70.3	
459.18	0.075	0.000	0.000	0.00	0.00	459.18	0.075	0	3854	70.8	
459.19	0.075	0.000	0.000	0.00	0.00	459.19	0.075	0	3976	71.2	
459.20	0.076	0.000	0.000	0.00	0.00	459.20	0.076	0	4098	71.7	
459.21	0.077	0.000	0.000	0.00	0.00	459.21	0.077	0	4220	72.1	
459.22	0.078	0.001	0.000	0.00	0.00	459.22	0.078	0	4342	72.5	
459.23	0.079	0.002	0.000	0.00	0.00	459.23	0.079	0	4464	73.0	
459.24	0.079	0.003	0.000	0.00	0.00	459.24	0.082	0	4587	73.4	
459.25	0.080	0.004	0.000	0.00	0.00	459.25	0.084	0	4710	73.8	
459.26	0.081	0.006	0.000	0.00	0.00	459.26	0.087	0	4833	74.2	
459.27	0.082	0.007	0.000	0.00	0.00	459.27	0.089	0	4956	74.6	
459.28	0.082	0.009	0.000	0.00	0.00	459.28	0.092	0	5079	75.0	
459.29	0.083	0.011	0.000	0.00	0.00	459.29	0.094	0	5202	75.3	
459.30	0.084	0.014	0.000	0.00	0.00	459.30	0.097	0	5326	75.7	
459.31	0.084	0.016	0.000	0.00	0.00	459.31	0.100	0	5449	76.0	
459.32	0.085	0.018	0.000	0.00	0.00	459.32	0.104	0	5573	76.4	
459.33	0.086	0.021	0.000	0.00	0.00	459.33	0.107	0	5697	76.7	
459.34	0.086	0.024	0.000	0.00	0.00	459.34	0.110	0	5821	77.0	
459.35	0.087	0.027	0.000	0.00	0.00	459.35	0.114	0	5946	77.3	
459.36	0.088	0.027	0.000	0.00	0.00	459.36	0.115	0	6070	77.6	
459.37	0.089	0.039	0.000	0.00	0.00	459.37	0.127	0	6195	77.9	
459.38	0.089	0.049	0.000	0.00	0.00	459.38	0.138	0	6320	78.2	
459.39	0.090	0.057	0.000	0.00	0.00	459.39	0.147	0	6445	78.4	
459.40	0.091	0.064	0.000	0.00	0.00	459.40	0.155	0	6570	78.6	
459.41	0.091	0.070	0.000	0.00	0.00	459.41	0.162	0	6695	78.9	
459.42	0.092	0.076	0.000	0.00	0.00	459.42	0.168	0	6820	79.1	
459.43	0.092	0.082	0.000	0.00	0.00	459.43	0.174	0	6946	79.3	
459.44	0.093	0.087	0.000	0.00	0.00	459.44	0.180	0	7072	79.5	
459.45	0.094	0.092	0.000	0.00	0.00	459.45	0.186	0	7198	79.7	
459.46	0.094	0.096	0.000	0.00	0.00	459.46	0.191	0	7324	79.9	
459.47	0.095	0.101	0.000	0.00	0.00	459.47	0.196	0	7450	80.0	
459.48	0.096	0.105	0.000	0.00	0.00	459.48	0.201	0	7576	80.2	
459.49	0.096	0.109	0.000	0.00	0.00	459.49	0.205	0	7703	80.4	
459.50	0.097	0.113	0.000	0.00	0.00	459.50	0.210	0	7829	80.6	
459.51	0.098	0.117	0.000	0.00	0.00	459.51	0.214	0	7956	80.7	
459.52	0.098	0.120	0.000	0.00	0.00	459.52	0.218	0	8083	80.9	
459.53	0.099	0.124	0.000	0.00	0.00	459.53	0.223	0	8211	81.0	
459.54	0.099	0.127	0.000	0.00	0.00	459.54	0.227	0	8338	81.2	
459.55	0.100	0.131	0.000	0.00	0.00	459.55	0.231	0	8465	81.4	
459.56	0.101	0.134	0.000	0.00	0.00	459.56	0.234	0	8593	81.5	
459.57	0.101	0.137	0.000	0.00	0.00	459.57	0.238	0	8721	81.7	
459.58	0.102	0.140	0.000	0.00	0.00	459.58	0.242	0	8849	81.8	
459.59	0.102	0.143	0.000	0.00	0.00	459.59	0.246	0	8977	82.0	
459.60	0.103	0.146	0.000	0.00	0.00	459.60	0.249	0	9105	82.1	
459.61	0.103	0.149	0.000	0.00	0.00	459.61	0.253	0	9234	82.2	
459.62	0.104	0.152	0.000	0.00	0.00	459.62	0.256	0	9362	82.4	
459.63	0.105	0.155	0.000	0.00	0.00	459.63	0.259	0	9491	82.5	
459.64	0.105	0.158	0.000	0.00	0.00	459.64					

OUTFLOW SUMMARY WEST POND

Starting Water Level (m) = 458.60
Elevation Increment (m) = 0.01

Shading represents Storage-Discharge pairings used in VO modelling

Upstream Elevation (m)	Orifice 1 Outflow (cms)	Orifice 2 Outflow (cms)	Weir 1 Outflow (cms)	Ditch Inlet Outflow (cms)	Backwater Elevation (m)	Stage (m)	Total Flow (cms)	Storage Below Extended Detention (m ³)	Storage (m ³)	Detention Time (hrs)	Design Storm 1
459.65	0.106	0.160	0.000	0.00	0.00	459.65	0.266	0	9749	82.8	Emergency Spillway
459.66	0.106	0.163	0.106	0.00	0.00	459.66	0.375	0	9678	82.9	
459.67	0.107	0.166	0.300	0.00	0.00	459.67	0.573	0	10008	83.0	
459.68	0.107	0.168	0.552	0.00	0.00	459.68	0.828	0	10137	83.0	
459.69	0.108	0.171	0.852	0.00	0.00	459.69	1.130	0	10267	83.1	
459.70	0.109	0.173	1.192	0.00	0.00	459.70	1.474	0	10397	83.1	
459.71	0.109	0.176	1.570	0.00	0.00	459.71	1.855	0	10527	83.1	
459.72	0.110	0.178	1.981	0.00	0.00	459.72	2.269	0	10657	83.1	
459.73	0.110	0.181	2.425	0.00	0.00	459.73	2.716	0	10787	83.1	
459.74	0.111	0.183	2.898	0.00	0.00	459.74	3.192	0	10918	83.2	
459.75	0.111	0.185	3.400	0.00	0.00	459.75	3.696	0	11048	83.2	
459.76	0.112	0.188	3.929	0.00	0.00	459.76	4.228	0	11179	83.2	
459.77	0.112	0.190	4.484	0.00	0.00	459.77	4.786	0	11310	83.2	
459.78	0.113	0.192	5.064	0.00	0.00	459.78	5.369	0	11441	83.2	
459.79	0.113	0.194	5.669	0.00	0.00	459.79	5.977	0	11573	83.2	
459.80	0.114	0.197	6.297	0.00	0.00	459.80	6.608	0	11704	83.2	
459.81	0.114	0.199	6.948	0.00	0.00	459.81	7.262	0	11836	83.2	
459.82	0.115	0.201	7.622	0.00	0.00	459.82	7.938	0	11967	83.2	
459.83	0.115	0.203	8.318	0.00	0.00	459.83	8.637	0	12099	83.2	
459.84	0.116	0.205	9.035	0.00	0.00	459.84	9.357	0	12231	83.2	
459.85	0.116	0.207	10.328	0.00	0.00	459.85	10.652	0	12364	83.2	
459.86	0.117	0.209	11.131	0.00	0.00	459.86	11.457	0	12496	83.2	
459.87	0.117	0.211	11.954	0.00	0.00	459.87	12.283	0	12629	83.2	
459.88	0.118	0.214	12.799	0.00	0.00	459.88	13.130	0	12761	83.2	
459.89	0.118	0.216	13.665	0.00	0.00	459.89	13.999	0	12894	83.2	
459.90	0.119	0.218	14.551	0.00	0.00	459.90	14.887	0	13027	83.2	
459.91	0.119	0.219	15.457	0.00	0.00	459.91	15.796	0	13160	83.2	
459.92	0.120	0.221	16.384	0.00	0.00	459.92	16.725	0	13294	83.2	
459.93	0.120	0.223	17.330	0.00	0.00	459.93	17.674	0	13427	83.3	
459.94	0.121	0.225	18.295	0.00	0.00	459.94	18.642	0	13561	83.3	
459.95	0.121	0.227	19.151	0.00	0.00	459.95	19.500	0	13695	83.3	
459.96	0.122	0.229	20.149	0.00	0.00	459.96	20.500	0	13829	83.3	
459.97	0.122	0.231	21.165	0.00	0.00	459.97	21.518	0	13963	83.3	
459.98	0.123	0.233	22.200	0.00	0.00	459.98	22.555	0	14097	83.3	
459.99	0.123	0.235	23.253	0.00	0.00	459.99	23.611	0	14232	83.3	
460.00	0.124	0.237	24.325	0.00	0.00	460.00	24.685	0	14366	83.3	
460.01	0.124	0.238	25.415	0.00	0.00	460.01	25.777	0	14501	83.3	
460.02	0.125	0.240	26.522	0.00	0.00	460.02	26.887	0	14636	83.3	
460.03	0.125	0.242	27.648	0.00	0.00	460.03	28.016	0	14771	83.3	
460.04	0.126	0.244	28.792	0.00	0.00	460.04	29.161	0	14906	83.3	
460.05	0.126	0.246	29.953	0.00	0.00	460.05	30.325	0	15042	83.3	
460.06	0.127	0.247	31.132	0.00	0.00	460.06	31.506	0	15177	83.3	
460.07	0.127	0.249	32.328	0.00	0.00	460.07	32.704	0	15313	83.3	
460.08	0.128	0.251	33.542	0.00	0.00	460.08	33.920	0	15449	83.3	
460.09	0.128	0.252	34.773	0.00	0.00	460.09	35.153	0	15585	83.3	
460.10	0.129	0.254	36.021	0.00	0.00	460.10	36.403	0	15721	83.3	
460.11	0.129	0.256	37.286	0.00	0.00	460.11	37.671	0	15858	83.3	
460.12	0.130	0.258	38.568	0.00	0.00	460.12	38.955	0	15994	83.3	
460.13	0.130	0.259	39.867	0.00	0.00	460.13	40.256	0	16131	83.3	
460.14	0.130	0.261	41.182	0.00	0.00	460.14	41.574	0	16268	83.3	
460.15	0.131	0.263	41.940	0.00	0.00	460.15	42.334	0	16405	83.3	
460.16	0.131	0.264	43.271	0.00	0.00	460.16	43.667	0	16542	83.3	
460.17	0.132	0.266	44.619	0.00	0.00	460.17	45.016	0	16679	83.3	
460.18	0.132	0.267	45.982	0.00	0.00	460.18	46.382	0	16817	83.3	
460.19	0.133	0.269	47.362	0.00	0.00	460.19	47.764	0	16954	83.3	
460.20	0.133	0.271	48.758	0.00	0.00	460.20	49.162	0	17092	83.3	
460.21	0.134	0.272	50.171	0.00	0.00	460.21	50.576	0	17230	83.3	
460.22	0.134	0.274	51.599	0.00	0.00	460.22	52.007	0	17368	83.3	
460.23	0.135	0.275	53.043	0.00	0.00	460.23	53.453	0	17506	83.3	
460.24	0.135	0.277	54.504	0.00	0.00	460.24	54.916	0	17645	83.3	
460.25	0.135	0.278	55.980	0.00	0.00	460.25	56.394	0	17783	83.3	
460.26	0.136	0.280	57.472	0.00	0.00	460.26	57.888	0	17922	83.3	
460.27	0.136	0.282	58.980	0.00	0.00	460.27	59.398	0	18061	83.3	
460.28	0.137	0.283	60.504	0.00	0.00	460.28	60.924	0	18200	83.3	
460.29	0.137	0.285	62.044	0.00	0.00	460.29	62.466	0	18339	83.3	
460.30	0.138	0.286	63.600	0.00	0.00	460.30	64.023	0	18479	83.3	
460.31	0.138	0.288	65.171	0.00	0.00	460.31	65.596	0	18618	83.3	
460.32	0.138	0.289	66.758	0.00	0.00	460.32	67.185	0	18758	83.3	
460.33	0.139	0.291	68.360	0.00	0.00	460.33	68.790	0	18898	83.3	
460.34	0.139	0.292	69.978	0.00	0.00	460.34	70.410	0	19038	83.3	
460.35	0.140	0.294	71.612	0.00	0.00	460.35	72.045	0	19178	83.3	
460.36	0.140	0.295	73.261	0.00	0.00	460.36	73.696	0	19318	83.3	
460.37	0.141	0.297	74.926	0.00	0.00	460.37	75.363	0	19459	83.3	
460.38	0.141	0.298	76.606	0.00	0.00	460.38	77.045	0	19599	83.3	
460.39	0.141	0.299	78.302	0.00	0.00	460.39	78.742	0	19740	83.3	
460.40	0.142	0.301	80.013	0.00	0.00	460.40	80.455	0	19881	83.3	
460.41	0.142	0.302	81.739	0.00	0.00	460.41	82.184	0	20022	83.3	
460.42	0.143	0.304	83.481	0.00	0.00	460.42	83.927	0	20163	83.3	
460.43	0.143	0.305	85.238	0.00	0.00	460.43	85.686	0	20305	83.3	
460.44	0.144	0.307	87.011	0.00	0.00	460.44	87.461	0	20446	83.3	
460.45	0.144	0.308	88.799	0.00	0.00	460.45	89.251	0	20588	83.3	
460.46	0.144	0.309	90.602	0.00	0.00	460.46	91.056	0	20730	83.3	
460.47	0.145	0.311	92.420	0.00	0.00	460.47	92.876	0	20872	83.3	
460.48	0.145	0.312	94.254	0.00	0.00	460.48	94.712	0	21014	83.3	
460.49	0.146	0.314	96.103	0.00	0.00	460.49	96.562	0	21157	83.3	
460.50	0.146	0.315	97.967	0.00	0.00	460.50	98.428	0	21299	83.3	
460.51	0.146	0.316	99.847	0.00	0.00	460.51	100.310	0	21442	83.3	
460.52	0.147	0.318	101.742	0.00	0.00	460.52	102.206	0	21586	83.3	
460.53	0.147	0.319	103.651	0.00	0.00	460.53	104.118	0	21728	83.3	
460.54	0.148	0.320	105.576	0.00	0.00	460.54	106.044	0	21871	83.3	
460.55	0.148	0.322	107.517	0.00	0.00	460.55	107.986	0	22014	83.3	
460.56	0.148	0.323	109.472	0.00	0.00	460.56	109.943	0	22158	83.3	
460.57	0.149	0.324	111.442	0.00	0.00	460.57	111.916	0	22302	83.3	
460.58	0.149	0.326	113.428	0.00	0.00	460.58	113.903	0	22445	83.3	
460.59	0.150	0.327	115.429	0.00	0.00	460.59	115.905	0	22589	83.3	
460.60	0.150	0.328	117.445	0.00	0.00	460.60	117.923	0	22733	83.3	

Appendix F Hydrologic Modelling



Appendix F1 Hydrology Parameters

Proposed Conditions VO Parameter Summary

NASHYD

Number	301	316	317	318	401	402	403
Description							
DT(min)	1	1	1	1	1	1	1
Area (ha)	0.27	1.97	2.62	1.32	64.15	93.38	0.63
CN*	73.0	72.0	72.0	82.0	88.0	86.0	86.0
CN (AMC III)	88.0	86.0	86.0	90.0	93.0	92.0	92.0
IA(mm)	5.0	8.0	8.0	9.0	7.1	7.9	8.0
TP Method	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands
TP (hr)	0.03	0.36	0.36	0.14	2.48	2.24	0.04

STANDHYD

Number	302	303	304	305	306	307	308	309	310	311	312	313	314	315
Description														
DT(min)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Area (ha)	1.92	0.5	11.12	1.5	0.32	0.24	0.76	4.83	0.35	0.5	2.45	0.6	1.38	22.70
TIMP ²	0.50	0.78	0.60	0.63	0.62	0.67	0.52	0.65	0.48	0.67	0.50	0.67	0.67	0.60
XIMP ^{1,2}	0.50	0.78	0.60	0.62	0.61	0.67	0.51	0.65	0.47	0.67	0.50	0.67	0.67	0.59
CN*	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0
CN (AMC III)	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0	88.0
IA(mm)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
SLPP(%)	2	2	2	2	2	2	2	2	2	2	2	2	2	2
LGPM(m)	40	40	40	40	40	40	40	40	40	40	40	40	40	40
MNP	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
DPSI (mm)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
SLPI(%)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
LGI(m)	113.14	57.74	272.27	100.00	46.19	40.00	71.18	179.44	48.30	57.74	127.80	63.25	95.92	389.01
MNI	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013

¹Note that where there is NO directly connected area (ie: roof runoff to grassed areas), the hydrology program does not accept XIMP=0%, therefore, XIMP = 1% has been used

²Note that where there is NO pervious area, the hydrology program does not accept TIMP and XIMP=100%, therefore, TIMP and XIMP = 99% has been use

Total Area = 129.4 ha

Proposed Conditions CN Calculations

Site Soils: (per Ag Maps)

Soil Type

Perth Silt Loam

Hydrologic Soil Group

C

TABLE OF CURVE NUMBERS (CN's)**										
Land Use	Hydrologic Soil Type								Manning's 'n'	Source
	A	AB	B	BC	C	CD	D			
Meadow "Good"	30	44	58	64.5	71	74.5	78	0.40	MTO	
Woodlot "Fair"	36	48	60	66.5	73	76	79	0.40	MTO	
Gravel	76	80.5	85	87	89	90	91	0.30	USDA	
Lawns "Good"	39	50	61	67.5	74	77	80	0.25	USDA	
Pasture/Range	58	61.5	65	70.5	76	78.5	81	0.17	MTO	
Crop	66	70	74	78	82	84	86	0.13	MTO	
Fallow (Bare)	77	82	86	89	91	93	94	0.05	MTO	
Low Density Residences	57	64.5	72	76.5	81	83.5	86	0.25	USDA	
Streets, paved	98	98	98	98	98	98	98	0.01	USDA	

1. MTO Drainage Manual (1997), Design Chart 1.09-Soil/Land Use Curve Numbers
2. USDA (1986), Urban Hydrology for Small Watersheds, Table 2.2-Runoff Curve Numbers for Urban Areas

HYDROLOGIC SOIL TYPE (%) - Proposed Conditions								
Catchment	Hydrologic Soil Type							TOTAL
	A	AB	B	BC	C	CD	D	
301					100			100
316					100			100
317					100			100
318					100			100
401					100			100
402					100			100
403					100			100
302					100			100
303					100			100
304					100			100
305					100			100
306					100			100
307					100			100
308					100			100
309					100			100
310					100			100
311					100			100
312					100			100
313					100			100
314					100			100
315					100			100

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
301				100.0						100.0
316	100.0									100.0
317	100.0									100.0
318		49.2				50.8				100.0
401		2.1	8.9	3.8		80.6		4.6		100.0
402		1.4	1.1			97.1		0.4		100.0
403						100.0				100.0
302				100.0						100.0
303				100.0						100.0
304				100.0						100.0
305				100.0						100.0
306				100.0						100.0
307				100.0						100.0
308				100.0						100.0
309				100.0						100.0
310				100.0						100.0
311				100.0						100.0
312				100.0						100.0
313				100.0						100.0
314				100.0						100.0
315				100.0						100.0

Note: Where STANDHYD command used (shaded), impervious fraction is not considered in CN determination, since %Imp directly input in STANDHYD command



Proposed Conditions CN Calculations

Arthur Wellington North Development
 Project Number: 2504
 Date: February 2025
 Designer Initials: J.H.

CURVE NUMBER (CN) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
301	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
316	71.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	71
317	71.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	71
318	0.0	35.9	0.0	0.0	0.0	41.7	0.0	0.0	0.0	78
401	0.0	1.5	7.9	2.8	0.0	66.1	0.0	0.0	4.5	83
402	0.0	1.0	1.0	0.0	0.0	79.6	0.0	0.0	0.4	82
403	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82
302	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
303	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
304	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
305	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
306	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
307	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
308	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
309	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
310	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
311	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
312	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
313	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
314	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
315	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74

** AMC II assumed



Proposed Conditions CN Calculations

Arthur Wellington North Development
 Project Number: 2504
 Date: February 2025
 Designer Initials: J.H.

Input Values									
Subcatchment:	301		316	317	318	401	402	403	
CN (AMC II):	74		71	71	78	83	82	82	
CN (AMC III) =	88		86	86	90	93	92	92	
100 Year Precipitation, P =	71.5	mm	71.5	71.5	71.5	71.5	71.5	71.5	

Proposed Conditions IA Calculations

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
301				100.0						100.0
316	100.0									100.0
317	100.0									100.0
318		49.2				50.8				100.0
401		2.1	8.9	3.8		80.6			4.6	100.0
402		1.4	1.1			97.1			0.4	100.0
403						100.0				100.0
302				100.0						100.0
303				100.0						100.0
304				100.0						100.0
305				100.0						100.0
306				100.0						100.0
307				100.0						100.0
308				100.0						100.0
309				100.0						100.0
310				100.0						100.0
311				100.0						100.0
312				100.0						100.0
313				100.0						100.0
314				100.0						100.0
315				100.0						100.0

IA VALUES (mm) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
IA (mm)	8	10	2	5	8	8	3	2	2	
301				5.0						5.0
316	8.0									8.0
317	8.0									8.0
318		4.9				4.1				9.0
401		0.2	0.2	0.2		6.5			0.1	7.1
402		0.1	0.0			7.8			0.0	7.9
403						8.0				8.0
302				5.0						5.0
303				5.0						5.0
304				5.0						5.0
305				5.0						5.0
306				5.0						5.0
307				5.0						5.0
308				5.0						5.0
309				5.0						5.0
310				5.0						5.0
311				5.0						5.0
312				5.0						5.0
313				5.0						5.0
314				5.0						5.0
315				5.0						5.0

* IA values based on TRCA guidelines

Proposed Conditions Time to Peak Calculations

Uplands Method:

Catchment ID	High Elevation	Low Elevation	Length (m)	Slope (%)	Land Cover Type	Velocity (m/s)	Time of Concentration (s)	Time of Concentration (hr)	Time to Peak (hr)
301a	460.19	458.80	60	2.32	Pasture	0.33	180.6	0.05	0.03
301									0.03
316a	462.13	459.48	607	0.44	Waterway	0.32	1921.5	0.53	0.36
316									0.36
317a	462.13	459.48	607	0.44	Waterway	0.32	1921.5	0.53	0.36
317									0.36
318a	461.81	459.63	59	3.69	Pasture	0.42	140.3	0.04	0.03
318b	459.63	458.77	199	0.43	Waterway	0.31	633.1	0.18	0.12
318									0.14
401a	474.50	471.95	147	1.73	Cultivated Straight Row	0.37	399.8	0.11	0.07
401b	471.95	469.98	317	0.62	Cultivated Straight Row	0.22	1436.3	0.40	0.27
401c	469.98	466.16	912	0.42	Cultivated Straight Row	0.18	5027.7	1.40	0.94
401d	466.16	465.25	671	0.14	Cultivated Straight Row	0.10	6480.3	1.80	1.21
401									2.48
402a	473.78	468.50	430	1.23	Cultivated Straight Row	0.31	1388.7	0.39	0.26
402b	468.50	463.00	1424	0.39	Cultivated Straight Row	0.17	8173.2	2.27	1.52
402c	463.00	462.13	347	0.25	Cultivated Straight Row	0.14	2469.0	0.69	0.46
402									2.24
403a	461.97	460.31	86	1.93	Cultivated Straight Row	0.39	221.8	0.06	0.04
403									0.04

Proposed Conditions Percent Impervious Calculations

			StandHyd IDs														
			302	303	304	305	306	307	308	309	310	311	312	313		314	315
Catchment Area (ha)			1.92	0.5	11.12	1.50	0.32	0.24	0.76	4.83	0.35	0.5	2.45	0.6	1.38	22.70	
Land Use Areas	Timp	Ximp	Land Use Areas												Total		
Parks	17%	13%			1.16	0.23	0.08		0.28		0.13					1.63	3.51
SWM Pond	50%	50%	1.92										2.45				4.37
Single Houses	67%	67%			2.37			0.24	0.20	0.80		0.50		0.60	0.75	14.65	
20m ROW	54%	54%			3.32	0.35				1.87					6.42	11.96	
Semi-detached	67%	67%			1.63					0.73	0.22				0.63		3.21
Townhouses	78%	78%		0.50	2.64	0.92	0.24		0.28	1.43						6.01	
Total Land Use =			1.92	0.50	11.12	1.50	0.32	0.24	0.76	4.83	0.35	0.50	2.45	0.60	1.38	22.70	49.17
Timp =			50%	78%	60%	63%	62%	67%	52%	65%	48%	67%	50%	67%	67%	60%	33%
Ximp =			50%	78%	60%	62%	61%	67%	51%	65%	47%	67%	50%	67%	67%	59%	32%

Proposed Conditions VO Parameter Summary

NASHYD

Number	101	102	103	104	105	201	202
Description							
DT(min)	1	1	1	1	1	1	1
Area (ha)	29.43	8.01	13.30	1.03	3.84	157.60	0.63
CN	82.0	82.0	82.0	82.0	82.0	82.0	82.0
CN (AMC III)	92.0	92.0	92.0	92.0	92.0	92.0	92.0
IA(mm)	4.1	4.7	4.0	4.0	4.0	4.0	4.0
TP Method	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands	Uplands
TP (hr)	0.56	0.33	0.34	0.23	0.20	2.24	0.04

Total Area = 213.2 ha

Site Soils: (per Ag Maps)

Soil Type

Perth Silt Loam

Hydrologic Soil Group

C

TABLE OF CURVE NUMBERS (CN's)**										
Land Use	Hydrologic Soil Type								Manning's 'n'	Source
	A	AB	B	BC	C	CD	D			
Meadow "Good"	30	44	58	64.5	71	74.5	78	0.40	MTO	
Woodlot "Fair"	36	48	60	66.5	73	76	79	0.40	MTO	
Gravel	76	80.5	85	87	89	90	91	0.30	USDA	
Lawns "Good"	39	50	61	67.5	74	77	80	0.25	USDA	
Pasture/Range	58	61.5	65	70.5	76	78.5	81	0.17	MTO	
Crop	66	70	74	78	82	84	86	0.13	MTO	
Fallow (Bare)	77	82	86	89	91	93	94	0.05	MTO	
Low Density Residences	57	64.5	72	76.5	81	83.5	86	0.25	USDA	
Streets, paved	98	98	98	98	98	98	98	0.01	USDA	

1. MTO Drainage Manual (1997), Design Chart 1.09-Soil/Land Use Curve Numbers
2. USDA (1986), Urban Hydrology for Small Watersheds, Table 2.2-Runoff Curve Numbers for Urban Areas

HYDROLOGIC SOIL TYPE (%) - Existing Conditions								
Catchment	Hydrologic Soil Type							TOTAL
	A	AB	B	BC	C	CD	D	
101					100			100
102					100			100
103					100			100
104					100			100
105					100			100
201					100			100
202					100			100

LAND USE (%) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101		1.9				97.6			0.4	100.0
102		15.9	5.6			72.7			5.8	100.0
103						100.0				100.0
104						100.0				100.0
105						100.0				100.0
201		1.7	4.3	1.5		90.4			2.1	100.0
202						100.0				100.0

Note: Where STANDHYD command used (shaded), impervious fraction is not considered in CN determination, since %Imp directly input in STANDHYD command

CURVE NUMBER (CN) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
101	0.0	1.4	0.0	0.0	0.0	80.1	0.0	0.0	0.4	82
102	0.0	11.6	4.9	0.0	0.0	59.6	0.0	0.0	5.7	82
103	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82
104	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82
105	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82
201	0.0	1.2	3.8	1.1	0.0	74.1	0.0	0.0	2.1	82
202	0.0	0.0	0.0	0.0	0.0	82.0	0.0	0.0	0.0	82

** AMC II assumed



Proposed Conditions CN Calculations

Arthur Wellington North Development
 Project Number: 2504
 Date: February 2025
 Designer Initials: K.T.

Input Values									
Subcatchment:	101		102	103	104	105	201	202	
CN (AMC II):	82		82	82	82	82	82	82	82
CN (AMC III) =	92		92	92	92	92	92	92	92
100 Year Precipitation, P =	71.5	mm	71.5	71.5	71.5	71.5	71.5	71.5	71.5

Existing Conditions IA Calculations

LAND USE (%) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101		1.9				97.6			0.4	100.0
102		15.9	5.6			72.7			5.8	100.0
103						100.0				100.0
104						100.0				100.0
105		0.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0
201		1.7	4.3	1.5		90.4			2.1	100.0
202						100.0				100.0

IA VALUES (mm) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
IA (mm)	8	10	2	5	8	4	3	2	2	
101		0.2				3.9			0.0	4.1
102		1.6	0.1			2.9			0.1	4.7
103						4.0				4.0
104						4.0				4.0
105		0.0	0.0	0.0	0.0	4.0	0.0	0.0	0.0	4.0
201		0.2	0.1	0.1		3.6			0.0	4.0
202						4.0				4.0

* IA values based on CVC guidelines

Existing Conditions Time to Peak Calculations

Uplands Method:

Catchment ID	High Elevation	Low Elevation	Length (m)	Slope (%)	Land Cover Type	Velocity (m/s)	Time of Concentration (s)	Time of Concentration (hr)	Time to Peak (hr)
101a	465.50	461.70	388	0.98	Cultivated Straight Row	0.28	1402.1	0.39	0.26
101b	461.70	459.89	477	0.38	Waterway	0.30	1615.8	0.45	0.30
101									0.56
102a	462.13	459.89	542	0.41	Waterway	0.31	1761.8	0.49	0.33
102									0.33
103a	463.81	458.51	519	1.02	Cultivated Straight Row	0.28	1837.0	0.51	0.34
103									0.34
104a	464.86	461.73	333	0.94	Cultivated Straight Row	0.27	1228.2	0.34	0.23
104									0.23
105a	463.81	459.63	187	2.24	Cultivated Straight Row	0.42	448.3	0.12	0.08
105b	459.63	458.77	199	0.43	Waterway	0.31	633.1	0.18	0.12
105									0.20
201a	473.78	468.50	430	1.23	Cultivated Straight Row	0.31	1388.7	0.39	0.26
201b	468.50	463.00	1424	0.39	Cultivated Straight Row	0.17	8173.2	2.27	1.52
201c	463.00	462.13	347	0.25	Cultivated Straight Row	0.14	2469.0	0.69	0.46
201									2.24
202a	461.97	460.31	86	1.93	Cultivated Straight Row	0.39	221.8	0.06	0.04
202									0.04

Appendix F2 Hydrologic Modelling

Digital Report and Modelling Files

The following FileSafe Cloud link includes:

665 Eliza Street, Arthur Visual Otthymo 6.2 Modelling – Functional Servicing and Stormwater Management Report (February 2025)

Contents:

<https://filesafecloud.scsconsultinggroup.com/url/nrxir2xgce32vjfg>



Appendix G Sanitary Flow Calculations



<u>Proposed Sanitary Flow Calculations</u>	
Average Flow Rate ¹	400 litres/capita/day
Equivalent Residential Units	406 units
Residential Population (2.7 persons/unit) ²	1,096 persons
Peaking Factor	3.77
Peak Flow	19.153 L/s
Total Site Area	23.723 ha
Infiltration (0.15 litres/second/ha)	3.559 L/s
Total Proposed Peak Sanitary Flow	22.71 L/s
Total Proposed Average Daily Flow	438.48 m³/day

¹ MECP recommended per person flow rate of 400 L/C/D per Arthur Water & Sanitary Systems Technical Study (Triton, 2020)

² Capita per ERU per Wellington North Community Growth Plan (GSP Group Inc., 2018)

<u>Proposed Sanitary Flow Calculations</u>	
Average Flow Rate ¹	400 litres/capita/day
Equivalent Residential Units	435 units
Residential Population (2.7 persons/unit) ²	1,175 persons
Peaking Factor	3.75
Peak Flow	20.412 L/s
Total Site Area	20.942 ha
Infiltration (0.15 litres/second/ha)	3.141 L/s
Total Proposed Peak Sanitary Flow	23.55 L/s
Total Proposed Average Daily Flow	469.80 m³/day

¹ MECP recommended per person flow rate of 400 L/C/D per Arthur Water & Sanitary Systems Technical Study (Triton, 2020)

² Capita per ERU per Wellington North Community Growth Plan (GSP Group Inc., 2018)

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