Shining Hill Estates Phase 3 (Aurora)
Town of Aurora

Functional Servicing and Stormwater Management Report

December 2021

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SUBMISSION HISTORY

Submission	Date	In Support Of	Distributed To
1st	March 2021	Re-Zoning, Official Plan Amendment, Draft Plan Approval	Town of Aurora, LSRCA, York Region, Shining Hill Estates Collection Inc.
2nd	December 2021	Re-Zoning, Official Plan Amendment, Draft Plan Approval	Town of Aurora, LSRCA, York Region, Shining Hill Estates Collection Inc.

1.0 INTRODUCTION

SCS Consulting Group Ltd. has been retained by Shining Hill Estates Collection Inc. to prepare a Functional Servicing and Stormwater Management Report for a proposed development in the Town of Aurora.

1.1 Purpose of the Functional Servicing Report

The Functional Servicing and Stormwater Management Report (FSSR) has been prepared in support of an Official Plan Amendment, Zoning Bylaw Amendment, and Plan of Subdivision applications 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,
- a neighbourhood park,
- open space,
- a private school (St. Anne's School (SAS)), and
- proposed roads.

The purpose of this report is to demonstrate that the development can be graded and serviced in accordance with the Town of Aurora, Lake Simcoe Region Conservation Authority (LSRCA), the Ontario Building Code, and the Ministry of Environment, Conservation and Parks (MECP) design criteria.

1.2 Study Area

The study area is a land assembly approximately 31.8 ha in size and is bound by St. John's Sideroad to the south, the Shining Hill Estates Phase 2 development to the southwest, a tributary of Tannery Creek to the east, existing residential development to the west, and the municipal boundary of Aurora-Newmarket to the north (see **Figure 1.1**).

The existing subject lands are comprised of estate residential uses including two dwellings, ancillary structures and open space areas.

1.3 Background Servicing Information

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

- South Georgian Bay Lake Simcoe Source Protection Plan (SGBLS SPP) (Approval Date: January 26, 2015, Effective: July 1, 2015, Amended: May 14, 2015);
- Town of Aurora Design Criteria Manual for Engineering Plans (Revised June 2021);
- Technical Guidelines for Stormwater Management Submissions, Lake Simcoe Region Conservation Authority (September, 2016);
- Phosphorus Offsetting Policy, Lake Simcoe Region Conservation Authority (July 2021);

- Phosphorus Budget Tool in Support of Sustainable Development for the Lake Simcoe Watershed (March, 2012);
- Water Budget Offsetting Policy, Lake Simcoe Region Conservation Authority (July 2021);
- Lake Simcoe Protection Plan (July 2009); and
- Ministry of Environment, Conservation and Parks (MECP) Stormwater Management Planning and Design Manual (March 2003).

The site servicing and SWM strategies are also based on the following approved Engineering Drawings as well as the following reports for this Draft Plan of Subdivision:

- St. Andrews on The Hill Engineering Drawings, Revision date March 1988, prepared by PMG Consulting Engineers;
- Hydrogeological Investigation - Revised, Shining Hill (Phase 3), 162, 306, 370, 434 & 488 St. John's Sideroad West, Aurora, Ontario, prepared by Golder Associates, dated December 20 2021;
- A Geotechnical Investigation and Slope Stability Assessment for Proposed Residential Development, Soil Engineers Ltd., November 19 2021; and
- A Geotechnical Investigation and for Proposed School Block, Soil Engineers Ltd., November 19 2021.

Excerpts from the above listed documents are included in **Appendix B**.

A Rainscaping design charette with the Town of Aurora and LSRCA was held on December 15, 2020. The meeting minutes are included in **Appendix B**.

2.0 STORMWATER MANAGEMENT

2.1 Stormwater Runoff Control Criteria

The following stormwater runoff control criteria have been established based on the greatest requirements of each of the design guidelines and standards listed in **Section 1.3**. The stormwater runoff criteria are summarized below in **Table 2.1**:

Table 2.1 – Stormwater Runoff Control Criteria

Criteria	Control Measure
Quantity Control	Peak Flow: Control proposed peak flows to existing peak flows for the 2 through 100 year storm events. (Town, LSRCA) Volume Control: Proposed runoff volume from a 25 mm rainfall event over the total impervious area shall be captured and retained/treated on-site or in accordance with LRCA's Flexible Treatment guidelines if full compliance with the 25 mm guideline is not possible. (LSRCA)
Quality Control	Total Suspended Solids: MECP Enhanced Level Protection (80% TSS Removal). (MECP, LSRCA, Town) Phosphorus: Per Lake Simcoe Protection Plan, a Phosphorus Loading Study is to be done to determine the existing and proposed phosphorus loading rates. Per the LSPOP, target 100% control and net-zero phosphorus export. (LSRCA)
Erosion Control	Detention of the 25 mm rainfall runoff for a minimum of 24 hours. (LSRCA)
Water Budget	As the site is within a Wellhead Protection Area (WHPA) Q1/Q2, maintain the existing water budget through the use of best management practices such as Low Impact Development measures. (SGBLS SPP)

2.2 Existing Drainage

The subject lands are located within the Tannery Creek Watershed in the Town of Aurora. A tributary of the Tannery Creek travels west to east along the southern limits of the property, crosses south under St. John's Sideroad, and eventually crossing back north under St. John's Sideroad where it travels south to north east of the subject lands, and ultimately east toward Yonge Street away from the subject property.

As shown on **Figure 2.1**, there are five outlets for the site that all drain to the Tannery Creek:

1. Southwest outlet via sheet flow to the tributary from the SAS site (Catchment 105 – 2.68 ha),
2. Southwest outlet via the tributary (Catchment 101 – 4.12 ha),

3. North outlet via sheet flow from the SAS site toward a drainage draw located to the north in Newmarket (Catchment 104 – 1.06 ha), and
4. North outlet toward a wetland located to the north in Newmarket (Catchment 102 – 3.62 ha), and
5. East outlet via sheet flow down the valley wall (Catchment 103 – 2.39 ha).

Drainage from the Outlets 1 and 3 are wholly from the SAS site, which the development of that block will be subject to Site Plan Control. For the purpose of this FSSR, the hydrology of those catchments will not be assessed as it will be completed through a future the Site Plan Control application.

2.2.1 Existing Site Characterization

The soil classifications were identified using the Ontario Soil Survey Complex from OMAFRA and land uses visible in recent aerial photography and verified through a site visit. The mapping identifies that the soils within the study limits are Schomberg Clay Loam. According to the Design Flood Estimation Design Chart H2-6A, the soils are considered as Hydrologic Soil Group C. This is consistent with Golder Associations Hydrogeological Investigation 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.

Golder Associates completed in-situ infiltration testing which found a range of estimated infiltration rates of 30 – 75 mm/hr. Applying a safety correction factor yields a design infiltration rates ranging from 12 – 30 mm/hr. Golder Associates completed monitoring of groundwater level across the site with readings from September 2020 to November 2021. Refer to **Appendix B** for excerpts from the Hydrogeological Assessment for the infiltration test results and groundwater monitoring results.

2.2.2 Existing Hydrologic Modelling

Hydrologic modelling was undertaken using the Visual Otthymo Version 6.0 software (VO6) based on the 4-hour Chicago, 12-hour SCS Type II, and 24-hour SCS Type II Distribution methods. The IDF rainfall information was obtained from the Town of Aurora Design Criteria Manual to determine the existing peak flows to outlet locations. The existing flows from the study area to the outlet locations are summarized in **Table 2.2**.

Table 2.2: Summary of Existing Flows

Return Period Storm	Outlet 2 (Catchment 101) (m^3/s)			Outlet 5 (Catchment 102) (m^3/s)			Outlet 4 (Catchment 103) (m^3/s)		
	4-Hour Chi	12-Hour SCS	24-Hour SCS	4-Hour Chi	12-Hour SCS	24-Hour SCS	4-Hour Chi	12-Hour SCS	24-Hour SCS
2 Year	0.051	0.081	0.092	0.109	0.177	0.196	0.090	0.138	0.151
5 Year	0.098	0.138	0.151	0.208	0.289	0.307	0.170	0.221	0.234
10 Year	0.134	0.188	0.205	0.286	0.384	0.408	0.234	0.292	0.309
25 Year	0.175	0.243	0.266	0.375	0.488	0.518	0.307	0.368	0.390
50 Year	0.228	0.286	0.302	0.483	0.567	0.582	0.392	0.426	0.437
100 Year	0.280	0.331	0.347	0.580	0.647	0.661	0.467	0.486	0.494

A summary of modelling parameters and an existing VO6 schematic are provided in **Appendix C**. A USB drive containing the VO6 hydrology model is also provided in **Appendix C**, or available on request via file transfer.

2.3 Proposed Storm Drainage

The proposed storm drainage plan is shown on **Figure 2.2**, while the proposed servicing plan is shown on **Figure 2.3**. Impervious coverage was estimated based on the maximum impervious areas using the anticipated zoning, and is illustrated on **Figure 2.4**.

<u>Lot Level Drainage</u>

Split draining lots will use a rear yard infiltration trench to infiltrate runoff from the back half of the roofs where 1 m of separation to the high groundwater level can be provided. Infiltration measure are required by the Ontario Building Code to be a minimum of 5 m from a foundation. The front yard setbacks are 4.5 m per the zoning bylaw which eliminates the possibility for infiltration measures in the front yard for runoff from the front half of the roofs and driveways. Therefore, infiltration measures for the front half of the roofs and driveways can only be located in the road right-of-way or end-of-pipe.

All roof downspouts are to drain to grassed areas.

<u>Outlets 1 and 3 – SAS Site</u>

Runoff to Outlet's 1 and 3 will not be modified as part of the subdivision development. Future development of the site will be subject to Site Plan Control, and the proposed development will have to demonstrate compliance with all of the stormwater runoff control criteria.

<u>Outlet 2</u>

Clean runoff from 0.56 ha of rear & front yards (Catchment 207), and major system runoff from 0.42 ha (Catchment 204) will drain via overland flow directly to Outlet 2. Runoff from approximately 2.33 ha (Catchment 206) will be captured by the storm sewer system, controlled to the stormwater runoff control criteria using low impact development (LID) measures within the municipal road right-of-way and superpipes and conveyed to Outlet 2 via a storm sewer.

The capacity of the St. John's Sideroad ditch will be assessed at detailed design to confirm there is adequate capacity. It is noted that there is a small drainage area to the ditch because there is a high point in the ditch approximately 50 m west of the proposed Street A intersection.

Outlet 4

Clean runoff from 0.23 ha of rear & front yards (Catchment 205) will drain via overland flow directly to Outlet 4. Major and minor system runoff from 2.38 ha (Catchment 203), and major system runoff from 0.47 ha (Catchment 202) will be captured by the storm sewer system, controlled to the stormwater runoff control criteria using LIDs within the municipal road right-of-way and an superpipes and conveyed to Outlet 4 via a storm sewer draining east, located within a municipal easement, north of the St. John's Sideroad right-of-way, discharging at the bottom of the valley wall to the Tannery Creek tributary.

The capacity of the St. John's Sideroad ditch will be assessed at detailed design to confirm there is adequate capacity. It is noted that there is a small drainage area to the ditch because there is a high point in the ditch approximately 50 m west of the proposed Street A intersection.

Outlet 5

Major and minor system runoff from 3.53 ha (Catchments EXT1, 201 and 208), and minor system runoff from 0.47 ha (Catchment 202) will be captured by the storm sewer system, controlled to the stormwater runoff control criteria using LIDs and an end of pipe underground stormwater management facility located under the Neighbourhood Park and conveyed to Outlet 5 via a storm sewer discharging to the Tannery Creek. The runoff from the SAS site that drains to Outlet 5 (Catchment EXT1) will be accommodated for in the end-of-pipe SWM facility for peak flow control, but will be required to provide on-site volume control, quality control (TSS, phosphorus), erosion control, and water balance. Runoff from 0.16 ha (Catchment 209) will drain via overland flow to Outlet 5.

2.4 Best Management Practices

In accordance with the Ministry of Environment Stormwater Management Planning and Design Manual (2003) and LSRCA objectives, a review of stormwater management LID measures and best management practices (BMP) was completed. The review included a focus on the treatment train approach, evaluating lot level, conveyance system and end-of-pipe practices.

As part of the review of the LIDs, a "RainScaping" design charrette meeting was held on December 15, 2020. The RainScaping charrette was a meeting with the Town of Aurora, Town of Newmarket, and LSRCA staff, as well as the applicant and the applicant's consultants to discuss, review and develop LID strategies, opportunities and constraints for the subject development.

The meeting minutes from the RainScaping charrette are included in **Appendix B**. **Table 2.3** notes the various LIDs and whether they are recommended.

Table 2.3 – Recommended Stormwater LID & BMP Practices

STORMWATER MANAGEMENT PRACTICE	RECOMMENDED (Yes/No)
Reduced Lot Grading	Yes
Increased Topsoil Depth	Yes
Passive Landscaping/Bio-Retention	No
Roof Leader to Rear Yard Infiltration Trenches	Yes
Roof Runoff to Retention Cisterns	No
Green Roofs	No
Rooftop and/or Parking Lot Detention Storage	No
Roof overflow to Grassed Areas	Yes
Pervious Pavement	No
Vegetated Filter Strips	No
Bioswale/Rain Garden	Yes
Exfiltration at Rear Lot Catchbasins	No
Street Catchbasin Infiltration/Filtration System	Yes
Underground Stormwater Detention Facility	Yes
Wet Ponds, Wetlands, Dry Ponds	No

Reduced Lot Grading – Reducing lot grades from a maximum of 5% to a minimum of 2% is suggested wherever possible to maximize infiltration and evapotranspiration of stormwater runoff at the lot level.

Increased Topsoil Depth – A minimum topsoil restoration depth of 0.3 meters is proposed in all landscaped areas.

Roof overflow to Grassed Areas –Roof leaders can be directed to grassed areas where there is grass.

Bioswale/Rain Garden – A grassed swale in the boulevard to receive street runoff is proposed running parallel to single loaded roads/laneways and roads without driveway access.

Roof Runoff to Rear Yard Infiltration Trenches – Directing roof runoff to subsurface infiltration trenches can be used to promote infiltration. By promoting infiltration water quality and quantity control is provided for the volume of water retained. Infiltration of roof runoff can provide a significant SWM benefits as part of the overall treatment train approach for the proposed development. All split draining lots will use a rear yard infiltration trenches to infiltrate runoff from the back half of the roofs. Infiltration measure are required by the Ontario

Building Code to be a minimum of 5 m from a foundation. The front yard setbacks are 4.5 m per the zoning bylaw which eliminates the possibility for infiltration measures in the front yard for runoff from the front half of the roofs and driveways.

Street Catchbasin Infiltration/Filtration System – Proposed to treat runoff from the street, there will be a connection from the street catchbasins to an infiltration or filtration trench (groundwater dependent) located in the road boulevard. Where feasible, the infiltration/filtration trenches will be sized for the volume control or water quality control criteria, whichever is a greater volume. Preliminary sizing is discussed further in **Section 2.6**.

Wet Ponds, Wetlands, Dry Ponds, Underground Storage – As discussed during the RainsScaping design charette, wet ponds are discouraged by the LSRCA. Underground storage systems are preferred to be located under park areas to utilize dual land uses. Underground storage will be utilized under the neighbourhood park at achieve the peak flow and erosion control criteria.

Superpipes – To meet quantity and erosion control targets, stormwater storage will be provided by the use of superpipes prior to discharging to the drainage outlets.

Manufactured Treatment Device - A properly sized manufactured treatment device (MTD) can assist in providing MECP Enhanced (Level 1) treatment and can contribute to the treatment train approach for water quality control. MTD's can be used as standalone devices or as pre-treatment to infiltration or filtration systems and could include catchbasin inserts (such as goss traps), oil-grit separators, or stormwater filters.

The location of the proposed LID measures is shown on **Figure 2.6**. The infiltration LID locations have been selected for locations where a minimum of 1 m separation between the proposed ground and the seasonally high groundwater table can be provided. Golder Associate's Hydrogeological Investigation assessed Site Sections 'A-A', 'B-B' and 'C-C' which illustrate the proposed ground and seasonally high groundwater table (refer to excepts in **Appendix B3**).

2.5 Proposed Stormwater Management Plan

2.5.1 Quantity Control

Peak Flow

The proposed superpipe and underground storage system will control proposed flows from the site to existing flow rates for the 2 to 100 year storm events. The preliminary design of these facilities and a comparison of the proposed and existing peak flow rates are discussed further in following sections.

Volume

The proposed development targets a volume control criteria to capture and treat or retain the runoff volume from the 25 mm rainfall event from new and/or fully reconstructed impervious areas. Proposed LIDs and BMPs have been sized to provide this storage volume where feasible. The preliminary design of these facilities are discussed further in following sections.

2.5.2 Quality Control

Quality control to provide TSS and phosphorus removal will be provided by a treatment train of LID techniques which will include additional topsoil depth on all grassed areas, reduced lot grading where possible, rear yard infiltration trenches, bioswales, a street catchbasin infiltration or filtration system, and an end-of-pipe underground storage system. The preliminary design requirements of the SWM infrastructure to provide the water quality treatment and a detailed phosphorus budget are provided in following sections.

2.5.3 Erosion Control

The erosion control criteria is to provide a minimum of 24 hour extended detention of the runoff from a 25 mm rainfall event and will be provided in the superpipe for Outlet 2 and Outlet 4, and in the end-of-pipe underground storage system for Outlet 5. The preliminary design requirements of the facilities are discussed further in a following section.

2.5.4 Water Budget

Where feasible, measures to minimize impacts on the water budget will be incorporated into the development design. As noted in the Hydrogeological Investigation, the estimated existing infiltration volume on the proposed development is approximately 16,740 m^3. Without mitigation the proposed development infiltration volume is approximately 9,240 m^3.

Low impact development measures will be implemented as previously described to maintain or increase existing infiltration rates. Per the Hydrogeological Investigation, it is anticipated that a proposed infiltration volume of approximately 16,205 m^3 can be achieved through the proposed mitigation measures.

The Hydrogeological Investigation also assessed catchment based water budgets to the receiving tributaries and wetlands. Refer to the Hydrogeological Investigation submitted under separate cover for the results.

2.6 Phosphorus Budget

Under the Lake Simcoe Protection Plan, a stormwater management plan must demonstrate how phosphorus loadings are minimized between existing and proposed. Furthermore, LSRCA's Lake Simcoe Phosphorus Offsetting Policy (September 2017) states that:

> *"The phosphorous load from the proposed development on the property will be zero. In situations where the phosphorous load cannot be met or demonstrated in a post-development scenario to achieve the Zero Phosphorous, the developer or proponent shall be required to provide phosphorous off setting to the LSRCA."*

The MECP database application *Lake Simcoe Phosphorus Loading Development Tool* (v2, 01-April-2012 update) was used to complete the phosphorus budget for the proposed development. Due to the complex treatment train provided by the SWM measures outlined above a spreadsheet based on the MECP database application was developed to determine the proposed conditions phosphorus budget.

Existing Phosphorus Loadings

The existing phosphorus loading is based on the land uses based on the Ecological Land Classification (ELC) community type for existing conditions shown in the Natural Heritage Evaluation, prepared by Beacon Environmental, submitted under separate cover. The existing land uses are shown on **Figure 2.7**. Based on the Phosphorus Loading Development Tool, the existing annual phosphorus loadings were calculated to be 1.71 kg/year. Refer to **Appendix D** for the phosphorus loading tool output.

Proposed Phosphorus Loadings

The proposed land uses for the proposed development are shown on **Figure 2.8**. The proposed residential development is considered high intensity development according to the MECP Phosphorus Tool. The SAS Blocks will be subject to Site Plan control, and therefore will be required to complete their own Phosphorus Budget analysis at the Site Plan control stage. The runoff from these blocks that drains to the proposed end-of-pipe subdivision infrastructure will be partially treated for phosphorus removal at those end-of-pipe facilities. The phosphorus from the site plan block (SAS) that is not removed by the end-of-pipe facility will need to be removed to achieve the zero phosphorus target for those blocks, either through additional on-site controls or offsetting, demonstrated at the Site Plan control stage.

The majority of the development will be treated by sorbtive media interceptors, which will be created by adding 5% iron filings by weight to the proposed filtration and infiltration facilities. This is considered to be a standard sizing guideline for sorbtive media interceptors. Please refer to **Appendix D** for relevant sizing information.

The proposed phosphorus loading with no best management practices (BMPs) was calculated to be 20.37 kg/yr (refer to **Appendix D**).

The proposed phosphorus loading with BMPs was calculated to be 2.48 kg/yr (see **Appendix D**). **Table 2.4** provides a summary of the land use, BMP, and phosphorus removal efficiencies for the proposed condition.

Table 2.4: Phosphorus Budget Summary

Phosphorus Loading (kg/yr)		
Existing	Proposed without BMPs	Proposed with BMPs
1.71	20.37	2.48

As per LSRCA's Phosphorus Offsetting Policy, the increase in phosphorus loading will be offset at a rate of $35,770/kg/year, at a 2.5:1 ratio. The cost of the phosphorous offsetting will total $255,232.08, which includes a 15% administration cost. As previously noted, this calculation was completed assuming that the SAS Blocks will remove 100% phosphorus and therefore may be subject to additional phosphorus offsetting to be calculated at the Site Plan Control stage.

2.7 Rear Yard Infiltration Trenches

Rear yard infiltration trenches are proposed for split draining lots to receive runoff from the back half of the roofs where 1 m of separation to the high groundwater level can be provided. The trenches will be located beneath the rear yard swales and will receive runoff from the back half of the roofs by overland runoff from roof leaders directed to the rear yard swales. They will be composed of washed clear stone with approximate dimensions of 0.6 m deep and 1.0 m wide, which will capture a minimum of 25 mm of runoff from the back half of the roofs. The length of the trench will vary depending on the size of the lots. Based on the design infiltration rate of 12 mm/hr, the runoff storage volume in the trench can be infiltrated with 48 hours. Refer to **Figure 2.9** for details. Calculations are provided in **Appendix E**.

2.8 Bioswale/Rain Garden: Roads

The proposed bioswale/rain garden will collect runoff from half of the road right-of-way via proposed curb cuts to facilitate retention and filtration via the proposed engineered soil media and stone base. The curb cuts are proposed along the length of the respective bioswale to maximize conveyed drainage area. Curb cuts are proposed upstream of catchbasins to ensure runoff is conveyed to the bioswale prior to discharging to the proposed storm sewers. In storm events where the capacity of the bioswale is exceeded, runoff will discharge back to the road where it will be captured by catchbasins located immediately downstream of the lowest curb cut. Because there will not be 1.0 m of separation from the bottom of the bioswales to the seasonally high groundwater, the bioswales will be wrapped in an impermeable liner and have an underdrain.

The bioswales are sized for the greater of the water quality treatment volume per Table 3.2 of the MECP SWM Planning and Design Manual or the 25 mm volume from impervious surfaces. The bioswale on the 16.5 m road right-of-way provides storage for 25 mm/impervious area, and the bioswale on the 15 m road right-of-way provides storage for 28.2 mm/impervious area. Right-of-way cross sections and the details are discussed further in **Section 6.0** and calculations are provided in **Appendix E**.

2.9 Bioswale Infiltration: Street D

The proposed bioswale infiltration system will collect runoff from the front half of the roofs, driveways, and Street D via overland flow to the LID located in an island of the cul-de-sac to facilitate infiltration via the proposed engineered soil media and stone base. The bioswale infiltration system is sized for the greater of the water quality treatment volume per Table 3.2 of the MECP SWM Planning and Design Manual or the 25 mm volume from impervious surfaces. The bioswale provides storage for 22.9 mm/impervious area. Details are discussed further in **Section 6.0** and calculations are provided on **Figure 6.7**.

2.10 Catchbasin Filtration Trench

Catchbasin filtration trenches are proposed to provide quality control for the municipal road right-of-way and lots draining to the catchbasins. Runoff entering a catchbasin will be directed through a catchbasin pretreatment device (e.g. "goss trap" and sump) before entering a lead directed to the trenches. Runoff in excess of the capacity of the lead, or if a filtration trench has reached capacity, will be directed through an overflow lead into the minor system. The trenches will be located beneath the right-of-way boulevard. However, they can only fit in one side of the right-of-way due to conflicts with the watermain separation. Therefore, any catchbasin which isn't directly connected to a trench will have its lead connected to a catchbasin that is directly connected to a trench. The proposed road right-of-way cross section with the catchbasin filtration system is discussed further in **Section 6.0**.

As there will not be a minimum of 1.0 m of separation to the seasonally high groundwater level, the system will be designed as a filter trench with an impermeable liner to prevent groundwater inflow and a subdrain returning water back to the storm sewer.

The catchbasin filtration trenches will be composed of washed clear stone on top of 0.4 m of brick sand. A perforated drain within the brick sand layer connected to the minor system will be provided at the downstream end of the filtration facility. The proposed road right-of-way cross section with the catchbasin filtration system is discussed further in **Section 6.0**.

The filtration trenches are sized for a minimum of the water quality treatment volume per Table 3.2 of the MECP SWM Planning and Design Manual. Due to potential conflicts with the service laterals, other utilities in the boulevard, and potential future maintenance, it is not feasible to achieve the 25 mm volume from impervious surfaces. The trenches all provide a minimum of the water quality treatment volume. The trenches provide the following volume from the contributing impervious areas:

- 18 m road right-of-way: 10.1 mm/impervious area,
- 23 m road right-of-way:9.0 mm/impervious area,
- Half of 16.5 m road right-of-way: 39.6 mm/impervious area, and
- Half of 15 m road right-of-way: 21.8 mm/impervious area.

Calculations are provided in **Appendix E**.

2.11 SWM/LID Design Criteria Conformance

The SWM/ LID's throughout the site have been designed in order to meet MECP's criteria for infiltration trenches. **Table 2.5** below describes the MECP criteria and how the LID design meets it.

Table 2.5: MECP LID Criteria

Design Element	Criteria	Design Conformance
Drainage Area	< 2 hectares	Achieved, refer to **Figure 2.6**.
Land Use	Residential land only	Achieved.
Depth	The seasonally high groundwater table depth should be > 1 m below the bottom of the infiltration trench	Achieved where possible. Not achieved – the system will be designed as a filter trench with impermeable liner.
Drawdown	24 -48 hr drawdown	Achieved, refer to Appendix E.

Design Element	Criteria	Design Conformance
Storage Media	50 mm diameter clear stone	Due to constructability, 19 mm diameter clear stone is typical. This provides the same porosity, therefore achieving the criteria. Refer to **Figures 6.7** and **6.8**.
Filter Layer	150 – 300 mm sand filter	Achieved, refer to **Figures 6.7** and **6.8**.
Distribution Pipes	≥100 mm diameter pipe 75 – 150 mm from the top of the storage layer	Achieved, refer to **Figures 6.7** and **6.8**.

2.12 End-of-Pipe Underground Storage (Outlet 5)

Catchments 201, 202 and EXT1 will be controlled for erosion and quantity control using an underground storage system, such as "Greenstorm".

2.12.1 Extended Detention

The attenuation of the extended detention volume in the underground storage system will provide erosion protection for the downstream watercourse. The extended detention volume 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 Catchment 201, 202, 208 and EXT1 (Outlet 5) is 712 m^3 (see **Appendix F**). The peak release rate for the extended detention volume is approximately 0.012 m^3/s.

2.12.2 Quantity Control: Peak Flow

The proposed underground storage will control proposed 2 - 100 year flows from the site to the existing peak flow rates. Proposed hydrology modelling was completed using the VO6 model to determine the required detention storage volume. Refer to the USB drive containing the VO6 hydrology model provided in **Appendix C**. A summary of the resulting storage requirements for the underground storage system is provided in **Table 2.5**.

Table 2.6: Outlet 5 Underground Storage System Storage Requirements

Return Period Storm	4 Hour Chicago		12 Hour SCS Type II		24 Hour SCS Type II	
	Discharge (m^3/s)	Storage (m^3)	Discharge (m^3/s)	Storage (m^3)	Discharge (m^3/s)	Storage (m^3)
2 Year	0.049	762	0.101	834	0.128	**870**
5 Year	0.145	893	0.180	1049	0.190	**1100**
10 Year	0.173	1015	0.229	1232	0.263	**1289**
25 Year	0.202	1158	0.308	1403	0.319	**1474**
50 Year	0.278	1315	0.336	1533	0.357	**1575**

Return Period Storm	4 Hour Chicago		12 Hour SCS Type II		24 Hour SCS Type II	
	Discharge (m^3/s)	**Storage (m^3)**	**Discharge (m^3/s)**	**Storage (m^3)**	**Discharge (m^3/s)**	**Storage (m^3)**
100 Year	0.316	1456	0.396	1649	0.415	**1687**

Note: Bold values indicate the more conservative (higher) proposed storage volumes

2.13 Superpipe: Catchment 203 (Outlet 4)

Catchment 203 will be controlled for erosion and quantity control by superpipe storage.

2.13.1 Extended Detention – Catchment 203

The attenuation of the extended detention volume in the underground storage system will provide erosion protection for the downstream watercourse. The extended detention volume 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 is 356 m^3 (see **Appendix F**). The peak release rate for the extended detention volume is approximately 0.006 m^3/s.

2.13.2 Quantity Control: Peak Flow – Catchment 203

The proposed superpipe will control proposed 2 - 100 year flows from the site to the existing peak flow rates. Proposed hydrology modelling was completed using the VO6 model to determine the required detention storage volume. Refer to the USB drive containing the VO6 hydrology model provided in **Appendix C**. A summary of the resulting storage requirements for the superpipe is provided in **Table 2.6**.

Table 2.7: Superpipe Storage Requirements – Catchment 203

Return Period Storm	4 Hour Chicago		12 Hour SCS Type II		24 Hour SCS Type II	
	Discharge (m^3/s)	**Storage (m^3)**	**Discharge (m^3/s)**	**Storage (m^3)**	**Discharge (m^3/s)**	**Storage (m^3)**
2 Year	0.044	370	0.100	391	0.104	**411**
5 Year	0.107	426	0.124	517	0.137	**549**
10 Year	0.123	508	0.165	617	0.178	**653**
25 Year	0.156	594	0.199	735	0.210	**772**
50 Year	0.192	706	0.226	818	0.236	**843**
100 Year	0.224	813	0.265	896	0.273	**911**

Note: Bold values indicate the more conservative (higher) proposed storage volumes

2.14 Superpipe: Catchment 206 (Outlet 2)

Catchment 206 will be controlled for erosion and quantity control by superpipe storage.

2.14.1 Extended Detention – Catchment 206

The attenuation of the extended detention volume in the underground storage system will provide erosion protection for the downstream watercourse. The extended detention volume 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 is 274 m^3 (see **Appendix F**). The peak release rate for the extended detention volume is approximately 0.005 m^3/s.

2.14.2 Quantity Control: Peak Flow – Catchment 206

The proposed superpipe will control proposed 2 - 100 year flows from the site to the existing peak flow rates. Proposed hydrology modelling was completed using the VO6 model to determine the required detention storage volume. Refer to the USB drive containing the VO6 hydrology model provided in **Appendix C**. A summary of the resulting storage requirements for the superpipe is provided in **Table 2.8**.

Table 2.8: Superpipe Storage Requirements - Catchment 206

Return Period Storm	4 Hour Chicago		12 Hour SCS Type II		24 Hour SCS Type II	
	Discharge (m^3/s)	Storage (m^3)	Discharge (m^3/s)	Storage (m^3)	Discharge (m^3/s)	Storage (m^3)
2 Year	0.023	303	0.047	342	0.057	**358**
5 Year	0.061	371	0.075	451	0.081	**481**
10 Year	0.074	442	0.095	556	0.102	**597**
25 Year	0.088	522	0.113	657	0.131	**702**
50 Year	0.107	625	0.159	722	0.186	**741**
100 Year	0.146	713	0.231	765	0.255	**778**

Note: Bold values indicate the more conservative (higher) proposed storage volumes

2.15 Comparison of Existing Targets and Proposed Flows

To the extent possible, the proposed development was designed to control proposed runoff to the existing levels. **Table 2.9**, **Table 2.10** and **Table 2.11** provides a comparison of existing and proposed flows at outlet locations 2, 4 and 5.

Table 2.9: Comparison of Existing Targets & Proposed Flows – 4-Hour Chicago

Return Period Storm	Outlet 2 (m^3/s)		Outlet 4 (m^3/s)		Outlet 5 (m^3/s)	
	Ex.	Prop.	Ex.	Prop.	Ex.	Prop.
2 Year	0.051	0.032	0.090	0.047	0.109	0.051
5 Year	0.098	0.080	0.170	0.118	0.208	0.152
10 Year	0.134	0.112	0.234	0.136	0.286	0.183
25 Year	0.175	0.145	0.307	0.169	0.375	0.214
50 Year	0.228	0.211	0.392	0.212	0.483	0.293
100 Year	0.280	0.279	0.467	0.244	0.580	0.341

Table 2.10: Comparison of Existing Targets & Proposed Flows – 12-Hour SCS Type II

Return Period Storm	Outlet 2 (m3/s)		Outlet 4 (m3/s)		Outlet 5 (m3/s)	
	Ex.	Prop.	Ex.	Prop.	Ex.	Prop.
2 Year	0.081	0.059	0.138	0.109	0.177	0.107
5 Year	0.138	0.120	0.221	0.141	0.289	0.192
10 Year	0.188	0.155	0.292	0.180	0.384	0.244
25 Year	0.243	0.203	0.368	0.223	0.488	0.332
50 Year	0.286	0.243	0.426	0.251	0.567	0.359
100 Year	0.331	0.286	0.486	0.290	0.647	0.423

Table 2.11: Comparison of Existing Targets & Proposed Flows – 24-Hour SCS Type II

Return Period Storm	Outlet 2 (m3/s)		Outlet 4 (m3/s)		Outlet 5 (m3/s)	
	Ex.	Prop.	Ex.	Prop.	Ex.	Prop.
2 Year	0.092	0.073	0.151	0.117	0.196	0.136
5 Year	0.151	0.128	0.234	0.151	0.307	0.204
10 Year	0.205	0.170	0.309	0.196	0.408	0.280
25 Year	0.266	0.219	0.390	0.232	0.518	0.344
50 Year	0.302	0.249	0.437	0.261	0.582	0.382
100 Year	0.347	0.310	0.494	0.299	0.661	0.444

As shown in **Tables 2.9**, **Table 2.10** and **Table 2.11**, the proposed flows are less than or equal to the existing flows for the 2 through 100 year storm events at all target locations. As noted above, discharge rates to Outlets 1 and 3 will be addressed through a subsequent Site Plan Application process for St. Anne's school.

2.16 Storm Servicing

The storm sewer system (minor system) will be designed for the 5 year return storm as per the Town of Aurora standards.

The major system flow drainage (up to the 100 year storm event) will generally be conveyed overland along the road right-of-ways (ROW).

The storm sewer system will typically be designed with grades between 0.5% and 2.0%. Throughout the site, the storm sewer will be constructed at a minimum depth of 1.5 m to provide frost protection and 2.8 m to service basements. It is anticipated that all storm sewers will be able to be provided deep enough to service basements by gravity, however due to the superpipe storage, it is anticipated that portions of the site will require sump pumps to avoid basement flooding due to the hydraulic grade line in the sewer.

The storm drainage system will be designed in accordance with the Town of Aurora 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: 0.45 m/s for first run, 0.6 m/s for second to fourth run, 0.75 m/s for subsequent runs; and
- Minimum Pipe Depth: 1.5 m to obvert, 2.8 m to obvert to service basements.

The rainfall intensity will be calculated based on Town of Aurora parameters listed below in **Table 2.12**:

Table 2.12 – Rainfall Intensity Parameters

Return Period Storm	A	B	C
2 Year	647.7	4	0.784
5 Year	929.8	4	0.798
10 Year	1021	3	0.787
25 Year	1100	2	0.776
50 Year	1448	3	0.803
100 Year	1770	4	0.820

2.17 Overland Flow

Major system flows (greater than the 5 year up to the 100 year storm event) will be conveyed within the road right-of-ways to 100 year capture points. At detailed design, the 100 year capture points will be designed to capture the 100 year flows assuming 50% blockage at a depth not exceeding the maximum ponding depth per Town of Aurora criteria.

2.18 Regulatory Floodline

Based on LSRCA's floodplain mapping, the Regulatory floodplain associated with the tributary of Tannery Creek to the east is well below the proposed development. The Regulatory floodline associated with the tributary of Tannery Creek to the west/southwest was updated during the approval of the Shining Hill Estates Phase 2 development, and is plotting on the **Figure 5.1**, which shows that the proposed development is outside of the Regulatory floodline.

3.0 SANITARY SERVICING

3.1 Existing Sanitary Sewer System

The existing buildings on the subject lands are serviced with an existing 200 mm diameter sanitary service connection at the property line, opposite of Willow Farm Lane. It is currently unknown what the size of the private sanitary sewer on the property is, however, the existing sanitary manholes were surveyed which indicates the location of the sewer.

The existing sanitary sewer servicing the subject lands discharges to an existing 200 mm diameter sanitary sewer that crosses St. John's Sideroad where it continues south on Willow Farm Lane, east on Heatherfield Lane as a 300 mm diameter, through an easement east and north to St. John's Sideroad, east along St. John's Sideroad, and discharges into a 975 mm diameter trunk sanitary sewer on Yonge Street. The existing sewer system is shown on **Figure 3.1**.

The existing sanitary sewer system was sized to accommodate an area of 12.0 ha and a population of 750 from the subject lands.

A downstream analysis of the existing system up to the Yonge Street trunk is provided in **Appendix G**, which includes the addition of the approved Shining Hill Estates Phase 2 development together with the 12.0 ha and population of 750 from the subject lands. The results show that several runs of the sanitary sewer system are between 90% and 95% capacity.

3.2 Proposed Sanitary Sewer System

The preliminary layout for the proposed sanitary sewer within the subject lands is provided on **Figure 3.1** and **Figure 3.2**.

The sanitary sewers within the proposed development will have slopes ranging between 0.5% and 2% (typically) and will be provided at 3 m to 5 m deep.

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

- Residential Sanitary Generation Rate: 400 l/c/d,
- Population Density:
  - 3.8 people/unit (Single Family)
  - 3.5 people/unit (Townhouse)
  - 0.30 persons/student (School)
    - Note that SAS will be a boarding school and therefore the ultimate population has been used (800 persons total), without apply the 0.3 persons/student rate.
- Peaking Factor: Harmon (Min. 2.0, Max. 4.0),
- Infiltration Rate: 0.26 L/s/ha,
- Minimum Pipe Size: 200 mm diameter,
- Minimum Pipe Cover: 2.8 m,
- Minimum Full Flow Velocity: 0.60 m/s, and
- Maximum Velocity: 3.0 m/s.

The downstream analysis to the Yonge Street trunk sewer was updated to add the proposed development flows. The proposed development includes 13.61 ha and an equivalent population of 1284 (including residential units, neighbourhood park, and the St. Anne's School). Refer to **Appendix G**, for the sanitary sewer design sheet. The results show that with the addition of the proposed development, that four (4) 300 mm diameter sewer runs on St. John's Sideroad would theoretically be between 104% to 107% capacity, and that three (3) runs on Heatherfield Lane and one (1) run in the easement would be between 100% to 108% capacity.

Further analysis and consultation with the Town will be completed at detailed design to confirm whether the surcharging of the Heatherfield Lane sanitary sewer is acceptable. If it is not and to avoid sewer upgrades on Heatherfield Lane, an option is to install a new sanitary sewer parallel to St. John's Sideroad to discharge into the existing 300 mm diameter sewer on St. John's Sideroad at existing manhole MH72A, as shown on **Figure 3.1**. The St. John's Sideroad sewer is significantly lower in elevation than the existing and proposed development and has a drop structure at the junction of the easement and St. John's Sideroad. A hydraulic grade line analysis was completed that shows that the 300 mm diameter sewer on St. John's Sideroad can convey the proposed flows without surcharging the sanitary sewer in the easement (upstream of existing MH72A). There are two existing service connections to the St. John's Sideroad sewer that service 77 St. John's Sideroad and 15900 Yonge Street. Based on site reconnaissance, these dwellings are significantly higher than St. John's Sideroad, and based on LSRCA's floodplain mapping the elevation of the dwellings are at least 251.86, which is more than 4.5 m higher than the proposed hydraulic grade line in the sewer. A maximum of 0.12 m hydraulic grade line surcharge on the St. John's Sideroad sanitary sewer at MH72A will not impact these service connections. Preliminary sanitary sewer design sheets and the hydraulic grade line analysis are provided in **Appendix G**.

4.0 WATER SUPPLY AND DISTRIBUTION

4.1 Existing Water Distribution

The existing buildings on the subject lands are serviced with an existing private watermain that extends from a 200 mm diameter service connection at the property line, opposite of Willow Farm Lane. The size of the private watermain has been reported to be 150 mm diameter, although drawings are not available. Several existing private hydrants were surveyed and a subsurface utility investigation is being completed to verify the location of the private watermain.

The existing watermain servicing the subject lands crosses St. John's Sideroad with a 200 mm diameter watermain where there is a tee connection to the existing ductile iron watermain on the south boulevard of St. John's Sideroad at the intersection of Willow Farm Lane. At the tee, the watermain is a 300 mm diameter to the east, 200 mm diameter to the west, and 200 mm diameter south on Willow Farm Lane. The existing watermain system is illustrated on **Figure 2.3**.

4.2 Proposed Water System

Two connections to the existing system are proposed at St. John's Sideroad. When the future development of the Shining Hill lands in the Town of Newmarket to the north proceed, then the watermain system could be connected through Newmarket subject to future analysis. In that case, a boundary water meter and pressure reducing valve is expected at the municipal boundary, to be provided in the future when the Newmarket phase proceeds. The preliminary layout for the proposed watermain system is provided on **Figure 2.3.**

Municipal Engineering Solutions (MES) has been completed a Water Distribution Analysis for the proposed development (refer to **Appendix H**).

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

- Residential water usage rate: 400 l/c/d,
- Schools water usage rate: 110 L/student/d,
- Population Density:
  - 3.8 people/unit (Single Family)
  - 3.5 people/unit (Townhouse)
  - 2.5 people/unit (Apartment)
- Minimum Pipe Size: 200 mm diameter (150 mm diameter for cul-de-sacs, at the discretion of the Town),
- Minimum Pipe Depth: 1.8 m, and
- Maximum Hydrant Spacing: 150 m.

5.0 GRADING

5.1 Existing Grading Conditions

Under existing conditions, the site slopes in several directions to several drainage draws to the south, east, and north. East of the proposed development is a deep valley. Site grading alteration has been completed on the property in the past to accommodate the past estate residential use. This includes berms, driveways, parking areas, structures, gardens/landscaping and leveling of fields for recreational use. The existing topography has slopes that range from nearly flat at the south-central portion of the site to approximately 30% at existing embankments. The ground surface elevations through the study area range from approximately 274.25 m in the west to approximately 260 m in the northeast corner.

5.2 Proposed Grading Concept

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

- Satisfy the Town of Aurora lot and road grading criteria including:
  - Minimum Road Grade: 0.5%
  - Maximum Road Grade: 6.0%
  - Minimum Lot Grade: 2%
  - Maximum Lot Grade: 5%
- 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; and
- Achieve the stormwater management objectives required for the proposed development.

A preliminary grading plan is provided on **Figure 5.1**.

At the detailed design stage, the preliminary grading shown on **Figure 5.1** will be subject to a more in-depth analysis in an attempt to balance the cut and fill volumes and minimize slopes and walls.

6.0 RIGHT-OF-WAYS AND SIDEWALKS

The proposed road right-of-way cross-sections are provided on **Figures 6.1**, **6.2**, **6.3**, **6.4**, and **6.5**. The sections have been developed to facilitate the LID measures in the boulevard, while still maintaining the general geometric layout of the pavement and street furniture per the Town's standard cross-section as close as possible. The 23 m wide collector road right-of-way has been designed in consultation with the transportation consultant and planning consultant to incorporate a proposed multi-use path and street parking.

The proposed parking and sidewalk location plan is provided on **Figure 6.5**. For the areas where sidewalk will be provided along one side of the street, sidewalks will be typically be located on north or east side of the boulevard or the boulevard side where the larger number of frontages can be serviced.

7.0 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 Stormwater Management Technical Guidelines document (LSRCA, 2016). A detailed erosion and sediment control plan will be prepared for review and approval by the Town of Aurora and LSRCA 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.

8.0 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.

9.0 SUMMARY

This Functional Servicing and Stormwater Management Report has been prepared in support of the Draft Plan of Subdivision and Zoning By-law Amendment applications for the proposed Shining Hill Estates Phase 3 development in the Town of Aurora. This report outlines the means by which the proposed development can be graded and serviced in accordance with the Town of Aurora, Lake Simcoe Region Conservation Authority, Lake Simcoe Protection Plan, and the Ministry of Environment, Conservation and Parks design criteria and policies.

General Information

- The existing land use is estate residential;
- The site is located in the East Holland River Watershed draining to the Tannery Creek; and
- The proposed development consists of low density residential, a neighbourhood park, open space, a private school (St. Anne's School (SAS)), and proposed roads.

Stormwater Management and Storm Servicing

- Quantity, Peak Flow Control: Peak flow control will be provided by the underground storage and superpipes to control proposed runoff rates in the 2 through 100 year storm events;
- Quantity, Volume Control: The on-site retention/detention of the 25 mm rainfall runoff will be provided to the extent feasible by a treatment train of LIDs and BMPs through the use of rear yard infiltration trenches, rain garden/bioswales, and catchbasin filtration trenches in the right-of-way boulevard;
- Quality Control, TSS: MECP Enhanced (Level 1) water quality protection will be provided using a treatment train of LIDs and BMPs including catchbasin sumps and "goss traps", rear yard infiltration trenches, rain garden/bioswales, catchbasin filtration trenches in the right-of-way boulevard;
- Quality Control, Phosphorus: A phosphorus budget analysis was completed using the MECP phosphorus budget tool, which shows that the proposed phosphorus export will be approximately 2.48 kg/yr. The phosphorus export is being mitigated through the use of rear yard infiltration trenches, rain garden/bioswales with sorbtive media, catchbasin infiltration/filtration trenches with sorbtive media in the right-of-way boulevard, and underground storage. An offsetting fee will also be paid to LSRCA in lieu of meeting the zero export criteria;
- Erosion Control: The runoff volume from a 25 mm rainfall event will be detained over 24 hours, to the extent feasible by the underground storage and superpipes;
- Water Budget: Golder Associates has completed a water budget analysis to demonstrate that the proposed site water annual infiltration rates will be approximately equal to existing rates. Catchment based water budgets have been completed to the receiving tributaries and wetlands;
- Storm Servicing:
  - Storm runoff will be conveyed by storm sewers designed in accordance with Town of Aurora and MECP criteria;
  - Storm sewers will generally be designed for the 5 year storm event; and
  - Adequate 100 year overland flow routes will be provided.

Sanitary Servicing

- There is an existing 200 mm diameter sanitary sewer service connection that services the property that discharges to the sanitary sewer in the St. Andrew's on The Hill subdivision, ultimately discharging to the 975 mm diameter trunk sanitary sewer at Yonge Street and St. John's Sideroad;
- A downstream sanitary sewer system analysis has been completed;
- The existing St. John's Sideroad sanitary sewer will theoretically flow slightly above 100% capacity, however, a hydraulic grade line analysis has been completed that demonstrates that the surcharging will not negatively affect any existing service connections; and
- The downstream sanitary sewer analysis shows that the Heatherfield Lane sanitary sewer may flow slightly above 100% capacity if the proposed sanitary sewer flows are discharged to the St. Andrew's on The Hill sanitary sewer system. Further analysis and consultation with the Town will be completed at detailed design to confirm whether the surcharging of the Heatherfield Lane sanitary sewer is acceptable ;
    - Alternatively, a new external sanitary sewer running parallel to St. John's Sideroad is possible to convey sanitary flows, connecting to the 300 mm diameter sanitary sewer on St. John's Sideroad at MH72A.

Water Supply and Distribution

- There is an existing 200 mm transitioning to a 300 mm diameter watermain on St. John's Sideroad;
- The development is proposed to be serviced with two connections to the St. John's Sideroad watermain;
- MES has completed a watermain hydraulic analysis to ensure that there will be sufficient domestic and fire flows to service the development;
- If a watermain connection for future development in Newmarket be provided at the north limit, then a water meter and pressure reducing valve are expected at the municipal boundary between Aurora and Newmarket when development in Newmarket proceeds and the watermain is connected, subject to future analysis; and
- Water supply allocation is required from the Town.

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 lot grading will be subject to further grading design at the architectural design stage prior to the building permit applications.

Right-of-Ways and Sidewalks

- Site specific right-of-way cross sections are proposed to facilitate the low impact development measures in the boulevard, street parking, and multi-use paths.

Erosion and Sediment Control during Construction

- An erosion and sediment control plan will be prepared at the detailed engineering stage, in accordance with the Stormwater Management Technical Guidelines document (LSRCA, 2016).

Utility Considerations

- The utility companies have been contacted to confirm whether there is sufficient servicing capacity.

Respectfully Submitted:

SCS Consulting Group Ltd.

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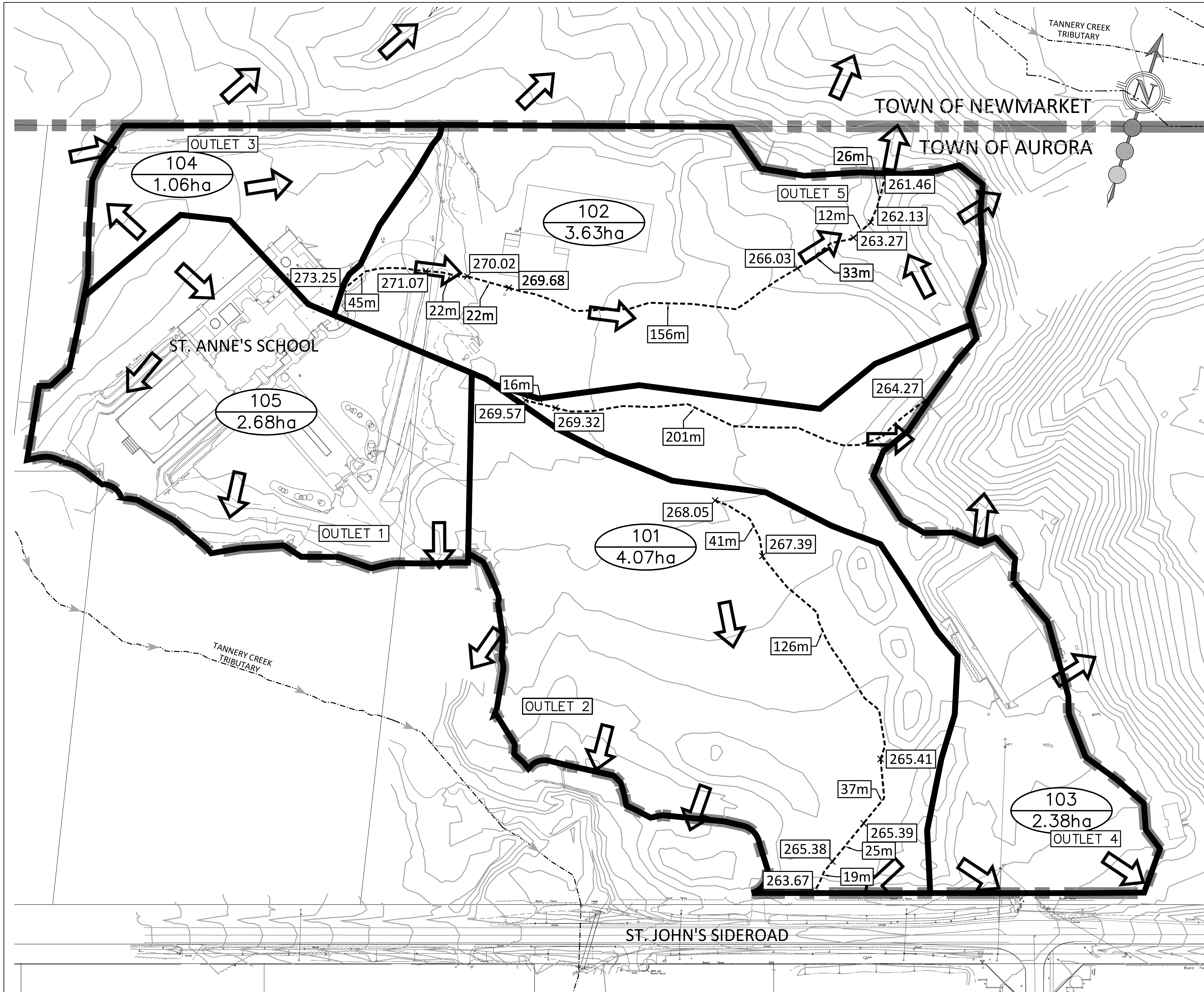

LEGEND:

- MUNICIPAL BOUNDARY
- LIMIT OF DEVELOPMENT
- 255.00 EXISTING CONTOUR AND ELEVATION
- WETLAND BOUNDARY
- STORM DRAINAGE BOUNDARY
- TIME TO PEAK LENGTH
- ×273.25 TIME TO PEAK ELEVATION
- OVERLAND FLOW DIRECTION
- 0.50 CATCHMENT ID
- 0.20ha DRAINAGE AREA (HECTARES)

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
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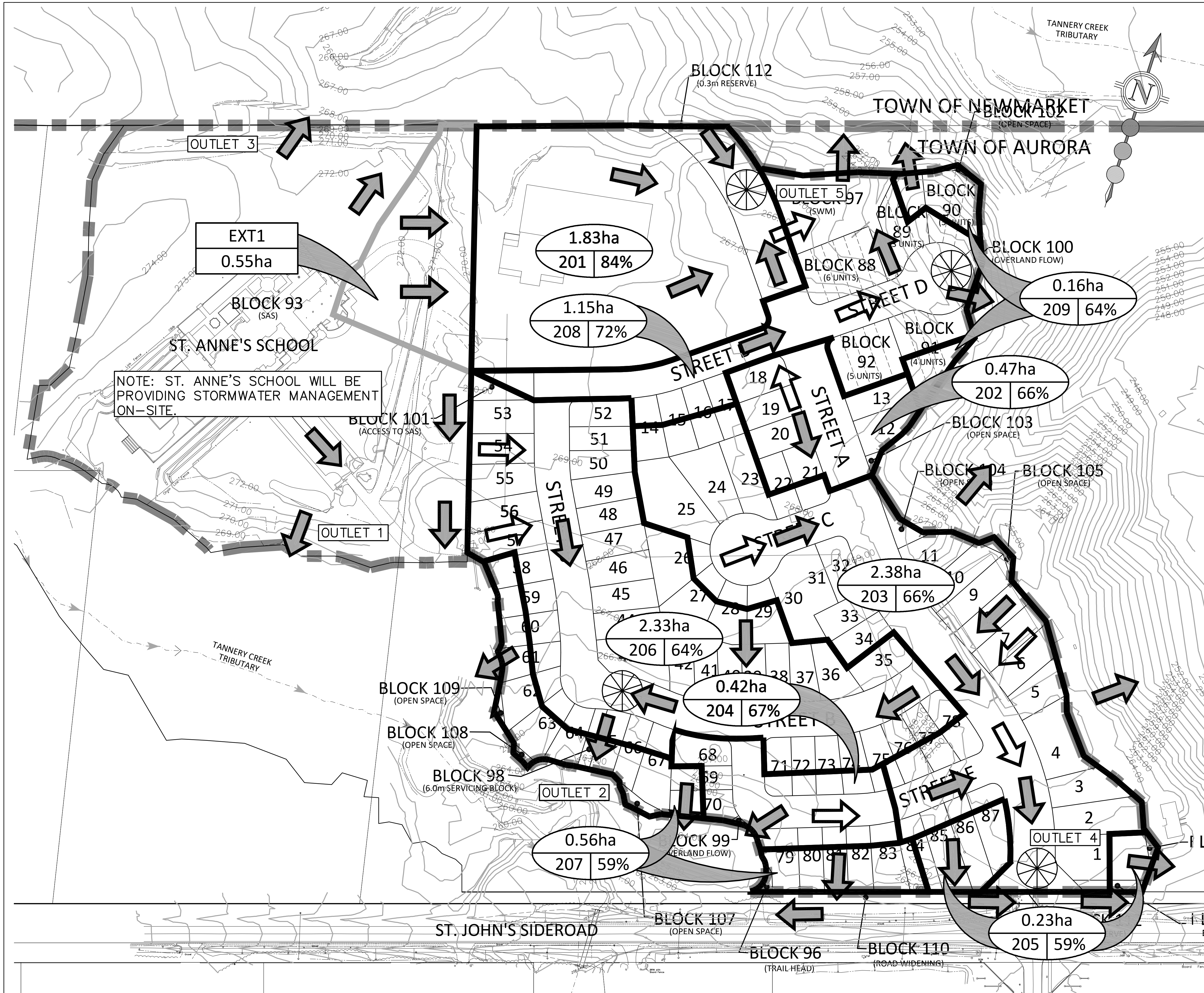
FAX: (905) 475-8335

SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

EXISTING STORM DRAINAGE

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.1



LEGEND:

- MUNICIPAL BOUNDARY
- LIMIT OF DEVELOPMENT
- 255.00 EXISTING CONTOUR AND ELEVATION
- MAJOR SYSTEM - OVERLAND FLOW
- MINOR SYSTEM - STORM SEWER
- WETLAND BOUNDARY
- STORM DRAINAGE BOUNDARY
- EXTERNAL STORM DRAINAGE BOUNDARY
- 100 YEAR CAPTURE LOCATION
- 2.39ha DRAINAGE AREA (HECTARES)
- 206 CATCHMENT ID
- 64% IMPERVIOUSNESS
- EXT1 EXTERNAL CATCHMENT ID
- 0.55ha DRAINAGE AREA

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

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SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

PROPOSED STORM DRAINAGE

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.2

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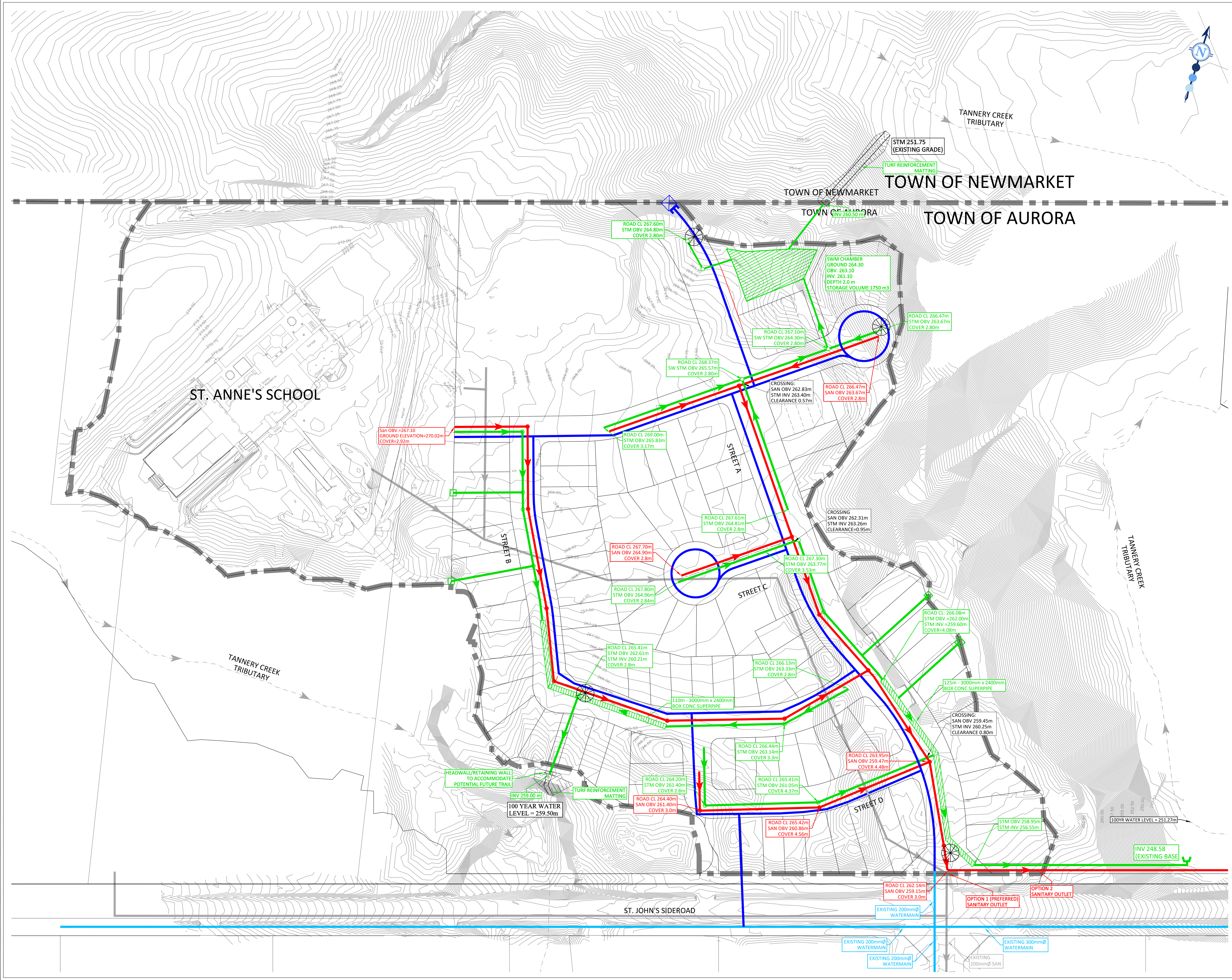

LEGEND:

- LIMIT OF DEVELOPMENT
- MUNICIPAL BOUNDARY
- STORM SEWER
- SUPERPIPE STORM SEWER
- SANITARY SEWER
- EXISTING SANITARY SEWER
- WATERMAIN
- EXISTING WATERMAIN
- REAR YARD CATCHBASIN
- PROPOSED UNDERGROUND STORAGE
- METER CHAMBER AND PRESSURE REDUCING VALVE
- 100 YEAR CAPTURE LOCATION

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

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SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

PROPOSED SERVICING PLAN

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:1000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.3

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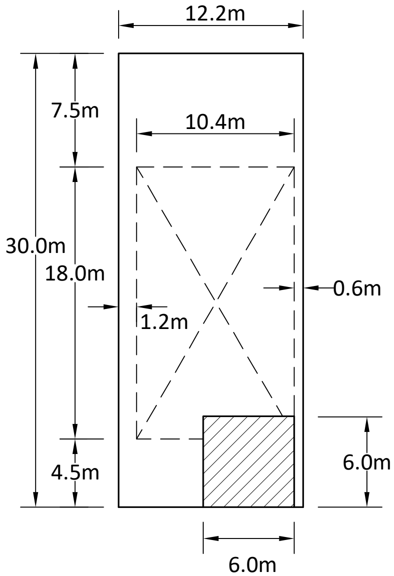
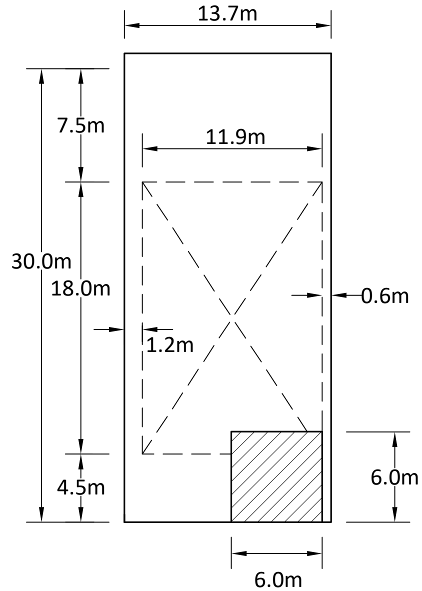
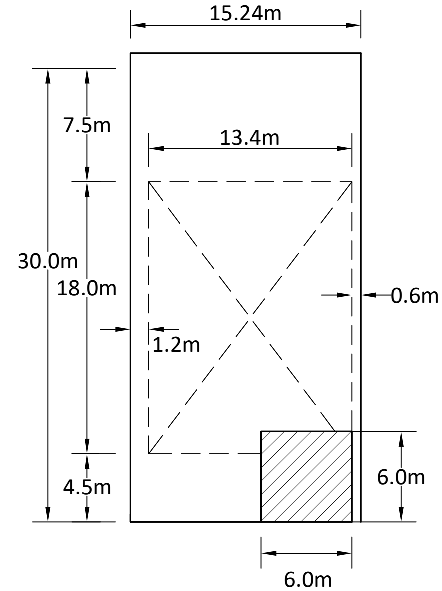
TYPICAL 12.2m x 30m SINGLE
DETACHED DWELLING



TYPICAL 13.7m x 30m SINGLE
DETACHED DWELLING



TYPICAL 15.24m x 30m SINGLE
DETACHED DWELLING



CALCULATED PERCENT IMPERVIOUS = 59%

CALCULATED PERCENT IMPERVIOUS = 59%

CALCULATED PERCENT IMPERVIOUS = 59%

NOTE: SETBACKS PER ZONING

LEGEND:
DRIVEWAY
BUILDING ENVELOPE

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

TYPICAL SINGLE LOT COVERAGE

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

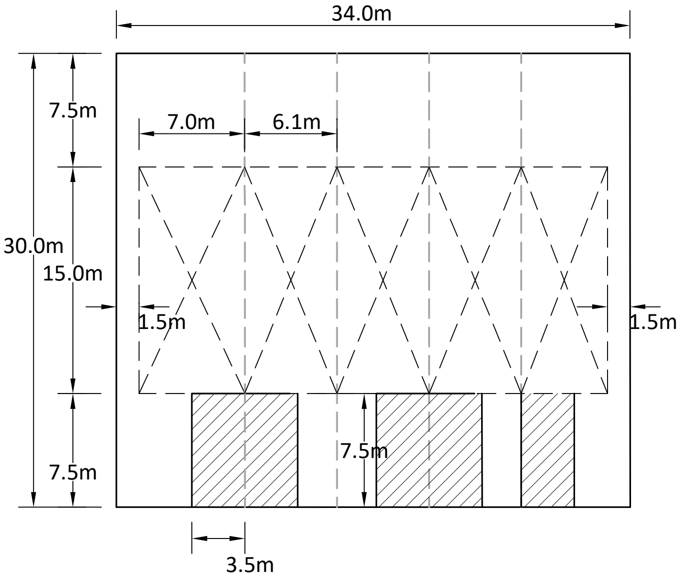
DESIGNED BY:	E.T.C.K.	CHECKED BY:	E.T.C.K.
SCALE:	1:500	DATE:	DECEMBER 2021

PROJECT No: 2183

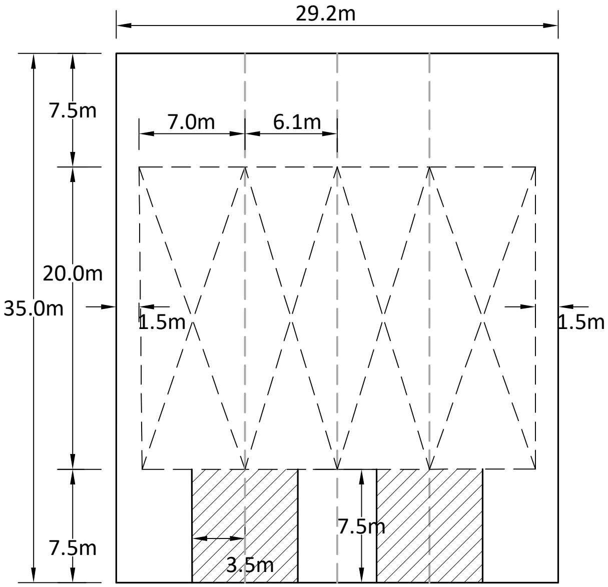
FIGURE No: 2.4

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-LOTS-2.4.dwg - Revised by <LYULE> : Mon, Mar 01 2021 - 1:21pm

TYPICAL 6.1m x 30m
TOWNHOUSE BLOCK



TYPICAL 6.1m x 35m
TOWNHOUSE BLOCK



CALCULATED PERCENT IMPERVIOUS = 58%

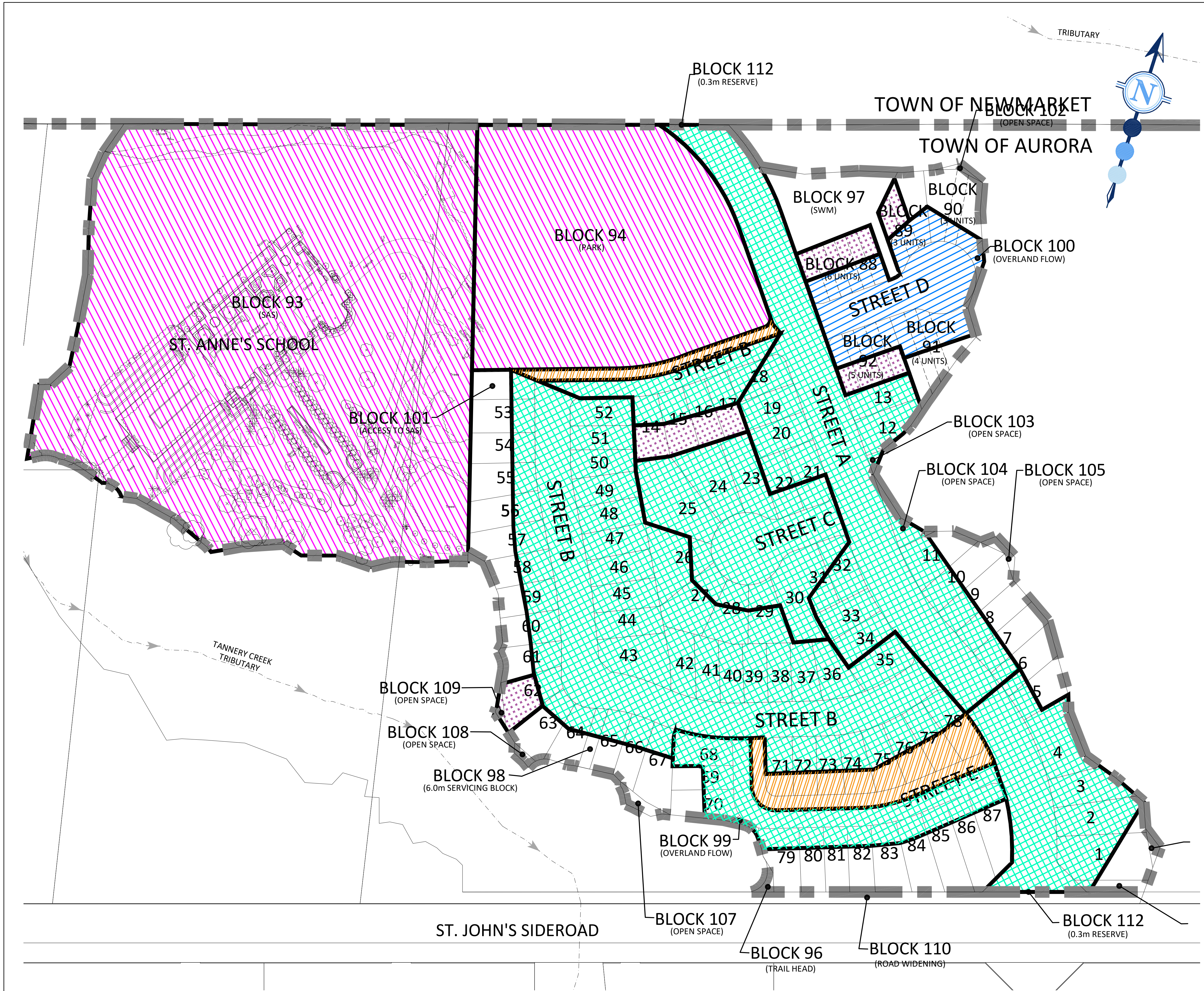
CALCULATED PERCENT IMPERVIOUS = 64%

NOTE: SETBACKS PER ZONING

LEGEND: DRIVEWAY; BUILDING ENVELOPE	SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)	TYPICAL TOWNHOUSE LOT COVERAGE

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

DESIGNED BY:	E.T.C.K.	CHECKED BY:	E.T.C.K.	PROJECT No:	FIGURE No:
SCALE:	1:500	DATE:	DECEMBER 2021	2183	2.5



LEGEND:

- MUNICIPAL BOUNDARY
- LIMIT OF DEVELOPMENT
- STORM DRAINAGE BOUNDARY
- REAR YARD INFILTRATION TRENCH (25 mm/imp-ha)
- CATCHBASIN FILTRATION SYSTEM
- ON-SITE INFILTRATION (25 mm/imp-ha)
- BIOSWALE INFILTRATION (25 mm/imp-ha)
- BIOSWALE FILTRATION (25 mm/imp-ha)

*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

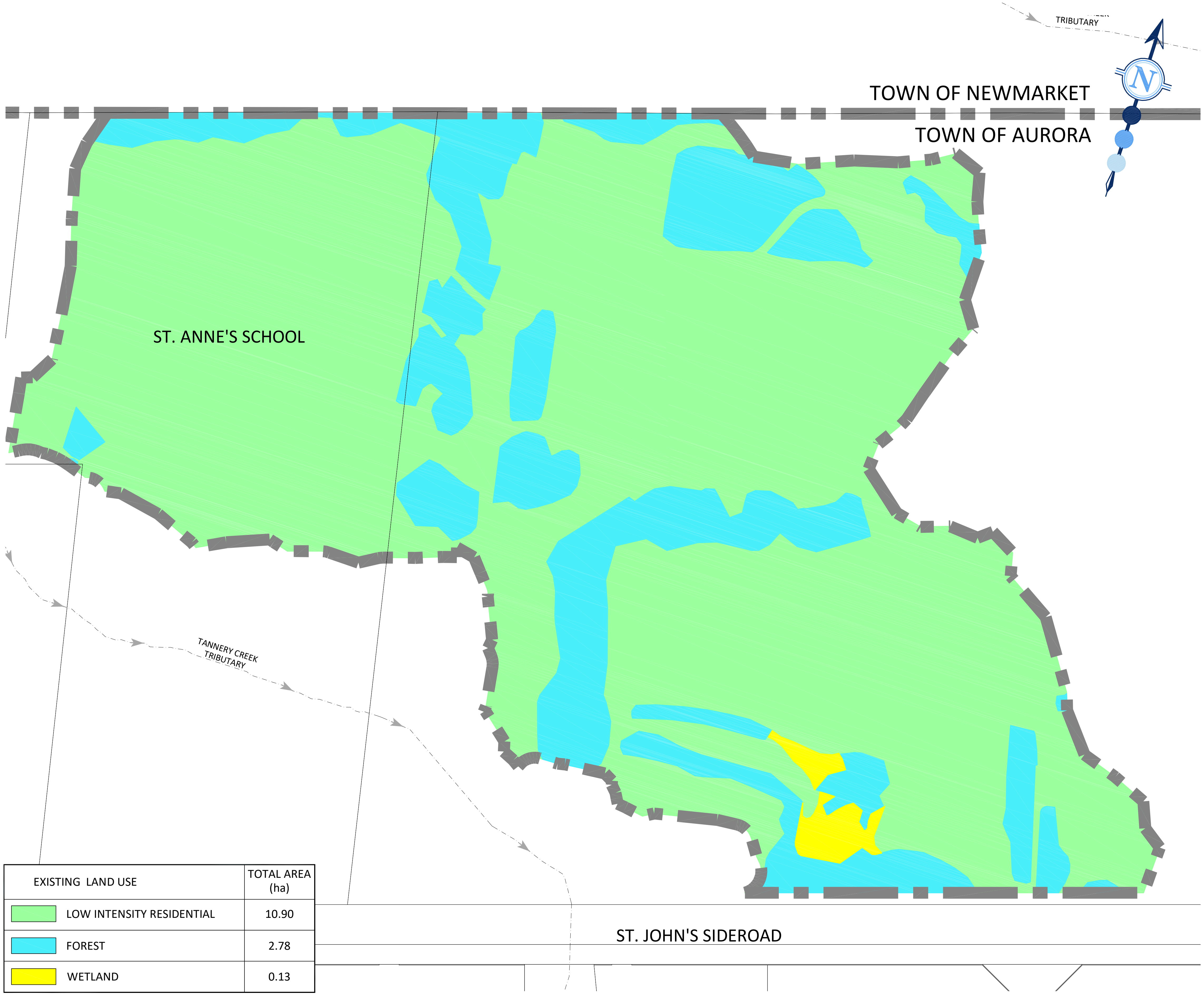
SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

LID PLAN

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.6

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-LIDS-2.5.dwg - Revised by <LYULE> : Mon, Mar 01 2021 - 1:22pm



EXISTING LAND USE		TOTAL AREA (ha)
	LOW INTENSITY RESIDENTIAL	10.90
	FOREST	2.78
	WETLAND	0.13

LEGEND:

MUNICIPAL BOUNDARY

LIMIT OF DEVELOPMENT

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

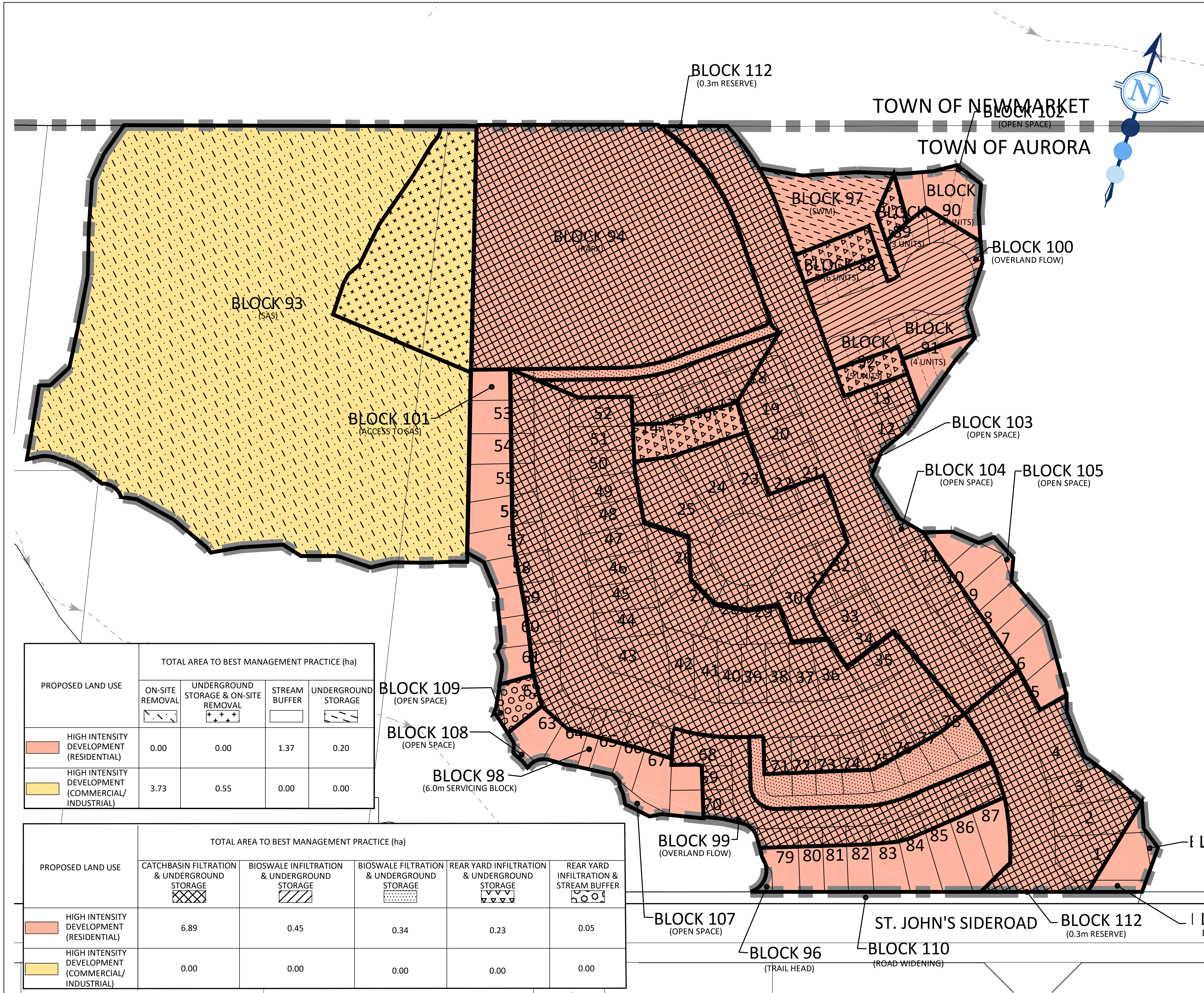
SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

EXISTING PHOSPHORUS BUDGET

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.7

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-PHSB-EXST-2.6.dwg - Revised by <LYULE> : Mon, Mar 01 2021 - 1:20pm



PROPOSED LAND USE	TOTAL AREA TO BEST MANAGEMENT PRACTICE (ha)			
	ON-SITE REMOVAL	UNDERGROUND STORAGE & ON-SITE REMOVAL	STREAM BUFFER	UNDERGROUND STORAGE
HIGH INTENSITY DEVELOPMENT (RESIDENTIAL)	0.00	0.00	1.37	0.20
HIGH INTENSITY DEVELOPMENT (COMMERCIAL/ INDUSTRIAL)	3.73	0.55	0.00	0.00

PROPOSED LAND USE	TOTAL AREA TO BEST MANAGEMENT PRACTICE (ha)				
	CATCHBASIN FILTRATION & UNDERGROUND STORAGE	BIOSWALE INFILTRATION & UNDERGROUND STORAGE	BIOSWALE FILTRATION & UNDERGROUND STORAGE	REAR YARD INFILTRATION & UNDERGROUND STORAGE	REAR YARD INFILTRATION & STREAM BUFFER
HIGH INTENSITY DEVELOPMENT (RESIDENTIAL)	6.89	0.45	0.34	0.23	0.05
HIGH INTENSITY DEVELOPMENT (COMMERCIAL/ INDUSTRIAL)	0.00	0.00	0.00	0.00	0.00

LEGEND:

MUNICIPAL BOUNDARY

LIMIT OF DEVELOPMENT

STORM DRAINAGE BOUNDARY

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

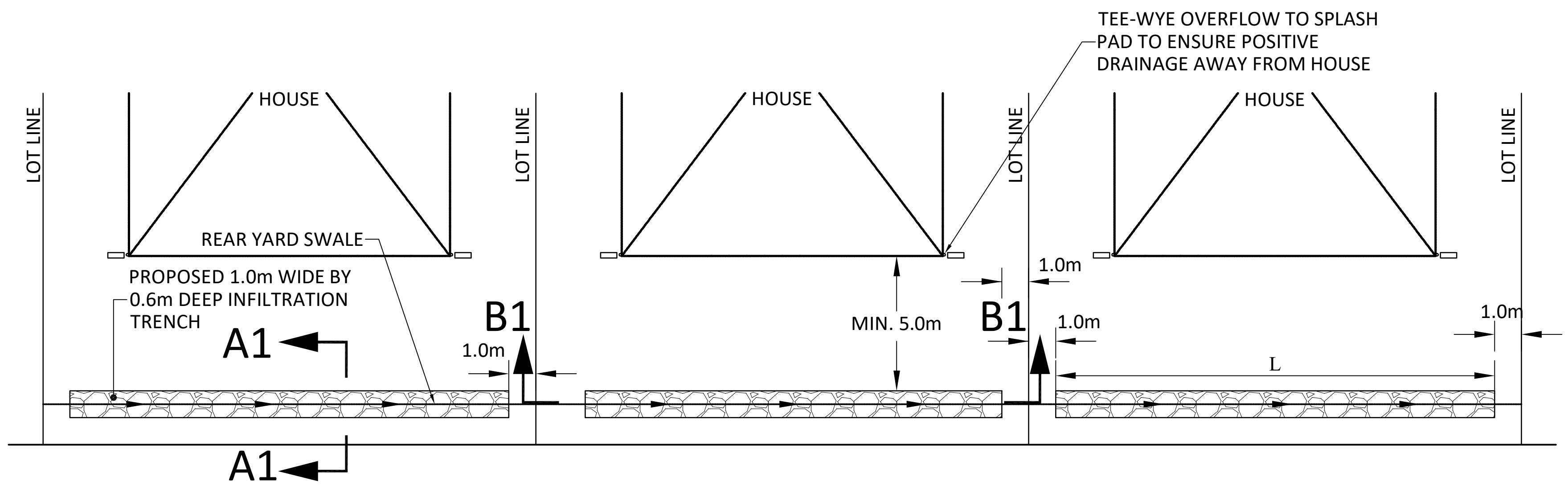
SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

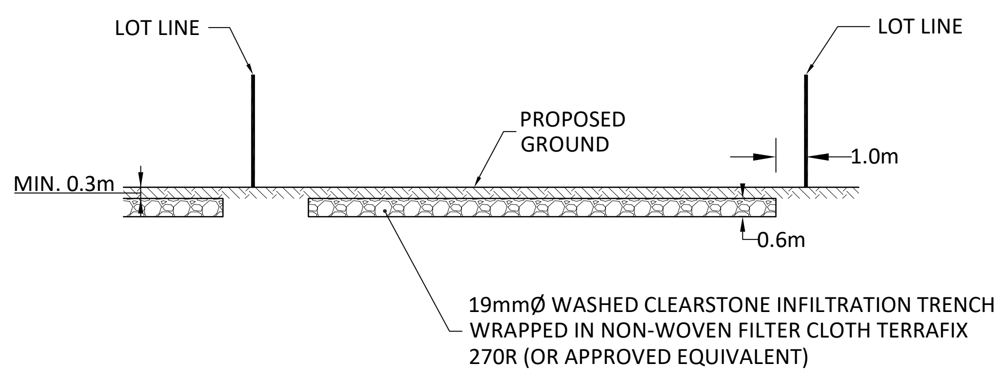
PROPOSED PHOSPHORUS BUDGET

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	2.8



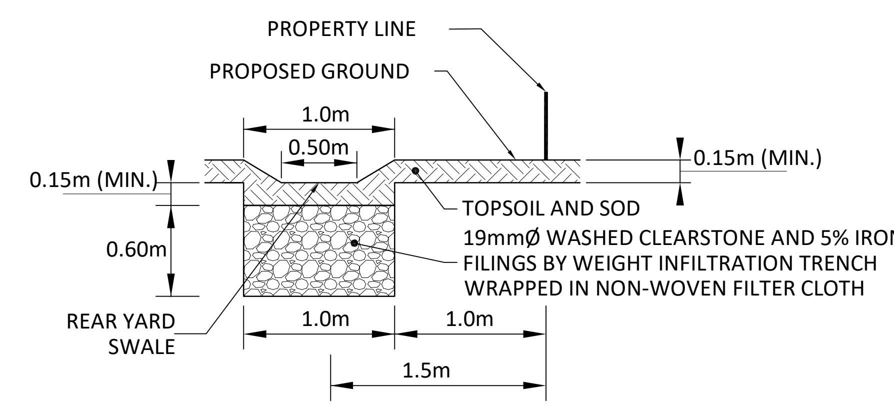
SPLIT DRAINING LOTS PLAN

SCALE 1:250



SECTION B1-B1

SCALE 1:250



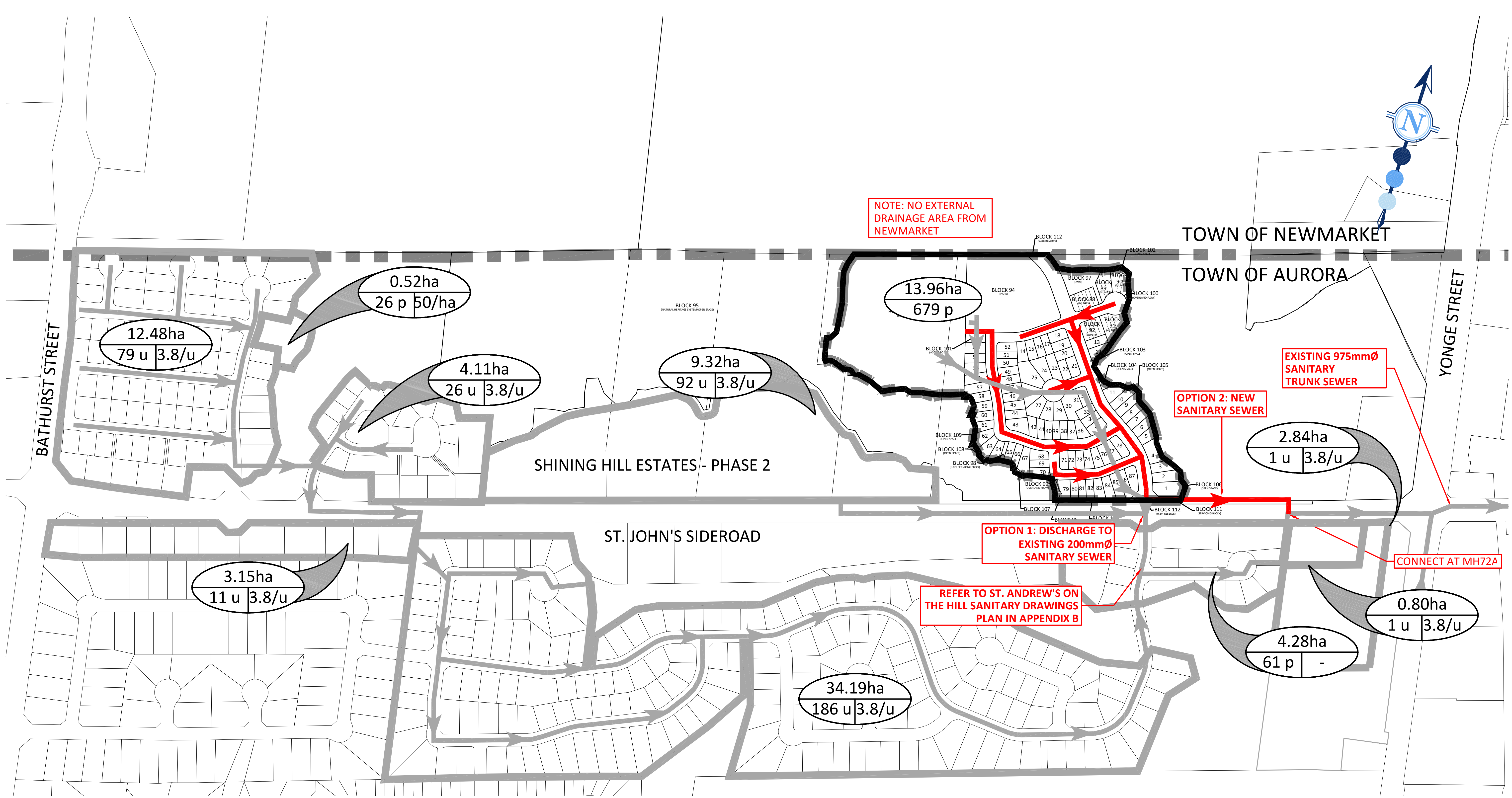
SECTION A1-A1 INFILTRATION TRENCH ASSEMBLY

SCALE 1:50

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

SHINING HILL ESTATES COLLECTION INC.	LEGEND:	SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)	REAR YARD INFILTRATION TRENCH DETAIL
SCS consulting group ltd, 30 CENTURIAN DRIVE, SUITE 100, MARKHAM, ONTARIO L3R 8B8, TEL: (905) 475-1900, FAX: (905) 475-8335		DESIGNED BY: E.T.C.K. / CHECKED BY: S.E.K.	PROJECT No: 2183
		SCALE: AS SHOWN / DATE: DECEMBER 2021	FIGURE No: 2.9

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-DETL-REAR-YARD-INFL-2.8.dwg - Revised by <DDILAY> : Tue, Mar 02 2021 - 2:28pm



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

LEGEND:

1.60ha / 80p / 50/u — SANITARY DRAINAGE AREA; PERSONS PER UNIT OR PERSONS PER AREA; TOTAL POPULATION (p) OR TOTAL UNITS

- MUNICIPAL BOUNDARY
- LIMIT OF DEVELOPMENT
- PROPOSED SANITARY DRAINAGE BOUNDARY
- EXTERNAL SANITARY DRAINAGE BOUNDARY
- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER

SHINING HILL ESTATES COLLECTION INC.

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

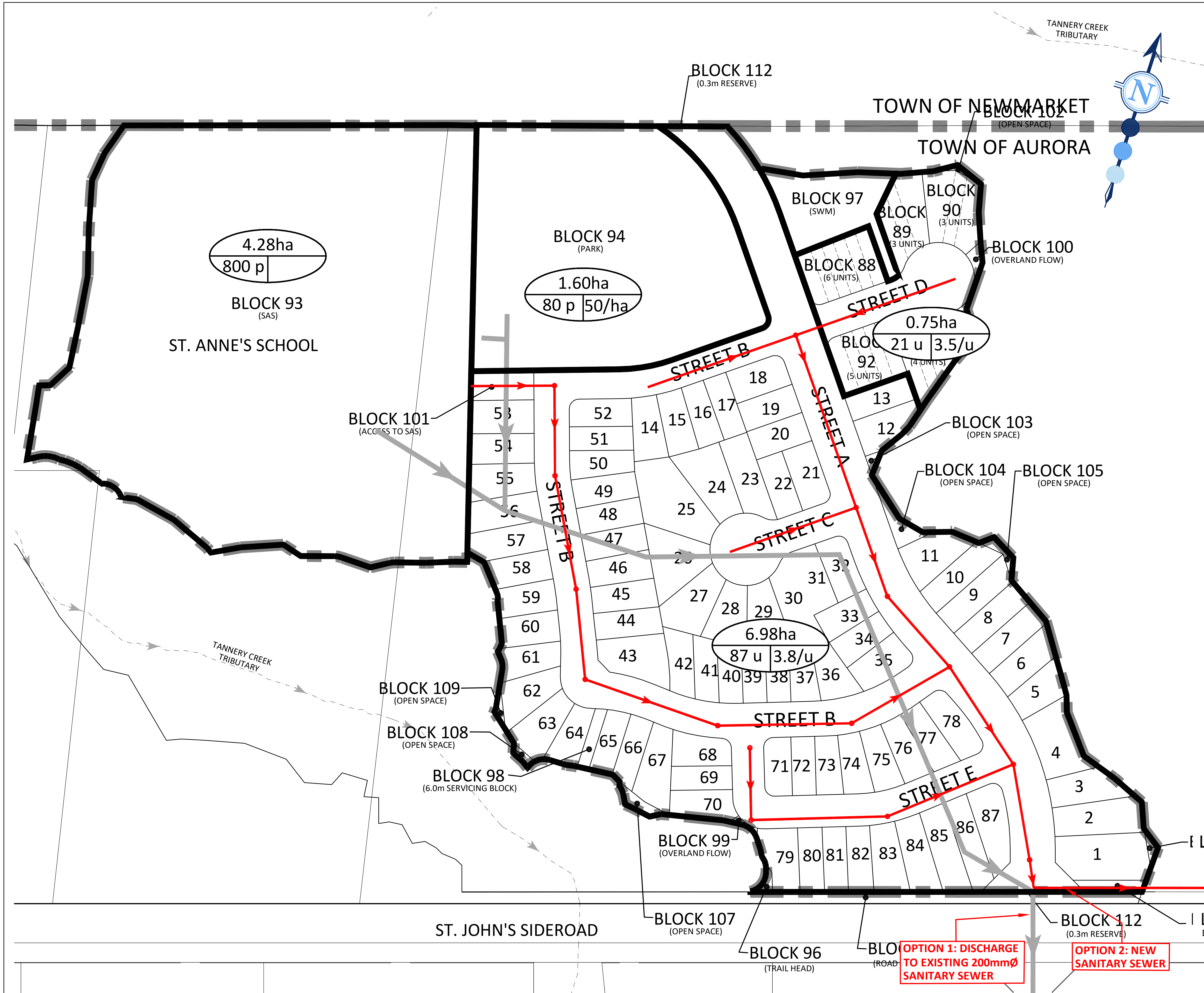
SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

EXTERNAL SANITARY DRAINAGE PLAN

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:6000	DATE:	DECEMBER 2021

PROJECT No: 2183

FIGURE No: 3.1



LEGEND:

MUNICIPAL BOUNDARY

LIMIT OF DEVELOPMENT

SANITARY DRAINAGE BOUNDARY

SANITARY SEWER AND FLOW DIRECTION

EXISTING SANITARY SEWER AND FLOW DIRECTION

1.60ha / 80p / 50/ha — SANITARY DRAINAGE AREA; PERSONS PER UNIT OR PERSONS PER AREA; TOTAL POPULATION (p) OR TOTAL UNITS

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

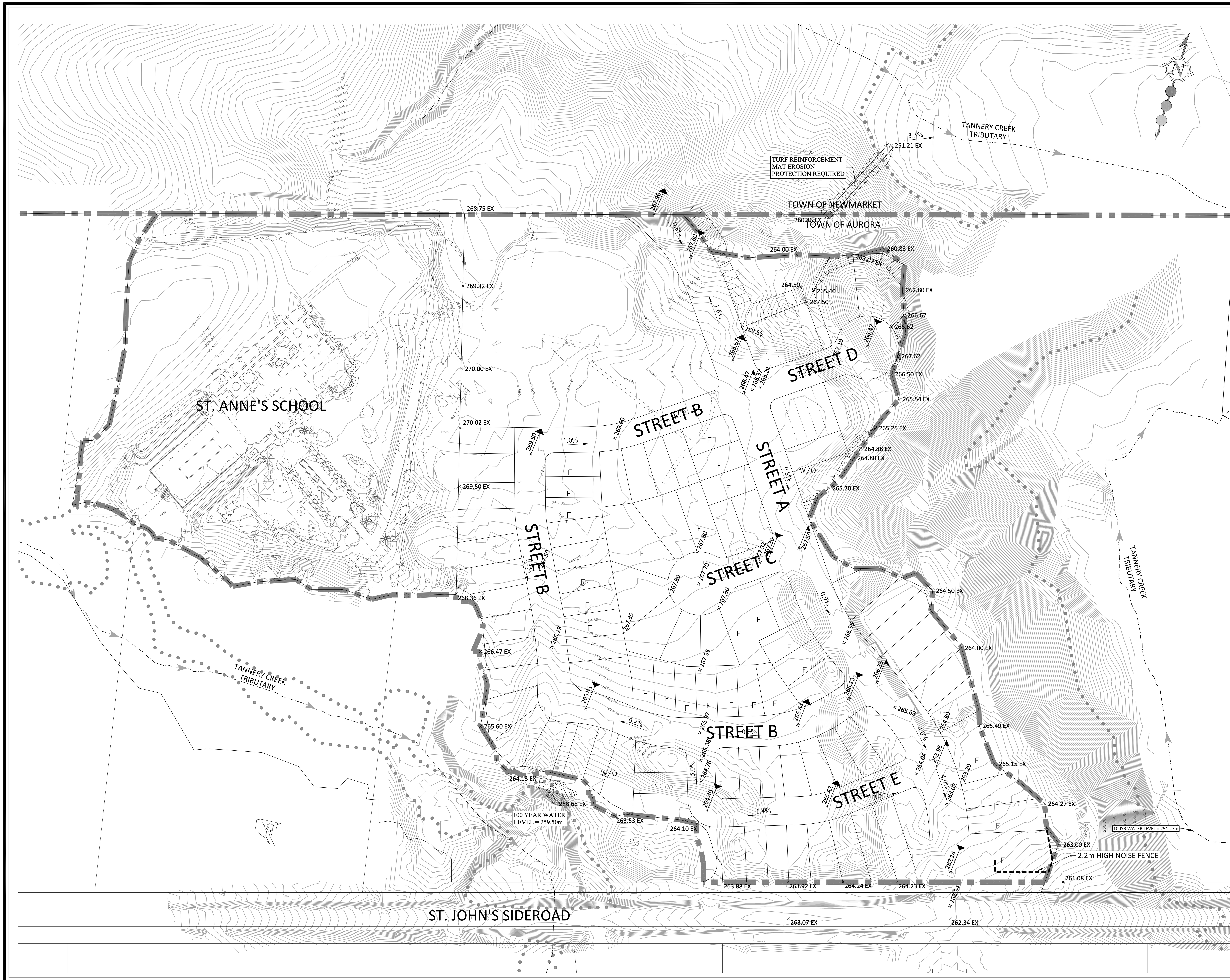
SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

PRELIMINARY SANITARY DRAINAGE PLAN

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	3.2



NOTE: ALL SINGLE RESIDENTIAL LOTS ARE SPLIT DRAINING UNLESS OTHERWISE NOTED.
*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

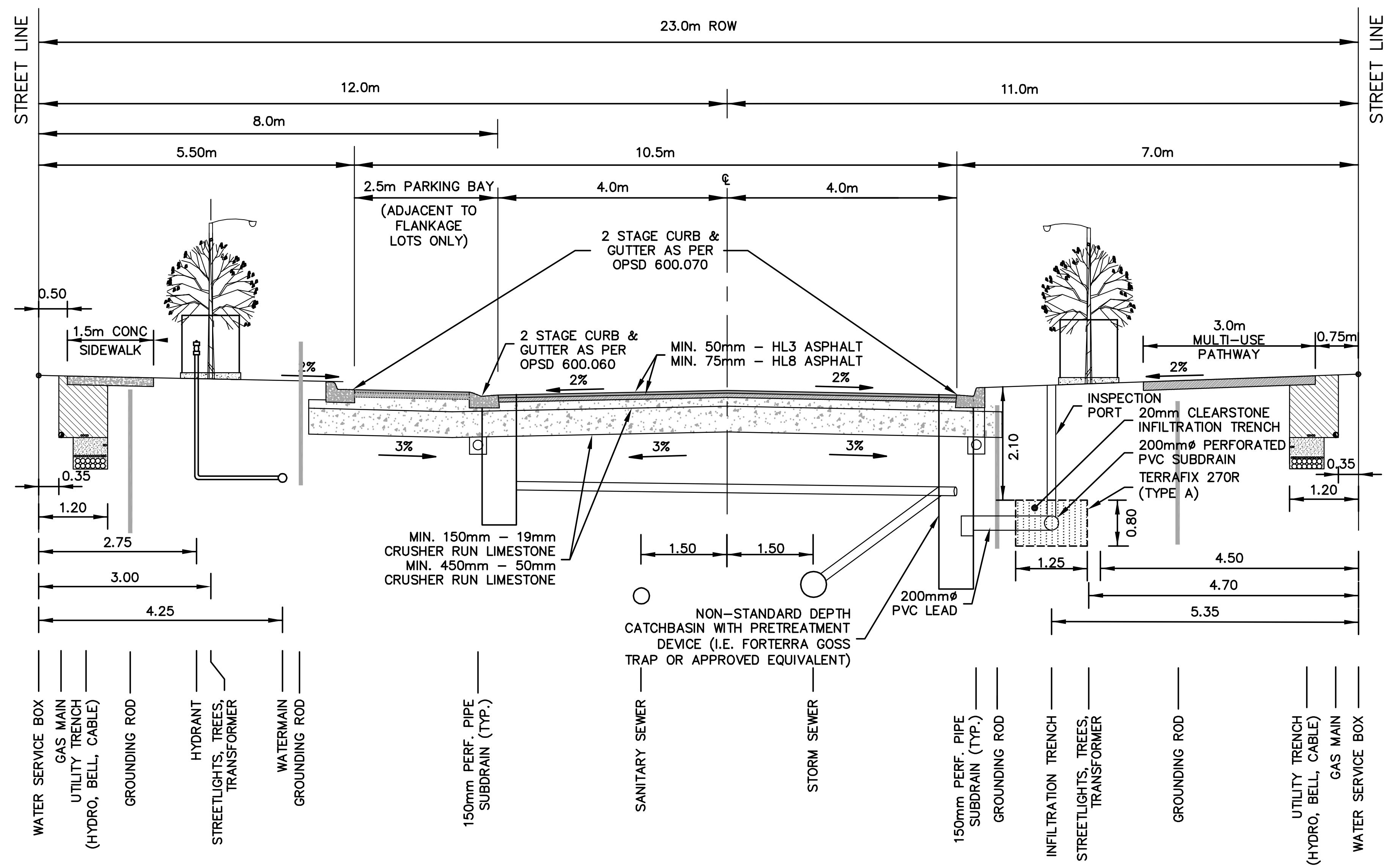
SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

PRELIMINARY GRADING PLAN

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:1000	DATE:	DECEMBER 2021
PROJECT No: 2183		FIGURE No: 5.1	

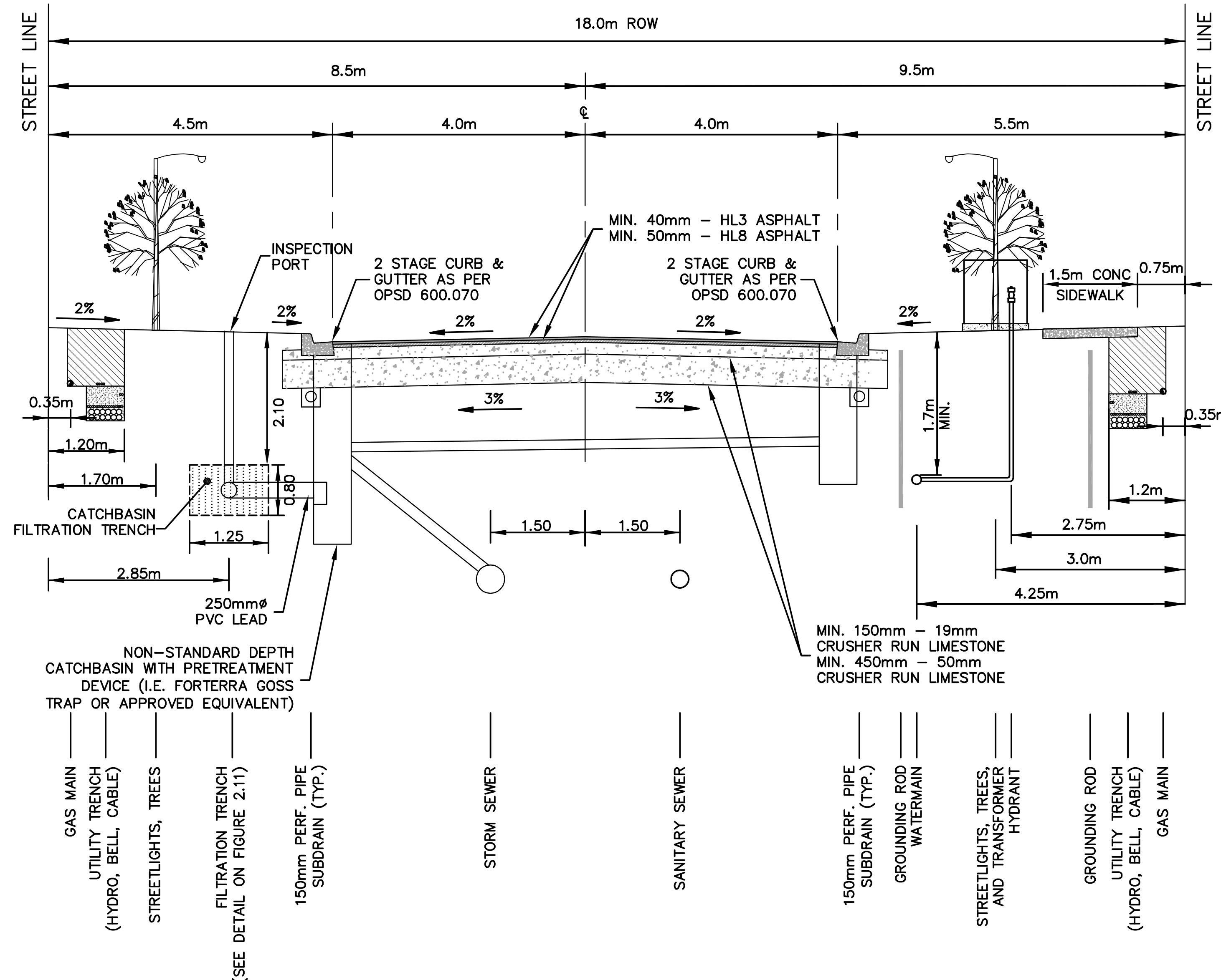
File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-GRAD-5.1.dwg - Revised by <YKASPIN> : Mon, Nov 01 2021 - 9:48am



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

SHINING HILL ESTATES COLLECTION INC.	LEGEND:	SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)		23.0m ROAD SECTION ST. JOHN'S SR TO STREET B/D INTERSECTION	
SCS consulting group ltd 30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335		DESIGNED BY: E.T.C.K.	CHECKED BY: S.E.K.	PROJECT No: 2183	FIGURE No: 6.1
		SCALE: 1:100	DATE: DECEMBER 2021		

File: P:\2183 Shining Hill Estates\Drawings\FSSR–Phase 3\Fig\Report Figures\2183P–FIGS–ROW–6.1–6.5.dwg – Revised by <TMCNEIL> : Wed, Nov 03 2021 – 10:37am



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

SHINING HILL ESTATES COLLECTION INC.

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

LEGEND:

SHINING HILL ESTATES
PHASE 3 - FSSR (AURORA)

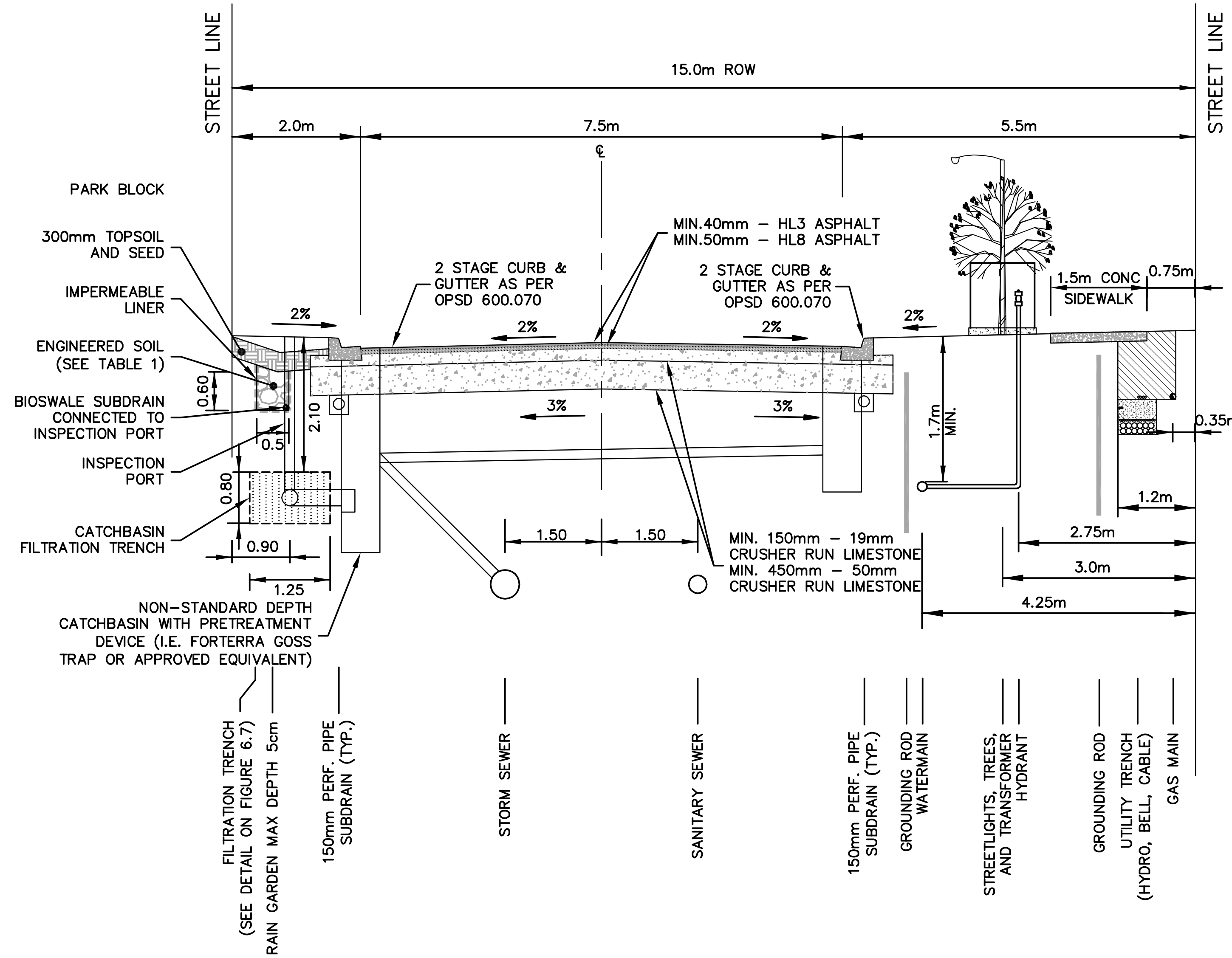
18.0m LOCAL ROAD SECTION

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:100	DATE:	DECEMBER 2021

PROJECT No: 2183

FIGURE No: 6.2

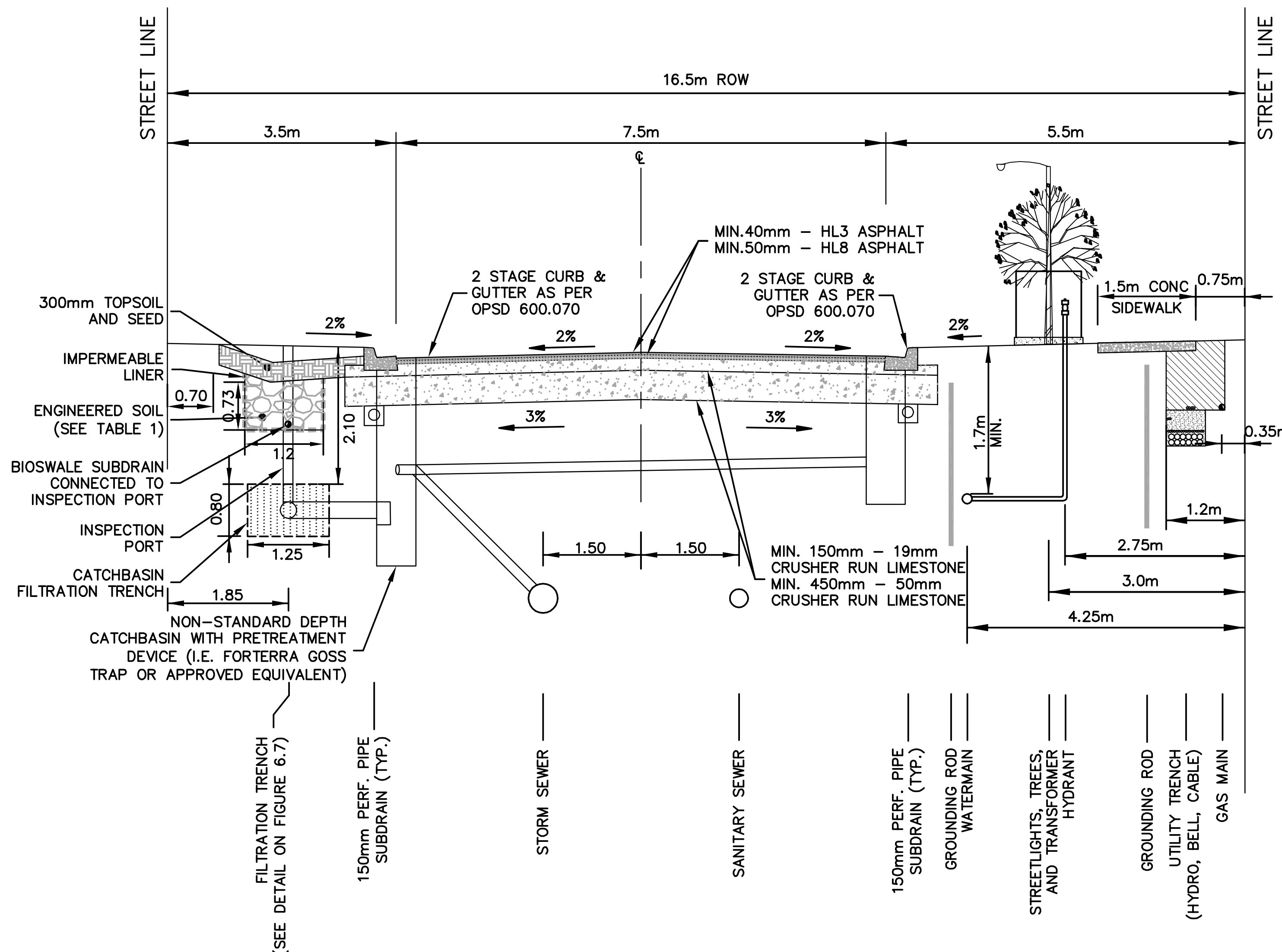
File: P:\2183 Shining Hill Estates\Drawings\FSSR–Phase 3\Fig\Report Figures\2183P–FIGS–ROW–6.1–6.5.dwg – Revised by <TMCNEIL> : Wed, Nov 03 2021 – 10:37am



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

SHINING HILL ESTATES COLLECTION INC.	LEGEND:	SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)		15.0m LOCAL ROAD SECTION	
SCS consulting group ltd 30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335		DESIGNED BY: E.T.C.K.	CHECKED BY: S.E.K.	PROJECT No: 2183	FIGURE No: 6.3
		SCALE: 1:100	DATE: DECEMBER 2021		

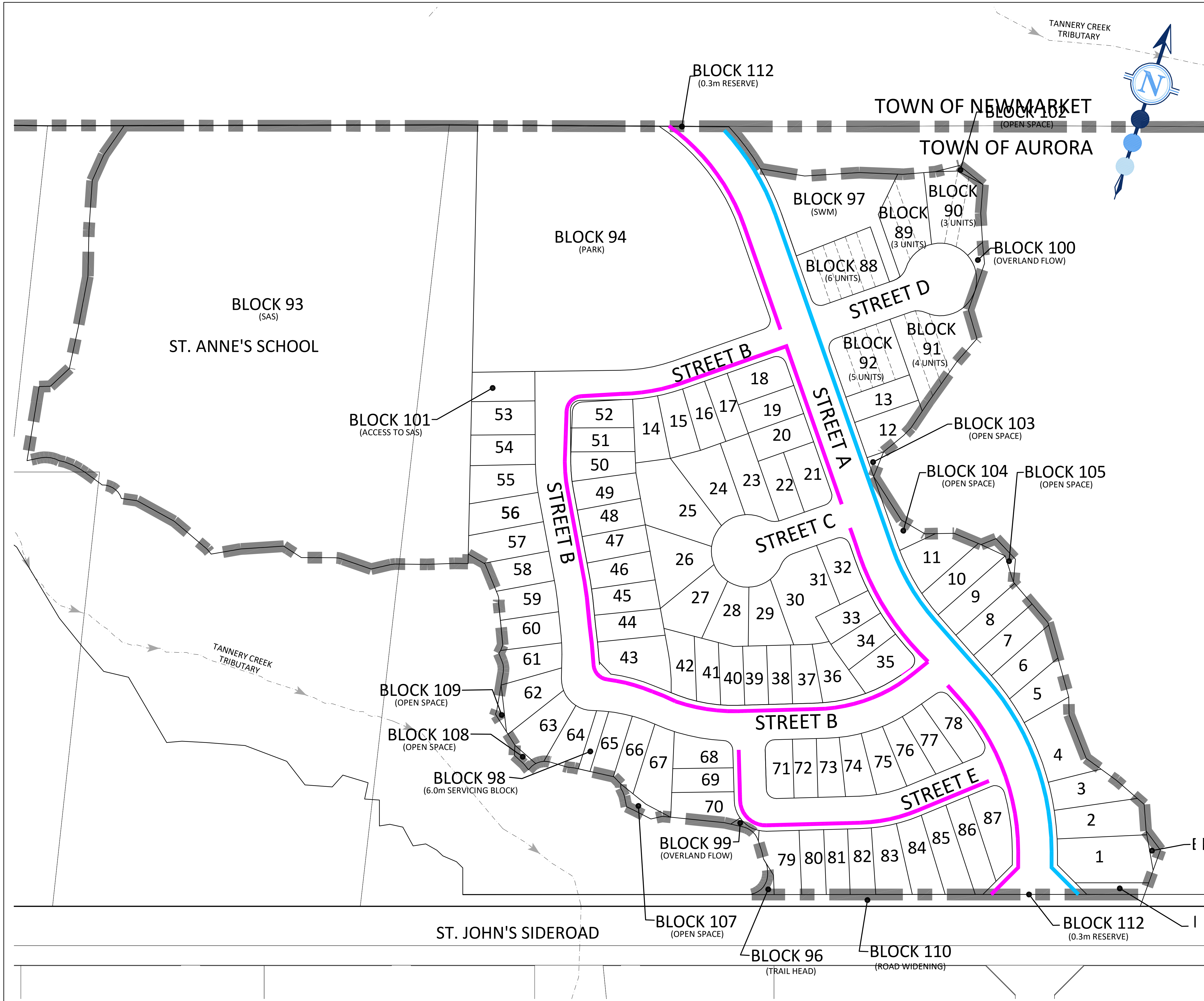
File: P:\2183 Shining Hill Estates\Drawings\FSSR–Phase 3\Fig\Report Figures\2183P–FIGS–ROW–6.1–6.5.dwg – Revised by <TMCNEIL> : Wed, Nov 03 2021 – 10:37am



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

SHINING HILL ESTATES COLLECTION INC.	LEGEND:	SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)		16.5m LOCAL ROAD SECTION	
SCS consulting group ltd — 30 CENTURIAN DRIVE, SUITE 100, MARKHAM, ONTARIO L3R 8B8, TEL: (905) 475-1900, FAX: (905) 475-8335		DESIGNED BY: E.T.C.K.	CHECKED BY: S.E.K.	PROJECT No: 2183	FIGURE No: 6.4
		SCALE: 1:100	DATE: DECEMBER 2021		

File: P:\2183 Shining Hill Estates\Drawings\FSSR–Phase 3\Fig\Report Figures\2183P–FIGS–ROW–6.1–6.5.dwg – Revised by <TMCNEIL> : Wed, Nov 03 2021 – 10:37am



LEGEND:

MUNICIPAL BOUNDARY

LIMIT OF DEVELOPMENT

SIDEWALK

MULTI-USE PATH

*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

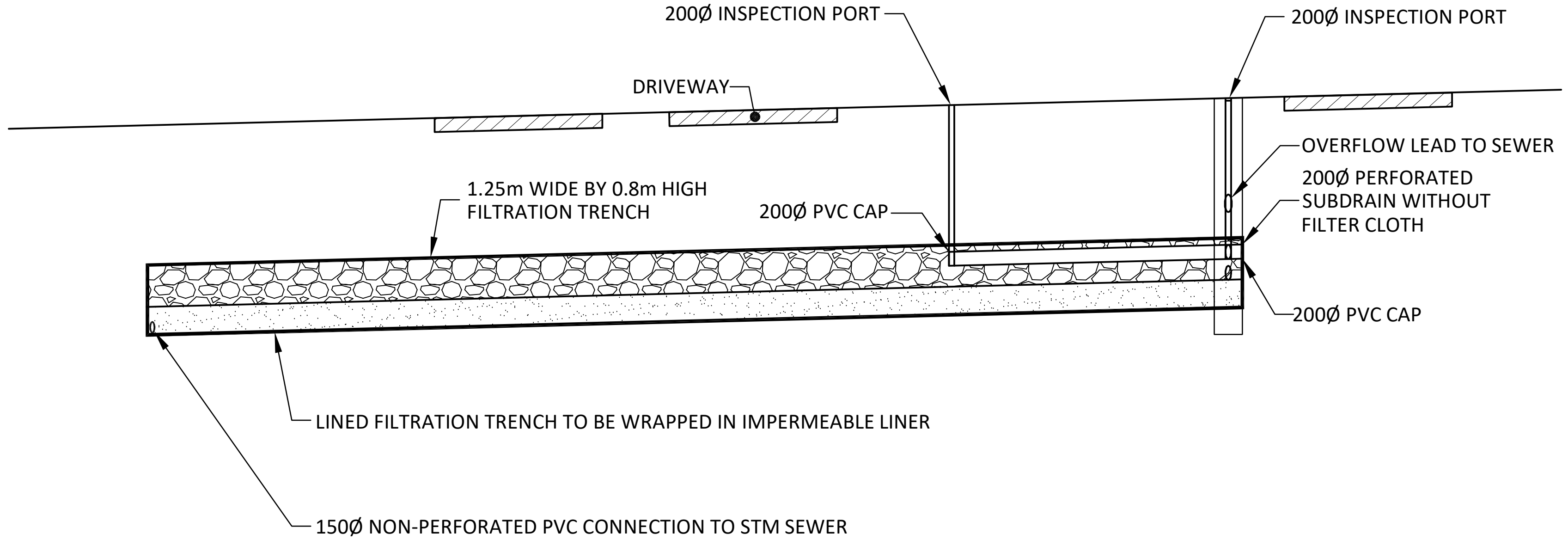
SHINING HILL ESTATES COLLECTION INC.

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

SIDEWALK PLAN

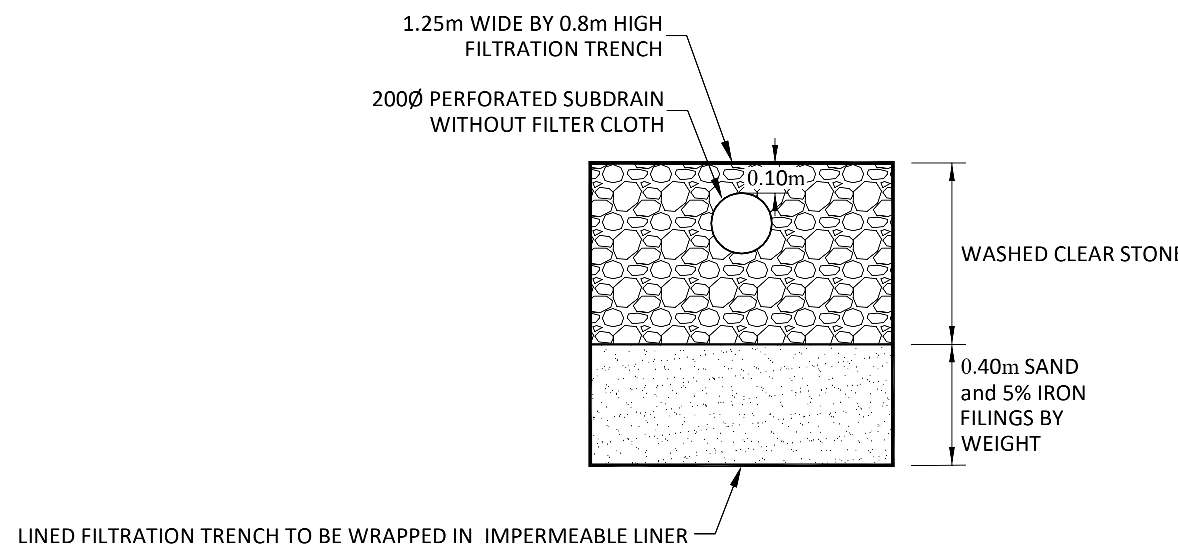
DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	1:2000	DATE:	DECEMBER 2021
PROJECT No:	2183	FIGURE No:	6.5

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-SDWK-6.6.dwg – Revised by <TMCNEIL> : Wed, Nov 03 2021 – 10:44am



PRELIMINARY FILTRATION TRENCH - PROFILE VIEW

SCALE: N.T.S.



PRELIMINARY FILTRATION TRENCH CROSS-SECTION

SCALE: N.T.S.

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

SHINING HILL ESTATES COLLECTION INC.

LEGEND:

SHINING HILL ESTATES PHASE 3 - FSSR (AURORA)

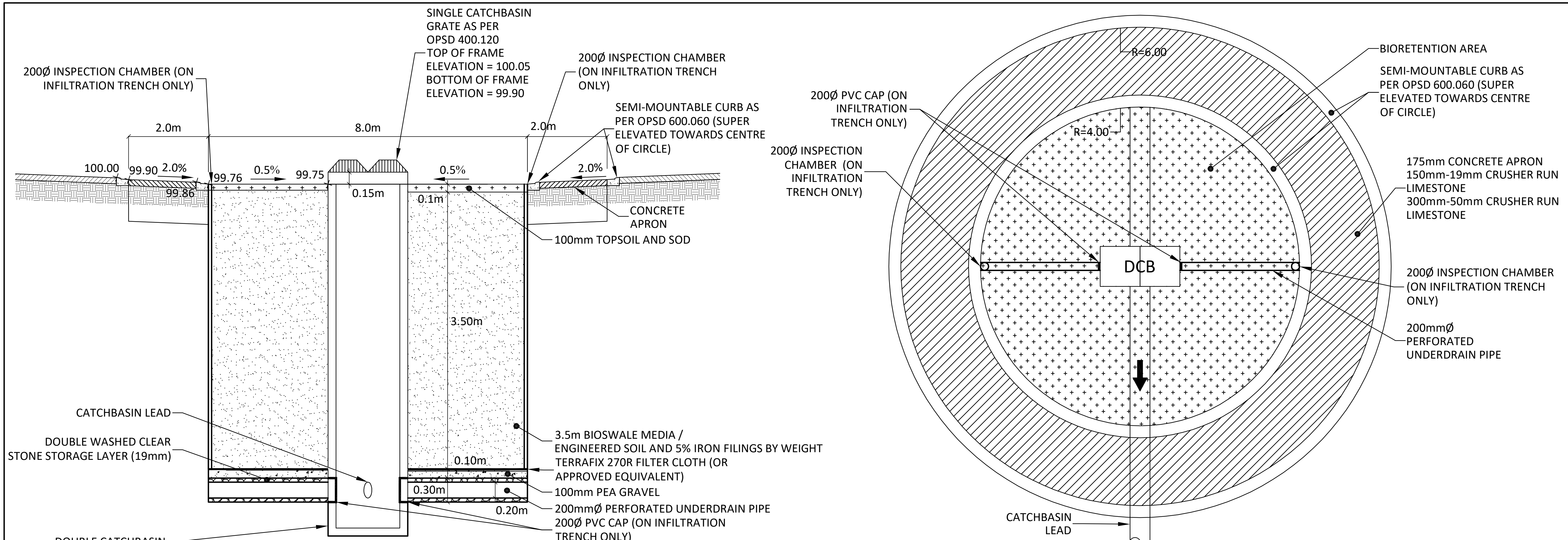
CATCHBASIN FILTRATION TRENCH DETAIL

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

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	AS SHOWN	DATE:	DECEMBER 2021

PROJECT No: 2183

FIGURE No: 6.6



REVERSE GRADED BIOSWALE/INFILTRATION CUL-DE-SAC DETAIL

SCALE: H 1:100 V 1:50

* ELEVATIONS SHOWN ARE FOR REFERENCE PURPOSES ONLY

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

SHINING HILL ESTATES COLLECTION INC.

LEGEND:

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

SHINING HILL ESTATES
PHASE 3 - FSSR (AURORA)

BIOSWALE/INFILTRATION DETAIL

DESIGNED BY:	E.T.C.K.	CHECKED BY:	S.E.K.
SCALE:	AS SHOWN	DATE:	DECEMBER 2021

PROJECT No: 2183

FIGURE No: 6.7

File: P:\2183 Shining Hill Estates\Drawings\FSSR-Phase 3\Fig\Report Figures\2183P-FIGS-DETL-CSBN-INFL-6.8.dwg - Revised by <LYULE> : Mon, Mar 01 2021 - 12:46pm

APPENDIX A

DRAFT PLAN OF SUBDIVISION

DRAFT PLAN OF SUBDIVISION

Part of Lot 86, Concession 1
Town of Aurora
Regional Municipality of York

KEY PLAN

SUBJECT PROPERTY
OTHER LANDS OWNED BY APPLICANT

SCHEDULE OF LAND USE

LOT / BLOCK	LAND USE		UNITS	AREA
1-78	Single Detached Min. 15.24m	□	23	1.46
	Single Detached Min. 13.70m	+	28	1.43
	Single Detached Min. 12.20m	O	27	1.18
79-87	Lane Access Single Detached Min. 13.70m	^	5	0.30
	Lane Access Single Detached Min. 12.20m	~	4	0.18
88-92	Townhouses Min. 6.1m	=	21	0.54
93	Saint Anne's School			4.28
94	Neighbourhood Park			1.61
95	Natural Heritage / Open Space			17.77
96-97	SWM / Trailhead			0.21
98 & 111	Servicing Blocks			0.04
99-100	Overland Flow			0.01
101	Access to Saint Anne's School			0.05
102-109	Vista's / Open Space			0.07
110	Road Widening			0.21
112-113	0.3m Reserves			0.01
Street A	23.0m Right of Way 436m			1.02
Street B-D	18.0m Right of Way 490m			0.96
Street E	16.5m Right of Way 165m			0.27
Street B	15.0m Right of Way 160m			0.19
TOTAL			108	31.79

OWNER'S AUTHORIZATION

I hereby authorize Malone Given Parsons Ltd. to prepare and submit this Draft Plan of Subdivision to the Town of Aurora.

See Original
Angelo DeGasperis

See Original
Date

SURVEYOR'S CERTIFICATE

I hereby certify that the boundaries of the lands to be subdivided and their relationship to the adjacent lands are correctly shown.

See Original
Neil A. LeGrow

See Original
Date

ADDITIONAL INFORMATION

AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT, CHAPTER P.13(R.S.O. 1990).
(a),(e),(f),(g),(j),(l) - As shown of the Draft Plan.
(b),(c) - As shown on the Draft and Key Plan.
(d) - Land to be used in accordance with the Schedule of Land Use.
(i) - Soil is clay loam and sandy loam.
(h),(k) - Full municipal services to be provided.

Prepared by:

140 Renfrew Drive, Suite 201
Markham, Ontario, L3R 6B3
Tel: (905) 513-0170
www.mgp.ca

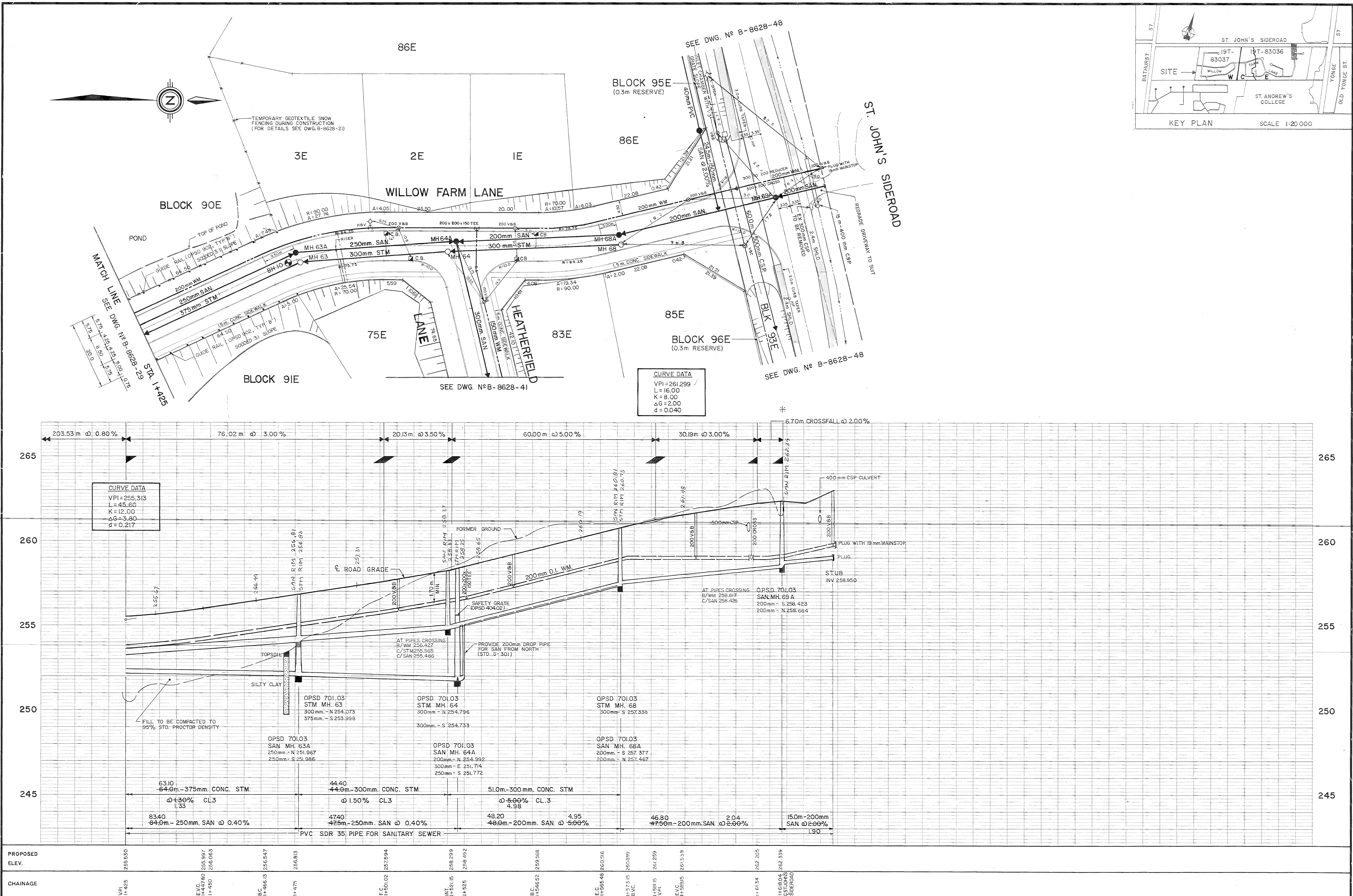
Prepared for:
SHINING HILL ESTATES COLLECTION INC.

Date: March 8, 2021 | Project No.: 15-2374

Date	Revision	By
Oct 7/21	Revise the plan according to Town's comments	DR
Nov 1/21	Add servicing block and temporary turning circle	DR

APPENDIX B1

BACKGROUND DRAWINGS



GENERAL NOTES

Nº	REVISIONS	DATE	APPROVED
1.	REVISED AS PER TOWN'S COMMENTS	NOV. 87	
2	REVISED AS PER TOWN'S COMMENTS	FEB. 88	
3.	SAN., WM. AND ROAD GRADE REVISED	MAR. 88	
4.	REVISED AS CONSTRUCTED SEWERS INVERTS	JAN. 89	
5.	AS CONSTRUCTED SEWERS LENGTH AND GRADE	JAN. 91	
6.	AS CONSTRUCTED ℄ ROADS AND WM TIES	NOV. 94	

REGISTERED PROFESSIONAL ENGINEER
R. C. HINDE
PROVINCE OF ONTARIO

TOWN OF AURORA
APPROVED
DATE Aug. 10th 19 88
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON · THE · HILL

PLAN AND PROFILE

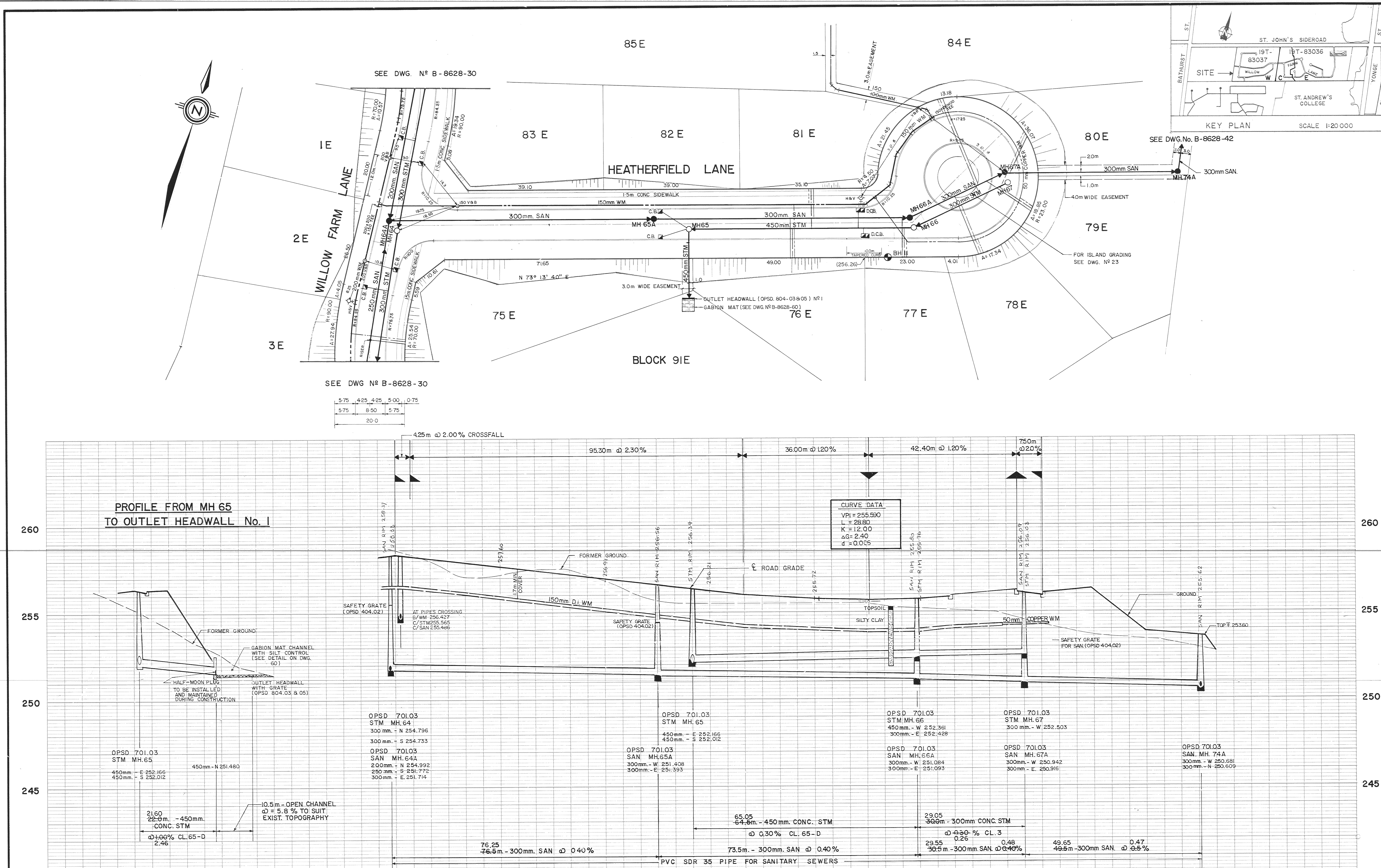
WILLOW FARM LANE

STA. 1+425 to 1+612.98

TOWN OF AURORA

ENGINEERING DEPARTMENT

DESIGN Y.R.	SCALE HOR. 1:500	VERT. 1:100
DRAWN D.K., L.J.K.	REVIEWED T.E.	DRAWING Nº B-8628-30
DATE JUNE '87	SHEET Nº 7 OF 7	



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CHAINAGE	0+000	0+004.25	0+025	0+050	0+075	0+099.55	B.V.C. 0+121.15	V.P.I. 0+135.55	0+149.95 E.V.C.	0+177.95	0+185.45

GENERAL NOTES

Nº	REVISIONS	DATE	APPROVED
1.	REVISED AS PER TOWN'S COMMENTS	NOV. 87	
2.	REVISED AS PER TOWN'S COMMENTS	FEB.88	
3.	SAN., STM. SEWERS AND ROAD GRADES REVISED	MAR.88	
4.	REVISED AS CONSTRUCTED SEWERS INVERTS	JAN. 89	
5.	AS CONSTRUCTED SEWERS LENGTH AND GRADE	JAN.91	
6.	AS CONSTRUCTED ℄ ROADS & WM TIES	NOV.94	

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Telephone: (416) 787-4935 / 0348

REGISTERED PROFESSIONAL ENGINEER
R. C. ... NOE

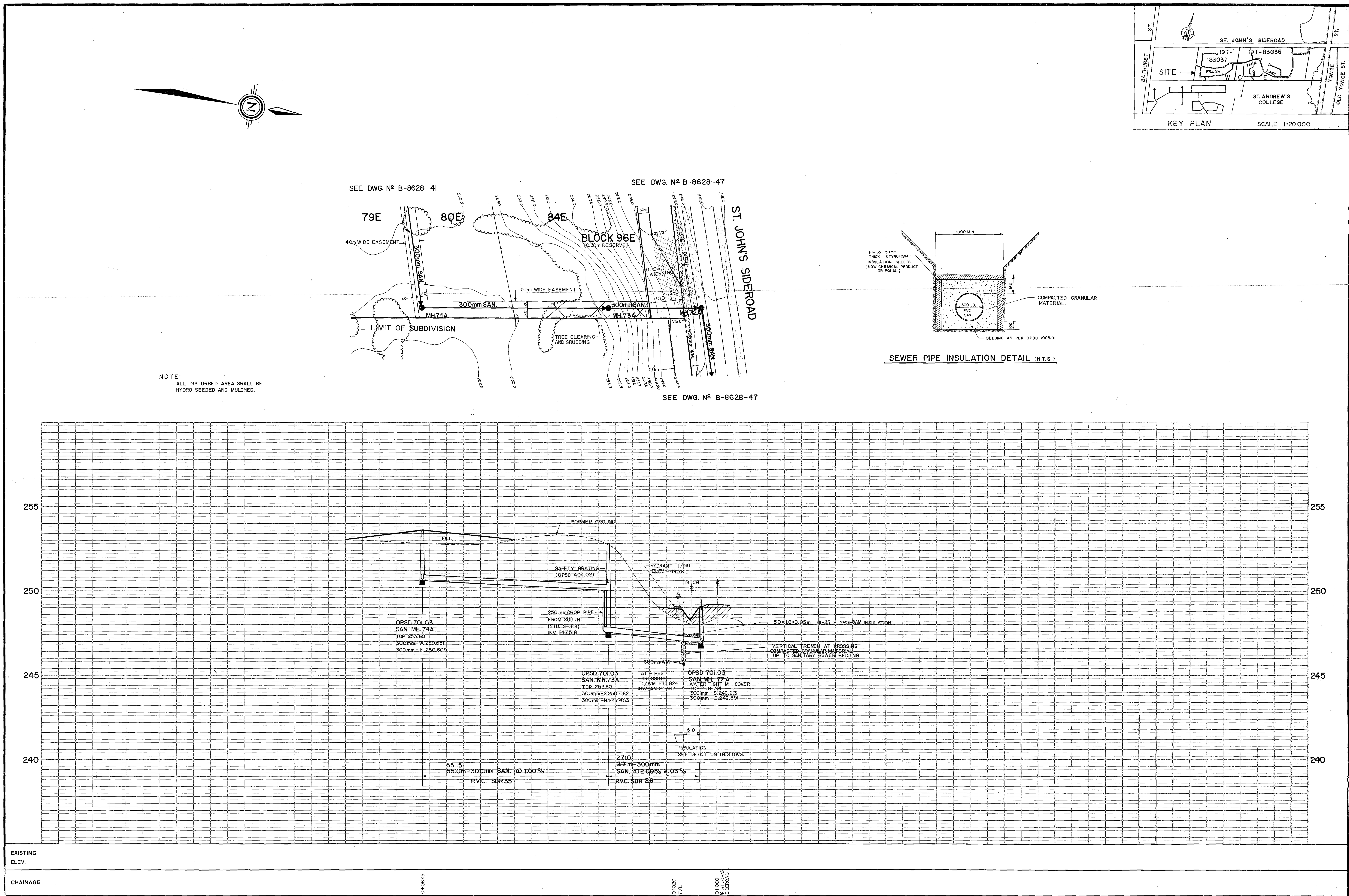
TOWN OF AURORA
APPROVED
DATE Aug. 10th 1988
William G. Walker
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON · THE · HILL
PLAN AND PROFILE
HEATHERFIELD LANE
STA. 0+000 to 0+184.45

TOWN OF AURORA
ENGINEERING DEPARTMENT

DESIGN Y.R.	SCALE HOR. 1:500	VERT. 1:100	
DRAWN L.J.K., K.A.	REVIEWED Y.R.	DRAWING Nº B-8628-41	
DATE JUNE '87	SHEET Nº OF		

4385-1-1



EXISTING ELEV.

CHAINAGE 0+087.5 0+020 P/L 0+000 ℄ ST. JOHN'S SIDEROAD

GENERAL NOTES

Nº	REVISIONS	DATE	APPROVED
1.	SANITARY SEWER RELOCATED	MAR. 88	
2	REVISED AS CONSTRUCTED SAN. SEWER INVERTS	JAN. 89	
3.	AS CONSTRUCTED SAN. SEWER LENGTH AND GRADE	JAN. 91	

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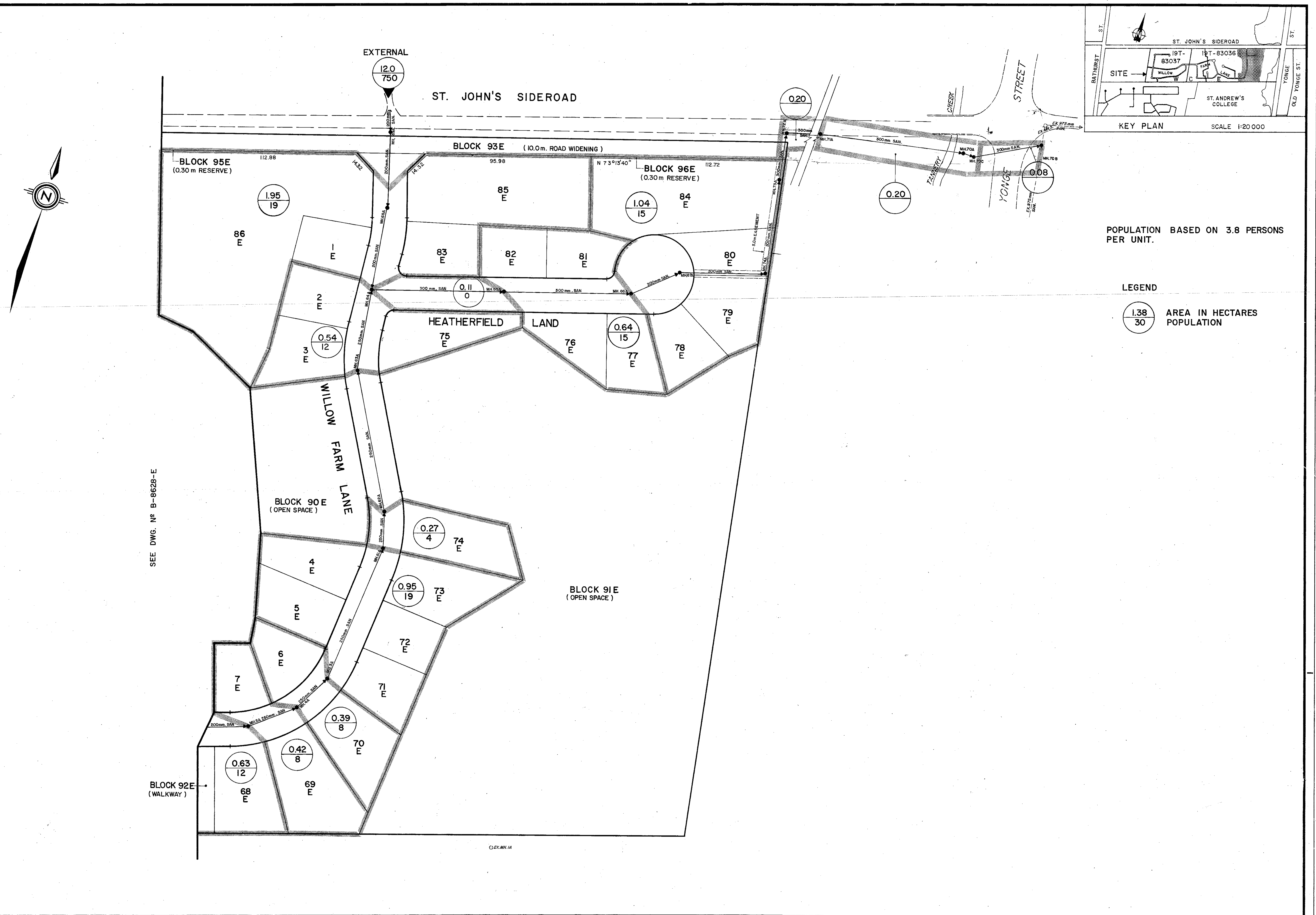
REGISTERED PROFESSIONAL ENGINEER — E. C. HINDE — PROVINCE OF ONTARIO

TOWN OF AURORA
APPROVED
DATE Aug. 10th 1988
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON · THE · HILL
PLAN AND PROFILE
SANITARY SEWER EASEMENT Nº 1.
STA.0+000 to 0+087.500

TOWN OF AURORA
ENGINEERING DEPARTMENT

DESIGN	Y.R.	SCALE HOR. 1:500	VERT. 1:100		
DRAWN	K.A.	REVIEWED T.E.	DRAWING Nº B-8628-42		
DATE	JUNE '87	SHEET Nº 1 OF 1			



GENERAL NOTES

Nº	REVISIONS	DATE	APPROVED
1.	REVISED AS PER TOWN'S COMMENTS	NOV. 87	
2.	SAN. SEWERS REVISED	MAR. 88	

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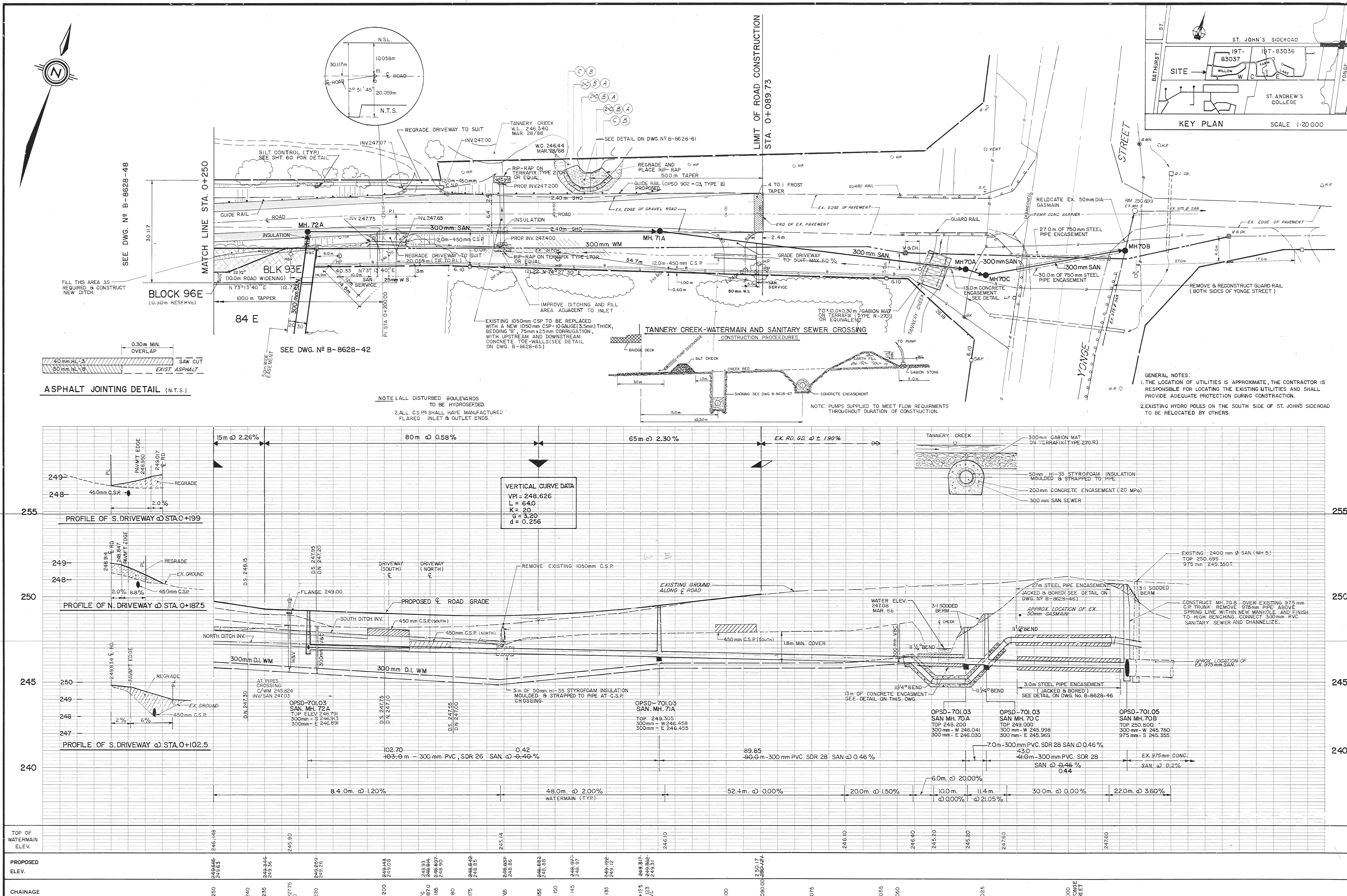
TOWN OF AURORA
APPROVED
DATE Aug. 10th 19 88
William G. Walker
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON · THE · HILL

SANITARY DRAINAGE PLAN

TOWN OF AURORA
ENGINEERING DEPARTMENT

DESIGN Y.R.	SCALE 1:1000		
DRAWN R.Z., L.J.K.	REVIEWED Y.R.	DRAWING Nº B-8628-F	
DATE AUG. '87	SHEET Nº 3 OF 3		



Nº	REVISIONS	DATE	APPROVED
7.	REVISED ROADWAY CULVERT TO CONCRETE	APR. 89	
8.	REVISED ROADWAY CULVERT TO CSP	MAY. 89	
9.	REVISED LOCATION OF MH.70A AND MH.70B	MAY. 89	
10	REVISED LOCATION OF MH.70 A AND NEW MH. 70C	SEP. 90	
11.	AS CONSTRUCTED SAN. SEWER LENGTH AND GRADE	JAN. 91	
12.	AS BUILT ℄ ROAD AND WM. TIES	MAR. 91	

Nº	REVISIONS	DATE	APPROVED
1.	REVISED AS PER TOWN'S COMMENTS	NOV. 87	
2.	REVISED AS PER TOWN'S COMMENTS	JAN. 88	
3.	REVISED AS PER TOWN'S COMMENTS	MAR. 88	
4	ALIGNMENT OF ℄ ROAD, DITCH LINING AND DITCH GRADING.	MAY 88	
5	REVISEDWM. OFFSET	MAY 88	
6.	REVISED AS CONSTRUCTED SAN. SEWER INVERTS	JAN. 89	
13.	AS BUILT WATER & SANITARY SERVICE TIES	MAY 92	

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REGISTERED PROFESSIONAL ENGINEER
CIVIL
R. C. HINDE
PROVINCE OF ONTARIO

TOWN OF AURORA
APPROVED
DATE Aug. 10th 1988
William G. Maier
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON·THE·HILL

PLAN AND PROFILE

ST. JOHN'S SIDEROAD

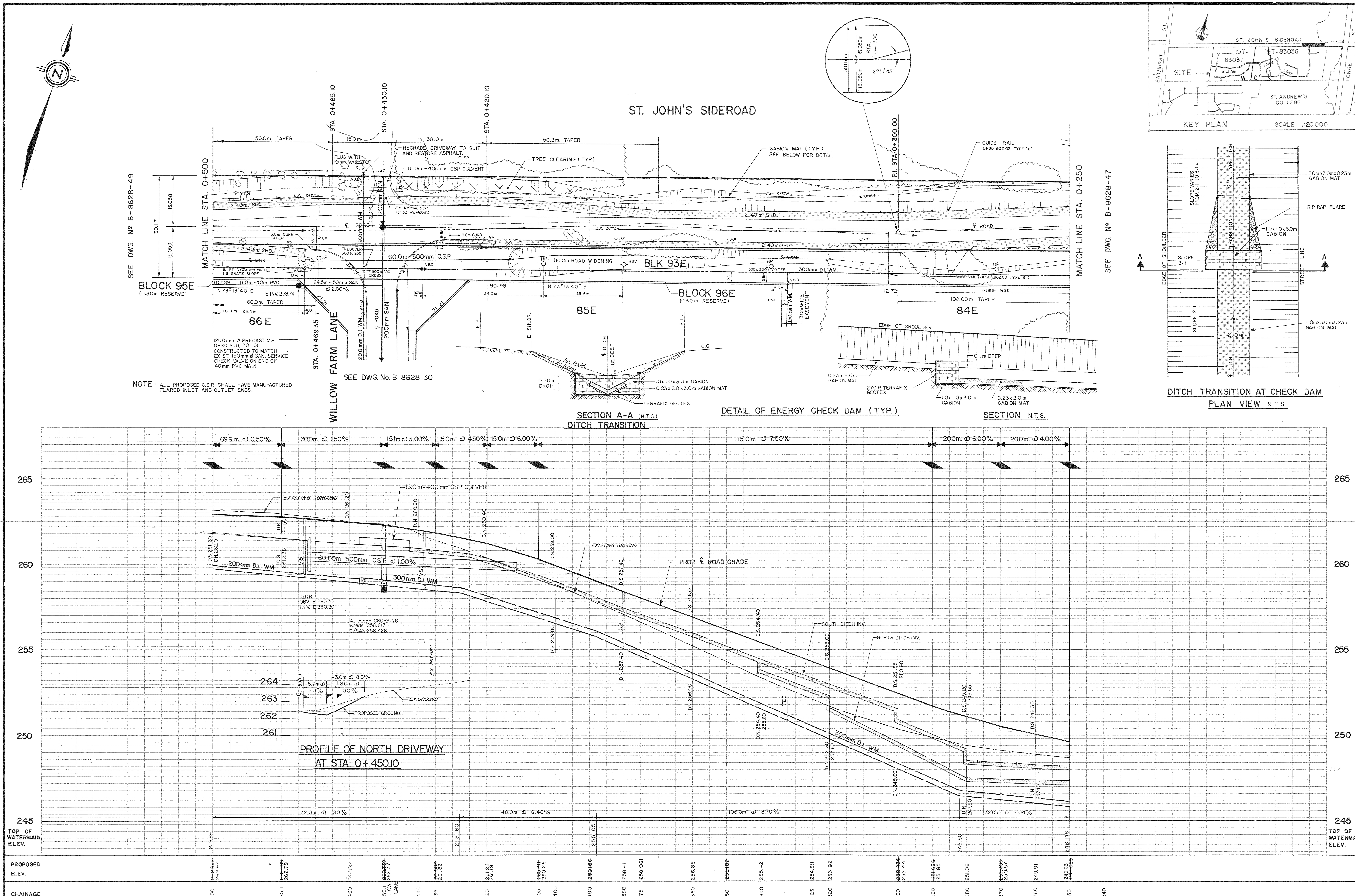
STA. 0+000 to 0+250

TOWN OF AURORA

ENGINEERING DEPARTMENT

DESIGN Y.R.	SCALE HOR. 1:500	VERT. 1:100	
DRAWN M.J.B.	REVIEWED Y.R.	DRAWING Nº B-8628-47	
DATE JUNE '87	SHEET Nº OF		

4035-1-2
5070-1-2
4035-1-2



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CHAINAGE	0+500	0+480.1	0+460	0+450.1 ℄ WILLOW FARM LANE	0+440	0+435	0+420	0+405	0+400	0+390	0+380	0+375	0+360	0+350	0+340	0+325	0+320	0+300	0+290	0+280	0+270	0+260	0+250	0+240	

Nº	REVISIONS	DATE	APPROVED
7.	LENGTHENED GUIDE RAIL, REVISED DITCH. LINES	APR. 89	
8.	AS BUILT ℄ ROAD AND WM. TIES.		

Nº	REVISIONS	DATE	APPROVED
1	AS PER TOWN'S COMMENTS	NOV. 87	
2.	AS PER TOWN'S COMMENTS	JAN. 88	
3.	AS PER TOWN'S COMMENTS	MAR. 88	
4.	ALIGNMENT OF ROAD ℄, DITCH CHECK DAMS, DITCH LINING AND DITCH GRADING.	MAY 88	
5.	REVISE WM. OFFSET, LENGTHEN 58m CSP TO 60m RELOCATE DICB TO 10m S. of ROAD ℄	NOV. 88	
6.	SAN. SERVICE	NOV. 88	

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CIVIL
R. C. HINDE
PROVINCE OF ONTARIO

TOWN OF AURORA
APPROVED
DATE August 19 88
William G. Walker
DIRECTOR OF PUBLIC WORKS

ST. ANDREWS ON·THE·HILL
PLAN AND PROFILE
ST. JOHN'S SIDEROAD
STA. 0+250 to 0+500

TOWN OF AURORA
ENGINEERING DEPARTMENT

DESIGN Y. R.	SCALE HOR. 1:500	VERT. 1:100	
DRAWN M. J., LJ.K.	REVIEWED T. E.	DRAWING NºB-8628-48	
DATE JUNE '87	SHEET Nº OF		

Table 1. Description of Berger (2010) Land Uses in the Lake Simcoe Watershed

Berger (2010) Land Use	Included LSRCA Land Use(s)	Land Use Description
Hay / Pasture	Non-intensive Agriculture	Hay and pasture fields, including the related agricultural buildings such as barns, silos and the farm residence. Fields are dominated with herbaceous vegetation and grasses with an understory of similar material in a state of decay. Weedy hay and/or pasture covers more than 50% of the area.
Crop Land	Intensive Agriculture	Cultivated row crops, including the related agricultural buildings (e.g., barns, silos and the farm residence), producing crops in varying degrees (e.g., corn and wheat) and includes specialty agriculture (i.e., orchards, market gardens, Christmas tree plantations and nurseries).
Sod Farm / Golf Course	Sod Farm	Sod farms.
	Golf Course	Golf courses, including lane ways, but not the isolated woodlots within, unless the area of the woodlots is < 0.5 ha.
Low Intensity Development	Estate Residential	A home including the manicured area around the home and driveway, within a natural heritage feature. The natural heritage feature is not included in the Estate Residential land use classification.
	Manicured Open Space	Cleared areas with a low density of trees, including lawns and landscaping. Land use is dominated by gardens, parkland and lawns, e.g., cemeteries, urban parks, ski hills and residential/industrial open space with a minimum size of 2 ha.
	Rail	Rail lines and the associated cleared adjacent areas.
	Rural Development	Properties not directly associated with an agricultural operation and that contain residential, commercial or other buildings, as well as a manicured open space, within a natural heritage or agricultural feature (e.g., estate residential or service station). On developed portions, these properties are under intensive use. Based on canopy cover, these areas will often appear as Cultural Savannah or Cultural Woodland in aerial photographs or satellite imagery. However, the presence of buildings and manicured lands identify the properties as Rural Development.
High Intensity Development[1] (Commercial /Industrial)	Commercial	Impervious properties that contain a building and an adjacent parking lot (e.g., shopping and strip malls, power centres, scrap yards). Excludes green land areas such as parks or river valleys.
	Industrial	Impervious properties that are not commercial and include industrial operations e.g., factories, manufacturing facilities, processing facilities, bulk fuel storage. Excludes green land areas such as parks or river valleys.
	Institutional	Schools, hospitals and other institutional structures. May include large storm water management ponds. Excludes green land areas such as parks or river valleys.
High Intensity Development[1] (Residential)	Urban	Urban related land uses including continuous ribbon development. Interpreted from aerial photographs or satellite imagery by many roof tops and/or groupings of 5 or more residential properties with a combined area of ≥ 2 ha. Residential properties include single and semi-detached dwellings, apartment buildings and associated out-buildings, driveways and parking lots. Excludes green land areas such as parks or river valleys.
Quarry	Active Aggregate	Areas that are currently being excavated or have recently been excavated. Identified by pits, extraction machinery, unvegetated landscape and/or piles of extracted materials. Active aggregate areas may contain open water.
	Inactive Aggregate	Former aggregate sites that have been recently revegetated; vegetation is established and growing. Depending on their characteristics, in aerial photographs or satellite imagery, these properties may appear to be comparable to an abandoned field or forming wetland.
Road	Road	Unpaved roads, including the shoulder. Does not include driveways.

APPENDIX B2

RAINSCAPING MEETING MINUTES

MEETING MINUTES

File #:	**2183**
Date:	**January 5, 2021**

Project:	**Shining Hill Estates**
Purpose:	**Rainscaping Design Charrette**
Date/Time of Meeting:	**December 15, 2020 – 1:00 pm to 3:30 pm**
Location:	**SCS hosted Zoom**
Next Meeting:	**TBD**

	Recipient(s):	Email:
Attendees:	Rob Baldwin, LSRCA	r.baldwin@lsrca.on.ca
	Melinda Bessey, LSRCA	m.bessey@lsrca.on.ca
	Phil Thase, LSRCA	p.thase@lsrca.on.ca
	Dave Ruggle, LSRCA	d.ruggle@lsrca.on.ca
	Jessica Chan, LSRCA	j.chan@lsrca.on.ca
	Shelly Cuddy, LSRCA	s.cuddy@lsrca.on.ca
	Bill Butler, Aurora	bbutler@aurora.ca
	Glen McArthur, Aurora	gmcArthur@aurora.ca
	Rosanna Punit, Aurora	rpunit@aurora.ca
	Brian Jakovina, Aurora	bjakovina@aurora.ca
	Peter Noehammer, Newmarket	pnoehammer@newmarket.ca
	Jason Unger, Newmarket	junger@newmarket.ca
	Craig Schritt, Newmarket	cschritt@newmarket.ca
	Meghan White, Newmarket	mwhite@newmarket.ca
	Adrian Cammaert, Newmarket	acammaert@newmarket.ca
	Jeff Bond, Newmarket	jbond@newmarket.ca
	Paul Bailey, Shining Hill Estates Collection Inc.	paul@bazil.ca
	Brian Henshaw, Beacon	bhenshaw@beaconenviro.com
	Chana Steinberg, Beacon	csteinberg@beaconenviro.com
	Don Given, MGP	dgiven@mgp.ca
	Lincoln Lo, MGP	llo@mgp.ca
	Diane Russelle, MGP	drusselle@mgp.ca
	Rohan Sovig, MGP	rsovig@mgp.ca
	Allyssa Hrynyk, MGP	ahrynyk@mgp.ca
	Steve Schaefer, SCS	sschaefer@scsconsultinggroup.com

	Sarah Kurtz, SCS	skurtz@scsconsultinggroup.com
	Erich Knechtel, SCS	eknechtel@scsconsultinggroup.com
	Ben O'Neill, SCS	boneill@scsconsultinggroup.com
Absentees:	Rachel Prudhomme, Newmarket	rprudhomme@newmarket.ca
	Sepideh Majdi, Newmarket	smajdi@newmarket.ca
	Victoria Klyuev, Newmarket	vklyuev@newmarket.ca
	Mark Agnoletto, Newmarket	magnoletto@newmarket.ca
	Gordon Macmillan, Newmarket	gmacmillan@newmarket.ca
	David Waters, Aurora	dwaters@aurora.ca
	Gary Greidanus, Aurora	ggreidanus@aurora.ca
	Jim Tree, Aurora	jtree@aurora.ca
cc:	Mumta Mistry, Soil Engineers Ltd.	mumta@soilengineersltd.com
	Joel Gopaul, Golder	joel_gopaul2@golder.com
	Chris Kozuskanich, Golder	chris_kozuskanich@golder.com

The following is considered to be a true and accurate record of the items discussed. Any errors or omissions in these minutes should be provided in writing to the author immediately.

The purpose of this meeting was to establish a suite of LID and SWM solutions in consultation with Aurora, Newmarket, and the LSRCA, for application in the Shining Hill Estates Phase 3 functional servicing design.

Item:	**Action:**
1.0 Rainscaping Charrette Presentation	
1.1 Planning Status	
• St. Anne's School (Aurora) occupancy targeted for September 2022 • Phase 3 Newmarket requires Official Plan amendment and urban zoning designation. Secondary plan level reports required prior to draft plan approval.	Info
1.2 Geotechnical Investigation	
• Long term stable top of slope (LTSTOS) generally follows physical top of slope except in localized area in south east of Phase 3 (Aurora). • LTSTOS still to be evaluated near SAS driveway and all of Phase 3 within Newmarket. • Soils generally suitable for surface retention, clay liner required in sandy areas. • Varved clays encountered in many areas having a low estimated percolation rate.	Info

Item:	**Action:**
1.3 Hydrogeology (Golder)	
• Municipal wells in Aurora near the site (southeast corner of Yonge Street and St. John's Sideroad) tap into the deep Thorncliffe aquifer. Impacts to water quality from the proposed development are expected to be minimal because of the depth of the wells, the low permeability clay, and the groundwater gradient which generally discharges to the Tannery Creek tributaries within the site. • Groundwater depth ranges from 2 m to 6.5 m, most shallow towards the east. Measured in December 2020, spring monitoring required to establish seasonal high elevations. • WHPA-Q1 requires matching pre-development recharge.	Info
1.4 Ecology and Constraints	
• Recommending minimum vegetation protection zone (MVPZ) of: 10 m to dripline/woodland, 30 m to watercourse, and 15 m to wetlands. • A reduced MVPZ is recommended adjacent to the existing St. Anne's School driveway access. It is 3 m to the woodland/dripline and 6 m to the wetland. This is consistent with the existing condition. • Regulatory floodline generally not the limiting constraint due to deep valley corridors. • Existing drainage boundaries map is attached.	Info
1.5 Development Concept and Preliminary Engineering	
• Steep road connections expected to St. John's and Bathurst. • Expecting road grades between 0.7% and 5%, and lot grades between 2% and 5%. • Sloping or walls could be required at some locations around the site perimeter to make up grade.	Info
1.6 Stormwater Management	
• LSRCA's guidelines are the principal SWM criteria. • Constraints are low permeability soils, steep topography for grading, and shallow to moderate groundwater depth. • Opportunities are: underground storage in park blocks, infiltration/filtration in boulevard LIDs, steep topography for storm outfall flexibility.	Info

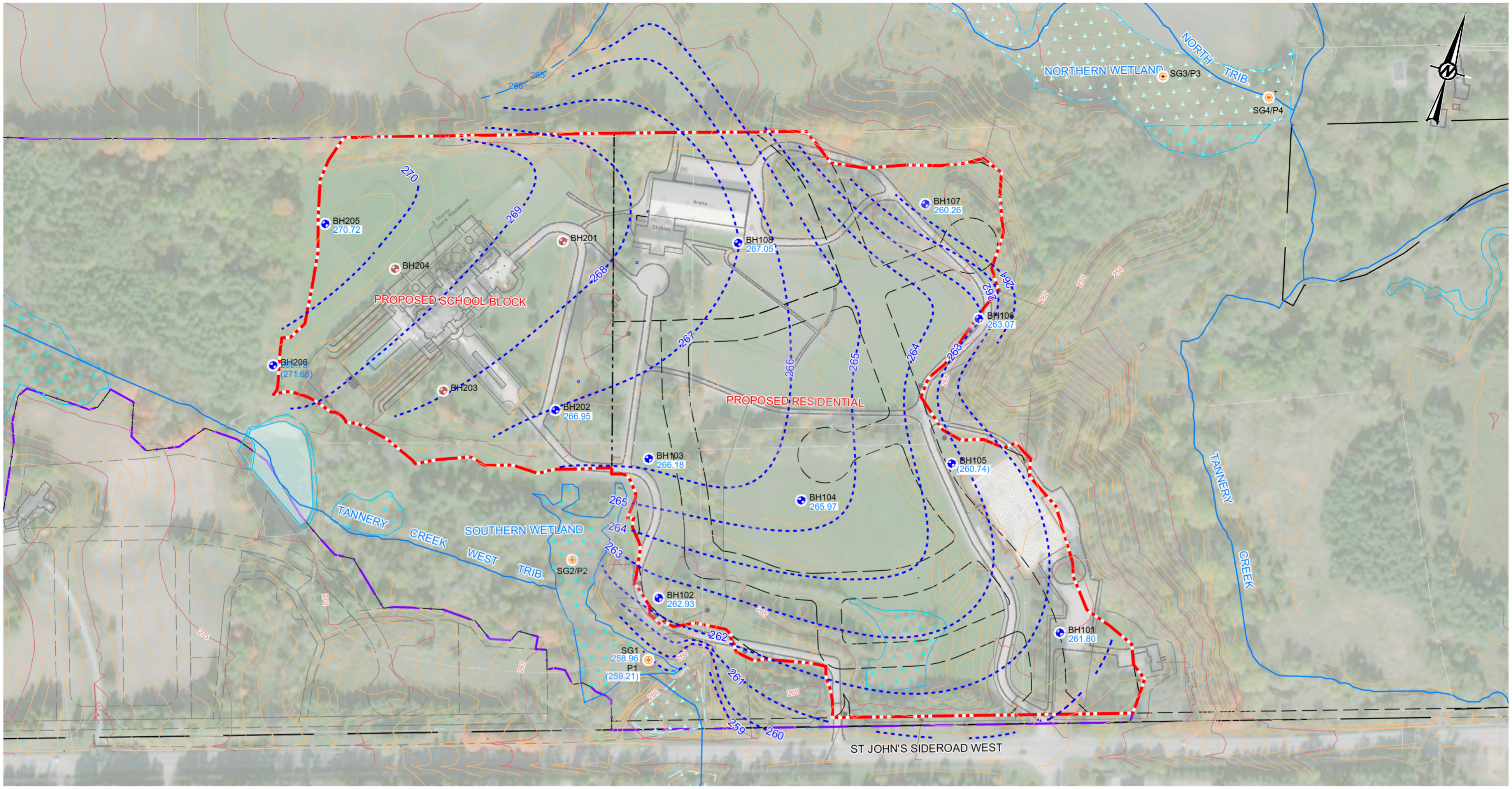
Item:	**Action:**
2.0 Municipal Feedback on LIDs and SWM - Aurora	
2.1 Bioswales and Grassed Swales (in boulevard or elsewhere)	
• Aurora has some experience maintaining grassed bioswales and grassed swales that have worked well. Has experienced some issues with sediment build up at curb cut inlets and short circuiting. • Aurora open to implementing bioswales/grassed swales in boulevards in the future. Notes that more focus should be given to operations and maintenance manuals, and protection during construction. • Driveways will limit these LIDs, but many single loaded roads in the plan present opportunities.	Info
2.2 Permeable Pavement	
• Aurora has had issues with clogging and short circuiting.	Info
2.3 Catchbasin Infiltration/Exfiltration	
• Aurora currently operating and monitoring some of these systems. But only in a small number. No issues thus far. • Unlikely to accept them under the road or hard surfaces. • Not preferred relative to surface LIDs because easy visual inspection from the surface is not possible. • LSRCA/SCS note there are design alternatives to improve ease of inspection and maintenance, such as inspection ports and cleanouts for flushing. • Access in the event of reconstruction is good when this LID is located under a pervious surface behind the curb line, and they are also at a shallower depth in this configuration.	Info

Item:	**Action:**
2.4 General/Other	
• Town has concerns over yearly sediment load in catchbasins. • It will be easier for the Town to accept LIDs if they are well protected during construction and ready to be certified immediately. Focus should be on managing impacts of sedimentation on Town operations. • Aurora does not accept rain gardens on lots for municipal maintenance (i.e. easement and municipal maintenance agreement). SCS notes that privately owned ones can still contribute to water balance and phosphorous removal. • LSRCA/SCS note possibility of super elevating roads draining to rain gardens on one side of the ROW. It was noted that Highland Gate utilized superelevated roads. Town noted that Operations may not be favourable to superelevated roads and that further discussions would be required. • Aurora Development Engineering supports underground storage usage in parks, but noted that discussions with Parks is necessary (Brian Jakovina to confirm with Parks). Easy truck and maintenance access are key. Also supports this approach to optimize land use. It was noted that LSRCA is working with City of Barrie to utilize underground storage/infiltration below programmed parks extensively, which provide good truck access utilizing hard surfaces from the programmed parks.	Info
3.0 Municipal Feedback on SWM and LIDs - Newmarket	
3.1 General	
• Newmarket not excluding any LIDs or SWM methods at this stage. • Town has ceased using sand for winter road maintenance which should make LID maintenance easier in the future.	Info
4.0 Design Charrette (Aurora)	
4.1 The owner anticipates the Aurora medium density block will be a mid-rise apartment-style building. There are opportunities for a variety of LIDs given that this will be a private site plan.	Info
4.2 MGP noted that minor building additions are expected on the SAS site in the long term. MGP and Aurora to discuss greenspace access for the school. Accommodation for increased impervious should be included in the SWM design.	Aurora/MGP
4.3 Aurora prefers LIDs under grassed areas rather that under hard surfaces.	Info

Item:		**Action:**
4.4	Aurora noted that all LIDs on a lot must be privately owned. Aurora will not provide any maintenance for such LIDs. SCS noted that such LIDs can be used toward water balance and phosphorus removal credit. Developer obtains an ECA for the private lot LIDs, which are removed from the ECA upon transfer to the municipality.	Info
4.5	Three outlets are expected for the Aurora lands. Likely superpipe to the southwest with outfall to the westerly Tannery Creek tributary; underground SWM in park with outfall to the northeast; and a superpipe connecting to St. John's Sideroad and discharging northeast to Tannery Creek.	Info
4.6	Aurora noted limited experience with curb cuts relative to the amount sketched on the plan. Bioswales are viable, but curb cut inlets have been a source of maintenance issues. Suggested the developer/engineer explore mitigation measures such as wider inlets.	Info
5.0	**Design Charrette (Newmarket)**	
5.1	The engineering preference for the northern most watercourse crossing is to align it south of the existing farm crossing. This alignment locates the SWM block in a lower area, and eliminates unnecessary bends in the road.	Info
5.2	Newmarket noted the use of underground SWM infrastructure with park land above would have to be discussed at a later date. Newmarket is open to this idea if it uses land more wisely. Newmarket has experience with this approach in the Mosaik subdivision, and is learning from the ongoing maintenance of this SWM infrastructure.	Info
5.3	LSRCA suggested mandating the school block provide on-site LID control.	Info
5.4	LSRCA noted the restoration/trail block is a good location for compensation plantings for proposed crossing disturbances. Newmarket expressed interest in increasing woodland continuity using this block. MGP, Newmarket, LSRCA, Beacon to discuss further.	MGP Newmarket LSRCA Beacon
5.5	Newmarket noted to consider boulevard swale depth at detailed design and that it does not inhibit grass cutting, or else it could lead to homeowner tampering.	Info
6.0	**Design Charrette (General)**	
6.1	LSRCA noted opportunities for localized SWM treatment at end of the cul-de-sacs in the concept plan.	Info
6.2	Many single loaded roads exist in the plan, and present opportunities for boulevard LIDs at the surface (e.g.. bioswales).	Info

APPENDIX B3

HYDROGEOLOGICAL ASSESSMENT EXCERPTS



0 50 100 150 m
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LEGEND

- PHASE 3 (AURORA) BOUNDARY
- SOIL ENG MONITORING WELL
- SOIL ENG BOREHOLE
- GOLDER PIEZOMETER / STAFF GAUGE
- 262.42 STATIC WATER LEVEL (JANUARY 2021)
- (258.99) LOWER SCREENED POTENTIAL (masl)
- WATER TABLE CONTOUR (masl)
- WETLAND (BEACON)
- SITE CONTOURS (1 masl INTERVAL)

REFERENCES AND NOTES

1. MAPPING BASED ON ESRI GEOGRAPHY NETWORK OBM FEATURES AND CLIENT CAD COMPILATIONS
2. WETLAND AND ECOLOGICAL FEATURES, BEACON, FEBRUARY 2021
3. MAPPED FEATURES AND LOCATIONS ARE APPROXIMATE AND NOT TO SCALE
4. BETWEEN THE WELLS AND TEST WELLS, CONTOURED SURFACES ARE NOT PROVEN.
5. LOWER SCREENED POTENTIALS (LEVELS IN BRACKETS) NOT USED FOR INTERPOLATION.

CLIENT
SHINING HILL ESTATES COLLECTION INC.
C/O SCS CONSULTING GROUP LTD.

CONSULTANT
GOLDER
MEMBER OF WSP

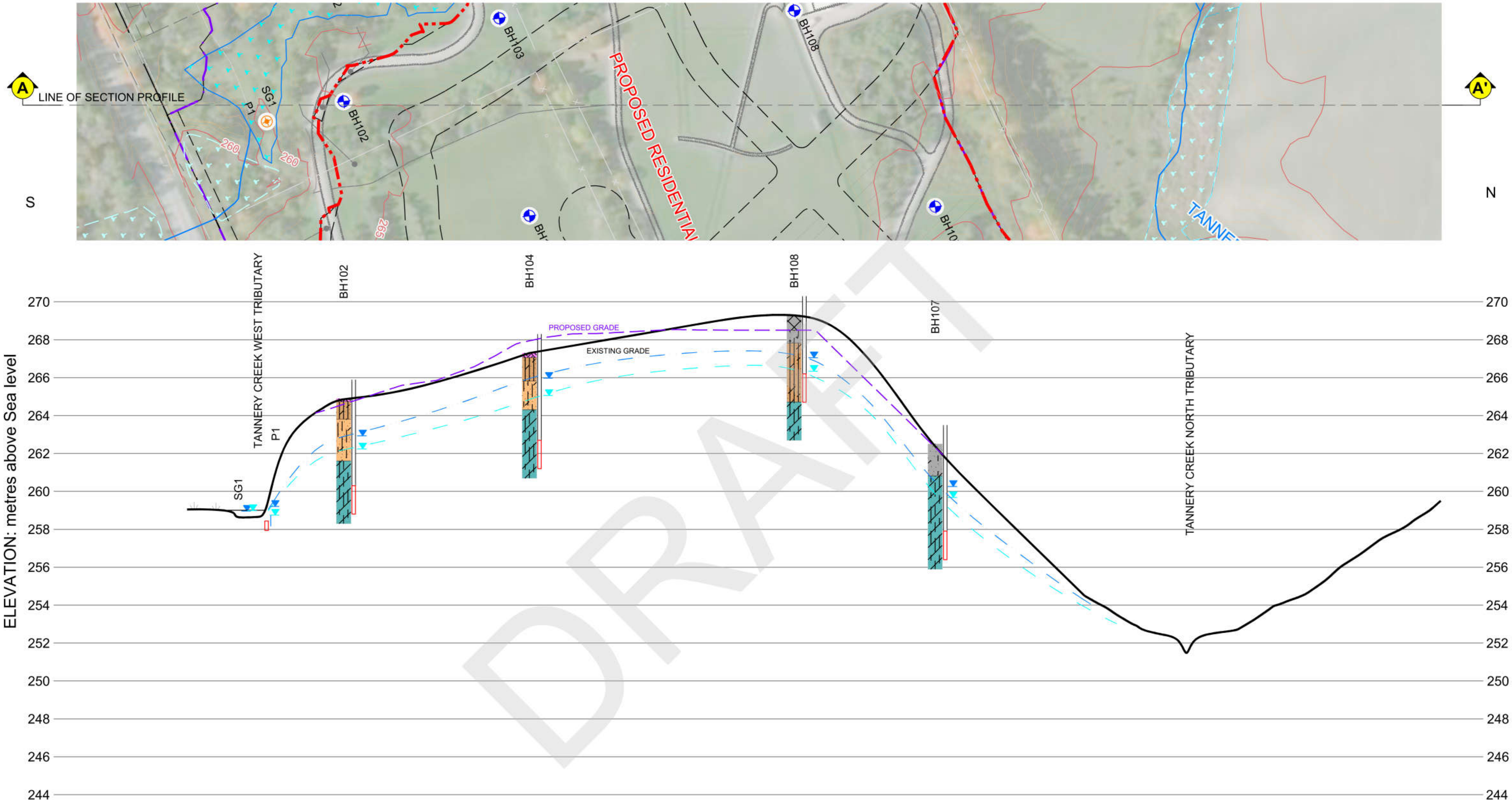
YYYY-MM-DD	2021-12-15
DESIGNED	
PREPARED	JPR
REVIEWED	JG
APPROVED	CMK

PROJECT
HYDROGEOLOGICAL INVESTIGATION
SHINING HILL (PHASE 3)
162, 306, 370, 434 & 488 ST. JOHN'S SIDEROAD WEST, AURORA

TITLE
WATER TABLE JANUARY 2021

PROJECT NO.	CONTROL	REV.	FIGURE
20360612	0001	B	8

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



SOILS LEGEND

- EARTH / TOPSOIL FILL
- GRANULAR FILL
- SAND
- SAND TRACE SILT
- SILTY SAND
- TOPSOIL
- SILTY SAND TRACE CLAY
- SANDY SILT WITH TO TRACE CLAY
- SILTY CLAY

SECTION LEGEND

- BH1 BOREHOLE ID
- RECORDED STATIC WATER LEVEL 1st DECEMBER 2020
- RECORDED STATIC WATER LEVEL JANUARY 2021
- SCREEN

BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DETERMINED ONLY AT WELL AND TEST WELL LOCATIONS. BETWEEN THE WELLS AND TEST WELLS, BOUNDARIES ARE NOT PROVEN BUT ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

0 40 80 120 m

1:2000

CLIENT
SHINING HILL ESTATES COLLECTION INC.
C/O SCS CONSULTING GROUP LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-12-15
DESIGNED	
PREPARED	JPR
REVIEWED	JG
APPROVED	CMK

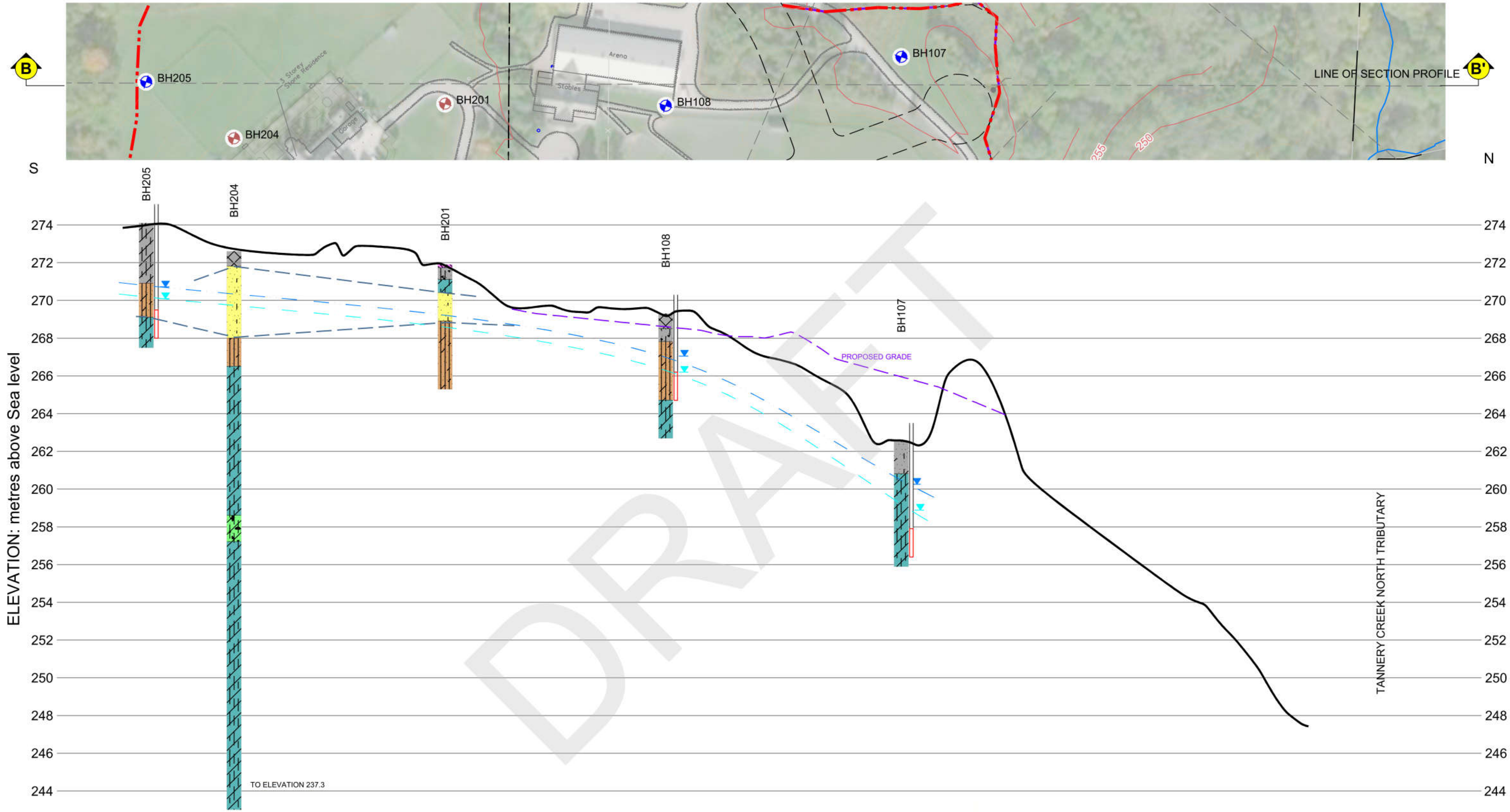
PROJECT
HYDROGEOLOGICAL INVESTIGATION
SHINING HILL (PHASE 3)
162, 306, 370, 434 & 488 ST. JOHN'S SIDEROAD WEST, AURORA

TITLE
SITE SECTION A - A'

PROJECT NO.	CONTROL	REV.	FIGURE
20360612	0001	B	7A

25 mm

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



SOILS LEGEND

- EARTH / TOPSOIL FILL
- GRANULAR FILL
- SAND
- SAND TRACE SILT
- SILTY SAND
- TOPSOIL
- SILTY SAND TRACE CLAY
- SANDY SILT WITH TO TRACE CLAY
- SILTY CLAY
- SILTY SAND DIAMICT
- SILTY CLAY DIAMICT

SECTION LEGEND

- BH1 BOREHOLE ID
- RECORDED STATIC WATER LEVEL 1st DECEMBER 2020
- RECORDED STATIC WATER LEVEL JANUARY 2021
- SCREEN

BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DETERMINED ONLY AT WELL AND TEST WELL LOCATIONS. BETWEEN THE WELLS AND TEST WELLS, BOUNDARIES ARE NOT PROVEN BUT ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

0 40 80 120 m
1:2000

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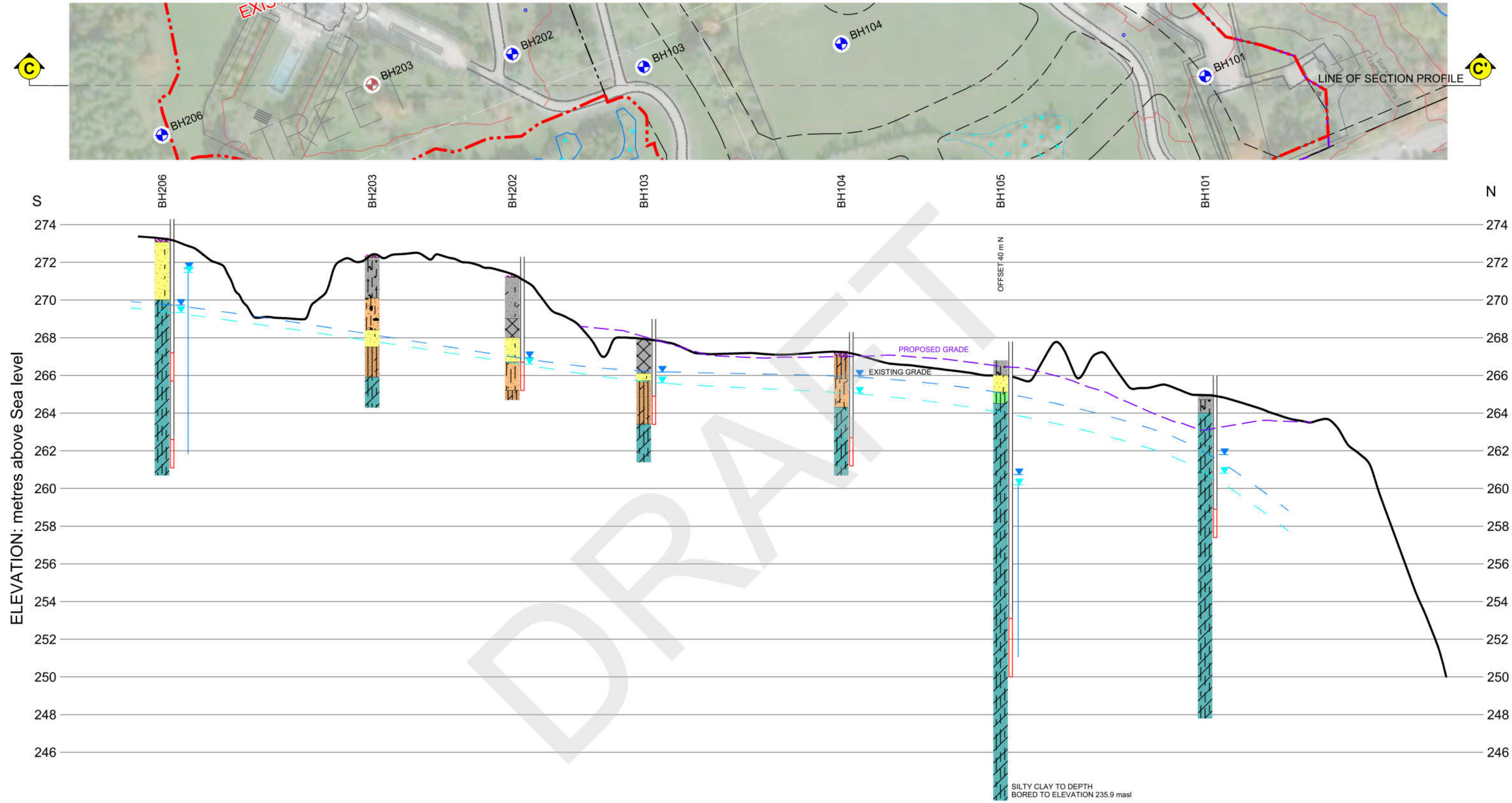
SHINING HILL (PHASE 3)
162, 306, 370, 434 & 488 ST. JOHN'S SIDEROAD WEST, AURORA

TITLE
SITE SECTION B - B'

PROJECT NO.	CONTROL	REV.	FIGURE
20360612	0001	B	7B



SOILS LEGEND

- EARTH / TOPSOIL FILL
- GRANULAR FILL
- SAND
- SAND TRACE SILT
- SILTY SAND
- TOPSOIL
- SILTY SAND TRACE CLAY
- SANDY SILT WITH TO TRACE CLAY
- SILTY CLAY
- SILTY SAND DIAMICT
- SILTY CLAY DIAMICT

SECTION LEGEND

- BH1 BOREHOLE ID
- RECORDED STATIC WATER LEVEL 1st DECEMBER 2020
- RECORDED STATIC WATER LEVEL JANUARY 2021
- SCREEN

BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DETERMINED ONLY AT WELL AND TEST WELL LOCATIONS. BETWEEN THE WELLS AND TEST WELLS, BOUNDARIES ARE NOT PROVEN BUT ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

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PROJECT
HYDROGEOLOGICAL INVESTIGATION
SHINING HILL (PHASE 3)
162, 306, 370, 434 & 488 ST. JOHN'S SIDEROAD WEST, AURORA

TITLE
SITE SECTION C - C'

PROJECT NO.	CONTROL	REV.	FIGURE
20360612	0001	B	7C

Staff gauge SG2, located in the palustrine portion of the Southern Wetland, was dry and groundwater levels below the base of the piezometer (June) or below grade (September and November) were recorded at piezometer P2 on four monitoring events in June, September and November 2021. Shallow groundwater depths with an increasing trend from early-October through mid-November 2021 at P2 are illustrated on the hydrograph presented on Figure E-6. These observations are consistent with the location of P2/SG2 in the palustrine portion of the wetland and suggest this portion of the wetland is supported by at least seasonally high groundwater levels.

Staff gauge SG3 and SG4, located in the Northern Wetland, were dry on all five monitoring events in June and September 2021. Below-grade heads were recorded at piezometers P3 and P4, with fluctuating groundwater levels ranging in depth from 1.23 mbgs (P3 on June 2, 2021) to 0.05 mbgs (P4 on September 29, 2021). These observations are consistent with the classification of the Tannery Creek North Tributary as intermittent, and suggest that the Northern Wetland is supported in part by groundwater levels that fluctuate at times close to grade.

3.5 Hydraulic Testing

Single well response testing (i.e., rising head tests) was carried out at BH103, BH108 and BH202 on November 16, 2020, and at BH101 and BH206-S (shallow) on December 1, 2020. The rising head tests were carried out by rapidly lowering the water levels by purging with a dedicated Waterra footvalve and tubing. The resulting water level recoveries were monitored with an electronic water level tape and automatic data logger. The recovery data were analyzed using the AQTESOLV for Windows (1996 – 2007) Version 4.5 software. The Bouwer and Rice (1976) method for unconfined conditions was applied to the rising head test data. Estimates of hydraulic conductivity (K) obtained from the rising head tests are summarized below in Table 1. Summary printouts of the rising head test data and results from AQTESOLV are included in Appendix F.

Table 1: Summary of Estimated Hydraulic Conductivity

Borehole	Unit Screened	Depth of Monitoring Well (mbgs)	Method	K (cm/s)
BH101	SILTY CLAY	7.6	Bouwer and Rice (1976), unconfined	$2x10^{-6}$
BH103	SANDY SILT	4.6	Bouwer and Rice (1976), unconfined	$1x10^{-3}$
BH108	SANDY SILT	4.6	Bouwer and Rice (1976), unconfined	$5x10^{-4}$
BH202	SILTY FINE SAND	6.1	Bouwer and Rice (1976), unconfined	$4x10^{-4}$
BH206-S (shallow)	SILTY CLAY	7.6	Bouwer and Rice (1976), unconfined	$3x10^{-6}$

Notes:
mbgs – metres below ground surface. cm/s – centimetres per second

The hydraulic conductivity estimates for the non-cohesive sandy silt and silty fine sand soils ranged from $4x10^{-4}$ cm/s to $1x10^{-3}$ cm/s, with a geometric mean of $6x10^{-4}$ cm/s (n=3). The hydraulic conductivity estimates for the silty clay unit ranged from $2x10^{-6}$ cm/s to $3x10^{-6}$ cm/s, with a geometric mean of $3x10^{-6}$ cm/s (n=2). The estimated hydraulic conductivity values are considered reasonable for the units tested.

3.6 Guelph Permeameter Testing

Soil infiltration rate testing was carried out on November 24, 2020 in the unsaturated zone using a Guelph Permeameter (Soilmoisture Equipment Corp., Model 2800K1). The Guelph Permeameter was operated in general accordance with the procedures outlined by the manufacturer (Soilmoisture Equipment Corp., 2012) using a single head method. The apparatus was installed at the base of hand-augered test holes.

Once the outflow of water at the depth of installation reached a steady-state flow rate, the field-saturated hydraulic conductivity, K_{fs}, of the soil was estimated using the following equation (Elrick et. al., 1989):

$$K_{fs} = \frac{C_1 Q_1}{2\,\pi\, H_1^2 + \pi\, a^2\, C_1 + 2\,\pi \frac{H_1}{\alpha^*}}$$

Where: C_1 = shape factor
Q_1 = flow rate (cm^3/s)
H_1 = water column height (cm)
a = well radius (cm)
α^* = alpha factor (0.12 cm^{-1} for Type 3 soils)

The field data and analysis of the infiltration rate tests are presented as Figures F-1 to F-5, Appendix F. Based on the resulting K_{fs} (cm/s), the corresponding infiltration rates (mm/hr) were estimated using the approximate relationship presented in the *Low Impact Development Stormwater Management Planning and Design Guide* (or "*Design Guide*") (TRCA and CVCA, 2010). A summary of the infiltration rate test results is presented in Table 2, below.

Table 2: Summary of Estimated Infiltration Rates

Test	Soil Description	Test Depth Relative to Grade (mbgs)	Approximate Test Elevation [6] (masl)	Est. Field-Saturated Hydraulic Conductivity K_{fs} (cm/s)	Estimated Infiltration Rate [1] (mm/hr)	Correction Factor	Corrected Estimated Infiltration Rate [2] (mm/hr)
GP-101 (near BH101)	Inferred SILTY SAND (FILL)[3]	1.0	264.0	$9x10^{-5}$	49	3.5	14
GP-102 (near BH102)	SILT	0.7	264.2	$1x10^{-5}$	30	2.5[5]	12
GP-105 (near BH105)	SAND	0.8	266.0	$1x10^{-4}$	50	3.5	14
GP-106 (near BH106)	Inferred SILTY FINE SAND[4]	1.1	264.2	$3x10^{-4}$	62	3.5	18

Test	Soil Description	Test Depth Relative to Grade (mbgs)	Approximate Test Elevation [6] (masl)	Est. Field-Saturated Hydraulic Conductivity K_{fs} (cm/s)	Estimated Infiltration Rate [1] (mm/hr)	Correction Factor	Corrected Estimated Infiltration Rate [2] (mm/hr)
GP-206 (near BH206)	SAND	0.7	272.6	$1x10^{-3}$	75	2.5[5]	30

Notes:
mbgs – metres below ground surface. cm/s - centimetres per second. mm/hr – millimetres per hour
[1] – based on Table C1 from TRCA and CVCA (2010).
[2] – correction factor in accordance with Table C2 from TRCA and CVCA (2010).
[3] – the base of the test hole was near the contact point between silty sand fill and the underlying silty clay unit. In Golder's opinion, this result is more representative of silty sand fill.
[4] – the base of the test hole was near the contact point between silty fine sand and the underlying silty clay unit. In Golder's opinion, this result is more representative of silty fine sand.
[5] – should the clearance between the invert of the LID feature(s) and the underlying silty clay unit be less than 1.5 m, the correction factor should be increased to 3.5.
[6] – approximate elevation of infiltration rate test based on nearby borehole as indicated.

The field-saturated hydraulic conductivity values of the silty sand fill, silt, silty fine sand, and sand ranged from approximately $1x10^{-5}$ cm/s to $1x10^{-3}$ cm/s, with corresponding infiltration rates ranging from 30 mm/hr to 75 mm/hr.

The infiltration rate estimates from this investigation are based on the test methods discussed above and are for the corresponding fill/soil types encountered. They represent the fill/soil conditions at the tested locations and depths only; conditions may vary between and beyond the tested locations. Care should be taken during construction of any proposed infiltration measures to preserve the existing soil structure and avoid compaction and re-working which could reduce its infiltrative properties.

For preliminary design purposes, a correction factor was applied to estimate the design infiltration rate in accordance with guidance provided in TRCA and CVCA (2010), to account for potential reductions in soil permeability due to compaction, smearing during the construction of a given infiltration feature and the gradual accumulation of fine sediments over the lifespan of the infiltration feature. Based on the guidance, a correction factor of 2.5 to 3.5 was applied to the estimated infiltration rates. The corrected infiltration rate estimate ranges from approximately 12 mm/hr to 30 mm/hr, with a geometric mean of 17 mm/hr (n=5). As noted above in Table 2, should the clearance between the invert of the LID feature(s) and the underlying silty clay unit be less than 1.5 m, the correction factor should be increased to 3.5 where applicable.

3.7 Summary

The Phase 3 development site is currently occupied by a three-storey residence in the northwest portion of the site, a swimming pool, an indoor horse arena with stables, several outdoor horse arenas, a one-storey residence in the southeast corner of the site, and private roadways. The majority of the site is grass-covered with paved areas adjacent to the three-storey residence, indoor horse arena and a former ice rink. The site is generally located on tableland areas between Tannery Creek and the North and West Tributaries of Tannery Creek. Portions of the site are mapped within LSRCA regulated areas, and the western portion of the site is mapped within the Oak Ridges Moraine Conservation Plan Area and Greenbelt.

- WHCs were chosen based on Table 3.1 in the MOE *SWM Manual* (2003) corresponding to the silt loam soil type, existing land uses and proposed post-development conditions.
  - Mineral Cultural Thicket / Mixed Forest / Hedgerow / Plantation (Mature Forest): 400 mm WHC and 0.65 infiltration factor (pre- and post-development conditions).
  - Mineral Cultural Meadow (Pasture and Shrubs): 250 mm WHC and 0.55 infiltration factor (pre- and post-development conditions).
  - Lawn / Vistas & Open Space / Overland Flow Area (Urban Lawn): 125 mm WHC and 0.50 or 0.55 infiltration factor, depending on topography (pre- and post-development conditions).
  - Mineral Meadow Marsh (MAM2): Surplus assumed to equal precipitation minus potential evapotranspiration, with a null (i.e., 0%) infiltration factor.
  - Neighborhood Park (Urban Lawn): 125 mm WHC and 0.60 infiltration factor (post-development).
  - Underground SWM Facility (Urban Lawn): 125 WHC, with a null (i.e., 0%) infiltration factor.
  - Impervious Areas (i.e., paved parking lots, artificial turf, roads and rooftops): Surplus assumed as 90% of precipitation and null (i.e., 0%) infiltration factor (Conservation Authorities Geoscience Group, 2013).
- Net surplus was estimated by multiplying the estimated monthly surplus (mm/month) for the assumed WHC by the associated drainage area. Annual evapotranspiration and surplus values were obtained from the meteorological data from the Toronto Buttonville A ECCC Meteorological Station based on the WHC assigned to each land use area.
- Runoff was calculated as the difference between surplus and infiltration.

5.3 Water Balance Results

Average annual water balance assessments were carried out on i) a site-wide basis; for the portions of the site contributing to ii) the entire Tannery Creek West Tributary sub-watershed, iii) the entire Tannery Creek North Tributary sub-watershed and iv) the Tannery Creek sub-watershed upstream of Yonge Street, and for portions of the site contributing to v) the palustrine section of the Southern Wetland and vi) the Northern Wetland bordered by Tannery Creek North Tributary), as described in Section 2.2. For the purposes of this report, the contributing areas of the site are referred to as catchments, which are distinct from the sub-watersheds defined in Section 2.2.1. The results for the pre-development, post-development, and mitigated post-development scenarios are presented in this section for each of the six assessments.

The names of the pre- and post-development contributing areas (catchments) referred to hereafter are summarized in the following table.

Pre-Development (see Figure 9)	Post-Development (see Figure 10)
Catchments 101 & 105 (6.75 ha)	Tannery Creek West (SAS) and Tannery Creek West, collectively the Tannery Creek West Tributary Catchment (5.36 ha)

Pre-Development (see Figure 9)	Post-Development (see Figure 10)
Catchments 102 & 104 (4.69 ha)	Tannery Creek North (SAS) and Tannery Creek North, collectively the Tannery Creek North Tributary Catchment (5.37 ha)
Catchment 103 (2.38 ha)	Tannery Creek Catchment (3.08 ha)
Southern Wetland (wetland catchment south) (1.31 ha)	Southern Wetland (wetland catchment south) (1.29 ha)
Northern Wetland (wetland catchment north) (2.40 ha)	Northern Wetland (wetland catchment north) (4.01 ha)
Site (sum of Catchments 101 to 105) (13.8 ha)	Site (sum of the Tannery Creek West and North Tributary and Tannery Creek Catchments) (13.8 ha)

Section 6.0 provides a discussion of the water balance results for the contributing areas relative to the entire sub-watershed areas for these features.

5.3.1 Pre-Development Condition

5.3.1.1 Site-Wide & Watercourse Catchments

Based on the results of the assessment, the average annual pre-development water balance was estimated on a site-wide and watercourse sub-watershed basis as summarized in Table 3, and as detailed in Tables G-3 to G-6, Appendix G.

Table 3: Pre-Development Average Annual Water Balance Results – Site Wide & Watercourse Catchments

Component	Average Annual Volume m^3/yr			
	Site-Wide (Catchments 101 to 105)	Catchments 101 & 105 (Tannery Creek West Tributary Catchment)	Catchments 102 & 104 (Tannery Creek North Tributary Catchment)	Catchment 103 (Tannery Creek Catchment)
Precipitation (P)	119,320	58,320	40,435	20,565
Evapotranspiration (ET)	71,755	36,320	24,020	11,415
Surplus (S)	47,225	21,835	16,285	9,105
Infiltration (I)	16,740	8,460	5,555	2,725
Runoff (R)	30,485	13,375	10,730	6,380

For the pre-development condition, the estimated average annual runoff from the total site is approximately 30,485 m^3 and the average annual infiltration on the site is approximately 16,740 m^3. The estimated average annual runoff from the Tannery Creek West Tributary Catchment is approximately 13,375 m^3 and the average annual infiltration within this catchment is 8,460 m^3. The estimated average annual runoff from the Tannery Creek North Tributary Catchment is approximately 10,730 m^3 and the average annual infiltration within this catchment is approximately 5,555 m^3. The estimated average annual runoff from the Tannery Creek Catchment is approximately 6,380 m^3 and the average annual infiltration within this catchment is approximately 2,725 m^3.

5.3.1.2 Wetland Catchments

Based on the results of the assessment, the average annual pre-development water balances for Southern Wetland (palustrine portion) and the Northern Wetland were estimated as summarized in Table 4, and as detailed in Tables G-7 to G-8, Appendix G.

Table 4: Pre-Development Average Annual Water Balance Results – Wetlands

Component	Average Annual Volume	
	Southern Wetland	Northern Wetland
Precipitation (P)	11,350	20,735
Evapotranspiration (ET)	6,125	12,075
Surplus (S)	5,190	8,605
Infiltration (I)	1,500	2,690
Runoff (R)	3,690	5,915

For the pre-development condition, the estimated average annual runoff contributing to the palustrine section of the Southern Wetland is approximately 3,690 m^3 and the average annual infiltration contributing to the palustrine section of the Southern Wetland is approximately 1,500 m^3. The estimated average annual runoff contributing to the Northern Wetland is approximately 5,915 m^3 and the average annual infiltration contributing to the Northern Wetland is approximately 2,690 m^3.

5.3.2 Post-Development Condition

5.3.2.1 Site Wide & Watercourse Catchments

Based on the results of the assessment, the average annual post-development water balance was estimated on a site-wide and watercourse catchment basis, as summarized in Table 5, and as detailed in Tables G-3 to G-6, Appendix G.

Table 5: Post-Development Average Annual Water Balance Results - Site Wide & Watercourse Catchments

Component	Average Annual Volume m^3/yr			
	Site-Wide	Tannery Creek West Tributary Catchment	Tannery Creek North Tributary Catchment	Tannery Creek Catchment
Precipitation (P)	119,320	46,310	46,400	26,610
Evapotranspiration (ET)	42,320	18,870	15,800	7,650
Surplus (S)	76,905	27,400	30,555	18,950
Infiltration (I)	9,240	4,430	3,145	1,665
Runoff (R)	67,665	22,970	27,410	17,285

For the post-development condition, the estimated average annual runoff from the site is approximately 67,665 m^3) on an average annual basis.

The estimated average annual runoff from the Tannery Creek West Tributary Catchment is approximately 22,970 m^3 and the estimated average annual infiltration within the catchment is approximately 4,430 m^3. As a result of land use changes from site development, runoff is expected to increase by 72% (i.e., 13,375 m^3 to 22,970 m^3) and infiltration is expected to decrease by 48% (i.e., 8,460 m^3 to 4,430 m^3) on an average annual basis.

The estimated average annual runoff from the Tannery Creek North Tributary Catchment is approximately 27,410 m^3 and the estimated average annual infiltration within the catchment is approximately 3,145 m^3. As a result of land use changes from site development, runoff is expected to increase by 155% (i.e., 10,730 m^3 to 27,410 m^3) and infiltration is expected to decrease by 43% (i.e., 5,555 m^3 to 3,145 m^3) on an average annual basis.

The estimated average annual runoff from the Tannery Creek Catchment is approximately 17,285 m^3 and the estimated average annual infiltration within the catchment is approximately 1,665 m^3. As a result of land use changes from site development, runoff is expected to increase by 171% (i.e., 6,380 m^3 to 17,285 m^3) and infiltration is expected to decrease by 39% (i.e., 2,725 m^3 to 1,665 m^3) on an average annual basis.

5.3.2.2 Wetland Catchments

Based on the results of the assessment, the average annual post-development water balance for Southern Wetland (palustrine portion) and the Northern Wetland were estimated as summarized in Table 6, and as detailed in Tables G-7 to G-8, Appendix G.

Table 6: Post-Development Average Annual Water Balance Results – Wetlands

Component	Average Annual Volume	
	Southern Wetland	Northern Wetland
Precipitation (P)	11,160	34,630
Evapotranspiration (ET)	5,405	8,880
Surplus (S)	5,740	25,735
Infiltration (I)	1,315	1,455
Runoff (R)	4,425	24,280

For the post-development condition, the estimated average annual runoff contributing to the palustrine portion of the Southern Wetland is approximately 4,425 m^3 and the estimated average annual infiltration contributing to the palustrine portion of the Southern Wetland is approximately 1,315 m^3. As a result of land use changes from site development, runoff is expected to increase by 20% (i.e., 3,690 m^3 to 4,425 m^3) and infiltration is expected to decrease by 12% (i.e., 1,500 m^3 to 1,315 m^3) on an average annual basis.

The estimated average annual runoff contributing to the Northern Wetland is approximately 24,280 m^3 and the estimated average annual infiltration contributing to the Northern Wetland is approximately 1,455 m^3. As a result of land use changes from site development, runoff is expected to increase by 310% (i.e., 5,915 m^3 to 24,280 m^3) and infiltration is expected to decrease by 46% (i.e., 2,690 m^3 to 1,455 m^3) on an average annual basis.

5.3.3 Post-Development Condition Including Mitigation

Average annual infiltration volumes at the site are expected to decrease relative to pre-development conditions and runoff volumes are expected to increase as a result of development. Groundwater recharge from the site assists to support the Tannery Creek West Tributary, Tannery Creek North Tributary and the Tannery Creek valley lands, which are classified as intermittent and permanent coldwater streams, and the associated natural heritage features, as described in Section 3. In addition, the western portion of the site is within the Oak Ridges Moraine Conservation Plan area, and the site is within the WHPA-Q1 (i.e., within the WHPA-B/C/D areas of York Region municipal wells to the east) and WHPA-Q2 areas. Therefore, it is considered prudent to incorporate low impact development (LID) measures into the development design to mitigate against reductions to post-development infiltration rates to the extent practical. Further, the use of LID measures for stormwater runoff from development sites assists to support the natural hydrologic cycle by helping to maintain groundwater recharge, provide additional water quality treatment and reduce the volume of runoff from a site.

The conceptual LID measures proposed for the site as part of the Functional Servicing design by SCS are presented on the LID Plan (SCS, 2021; see Appendix B), and are comprised of rear-yard infiltration trenches, catchbasin filtration trenches, bioswale filtration/infiltration trenches, downspout disconnection, and on-site infiltration as described below. Preliminary information provided by GEI Consultants Ltd. (GEI, 2021; see Appendix B) indicates that the school block (Block 93) will utilize enhanced grassed swales and filter strips to infiltrate runoff from impervious areas. The designed retention volumes for each of the measures in the Functional Servicing design were provided by SCS (i.e., excluding the school block LID measures). The LID measures are located throughout the site so that the enhancements to post-development infiltration rates and the attenuation of storm water volumes will benefit Tannery Creek as well as the North and West Tributaries of Tannery Creek. The following provides additional description of the LID measures.

Rear-Yard Infiltration Trenches

Rear-yard infiltration trenches along selected proposed single-detached units on Street B (Blocks 14 to 17 and Block 62) and select townhouse units along Street D (Blocks 88, 89 and 92), are proposed to capture flow from rear roof runoff for these proposed dwellings via overland flow.

The infiltration trenches should be designed with guidance from the *Low Impact Development Stormwater Management Planning and Design Guide* (TRCA & CVC, 2010). It is understood that the infiltration trenches will be designed to retain up to an approximately 22 mm storm event. The preliminary rear yard infiltration trench design is shown in the Rear Yard Infiltration Trench Detail in Appendix B (SCS, 2021).

A frequency analysis of precipitation observed at the Toronto Buttonville A station (1986 to 2017) was conducted based on the available storage of the proposed infiltration trenches. A resultant runoff reduction factor of 78% was applied to the area draining to the infiltration trenches.

Catchbasin Filtration Trenches

The Draft Plan of Subdivision (see Appendix B) includes 32 single-detached homes along Street B (Blocks 36-67), 20 along the Street E (Blocks 68-87), 12 on Street C (Blocks 21-32), and 18 on Street A (Blocks 1-13, 18-20, and 33-34). Catchbasin filtration trenches along the right of ways of Streets A, B, and C and E are proposed to capture flow from impervious paved surfaces for these proposed dwellings and adjoining street areas via overland flow.

The catchbasin filtration trenches should be designed in accordance with guidance from the *Low Impact Development Stormwater Management Planning and Design Guide* (TRCA & CVC, 2010). It is understood that the filtration trench will be designed with an impermeable liner, which will promote attenuation and settlement of sediments but will not increase infiltration. The preliminary filtration trench design is shown in the Catchbasin Filtration Trench Detail in Appendix B (SCS, 2021).

Bioswales

The following locations have been identified for the use of bioswales to collect and retain runoff (refer to the Draft Plan of Subdivision; Appendix B):

- The north half of Street B adjacent to the neighborhood park, the north half of Street E, and half of the single detached houses along the north side of Street E are proposed to incorporate a bioswale filtration trench which will be designed to capture flow from the impervious surfaces of these proposed dwellings and road via overland flow.
- Street D as well as the adjacent townhouse units are proposed to incorporate a bioswale infiltration trench which will be designed to capture flow from the front roof runoff and other impervious surfaces for these proposed dwellings and road via overland flow.

The bioswales should be designed with guidance from the *Low Impact Development Stormwater Management Planning and Design Guide* (TRCA & CVC, 2010). It is understood that the bioswales will be designed to retain up to an approximately 10 mm storm event. The preliminary bioswale infiltration trench design is shown in the

Bioswale/Infiltration Detail in Appendix B (SCS, 2021) and the preliminary bioswale sizing calculations are also provided in Appendix B (SCS, 2021).

A frequency analysis of precipitation observed at the Toronto Buttonville A station (1986 to 2017) was conducted based on the available storage of the proposed bioswale infiltration trench. A resultant runoff reduction factor of 69% was applied to the areas draining towards Street D. No runoff reduction was applied to the areas draining toward the proposed bioswale filtration trench, noting that the proposed design does not include an infiltration component.

On-site Infiltration – Saint Anne's School

The school block will utilize proposed LID measures as shown on the Site Grading Plan prepared by GEI Consultants Ltd. (GEI, 2021; see Appendix B). Enhanced grassed swales are proposed to capture flow from sections of paved road and other impervious surfaces directly in front of the existing residence within Block 93. The school block will also utilize a proposed vegetated filter strip in the southwest corner of Block 93 to infiltrate runoff from the roof of the portable school building near the west side of the property and the adjacent paved road, parking lot, and sidewalk area. In the absence of detailed design information, preliminary runoff reduction rates of 10% and 25% were adopted for this assessment for the enhanced grass swales and vegetated filter strip, respectively, based on the silt loam surficial soil type. Dry Swales are proposed along the north edge of the SAS block, but were not included in this assessment, due to the potential for less than 1 m of separation between the invert of the feature and the seasonally high groundwater table. Designs for these LID features are not available at this time but will be developed at the site plan control stage.

Downspout Disconnection

The Draft Plan of Subdivision (see Appendix B) includes 87 single-detached homes throughout the site. As an LID measure, it is proposed that all single-detached houses that are not serviced by rear-yard infiltration trenches or bioswales would incorporate downspout disconnection to direct flow from front and full roof downspouts to a pervious lawn area that drains away from the building. Downspouts should discharge to a gradual sloped pervious area 3 m away from the foundation that conveys runoff away from the building along a minimum flow path length of 5 m. A runoff reduction rate of 25% was adopted for this assessment, based on the silt loam surficial soil type (TRCA & CVC, 2010).

Groundwater Elevations

The types and locations of the conceptual LIDs consider the available groundwater elevation data and were selected for areas where it was deemed that the use of subsurface LIDs was feasible. The mitigated post-development scenario presented below assumes that a 1 m separation between the subsurface LID inverts and the inferred seasonally high groundwater elevations will be maintained. It is recommended that additional groundwater level monitoring continue through the detailed design stage.

Details such as final grades and the inverts of LID measures will be available as designs progress. The depth to groundwater could present challenges to the implementation of subsurface infiltration features used as LID measures in some areas of the site. In the event that a 1 m separation distance cannot be maintained, a subsurface LID would still enhance post-development infiltration rates, especially at times of seasonally low groundwater conditions, provided that the outlet or overflow of the LID remains above the seasonally high groundwater level. If the separation distance is less than 1 m, less average annual infiltration and more average annual runoff would be achieved than presented below. In any event, the infiltration features also function to capture and attenuate precipitation events at the site and provide a benefit to the storm water management scheme. The findings presented below should be re-assessed at the time of detailed design and on the basis of any additional groundwater elevation data and final grades.

5.3.3.1 Results – Site-Wide & Watercourse Catchments

Based on the above, the average annual mitigated post-development water balance for was estimated on a site-wide and watercourse catchment basis, as summarized in Table 7, and as detailed in Tables G-3 to G-6, Appendix G.

Table 7: Mitigated Post-Development Average Annual Water Balance Results – Site-Wide & Watercourse Catchments

Component	Average Annual Volume m^3/yr			
	Site-Wide	Tannery Creek West Tributary Catchment	Tannery Creek North Tributary Catchment	Tannery Creek Catchment
Precipitation (P)	119,320	46,310	46,400	26,610
Evapotranspiration (ET)	42,320	18,870	15,800	7,650
Surplus (S)	76,905	27,400	30,555	18,950
Infiltration (I)	16,205	6,800	5,770	3,635
Runoff (R)	60,700	20,600	24,785	15,315

The proposed LID mitigation scheme is estimated to increase average annual infiltration by approximately 6,965 m^3 and reduce average annual runoff, similarly, compared to the un-mitigated post-development condition. On a site-wide basis, average annual infiltration is estimated to decrease by 3% (i.e., 16,740 m^3 to 16,205 m^3) and average annual runoff is expected to increase by 99% (i.e., 30,485 m^3 to 60,700 m^3) as a result of development compared to pre-development conditions.

Considering the proposed LID mitigation, the estimated average annual runoff contributing to the Tannery Creek West Tributary Catchment is approximately 20,600 m^3 and the estimated average annual infiltration within the catchment is approximately 6,800 m^3. As a result of catchment boundary and land use changes from site development, runoff is expected to increase by 54% (i.e., 13,375 m^3 to 20,600 m^3) and infiltration is expected to decrease by 20% (i.e., 8,460 m^3 to 6,800 m^3) on an average annual basis.

Considering the proposed LID mitigation, the estimated average annual runoff contributing to the Tannery Creek North Tributary Catchment is approximately 24,785 m^3 and the estimated average annual infiltration within the catchment is approximately 5,770 m^3. As a result of catchment area and land use changes from site development, runoff is expected to increase by 131% (i.e., 10,730 m^3 to 24,785 m^3) and infiltration is expected to increase by 4% (i.e., 5,555 m^3 to 5,770 m^3) on an average annual basis.

Considering the proposed LID mitigation, the estimated average annual runoff contributing to the Tannery Creek Catchment is approximately 15,315 m^3 and the estimated average annual infiltration within the catchment is approximately 3,635 m^3. As a result of catchment boundary and land use changes from site development, runoff is expected to increase by 140% (i.e., 6,380 m^3 to 15,315 m^3) and infiltration is expected to increase by 33% (i.e., 2,725 m^3 to 3,635 m^3) on an average annual basis.

5.3.3.2 Results – Wetland Catchments

Based on the results of the assessment, the average annual mitigated post-development water balance for the Southern Wetland (palustrine portion) and the Northern Wetland were estimated as summarized in Table 8, and as detailed in Tables G-7 to G-8, Appendix G.

Table 8: Mitigated Post-Development Average Annual Water Balance Results – Wetlands

Component	Average Annual Volume	
	Southern Wetland	Northern Wetland
Precipitation (P)	11,160	34,630
Evapotranspiration (ET)	5,405	8,880
Surplus (S)	5,740	25,735
Infiltration (I)	1,720	4,080
Runoff (R)	4,020	21,655

Considering the proposed LID mitigation scheme and the relatively small changes in the catchment area, average annual infiltration contributing to the palustrine section of the Southern Wetland is estimated to increase by 15% (i.e., 1,500 m^3 to 1,720 m^3) and average annual runoff is expected to increase by 9% (i.e., 3,690 m^3 to 4,020 m^3) as a result of development compared to pre-development conditions.

Considering the proposed LID mitigation scheme and increase in the catchment area, average annual infiltration contributing to the Northern Wetland is estimated to increase by 52% (i.e., 2,690 m^3 to 4,080 m^3) and average annual runoff is expected to increase by 266% (i.e., 5,915 m^3 to 21,655 m^3) as a result of development compared to pre-development conditions.

6.0 DISCUSSION

The site is generally located on tableland areas between the Tannery Creek West Tributary, Tannery Creek North Tributary and Tannery Creek. Portions of the site are mapped within LSRCA regulated areas, and the western portion of the site is mapped within the Oak Ridges Moraine Conservation Plan Area and Greenbelt.

The findings of the subsurface investigation indicate that shallow native soils at the site are predominantly comprised of a non-cohesive silt to sand deposit with an average thickness of about 3.2 m and with moderate hydraulic conductivity. A thick underlying deposit of silty clay has moderate to low hydraulic conductivity. Based on MECP water well records, the thickness of the clay/till aquitard at the site is on the order of 50 m or more. Shallow groundwater flow is inferred to follow topography, with flow in an eastern direction towards Tannery Creek, in a northeast direction towards the Tannery Creek North Tributary, and in a south to southwest direction towards the Tannery Creek West Tributary, depending on location.

Off-site to the southwest in proximity to the Tannery Creek West Tributary, Beacon has identified several small wetlands that are riverine in nature. Further downstream, the Southern Wetland includes a riverine portion in proximity to the Tannery Creek West Tributary, and a palustrine portion at its north end (i.e., north of the Tannery Creek West Tributary) on the sloped portion of the valley lands.

Table E-1 - Water Level Depths and Elevations
Hydrogeological Investigation
Shining Hill, Aurora, Ontario

Monitoring Well ID	Ground Surface Elevation (masl)	29-Sep-20		16-Nov-20		24-Nov-20		01-Dec-20	
		Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)
BH101	265.00	4.50	260.50	4.13	260.87	4.19	260.81	3.89	261.11
BH102	264.90	2.80	262.10	2.66	262.24	2.67	262.24	2.48	262.42
BH103	268.00	2.50	265.50	2.40	265.61	2.40	265.61	2.26	265.75
BH104	267.30	2.70	264.60	2.24	265.06	2.24	265.06	2.18	265.12
BH105	266.80	7.20	259.60	6.78	260.02	6.72	260.08	6.60	260.20
BH106	265.30	DRY	DRY	6.92	258.38	5.92	259.38	3.69	261.61
BH107	262.50	4.20	258.30	3.56	258.94	3.61	258.89	2.82	259.68
BH108	269.30	3.20	266.10	3.08	266.22	3.10	266.20	2.97	266.34
BH202	271.30	4.60	266.70	4.69	266.61	4.70	266.61	4.64	266.67
BH205	274.10	3.80	270.30	3.97	270.13	4.00	270.10	3.78	270.33
BH206-D	273.30	2.00	271.30	1.83	271.48	1.84	271.47	1.73	271.57
BH206-S	273.30	3.90	269.40	3.92	269.38	3.92	269.38	3.89	269.42
P1	259.35			DRY	DRY	1.11	258.24	0.68	258.67
SG1	258.76			-0.27	259.03	-0.27	259.03	-0.29	259.05
P2	261.20								
SG2	261.22								
P3	250.37								
SG3	250.37								
P4	248.83								
SG4	248.89								

Notes:

1) mbgs = metres below ground surface
2) masl = metres above sea level
3) Monitoring wells 101 to 108, 202, 205 and 206D/S were installed by Soil Engineers Ltd. in September 2020. The elevations provided are understood to be referenced to a geodetic datum.
4) D = deep, S = shallow
5) P = piezometer, SG = staff gauge; P1/SG1 installed by Golder Associates Ltd. on November 16, 2020. P2/SG2 to P4/SG4 installed by Golder Associates Ltd. on June 2, 2021.
6) Elevation data for ground surface at the location of the P1/SG1 to P4/SG4 were surveyed by Golder Associates Ltd. and are referenced to a geodetic datum.
7) Groundwater level data from September 29, 2020, were measured by Soil Engineers Ltd.
8) Stabilized groundwater conditions may not have been present at BH106 on Sept. 29, Nov. 16, Nov. 24, and Dec. 1, 2020.

Table E-1 - Water Level Depths and Elevations
Hydrogeological Investigation
Shining Hill, Aurora, Ontario

Monitoring Well ID	Ground Surface Elevation (masl)	19-Jan-21		08-Apr-21		02-Jun-21		09-Jun-21	
		Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)
BH101	265.00	3.20	261.80	-	-	4.19	260.81	-	-
BH102	264.90	1.97	262.93	1.82	263.09	2.63	262.27	2.72	262.18
BH103	268.00	1.82	266.18	1.57	266.43	2.02	265.99	-	-
BH104	267.30	1.33	265.97	-	-	1.81	265.50	-	-
BH105	266.80	6.06	260.74	-	-	5.76	261.04	-	-
BH106	265.30	2.24	263.07	-	-	2.77	262.53	-	-
BH107	262.50	2.24	260.26	-	-	2.88	259.62	-	-
BH108	269.30	2.25	267.05	-	-	2.70	266.60	-	-
BH202	271.30	4.35	266.95	-	-	4.21	267.10	-	-
BH205	274.10	3.38	270.72	-	-	2.84	271.26	-	-
BH206-D	273.30	1.62	271.68	-	-	1.63	271.67	-	-
BH206-S	273.30	3.57	269.73	-	-	3.34	269.97	-	-
P1	259.35	0.22	259.13	0.01	259.34	0.09	259.27	0.14	259.21
SG1	258.76	-0.26	259.02	-0.25	259.01	-0.23	258.99	-0.24	259.00
P2	261.20					DRY	DRY	DRY	DRY
SG2	261.22					DRY	DRY	DRY	DRY
P3	250.37					1.23	249.15	0.50	249.87
SG3	250.37					DRY	DRY	DRY	DRY
P4	248.83					0.91	247.93	0.30	248.53
SG4	248.89					DRY	DRY	DRY	DRY

Notes:

1) mbgs = metres below ground surface

2) masl = metres above sea level

3) Monitoring wells 101 to 108, 202, 205 and 206D/S were installed by Soil Engineers Ltd. in September 2020. The elevations provided are understood to be referenced to a geodetic datum.

4) D = deep, S = shallow

5) P = piezometer, SG = staff gauge; P1/SG1 installed by Golder Associates Ltd. on November 16, 2020. P2/SG2 to P4/SG4 installed by Golder Associates Ltd. on June 2, 2021.

6) Elevation data for ground surface at the location of the P1/SG1 to P4/SG4 were surveyed by Golder Associates Ltd. and are referenced to a geodetic datum.

7) Groundwater level data from September 29, 2020, were measured by Soil Engineers Ltd.

8) Stabilized groundwater conditions may not have been present at BH106 on Sept. 29, Nov. 16, Nov. 24, and Dec. 1, 2020.

Input by: AGB/JJG
Checked by: CMK

Table E-1 - Water Level Depths and Elevations
Hydrogeological Investigation
Shining Hill, Aurora, Ontario

Monitoring Well ID	Ground Surface Elevation (masl)	03-Sep-21		09-Sep-21		29-Sep-21		12-Nov-21	
		Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)	Depth (mbgs)	Elevation (masl)
BH101	265.00	-	-	-	-	3.32	261.68	-	-
BH102	264.90	2.98	261.92	2.92	261.98	2.07	262.83	1.98	262.92
BH103	268.00	2.67	265.34	2.60	265.41	2.14	265.86	-	-
BH104	267.30	-	-	-	-	1.85	265.45	-	-
BH105	266.80	-	-	-	-	6.88	259.92	-	-
BH106	265.30	-	-	-	-	2.67	262.63	-	-
BH107	262.50	3.80	258.70	3.37	259.14	2.34	260.17	1.80	260.70
BH108	269.30	-	-	-	-	2.24	267.07	-	-
BH202	271.30	4.76	266.54	4.77	266.53	4.57	266.73	-	-
BH205	274.10	3.79	270.31	3.84	270.26	3.46	270.64	-	-
BH206-D	273.30	2.11	271.20	2.06	271.25	1.76	271.54	1.67	271.63
BH206-S	273.30	3.86	269.45	3.86	269.44	3.67	269.63	3.55	269.76
P1	259.35	0.83	258.52	0.85	258.51	0.55	258.80	0.10	259.25
SG1	258.76	-0.25	259.01	-0.25	259.01	-0.25	259.01	-0.26	259.02
P2	261.20	N/A	N/A	N/A	N/A	0.50	260.70	0.06	261.14
SG2	261.22	N/A	N/A	N/A	N/A	DRY	DRY	DRY	DRY
P3	250.37	1.05	249.32	0.74	249.64	0.14	250.23	-	-
SG3	250.37	DRY	DRY	DRY	DRY	DRY	DRY	-	-
P4	248.83	0.74	248.09	0.70	248.14	0.05	248.79	-	-
SG4	248.89	DRY	DRY	DRY	DRY	DRY	DRY	-	-

Notes:

1) mbgs = metres below ground surface

2) masl = metres above sea level

3) Monitoring wells 101 to 108, 202, 205 and 206D/S were installed by Soil Engineers Ltd. in September 2020. The elevations provided are understood to be referenced to a geodetic datum.

4) D = deep, S = shallow

5) P = piezometer, SG = staff gauge; P1/SG1 installed by Golder Associates Ltd. on November 16, 2020. P2/SG2 to P4/SG4 installed by Golder Associates Ltd. on June 2, 2021.

6) Elevation data for ground surface at the location of the P1/SG1 to P4/SG4 were surveyed by Golder Associates Ltd. and are referenced to a geodetic datum.

7) Groundwater level data from September 29, 2020, were measured by Soil Engineers Ltd.

8) Stabilized groundwater conditions may not have been present at BH106 on Sept. 29, Nov. 16, Nov. 24, and Dec. 1, 2020.

APPENDIX C

HYDROLOGY MODELLING AND PARAMETERS

The following secure link is being provided by **SCS Consulting Group** to share Shining Hill Estates PH3 (Aurora) FSSR related digital data:

https://filesafecloud.scsconsultinggroup.com/url/jukuts23hdxutgce

Please click on the link and download all files from this location.

- VO Modelling

Shining Hill Estates PH3 (Aurora)
Existing Hydrology Schematic
November 2021

Existing Conditions
VO2 Parameter Summary

NASHYD

Number	101	102	103
Description			
DT(min)	2	2	2
Area (ha)	4.07	3.63	2.38
CN*	75.0	80.0	80.0
IA(mm)	6.4	5.7	4.8
TP Method	Uplands	Uplands	Uplands
TP (hr)	0.43	0.16	0.12

Total Area = 10.1 ha

Existing Conditions
CN Calculations

Shining Hill Estates PH3 (Aurora)
Project Number: 2183
Date: November 2021
Designer Initials: MECM

Site Soils: (per OMAFRA County Soils Mapping)

Soil Type	**Hydrologic Soil Group**
Schomberg Clay Loam	C

TABLE OF CURVE NUMBERS (CN's)**									
Land Use	Hydrologic Soil Type							Manning's	Source
	A	AB	B	BC	C	CD	D	'n'	
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								
	Hydrologic Soil Type							
Catchment	A	AB	B	BC	C	CD	D	TOTAL
101	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
102	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
103	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100

HYDROLOGIC SOIL TYPE (%) - Existing Conditions								
	Hydrologic Soil Type							
Catchment	A	AB	B	BC	C	CD	D	TOTAL
101					100			100
102					100			100
103					100			100

LAND USE (%) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101	3.1	29.1	0.0	61.6	0.0	0.0	0.0	0.0	6.2	100.0
102	0.0	26.4	0.3	52.5	0.0	0.0	0.0	0.0	20.8	100.0
103	0.0	10.1	0.0	67.1	0.0	0.0	0.0	0.0	22.8	100.0

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

LAND USE (%) - Existing Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
101	3.1	29.1		61.6					6.2	100.0
102		26.4	0.3	52.5					20.8	100.0
103		10.1		67.1					22.8	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	2.2	21.2	0.0	45.6	0.0	0.0	0.0	0.0	6.1	75
102	0.0	19.3	0.3	38.8	0.0	0.0	0.0	0.0	20.4	79
103	0.0	7.4	0.0	49.7	0.0	0.0	0.0	0.0	22.3	79

** AMC II assumed

Existing Conditions
CN Calculations

Step	Input Values				
	Subcatchment:	101		102	103
1	**CN (AMC II):**	**75**		**79**	**79**
2	CN (AMC III) =	88		91	91
3	100 Year Precipitation, P =	95.98	mm	95.98	95.98

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \qquad S = \frac{(P - Ia)^2}{Q} - (P - Ia)$$

Q = rainfall excess or runoff, mm
S = potential maximum retention or available storage, mm

$$CN = \frac{25400}{S + 254} \qquad S = \frac{25400}{CN} - 254$$

CN* = modified SCS curve # that better reflects Ia conditions in Ontario

Step	Output Values				
	Subcatchment:	101		102	103
	S_{III} =	34.64	mm	25.12	25.12
	SCS Assumption of 0.2 S = Ia =	6.93	mm	5.02	5.02
4	Q_{III} =	64.12	mm	71.27	71.27
	Preferred Initial Abstraction, Ia =	6.4	mm	5.7	4.8
5	S^*_{III} =	35.65	mm	24.10	25.44
6	CN^*_{III} =	87.69	mm	91.33	90.90
	CN^*_{III}=	**88**	Rounded	**91**	**91**
7	**CN^*_{II}=**	**75**	convert	**80**	**80**

Explanation of Procedure

1 Determine CN based on typical AMC II conditions (attached)
2 Convert CN from AMC II to AMC III conditions (standard SCS tables)
3 Get precipitation depth P for 100 year storm
4 Using CN_{III} with Ia = 0.2S, compute Q_{III} for 100 year precipitation
5 For the same Q_{III}, compute S^*_{III} using Ia=1.5mm (or otherwise determined)
6 Compute CN^*_{III} using S^*_{III}
7 Calculate CN^*_{II} using SCS conversion table

Existing Conditions
IA Calculations

LAND USE (%) - Existing Conditions										
Catchment	Meadow 4.07	Woodlot 3.63	Gravel 2.38	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density C	Impervious	Total
101	3.1	29.1		61.6					6.2	100.0
102		26.4	0.3	52.5					20.8	100.0
103		10.1		67.1					22.8	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	8	3	2	2	
101	0.2	2.9		3.1					0.1	6.4
102		2.6	0.0	2.6					0.4	5.7
103		1.0		3.4					0.5	4.8

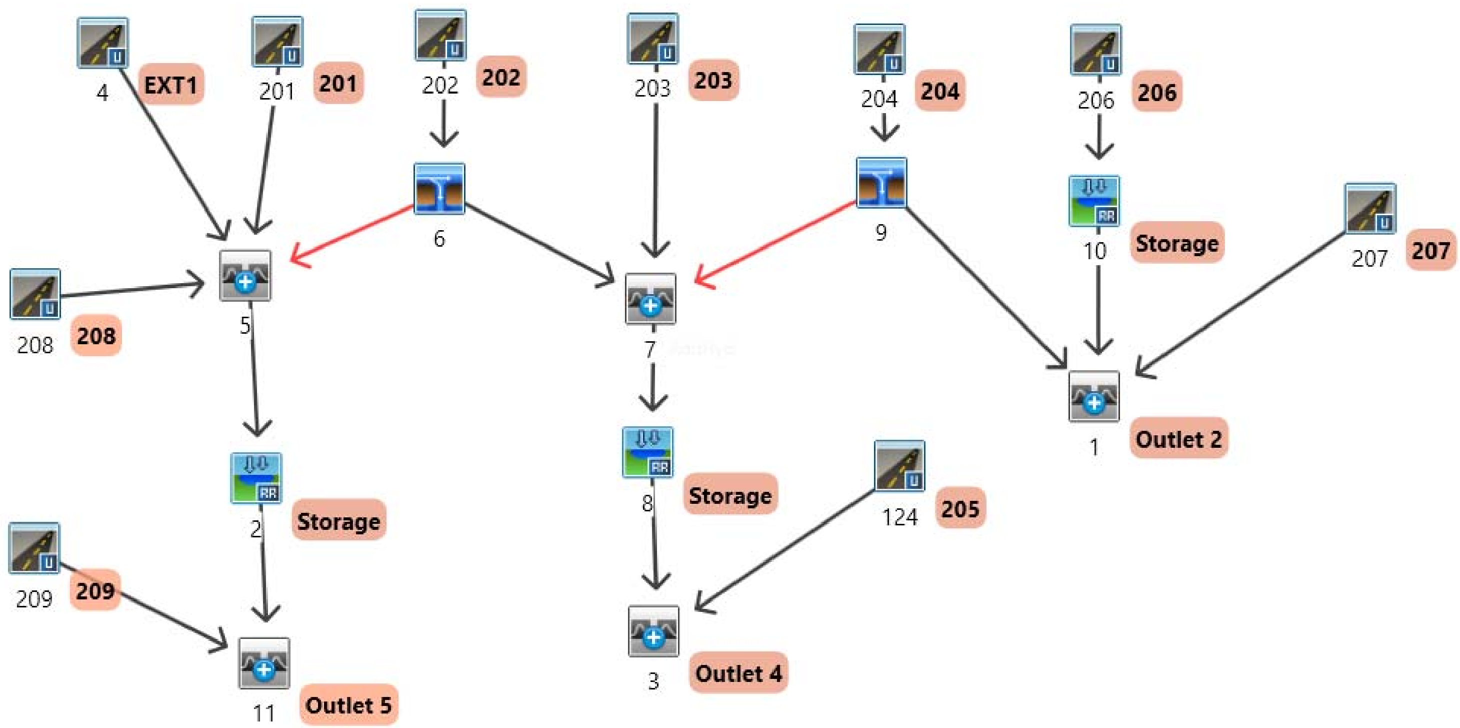
* IA values based on LRSCA guidelines

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	268.05	267.39	41	1.62	Woodland	0.19	211.8	0.06	0.04
101b	267.39	265.41	126	1.57	Pasture	0.27	461.0	0.13	0.09
101c	265.41	265.39	37	0.05	Woodland	0.04	1032.8	0.29	0.19
101d	265.39	265.38	25	0.04	Pasture	0.04	588.0	0.16	0.11
101e	265.38	263.67	19	9.10	Woodland	0.45	41.3	0.01	0.01
101									**0.43**
102a	273.25	271.07	45	4.81	Pasture	0.48	94.4	0.03	0.02
102b	271.07	270.02	22	4.88	Small Upland Gullies and Paved Areas	1.34	16.1	0.00	0.00
102c	270.02	269.68	22	1.55	Woodland	0.19	117.1	0.03	0.02
102d	269.68	266.03	156	2.33	Pasture	0.33	468.7	0.13	0.09
102e	266.03	263.27	33	8.29	Woodland	0.43	76.7	0.02	0.01
102f	263.27	262.13	12	9.64	Small Upland Gullies and Paved Areas	1.88	6.3	0.00	0.00
102g	262.13	261.46	26	2.56	Pasture	0.35	74.9	0.02	0.01
102									**0.16**
103a	269.57	269.32	16	1.58	Woodland	0.19	83.4	0.02	0.02
103b	269.32	264.27	201	2.51	Pasture	0.35	581.2	0.16	0.11
103									**0.12**

Shining Hill Estates PH3 (Aurora)
Proposed Hydrology Schematic
November 2021



Proposed Conditions
VO2 Parameter Summary

STANDHYD

Number	201	202	203	204	205	206	207	208	209	EXT1
Description										
DT(min)	2	2	2	2	2	2	2	2	2	2
Area (ha)	1.83	0.47	2.38	0.42	0.23	2.33	0.56	1.15	0.16	0.55
XIMP[1,2]	0.84	0.33	0.32	0.37	0.07	0.26	0.07	0.54	0.13	0.65
TIMP[2]	0.84	0.66	0.66	0.67	0.59	0.64	0.59	0.72	0.64	0.65
CN*	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0	73.0
IA(mm)	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
LGP(m)	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
DPSI (mm)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
SLPI(%)	1	1	1	1	1	1	1	1	1	1
LGI(m)	110.45	55.98	125.96	52.92	39.16	124.63	61.10	87.56	32.66	60.55
MNI	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013

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

Total Area = 10.1 ha

Shining Hill Estates
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Designer Initials: M.E.C.M.

Proposed Conditions
CN Calculations

Site Soils: (per OMAFRA County Soils Mapping)

Soil Type	**Hydrologic Soil Group**
Schomberg Clay Loam	C

TABLE OF CURVE NUMBERS (CN's)**

Land Use	Hydrologic Soil Type							Manning's	Source
	A	AB	B	BC	C	CD	D	'n'	
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	
201	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
202	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
203	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
204	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
205	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
206	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
207	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
208	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
209	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100
EXT1	0.0	0.0	0.0	0.0	100.0	0.0	0.0	100

HYDROLOGIC SOIL TYPE (%) - Proposed Conditions

Catchment	Hydrologic Soil Type							TOTAL
	A	AB	B	BC	C	CD	D	
201					100			100
202					100			100
203					100			100
204					100			100
205					100			100
206					100			100
207					100			100
208					100			100
209					100			100
EXT1					100			100

LAND USE (%) - Existing Conditions

Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
201	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
202	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
203	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
204	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
205	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
206	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
207	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
208	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
209	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0
EXT1	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	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

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
201				100.0						100.0
202				100.0						100.0
203				100.0						100.0
204				100.0						100.0
205				100.0						100.0
206				100.0						100.0
207				100.0						100.0
208				100.0						100.0
209				100.0						100.0
EXT1				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) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Weighted CN
201	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
202	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
203	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
204	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
205	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
206	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
207	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
208	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
209	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74
EXT1	0.0	0.0	0.0	74.0	0.0	0.0	0.0	0.0	0.0	74

** AMC II assumed

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Proposed Conditions
CN Calculations

Step	Input Values												
	Subcatchment:			201	202	203	204	205	206	207	208	209	EXT1
1	**CN (AMC II):**			**74**	**74**	**74**	**74**	**74**	**74**	**74**	**74**	**74**	**74**
2	CN (AMC III) =			88	88	88	88	88	88	88	88	88	88
3	100 Year Precipitation, P =	95.98	mm	95.98	95.98	95.98	95.98	95.98	95.98	95.98	95.98	95.98	95.98

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \qquad S = \frac{(P - Ia)^2}{Q} - (P - Ia)$$

Q = rainfall excess or runoff, mm
S = potential maximum retention or available storage, mm

$$CN = \frac{25400}{S + 254} \qquad S = \frac{25400}{CN} - 254$$

CN* = modified SCS curve # that better reflects Ia conditions in Ontario

Step	Output Values												
	Subcatchment:			201	202	203	204	205	206	207	208	209	EXT1
	S_{III} =		mm	34.64	34.64	34.64	34.64	34.64	34.64	34.64	34.64	34.64	34.64
	SCS Assumption of 0.2 S = Ia =		mm	6.93	6.93	6.93	6.93	6.93	6.93	6.93	6.93	6.93	6.93
4	Q_{III} =		mm	64.12	64.12	64.12	64.12	64.12	64.12	64.12	64.12	64.12	64.12
	Preferred Initial Abstraction, Ia =		mm	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
5	S^*_{III} =		mm	38.12	38.12	38.12	38.12	38.12	38.12	38.12	38.12	38.12	38.12
6	CN^*_{III} =		mm	86.95	86.95	86.95	86.95	86.95	86.95	86.95	86.95	86.95	86.95
	CN^*_{III}=		Rounded	**87**	**87**	**87**	**87**	**87**	**87**	**87**	**87**	**87**	**87**
7	**CN^*_{II}=**		convert	**73**	**73**	**73**	**73**	**73**	**73**	**73**	**73**	**73**	**73**

Explanation of Procedure

1 Determine CN based on typical AMC II conditions (attached)
2 Convert CN from AMC II to AMC III conditions (standard SCS tables)
3 Get precipitation depth P for 100 year storm
4 Using CN_{III} with Ia = 0.2S, compute Q_{III} for 100 year precipitation
5 For the same Q_{III}, compute S^*_{III} using Ia=1.5mm (or otherwise determined)
6 Compute CN^*_{III} using S^*_{III}
7 Calculate CN^*_{II} using SCS conversion table

Proposed Conditions
IA Calculations

LAND USE (%) - Proposed Conditions										
Catchment	Meadow	Woodlot	Gravel	Lawns	Pasture Range	Crop	Fallow (Bare)	Low Density Residences	Impervious	Total
201				100.0						100.0
202				100.0						100.0
203				100.0						100.0
204				100.0						100.0
205				100.0						100.0
206				100.0						100.0
207				100.0						100.0
208				100.0						100.0
209				100.0						100.0
EXT1				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	
201				5.0						5.0
202				5.0						5.0
203				5.0						5.0
204				5.0						5.0
205				5.0						5.0
206				5.0						5.0
207				5.0						5.0
208				5.0						5.0
209				5.0						5.0
EXT1				5.0						5.0

* IA values based on LRSCA guidelines

SCS consulting group ltd

Shining Hill Estates
Project Number: 2183
Date: November 2021
Designer Initials: M.E.C.M.

Proposed Conditions
Percent Impervious Calculations

			StandHyd IDs									
			201	202	203	204	205	206	207	208	209	EXT1
Catchment Area (ha)			1.83	0.47	2.38	0.42	0.23	2.33	0.56	1.15	0.16	0.55
Land Use Areas	Timp	Ximp	Land Use Areas									
Neighbourhood Park	85%	85%	1.60							0.20		
Townhouses	64%	13%								0.39	0.16	
Single House - Rear Lot	45%	0%										
Single Houses	59%	7%		0.30	1.56	0.24	0.23	1.72	0.56	0.02		
15m ROW	69%	69%								0.28		
16.5m ROW	77%	77%			0.09	0.18						
23.0m ROW	80%	80%	0.23	0.17	0.56					0.07		
18.0m ROW	78%	78%			0.17			0.61		0.19		
External Area	65%	65%										0.55
Laneway - Uncontrolled	100%	0%										
Laneway	100%	100%										
		Total Land Use =	1.83	0.47	2.38	0.42	0.23	2.33	0.56	1.15	0.16	0.55
		Timp =	84%	66%	66%	67%	59%	64%	59%	72%	64%	65%
		Ximp =	84%	33%	32%	37%	7%	26%	7%	54%	13%	65%

Percent Impervious (Timp) Calculations per Typical Layout for Single Detached Dwelling

Land Use	Lot Type				
Single Houses	**12.2x30m**	**13.7x30m**	**15.24x30m**	**Max. Timp**	**Max. Ximp**
	59%	59%	59%	59%	7%
Townhouses	**6.1x30m**	**6.1x35m**			
	58%	64%		64%	13%

***Ximp calculations based on total impervious areas directly connected**

APPENDIX D

PHOSPHORUS BUDGET

Existing Phosphorus Budget

Watershed East Holland River

Land Cover	TP Loading (kg/ha/yr)	Area (ha)	TP Loading (kg/yr)
Low Intensity Development	0.13	10.90	1.417
Forest	0.10	2.78	0.278
Wetland	0.10	0.13	0.013
TOTAL		13.81	1.708

Subwatershed	Phosphorus Export (kg/ha/yr)											
	Cropland	Hay-Pasture	Sod Farm/Golf Course	High Intensity Development: Commercial /Industrial	High Intensity Development: Residential	Low Intensity Development	Quarry	Unpaved Road	Forest	Transition	Wetland	Open Water
Monitored Subwatersheds												
Beaver River	0.22	0.04	0.01	1.82	1.32	0.19	0.06	0.83	0.02	0.04	0.02	0.26
Black River	0.23	0.08	0.02	1.82	1.32	0.17	0.15	0.83	0.05	0.06	0.04	0.26
East Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hawkestone Creek	0.19	0.10	0.06	1.82	1.32	0.09	0.10	0.83	0.03	0.04	0.03	0.26
Lovers Creek	0.16	0.07	0.17	1.82	1.32	0.07	0.06	0.83	0.06	0.06	0.05	0.26
Pefferlaw/Uxbridge Brook	0.11	0.06	0.02	1.82	1.32	0.13	0.04	0.83	0.03	0.04	0.04	0.26
Whites Creek	0.23	0.10	0.42	1.82	1.32	0.15	0.08	0.83	0.10	0.11	0.09	0.26
Unmonitored Subwatersheds												
Barrie Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
GeorginaCreeks	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hewitts Creek	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Innisfil Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Maskinonge River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Oro Creeks North	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Oro Creeks South	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Ramara Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Talbot/Upper Talbot River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
West Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26

Proposed Conditions Phosphorus Budget

Watershed: East Holland River

Description	Land Cover	TP Loading (kg/ha/yr)	Area (ha)	TP Loading (kg/yr)	BMP #1: BMP	BMP #1: TP Removal Rate (%)	BMP #1: TP Export (kg/yr)	BMP #2: BMP	BMP #2: TP Removal Rate (%)	BMP #2: TP Export (kg/yr)	Combined Removal Efficiency	Unmitigated P_{load} (kg/year)	Mitigated P_{load} (kg/year)
School Block	High Intensity Dev. - Commercial/Industrial	1.82	3.73	6.792	On-Site Removal	100%	0.000	None	0%	0.000	100%	6.79	0.000
School Block	High Intensity Dev. - Commercial/Industrial	1.82	0.55	0.996	Underground Storage	25%	0.747	On-Site Removal	100%	0.000	100%	1.00	0.000
Catchbasin Filtration and Underground Storage	High Intensity Dev. - Residential	1.32	6.89	9.095	Sand or Media Filters	45%	5.002	Underground Storage	25%	3.752	59%	9.09	3.752
Bioswale Infiltration and Underground Storage	High Intensity Dev. - Residential	1.32	0.45	0.595	Infiltration Trenches	60%	0.238	Underground Storage	25%	0.178	70%	0.59	0.178
Bioswale Filtration and Underground Storage	High Intensity Dev. - Residential	1.32	0.34	0.449	Sand or Media Filters	45%	0.247	Underground Storage	25%	0.185	59%	0.45	0.185
Rear Yard Infiltration and Underground Storage	High Intensity Dev. - Residential	1.32	0.23	0.304	Infiltration Trenches	60%	0.121	Underground Storage	25%	0.091	70%	0.30	0.091
Rear Yard Infiltration and Stream Buffer	High Intensity Dev. - Residential	1.32	0.05	0.066	Infiltration Trenches	60%	0.026	Vegetated Filter Strips / Stream Buffers	65%	0.009	86%	0.07	0.009
SWM Block	High Intensity Dev. - Residential	1.32	0.20	0.266	Underground Storage	25%	0.200	None	0%	0.200	25%	0.27	0.200
Rear Yards	High Intensity Dev. - Residential	1.32	1.37	1.808	Vegetated Filter Strips / Stream Buffers	65%	0.633	None	0%	0.633	65%	1.81	0.633
		Total	13.81								**Total**	20.37	5.048
												Removal Rate	**75%**

Subwatershed	Phosphorus Export (kg/ha/yr): Cropland	Hay-Pasture	Sod Farm/Golf Course	High Intensity Development: Commercial /Industrial	High Intensity Development: Residential	Low Intensity Development	Quarry	Unpaved Road	Forest	Transition	Wetland	Open Water
Monitored Subwatersheds												
Beaver River	0.22	0.04	0.01	1.82	1.32	0.19	0.06	0.83	0.02	0.04	0.02	0.26
Black River	0.23	0.08	0.02	1.82	1.32	0.17	0.15	0.83	0.05	0.06	0.04	0.26
East Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hawkestone Creek	0.19	0.10	0.06	1.82	1.32	0.09	0.10	0.83	0.03	0.04	0.03	0.26
Lovers Creek	0.16	0.07	0.17	1.82	1.32	0.07	0.06	0.83	0.06	0.06	0.05	0.26
Pefferlaw/Uxbridge Brook	0.11	0.06	0.02	1.82	1.32	0.13	0.04	0.83	0.03	0.04	0.04	0.26
Whites Creek	0.23	0.10	0.42	1.82	1.32	0.15	0.08	0.83	0.10	0.11	0.09	0.26
Unmonitored Subwatersheds												
Barrie Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
GeorginaCreeks	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hewitts Creek	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Innisfil Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Maskinonge River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Oro Creeks North	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Oro Creeks South	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Ramara Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Talbot/Upper Talbot River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
West Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26

Lake Simcoe Phosphorous Offsetting Policy Calculation

Phosphorus Export =	5.05	kg/yr
Offset Ratio =	2.5	:1
Offsetting Value =	$ 35,770.00	/kg/year
Offsetting Cost =	$ 451,404.44	
Administration Fee =	15%	
	$ 67,710.67	
TOTAL PHOSPHORUS OFFSETTING FEE =	**$ 519,115.11**	

Sorbtive media

What is it?

Granular Sorptive media

Sorbtive media is an oxide-based, high surface area reactive engineered media that absorbs and retains large amounts of dissolved phosphorus. It does not desorb (leach) pollutants and has a low total phosphorus effluent concentration (< 0.1 mg/L). Sorbtive Media controls phosphorus by two mechanisms:

1. Physical filtration is the removal of particulate-bound phosphorus and sediment, and
2. Sorption is the physio-chemical removal of dissolved phosphorus (the biologically available portion).[1].

How is it being used?

The "Sorbtive Media" website highlights multiple projects where their product was used for phosphorus removal. At the Sturgeon Meadows Stormwater Management Facility in Leamington, Ontario, Sorbtive Media was applied as a retrofit component to enhance pollutant removal withing an existing dry pond as part of a treatment train. A 30 cm layer was applied within retrofitted trenches in combination with washed stone and rip rap rock to manage the expected treatment flow.

The Rumble Pond Retrofit project in Richmond Hill, Ontario used Sorbtive Media in combination with permeable interlocking pavers to enhance overall capacity of the pervious pavers.

A partnership between Credit Valley Conservation and the University of Guelph completed a project at the IMAX Corporation headquarters in which Sorbtive Media was used downstream of a bioretention cell to provide tertiary nutrient treatment.

A project at Mayville Park in Upstate New York used six retrofit filtration cells surrounding draining inlets near a community centre, which previously had no stormwater treatment on-site. [1] In addition to these projects included on their website, Sorbtive Media was installed at the Colony Trail retrofit in East Gwillimbury. The Imbrium Sorbtive Media chamber removed an average of 66 % of dissolved phosphate from the site. [2]

Benefits

A pilot study was undertaken by researchers at Fleming College in Ontario, Canada to assess the phosphorus removal performance of bioretention soil mix amended with Sorbtive Media. Five bioretention cells were constructed and filled with a soil mix comprised of sand, peat moss, and various percentages of Sorbtive Media. Batches of artificial stormwater containing differing concentrations of phosphorus were used to simulate storm events on the bioretention cells. Through analysis of the influent and effluent concentrations, it was determined that the amended bioretention cells demonstrated substantial improvement in phosphorus removal. Each of the amended cells maintained removal efficiency of up to 99 % and at least 84 % for the duration of the study, even when blended into the soil mix at only 3 - 5 % volume basis.[3]

1. Imbrium Stormwater Treatment Solutions. Sorbtive Media. http://www.imbriumsystems.com/stormwater-treatment-solutions/sorbtive-media. Accessed October 6, 2017
2. Lake Simcoe Region Conservation Authority (LSRCA). Showcasing Water Innovation: Stormwater Performance Monitoring Report. 2013. http://www.lsrca.on.ca/Shared%20Documents/reports/swi_monitoring_2013.pdf. Accessed October 6, 2017.
3. Balch G. Broadbent H, Wootton B, Collins S. Phosphorus Removal Performance of Bioretention Soil Mix Amended with Imbrium Systems Sorbtive Media. 2013. Centre for Alternative Wastewater Treatment, in association with Fleming College. http://www.imbriumsystems.com/Portals/0/documents/sm/technical_docs/Fleming%20College%20CAWT%20Report%20on%20Sorbtive%20Media%20Pe

APPENDIX E

LID PRELIMINARY DESIGN

REAR YARD INFILTRATION TRENCH SIZING

Estimate imperviousness of drainage area from back half of roof draining to rear yard infiltration trenches, using a sample 13.7 m wide lot.

Total Area (1/2 of Lot Depth x Lot Frontage Width)	13.7 x 15 =	205.50 m^2
Imp Area (back 1/2 Roof)	(11.9 x 18 x 0.5)	107.10 m^2
Total Imp. Area		107.10 m^2
Imperviousness		**52.1%**

Sample Drainage Area 205.5 0.02055 ha/m

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003) 29.0 m^3/ha

Required Water Quality Infiltration Volume **0.597** m^3/Lot

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event 130.3 m^3/ha

Required 25 mm Storm Event Volume **2.678** m^3/Lot

Required Bioswale Volume **2.678 m^3/Lot**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS (FROM MOE 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 (ContinuousFlow)	90	150	200	240

Bioswale Design - Provided

	Units	Total to Bioswale
D - Depth	m	0.60
W - Width	m	1.0
L - Length	m	11.70
A - Bottom Area	m^2	11.7
Total Volume of the Bioswale (i.e. media volume)	m^3	7.0
n - Media Porosity		0.40
Total Runoff Storage Volume of the Bioswale	m^3	2.81
Total Runoff Storage Volume of the Bioswale	mm	26.2

Half of 15m ROW BIOSWALE SIZING

Estimate imperviousness of drainage area from road area draining to bioswale

Total Area (assume 1 m sample section, crown of road to ROW limit)	5.75 x 1 =	5.75 m^2
Imp Area (Roof)		0 m^2
Imp Area (Driveway)		0 m^2
Imp Area (Sidewalk/Trail)		0 m^2
Imp Area (Pavement+Curb)	3.75 + 0.5	4.25 m^2
Total Imp. Area		4.25 m^2
Imperviousness		**73.9%**
Sample Drainage Area	5.75 m2/m-road	0.000575 ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	36.3 m^3/ha
Required Water Quality Infiltration Volume	**0.021** m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	184.8 m^3/ha
Required 25 mm Storm Event Volume	**0.106** m^3/m-road

Required Trench Volume	**0.106 m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MOE 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 (ContinuousFlow)	90	150	200	240

Bioswale Design - Provided

	Units	Total to Bioswale
D - Depth	m	0.60
W - Width	m	0.5
L - Length	m	1.00
A - Bottom Area	m^2	0.5
Total Volume of the Bioswale (i.e. media volume)	m^3	0.3
n - Media Porosity		0.40
Total Runoff Storage Volume of the Bioswale	m^3	0.12
Total Runoff Storage Volume of the Bioswale	mm	28.2

Half of 16.5m ROW BIOSWALE SIZING

Estimate imperviousness of drainage area from half of the road area and half of the adjacent 13.7m lot draining to bioswale

Total Area (assume 1 m sample section, crown of road to lot split point)	13.5 + 7.25 x 1 =	20.75 m^2
Imp Area (Roof)	11.9 / 13.7 x 1 x 9 =	7.82 m^2
Imp Area (Driveway)	6 / 13.7 x 1 x 4.5 =	1.97 m^2
Imp Area (Sidewalk/Trail/Multi-Use Pathway)		0 m^2
Imp Area (Pavement+Curb)	3.75 + 0.5 =	4.25 m^2
Total Imp. Area		14.04 m^2
Imperviousness		**67.7%**
Sample Drainage Area	20.75 m2/m-road	0.002075 ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	34.2 m^3/ha
Required Water Quality Infiltration Volume	**0.071** m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	169.1 m^3/ha
Required 25 mm Storm Event Volume	**0.351** m^3/m-road
Required Trench Volume	**0.351 m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MOE 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 (ContinuousFlow)	90	150	200	240

Bioswale Design - Provided

	Units	Total to Bioswale
D - Depth	m	0.73
W - Width	m	1.20
L - Length	m	1.00
A - Bottom Area	m^2	1.20
Total Volume of the Bioswale (i.e. media volume)	m^3	0.9
n - Media Porosity		0.40
Total Runoff Storage Volume of the Bioswale	m^3	0.35
Total Runoff Storage Volume of the Bioswale	mm	25.0

Based on the maximum dimensions of the bioswale to avoid conflicts with service laterals and utilities in the boulevard, the bioswale provides 25 mm/impervious area of storage.

15m HALF ROW BOULEVARD FILTRATION TRENCH SIZING

Estimate imperviousness of drainage area from roofs, driveway, and road areas draining to filtration trench. Assume a section of road with a 12.2 m frontage lot with a split draining lot

Total Area	12.2 x 24 =	292.80 m^2
Imp Area (Roof)	(187 x 1/2) =	93.50 m^2
Imp Area (Driveway, including boulevard driveway)	(6 x 6) + (5.5 x 6) =	69 m^2
Imp Area (Sidewalk, less driveway overlap)	(1.5 x 12.2) - (6 x 1.5)=	9.3 m^2
Imp Area (Pavement+Curb)	(3.75 + 0.5) x 12.2=	51.85 m^2
Total Imp. Area		223.65 m^2
Imperviousness		**76.4%**
Sample Drainage Area	24 m2/m-road	0.0024 ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	37.1 m^3/ha
Required Water Quality Infiltration Volume	**0.089** m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	191.0 m^3/ha
Required 25 mm Storm Event Volume	**0.458** m^3/m-road

Required Trench Volume	**0.458 m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MOE 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 (ContinuousFlow)	90	150	200	240

Filtration Trench Design - Provided

	Units	Total to Filtration Trench
D - Depth	m	0.80
W - Width	m	1.25
L - Length	m	1.00
A - Bottom Area	m^2	1.3
Total Volume of the Filtration Trench (i.e. stone volume)	m^3	1.0
n - Media Porosity		0.40
Total Runoff Storage Volume of the Filtration Trench	m^3	0.40
Total Runoff Storage Volume of the Filtration Trench	mm	21.8

Based on the maximum dimensions of the filtration trench to avoid conflicts with service laterals and utilities in the boulevard, the filtration trench provides 21.8 mm/impervious area of storage.

16.5m ROW BOULEVARD FILTRATION TRENCH SIZING

Estimate imperviousness of drainage area from roofs, driveway, and road areas draining to filtration trench. Assume a section of road with a 13.7 m frontage lot with a split draining lot on one side.

Total Area	13.7 x 24.25 =	332.23	m^2
Imp Area (Roof)	214 x 1/2 =	107.00	m^2
Imp Area (Driveway, including boulevard driveway)	(6 x 6) + (5.5 x 6) =	69	m^2
Imp Area (Sidewalk, less driveway overlap)	(1.5 x 13.7) - (6 x 1.5)=	11.55	m^2
Imp Area (Pavement+Curb)	(3.7 + 0.5) x 13.7 =	57.54	m^2
Total Imp. Area		245.09	m^2
Imperviousness		**73.8%**	
Sample Drainage Area	13.7 m2/m-road	0.00137	ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	36.3	m^3/ha
Required Water Quality Infiltration Volume	**0.050**	m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	184.4	m^3/ha
Required 25 mm Storm Event Volume	**0.253**	m^3/m-road

Required Trench Volume	**0.253**	**m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS (FROM MOE 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 (ContinuousFlow)	90	150	200	240

Filtration Trench Design - Provided

	Units	Total to Filtration Trench
D - Depth	m	0.80
W - Width	m	1.25
L - Length	m	1.00
A - Bottom Area	m^2	1.3
Total Volume of the Filtration Trench (i.e. stone volume)	m^3	1.0
n - Media Porosity		0.40
Total Runoff Storage Volume of the Filtration Trench	m^3	0.40
Total Runoff Storage Volume of the Filtration Trench	mm	39.6

Based on the maximum dimensions of the filtration trench to avoid conflicts with service laterals and utilities in the boulevard, the filtration trench provides 39.6 mm/impervious area of storage.

18m ROW BIOSWALE INFILTRATION SIZING
STREET D CUL-DE-SAC

Estimate imperviousness of drainage area from road area and half of the adjacent Townhouse lots draining to bioswale

Total Area (assume 1 m sample section, whole ROW plus 2 half lots)	(18 + 15 + 15) x 1 =	48.00 m^2
Imp Area (Roof)	2 x 31.05 / 34 x 1 x 7.5 =	13.70 m^2
Imp Area (Driveway)	2 x 3.5 x 5 / 34 x 1 x 7.5 =	7.72 m^2
Imp Area (Sidewalk/Trail)	1.5 x 1 =	1.5 m^2
Imp Area (Pavement+Curb)	2 x (4 + 0.5) =	9 m^2
Total Imp. Area		31.92 m^2
Imperviousness		**66.5%**
Total Drainage Area to Biofiltration Infiltration System	4500 m2	0.45 ha

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	33.8 m^3/ha
Required Water Quality Infiltration Volume	**15.225** m^3

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	166.2 m^3/ha
Required 25 mm Storm Event Volume	**74.810** m^3

Required Trench Volume	**74.810** m^3

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MOE 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 (ContinuousFlow)	90	150	200	240

Bioswale Design - Provided

	Units	Total to Bioswale
D - Depth	m	3.50
A - Bottom Area	m^2	48.93
Total Volume of the Bioswale (i.e. media volume)	m^3	171.3
n - Media Porosity		0.40
Total Runoff Storage Volume of the Bioswale	m^3	68.50
Total Runoff Storage Volume of the Bioswale	mm	22.9

(Total area of Cul-de-sac Bioswale Infiltration Trench, less area of catchbasins. See **Figure 6.7**.)

18m ROW BOULEVARD FILTRATION TRENCH SIZING

Estimate imperviousness of drainage area from roofs, driveway, and road areas draining to filtration trench. Assume a section of road with a 13.7 m frontage lot with a split draining lot on one side and front draining lot on the other.

Total Area	13.7 x 68 =	931.60 m^2
Imp Area (Roof)	(185 x 1/2) + (185) =	277.50 m^2
Imp Area (Driveway, including boulevard driveway)	(6 x 6 x 2) + (5 x 6 x 2) =	132 m^2
Imp Area (Sidewalk, less driveway overlap)	(1.5 x 13.7) - (6 x 1.5)=	11.55 m^2
Imp Area (Pavement+Curb)	(8 + 0.5 + 0.5) x 13.7=	123.3 m^2
Total Imp. Area		544.35 m^2

Imperviousness **58.4%**

Sample Drainage Area 68 m2/m-road 0.0068 ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003) 31.1 m^3/ha
Required Water Quality Infiltration Volume **0.212** m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event 146.1 m^3/ha
Required 25 mm Storm Event Volume **0.993** m^3/m-road

Required Trench Volume	**0.993 m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS
(FROM MOE 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 (ContinuousFlow)	90	150	200	240

Filtration Trench Design - Provided

	Units	Total to Filtration Trench
D - Depth	m	0.80
W - Width	m	1.25
L - Length	m	1.00
A - Bottom Area	m^2	1.3
Total Volume of the Filtration Trench (i.e. stone volume)	m^3	1.0
n - Media Porosity		0.40
Total Runoff Storage Volume of the Filtration Trench	m^3	0.40
Total Runoff Storage Volume of the Filtration Trench	mm	10.1

Based on the maximum dimensions of the filtration trench to avoid conflicts with service laterals and utilities in the boulevard, the filtration trench provides 10.1 mm/impervious area of storage.

23m ROW BOULEVARD FILTRATION TRENCH SIZING

Estimate imperviousness of drainage area from roofs, driveway, and road areas draining to filtration trench. Assume a section of road with a 15.2 m frontage lot with a split draining lot on one side and front draining lot on the other.

Total Area	15.2 x 68 =	1033.60	m^2
Imp Area (Roof)	(208 x 1/2) + (208) =	312.00	m^2
Imp Area (Driveway, including boulevard driveway)	(6 x 6 x 2) + (5.5 x 6) + (7 x 6) =	147	m^2
Imp Area (Sidewalk, MUP, less driveway overlap)	(3 x 15.2) + (1.5 x 15.2) - (6 x 1.5) - (6 x 3) =	41.4	m^2
Imp Area (Pavement and curbs)	(10.5+0.5+0.5) x 15.2=	174.8	m^2
Total Imp. Area		675.20	m^2
Imperviousness		**65.3%**	
Sample Drainage Area	68 m2/m-road	0.0068	ha/m-road

Required Volume per Hectare (Water Quality Requirements)

(as per Table 3.2, MOE, 2003)	33.4	m^3/ha
Required Water Quality Infiltration Volume	**0.227**	m^3/m-road

Required Volume per Hectare (25 mm Storm Requirements)

as per 25 mm Storm Event	163.3	m^3/ha
Required 25 mm Storm Event Volume	**1.111**	m^3/m-road

Required Trench Volume	**1.111**	**m^3/m-road**

TABLE 3.2 - WATER QUALITY STORAGE REQUIREMENTS (FROM MOE 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 (ContinuousFlow)	90	150	200	240

Filtration Trench Design - Provided

	Units	Total to Filtration Trench
D - Depth	m	0.80
W - Width	m	1.25
L - Length	m	1.00
A - Bottom Area	m^2	1.3
Total Volume of the Filtration Trench (i.e. stone volume)	m^3	1.0
n - Media Porosity		0.40
Total Runoff Storage Volume of the Infiltration/Filtration Trench	m^3	0.40
Total Runoff Storage Volume of the Infiltration/Filtration Trench	mm	9.0

Based on the maximum dimensions of the filtration trench to avoid conflicts with service laterals and utilities in the boulevard, the filtration trench provides 9 mm/impervious area of storage.

APPENDIX F

DETENTION STORAGE PRELIMINARY DESIGN

SCS consulting group ltd

Permanent Pool and Extended Detention Sizing - Outlet 2

Weighted Impervious Calculation

Catchment ID	Total Area	Imperviousness	Impervious Area
	(ha)	(%)	(ha)
206	2.33	64	1.49
Total	**2.33**	**64**	**1.49**

Water Quality and Extended Detention Sizing - Outlet 2

Shining Hill PH3 (Aurora)
Project Number: 2183
Date: November 2021
Designer Initials: MECM

EXTENDED DETENTION

Using the 25mm - 4 hour Chicago Storm

Erosion Control Volume (V) = Runoff Depth (mm) x Drainage Area (ha) x 10 (m^3) / (mm)(ha)

Erosion Control Volume (V) = 11.75 mm x 2.33 ha x 10 m^3 / mm·ha

Erosion Control Volume (V) = 274 m^3

Peak Flowrate (Q_p) = Extended Detention Volume (m^3) / Detention Time (hr) x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 274 m^3 / 24 hr x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 0.005 m^3/s

Permanent Pool and Extended Detention Sizing - Outlet 4

Weighted Impervious Calculation

Catchment ID	Total Area	Imperviousness	Impervious Area
	(ha)	(%)	(ha)
203	2.40	66	1.58
204	0.42	67	0.28
Total	**2.82**	**66**	**1.87**

Water Quality and Extended Detention Sizing - Outlet 4

EXTENDED DETENTION

Using the 25mm - 4 hour Chicago Storm

Erosion Control Volume (V) = Runoff Depth (mm) x Drainage Area (ha) x 10 (m^3) / (mm)(ha)

Erosion Control Volume (V) = 12.72 mm x 2.82 ha x 10 m^3 / mm·ha

Erosion Control Volume (V) = 359 m^3

Peak Flowrate (Q_p) = Extended Detention Volume (m^3) / Detention Time (hr) x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 359 m^3 / 24 hr x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 0.006 m^3/s

Permanent Pool and Extended Detention Sizing - Outlet 5

Weighted Impervious Calculation

Catchment ID	Total Area	Imperviousness	Impervious Area
	(ha)	(%)	(ha)
201	1.83	84	1.54
202	0.47	66	0.31
208	1.15	72	0.83
EXT1	0.55	65	0.36
Total	**4.00**	**76**	**3.03**

Water Quality and Extended Detention Sizing - Outlet 5

Shining Hill PH3 (Aurora)
Project Number: 2183
Date: November 2021
Designer Initials: MECM

EXTENDED DETENTION

Using the 25mm - 4 hour Chicago Storm

Erosion Control Volume (V) = Runoff Depth (mm) x Drainage Area (ha) x 10 (m^3) / (mm)(ha)

Erosion Control Volume (V) = 17.81 mm x 4.00 ha x 10 m^3 / mm·ha

Erosion Control Volume (V) = 712 m^3

Peak Flowrate (Q_p) = Extended Detention Volume (m^3) / Detention Time (hr) x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 712 m^3 / 24 hr x 1 (hr) / 3600 (s) x 1.5 (peaking factor)

Peak Flowrate (Q_p) = 0.012 m^3/s

APPENDIX G

SANITARY FLOW CALCULATIONS

SCS consulting group ltd

Sanitary Design Sheet - Option 1 Downstream Analysis of Willow Farm & Heatherfield Sewers
Shining Hill Estates
Phase 3 (Aurora) - FSSR
Aurora, York Region

Project: Shining Hill Estates
Project No. 2183
Date: 20-Dec-21
Designed By: E.T.C.K.
Reviewed By: S.E.K.

Minimum Sewer Diameter (mm) =	**200**	**Avg. Domestic Flow (l/cap/day) =**	**400**
Mannings n =	**0.013**	**Infiltration Rate (l/s/ha) =**	**0.26**
Minimum Velocity (m/s) =	**0.60**	**Max. Harmon Peaking Factor =**	**4.0**
Maximum Velocity (m/s) =	**3**	**Min. Harmon Peaking Factor =**	**2.0**
Minimum Pipe Slope (%) =	**1.00**	**NOMINAL PIPE SIZE USED**	

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LOCATION			RESIDENTIAL							FLOW CALCULATIONS								PIPE DATA				
STREET	MANHOLE FROM	MANHOLE TO	AREA (ha)	ACCUM. AREA (ha)	UNITS (#)	DENSITY PER UNIT (p/unit)	DENSITY PER HA (p/ha)	RESIDENTIAL POPULATION	ACCUM. RESIDENTIAL POPULATION	INFILTRATION (L/s)	TOTAL ACCUM. POPULATION	AVG. DOMESTIC FLOW (L/s)	ACCUM. AVG. DOMESTIC FLOW (L/s)	PEAKING FACTOR	PEAKED RESIDENTIAL FLOW (L/s)	ICI FLOW (L/s)	TOTAL FLOW (L/s)	LENGTH (m)	PIPE DIAMETER (mm)	SLOPE (%)	FULL FLOW CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)
Townhouse Residential	MH1	MH99	0.75	0.75	21	3.5		74	74	0.2	74	0.3	0.3	4.00	1.4	0.0	1.6	100.0	200	1.00	32.8	1.04
Neighbourhood Park	MH2	MH99	1.6	1.6	0		50	80	80	0.4	80	0.4	0.4	4.00	1.5	0.0	1.9	100.0	200	1.00	32.8	1.04
Single Family Residential	MH3	MH99	6.98	6.98	87	3.8		331	331	1.8	331	1.5	1.5	4.00	6.1	0.0	7.9	100.0	200	1.00	32.8	1.04
St. Anne's School	MH4	MH99	4.28	4.28	800	1		800	800	1.1	800	3.7	3.7	3.86	14.3	0.0	15.4	100.0	200	1.00	32.8	1.04
Phase 3 Connection	MH99	MH69A	0	13.61	0			0	1284	3.5	1284	0.0	5.9	3.73	22.2	0.0	25.7	100.0	200	1.90	45.2	1.44
Phase 2 External	ext3	MH69A	9.32	9.32	92	3.8		350	350	2.4	350	1.6	1.6	4.00	6.5	0.0	8.9	46.8	200	2.04	46.8	1.49
Willow Farm Lane	MH69A	MH68A	0	22.93	0			0	1634	6.0	1634	0.0	7.6	3.65	27.6	0.0	33.6	46.8	200	2.04	46.8	1.49
External to Willow Farm	ext4	MH63A	53.93	53.93	302	3.8		1148	1148	14.0	1148	5.3	5.3	3.76	20.0	0.0	34.0	100.0	250	1.00	59.4	1.21
Willow Farm Lane (south)	MH63A	MH64A	0.54	54.47	12	1		12	1160	14.2	1160	0.1	5.4	3.76	20.2	0.0	34.3	100.0	250	0.40	37.6	0.77
Willow Farm Lane	MH68A	MH64A	1.95	24.88	19	1		19	1653	6.5	1653	0.1	7.7	3.65	27.9	0.0	34.4	48.2	200	4.95	72.9	2.32
Heatherfield Lane	MH64A	MH65A	0.11	79.46	0			0	2812	20.7	2812	0.0	13.0	3.47	45.1	0.0	65.8	76.3	300	0.40	61.1	0.86
Heatherfield Lane	MH65A	MH66A	0.64	80.1	15	1		15	2827	20.8	2827	0.1	13.1	3.46	45.3	0.0	66.2	73.5	300	0.40	61.1	0.86
Heatherfield Lane	MH66A	MH67A	1.04	81.14	15	1		15	2842	21.1	2842	0.1	13.2	3.46	45.6	0.0	66.7	29.6	300	0.48	67.0	0.95
Easement	MH67A	MH74A	0	81.14	0			0	2842	21.1	2842	0.0	13.2	3.46	45.6	0.0	66.7	49.7	300	0.47	66.3	0.94
Easement	MH74A	MH73A	0	81.14	0			0	2842	21.1	2842	0.0	13.2	3.46	45.6	0.0	66.7	55.2	300	1.00	96.7	1.37
Easement	MH73A	MH72A	0	81.14	0			0	2842	21.1	2842	0.0	13.2	3.46	45.6	0.0	66.7	27.1	300	2.03	137.7	1.95
St. John's Sideroad	MH72A	MH71A	0.8	81.94	1	3.8		4	2846	21.3	2846	0.0	13.2	3.46	45.6	0.0	66.9	102.7	300	0.42	62.6	0.89
St. John's Sideroad	MH71A	MH70A	3.63	85.57	1	3.8		4	2850	22.2	2850	0.0	13.2	3.46	45.7	0.0	67.9	89.9	300	0.46	65.6	0.93
St. John's Sideroad	MH70A	MH70C	0	85.57	0			0	2850	22.2	2850	0.0	13.2	3.46	45.7	0.0	67.9	7.0	300	0.46	65.6	0.93
Yonge Street	MH70C	MH70B	0.08	85.65	0			0	2850	22.3	2850	0.0	13.2	3.46	45.7	0.0	67.9	43.0	300	0.44	64.1	0.91

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Sanitary Design Sheet - Option 2 New Sanitary Sewer on St. John's SR
Shining Hill Estates
Phase 3 (Aurora) - FSSR
Aurora, York Region

Minimum Sewer Diameter (mm) =	**200**	**Avg. Domestic Flow (l/cap/day) =**	**400**
Mannings n =	**0.013**	**Infiltration Rate (l/s/ha) =**	**0.26**
Minimum Velocity (m/s) =	**0.60**	**Max. Harmon Peaking Factor =**	**4.0**
Maximum Velocity (m/s) =	**3**	**Min. Harmon Peaking Factor =**	**2.0**
Minimum Pipe Slope (%) =	**1.00**	**NOMINAL PIPE SIZE USED**	

Project: Shining Hill Estates
Project No. 2183
Date: 20-Dec-21
Designed By: E.T.C.K.
Reviewed By: S.E.K.

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LOCATION			RESIDENTIAL							FLOW CALCULATIONS								PIPE DATA				
STREET	MANHOLE FROM	MANHOLE TO	AREA (ha)	ACCUM. AREA (ha)	UNITS (#)	DENSITY PER UNIT (p/unit)	DENSITY PER HA (p/ha)	RESIDENTIAL POPULATION	ACCUM. RESIDENTIAL POPULATION	INFILTRATION (L/s)	TOTAL ACCUM. POPULATION	AVG. DOMESTIC FLOW (L/s)	ACCUM. AVG. DOMESTIC FLOW (L/s)	PEAKING FACTOR	PEAKED RESIDENTIAL FLOW (L/s)	ICI FLOW (L/s)	TOTAL FLOW (L/s)	LENGTH (m)	PIPE DIAMETER (mm)	SLOPE (%)	FULL FLOW CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)
Townhouse Residential	MH1	MH99	0.75	0.75	21	3.5		74	74	0.2	74	0.3	0.3	4.00	1.4	0.0	1.6	100.0	200	1.00	32.8	1.04
Neighbourhood Park	MH2	MH99	1.6	1.6	0		50	80	80	0.4	80	0.4	0.4	4.00	1.5	0.0	1.9	100.0	200	1.00	32.8	1.04
Single Family Residential	MH3	MH99	6.98	6.98	87	3.8		331	331	1.8	331	1.5	1.5	4.00	6.1	0.0	7.9	100.0	200	1.00	32.8	1.04
St. Anne's School	MH4	MH99	4.28	4.28	800	1		800	800	1.1	800	3.7	3.7	3.86	14.3	0.0	15.4	100.0	200	1.00	32.8	1.04
Phase 3 Connection	MH99	MH72A	0	13.61	0			0	1284	3.5	1284	0.0	5.9	3.73	22.2	0.0	25.7	100.0	200	1.90	45.2	1.44
Phase 2 External	ext3	MH69A	9.32	9.32	92	3.8		350	350	2.4	350	1.6	1.6	4.00	6.5	0.0	8.9	46.8	200	2.04	46.8	1.49
Willow Farm Lane	MH69A	MH68A	0	9.32	0			0	350	2.4	350	0.0	1.6	4.00	6.5	0.0	8.9	46.8	200	2.04	46.8	1.49
External to Willow Farm	ext4	MH63A	53.93	53.93	302	3.8		1148	1148	14.0	1148	5.3	5.3	3.76	20.0	0.0	34.0	100.0	250	1.00	59.4	1.21
Willow Farm Lane (south)	MH63A	MH64A	0.54	54.47	12	1		12	1160	14.2	1160	0.1	5.4	3.76	20.2	0.0	34.3	100.0	250	0.40	37.6	0.77
Willow Farm Lane	MH68A	MH64A	1.95	11.27	19	1		19	369	2.9	369	0.1	1.7	4.00	6.8	0.0	9.8	48.2	200	4.95	72.9	2.32
Heatherfield Lane	MH64A	MH65A	0.11	65.85	0			0	1528	17.1	1528	0.0	7.1	3.67	26.0	0.0	43.1	76.3	300	0.40	61.1	0.86
Heatherfield Lane	MH65A	MH66A	0.64	66.49	15	1		15	1543	17.3	1543	0.1	7.1	3.67	26.2	0.0	43.5	73.5	300	0.40	61.1	0.86
Heatherfield Lane	MH66A	MH67A	1.04	67.53	15	1		15	1558	17.6	1558	0.1	7.2	3.67	26.5	0.0	44.0	29.6	300	0.48	67.0	0.95
Easement	MH67A	MH74A	0	67.53	0			0	1558	17.6	1558	0.0	7.2	3.67	26.5	0.0	44.0	49.7	300	0.47	66.3	0.94
Easement	MH74A	MH73A	0	67.53	0			0	1558	17.6	1558	0.0	7.2	3.67	26.5	0.0	44.0	55.2	300	1.00	96.7	1.37
Easement	MH73A	MH72A	0	67.53	0			0	1558	17.6	1558	0.0	7.2	3.67	26.5	0.0	44.0	27.1	300	2.03	137.7	1.95
St. John's Sideroad	MH72A	MH71A	0.8	81.94	1	3.8		4	2846	21.3	2846	0.0	13.2	3.46	45.6	0.0	66.9	102.7	300	0.42	62.6	0.89
St. John's Sideroad	MH71A	MH70A	3.63	85.57	1	3.8		4	2850	22.2	2850	0.0	13.2	3.46	45.7	0.0	67.9	89.9	300	0.46	65.6	0.93
St. John's Sideroad	MH70A	MH70C	0	85.57	0			0	2850	22.2	2850	0.0	13.2	3.46	45.7	0.0	67.9	7.0	300	0.46	65.6	0.93
Yonge Street	MH70C	MH70B	0.08	85.65	0			0	2850	22.3	2850	0.0	13.2	3.46	45.7	0.0	67.9	43.0	300	0.44	64.1	0.91

SCS consulting group ltd

Sanitary Sewer Hydraulic Grade Line Analysis
Shining Hill Estates
Phase 3 (Aurora) - FSSR
Aurora, York Region

Project: Shining Hill Estates
Project No. 2183
Date: 20-Dec-21
Designed By: ETCK
Reviewed By: SEK

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LOCATION			INVERTS		FLOW	PIPE DATA								PIPE LOSS CALCULATIONS					MH LOSS CALCULATIONS		TOTAL LOSS	HYDRAULIC GRADE LINE			
STREET	FROM (U/S)	TO (D/S)	U/S (m)	D/S (m)	TOTAL PIPE FLOW (Qdes) (L/s)	PIPE DIAMETER (mm)	LENGTH (m)	MANNING's 'n'	PIPE AREA (m2)	HYD. RAD$^{2/3}$	SLOPE (%)	Qcap. (L/s)	Qdes/Qcap (%)	L/D	f	Vf	$V^2/2g$	TOTAL PIPE LOSS (m)	MH LOSS (m)	PIPE BEND LOSS (m)	TOTAL LOSS (m)	HGL (U/S) (m)	HGL SURCHARGE ABOVE U/S OBV. (m)	HGL (D/S) (m)	MH TOP (U/S) (m)
St. John's Sideroad	MH72A	MH71A	246.889	246.458	66.9	300	102.7	0.013	0.071	0.178	0.42	62.6	1.07	342.333	0.031	0.946	0.046	0.492	0.03	0.00	0.52	247.308	0.118	246.786	248.79
St. John's Sideroad	MH71A	MH70A	246.454	246.041	67.9	300	89.9	0.013	0.071	0.178	0.46	65.6	1.04	299.500	0.031	0.961	0.047	0.443	0.00	0.00	0.45	246.786	0.032	246.341	249.31
St. John's Sideroad	MH70A	MH70C	246.030	245.998	67.9	300	7.0	0.013	0.071	0.178	0.46	65.6	1.04	23.333	0.031	0.961	0.047	0.035	0.00	0.00	0.04	246.341	0.010	246.304	248.20
Yonge Street	MH70C	MH70B	245.969	245.780	67.9	300	43.0	0.013	0.071	0.178	0.44	64.1	1.06	143.333	0.031	0.961	0.047	0.212	0.01	0.00	0.22	246.304	0.035	246.080	249.00

APPENDIX H

WATER DISTRIBUTION ANALYSIS LETTER

November 9, 2021 Project No. 17002-102

Sent via email
Mr. Paul Bailey
Shining Hill Estate Collection Inc.
2235 Sheppard Avenue East, Suite 903
Toronto, ON M2J 5B5

Subject: Shining Hill Phase 3 Development
Water Distribution Modeling
Town of Aurora, Region of York

Dear Mr. Bailey,

We are pleased to submit our report entitled “Shining Hill Phase 3 Development Watermain Analysis” outlining the results of our water distribution analysis for the proposed residential development in the Town of Aurora, Region of York.

A WaterCAD model of the immediate area was developed utilizing the design information provided to Municipal Engineering Solutions and a hydrant test performed by The Ontario Clean Water Agency in April 2021. The findings of our analysis are summarized in the following report.

We trust you find this report satisfactory. Should you have any questions or require further clarification, please call.

Yours truly,

Municipal Engineering Solutions

Kristin St-Jean, P.Eng.

/KS

File Location: C:\Users\krist\Documents\Projects\17002-102 Shining Hill, Aurora\5.0 Report\2021-11 Report Update\17002-102 Shining Hill Phase 3 Development Watermain Analysis_20211109.docx

55 Gilbank Drive, Aurora, Ontario L4G 6H9

Tel: 905.726.1016 Cell: 416.434.0186 Fax: 905.726.1225

SHINING HILL PHASE 3 DEVELOPMENT

WATERMAIN ANALYSIS

PREPARED BY:

MUNICIPAL ENGINEERING SOLUTIONS

Municipal Engineering
Solutions

FOR:

SHINING HILL ESTATE COLLECTION INC.
Updated November 2021

Project Number: 17002-102

TABLE OF CONTENTS

Section 1 – INTRODUCTION

Municipal Engineering Solutions ("MES") was retained by Shining Hill Estate Collection Inc. to conduct a hydraulic water analysis for the proposed development located on the north side of St. John's Sideroad, west of Yonge Street in the Town of Aurora (Region of York). As part of this hydraulic assessment MES was requested to undertake the following:

1. Calculate/verify water demands for the proposed development using Town of Aurora, provincial and industry design standards;
2. Add the subject watermains/development to the development water model;
3. Run the model to size the subject mains to achieve service criteria during Average Day, Peak Hour and fire flow during Maximum Day demand; and
4. Prepare a Report summarizing the modeling results for agency review and design purposes.

1.1 Development Background

The development site is located on the north side of St. John's Sideroad (north of Willow Farm Lane) and west of Yonge Street in the Town of Aurora. The proposed development is made up of 87 single family detached homes and 21 townhouses. A future school will be located west of this development which will be serviced from the proposed watermains. The breakdown of the buildings is shown in **Appendix A**. The proposed development is shown below on **Figure 1**.

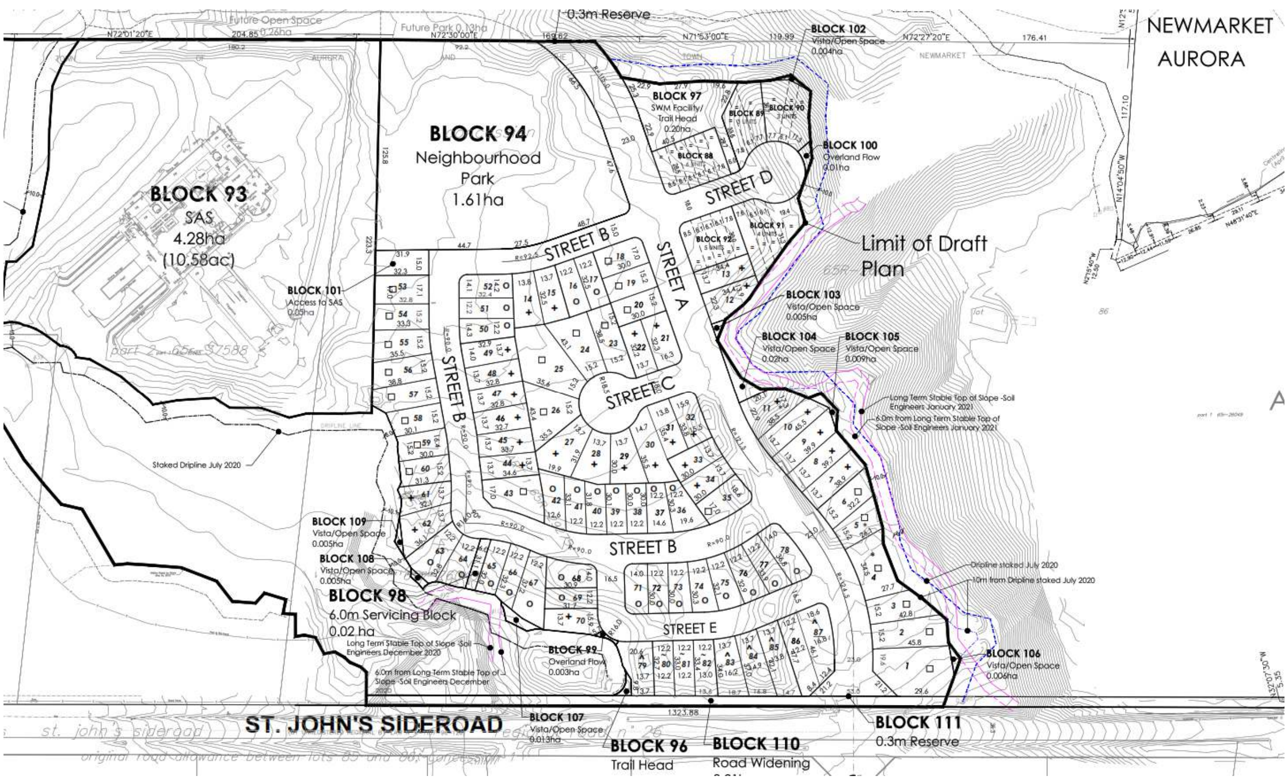


Figure 1 – Proposed Shining Hill Phase 3 Development

Section 2 – WATERMAIN DESIGN CRITERIA

The design criteria utilized to estimate the water demands for the hydraulic water model follows general industry standards and is calculated using the design criteria and guidelines outlined in the Town of Aurora Design Criteria and the Ministry of the Environment, Conservation and Parks (MECP) Watermain Design Criteria.

The following sections summarize the specific design criteria used to carry out the hydraulic watermain assessment for this development.

2.1 Equivalent Population Densities & Water Design Factors

To calculate the equivalent population and water design factors for this development MES used Town of Aurora standard demand rates as noted in the *Design Criteria Manual for Engineering Plans (June 2021)*. The ultimate population of the school is estimated to be 800 people (staff and students). **Table 1** summarizes the residential population densities and **Table 2** summarizes the average daily demand and peaking factors used for the calculations. The residential demand rate was also used for the school population because it will be a boarding school.

Table 1 – Equivalent Population Density

Type of Development	Equivalent Population (Persons/Unit)
Single Detached Dwellings	3.8
Townhouses	3.5

Source: Design Criteria Manual for Engineering Plans, June 2021

Table 2 - Water Design Factors

Type of Development	Average Daily Demand	Maximum Daily Demand Peaking Factor	Peak Hourly Demand Peaking Factor
Residential	400 l/capita/day	2.0	3.0

Source: Town of Aurora Design Criteria Manual, June 2021

Section 3 –FLOW DEMANDS

Utilizing the demand criteria and the Average Day, Maximum Day and Peak Hour peaking factors from Table 1 the water demands for this development were calculated.

3.1 Equivalent Population Flow Demands

The calculated demands for the development are summarized in **Table 3**. For additional details on the development water demands and assigned demand nodes used in the water model see **Appendix A**.

Table 3 – Water Demand for the Shining Hill Phase 3 Development

	Average Day Demand (L/s)	Maximum Day Demand (L/s)	Peak Hour Demand (L/s)
Water Demands	5.57	11.15	16.72

3.2 Fire Flow Demands

The fire flow demands for the development were taken from the Town of Aurora *Design Criteria Manual for Engineering Plans (June 2021)*. The minimum required fire flow for the school was assumed to be 250 L/s as per the Town's Criteria, however the school is a retrofit of an existing building and the fire flow for the school has yet to be finalized. The minimum required fire flows assumed for this development are summarized in **Table 4**.

Table 4 - Fire Flow Requirements

Type of Development	Fire Flow (L/s)
Single Family Homes	117
Townhouses	125
School	250

Source: Town of Aurora Design Criteria Manual, June 2021

The fire flow utilized in this analysis is based on the Town's minimum fire flow requirements. The fire flow noted in Table 4 must be reviewed and confirmed by the appropriate designer (architect) with detailed design data (floor area and type of construction) for these buildings prior to implementation and construction. A greater fire flow than currently noted within the Town's Criteria may be required for the school building or the required fire flows may need to be calculated using the Fire Underwriters Survey. Regardless, the buildings will need to be designed to suit the available flow and pressure. Any design/criteria changes are to be reported to MES.

Section 4 – OTHER SYSTEM REQUIREMENTS

4.1 System Pressure Requirements

In addition to meeting the various flow requirements, the system must also satisfy minimum and maximum pressure requirements as outlined by the Town. The Town's pressure requirements are outlined in the Design Criteria and stipulate the following:

1. The pressure range during maximum daily demand shall be 350 kPa to 620 kPa (50 to 90 psi)
2. The maximum system pressure under static load or during minimum hourly demand shall be 700 kPa (100 psi).
3. The minimum pressure during peak hourly demand shall be 275 kPa (40 psi).
4. The minimum system pressure when the system is tested for fire flow during maximum day demands shall be 140 kPa (20 psi).

To comply with the Ontario Building Code, reduction of pressures to 550 kPa (80 psi) is required, normally by having reducing valves installed on individual services.

4.2 Watermain Sizing

The Town of Aurora stipulates a minimum pipe size of 200 mm for residential areas and 250 mm diameter for industrial, commercial and institutional areas. All watermains are adequately sized to maintain demand flows at the required pressures without causing excessive energy loss or result in water quality decay. The watermain system must therefore be designed to accommodate the greater of the following:

- Maximum day plus fire demand
- Peak hour demand

For distribution systems providing fire protection the minimum pipe size shall be 150 mm diameter in accordance with Ministry of the Environment, Conservation and Parks (MECP) and NFPA requirements.

To provide appropriate fire protection, reliable supply and pressures the water distribution system should be looped wherever possible to improve supply security and water quality.

4.3 Watermain C-Factor

In designing and modeling of the pipes the Coefficient of Roughness (C-Factor) factors from the Town's Design Criteria were utilized. The Coefficient of Roughness assigned to each pipe size is summarized in **Table 5** below.

Table 5 - Hazen-Williams Coefficient of Roughness (C-Factors)

Size of Pipe (Diameter in mm)	Coefficient of Roughness (C)
150 mm	100
200 mm to 250 mm	110
300 mm	120
400 mm to 450 mm	130
600 mm or Greater	140

Source: Town of Aurora Design Criteria Manual, June 2021

Section 5 – ANALYSIS & MODELING RESULTS

In order to conduct the hydraulic water analysis for the proposed development the water demands were estimated by MES using the design criteria previously discussed and incorporated into a WaterCAD model created for the immediate area using boundary conditions from a hydrant test. The following sections discuss the model setup and results.

5.1 Model Setup

A hydrant test was performed on St. John's Sideroad by The Ontario Clean Water Agency on April 28th, 2021. The hydrant test results are included in **Appendix B**.

The development is located in the Aurora Central (Zone 1) Pressure District. The proposed water supply for the development is from two connections to the existing 300 mm/200mm diameter watermain on St. John's Sideroad.

At the north extent of the development, the watermain will connected in the future to the Newmarket System. The future connection will need to have a Water Meter and a Pressure Reducing Valve (PRV) with an anti-stagnation device. The direction of the PRV and meter will depend on which Newmarket pressure zone the watermain is connected to (Newmarket Central or Newmarket West).

Friction factor for all new pipes added to the model were assigned according to Table 5. Fire flows were based on the Town of Aurora Design Criteria. Elevations within the development vary from approximately 262.1 m to 270.0 m.

5.2 Watermain Sizing and System Pressures

The analysis was conducted under existing servicing conditions for Average Day, Maximum Day, Peak Hour and Maximum day plus Fire demands to size the watermains and meet the pressure requirements. The pipe sizes and layout are shown in **Appendix A**.

Modeled service pressures for the development are summarized in **Table 6**. All pressures lie within the required operating range under average day, maximum day, maximum day plus fire flow and peak hour demands. The modelling indicates that pressures are not expected to exceed 550 kPa within the proposed development. Since modeling was done using a single demand scenario for boundary conditions (hydrant test), it is anticipated that pressures will be lower during peak hour and higher during minimum hour than indicated in the modeling.

Detailed pipe and node tables for the various scenarios modeled are attached to this report in **Appendix C**.

Table 6 - Modeled Service Pressures

Scenario	Average Day	Maximum Day	Peak Hour	Max. Day + Fire
Existing	401 – 525 kPa (58.2 to 76.1 psi)	401 – 524 kPa (58.1 to 76.1 psi)	400 – 524 kPa (58.1 to 76.1 psi)	202 to 353 L/s @ 140 kPa

Section 6 – CONCLUSIONS/RECOMMENDATIONS

The proposed watermain layout for the Shining Hill Phase 3 Development can achieve hydraulic requirements as prescribed by the Town of Aurora watermain design criteria as summarized below.

- The service pressures from the proposed watermain layout are expected to range between 400 kPa to 525 kPa (58.1 psi to 76.1 psi).
- The available fire flow meets or exceeds the minimum fire flow demands utilized for this assessment at the minimum pressure of 140 kPa. Assumptions made within this report must be confirmed when additional building information becomes available.
- Once the building designs are completed and the specifics are known, the fire flow demands used in this analysis and summarized in Table 4, including all assumptions, must be reviewed and confirmed by the designer(s), architect and mechanical consultant to ensure the fire flows used within this report are still valid prior to implementation and construction and to confirm that the water supply is adequate.
- The fire flows utilized in this analysis are based on the Town's minimum fire flow requirements. Should it be determined, based on the final site and building design, that a greater fire flow is required or that the fire flows need to be calculated using the Fire Underwriters Survey formula the pipe sizes may need to be upsized to suit the higher fire flows or the building construction designed to suit the flow available.
- The minimum required fire flow for the school was assumed to be 250 L/s as per the Town's Criteria, however the school is a retrofit of an existing building and the required fire flow for the school has yet to be finalized. The required fire flow for the school building must be confirmed by the Town and the school architect. Regardless, the building will need to be designed to suit the available flow and pressure. Any design/criteria changes are to be reported to MES.
- Confirmation and/or changes to the criteria should also be provided to and reviewed with MES prior to the finalization of the detailed design drawings and construction of the watermain system. Final design parameters are to be provided to MES prior to construction for further review to confirm that the actual (final) site conditions and building design(s) reflect those modeled by MES within this report.
- The hydrant test used for the boundary conditions provides a snapshot of the system performance and does not capture the system variation as accurately as boundary information from a calibrated model or system monitoring. The Town of Aurora must confirm that the results presented in this report are in keeping with the pressures currently measured in the area.
- This report, including all modeling assumptions used, is to be submitted to and reviewed by the water operating authority (municipality) to confirm that the modeling parameters used are acceptable to the operating authority and/or confirm if modified domestic or fire flow requirements are required or should be implemented for this particular development.

Appendix A

Demands

Town of Aurora

Design Criteria Manual of Engineering Plans, June 2021 (unless otherwise stated)

Equivalent Population by Unit

Type of Development	Equivalent Population Density *(Person/Unit)*
Single Family Homes/Semi-Detached	3.8
Townhouses	3.5
Apartments	2.5

Water Design Factors

Average Daily Demand (litres/capita/day)	400
Maximum Daily Demand P.F.	2.00
Peak Hourly Demand P.F.	3.00

Coefficient of Roughness

Size of Pipe (mm Dia.)	Coefficient of Roughness (C)
150	100
200-250	110
300	120
400-450	130
Over 600	140

Minimum Pipe Size

Type of Development	Size of Pipe (mm Dia.)
Residential	200
Industrial/Commercial/Institutional	250

(For cul-de-sacs only, a 150mm watermain may be permitted at the discretion of the Town.)

Working Pressures

Parameter	Pressure
Normal Condition	
Minimum Pressure (Maximum Day)	275 kPa (40 psi)
Normal Operating Pressure (Maximum Day)	350 kPa to 620 kPa (50 to 90 psi)
Maximum (Building Code)	550 kPa (80 psi)
Maximum recommended	700 kPa (100 psi)
Fire Flow Conditions	
Minimum Pressure	140 kPa (20 psi)

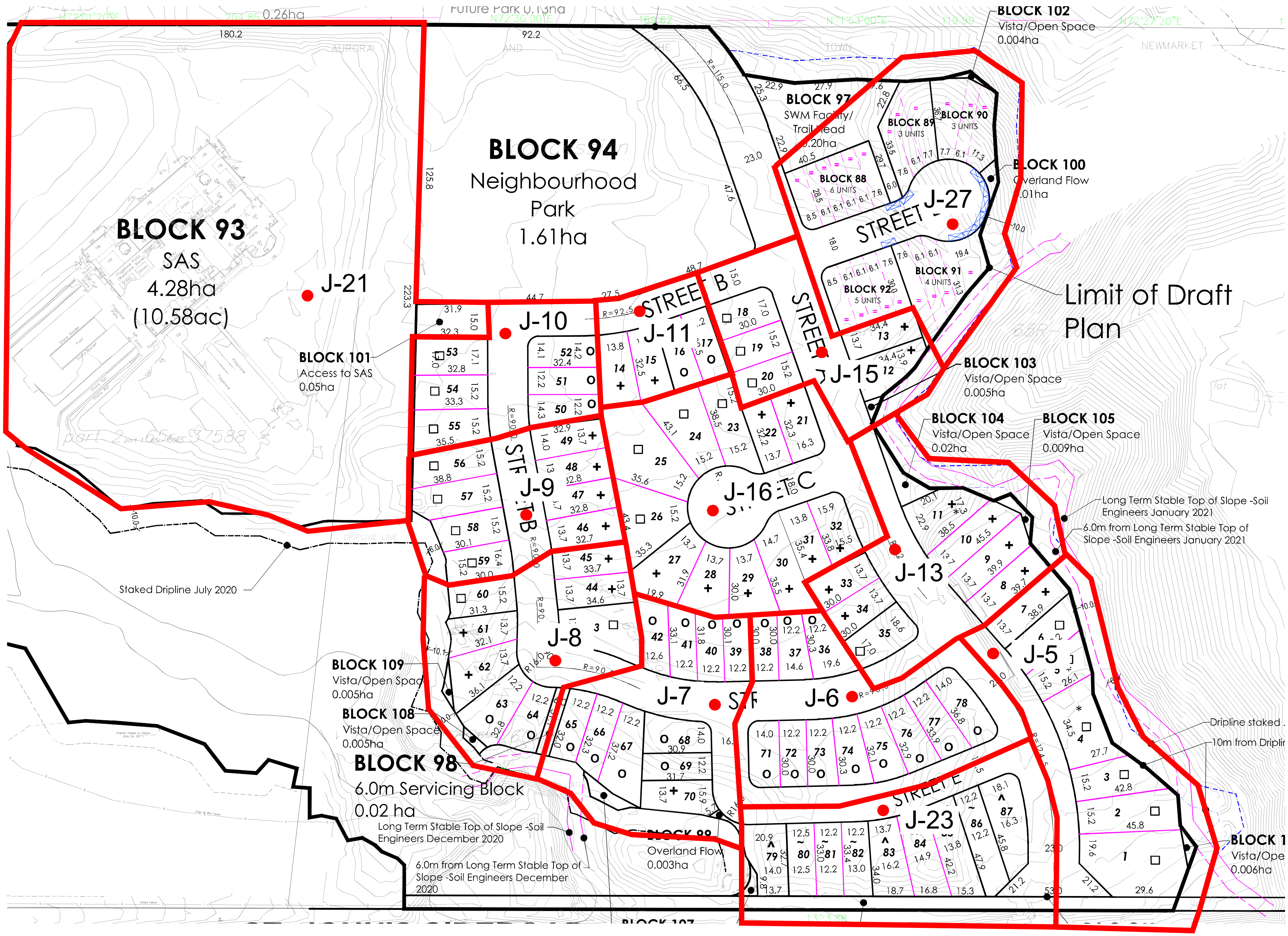
Fire Flow Demands

Type of Development	Fire Demand (L/s)
Single Family/Semi-Detached	117
Townhouse/Row House	125
Apartment	150
Commercial	200
Institutional/Industrial	250

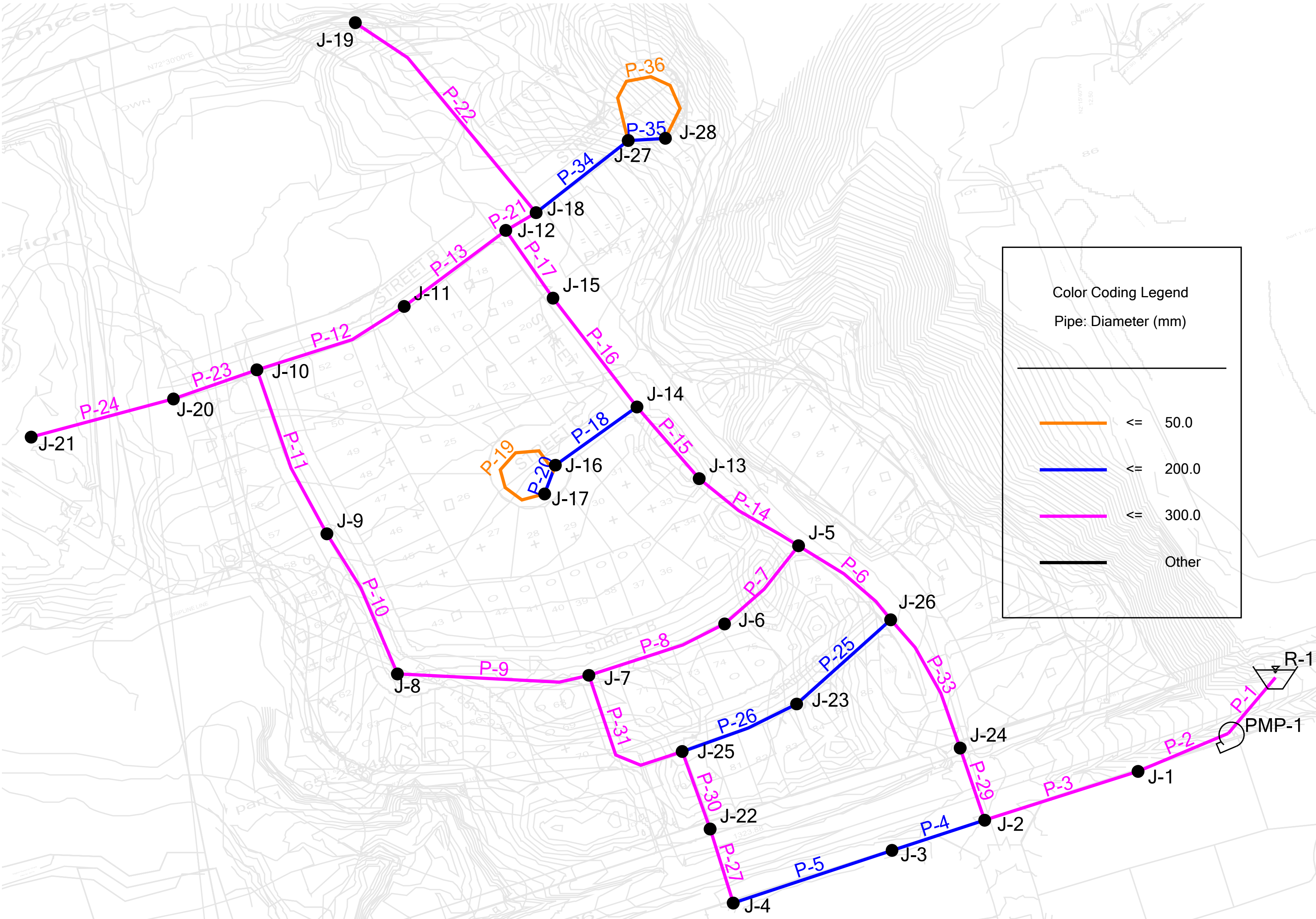
Water Demands
Shining Hill Phase 3 Development, Aurora
November 2021

Node	Type of Development			Equivalent Population		Demands		
	Detached	Townhouse	Institutional	Total Population	Total Population	Avg Day	Max Day	Peak Hour
	(units)	*(units)*	*(people)*	*(Residential)*	*(ICI)*	*(L/s)*	*(L/s)*	*(L/s)*
J-5	*7*			27	0	0.12	0.24	0.36
J-6	*11*			42	0	0.19	0.38	0.57
J-7	*10*			38	0	0.18	0.36	0.54
J-8	*8*			30	0	0.14	0.28	0.42
J-9	*8*			30	0	0.14	0.28	0.42
J-10	*6*			23	0	0.11	0.22	0.33
J-11	*4*			15	0	0.07	0.14	0.21
J-13	*7*			27	0	0.12	0.24	0.36
J-15	*5*			19	0	0.09	0.18	0.27
J-16	*12*			46	0	0.21	0.42	0.63
J-21			*800*	0	800	3.70	7.41	11.11
J-23	*9*			34	0	0.16	0.32	0.48
J-27		*21*		74	0	0.34	0.68	1.02
TOTAL	**87**	**21**	**800**	**404**	**800**	**5.57**	**11.15**	**16.72**

Demand Allocation
Shining Hill Phase 3



Pipe and Node IDs



Appendix B

Boundary Information

HYDRANT INSPECTION & FLOW REPORT

SUGGESTED NFPA RATING	
BLUE	CLASS AA
6582 gpm	@ 20 psi (138 kPa)

ONTARIO CLEAN WATER AGENCY
AGENCE ONTARIENNE DES EAUX

Prepared By: The Ontario Clean Water Agency
Prepared For: Bazil Developments Attn Paul Bailey
Test Hydrant Andrew Cruickshank
Flow Hydrant(s) Sergio Mailhos, Cody Flatt

Date: 28-Apr-21 Time: 8:47 AM

HYDRANT DESCRIPTION

Hydrant ID:	5070-07	Side of Street:	SOUTH	Make:	Concord	Open Dir:	Left
Address:	St John's Sdrd - East of Willow Farm Lane			Model:	D-67M	Latitude:	
Location:	AURORA ONTARIO			Year:	1988	Longitude:	

GENERAL INSPECTION

OK - Good Condition *FR - Future Repair Required* *N/A - Not Applicable* *CF - Component Failure*

Upper Section	OK	FR	N/A	CF
Bonnet	☐	☐	☑	☐
Operating Nut	☐	☐	☑	☐
Gaskets / Bolts	☐	☐	☑	☐
O-Ring(s)	☐	☐	☑	☐

Mid Section	OK	FR	N/A	CF
Port Height	☐	☐	☑	☐
Caps / Nozzles	☐	☐	☑	☐
Chains	☐	☐	☑	☐
Traffic Flange	☐	☐	☑	☐

General	OK	FR	N/A	CF
Accessibility	☐	☐	☑	☐
Position / Height	☐	☐	☑	☐
Paint Cond	☐	☐	☑	☐
Drain Ports	☐	☐	☑	☐

Hydrostatic Leak Testing		
Hydrant Closed	Above Grade Leak	N/A
	Subsurface Leak	N/A
Hydrant Open	Above Grade Leak	N/A
	Subsurface Leak	N/A

Maintenance	
Lubricate Operating Nut	N/A
Lubricate & Clean Nozzle Threads	N/A
Lubricate & Clean Cap Threads	N/A
Water Removed (if non-draining)	N/A

Auxiliary / Secondary Valve	
Located / Accessible	N/A
Operated/Exercised	N/A
Number of Turns	N/A
Open Direction	

Comments:

Auxiliary Valve Location:

FLUSHING

*If hydrants are being flow tested, inspections and flushing are completed prior to testing

Hydrant Operated	Clear Flow Obtained	Cl2 Residual	Time Flushed	Flow	Total Flow	Dechlorinated
Yes - Easily Operated	Yes	N/A	5 minutes	3407 gal	17033 gal	Yes

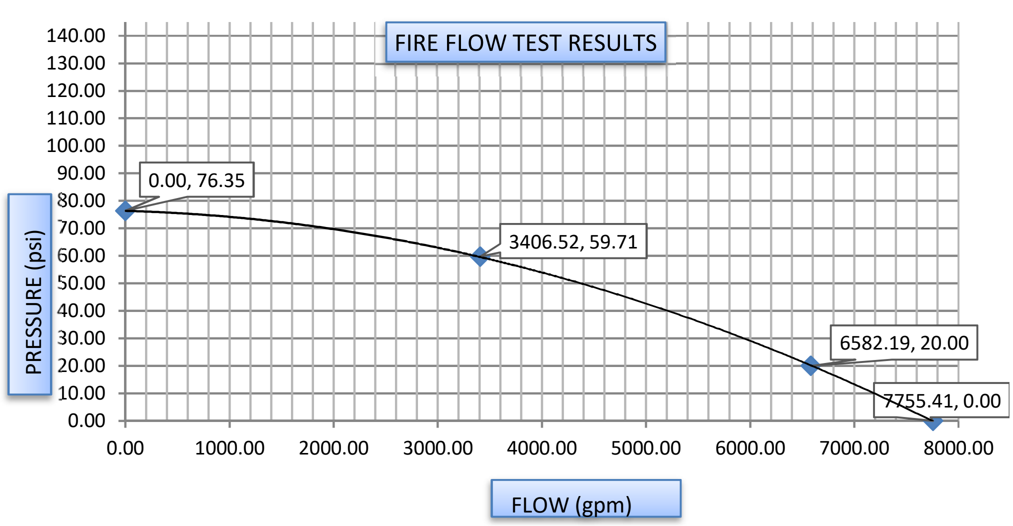
Comments: **STATIC AFTER FLOW TEST WAS PERFORMED 75.87 PSI**

FLOW TESTING

*Flow testing results may be from previous year(s). Note date & time

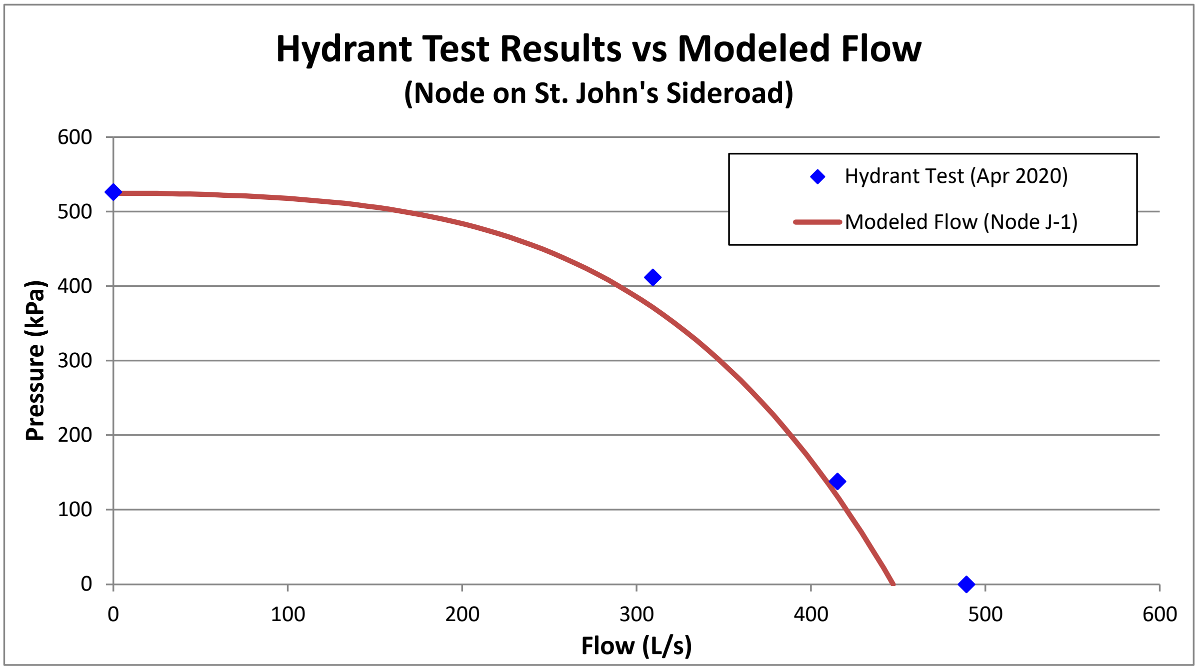
Date: 28-Apr-21 Time: 8:47 AM

Flow Hydrant								Test Hydrant		
ID	Flow Device Used	Size	Coefficient	Time Flushed	Flow	Total Flow	Pitot	ID	Static	Residual
5070-14	Pollard Diffuser	2.5"	0.832	5.0 minutes	981 gal	4905 gal	40 psi	5070-07	76.35	59.71
5070-14	Pollard Diffuser	2.5"	0.832	5.0 minutes	956 gal	4781 gal	38 psi			
5070-10	Pollard Diffuser	2.5"	0.832	3.0 minutes	776 gal	2327 gal	25 psi			
5070-10	Pollard Diffuser	2.5"	0.832	3.0 minutes	694 gal	2081 gal	20 psi			



Calculated Results	
Calculated Flow @ 20 psi	**6582 gpm**
Calculated Flow @ 0 psi	**7755 gpm**
Pressure Drop	**21.79%**

Comments:



	Static Pressure	Residual Pressure	Test Flow	Theoretical Flow at 140 kPa
	(kPa)	(kPa)	(L/s)	(L/s)
Hydrant Test	526.4	411.7	309.5	415.3
Model	524.6	407.9	283.9	410.1

Appendix C

Model Results

Results
Shining Hill Phase 3 Development, Aurora
November 2021

Municipal Engineering Solutions

Average Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (kPa)
J-1	0.00	257.40	311.00	524.55
J-2	0.00	262.34	310.99	476.18
J-3	0.00	262.80	310.99	471.67
J-4	0.00	262.60	310.99	473.61
J-5	0.12	266.35	310.99	436.90
J-6	0.19	266.44	310.99	436.02
J-7	0.18	265.92	310.99	441.11
J-8	0.14	265.59	310.99	444.33
J-9	0.14	268.50	310.99	415.85
J-10	0.11	269.50	310.99	406.06
J-11	0.07	269.00	310.99	410.95
J-12	0.00	268.47	310.99	416.14
J-13	0.12	266.95	310.99	431.03
J-14	0.00	267.50	310.99	425.64
J-15	0.09	268.00	310.99	420.74
J-16	0.21	267.70	310.99	423.68
J-17	0.00	267.70	310.99	423.68
J-18	0.00	268.47	310.99	416.14
J-19	0.00	267.90	310.99	421.72
J-20	0.00	270.00	310.99	401.16
J-21	3.70	270.00	310.99	401.15
J-22	0.00	263.88	310.99	461.08
J-23	0.16	265.42	310.99	446.01
J-24	0.00	262.14	310.99	478.13
J-25	0.00	264.20	310.99	457.95
J-26	0.00	263.95	310.99	460.40
J-27	0.34	268.47	310.99	416.14
J-28	0.00	268.47	310.99	416.14

MIN		**257.40**		**401.15**
MAX		**270.00**		**524.55**

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
P-1	R-1	PMP-1	32.47	300	120	5.57	0.08
P-2	PMP-1	J-1	43.48	300	120	5.57	0.08
P-3	J-1	J-2	71.56	300	120	5.57	0.08
P-4	J-2	J-3	43.33	200	110	1.34	0.04
P-5	J-3	J-4	74.34	200	110	1.34	0.04
P-6	J-26	J-5	52.99	300	120	3.52	0.05
P-7	J-5	J-6	48.01	300	120	0.94	0.01
P-8	J-6	J-7	64.68	300	120	0.75	0.01
P-9	J-7	J-8	85.51	300	120	2.46	0.03
P-10	J-8	J-9	70.02	300	120	2.32	0.03
P-11	J-9	J-10	79.49	300	120	2.18	0.03
P-12	J-10	J-11	71.85	300	120	-1.63	0.02
P-13	J-11	J-12	56.46	300	120	-1.70	0.02
P-14	J-5	J-13	53.46	300	120	2.46	0.03
P-15	J-13	J-14	42.21	300	120	2.34	0.03
P-16	J-14	J-15	61.09	300	120	2.13	0.03
P-17	J-15	J-12	36.66	300	120	2.04	0.03
P-18	J-14	J-16	44.53	200	110	0.21	0.01
P-19	J-16	J-17	62.06	50	100	0.00	0.00
P-20	J-17	J-16	13.67	200	110	0.00	0.00
P-21	J-12	J-18	15.60	300	120	0.34	0.00
P-22	J-18	J-19	117.54	300	120	0.00	0.00
P-23	J-10	J-20	39.25	300	120	3.70	0.05
P-24	J-20	J-21	65.47	300	120	3.70	0.05
P-25	J-23	J-26	56.14	200	110	-0.71	0.02
P-26	J-25	J-23	55.20	200	110	-0.55	0.02
P-27	J-4	J-22	34.42	300	120	1.34	0.02
P-29	J-2	J-24	33.88	300	120	4.23	0.06
P-30	J-22	J-25	36.66	300	120	1.34	0.02
P-31	J-25	J-7	68.79	300	120	1.89	0.03
P-33	J-24	J-26	65.59	300	120	4.23	0.06
P-34	J-18	J-27	52.03	200	110	0.34	0.01
P-35	J-27	J-28	16.54	200	110	0.00	0.00
P-36	J-28	J-27	74.31	50	100	0.00	0.00

Municipal Engineering Solutions

Maximum Day

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (kPa)
J-1	0.00	257.40	310.99	524.48
J-2	0.00	262.34	310.98	476.04
J-3	0.00	262.80	310.98	471.51
J-4	0.00	262.60	310.97	473.41
J-5	0.24	266.35	310.97	436.69
J-6	0.38	266.44	310.97	435.81
J-7	0.36	265.92	310.97	440.90
J-8	0.28	265.59	310.97	444.10
J-9	0.28	268.50	310.97	415.61
J-10	0.22	269.50	310.96	405.80
J-11	0.14	269.00	310.97	410.71
J-12	0.00	268.47	310.97	415.90
J-13	0.24	266.95	310.97	430.81
J-14	0.00	267.50	310.97	425.41
J-15	0.18	268.00	310.97	420.51
J-16	0.42	267.70	310.97	423.46
J-17	0.00	267.70	310.97	423.46
J-18	0.00	268.47	310.97	415.90
J-19	0.00	267.90	310.97	421.48
J-20	0.00	270.00	310.96	400.89
J-21	7.41	270.00	310.96	400.85
J-22	0.00	263.88	310.97	460.88
J-23	0.32	265.42	310.97	445.81
J-24	0.00	262.14	310.98	477.97
J-25	0.00	264.20	310.97	457.74
J-26	0.00	263.95	310.97	460.21
J-27	0.68	268.47	310.97	415.90
J-28	0.00	268.47	310.97	415.90
MIN		**257.40**		**400.85**
MAX		**270.00**		**524.48**

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
P-1	R-1	PMP-1	32.47	300	120	11.15	0.16
P-2	PMP-1	J-1	43.48	300	120	11.15	0.16
P-3	J-1	J-2	71.56	300	120	11.15	0.16
P-4	J-2	J-3	43.33	200	110	2.69	0.09
P-5	J-3	J-4	74.34	200	110	2.69	0.09
P-6	J-26	J-5	52.99	300	120	7.05	0.10
P-7	J-5	J-6	48.01	300	120	1.88	0.03
P-8	J-6	J-7	64.68	300	120	1.50	0.02
P-9	J-7	J-8	85.51	300	120	4.92	0.07
P-10	J-8	J-9	70.02	300	120	4.64	0.07
P-11	J-9	J-10	79.49	300	120	4.36	0.06
P-12	J-10	J-11	71.85	300	120	-3.27	0.05
P-13	J-11	J-12	56.46	300	120	-3.41	0.05
P-14	J-5	J-13	53.46	300	120	4.93	0.07
P-15	J-13	J-14	42.21	300	120	4.69	0.07
P-16	J-14	J-15	61.09	300	120	4.27	0.06
P-17	J-15	J-12	36.66	300	120	4.09	0.06
P-18	J-14	J-16	44.53	200	110	0.42	0.01
P-19	J-16	J-17	62.06	50	100	0.01	0.00
P-20	J-17	J-16	13.67	200	110	0.01	0.00
P-21	J-12	J-18	15.60	300	120	0.68	0.01
P-22	J-18	J-19	117.54	300	120	0.00	0.00
P-23	J-10	J-20	39.25	300	120	7.41	0.10
P-24	J-20	J-21	65.47	300	120	7.41	0.10
P-25	J-23	J-26	56.14	200	110	-1.41	0.05
P-26	J-25	J-23	55.20	200	110	-1.09	0.03
P-27	J-4	J-22	34.42	300	120	2.69	0.04
P-29	J-2	J-24	33.88	300	120	8.46	0.12
P-30	J-22	J-25	36.66	300	120	2.69	0.04
P-31	J-25	J-7	68.79	300	120	3.78	0.05
P-33	J-24	J-26	65.59	300	120	8.46	0.12
P-34	J-18	J-27	52.03	200	110	0.68	0.02
P-35	J-27	J-28	16.54	200	110	0.01	0.00
P-36	J-28	J-27	74.31	50	100	0.01	0.00

Municipal Engineering Solutions

Peak Hour

Node Table

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (kPa)
J-1	0.00	257.40	310.98	524.37
J-2	0.00	262.34	310.96	475.83
J-3	0.00	262.80	310.95	471.26
J-4	0.00	262.60	310.94	473.10
J-5	0.36	266.35	310.94	436.37
J-6	0.57	266.44	310.94	435.49
J-7	0.54	265.92	310.94	440.57
J-8	0.42	265.59	310.93	443.75
J-9	0.42	268.50	310.93	415.23
J-10	0.33	269.50	310.92	405.41
J-11	0.21	269.00	310.93	410.32
J-12	0.00	268.47	310.93	415.53
J-13	0.36	266.95	310.93	430.47
J-14	0.00	267.50	310.93	425.06
J-15	0.27	268.00	310.93	420.14
J-16	0.63	267.70	310.93	423.10
J-17	0.00	267.70	310.93	423.10
J-18	0.00	268.47	310.93	415.53
J-19	0.00	267.90	310.93	421.10
J-20	0.00	270.00	310.92	400.47
J-21	11.11	270.00	310.91	400.39
J-22	0.00	263.88	310.94	460.57
J-23	0.48	265.42	310.94	445.50
J-24	0.00	262.14	310.95	477.74
J-25	0.00	264.20	310.94	457.43
J-26	0.00	263.95	310.94	459.92
J-27	1.02	268.47	310.93	415.52
J-28	0.00	268.47	310.93	415.52

MIN		**257.40**		**400.39**
MAX		**270.00**		**524.37**

Pipe Table

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness (C)	Flow (L/s)	Velocity (m/s)
P-1	R-1	PMP-1	32.47	300	120	16.72	0.24
P-2	PMP-1	J-1	43.48	300	120	16.72	0.24
P-3	J-1	J-2	71.56	300	120	16.72	0.24
P-4	J-2	J-3	43.33	200	110	4.03	0.13
P-5	J-3	J-4	74.34	200	110	4.03	0.13
P-6	J-26	J-5	52.99	300	120	10.57	0.15
P-7	J-5	J-6	48.01	300	120	2.82	0.04
P-8	J-6	J-7	64.68	300	120	2.25	0.03
P-9	J-7	J-8	85.51	300	120	7.38	0.10
P-10	J-8	J-9	70.02	300	120	6.96	0.10
P-11	J-9	J-10	79.49	300	120	6.54	0.09
P-12	J-10	J-11	71.85	300	120	-4.90	0.07
P-13	J-11	J-12	56.46	300	120	-5.11	0.07
P-14	J-5	J-13	53.46	300	120	7.39	0.10
P-15	J-13	J-14	42.21	300	120	7.03	0.10
P-16	J-14	J-15	61.09	300	120	6.40	0.09
P-17	J-15	J-12	36.66	300	120	6.13	0.09
P-18	J-14	J-16	44.53	200	110	0.63	0.02
P-19	J-16	J-17	62.06	50	100	0.01	0.00
P-20	J-17	J-16	13.67	200	110	0.01	0.00
P-21	J-12	J-18	15.60	300	120	1.02	0.01
P-22	J-18	J-19	117.54	300	120	0.00	0.00
P-23	J-10	J-20	39.25	300	120	11.11	0.16
P-24	J-20	J-21	65.47	300	120	11.11	0.16
P-25	J-23	J-26	56.14	200	110	-2.12	0.07
P-26	J-25	J-23	55.20	200	110	-1.64	0.05
P-27	J-4	J-22	34.42	300	120	4.03	0.06
P-29	J-2	J-24	33.88	300	120	12.69	0.18
P-30	J-22	J-25	36.66	300	120	4.03	0.06
P-31	J-25	J-7	68.79	300	120	5.67	0.08
P-33	J-24	J-26	65.59	300	120	12.69	0.18
P-34	J-18	J-27	52.03	200	110	1.02	0.03
P-35	J-27	J-28	16.54	200	110	0.01	0.00
P-36	J-28	J-27	74.31	50	100	0.01	0.00

Results
Shining Hill Phase 3 Development, Aurora
November 2021

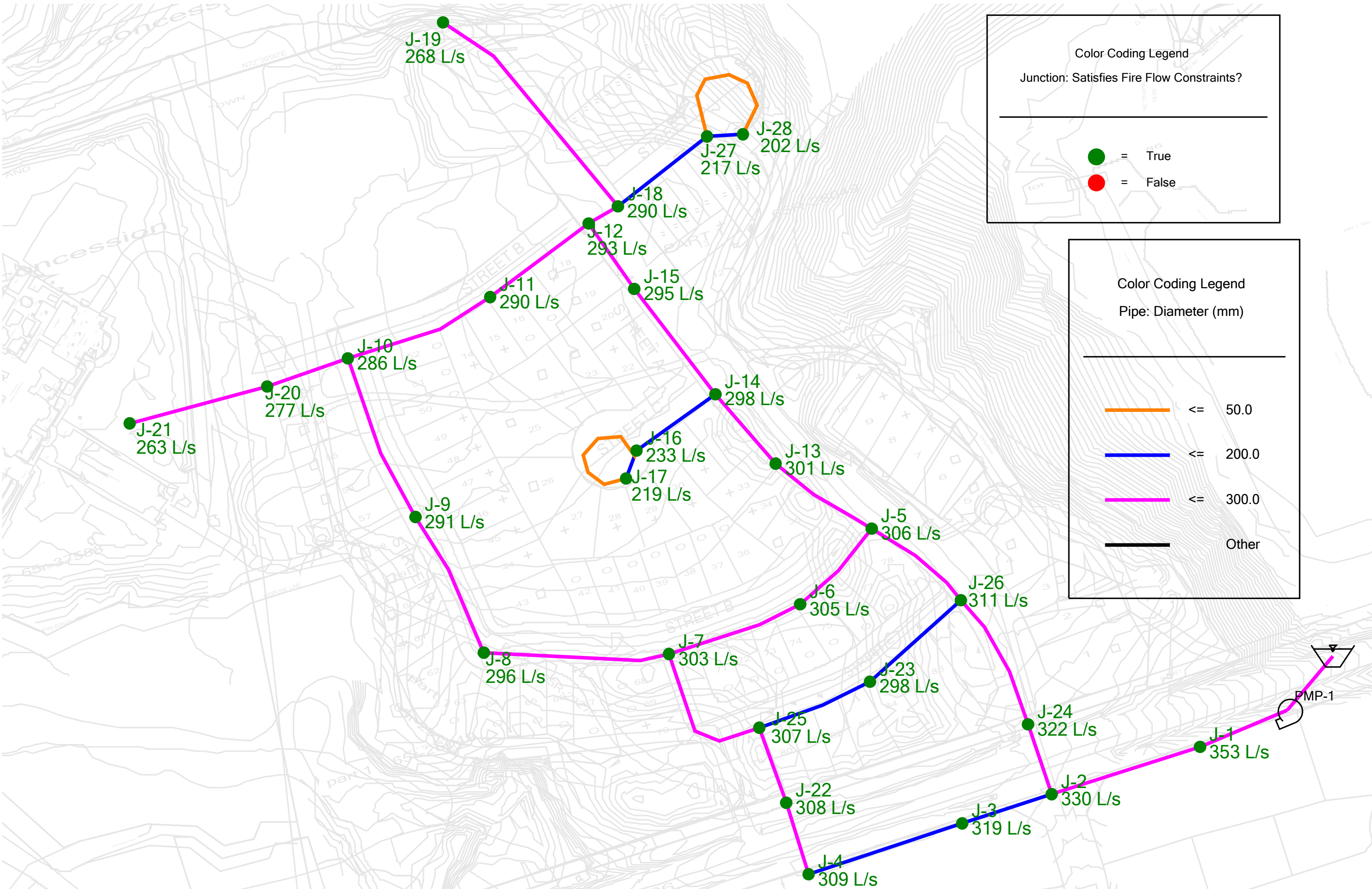
Municipal Engineering Solutions

Fire Flow Table					
ID	Fire Flow Demand (L/s)	Total Demand (L/s)	Total Available Flow (L/s)	Available Fire Flow (L/s)	**Fire Flow Met?**
J-1	117.00	117.00	352.80	352.80	TRUE
J-2	117.00	117.00	329.87	329.87	TRUE
J-3	117.00	117.00	318.87	318.87	TRUE
J-4	117.00	117.00	308.88	308.88	TRUE
J-5	117.00	117.24	305.76	305.52	TRUE
J-6	117.00	117.38	305.34	304.96	TRUE
J-7	117.00	117.36	303.82	303.46	TRUE
J-8	117.00	117.28	296.49	296.21	TRUE
J-9	117.00	117.28	291.49	291.21	TRUE
J-10	117.00	117.22	286.03	285.81	TRUE
J-11	117.00	117.14	290.03	289.89	TRUE
J-12	117.00	117.00	292.94	292.94	TRUE
J-13	117.00	117.24	301.24	301.00	TRUE
J-14	117.00	117.00	298.31	298.31	TRUE
J-15	117.00	117.18	295.10	294.92	TRUE
J-16	117.00	117.42	233.74	233.32	TRUE
J-17	117.00	117.00	218.89	218.89	TRUE
J-18	117.00	117.00	290.27	290.27	TRUE
J-19	117.00	117.00	268.36	268.36	TRUE
J-20	250.00	250.00	276.70	276.70	TRUE
J-21	250.00	257.41	270.09	262.68	TRUE
J-22	117.00	117.00	307.84	307.84	TRUE
J-23	117.00	117.32	297.95	297.63	TRUE
J-24	117.00	117.00	322.35	322.35	TRUE
J-25	117.00	117.00	306.63	306.63	TRUE
J-26	117.00	117.00	311.40	311.40	TRUE
J-27	125.00	125.68	217.81	217.13	TRUE
J-28	125.00	125.00	202.26	202.26	TRUE

MIN	**202.26**
MAX	**352.80**

Scenario: Maximum Day

Available Fire Flow



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