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# Beachwood Development

## Beachwood Developments Inc.

## Functional Servicing & Stormwater Management Report

**September 2020**

**The Jones Consulting Group Ltd.**
**4-229 Mapleview Drive East, Barrie ON L4N 0W5**

**ROM-17026 (70)**

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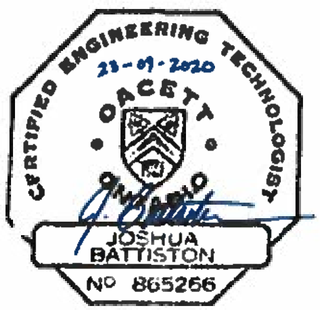


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# Functional Servicing & Stormwater Management Report

Beachwood Developments Inc. – Beachwood Development
Town of Wasaga Beach

# 1. Introduction

## 1.1. Appointment

The Jones Consulting Group Ltd. (TJCG) was retained by Beachwood Developments Inc. (Client) to provide engineering services for a proposed residential subdivision development located south of Shore Lane and north of Beachwood Road in the Town of Wasaga Beach (Town). The subdivision is to be known as the Beachwood Development.

This *Functional Servicing & Stormwater Management Report* (FSR/SWMR) has been prepared in support of the proposed *Draft Plan of Vacant Land Condominium* prepared by TJCG, dated July 27, 2020, and demonstrates how the lands will be serviced by the surrounding municipal infrastructure.

In particular, this report examines the property's servicing in relation to:

- Stormwater Servicing
- Stormwater Management
- Water Servicing
- Sanitary Servicing
- Transportation
- Site Grading
- Utility Servicing

## 1.2. Property Description

The land presents an irregular shape and comprises of approximately 5.88 hectares (ha) of land. The subject property is legally described as Part of Lot 34, Concession 3, in the Town of Wasaga Beach, County of Simcoe. A copy of the subject lands proposed proposed *Draft Plan of Vacant Land Condominium* prepared by The Jones Consulting Group, dated July 27, 2020, has been attached in **Appendix D**.

The site is situated south of Shore Lane, north of Beachwood Road and west of 74th Street North. It is bound to the north by existing residential lots and undeveloped land, to the west and east by existing residential lots, to the south by Beachwood Road. The location of the subject property is shown in **Figure 1**.

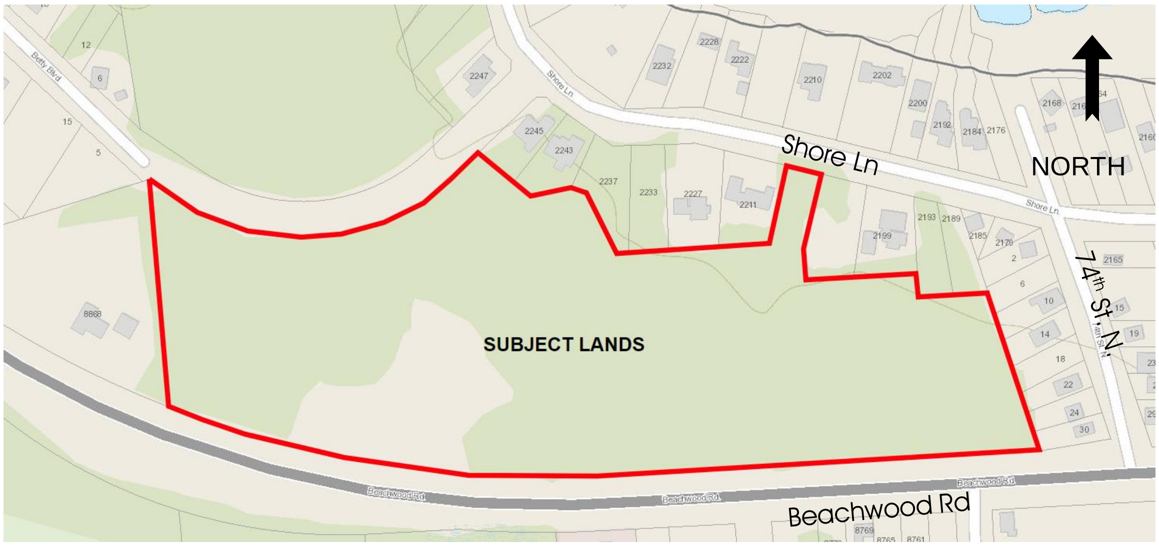



**Figure 1 - Site Location**

The site topography ranges in elevation from a maximum elevation of 184 meters at the southwest corner of the site adjacent to Beachwood Road to a minimum elevation of 178 meters at northern most point of the site adjacent to Shore Lane. The site generally drains from south to north towards Shore Lane. Existing slopes range from 0.7% to 7%.

The soils encountered on the subject lands are predominately an Eastport Sand (Est). These underlying soils belong to *Group A* of the SCS Hydrological Soils Group Classification System. Soil Series are determined from the *Soil Map of Simcoe County*, *Report No. 29* of the *Ontario Soil Survey* produced by the Canadian Department of Agriculture with the Ontario Department of Agriculture. Further background geotechnical and hydrogeological investigations have been completed by Cambium Inc., dated February 14, 2020 and June 18, 2020 respectively.

## 1.3. Existing and Proposed Land Use

The lands are comprised of undeveloped land and forested areas. There are currently no dwellings on the site. Lands from the south drain through the subject lands towards Nottawasaga Bay, forming part of Georgian Bay and Lake Huron. Refer to the *West End Drainage Study, Town* *of Wasaga Beach* prepared by Ainley Group, dated August 2019, with excerpts provided in **Appendix D** for existing and proposed external drainage details.

The Beachwood Development proposed the development of 82 units based on single family residential units and street townhomes units, as well as two (2) high density blocks consisting of 134 units.

The property is currently zoned primarily as Residential with portions of the property designated for Low and High Density. Low Density is reserved for singles ranging in standard frontage size from 9.0m, 10.0m, 12.0m with frontages up to 32.0m in special cases, as well as townhouse blocks with unit frontages of 6.0m.

There are also land allocations provided for Stormwater Management, Parkland and Municipal Drainage. For further detail, refer to the *Proposed Draft Plan of Subdivision* prepared by TJCG, dated July 27, 2020 which has been included in **Appendix D**.

## 1.4. Supporting Documents

The following documents have been referenced in the preparation of this report:

- Town of Wasaga Beach Engineering Guidelines, March 2015;
- Nottawasaga Valley Conservation Authority, Stormwater Technical Guide, 2013;
- Credit Valley Conservation Authority & Toronto Region Conservation Authority, Low Impact Development Stormwater Management Planning and Design Guide, 2010;
- Ministry of the Environment, Stormwater Management Planning and Design Manual, March 2003;
- Ministry of Transportation, Drainage Management Manual, February 2008;
- Ministry of the Environment, Design Guidelines for Drinking-Water Systems, 2008;
- Soil Survey of Simcoe County, Report 29 of the Ontario Soil Survey;
- Simcoe County online Mapping;
- Nottawasaga Valley Conservation Authority online Mapping;
- West End Drainage Study, Town of Wasaga Beach, prepared by Ainley Group, August 2019;
- Geotechnical Investigation – Part Lot 34 & 35, Concession 3, Wasaga Beach, Ontario, prepared by Cambium Inc., February 14, 2020.
- Hydrogeological Assessment Report – Shore Lane Development, Wasaga Beach, Ontario, prepared by Cambium Inc., June 18, 2020.

## 1.5. Overview of Municipal Infrastructure

### 1.5.1. *Stormwater Servicing*

Currently, there are no existing storm sewers or existing stormwater management facilities on the subject lands. The lands to the north of the site have existing drainage facilities but provide no stormwater management controls for the subject site. As such, on-site all proposed storm sewer systems and a stormwater management facility are designed to control and convey drainage prior to discharge to the existing downstream infrastructure.

The proposed storm sewer system and stormwater management facilities will be designed and constructed in accordance with the latest standards prepared by the Town of Wasaga Beach, Nottawasaga Valley Conservation Authority (NVCA), and Ministry of Environment and Conservation and Parks (MOE). In addition to traditional end-of-pipe SWM facilities, Low Impact Development (LID) controls are proposed throughout the site as a means to meet Runoff Volume Control Targets (RVCT) and provide additional quality control through infiltration and filtration of stormwater runoff.

All flows from the proposed stormwater management facility will be discharge to Betty Boulevard; minor flows connecting to the proposed Storm Sewer system and emergency overflows to the roadside ditch. Furthermore, there are two drainage easements, otherwise referred to as the west and east easements, which convey external flows from lands located to the south of Beachwood Road as well as some uncontrolled internal catchments, through the subject lands to the extension of Betty Boulevard and Shore Lane, respectively.

### 1.5.2. *Water Servicing*

According to the *Chapman Property, Shore Lane Plan and Profile Drawing*, prepared by WMI & Associates Ltd., (Drawing P1, included in **Appendix D**), there is an existing 300mm diameter watermain on the north side of Shore Lane. According to the Town of Wasaga Beach's *Record Drawing of Highway No. 26 (Beachwood Road)* prepared by Ainley Group (Drawing 105120-SW10-RD Sta. 2+000 to Sta. 2+275 included in **Appendix D**), there is an existing 300mm diameter watermain located in the south boulevard of Beachwood Road. Connections are proposed to the existing watermain on Shore Lane, which will allow for the development to be adequately serviced with the required domestic and fire flows, and a second connection to the existing watermain within Betty Boulevard to create a looped system and a second feed to the development.

The Town of Wasaga beach will confirm through their consultant that adequate capacity exists within the existing and proposed (water tower and works yard to the south or Beachwood Boulevard) water supply network. Refer to Section 3 of this report for further design details.

### 1.5.3. *Sanitary Servicing*

According to the *Chapman Property, Shore Lane Plan and Profile Drawing*, prepared by WMI & Associates Ltd., (Drawing P1, included in **Appendix D**), there is an existing 450mm diameter sanitary sewer within Shore Lane. According to the Town of Wasaga Beach's *Record Drawing of Highway No. 26 (Beachwood Road)* prepared by Ainley Group (Drawing 105120-SW10-RD Sta. 2+000 to Sta. 2+275 included in **Appendix D**), there is an existing 250mm diameter sanitary sewer located in the south boulevard of Beachwood Road.

Flows generated from the site will be collected by the internal sanitary collection system and will discharge out between Lot 1 and 2199 Shore Lane via a 6.0m servicing easement, connecting to the existing 450mm dia. sanitary sewer on Shore Lane.

The Town of Wasaga beach has confirmed that adequate capacity exists within the existing sanitary sewer network with planned upgrades at SP1. Refer to Section 4 of this report for further design details.

### 1.5.4. *Road Network*

The subject property is generally bound by Beachwood Road to the south, existing residential to the east (74$^{th}$ Street North), north (Shore Lane) and west (8884 Beachwood Road). The proposed internal road network consists of private paved roadways with full movement stop controlled connections to Beachwood Road (mid-frontage) and the future extension of Betty Boulevard (northwest frontage).

# 2. Stormwater Management

## 2.1. Introduction & Stormwater Control Criteria

The stormwater management plan is intended to provide an environmentally sound approach to stormwater and drainage issues. The issues can be divided generally into five categories: runoff volume control, stormwater quality control, stormwater quantity control, water balance, and erosion & sediment control.

This report outlines a proposed design for the Beachwood Development's stormwater management system to meet the compulsory post-development quantity and quality control requirements of the Town, NVCA, and MECP.

The Town of Wasaga Beach and the NVCA require Enhanced (Level 1) quality control for the stormwater runoff generated from this site. The proposed downstream Stormwater Management Facility (Sand Filter with Forebay) has been sized to provide Level 1 quality control, quantity control up to the 100-year post to pre-development flows and the extended detention (48 hour drawdown) for the 25mm storm event for erosion control. Per the NVCA *Stormwater Technical Guide*, there is an additional requirement to capture and retain the first 5 mm of each and every rainfall event on-site. Consequently, the Beachwood Development is proposed to implement at-source LID's to meet this runoff volume control target, which will also assist in achieving established quality and quantity control targets and site-specific water balance.

The design of the LID's is outlined in **Section 2.5** in this Report, and the design of the SWMF is outlined in **Sections 2.7.1** & **2.7.2**.

## 2.2. Existing Drainage Conditions

The subject lands consist of approximately 5.88 hectares of land which is primarily unused forested areas. The land's topography contains slopes ranging from 0.7 to 7% with an average slope of 2.0%, generally draining from south to north.

There are significant external areas south of Beachwood Road draining towards the subject lands. The *West End Drainage Study*, prepared by Ainley Group, dated August 2019 provided details of the existing off-site drainage conditions as well as those of the site. A review of the existing site topography confirms that the two (2) catchments identified by Ainley are generally consistent with their modelling and routing identified in the August 2019 report.

There are no existing stormwater management facilities on the subject lands and all flows are released uncontrolled to the Shore Lane, future development lands to the north and Beachwood Road's existing drainage systems. The existing conditions are identified as on drawing **SWM-1** included in **Appendix C** and on Ainley Group drawings **118038-DR1**, **DRAFT-1** and **118038-DR1** included in **Appendix D**.

## 2.3. Drainage Background

Drainage in the area of, and upstream of the subject lands generally flows from south to north, eventually reaching Georgian Bay. However, over the past two decades the areas built form has changed, primarily with the construction of New Highway 26.

The new 4-lane divided Highway 26 was built in early-mid 2000 to divert traffic travelling between Stayner/Wasaga Beach and Collingwood while allowing local traffic to remain on highway 26-former, renamed Beachwood Road.

The construction of new highway 26 included the installation of thirty plus (30+) new culverts upstream of the new roadway. Culverts were designed in accordance with the MTO Drainage Design Standard WC-7 (Jan. 2008) and SD-13 (Jan 2008) which stipulate culvert capacity and free board depth at the new roadway, resulting in backwater effect upstream of the culverts.

The subject, Beachwood Development lands fall within the drainage boundaries of Culverts 11 and 12 (Ainley: culverts #16 & #17, respectively).

Subsequent to the construction of new Highway 26, the Town of Wasaga Beach in January 2017 completed the West End Water Storage Facility and Maintenance Depot Class Environmental Assessment which concluded that the lands located between current Hwy 26 and Beachwood Road, south of the proposed Beachwood Development was the preferred location for the Town's new water storage and maintenance depot.

During the site selection, public concerns were expressed regarding on going drainage issues in the area. As such, Ainley Associates, on behalf of the Town, prepared and issued the August 2019 West End Drainage Study report. The report's recommendations include the construction of a wet stormwater management retention facility upstream of Beachwood Road to provide quality and quantity controls, and channelization of flow to Beachwood Road (culvert #16) and within the lands north of Beachwood Rd., discharging to Georgian Bay via two existing outlets, as drainage paths north or Beachwood Rd. are poorly defined.

The proposed Beachwood Development application makes provision for conveyance of Culvert #17 flows through the site to the extension of Betty Boulevard.

The report also recommends flows from upstream of Culvert #16, located east of Joan Ave be controlled through orifice controlled oversized storm pipes before discharging through the Beachwood Developments lands to Shore Lane and ultimately to Georgian Bay by the second of two existing culvert.

The proposed Beachwood Development application makes provision for conveyance of Culvert #16 flows through the site to Shore Lane.

## 2.4. Proposed Drainage Conditions

Development of the subject lands will consist of low density residential lots, low density townhouse lots, a Parkland block, a Municipal 6.0m Drainage block, a Stormwater Management block, two (2) high density apartment building blocks and an internal road network. The proposed internal storm sewer system will connect to the proposed stormwater management facility which ultimately discharges to the future Betty Boulevard storm sewer system.

The proposed lots and blocks will be developed with buildings, driveways, parking lots, and landscaped areas. The grading of the lots will direct stormwater runoff to the internal road network, which will contain a proposed storm sewer system to convey minor flow (up to the 5 Year event) to the proposed end-of-pipe Stormwater Management Facilities (SWMF). The internal roads will also be utilized to convey the major overland flow (>5 Year event) within the right-of-way to the proposed SWMF. The proposed end-of-pipe SWMF located along the north boundary of the site is a sand filter complete with a forebay and underdrain system, aiming to provide extended detention and flood control, along with quantity and quality control functions.

The proposed drainage system will also include a suite of lot level Low Impact Development (LID) source and conveyance control measures to provide controls that reduce peak flows and runoff volumes, provide upstream treatment for contaminants, promote groundwater recharge and assist in maintaining water balance to pre-development levels. LIDs proposed for the site include rear yard soakaway pits and rooftop storage on the high-density buildings.

Two drainage easements are provided to convey flows from external lands to the south, further details on the easements and conveyance infrastructure proposed are summarized in **Section 2.12.**

The storm sewer sub-catchment plan drawing, referenced as drawing **STM-1**, is provided in **Appendix C** and the storm sewer design sheet in **Appendix A.**

The Post-Development Storm Drainage Plan, referenced as drawing **SWM-2**, is included in **Appendix C** for reference.

## 2.5. Best Management Practices

### 2.5.1. *Lot Level Controls*

Lot-level LID measures, elsewhere referred to as the Rear Yard Soakaways, are proposed as a subsurface detention and infiltration system within the rear yards of all single detached and townhouse type units. The soakaways will promote infiltration of collected runoff and reduce the overall effective imperviousness of the site. These facilities will be sized to capture and infiltrate the full runoff volume from the 25 mm storm generated from half of the roof area of each dwelling, with full facility drawdown occurring within 24 hours. The infiltration rate as estimated through review of soil mapping was between 70 and 120 mm/hr on average, and a factored infiltration rate of 48 mm/hr will be used in the detailed design.

### 2.5.2. *End-of-Pipe Controls*

The proposed end-of-pipe SWMF located in the northwest corner of the site is programmed as a sand filter complete with a forebay in accordance with Section 4.9 of the MECP *Stormwater Management Planning and Design Manual, 2003*. The aim of this facility is to provide extended detention and flood control, along with quantity and quality control functions.

Further details on the SWMF are provided in **Section 2.7.1.** and **2.7.2.**

### 2.5.3. *Enhanced Grass Swales*

All proposed swales within the site are designed as water quality swales to promote infiltration of a portion of the captured runoff. The aim of this facility is to provide extended detention and flood control, along with quantity and quality control functions.

Detailed sizing calculations for each of the respective swales will be provided during the detailed design process.

### 2.5.4. *Additional Low Impact Development Practices*

It is recommended that all catchments should incorporate an increased depth of absorbent topsoil at least 300mm thick to promote at-source infiltration on pervious surfaces on lots. It is further recommended that any absorbent topsoil be amended with organic content (compost) as recommended in the *CVC Low Impact Development Design Guidelines* while scarifying subsoils and remaining as unconsolidated as reasonably possible to maintain void spaces.

A study conducted in BC has asserted reductions in runoff volume and peak flows up to 50% from the placement of 300mm of absorbent landscaping (*British Columbia Ministry of Land, Water and Air Protection, May 2002*). Another study conducted in Ontario through the *Sustainable Technologies Evaluation Program (STEP)* has confirmed similar findings with a reduction in runoff of up to 27% (*STEP, Residential Lot Level SWM Practices, 2013*).

Other recommendations include downspout disconnection (where not connected to a soakaway), where roof leaders are directed away from impervious surfaces. Additionally, it is recommended to incorporate rain barrels at the lot level where possible, to further reduce runoff volumes to downstream systems. The proposed design has accounted no credit for the aforementioned best management practices.

## 2.6. Hydrology

### 2.6.1. *Model*

The development was hydrologically modeled using the latest version of the PCSWMM Professional computer program by Computational Hydraulics Int. PCSWMM is a GIS-based hydrologic model capable of performing both event-based and continuous rainfall simulations for SWM Facility, LID design water balance & erosion threshold calculations, respectively. Furthermore, the PCSWMM model utilizes the Green-Ampt Method for determining infiltration losses, which allows for the direct incorporation of field-tested infiltration rates. Once the field-tested infiltration rates are obtained through geotechnical investigation, the model can be further advanced at the detailed design stage to incorporate the Guelph Permeameter field testing needed for LID design.

The PCSWMM model used for this design is developed based on the existing site topography and proposed site grading designs. The model input parameters (e.g. catchment area, flow lengths, proposed imperviousness) have been updated to reflect the proposed subdivision design.

The hydrologic modeling includes both pre-development and post-development conditions to established target flow rates ($m^3/s$) for each outlet for various return period events.

### 2.6.2. *Design Storms & Climatology*

The rainfall events used for the PCSWMM model simulations are based on the requirements of the Town of Wasaga Beach, NVCA and MECP, which include the SCS Type II and Chicago storm distributions. The following events have been modeled:

- 4-hour Chicago rainfall distribution for the 2 through 100-year storm events;
- 6, 12 & 24-hour SCS Type II rainfall distributions for the 2 through 100-year storm events;
- 25 mm 4-hour Chicago rainfall event; and
- Timmins Regional Storm Event.

### 2.6.3. *Soil Type and Land Use*

The *Soil Survey of Simcoe County Report No. 29* shows that the lands are comprised of Eastport Sand soils. This soil is well draining and is mostly sand and occurs in the form of dune. The profile is grey calcareous sand. The soils in this area are classified as Group A under the SCS Hydrologic Soils Group Classification. The underlying topography is rough and the slopes are short and steep.

### 2.6.4. *Geotechnical Investigation*

The site soils were investigated under a Geotechnical Investigation completed by Cambium Inc., dated February 14, 2020. The investigation included a total of seven (7) boreholes with monitoring wells. Site soils were determined to be topsoil overlying predominantly interbedded sand, silty sand, silt and sand and silty clay.

Grain size analyses were conducted on collected samples and estimated the coefficient of permeability for the subject lands at depth of 2.3m to 2.7m below original ground ranged from $10^{-1}$ to $10^{-3}$ cm/s, which are classified as pervious soils. Samples in the location of proposed SWMF's were in the typical range of $10^{-1}$ to $10^{-3}$ cm/s, indicating soils are pervious in this location.

A Groundwater Monitoring Wells were installed on four (4) of the boreholes. Measurements were taken monthly from December 2019 to May 2020 to better understand the groundwater regime for the subject lands. Findings from the monitoring data indicates a groundwater in each monitoring well throughout the monitoring period. The monthly groundwater measurement program is on-going. Proposed grading for the site has been developed to maintain minimum separation required from underside of structures, and as a result will require an average fill across the subject lands of approximately 1.4m.

### 2.6.5. *Discretization*

The hydrologic modeling includes both the pre-development and post-development conditions to established target flow rates ($m^3/s$) for the site.

The pre-development PCSWMM model was derived from a review of topographic survey data and previous studies. As flows generally travel from south to north, the site is split roughly into two halves: Catchment A104-R is a total of 3.39 ha and represents the west half of the development while Catchment A103-R is a total of 2.49 ha and represents the east half of the development. Each catchment area was assigned its own outlet as these flows are directed to different outlets to Georgian Bay.

The post-development PCSWMM model was derived from the grading for the proposed subdivision design. All areas that drain to the SWMF, including the SWMF itself, are included in Catchment 301, a total of 4.01 ha. Uncontrolled catchment areas have been added which generally consists of rear-yards that cannot be graded to drain to the pond. Catchments 302 and 303 are uncontrolled areas that total 0.76 ha and drain to the west outlet at the future Betty Boulevard extension via a series of dedicated swales and channels. Catchments 304 and 305 are uncontrolled areas that total 1.11 ha and drain to the east outlet at Shore Lane via a series of dedicated swales and channels.

The imperviousness of the subdivision catchment areas has been set at 66% to account for a variety of uses within the proposed development blocks. The higher imperviousness is conservative to ensure that the SWM pond block is adequately sized for a traditional end-of-pipe SWM facility which excludes the benefits to quality and quantity control that may be realized from the implementation of upstream LIDs.

The pre-development and post-development catchment areas are illustrated on **Drawings SWM-1** and **SWM-2** respectively and are provided in **Appendix C**. The PCSWMM model schematic is provided in **Appendix A** along with the model output.

**Table 1** summarizes the hydrologic parameters that were used for the PCSWMM model.

**Table 1 - Summary of Hydrologic Model Inputs**

| Area ID | Area (ha) | Flow Length (m) | Width (m) | Slope (%) | Zero Imperv. (%) | Imperv. (%) | DS Perv. (mm) | DS Imp. (mm) | Routing | Soils |
|---|---|---|---|---|---|---|---|---|---|---|
| **Pre-Development Model** | | | | | | | | | | |
| A104-R | 3.39 | 227 | 147 | 2.0 | 3 | 10 | 8 | 2 | Outlet | Sandy Loam |
| A103-R | 2.49 | 240 | 103 | 2.0 | 4.2 | 14 | 8 | 2 | Outlet | Sandy Loam |
| **Post-Development Model** | | | | | | | | | | |
| 301 | 4.01 | 200 | 200 | 2.0 | 0 | 66 | 5 | 2 | 46% Perv. | Sandy Loam |
| 302 | 0.25 | 20 | 123 | 2.0 | 0 | 63 | 5 | 2 | 84% Perv. | Sandy Loam |
| 303 | 0.51 | 96 | 53 | 2.0 | 0 | 9 | 5 | 2 | 100% Perv. | Sandy Loam |
| 304 | 0.67 | 90 | 74 | 2.0 | 0 | 19 | 5 | 2 | 100% Perv. | Sandy Loam |
| 305 | 0.45 | 30 | 148 | 2.0 | 0 | 4 | 5 | 2 | Outlet | Sandy Loam |

The infiltration parameters used in the model and summarized in **Table 1** reflect those referenced in the Geotechnical Investigation. Infiltration parameters will be revisited and updated for the detailed design upon completion of additional geotechnical investigations and testing.

Some of the PCSWMM model input parameters (not shown in **Table 1** above) are common among all catchment areas, including the following:

Manning's N for impervious / pervious area = 0.013 / 0.15

Depth of depression storage on impervious / pervious area (mm) = 2 / 5 or 8

### 2.6.6. *Model Results*

The hydrologic modeling results are summarized herein. The pre-development condition was modeled within PCSWMM to establish the allowable rates for the site. The allowable post-development rates and the resulting post-development peak flow rates are summarized in **Table 2** as computed by the PCSWMM model. The PCSWMM model schematics along with the output printout for the 100-year SCS 24-hour storm are provided in **Appendix A**. The model output printouts are provided in PDF format for all storm events on a USB provided with the digital modeling files appended at the back of the report.

**Table 2 - Hydrologic Modelling Results Summary: Pre-Development**

| Storm Peak Event Flow ($m^3/s$) | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Area (ha) | Storm Distribution | | | | | |
| **Return Period (years)** | | **2** | **5** | **10** | **25** | **50** | **100** |
| **Pre-Development Condition – Peak Flow Total (OF1 – West)** | | | | | | | |
| **Chicago - 4 hour** | 3.39 | 0.109 | 0.144 | 0.172 | 0.218 | 0.260 | 0.307 |
| **SCS Type II - 6 hour** | 3.39 | 0.065 | 0.086 | 0.102 | 0.153 | 0.208 | 0.272 |
| **SCS Type II - 12 hour** | 3.39 | 0.068 | 0.089 | 0.124 | 0.206 | 0.279 | 0.356 |
| **SCS Type II - 24 hour** | 3.39 | 0.070 | 0.110 | 0.203 | 0.316 | 0.388 | 0.466 |
| **25mm Storm** | 3.39 | 0.084 | | | | | |
| **Timmins Storm** | 3.39 | 0.173 | | | | | |
| **Pre-Development Condition – Peak Flow Total (OF2 – East)** | | | | | | | |
| **Chicago - 4 hour** | 2.49 | 0.111 | 0.148 | 0.175 | 0.218 | 0.255 | 0.295 |
| **SCS Type II - 6 hour** | 2.49 | 0.067 | 0.089 | 0.105 | 0.136 | 0.177 | 0.225 |
| **SCS Type II - 12 hour** | 2.49 | 0.070 | 0.092 | 0.116 | 0.174 | 0.228 | 0.286 |
| **SCS Type II - 24 hour** | 2.49 | 0.072 | 0.103 | 0.168 | 0.252 | 0.306 | 0.364 |
| **25mm Storm** | 2.49 | 0.085 | | | | | |
| **Timmins Storm** | 2.49 | 0.136 | | | | | |

**Table 3 - Hydrologic Modelling Results Summary: Post-Development**

| Storm Peak Event Flow ($m^3/s$) | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Area (ha) | Storm Distribution | | | | | |
| **Return Period (years)** | | **2** | **5** | **10** | **25** | **50** | **100** |
| **Post Development Condition – Total Peak Flow (OF1 – West)** | | | | | | | |
| **Chicago - 4 hour** | 4.77 | 0.039 | 0.062 | 0.082 | 0.112 | 0.135 | 0.161 |
| **SCS Type II - 6 hour** | 4.77 | 0.035 | 0.062 | 0.086 | 0.130 | 0.193 | 0.262 |
| **SCS Type II - 12 hour** | 4.77 | 0.040 | 0.076 | 0.113 | 0.212 | 0.284 | 0.362 |
| **SCS Type II - 24 hour** | 4.77 | 0.052 | 0.100 | 0.179 | 0.283 | 0.359 | 0.427 |
| **25mm Storm** | 4.77 | 0.022 | | | | | |
| **Timmins Storm** | 4.77 | 0.367 | | | | | |
| **Post Development Condition – Total Peak Flow (OF2 – East)** | | | | | | | |
| **Chicago - 4 hour** | 1.11 | 0.006 | 0.017 | 0.041 | 0.078 | 0.112 | 0.150 |
| **SCS Type II - 6 hour** | 1.11 | 0.005 | 0.042 | 0.082 | 0.136 | 0.178 | 0.219 |
| **SCS Type II - 12 hour** | 1.11 | 0.011 | 0.067 | 0.116 | 0.179 | 0.224 | 0.270 |
| **SCS Type II - 24 hour** | 1.11 | 0.027 | 0.121 | 0.175 | 0.232 | 0.270 | 0.308 |
| **25mm Storm** | 1.11 | 0.005 | | | | | |
| **Timmins Storm** | 1.11 | 0.091 | | | | | |

The results demonstrate that the proposed stormwater management facility will provides the required quantity control to reduce post-development peak flows to the respective allowable release rates for the West Outlet. Additional quantity controls are required for frequent return period events within the uncontrolled catchments 304 and 305 that drain to the East Outlet. Details of the additional controls will be provided at the detailed design stage. The function of the proposed SWMF is discussed in further detail in **Section 2.7.1.** and **2.7.2.**

## 2.7. Proposed Stormwater Management Plan

### 2.7.1. *Quantity Control*

As per the Town and NVCA requirements, *Enhanced* (Level 1) stormwater quality control is required for the site. The proposed sand filter will provide the required level of SWM quality control, as the calculations provided below will demonstrate. In addition to the typical quality control requirements, the incorporation of Low Impact Development (LID) measures will provide phosphorus removal benefits to ensure that best efforts are taken to reduce post-development phosphorus loadings to pre-development levels (or a minimum 80% removal of TP from post-development). Phosphorus Loading calculations are provided within **Section 2.9**.

A plan view and section drawing has been provided for the SWMF. Refer to **Drawings PND-1** and **PND-2** in **Appendix C**.

**<u>Sand Filter Sizing</u>**

The total land draining to the SWMF includes Catchment 301, totaling an area of 4.08 hectares. The total imperviousness used to size the pond is 60% given that catchment 301 is an internal area comprised of a combination of single detached, townhouses and two high density blocks. Based on the MOE requirements (extrapolation of Table 3.2), 31.7 $m^3$/ha of storage is required for enhanced quality control. Furthermore, the required extended detention volume is set at 40 $m^3$/ha to promote good drawdown characteristics.

The extended detention and water quality volumes, as per the MOE guidelines, are calculated as follows:

Enhanced (Level 1) Water Quality Protection

Total area draining to pond for quality control = 4.01 ha

(66% Net Impervious Area), (33.7 $m^3$/ha)

4.01 ha * 33.7 $m^3$/ha = 135 $m^3$ (water quality storage in filter)

4.01 ha * 40 $m^3$/ha = 160 $m^3$ (extended detention)

However, the extended detention volume must be designed to attenuate the erosion volume from the 4-hour 25 mm Chicago rainfall event. Based on the watershed characteristics and the PCSWMM model, the 25 mm storm event produces a runoff volume of 334 $m^3$ (8.34 mm * 4.01 ha). Therefore, the runoff volume from the 4-hour 25 mm Chicago storm event will govern.

The water quality storage is provided by the filter media in the sand filter as follows:

| Layer | Thickness | Void Ratio | Equivalent 1D Thickness |
|---|---|---|---|
| Sand Layer | 0.50 m | 0.30 | 0.15 m |
| Stone Layer | 0.50 m | 0.40 | 0.20 m |
| **Total** | | | **0.35 m** |

**Filter Surface Area** = 541 $m^2$

**Filter Volume** = Area * 1D Equivalent Thickness

= 541 $m^2$ x 0.35 m

= **189.4 $m^3$**

Extended detention volume is provided as a combination of storage within the filter media and active storage above the surface of the filter.

| | Required: | Provided: |
|---|---|---|
| Water Quality | 160 $m^3$ | 189 $m^3$ |
| Extended Detention | 334 $m^3$ | 657 $m^3$ |

The 24 hour extended detention release rate is the maximum target flow rate to ensure that 24 hour settling occurs in the facility. The extended detention outlet is 80 mm in diameter. The drawdown equation has been used to verify the detention time for the extended detention volume. The equation is given as follows:

$$t = \frac{0.66 C_2 h \sqrt{h} + 2 C_3 \sqrt{h}}{2.75 A_0}$$

Where:

t = drawdown time (seconds)

$C_2$ = slope coefficient from the area-depth linear regression (650.8)

$C_3$ = intercept from the area-depth linear regression (773.1)

$A_0$ = cross-sectional area of the extended detention orifice (0.0050 $m^2$)

h = maximum water elevation above the orifice, taken from the maximum extended detention elevation to centroid of orifice (0.46 m)

Given the above information, the actual extended detention storage time was determined to be approximately 23.8 hours, which approaches the MOE minimum criteria. This drawdown time does not account for the attenuation of flows through the filter media which will extend the drawdown time of the facility. When accounting for the flux through the filter media, the drawdown time is estimated to be 47.2 hours, as extracted from the PCSWMM output for the SWMF *Volume vs. Time* plot included in **Appendix A.** The area-depth curve can be found in **Appendix A** to illustrate how the above coefficients were determined.

**Forebay Sizing**

The forebay must be sized to provide sufficient length from the inlet to the forebay weir. It is recommended that the forebay be sized according to the length required for settlement of larger suspended particles. The forebay for the pond was designed according to the following criteria (MOE, 2003):

**Forebay Settling Length**

$$Dist = \sqrt{\frac{rQ_p}{V_s}}$$

Where:

Dist = the minimum forebay length (m)

r = the length to width ratio based on the dimensions at the permanent pool elevation 180.15 m (2.23:1)

$Q_p$ = the peak flow rate exiting the pond during the 4 hour 25 mm Chicago quality storm event (0.022 $m^3/s$)

$V_s$ = the settling velocity. It is recommended that a value of 0.0003 m/s be used in most cases.

The required settling lengths of the forebay is 12.8 metres.

**Forebay Dispersion Length**

$$Dist = \frac{8Q}{dV_f}$$

Where:

Dist = the minimum forebay length (m)

$Q$ = the 5-year peak inlet flow capacity (0.594 $m^3/s$)

$d$ = the depth of the permanent pool in the forebay (1.0 m)

$V_f$ = the desired velocity at forebay berm (0.5 m/s)

The required dispersion length of the forebay is 9.5 metres.

The proposed forebay accommodates both the settling and dispersion lengths. The shortest length is approximately 21.0 m measured from the inlet headwall to the face of the forebay overflow weir.

### 2.7.2. *Quantity Control*

The Stormwater Management Facility is located at the northwestern corner of the Beachwood Development, immediately east of the future Betty Boulevard extension and south of the existing residences fronting onto Shore Lane. The SWMF has been designed in accordance with the Town, NVCA, and MOE guidelines as a sand filter with a traditional wet forebay for pre-treatment.

The bottom elevation of the SWMF's forebay is 179.15 m, the surface elevation of the filter cell is 180.15 m, and the top of pond elevation is 181.71 m at the outer limits of the access road. This provides a total depth of active storage of 1.56 m in the main cell and forebay, and a surface depth equal to 0.35 m in the media filter cell. Sideslopes of 3H:1V are proposed throughout the filter cell and wet forebay, meanwhile 3H:1V slopes are proposed everywhere else in the facility. The backsloping of the pond is proposed to match existing grades within the site boundaries.

An allowance for a 3.0 m wide maintenance access road is provided around the entire perimeter of the SWMF. A total of 1,990 $m^3$ of active storage has been provided with no permanent pool or credit for the void storage in the sand filter. No ramp is provided within the forebay due to its overall geometry and proximity to the maintenance access road. During maintenance, the forebay can be drained by pumping to the control maintenance hole. A 0.15 m high flow spreader has been provided bordering the forebay and filter cell with 3H:1V sideslopes proposed throughout.

A sediment drying area will be provided for the temporary storage of sediment during forebay maintenance in the southwest corner of the SWMF block. The sediment drying area will be bermed and graded towards the forebay cell at positive slope. The sediment drying area will be appropriately sized in accordance with the MOE SWM Planning & Design Manual guidelines.

Minor events are collected through a system of conventional storm sewers and discharge through an inlet into the facility. Flows are then directed through the forebay to the filter cell which is complete with system of 100mm dia. underdrains connecting to a 250 mm dia header and draining to the outlet control structure at the west end of the facility. The sand filter is contained within an impermeable liner, therefore the flow rate through the filter is conservatively assumed to be constant as described by *Equation 4.20* and in *Section 4.9.6.* of the *Stormwater Management Planning and Design Manual, 2003.* The Sand Filter discharge is calculated based on the filter geometry, layer thicknesses and the proposed soils, as shown below.

**Sand Filter Discharge**

$$Q \quad = \quad f \times \left(\frac{P}{3,600,000}\right) \times (LW \times n)$$

Where:

f = longevity factor (per Table 4.12)

P = Percolation Rate for Sand (120 mm/h)

L = length of the filter (A = 541 $m^2$)

W = width of the filter (A = 541 $m^2$)

N = void ratio for media filter soils (0.30)

As such, the flow rate out of the media filter is calculated as 0.005 $m^3/s$, regardless of the available head on the filter.

Major overland flows are directed into the SWMF through an overland flow route directly into the forebay of the SWMF, acting as a protective measure for the sand filter.

The primary control for the SWMF is a reverse-sloped 450 mm diameter storm pipe directed to a control maintenance hole complete with a 80 mm diameter orifice plate, bolted to the inside of the control structure, with an invert set to the bottom of main cell elevation at 180.15 m. The primary outlet pipe will discharge via a double ditch inlet catchbasin manhole (DDICBMH), with its invert located at the surface of the sand filter. A secondary outlet will discharge via the same DDICBMH connected by a 450 mm diameter storm pipe to the control maintenance hole, with the pipe serving as the orifice inside of the control structure at an invert of 180.65 m.

A 10.0 m wide emergency overflow spill weir is incorporated at an elevation of 181.35 m to ensure safe conveyance of peak flows to the future Betty Boulevard extension during the Regional (Hazel) storm event. A detail of the overflow weir and overflow weir outfall channel are provided on **Drawing PND-1** and **PND-2.**

**Table 4 - SWMF Facility Design Summary**

| Key Elevations Table – SWMF | |
|---|---|
| **Location** | **Elev. (m)** |
| Bottom of Forebay | 179.15 |
| Underdrain Outlet | 179.20 |
| Sand Filter Surface | 180.15 |
| Top of Flow Spreader | 180.30 |
| Primary Outlet - 80mm dia. Orifice | 180.15 |
| Secondary Outlet - 475mm dia. Orifice | 180.65 |
| Emergency Overflow Weir - 30.0m Wide | 181.35 |
| Top of Pond (Access Road - Exterior) | 181.71 |

Based on the results of the *PF-Model,* an abbreviated version of the SWMF1 stage-storage-discharge table is presented in **Table 5** below. The full table is provided in **Appendix A**. The values for maximum required storage and pond outflow result from the proposed geometric configuration of the facility, design height and dimensions for the orifices and overflow weir. The *SCS 24-hour* storm distribution governs the maximum outflow and storage volume requirements. Refer to **Drawings PND-1** to **PND-2** in **Appendix C** for details. Refer to the PCSWMM Pond results in **Appendix A.**

**Table 5 - SWMF Performance Summary**

| Stage Storage Discharge Table – SWMF | | | | | |
|---|---|---|---|---|---|
| **Return Period (Years)** | **$Q_{PEAK}$ ($m^3/s$)** | **Total Storage Vol. ($m^3$)** | **Total Depth (m)** | **Active Depth (m)** | **Water Level (m)** |
| **Bottom of Forebay** | 0.000 | 0 | 0.00 | 0.00 | 179.15 |
| **Surface of Filter** | 0.005 | 189 | 1.00 | 0.00 | 180.15 |
| **25mm** | 0.007 | 304 | 1.00 | 0.00 | 180.15 |
| **2** | 0.017 | 835 | 1.54 | 0.54 | 180.69 |
| **5** | 0.091 | 1,064 | 1.73 | 0.73 | 180.88 |
| **10** | 0.155 | 1,204 | 1.84 | 0.84 | 180.99 |
| **25** | 0.240 | 1,385 | 1.98 | 0.98 | 181.13 |
| **50** | 0.285 | 1,539 | 2.09 | 1.09 | 181.24 |
| **Regional** | 0.309 | 1,693 | 2.16 | 1.16 | 181.31 |
| **100** | 0.326 | 1,702 | 2.20 | 1.20 | 181.35 |
| **Top (Interior)** | 2.895 | 1,915 | 2.45 | 1.45 | 181.65 |
| **Top (Exterior)** | 3.818 | 1,990 | 2.51 | 1.51 | 181.71 |

*Storage Volume includes 189.4 $m^3$ in filter.

As shown in the table above, the minimum freeboard of 0.30 m is provided under the Regional Storm event.

The results present above demonstrate that the proposed stormwater management facility will provide the required quantity control to reduce post-development peak flows to the allowable release rates for their respective areas.

## 2.8. Volume Control

Runoff Volume control is provided by a combination of the LID facilities as described in **Section 2.5**. Refer to the above mentioned sections for the corresponding details for the respective facilities.

The NVCA *Stormwater Technical Guidelines* provide guidance on volumetric control, setting an initial target of a volume capture equal to a depth of 5 mm over the total developable area of the site. However, a best efforts approach was undertaken for this site, given the underlying soils and proximity to the groundwater table as outlined in **Section 2.4.** Furthermore, the Town of Wasaga Beach prefers lot-level practices be implemented in an effort to minimize the associated ongoing operations and maintenance costs. Summarized below in **Table 6** are the volumetric control targets for the site, as well as the anticipated overall performance of the rear yard soakaways.

**Table 6 – Runoff Volume Control: Targets & Performance (25mm Event)**

| LID Performance: Area 6 - 25mm Event | | |
|---|---|---|
| **Description** | **Total Contributing Area (ha)** | **Infiltration Volume ($m^3$)** |
| **Runoff Volume Control Target (5mm)** | **5.88** | **294** |
| Soakaway Pits – Rooftops | 1.30 | *299.5 |
| High Density Blocks | 0.22 | **11.0 |
| **Best Efforts Sub-Totals** | | **310.5** |

* Infiltration Volume for Singles & Semis calculated as the runoff volume from the 25mm storm (23.04mm per PCSWMM) over the rooftop areas
** Infiltration Volume for High Density Blocks calculated as 5 mm of runoff volume over the entire roof area for each building

The results demonstrate that the proposed LID practices will achieve a volume capture of 310.5 $m^3$, corresponding to an equivalent depth of **5.28 mm over the total developable area**, is captured and infiltrated, achieving the $RVC_T$ volume of 292 $m^3$ as outlined in **Table 6.**

The results provided herein demonstrate that the proposed LID facilities will provide an adequate best efforts approach to achieving the required $RVC_T$.

## 2.9. Nutrient Management

As required by current NVCA policy, a pre to post development phosphorous balance has been undertaken for the site. The analysis has been completed using the Phosphorous Budget Tool created by Hutchison Environmental Sciences Ltd., and Stoneleigh Associates Inc. Members of The Jones Consulting Group Ltd. have been trained by Hutchinson Environmental Sciences Ltd. to complete a phosphorous analysis within the NVCA watershed using the Phosphorous Budget. Supporting software print-outs are provided in **Appendix A**.

The analysis revealed that in the pre development condition the total phosphorous load exported off site was determined to be 0.35 kg per year. In the post development condition without the considerations of BMP's, the total phosphorous exported off site is 2.62 kg per year. This corresponds to an increase in phosphorus export, but 'Best Management' practices were still applied to help sustain watershed health. With the consideration of the BMP's quality and quantity control features identified in previous sections, the phosphorous export off site will be reduced further to 0.99 kg per year. This amount is a reduction of 62% of the post development phosphorous export without BMP's; accordingly, further work will be done to limit the phosphorous exports post-development to achieve a balance for the site.

## 2.10. Water Balance

The primary objective of the NVCA's water balance target is to capture and manage annual rainfall on the development site to preserve pre-development hydrology (water balance) through a combination of infiltration, evapotranspiration, landscaping, rainwater reuse and/or other low impact development practices.

Various site specific characteristics contribute to the ability to achieve water balance. They include, but are not limited to: soil permeability, the ability to collect and direct drainage into the ground, groundwater table elevations and seasonal fluctuations. A site-specific water balance is required, and best efforts will be made via the SWM plan to maintain groundwater recharge while considering site specific characteristics.

Water Balance is an accounting of the available water resources in a given area and can be estimated by the following general equation:

**P = ET + R + I**

Where: P = Precipitation (Annual)

ET= Evapotranspiration/evaporation

R = Surface Water Runoff

I = Infiltration

The precise measurement of water balance components is difficult to ascertain. Typically, approximations and simplifications are made after reviewing precipitation and climatic data for a given time series and the annual quantities are estimated using statistical methods.

All water balance component data has been provided by Cambium Inc. in the Hydrogeological Assessment Report, dated June 18, 2020. The Water Balance component values have been summarized below in **Table 7**.

**Table 7 - Water Balance Components**

| Water Balance Component | Depth | Units |
|---|---|---|
| **Weighted Parameters** | | |
| **Pre-Development** | | |
| Precipitation (P) | 912 | mm/yr |
| Evapotranspiration (ET) | 545 | mm/yr |
| Infiltration Factor (MECP Method) | 0.55 | |
| Actual Infiltration | 202 | mm/yr |
| P – ET (Runoff for Pervious) | 165 | mm/yr |
| P – ET (Runoff for Impervious @ 90% Surplus) | 330 | mm/yr |
| **Post-Development** | | |
| Precipitation (P) | 912 | mm/yr |
| Evapotranspiration (ET) | 545 | mm/yr |
| Infiltration Factor (MECP Method) | 0.55 | |
| Actual Infiltration | 202 | mm/yr |
| P – ET (Runoff for Pervious) | 165 | mm/yr |
| P – ET (Runoff for Impervious @ 90% Surplus) | 330 | mm/yr |

Water balance components for both Pre- and Post- Development Scenarios have been weighted appropriately with respect to soil type (HSG-A) and ground cover (Woods and Urban Lawns) as described in the Hydrogeological Assessment Report prepared by Cambium Inc.

In order to assess the impact the proposed development will have on the subject site's water budget, the pre- and post-development infiltration volumes have been calculated by Cambium Inc. for the subject lands in **Table 8**, and a summary of the surplus runoff and infiltration deficit is provided in **Table 9**. The land areas used for the analysis are based on the *Draft Plan* (5.88 ha total area of developable land). The high density blocks will provide their own water balance subject to Site Plan control.

**Table 8 - Water Balance for Pre- & Post-Development Conditions**

| Land Use Description | Approx. Land Area ($m^2$) | Estimated Impervious Area (%) | TOTAL Runoff Volume ($m^3/a$) | TOTAL Infiltration Volume ($m^3/a$) |
|---|---|---|---|---|
| Mature Forest | 58,800 | 0 | 9,711 | 11,869 |
| **Totals** | **58,800** | **0** | **9,711** | **11,869** |
| Residential | 58,800 | 60 | 31,862 | 5,050 |
| **Totals** | **58,800** | **60** | **31,862** | **5,050** |

**Table 9 - Water Balance Summary**

| Total Development Area (m) | Pre Development Infiltration ($m^3/a$) | Post Development Infiltration ($m^3/a$) | Net Change (%) | Pre-to-Post Infiltration Deficit ($m^3/a$) |
|---|---|---|---|---|
| **58,800** | **11,869** | **5,050** | -57 | **6,819** |

LID measures for Stormwater Management have been incorporated into the servicing strategy for the subject lands in an effort to offset the infiltration deficit for the post-development condition by promoting groundwater recharge.

In following the NVCA runoff volume target with a best efforts approach, subcatchment areas have been established to be routed to rear yard soakways, each of which have been adequately sized to provide a storage volume to capture the runoff from the 25 mm event over the rear portion of each rooftop. Calculations below will demonstrate how the water balance target has been approached.

| | |
|---|---|
| Typical Rooftop Runoff Volume: | 2.70 $m^3$ (90th Percentile Storm) |
| Average Annual Rain Events: | 161.5 Events/Year |
| Average Annual Rain Events (>2mm): | 80.1 Events/Year |
| LID Capture: | 5.3 mm/Event |
| Total Rooftop Area + HD Blks to LID: | 1.52 ha |
| **LID Annual Infiltration Volume:** | **4,722 $m^3$** |

The **deficit is therefore reduced to 2,097 $m^3$/a** by introducing rear yard soakaways on each single and townhouse lot as well as future infiltration based LID measures on all high density blocks. Additional LID measures will be explored during the detailed design approval process to reduce the remaining deficit. The Detailed water balance calculations are included in **Appendix A** for reference.

## 2.11. Overland Flow

All minor stormwater runoff (up to the 5-year event) will be conveyed to the proposed stormwater management facilities via a network of storm sewers. The storm sewer system has been designed for the 5-year event using the Rational Method. A design sheet has been prepared for the 5-Year Storm Sewer. Refer to **Drawings STM-1** as well as **DS-1** for further details.

The major stormwater runoff will be conveyed along the roadway and dedicated drainage channels to the proposed stormwater management facilities and/or designated outlets. All channel sizing calculations have been completed to ensure they are adequately sized to convey the regulatory event. Hydraulic Conveyance Calculations for overland flow channels in drainage blocks, weirs within the SWMF and outfall channels are provided in **Appendix A.**

## 2.12. External Flow Conveyance

Drainage from lands south of Beachwood Road are collected by a pair of culverts as described in the *West End Drainage Study,* prepared by the Ainley Group, dated August 2019. The external flows which are to be conveyed through the subject lands are directed to one of two (2) easements, elsewhere referred to as the West and East drainage easements. An excerpt from the report prepared by Ainley Group is summarizing the flows to be conveyed is as follows:

**Table 10 – External Flows Summary (Table 8 – Ainley Report)**

| Culvert | Area | Design Event | Flows ($m^3/s$) | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 2-Yr | 5-Yr | 10-Yr | 25-Yr | 50-Yr | 100-Yr | Regional |
| Culvert #16 (West) | 15.80 ha | 24h SCS | 0.291 | 0.387 | 0.445 | 0.539 | 0.645 | 0.779 | 0.835 |
| Culvert #17 (East) | 9.10 ha | 24h SCS | 0.202 | 0.267 | 0.312 | 0.399 | 0.501 | 0.665 | 0.566 |

Major stormwater runoff will be conveyed through a series of dedicated culverts and drainage channels to the proposed outlets at the future Betty Boulevard extension and Shore Lane. All flows generated from Culvert #16 will be conveyed through the west easement via an 900mm x 1800mm concrete box culvert. All flows generated from Culvert #17 will be conveyed through the east easement first via a 750mm dia. concrete storm sewer and then through a 1.0m wide flat bottom drainage channel complete with 3:1 side slopes.

All channel sizing calculations have been completed to ensure they are adequately sized to convey the regulatory event. Hydraulic Conveyance Calculations for culverts and overland flow channels in drainage easements are provided in **Appendix A.**

## 2.13. Erosion & Sediment Control

The proposed works will require an *Erosion and Sediment Control (ESC) Plan* in order to provide adequate protection of downstream receiver systems throughout construction. The proposed ESC works are to be outlined and submitted during the detailed design approval process. The general approach to managing erosion and sediment control during construction is as follows:

During construction, the majority of the development's natural features will be removed and the topsoil stripped within the development area. The exposed surface will be susceptible to erosion, increasing the potential for sediment runoff. To minimize local and downstream impacts from erosion and sedimentation during construction, the following measures have been recommended:

- Excess earth and topsoil is to be stockpiled away from the creek limits and/or removed from site. Stockpiles shall be seeded or covered with erosion control if left for periods of greater than 30 days.

- Temporary sediment control fencing should be erected around the perimeter of all grading activities, including double silt fence along the north boundary.
- Temporary sediment fabric and stone filters should be installed on catch basins until surface cover has been stabilized.
- Temporary rock flow check dams should be installed within drainage cut-off swales.
- A temporary construction access mud mat should be installed at the construction accesses to reduce the amount of materials that may be transported off site.
- Temporary erosion and sediment control basins are to be constructed, complete with a Hickenbottom outlet control structure and overflow weir. The basins' purpose is to detain runoff long enough to allow the majority of soil particles to settle out of suspension.
- Construction during drier months should be monitored for wind-borne transport of sediments. At the direction of the engineer, the contractor may be directed to water down exposed earth areas with an aqueous solution of calcium chloride.
- All disturbed areas not under immediate construction for 30 days, or not intended for building activities within a 3-month time period, should be stabilized with seeding.
- Phased removal of temporary sediment basins during building phase of the development to coincide with upstream stabilization (established vegetation) of catchment areas.
- A weekly monitoring program to ensure all ESC measures are in place and not damaged by vandalism or a significant storm event.

Through proper implementation of the detailed erosion and sediment control measures, off-site impacts are expected to be minimized during the construction phase of the project.

# 3. Water Servicing

## 3.1. External Watermain Network

External to the proposed development and traversing the south property line from 8801 Beachwood Road (J.C. Cycle), continuing eastward there is an existing 300mm diameter watermain within the southern boulevard of Beachwood Road. Along the north limits of the site there is an existing 300mm diameter watermain within the Shore Lane right-of-way. Furthermore, within the future Betty Boulevard extension (by others) right-of-way there is an existing 300mm diameter watermain.

Connections are proposed at two (2) points, Shore Lane and the future extension of Betty Boulevard, with a redundant water supply.

## 3.2. Proposed Watermain System

The development's projected design population and water supply flow rates are estimated based on the Town of Wasaga Beach *Engineering Standards* (March 2015), and MOE *Design Guidelines for Drinking Water Systems* (2008). The following design criteria have been used for calculating the person per unit (PPU) for the various types of residential lots.

| **Unit Type** | **Design Population** |
|---|---|
| Low Density Residential | 2.6 persons/unit |
| High Density Residential | 2.6 persons/unit |

Based on the above design criteria and the *Proposed Draft Plan of Subdivision* statistics, the total population for the Beachwood Development has been calculated below.

| **Unit Type** | **Design Population** |
|---|---|
| 81 Low & Medium Density Residential units x 2.6 PPU | 211 people |
| 134 High Density Residential units x 2.6 PPU | 349 people |
| **Total projected population** | **560 people** |

In order to meet the development's potable and fire water demands, it is proposed that the subject lands be connected to the existing municipal system through proposed connections to the existing system at Shore Lane and the future Betty Boulevard extension. Internally, the subject lands will be serviced with watermains of 150mm diameter, including all appropriate appurtenances in accordance with the Town of Wasaga Beach standards. Operating pressures within the water supply and distribution system must remain a minimum pressure of 50 psi. A summary of the proposed demands for the development and scenarios is provided below.

**Design Criteria:**

- *Average Daily Flow (ADD) = 350 L/d/capita (Town of Wasaga Beach Engineering Standards)*
- *Peak Hour Demand Factor (PHD) = 4.13 (MOE Drinking Water Guideline, 2008)*
- *Max. Day Demand Factor (MDD) = 2.75 (MOE Drinking Water Guideline, 2008)*
- *Min. Hour Demand Factor (MHD) = 0.40 (MOE Drinking Water Guideline, 2008)*
- *Fire Flow – Residential = 8,000 L/min (Fire Underwriters Survey, 1999)*

**Peak Hour Demand:**

**Q = Pop. x ADD x PHD**

Q = (560 persons x 350 L/d/capita x 4.13)/86,400

Q = 9.37 L/s

**Minimum Hour Demand:**

**Q = Pop. x ADD x MHD**

Q = (560 persons x 350 L/d/capita x 0.40)/86,400

Q = 0.91 L/s

**Max. Day Demand:**

**Q = Pop. x ADD x MDD + FF**

Q = (560 persons x 350 L/d/capita x 2.75)/86,400 + 8,000 L/min / 60

Q = 139.57 L/s

## 3.3. Water System Analysis

A detailed Water System Analysis (WSA) will be completed by Ainley Group for the development's proposed water distribution system. Ainley has been entrusted with the Town's water model so that they can perform assessments on the watermain sizes for any proposed development within the municipal boundary. The WSA will demonstrate that the proposed water distribution network is capable of achieving the minimum standards in the MOE and Town of Wasaga Beach guidelines for water distribution systems. At the time of preparation of this report it is understood that currently proposed upgrades to the Town's water distribution system through construction of a water tower and works yard immediately south of Beachwood Road will satisfy the proposed flow demands (volume and pressure) of this development.

# 4. Sanitary Servicing

## 4.1. External Sanitary Servicing Framework

There are existing sanitary sewer systems in both Shore Lane and Beachwood Road. There is also a proposed (by others) sanitary system within the future Betty Boulevard extension right-of-way.

There are no external sanitary catchment areas draining into the subject land. Meanwhile, all internal sanitary drainage will be captured and conveyed by the internal system.

Refer to drawing **SAN-1** in **Appendix C** for more details.

## 4.2. Allocated and Projected Sanitary Flows

The Town of Wasaga Beach *Engineering Standards* (March 2015) provides theoretical design criteria for future developments and their projected sanitary loading demands. Specifically, the guidelines provide design criteria for development based on the proposed land use.

*Peak Domestic Sewage Flow Equation:*

$$Q_{tot} = Q_p + Q_i = \left[\frac{PQM}{86.4}\right] + [I \times A]$$

Where: $Q_{tot}$ = Total peak sewage flow (L/s)

$Q_p$ = Peak domestic sewage flow (L/s)

$Q_i$ = Extraneous sewage flow (L/s)

P = Design population in thousands (560 people = 0.560)

Q = Average daily flow (350 L/person/day)

A = Area of Development (ha), 5.88 ha

I = Units of extraneous flow (0.28 L/ha/s - Refer to the Town of Wasaga Beach Engineering Standards (2015))

M = Harmon Peaking Factor

M = **Harmon Peaking Factor**; $M = 1 + \frac{14}{(P)^{0.5}+4} \quad (M \geq 2.0)$

Therefore: $M = 1 + \frac{14}{(0.560)^{0.5}+4}$

M = 3.95

$$Q_{tot} = \frac{0.560 \times 350 \times 3.95}{86.4} + 0.28 \times 5.88$$

$Q_{tot}$ = 10.61 L/s (total peak sewage flow)

## 4.3. Proposed Sanitary Servicing Scenario

The proposed works involve the construction of an internal gravity sanitary collector sewer system that will collect wastewater flows and convey flow to the existing sanitary sewer system within Shore Lane.

The internal sanitary sewers will be constructed in accordance with the Town's engineering standards and the MOE guidelines. The sewers would consist of PVC DR35 pipe with a minimum diameter of 200mm. The design has ensured that the minimum and maximum velocities are maintained under full flow conditions as well as the flushing velocities outlined in the Town of Wasaga Beach standards.

Refer to drawings **SAN-1** in **Appendix C** which illustrate the proposed drainage scenario for the subject lands and delineates the sanitary sewer sub-catchment areas. A copy of the sanitary sewer design sheets is provided in **Appendix B**.

Due to road grading constraints at the future extension of Betty Boulevard and proposed Street 'A', three (3) lots at the north limit of Street 'A' have reduced depth sanitary laterals requiring basement sewage to be pumped and to a gravity lateral. Floors other than the basement will drain by gravity to the developments internal sanitary sewer network.

# 5. Transportation

## 5.1. External Road Network

The Town of Wasaga Beach's Official Plan *Schedule 'B Transportation System* identifies Shore Lane and Betty Boulevard as Local Municipal Roads and Beachwood Road as Provincial Highway/Future Collector Road. A copy of *Schedule 'B'* has been attached in **Appendix D**.

## 5.2. Proposed Site Entrance & Internal Road Network

The *Draft Plan of Vacant Land Condominium* proposes that the development be accessed via two (2) entrances. One entrance will be to the future Betty Boulevard extension (by others) to the northwest and the other entrance will be to Beachwood Road, to the south.

The subject lands will consist of an internal road network of private roads, with horizontal and vertical alignments conforming to the Town of Wasaga Beach's current Engineering Standards, Section 2.5. Development Right-of-Way (ROW) cross-sections proposed within the *Draft Plan* consist of only 10.0m widths. All proposed cross sections are in accordance with the Town of Wasaga Beach's current Engineering standards, drawing No. 2F.

Refer to the *Draft Plan Vacant Land Condominium,* dated July 27, 2020 attached in **Appendix D** for the proposed entrance locations and alignment, ROW widths and internal roadway networks.

A Traffic Impact Study has been undertaken by JD Engineering, dated July 10$^{th}$, 2020, in support of the proposed development.

# 6. Site Grading

The proposed grading will conform to the Town of Wasaga Beach *Engineering Standards* (March 2015). Road and Lot grading are generally directed towards the proposed end-of-pipe SWM facility and existing roadway right-of-ways.

The lot grading plans (Drawings **SG-1** and **SG-2**) are provided in **Appendix C** for reference. Some of the specific areas of interest are discussed in more detail below.

## 6.1. Easements and Blocks

Due to the site grading adjacent to existing development and requirement to convey flow through the site from lands to the south, two (2) servicing/drainage corridors are required to be provided, otherwise referred to as the West Block and East Easement.

Blocks/Easements are an essential part of allowing access for future possible maintenance to the proposed culverts and services.

The west block contains an 1800mm x 900mm concrete box culvert which conveys flows generated from south of the subject lands (Culvert #16) to the future Betty Boulevard extension, and ultimately north to Georgian Bay. The east easement contains a 750mm dia. concrete pipe which outlets into a flat bottom open channel to convey flows generated from south of the subject lands (Culvert #17) to Shore Lane. This corridor also contains a 150mm dia. PVC watermain and 200mm dia. PVC sanitary sewer required to service the proposed development.

# 7. Utilities

The lands will be serviced by secondary utilities including Bell, hydro, gas and cable TV. There are currently existing services on Beachwood Road and Shore Lane.

It is acknowledged that the utility providers for the area are Enbridge Gas, Wasaga Distribution Inc., Bell Canada and Rogers Cable. Each of these utility companies will be contacted in advance of the detailed design to ensure that sufficient capacity exists within the current installations and/or to arrange for future upgrades to support the proposed development.

# 8. Conclusions

This Functional Servicing & Stormwater Management Report identifies the recommended servicing design for the proposed Beachwood Development. This Report, read in conjunction with the *Civil Engineering Preliminary Design Drawing Set,* outlines the proposed infrastructure required to service the lands in terms of water, wastewater, stormwater management, roads, and grading.

This Report has been prepared based on the Town of Wasaga Beach Engineering Standards. The Report has recommended the proposed development can be adequately serviced based on the following:

- The provision of storm sewers, Low Impact Development measures, and end-of-pipe stormwater management facility as outlined in **Sections 2**.
- The provision of watermains as outlined in **Section 3**.
- The provision of gravity sanitary sewers as outlined in **Section 4**.
- The provision of Transportation Infrastructure as outlined in **Section 5**.
- The provision of Site Grading as outlined in **Section 6**.
- The provision of Utility Infrastructure as outlined in **Section 7**.

In conclusion, it is recommended that the approval authorities support the application for the Proposed *Draft Plan of Vacant Land Condominium*.

All of which is respectfully submitted,
**THE JONES CONSULTING GROUP LTD.**

# Appendix A

## Supporting Calculations – Stormwater Servicing

- SWMF Volumetric Sizing Calculations
- SWMF Volume Table
- SWMF Stage Storage Discharge Table
- LID Design Notes – Sand Filter
- Hydraulic Conveyance Calculations
    - West Drainage Easement Culvert
    - East Drainage Easement Culvert
    - East Drainage Easement Channel
    - SWMF Overland Flow Channel
    - SWMF Flow Spreader Weir
    - SWMF Emergency Overflow Weir
- NVCA P-Budget Tool Loading Calculations
- Storm Sewer Design Sheet
- PCSWMM Model Outputs

**Stormwater Management Facility No. 1**
**Sand Filter Volumetric Criteria Calculations**

**CLIENT:** Beachwood Developments Inc.
**PROJECT:** Beachwood Development
**FILE:** ROM-17026

**DATE:** September 2020
**DESIGN:** JB
**CHECKED:** DR

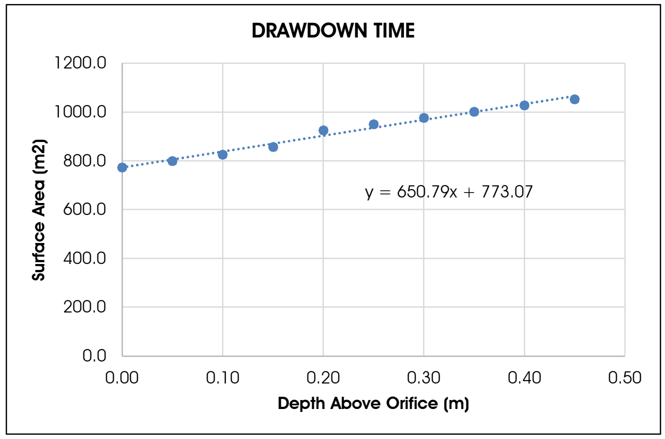
| | Area (ha) | TIMP(%) |
|---|---|---|
| Total - Area 301 | 4.012 | 66.0% |
| Post Development Drainage Area | 4.01 | 66.0% |

**Permanent Pool and Extended Detention Volumes:**

| | | |
|---|---|---|
| Drainage Area | 4.01 | ha |
| Imperviousness | 66.0% | |

| | Infiltration |
|---|---|
| **Imperviousness** | **Storage Vol.** |
| 35% | 25 $m^3$/ha |
| 55% | 30 $m^3$/ha |
| 70% | 35 $m^3$/ha |
| 85% | 40 $m^3$/ha |

*Excerpt - MOE Table 3.2, Enhanced Level*




**Volumetric Criteria:**

| | | |
|---|---|---|
| Water Quality Volumetric Criteria | 33.7 | $m^3$/ha |
| Erosion Control Volumetric Criteria | 40 | $m^3$/ha |

Source & Notes:
*MOE Table 3.2*

**SWMF Volume Requirements:**

| | | |
|---|---|---|
| Water Quality Volume (WQV) Required | 135 | $m^3$ |
| Erosion Control (EC) Required | 160 | $m^3$ |
| 25mm Event Runoff Volume Required | 342 | $m^3$ |
| Extended Detention Volume Provided | 739 | $m^3$ |

**SWMF Drawdown Requirements:**

| | | |
|---|---|---|
| Minimum Drawdown Time, MOE Table 4.8 | 24 | Hours |
| Preferred Drawdown Time, MOE Table 4.8 | 48 | Hours |

**Drawdown Calculations:**
**Based on Eqn. 4.11 MOE SWM Planning and Design Manual**

| **Extended Detention Drawdown** | | |
|---|---|---|
| Intercept of Regression, C3 | 773.1 | |
| Slope of Regression, C2 | 650.8 | |
| Orifice Area | 0.0050 | $m^2$ |
| Secondary Orifice - Inlet Elevation | 180.650 | m |
| Depth over Orifice (MAX WL) | 0.460 | $m^2$ |
| Drawdown Time - Max Ponding Level | 85,559 | Sec |
| | 23.8 | Hours |
| 25mm Event Water Level | 180.50 | m |
| Depth over Orifice (WQV WL) | 0.310 | m |
| Drawdown Time - 25mm Event Ponding Level | 67,642 | Sec |
| | 18.8 | Hours |

**SWMF Forebay Requirements:**

| | | |
|---|---|---|
| Forebay Length Provided | 21.0 | m |
| Forebay Depth Provided | 1.00 | m |
| Forebay Length : Width Ratio Provided | 2.23 | m |
| Minimum Forebay Length, MOE Table 4.8 | 12.8 | m |
| Minimum Forebay Depth, MOE Table 4.8 | 1.00 | m |
| Preferred Forebay Depth, MOE Table 4.8 | 1.50 | m |
| Minimum Forebay L : W, MOE Table 4.8 | 2.00 | :1 |
| Preferred Forebay L : W, MOE Table 4.8 | 4.00 - 5.00 | :1 |

**Forebay Calculations:**
**MOE Equation 4.5 - Forebay Settling Length**
Dist = SQRT((r*$Q_p$)/$V_s$)

| | | | |
|---|---|---|---|
| Forebay Length Required | Dist | 12.8 | m |
| Length-to-width ratio of forebay | r | 2.23 | :1 |
| $Q_P$ from the pond during design quality storm | $Q_p$ | 0.022 | $m^3$/s |
| Settling velocity | $V_s$ | 0.0003 | m/s |

**MOE Equation 4.6 - Dispersion Length**
Dist = (8*Q)/(d*$V_f$)

| | | | |
|---|---|---|---|
| Length of dispersion | Dist | 9.5 | m |
| Inlet (Pipe Capacity) flowrate-5 yr | Q | 0.594 | $m^3$/s |
| Depth of the permanent pool in the forebay | d | 1.00 | m |
| Desired velocity in the forebay | $V_f$ | 0.50 | m/s |

**Design Criteria Check:**

| | |
|---|---|
| Is Max. Required WQV Met? | YES |
| Is Max. Required ECV Met? | YES |
| Is Min. Required Drawdown Time Met? | NO |
| Is Preferred Drawdown Time Met? | NO |
| Is Required Forebay Length Provided? | YES |
| Is Minimum Forebay Depth Provided? | YES |
| Is Preferred Forebay Depth Provided? | NO |
| Is Minimum L : W Ratio Provided? | YES |
| Is Preferred L : W Ratio Provided? | NO |

Additional Notes:
Water Quality Volume (WQV) provided for Enhanced Level Treatment, Erosion Control provided by using the **discharge rates** for the sand filter

**Stormwater Management Facility No. 1**
**Storage Basin Volume Table**

**CLIENT:** Beachwood Developments Inc. | **DATE:** September 2020

**PROJECT:** Beachwood Development | **DESIGN:** JB

**FILE:** ROM-17026 | **CHECKED:** DR

Bottom Elevation 179.15 m
Stage 0.05 m

| Elevation | Depth | Total Area | Average Area | Incremental Active Storage Volume | Accumulated Active Storage Volume | Total Volume | |
|---|---|---|---|---|---|---|---|
| (m) | (m) | ($m^2$) | ($m^2$) | ($m^3$) | ($m^3$) | ($m^3$) | |
| 179.15 | 0.00 | 94 | 94.00 | 0.00 | 0.00 | 0.00 | Dead Storage Volume |
| 179.20 | 0.05 | 100 | 97.00 | 0.00 | 0.00 | 4.86 | |
| 179.25 | 0.10 | 107 | 103.50 | 0.00 | 0.00 | 10.05 | |
| 179.30 | 0.15 | 114 | 110.50 | 0.00 | 0.00 | 15.60 | |
| 179.35 | 0.20 | 122 | 118.00 | 0.00 | 0.00 | 21.50 | |
| 179.40 | 0.25 | 129 | 125.50 | 0.00 | 0.00 | 27.76 | |
| 179.45 | 0.30 | 137 | 133.00 | 0.00 | 0.00 | 34.41 | |
| 179.50 | 0.35 | 145 | 141.00 | 0.00 | 0.00 | 41.44 | |
| 179.55 | 0.40 | 153 | 149.00 | 0.00 | 0.00 | 48.87 | |
| 179.60 | 0.45 | 161 | 157.00 | 0.00 | 0.00 | 56.70 | |
| 179.65 | 0.50 | 169 | 165.00 | 0.00 | 0.00 | 64.95 | |
| 179.70 | 0.55 | 178 | 173.50 | 0.00 | 0.00 | 73.62 | |
| 179.75 | 0.60 | 187 | 182.50 | 0.00 | 0.00 | 82.72 | |
| 179.80 | 0.65 | 195 | 191.00 | 0.00 | 0.00 | 92.27 | |
| 179.85 | 0.70 | 205 | 200.00 | 0.00 | 0.00 | 102.28 | |
| 179.90 | 0.75 | 214 | 209.50 | 0.00 | 0.00 | 112.74 | |
| 179.95 | 0.80 | 224 | 219.00 | 0.00 | 0.00 | 123.68 | |
| 180.00 | 0.85 | 774 | 499.00 | 0.00 | 0.00 | 135.10 | |
| 180.05 | 0.90 | 800 | 787.00 | 0.00 | 0.00 | 174.45 | |
| 180.10 | 0.95 | 826 | 813.00 | 0.00 | 0.00 | 215.09 | |
| 180.15 | 1.00 | 858 | 842.00 | 0.00 | 0.00 | 257.04 | |
| 180.20 | 1.05 | 926 | 892.00 | 45.70 | 45.70 | 302.74 | Active Storage Volume |
| 180.25 | 1.10 | 951 | 938.50 | 46.95 | 92.65 | 349.69 | |
| 180.30 | 1.15 | 977 | 964.00 | 48.20 | 140.85 | 397.89 | |
| 180.35 | 1.20 | 1002 | 989.50 | 49.46 | 190.31 | 447.35 | |
| 180.40 | 1.25 | 1028 | 1015.00 | 50.74 | 241.05 | 498.09 | |
| 180.45 | 1.30 | 1053 | 1040.50 | 52.02 | 293.07 | 550.11 | |
| 180.50 | 1.35 | 1079 | 1066.00 | 53.32 | 346.39 | 603.43 | |
| 180.55 | 1.40 | 1105 | 1092.00 | 54.61 | 401.00 | 658.04 | |
| 180.60 | 1.45 | 1132 | 1118.50 | 55.93 | 456.93 | 713.97 | |
| 180.65 | 1.50 | 1158 | 1145.00 | 57.24 | 514.17 | 771.21 | |
| 180.70 | 1.55 | 1185 | 1171.50 | 58.57 | 572.74 | 829.78 | |
| 180.75 | 1.60 | 1212 | 1198.50 | 59.91 | 632.65 | 889.69 | |
| 180.80 | 1.65 | 1239 | 1225.50 | 61.26 | 693.91 | 950.95 | |
| 180.85 | 1.70 | 1266 | 1252.50 | 62.61 | 756.52 | 1013.56 | |
| 180.90 | 1.75 | 1293 | 1279.50 | 63.98 | 820.50 | 1077.54 | |
| 180.95 | 1.80 | 1321 | 1307.00 | 65.36 | 885.86 | 1142.90 | |
| 181.00 | 1.85 | 1349 | 1335.00 | 66.74 | 952.60 | 1209.64 | |
| 181.05 | 1.90 | 1377 | 1363.00 | 68.13 | 1020.73 | 1277.77 | |
| 181.10 | 1.95 | 1405 | 1391.00 | 69.53 | 1090.26 | 1347.30 | |

**Stormwater Management Facility No. 1**
**Storage Basin Volume Table**

**CLIENT:** Beachwood Developments Inc. **DATE:** September 2020

**PROJECT:** Beachwood Development **DESIGN:** JB

**FILE:** ROM-17026 **CHECKED:** DR

Bottom Elevation 179.15 m
Stage 0.05 m

| Elevation | Depth | Total Area | Average Area | Incremental Active Storage Volume | Accumulated Active Storage Volume | Total Volume | |
|---|---|---|---|---|---|---|---|
| (m) | (m) | ($m^2$) | ($m^2$) | ($m^3$) | ($m^3$) | ($m^3$) | |
| 181.15 | 2.00 | 1433 | 1419.00 | 70.95 | 1161.21 | 1418.25 | Active Storage Volume |
| 181.20 | 2.05 | 1462 | 1447.50 | 72.36 | 1233.57 | 1490.61 | |
| 181.25 | 2.10 | 1490 | 1476.00 | 73.80 | 1307.37 | 1564.41 | |
| 181.30 | 2.15 | 1519 | 1504.50 | 75.23 | 1382.60 | 1639.64 | |
| 181.35 | 2.20 | 1548 | 1533.50 | 76.68 | 1459.28 | 1716.32 | |
| 181.40 | 2.25 | 1577 | 1562.50 | 78.14 | 1537.42 | 1794.46 | |
| 181.45 | 2.30 | 1607 | 1592.00 | 79.61 | 1617.03 | 1874.07 | |
| 181.50 | 2.35 | 1637 | 1622.00 | 81.08 | 1698.11 | 1955.15 | |
| 181.55 | 2.40 | 2138 | 1887.50 | 94.21 | 1792.32 | 2049.36 | |
| 181.60 | 2.45 | 2243 | 2190.50 | 21.91 | 1814.23 | 2071.27 | |
| 181.65 | 2.50 | 2310 | 2276.73 | 100.89 | 1915.12 | 2172.16 | |
| 181.70 | 2.55 | 2459 | 2384.58 | 74.87 | 1989.99 | 2247.03 | |

## Stormwater Management Facility No. 1
## Infiltration Basin Stage Storage Discharge Details

**CLIENT:** Beachwood Developments Ltd.

**PROJECT:** Beachwood Development

**FILE:** ROM-17026

**DATE:** September 2020

**DESIGN:** JB

**CHECKED:** DR

| | | Orifice No. 1 | | Orifice No. 2 | | Orifice No. 3 | | Emergency Overflow Weir | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Elevation (m) | Depth Above Orifice Centroid (m) | Orifice No. 1 Flow ($m^3/s$) | Depth Above Orifice Centroid (m) | Orifice No. 2 Flow ($m^3/s$) | Depth Above Orifice Centroid (m) | Orifice No. 3 Flow ($m^3/s$) | Depth Above Overflow Weir (m) | Overflow Weir Flow ($m^3/s$) | Total Active Storage ($m^3$) | Total Flow ($m^3/s$) | Total Storage ($m^3$) |
| | 179.15 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 0 |
| | 179.20 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 11 |
| | 179.25 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 22 |
| | 179.30 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 32 |
| | 179.35 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 43 |
| | 179.40 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 54 |
| | 179.45 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 65 |
| | 179.50 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 76 |
| | 179.55 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 87 |
| | 179.60 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 97 |
| | 179.65 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 108 |
| | 179.70 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 118 |
| | 179.75 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 127 |
| | 179.80 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 137 |
| | 179.85 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 146 |
| | 179.90 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 156 |
| | 179.95 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 165 |
| | 180.00 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 174 |
| | 180.05 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 184 |
| | 180.10 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 193 |
| *Permanent Pool* | 180.15 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 0 | 0.000 | 203 |
| *25mm* | 180.20 | 0.01 | 0.001 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 46 | 0.001 | 303 |
| | 180.25 | 0.06 | 0.003 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 93 | 0.003 | 350 |
| | 180.30 | 0.11 | 0.005 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 141 | 0.005 | 398 |
| | 180.35 | 0.16 | 0.006 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 190 | 0.006 | 447 |
| | 180.40 | 0.21 | 0.006 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 241 | 0.006 | 498 |
| | 180.45 | 0.26 | 0.007 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 293 | 0.007 | 550 |
| | 180.50 | 0.31 | 0.008 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 346 | 0.008 | 603 |
| | 180.55 | 0.36 | 0.008 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 401 | 0.008 | 658 |
| | 180.60 | 0.41 | 0.009 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 457 | 0.009 | 714 |
| | 180.65 | 0.46 | 0.010 | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 | 514 | 0.010 | 771 |
| *2 yr* | 180.70 | 0.51 | 0.010 | 0.00 | 0.013 | 0.05 | 0.000 | 0.00 | 0.000 | 573 | 0.023 | 830 |
| | 180.75 | 0.56 | 0.010 | 0.00 | 0.027 | 0.10 | 0.000 | 0.00 | 0.000 | 633 | 0.037 | 890 |
| | 180.80 | 0.61 | 0.011 | 0.00 | 0.040 | 0.15 | 0.000 | 0.00 | 0.000 | 694 | 0.051 | 951 |
| | 180.85 | 0.66 | 0.011 | 0.00 | 0.055 | 0.20 | 0.000 | 0.00 | 0.000 | 757 | 0.066 | 1014 |
| *5 Yr* | 180.90 | 0.71 | 0.012 | 0.03 | 0.070 | 0.25 | 0.000 | 0.00 | 0.000 | 821 | 0.082 | 1078 |
| | 180.95 | 0.76 | 0.012 | 0.08 | 0.122 | 0.30 | 0.000 | 0.00 | 0.000 | 886 | 0.134 | 1143 |

## Stormwater Management Facility No. 1
## Infiltration Basin Stage Storage Discharge Details

**CLIENT:** Beachwood Developments Ltd.
**PROJECT:** Beachwood Development
**FILE:** ROM-17026

**DATE:** September 2020
**DESIGN:** JB
**CHECKED:** DR

| | | Orifice No. 1 | | Orifice No. 2 | | Orifice No. 3 | | Emergency Overflow Weir | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Elevation (m) | Depth Above Orifice Centroid (m) | Orifice No. 1 Flow ($m^3/s$) | Depth Above Orifice Centroid (m) | Orifice No. 2 Flow ($m^3/s$) | Depth Above Orifice Centroid (m) | Orifice No. 3 Flow ($m^3/s$) | Depth Above Overflow Weir (m) | Overflow Weir Flow ($m^3/s$) | Total Active Storage ($m^3$) | Total Flow ($m^3/s$) | Total Storage ($m^3$) |
| *10 Yr* | 181.00 | 0.81 | 0.013 | 0.13 | 0.157 | 0.35 | 0.000 | 0.00 | 0.000 | 953 | 0.170 | 1210 |
| | 181.05 | 0.86 | 0.013 | 0.18 | 0.186 | 0.40 | 0.000 | 0.00 | 0.000 | 1021 | 0.199 | 1278 |
| | 181.10 | 0.91 | 0.013 | 0.23 | 0.211 | 0.45 | 0.000 | 0.00 | 0.000 | 1090 | 0.224 | 1347 |
| *25 Yr* | 181.15 | 0.96 | 0.014 | 0.28 | 0.233 | 0.50 | 0.000 | 0.00 | 0.000 | 1161 | 0.246 | 1418 |
| | 181.20 | 1.01 | 0.014 | 0.33 | 0.253 | 0.55 | 0.000 | 0.00 | 0.000 | 1234 | 0.267 | 1491 |
| *50 Yr* | 181.25 | 1.06 | 0.014 | 0.38 | 0.272 | 0.60 | 0.000 | 0.00 | 0.000 | 1307 | 0.286 | 1564 |
| *Timmins* | 181.30 | 1.11 | 0.015 | 0.43 | 0.289 | 0.65 | 0.000 | 0.00 | 0.000 | 1383 | 0.304 | 1640 |
| *100 Yr* | 181.35 | 1.16 | 0.015 | 0.48 | 0.306 | 0.70 | 0.000 | 0.00 | 0.000 | 1459 | 0.321 | 1716 |
| | 181.40 | 1.21 | 0.015 | 0.53 | 0.322 | 0.75 | 0.000 | 0.05 | 0.084 | 1537 | 0.421 | 1794 |
| | 181.45 | 1.26 | 0.016 | 0.58 | 0.337 | 0.80 | 0.000 | 0.10 | 0.391 | 1617 | 0.744 | 1874 |
| | 181.50 | 1.31 | 0.016 | 0.63 | 0.351 | 0.85 | 0.000 | 0.15 | 0.835 | 1698 | 1.202 | 1955 |
| | 181.55 | 1.36 | 0.016 | 0.68 | 0.365 | 0.90 | 0.000 | 0.20 | 1.403 | 1792 | 1.784 | 2049 |
| | 181.60 | 1.41 | 0.017 | 0.73 | 0.378 | 0.95 | 0.000 | 0.25 | 2.090 | 1814 | 2.484 | 2071 |
| | 181.65 | 1.46 | 0.017 | 0.78 | 0.391 | 1.00 | 0.000 | 0.30 | 2.895 | 1915 | 3.302 | 2172 |
| *Top of Pond* | 181.70 | 1.51 | 0.017 | 0.83 | 0.403 | 1.05 | 0.000 | 0.35 | 3.818 | 1990 | 4.238 | 2247 |

### CONTROL MAINTENANCE HOLE - ORIFICE CONTROL

| Orifice No. 1 | | Orifice No. 2 | | Orifice No. 3 | |
|---|---|---|---|---|---|
| - Orifice diameter (m): | **0.080** | - Orifice diameter (m) | **0.450** | - Orifice diameter (m) | **0.000** |
| - Area ($m^2$) = | **0.005026** | - Area ($m^2$) = | **0.159038** | - Area ($m^2$) = | **0.000000** |
| - Orifice C = | **0.63** | - Orifice C = | **0.63** | - Orifice C = | **0.63** |
| - Invert (m)= | **180.15** | - Invert (m)= | **180.65** | - Invert (m)= | **180.65** |
| - Orifice Centroid (m) = | **180.19** | - Orifice Centroid (m) | **180.88** | - Orifice Centroid (m) | **180.65** |

### OVERFLOW WEIR

Overflow Weir

| | | |
|---|---|---|
| - Length of Weir(m) | **10** | |
| - Weir Sill(m) | **181.35** | |
| - Downstream Length of 'Overflow' Weir (m | **9** | @181.35m |
| - Weir Side Slopes (H:V) | **10** | :1 |

Submerged Orifice Equation:
Q = CxAx(2gH)^0.5
where;
Q = flow rate ($m^3$)
C = constant
A = area of opening($m^2$)
H = net head on the orifice
g = Acceleration due to gravity

Un-Submerged Orifice Equation (Flow Below Orifice Centroid):
$Q_w = 1.65([(pi*(D^2)/4)(2*cos^{-1}[(((D/2)-d)/(D/2))*180/pi)]/360)-((D/2-d)(Dd-d^2)^{0.5})]/d)d^{1.5}$
where;
Q = flow rate ($m^3/s$)
D=orifice diameter (m)
d=depth of flow above invert (m)

Broad Crested Weir
- Weir Equation: $Q = (C\ L\ (H^{3/2})) + (C(H^{5/2})Tan\ (\alpha/2))$
where ; Q = flow rate (cms)
C = constant (refer to Triangular and Rectangular 'C' Equations)
L = length (m)
H = head on the weir (m)
α = angle at apex of triangle (radians)

| Rectangular 'C' Equation | Triangular 'C' Equation |
|---|---|
| y=(a+bx)/(1+cx+dx^2) | y=(a+bx)/(1+cx+dx^2) |
| a -10383.4898 | a -1.0071E-05 |
| b 3418997.012 | b 143.5986704 |
| c 2131595.078 | c 114.5046511 |
| d -235014.247 | d -4.76857422 |

*x = head divided by downstream Length of Weir (H/L)

| | Project: | Beachwood Development | Date: | Sep-20 |
|---|---|---|---|---|
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| | Revisions: | | | |

BMP Type **LOT LEVEL / CONVEYANCE CONTROL / END OF PIPE**

Bioretention Systems, designed per *CVC Section 4.5 - Bioretention,* generally involves practices that temporarily store, treat and infiltrate collected runoff. Systems can be constructed with or without underdrains to achieve full or partial infiltration based on native soil characteristics. The primary component of a bioretention system is the Filter Media, generally comprised of a mixture of Sand, fines and organic materials which improves water quality. Typically systems are vegetated and topped with mulch, have pre-treatment devices and a overflow by-pass. These systems are adaptable to sites and can range from simple rain gardens on a Lot Level to Large Cells in End-of-Pipe facilities

**Common Concerns**

1 Risk of Groundwater Contamination
2 Risk of Soil Contamination
3 Location on Private Property / Enforcement
4 Proximity to Foundations & Seepage
5 Winter Operation
6 Roadway Stability
7 Pedestrian Traffic

**Physical Suitability and Constraints**

1 Proximity to Drinking Water Sources i.e. WHPA's, not within 2yr Travel Time
2 Site Topography, locate on slopes 1 - 5%
3 Available Head should be 1.0 - 1.5m to drive filter
4 Water Table, minimum 1.0m separation from Bottom of Facility to SHGWT
5 Site Soils & Infiltration Capacity, min 15 mm/hr
6 Drainage Area, 5:1 to 20:1 Impervious Drainage Area to BMP Area
7 Pollution Hot Spot Runoff
8 Setback from Buildings, minimum of 5m from foundations
9 Proximity to Underground Utilities
10 Overhead Wires, Check Tree Canopies will not interfere with O/H Systems in place

**Table 4.5.1 Ability of bioretention to meet SWM objectives**

| BMP | Water Balance Benefit | Water Quality Improvement | Stream Channel Erosion Control Benefits |
|---|---|---|---|
| Bioretention with no underdrain | Yes | Yes – size for water quality storage requirement | Partial – based on available storage volume and infiltration rates |
| Bioretention with underdrain | Partial – based on available storage volume beneath the underdrain and soil infiltration rate | Yes – size for water quality storage requirement | Partial – based on available storage volume beneath the underdrain and soil infiltration rate |
| Bioretention with underdrain and impermeable liner | Partial – some volume reduction through evapotranspiration | Yes – size for water quality storage requirement | Partial – some volume reduction through evapotranspiration |

**Table 4.5.2 Volumetric runoff reduction[1] achieved by bioretention**

| LID Practice | Location | % Runoff Reduction[1] | Reference |
|---|---|---|---|
| Bioretention without underdrain | Connecticut | 99% | Dietz and Clausen (2005) |
| | Pennsylvania | 80% | Ermilio (2005) |
| | Pennsylvania | 70% | Emerson and Traver (2004) |
| Bioretention with underdrain | North Carolina | 40 to 60% | Smith and Hunt (2007) |
| | North Carolina | 33 to 50% | Hunt and Lord (2006) |
| | Maryland and North Carolina | 20 to 50% | Li *et al.* (2009) |
| **Runoff Reduction Estimate[2]** | | **85% without underdrain<br>45% with underdrain** | |

**BMP Sizing Guidelines**

1 Commonly located near Impervious Surfaces generating runoff such as around parking lots, traffic islands, near buildings and in boulevards, online or offline config.
2 Geometry & Layout, typically linear or rectangular, can be orientated to fit many spaces but should be considered early in site layout & development
3 Maximum *Recommended* Footprint is typically based on Total Impervious Drainage Area, and ranges between 0.01 and 0.5 ha, with a max. of 0.8 ha.
4 Aim to orientate the cell to promote the spread of inflows evenly over the cell with flat slopes, multiple cells can be used in parallel or series
5 Pretreatment should be provided to ensure system longevity, forebay/gravel diaphragm, vegetated filter strips/grass swales or filter devices (OGS Units)
6 Conveyance by Direct Inlet, 3rd Pipe System, or Storm Sewer, an Overflow should be provided outletting to grade or a nearby Storm Sewer
7 Ponding levels are typically 150 - 250mm to limit length of inundation of planting. Deeper ponding depths may be recognized with a variation in plantings.
8 The infiltration rate of the soil in the pervious area should be at least 15 mm/hr
9 Filter Media will vary depending on type of system to be constructed. Typically includes a layer of Mulch (75mm) on Filter Medium (1000 - 1250mm), followed by a Pea Gravel Choking Layer (100mm) and the Storage Layer of 50mm Clear Stone (300 - 450mm) which may include an underdrain system.
10 Monitoring Wells 100 - 150mm in dia. should be installed to check for sediment buildup and ensure adequate drawndown times are being recognized
11 Underdrains should be placed a minimum of 100mm above the bottom of the Storage Layer, should be HDPE or equiv. with Smooth Interior Walls, and be a minimum of 200mm in dia. to accommodate freezing. Often a strip of Geotextile is placed between Filter Media and Pea Gravel Layer to prevent the migration of fines.
12 Facilities can be landscaped formally, and include a variety of plantings from low lying herbaceous materials to trees

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| | Revisions: | | | |

**BMP Sizing Calculations**

1) Depth of a facility is a function of native soil infiltration rate, porosity of storage media, and targeted drawdown time as well full or partial infiltration:

a) i) **Storage Layer Depth w/ Underdrain**

**Eq. 1.1** $d_{S\,MAX} = f' \times t / 0.4$ Where: $d_{S\,MAX}$ = Maximum Depth of Storage with Underdrain
$f'$ = Design Infiltration Rate (mm/hr)
$t$ = Infiltration Rate for Native Soils (mm/hr) - Use FS Rate, not actual

ii) **Storage Layer Depth w/o Underdrain**

**Eq. 1.2** $d_{S\,MAX} = f' \times 48 / 0.4$ Where: $d_{S\,MAX}$ = Maximum Depth of Storage without Underdrain
$f'$ = Depth of Ponding (mm)

2) Remaining facility depth established by ponding, layering, and corresponding thickness to establish 1D storage volume.

3) Determine the total volume of runoff (WQV) produced by the Water Quality Event (WQE) in PCSWMM.

3) Facility footprint is then determined based on the Water Quality Volume (WQV) to be captured, as well as the available 1D storage volume in the reservoir:

**Eq. 2** $A_P = WQV / (d_C * V_R)$ Where: $A_P$ = Footprint Area of Practice ($m^2$)
WQV = Water Quality Volume ($m^3$), Depth of Runoff (mm) * Catchment ($m^2$) OR Runoff Vol. ($m^3$)
$d_{P\,MAX}$ = Design Bioretention Cell Depth (m)

4) 3D facility draw down to confirm available capacity is within inter-event window: **Eq. 3** $t_D = (V_R / f') \times (A_P / P) \times \ln[(d_P + (A_P / P) / (AP / P)]$

5) Curb inlet sizing to ensure full capture of flow for side inlets (i.e. raingardens) **Eq. 4** $W_T = 0.817 \times Q^{0.42} \times S_O^{0.3} \times [1 / (n \times S_X)]^{0.6}$

6) Check flowrate through media to ensure ponding depth provided is adequate: **Eq. 5** $Q_{MAX,M} = (K_M \times A_P \times (h_{MAX} / d_m)) / 3.6 \times 10^6$

7) Check underdrain flowrate to ensure ponding depth provided is adequate: **Eq. 6** $Q_{MAX,P} = L \times B \times C_d \times Ao\,(2 \times g \times h_{MAX})^{0.5}$

8) Verify limiting flow rate is design peak inflow rate to mitigate ponding, provide additional hydraulic controls where required.

**Bioretention Cell Sizer:** = *user input* = *calculated / constant* = *design parameter*

1) $d_{S\,MAX} = f' \times t / 0.4$ → $d_{S\,MAX}$ = **3240 mm** OR $d_{S\,MAX} = f' \times 48 / 0.4$ → $d_{S\,MAX}$ = **2160 mm**

Where: $f'$ = 18 *mm/hr*; $t$ = 72 *hrs*; $t$ = 48 *hrs*

2)

| Layer | Depth (mm) | VR | 1D Storage (mm) |
|---|---|---|---|
| Freeboard | 150 | 1.0 | 150 |
| Ponding | 500 | 1.0 | 500 |
| Mulch / Topsoil | 100 | 0.4 | 40 |
| Filter Media | 500 | 0.3 | 150 |
| Choker Layer | 0 | 0.4 | 0 |
| Underdrain Dia. | 100 | 0.4 | 40 |
| Storage Layer | 400 | 0.4 | 160 |
| **Total Depth** | **1750** | | **1040** |

301 PCSWMM Catchment ID

3) **Bioretention Practice Footprint** Where: WQV = 345.3 $m^3$; $d_{P\,MAX}$ = 1.040 m

$A_P = WQV / d_P$

$A_{P\,MIN}$ = **332 $m^2$**

| | | |
|---|---|---|
| Depth of Runoff: | 8.47 | mm |
| U/S Catchment Area: | 4.077 | ha |
| XIMP: | 30% | |
| Imp. Area: | 1.22 | ha |

**Recommended Area Check (5:1 - 20:1 Ratio):**

**CAUTION! Consider additional protection against sedimentation**

4) **3D Facility Drawdown**

$t_D = (V_R / f') \times (A_P / P) \times \ln[(d_P + (A_P / P) / (AP / P)]$

$t_D$ = **18.2 hrs**

| | | |
|---|---|---|
| $A_{P\,MIN}$: | 332.04 | $m^2$ |
| Length: | 50.0 | m |
| Width: | 10.8 | m |
| $A_{P\,ACT}$: | 540 | $m^2$ |
| Aspect Ratio: | 4.62963 | :1 |
| P: | 121.6 | m (assumed rectangular) |
| $V_R$: | 0.35 | (weighted average) |

$A_{P\,MIN}$ = **611.6 $m^2$**
$A_{P\,MAX}$ = **2446 $m^2$**

**$I/P_{PROV}$ : 22.7 :1**

| | | | | |
|---|---|---|---|---|
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5) **Curb Inlet Sizing**

$W_T = 0.817 \times Q^{0.42} \times S_O^{0.3} \times [1 / (n \times S_X)]^{0.6}$

$W_T =$ 6.371 m

| | | | |
|---|---|---|---|
| $Q_{25mm}$ : | 0.441 | $m^3/s$ | (design peak flow) |
| $S_O$ : | 2.00 | % | (gutter slope) |
| $S_X$ : | 2.00 | % | (pavement crossfall) |
| n : | 0.013 | | (mannings coefficient) |

6) **Flow Through Media**

$Q_{MAX,M} = (K_M \times A_P \times (h_{MAX} / d_m)) / 3.6 \times 10^6$

$Q_{MAX,M} =$ 0.040 $m^3/s$

| | | | |
|---|---|---|---|
| $K_M$ : | 120 | mm /h | (hydaulic conductivity of media) |
| $A_{P\ ACT}$ : | 540 | $m^2$ | (area of practice) |
| $h_{MAX}$ : | 1.1 | m | (max. head = ponding + stone + media) |
| $d_m$ : | 0.5 | m | (depth of media) |

7) **Flow Through Underdrains**

$Q_{MAX,P} = L \times B \times C_d \times Ao\,(2 \times g \times h_{MAX})^{0.5}$

**Drain offset:** 0 mm

$Q_{MAX,P} =$ 3.958 $m^3/s$

| | | | |
|---|---|---|---|
| L : | 626 | m | (total length of pipe) |
| B : | 0.5 | | (clogging factor, typical 0.5) |
| $C_d$ : | 0.63 | | (coefficient of discharge) |
| Ao : | 0.0037 | m | (Open Area of Pipe / m length) |
| g : | 9.81 | $m/s^2$ | (gravitational constant) |
| $h_{MAX}$ : | 1.5 | m | (total head above top of pipe) |

8) **Check Limiting Flow Rate**

| | |
|---|---|
| $Q_{25mm}$ : | 0.441 |
| $Q_{MAX,M}$ : | 0.040 |
| $Q_{MAX,P}$ : | 3.958 |

**Is Q Limiting :** NO, SURFACE PONDING OCCURING

**ADDITIONAL NOTES:** $C_{LID} = 160997 \times g^{0.5} \times C_d \times Aorif / A_P$ *PCSWMM Drain Coefficient Parameter Estimate*

Where: $A_{ORIF}$ = cross-sectional flow area of underdrain x number of underdrain runs ($m^2$)

32 Underdrain Runs

**PCSWMM LID Editor Inputs:**

https://support.chiwater.com/77680/lid-control-editor

| LID Type: | Bio-Retention Cell / Rain Garden | | | | | Sub-Catchment Min. Area: | 540.0 $m^2$ | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Surface | | | | Storage | | | | Underdrain | | |
| Berm Height (mm) | Veg. Vol. (Frac.) | Mannings 'n' | Surf. Slope (%) | Thickness (mm) | Void Ratio | Seepage Rate (mm / hr) | Clogging Factor | Drain Coef. (mm / hr) | Drain Exp. | Drain Offset (mm) |
| 500 | 0 | 0.15 | 0.00% | 500 | 0.4 | 18 | 0 | 240.5 | 0.5 | 0 |

**Soil -** *Texture Class Required for Filter Media (Loamy Sand)*

| Thickness (mm) | Soil Texture Class | Porosity (frac) | Field Capacity (frac) | Wilting Point (frac) | Hydraulic Conductivity (mm / hr) | Conductivity Slope | Suction Head (mm) | |
|---|---|---|---|---|---|---|---|---|
| 500 | Loamy Sand | 0.437 | 0.105 | 0.047 | 29.97 | 5 | 60.96 | |

**Rerefences:** **STEP Low Impact Development Stormwater Management Planning & Design Guide:** *https://wiki.sustainabletechnologies.ca/wiki/Main_Page*

| Item | Page | Link |
|---|---|---|
| *Eq.'s 1.1, 1.2, 2, 3* | *Bioretention: Sizing* | *https://wiki.sustainabletechnologies.ca/wiki/Bioretention:_Sizing* |
| *Eq. 4* | *Curb Cuts* | *https://wiki.sustainabletechnologies.ca/wiki/Curb_cuts* |
| *Eq. 5* | *Flow Through Media* | *https://wiki.sustainabletechnologies.ca/wiki/Flow_through_media* |
| *Eq. 6* | *Flow Through Perforated Pipe* | *https://wiki.sustainabletechnologies.ca/wiki/Flow_through_perforated_pipe* |

# Worksheet for Culvert 17 - Box

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.001 | |
| Channel Slope | 0.00500 | m/m |
| Height | 0.90 | m |
| Bottom Width | 1.80 | m |
| Discharge | 0.78 | m³/s |

## Results

| | | |
|---|---|---|
| Normal Depth | 0.06 | m |
| Flow Area | 0.10 | m² |
| Wetted Perimeter | 1.91 | m |
| Hydraulic Radius | 0.05 | m |
| Top Width | 1.80 | m |
| Critical Depth | 0.27 | m |
| Percent Full | 6.3 | % |
| Critical Slope | 0.00004 | m/m |
| Velocity | 7.68 | m/s |
| Velocity Head | 3.00 | m |
| Specific Energy | 3.06 | m |
| Froude Number | 10.32 | |
| Discharge Full | 39.49 | m³/s |
| Slope Full | 12.83903 | m/m |
| Flow Type | Supercritical | |

## GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | m |
| Length | 0.00 | m |
| Number Of Steps | 0 | |

## GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | m |
| Profile Description | | |
| Profile Headloss | 0.00 | m |
| Average End Depth Over Rise | 0.00 | % |
| Normal Depth Over Rise | 6.26 | % |
| Downstream Velocity | Infinity | m/s |

# Worksheet for Culvert 17 - Box

## GVF Output Data

| | | |
|---|---|---|
| Upstream Velocity | Infinity | m/s |
| Normal Depth | 0.06 | m |
| Critical Depth | 0.27 | m |
| Channel Slope | 0.00500 | m/m |
| Critical Slope | 0.00004 | m/m |

# Cross Section for Culvert 17 - Box

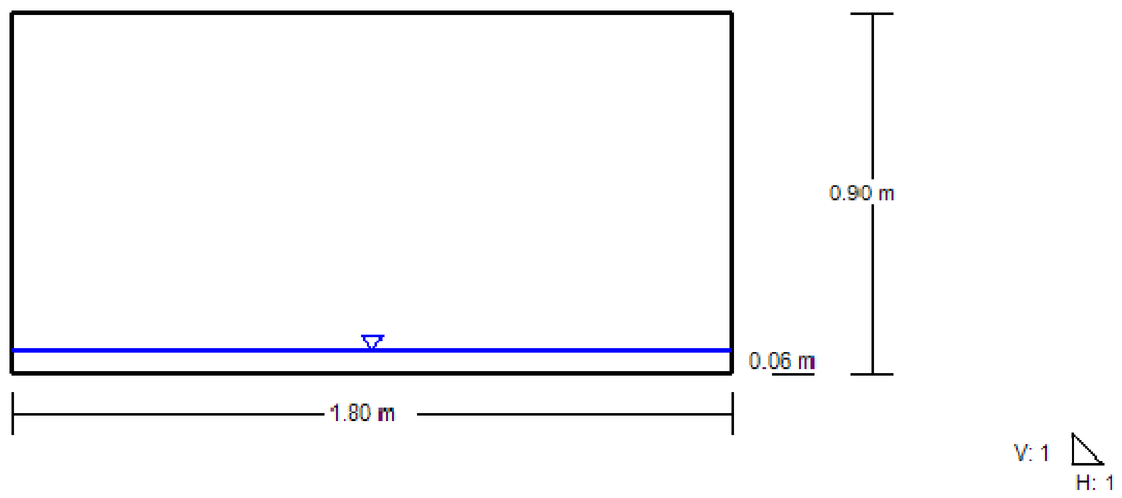
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.001 | |
| Channel Slope | 0.00500 | m/m |
| Normal Depth | 0.06 | m |
| Height | 0.90 | m |
| Bottom Width | 1.80 | m |
| Discharge | 0.78 | $m^3/s$ |

## Cross Section Image



# Worksheet for Culvert 16 - D/S Channel

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.025 | |
| Channel Slope | 0.02500 | m/m |
| Left Side Slope | 3.00 | m/m (H:V) |
| Right Side Slope | 3.00 | m/m (H:V) |
| Bottom Width | 1.00 | m |
| Discharge | 0.67 | $m^3/s$ |

## Results

| | | |
|---|---|---|
| Normal Depth | 0.22 | m |
| Flow Area | 0.37 | $m^2$ |
| Wetted Perimeter | 2.40 | m |
| Hydraulic Radius | 0.15 | m |
| Top Width | 2.33 | m |
| Critical Depth | 0.27 | m |
| Critical Slope | 0.01120 | m/m |
| Velocity | 1.81 | m/s |
| Velocity Head | 0.17 | m |
| Specific Energy | 0.39 | m |
| Froude Number | 1.45 | |
| Flow Type | Supercritical | |

## GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | m |
| Length | 0.00 | m |
| Number Of Steps | 0 | |

## GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | m |
| Profile Description | | |
| Profile Headloss | 0.00 | m |
| Downstream Velocity | Infinity | m/s |
| Upstream Velocity | Infinity | m/s |
| Normal Depth | 0.22 | m |
| Critical Depth | 0.27 | m |
| Channel Slope | 0.02500 | m/m |

# Worksheet for Culvert 16 - D/S Channel

## GVF Output Data

| | | |
|---|---|---|
| Critical Slope | 0.01120 | m/m |

# Cross Section for Culvert 16 - D/S Channel

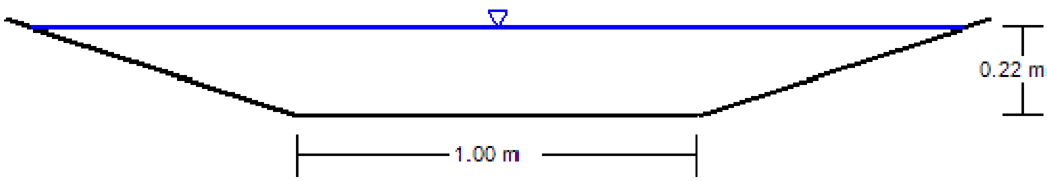
## Project Description

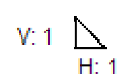
| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.025 | |
| Channel Slope | 0.02500 | m/m |
| Normal Depth | 0.22 | m |
| Left Side Slope | 3.00 | m/m (H:V) |
| Right Side Slope | 3.00 | m/m (H:V) |
| Bottom Width | 1.00 | m |
| Discharge | 0.67 | m³/s |

## Cross Section Image



# Worksheet for Culvert 16 - U/S Pipe

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.013 | |
| Channel Slope | 0.00500 | m/m |
| Diameter | 0.75 | m |
| Discharge | 0.67 | m³/s |

## Results

| | | |
|---|---|---|
| Normal Depth | 0.53 | m |
| Flow Area | 0.33 | m² |
| Wetted Perimeter | 1.49 | m |
| Hydraulic Radius | 0.22 | m |
| Top Width | 0.68 | m |
| Critical Depth | 0.51 | m |
| Percent Full | 70.5 | % |
| Critical Slope | 0.00564 | m/m |
| Velocity | 2.00 | m/s |
| Velocity Head | 0.20 | m |
| Specific Energy | 0.73 | m |
| Froude Number | 0.91 | |
| Maximum Discharge | 0.85 | m³/s |
| Discharge Full | 0.79 | m³/s |
| Slope Full | 0.00357 | m/m |
| Flow Type | SubCritical | |

## GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | m |
| Length | 0.00 | m |
| Number Of Steps | 0 | |

## GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | m |
| Profile Description | | |
| Profile Headloss | 0.00 | m |
| Average End Depth Over Rise | 0.00 | % |
| Normal Depth Over Rise | 70.49 | % |
| Downstream Velocity | Infinity | m/s |

# Worksheet for Culvert 16 - U/S Pipe

## GVF Output Data

| | | |
|---|---|---|
| Upstream Velocity | Infinity | m/s |
| Normal Depth | 0.53 | m |
| Critical Depth | 0.51 | m |
| Channel Slope | 0.00500 | m/m |
| Critical Slope | 0.00564 | m/m |

# Cross Section for Culvert 16 - U/S Pipe

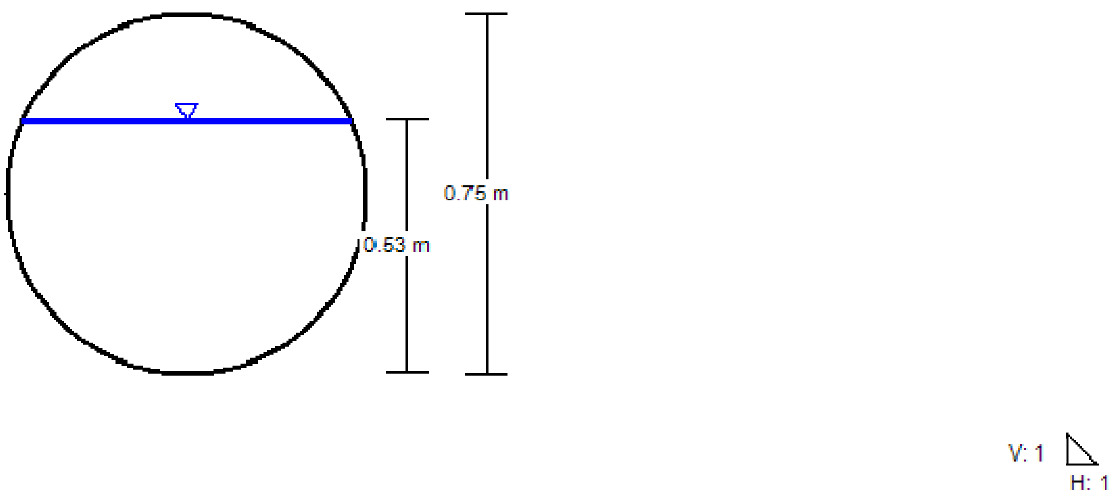
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.013 | |
| Channel Slope | 0.00500 | m/m |
| Normal Depth | 0.53 | m |
| Diameter | 0.75 | m |
| Discharge | 0.67 | m³/s |

## Cross Section Image



# Cross Section for SWMF Overland Flow

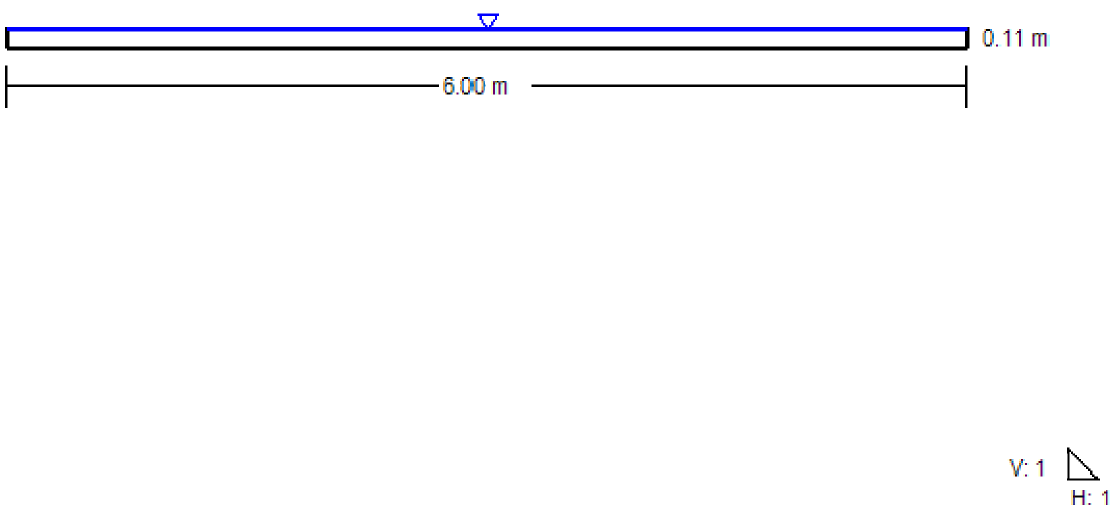
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Roughness Coefficient | 0.025 | |
| Channel Slope | 0.01000 | m/m |
| Normal Depth | 0.11 | m |
| Bottom Width | 6.00 | m |
| Discharge | 0.59 | m³/s |

## Cross Section Image



# Cross Section for Flow Spreader Weir

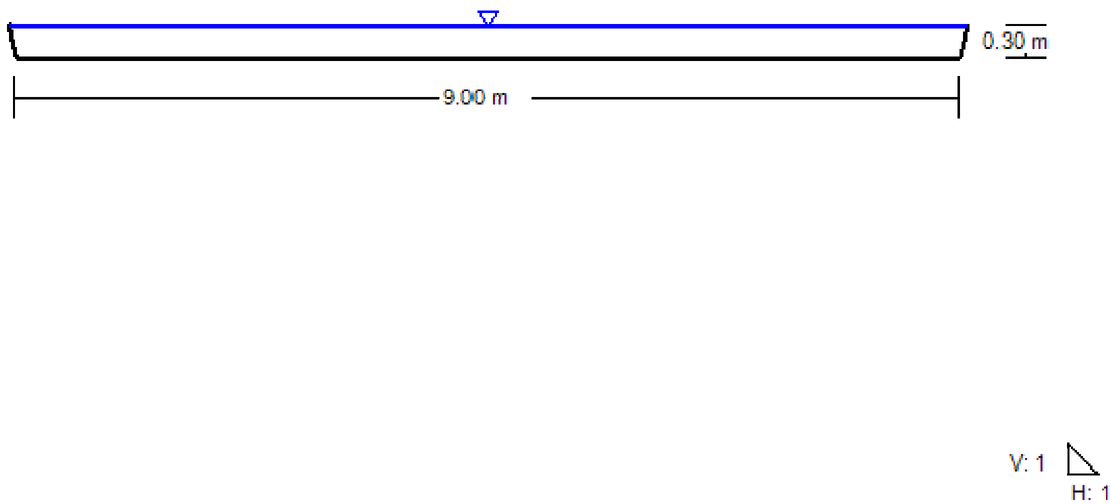
## Project Description

| | |
|---|---|
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Discharge | 2.72 | m³/s |
| Headwater Elevation | 180.60 | m |
| Crest Elevation | 180.30 | m |
| Tailwater Elevation | 180.15 | m |
| Weir Coefficient | 1.84 | SI |
| Crest Length | 9.00 | m |

## Cross Section Image



# Cross Section for SWMF Overflow Weir

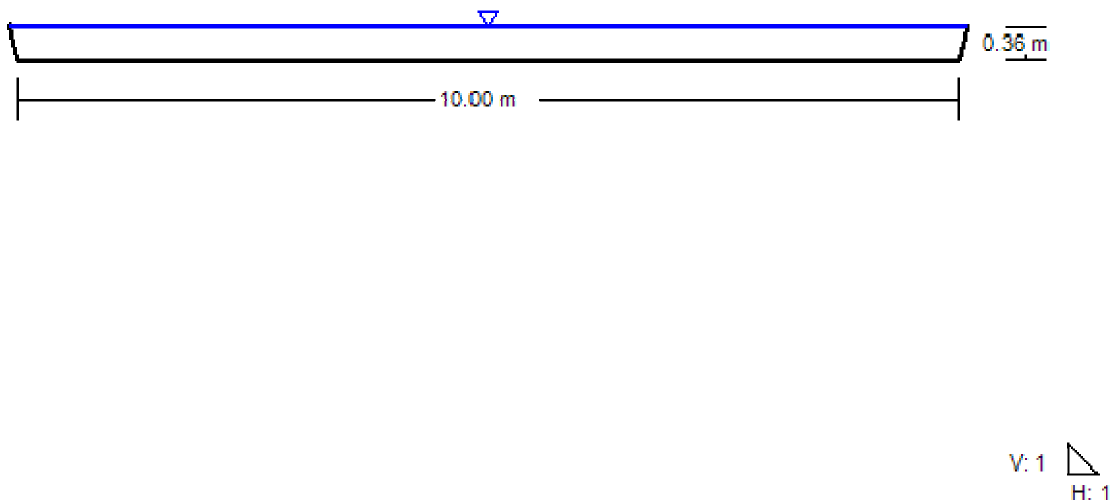
## Project Description

| | |
|---|---|
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Discharge | 3.97 | m³/s |
| Headwater Elevation | 181.71 | m |
| Crest Elevation | 181.35 | m |
| Tailwater Elevation | 181.15 | m |
| Weir Coefficient | 1.84 | SI |
| Crest Length | 10.00 | m |

## Cross Section Image



# Development Export Summary

## Development :Shore Lane - Wasaga Beach



# Pre-Development Phosphorus Export

DEVELOPMENT : Shore Lane - Wasaga Beach

| Landuse | | Area (ha) | P coeff (kg/ha) | Pload (kg/yr) |
|---|---|---|---|---|
| **Natural Heritage** | | | | |
| Forest | | 5.88 | 0.06 | 0.35 |
| | ***Natural Heritage Land use Class Total :*** | **5.88** | | **0.35** |
| | ***Development Total :*** | **5.88** | | **0.35** |





# Cropland Site Sediment & Phosphorus Pre-Development Export

DEVELOPMENT : Shore Lane - Wasaga Beach

COLOUR KEY : Site Specific Input | Constant / Lookup | Calculation

SubArea :

| | |
|---|---|
| Slope Area (ha) | R (rainfall / runoff for Lake Simcoe) |
| Surface Slope Gradient (%) | K (soil errodability factor) |
| Length of Slope (m) | NN (determined by slope) |
| Cropt Type Factor) | LS (slope length gradient factor) |
| Tillage Type Factor | C (crop management factor) |
| | P (prevention + capture) |
| | Soil Loss (kg/year) |
| | Phosphorus export (kg/ha/yr) |
| | Phosphorus load (kg/yr) |

PRE Developed Area (ha) :

Phosphorus export (kg/ha/yr) :

Phosphorus load (kg/yr) :

Updated : Sept 2014

# Post-Development Phosphorus Export

DEVELOPMENT : Shore Lane - Wasaga Beach

| Landuse | Area (ha) | P coeff (kg/ha) | Pload (kg/yr) |
|---|---|---|---|
| **Natural Heritage** | | | |
| Forest | 0.42 | 0.06 | 0.03 |
| ***Natural Heritage Land use Class Total :*** | **0.42** | | **0.03** |
| **Urban** | | | |
| Residential | 5.46 | 0.41 | 2.59 |
| ***Urban Land use Class Total :*** | **5.46** | | **2.59** |
| ***Development Total :*** | **5.88** | | **2.62** |

9/24/2020 Page 1 of 1

Updated : Sept 2014

# Cropland Site Sediment & Phosphorus Post-Development Export

DEVELOPMENT : Shore Lane - Wasaga Beach

COLOUR KEY : Site Specific Input | Constant / Lookup | Calculation

SubArea :

| | |
|---|---|
| Slope Area (ha) | R (rainfall / runoff for Lake Simcoe) |
| Surface Slope Gradient (%) | K (soil errodability factor) |
| Length of Slope (m) | NN (determined by slope) |
| Cropt Type Factor) | LS (slope length gradient factor) |
| Tillage Type Factor | C (crop management factor) |
| | P (prevention + capture) |
| | Soil Loss (kg/year) |
| | Phosphorus export (kg/ha/yr) |
| | Phosphorus load (kg/yr) |

PRE Developed Area (ha) :

Phosphorus export (kg/ha/yr) :

Phosphorus load (kg/yr) :

9/24/2020 Page 1 of 1

Updated : Sept 2014

# Post Dev BMP

| Area (ha) | Treated Area % | P coefficient | P coefficient | P Load Reduction (kg/yr) | Rationale |
|---|---|---|---|---|---|
| Best Management Practices (BMP) Applied (and Rationale) | | | | | |
| **Residential** | | | | | |
| 1.52 | 100 | 0.41 | 100 % | 0.62 | Rear Yard Soakaways for all rooftops, full infiltration of WQE |
| User Entry | | | | | |
| **Residential** | | | | | |
| 2.71 | 100 | 0.41 | 45 % | 0.50 | Remainder of 301 Catchment to SWMF |
| Sand or Media Filters | | | | | |
| **Residential** | | | | | |
| 1.23 | 100 | 0.41 | 100 % | 0.50 | Uncontrolled Areas to Grassed Swales, full infiltration of WQE |
| Enhanced Grass/Water Quality Swales | | | | | |
| **Forest** | | | | | |
| 0.42 | 100 | 0.06 | | | Uncontrolled NH Area |
| User Entry | | | | | |

# Development Area P and BMP Summary

| | |
|---|---|
| Total PreDevelopment Area (ha): | 5.88 |
| PreDevelopment Area excluding Wetlands (ha): | 5.88 |
| Total PostDevelopment Area (ha): | 5.88 |
| Total Area treated by BMP's (ha): | 5.88 |
| Treated Area total: | 5.88 |
| Total PreDevelopment Load (kg/yr): | 0.35 |
| Total PostDevelopment Load (kg/yr): | 2.62 |
| Total P Load Reduction with BMP's (kg/yr): | 1.62 |
| Minimum P Load Reduction Required: | 2.27 |
| Total PostDevelopment Load with BMP's (kg/yr) | 1.00 |

***Conclusion :*** No Net Increase in P Load.





# Post Dev Construction

**Shore Lane**
**5yr Storm Sewer Catchment Areas & Sizing**

**CLIENT:** Beachwood Developments
**PROJECT:** Beachwood Development
**FILE:** ROM-17026

**DATE:** September 2020
**DESIGN:** VS
**CHECKED BY:** JB / DR

**Storm Sewer Catchment Areas & Runoff Coefficients**

| Catchment Number | Area (Ha) | | | | Runoff Coefficient |
|---|---|---|---|---|---|
| | Landscape | Impervious (Roof) | Impervious (Parking, Sidewalk) | Total | Weighted |
| 201 | 1,222 | 1,244 | 133 | 2,599 | 0.59 |
| 202 | 744 | 1,109 | 436 | 2,289 | 0.69 |
| 203 | 799 | 1,191 | 447 | 2,436 | 0.69 |
| 204 | 1,006 | 1,377 | 861 | 3,244 | 0.70 |
| 205 | 321 | 425 | 402 | 1,148 | 0.72 |
| 206 | 443 | 660 | 520 | 1,623 | 0.72 |
| 207 | 716 | 1,066 | 400 | 2,182 | 0.69 |
| 208 | 449 | 669 | 296 | 1,413 | 0.69 |
| 209 | 81 | 0 | 297 | 378 | 0.76 |
| 210 | 1,376 | 1,093 | 3,743 | 6,212 | 0.76 |
| 211 | 999 | 1,093 | 4,338 | 6,429 | 0.80 |
| 212 | 756 | 318 | 1,266 | 2,340 | 0.69 |
| 213 | 324 | 284 | 469 | 1,077 | 0.70 |
| 214 | 498 | 527 | 767 | 1,792 | 0.72 |
| 215 | 3,725 | 1,128 | 108 | 4,961 | 0.41 |
| 101 | 913 | 1,309 | 246 | 2,468 | 0.66 |
| 102 | 4,395 | 167 | 0 | 4,562 | 0.27 |
| 103 | 207 | 310 | 0 | 517 | 0.64 |
| 104 | 4,267 | 0 | 192 | 4,459 | 0.28 |
| 105 | 2,977 | 0 | 0 | 2,977 | 0.25 |
| 106 | 2,020 | 584 | 0 | 2,604 | 0.40 |
| 107 | 435 | 651 | 0 | 1,086 | 0.64 |
| **Total:** | **28672.81** | **15204.84** | **14918.35** | **58796.00** | **0.58** |

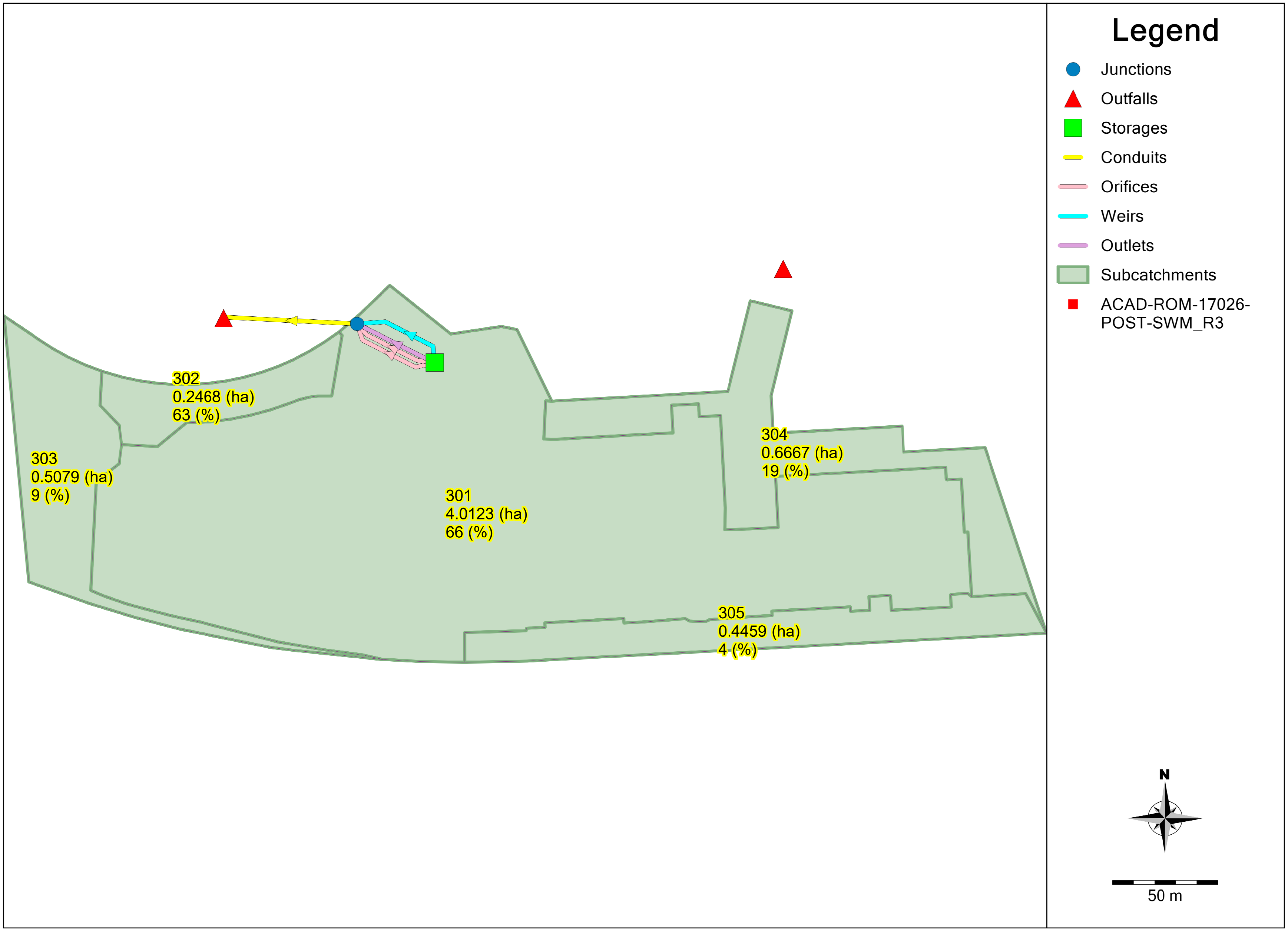
**Percent Impervious (%)= 51**

| Runoff Coefficient | | |
|---|---|---|
| Landscape | Impervious (Roof) | Impervious (Parking, Sidewalk) |
| 0.25 | 0.90 | 0.90 |

## 5 Year Storm Sewer Sizing

| STREET | AREAS | MANHOLE FROM | MANHOLE TO | LENGTH (m) | INCREMENT C | INCREMENT A | INCREMENT CA | TOTAL CA | FLOW TIME (min) TO | FLOW TIME (min) IN | I (mm/h) | TOTAL Q (cms) | S (%) | D (mm) | Q FULL (cms) | V FULL (m/s) | % FULL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| St. 'C' | 201 | STM 59 | STM 58 | 81.6 | 0.59 | 0.26 | 0.15 | 0.15 | 15.00 | 1.41 | 74 | 0.031 | **0.50** | **300** | 0.068 | 1.0 | **45.9** |
| St. 'C' | 202 | STM 58 | STM 57 | 43.3 | 0.69 | 0.23 | 0.16 | 0.31 | 16.41 | 0.75 | 69 | 0.060 | **0.50** | **300** | 0.068 | 1.0 | **87.6** |
| St. 'C' | 203 | STM 57 | STM 56 | 16.1 | 0.69 | 0.24 | 0.17 | 0.48 | 17.15 | 0.24 | 67 | 0.089 | **0.50** | **375** | 0.124 | 1.1 | **71.7** |
| St. 'C' | 204 | STM 56 | STM 55 | 45.3 | 0.70 | 0.32 | 0.22 | 0.70 | 17.39 | 0.60 | 66 | 0.129 | **0.50** | **450** | 0.202 | 1.3 | **64.2** |
| | | | | | | | | | | | | | | | | | |
| St. 'B' | 205 | DCB | STM 63 | 36.4 | 0.72 | 0.11 | 0.08 | 0.08 | 15.00 | 0.63 | 74 | 0.016 | **0.50** | **300** | 0.068 | 1.0 | **23.7** |
| St. 'B' | 206 | STM 63 | STM 64 | 41.1 | 0.72 | 0.16 | 0.12 | 0.19 | 15.63 | 0.71 | 72 | 0.039 | **0.50** | **300** | 0.068 | 1.0 | **56.5** |
| St. 'B' | 207 | STM 64 | STM 52 | 26.4 | 0.69 | 0.22 | 0.15 | 0.35 | 16.34 | 0.39 | 69 | 0.067 | **0.50** | **375** | 0.124 | 1.1 | **53.8** |
| St. 'B' | 208 | STM 52 | STM 55 | 44.6 | 0.69 | 0.14 | 0.10 | 0.44 | 16.73 | 0.66 | 68 | 0.084 | **0.50** | **375** | 0.124 | 1.1 | **67.7** |
| St. 'B' | | STM 55 | STM 65 | 23.1 | | | | 1.14 | 17.99 | 0.27 | 65 | 0.206 | **0.50** | **525** | 0.304 | 1.4 | **67.8** |
| | | | | | | | | | | | | | | | | | |
| St. 'D' | 209 | DCB | STM 73 | 9.0 | 0.76 | 0.04 | 0.03 | 0.03 | 15.00 | 0.16 | 74 | 0.006 | **0.50** | **300** | 0.068 | 1.0 | **9.1** |
| St. 'D' | 210 | HD UNIT 82 | STM 73 | 30.4 | 0.76 | 0.62 | 0.47 | 0.47 | 15.00 | 0.45 | 74 | 0.096 | **0.50** | **375** | 0.124 | 1.1 | **77.7** |
| St. 'D' | 211 | HD UNIT 83 | STM 73 | 38.4 | 0.80 | 0.64 | 0.51 | 0.51 | 15.00 | 0.57 | 74 | 0.105 | **0.50** | **375** | 0.124 | 1.1 | **84.4** |
| St. 'D' | | STM 73 | STM 49 | 36.3 | | | | 1.01 | 15.57 | 0.43 | 72 | 0.202 | **0.50** | **525** | 0.304 | 1.4 | **66.4** |
| St. 'B' | 212 | STM 49 | STM 50 | 58.9 | 0.69 | 0.23 | 0.16 | 1.17 | 16.00 | 0.70 | 70 | 0.229 | **0.50** | **525** | 0.304 | 1.4 | **75.3** |
| St. 'B' | 213 | STM 50 | STM 65 | 29.2 | 0.70 | 0.11 | 0.08 | 1.25 | 16.70 | 0.35 | 68 | 0.237 | **0.50** | **525** | 0.304 | 1.4 | **77.9** |
| | 214 | STM 65 | STM 54 | 40.5 | 0.72 | 0.18 | 0.13 | 2.52 | 18.26 | 0.41 | 64 | 0.450 | **0.50** | **675** | 0.594 | 1.7 | **75.6** |
| | | STM 54 | HEADWALL | 6.5 | | | | 2.52 | 18.67 | 0.07 | 63 | 0.443 | **0.50** | **675** | 0.594 | 1.7 | **74.5** |
| | | | | | | | | | | | | | | | | | |
| | Total | | | | | 3.50 | 2.52 | | | | | | | | | | |

Q= 0.0028*C*I*A (cms); C=RUNOFF COEFFICIENT; I-RAINFALL INTENSITY (5 Year) =493.013/(T.C.+0.081)^0.701 A=AREA (ha)


Legend
Junctions
Outfalls
Storages
Conduits
Orifices
Weirs
Outlets
Subcatchments
ACAD-ROM-17026-POST-SWM_R3
302
0.2468 (ha)
63 (%)
303
0.5079 (ha)
9 (%)
301
4.0123 (ha)
66 (%)
304
0.6667 (ha)
19 (%)
305
0.4459 (ha)
4 (%)
N
50 m

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.014)
------------------------------------------------------------

PROJECT NO: ROM-17026 (Beachwood Development, Wasaga Beach)
MODEL CREATED BY: The Jones Consulting Group Ltd.
DATE COMPLETED: May 2020 - PSWMR Submission

*************
Element Count
*************
Number of rain gages ...... 7
Number of subcatchments ... 5
Number of nodes ........... 4
Number of links ........... 5
Number of pollutants ...... 0
Number of land uses ....... 0

****************
Raingage Summary
****************

| Name | Data Source | Data Type | Recording Interval |
|---|---|---|---|
| 100y12h | 100y12h | INTENSITY | 6 min. |
| 100y24h | 100y24h | INTENSITY | 6 min. |
| 100y4h | 100y4h | INTENSITY | 5 min. |
| 100y6h | 100y6h | INTENSITY | 6 min. |
| Chicago_4h | Chicago_4h | INTENSITY | 5 min. |
| Timmins | Timmins | INTENSITY | 60 min. |
| WQE | WQE | INTENSITY | 5 min. |

********************
Subcatchment Summary
********************

| Name | Area | Width | %Imperv | %Slope | Rain Gage | Outlet |
|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| 301 | 4.01 | 200.62 | 66.00 | 2.0000 Chicago_4h | SWMF1 |
| 302 | 0.25 | 123.40 | 63.00 | 2.0000 Chicago_4h | OF1 |
| 303 | 0.51 | 52.91 | 9.00 | 2.0000 Chicago_4h | OF1 |
| 304 | 0.67 | 74.08 | 19.00 | 2.0000 Chicago_4h | OF2 |
| 305 | 0.45 | 148.63 | 4.00 | 2.0000 Chicago_4h | OF2 |

************
Node Summary
************

| Name | Type | Invert Elev. | Max. Depth | Ponded Area | External Inflow |
|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 179.30 | 2.20 | 0.0 | |
| OF1 | OUTFALL | 179.00 | 0.45 | 0.0 | |
| OF2 | OUTFALL | 180.00 | 0.00 | 0.0 | |
| SWMF1 | STORAGE | 179.75 | 1.95 | 0.0 | |

************
Link Summary
************

| Name | From Node | To Node | Type | Length | %Slope | Roughness |
|---|---|---|---|---|---|---|
| C1 | Cntrl_MH | OF1 | CONDUIT | 61.9 | 0.8078 | 0.0130 |
| OR1 | SWMF1 | Cntrl_MH | ORIFICE | | | |
| OR2 | SWMF1 | Cntrl_MH | ORIFICE | | | |
| EOW | SWMF1 | Cntrl_MH | WEIR | | | |
| Underdrains | SWMF1 | Cntrl_MH | OUTLET | | | |

*********************
Cross Section Summary
*********************

| Conduit | Shape | Full Depth | Full Area | Hyd. Rad. | Max. Width | No. of Barrels | Full Flow |
|---|---|---|---|---|---|---|---|
| C1 | CIRCULAR | 0.45 | 0.16 | 0.11 | 0.45 | 1 | 0.26 |

```
*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 04/24/2020 00:00:00
Ending Date .............. 04/27/2020 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 00:05:00
Routing Time Step ........ 5.00 sec
Variable Time Step ....... YES
Maximum Trials ........... 8
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         0.147        25.016
Evaporation Loss .........         0.000         0.000
```

Infiltration Loss ........ 0.107 18.208
Surface Runoff ........... 0.035 5.994
Final Storage ............ 0.006 1.019
Continuity Error (%) ..... -0.819

```
**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.035         0.352
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.035         0.352
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.000         0.000
Continuity Error (%) .....         0.066


***************************
Time-Step Critical Elements
***************************
None


********************************
Highest Flow Instability Indexes
********************************
All links are stable.


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     4.50 sec
Average Time Step           :     5.00 sec
```

Maximum Time Step : 5.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00
Percent Not Converging : 0.00

***************************
Subcatchment Runoff Summary
***************************

| Subcatchment | Total Precip mm | Total Runon mm | Total Evap mm | Total Infil mm | Imperv Runoff mm | Perv Runoff mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | Runoff Coeff |
|---|---|---|---|---|---|---|---|---|---|---|
| 301 | 25.02 | 0.00 | 0.00 | 15.61 | 15.44 | 0.00 | 8.34 | 0.33 | 0.26 | 0.333 |
| 302 | 25.02 | 0.00 | 0.00 | 18.86 | 14.59 | 3.24 | 5.57 | 0.01 | 0.02 | 0.223 |
| 303 | 25.02 | 0.00 | 0.00 | 24.85 | 2.08 | 0.00 | 0.00 | 0.00 | 0.00 | 0.000 |
| 304 | 25.02 | 0.00 | 0.00 | 24.67 | 4.41 | 0.00 | 0.00 | 0.00 | 0.00 | 0.000 |
| 305 | 25.02 | 0.00 | 0.00 | 24.02 | 0.93 | 0.00 | 0.93 | 0.00 | 0.00 | 0.037 |

******************
Node Depth Summary
******************

| Node | Type | Average Depth Meters | Maximum Depth Meters | Maximum HGL Meters | Time of Max Occurrence days hr:min | Reported Max Depth Meters |
|---|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 0.22 | 0.23 | 179.53 | 0 01:16 | 0.23 |
| OF1 | OUTFALL | 0.02 | 0.03 | 179.03 | 0 01:15 | 0.03 |
| OF2 | OUTFALL | 0.00 | 0.00 | 180.00 | 0 00:00 | 0.00 |
| SWMF1 | STORAGE | 0.13 | 0.39 | 180.14 | 0 04:26 | 0.39 |

*******************
Node Inflow Summary
*******************

| Node | Type | Maximum Lateral Inflow CMS | Maximum Total Inflow CMS | Time of Max Occurrence days hr:min | Lateral Inflow Volume 10^6 ltr | Total Inflow Volume 10^6 ltr | Flow Balance Error Percent |
|---|---|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 0.000 | 0.002 | 0 00:51 | 0 | 0.335 | 0.074 |
| OF1 | OUTFALL | 0.020 | 0.022 | 0 01:25 | 0.0137 | 0.348 | 0.000 |
| OF2 | OUTFALL | 0.005 | 0.005 | 0 01:20 | 0.00413 | 0.00413 | 0.000 |
| SWMF1 | STORAGE | 0.257 | 0.257 | 0 01:20 | 0.335 | 0.335 | -0.004 |

**********************
Node Surcharge Summary
**********************

No nodes were surcharged.

*********************
Node Flooding Summary
*********************

No nodes were flooded.

**********************
Storage Volume Summary
**********************

| Storage Unit | Average Volume 1000 m3 | Avg Pcnt Full | Evap Pcnt Loss | Exfil Pcnt Loss | Maximum Volume 1000 m3 | Max Pcnt Full | Time of Max Occurrence days hr:min | Maximum Outflow CMS |
|---|---|---|---|---|---|---|---|---|
| SWMF1 | 0.103 | 4 | 0 | 0 | 0.304 | 13 | 0 04:26 | 0.002 |

***********************

Outfall Loading Summary
***********************

| Outfall Node | Flow Freq Pcnt | Avg Flow CMS | Max Flow CMS | Total Volume 10^6 ltr |
|---|---|---|---|---|
| OF1 | 64.76 | 0.002 | 0.022 | 0.348 |
| OF2 | 4.58 | 0.000 | 0.005 | 0.004 |
| System | 34.67 | 0.002 | 0.005 | 0.352 |

********************
Link Flow Summary
********************

| Link | Type | Maximum \|Flow\| CMS | Time of Max Occurrence days hr:min | Maximum \|Veloc\| m/sec | Max/ Full Flow | Max/ Full Depth |
|---|---|---|---|---|---|---|
| C1 | CONDUIT | 0.002 | 0 01:15 | 0.48 | 0.01 | 0.06 |
| OR1 | ORIFICE | 0.000 | 0 00:00 | | | 0.00 |
| OR2 | ORIFICE | 0.000 | 0 00:00 | | | 0.00 |
| EOW | WEIR | 0.000 | 0 00:00 | | | 0.00 |
| Underdrains | DUMMY | 0.002 | 0 00:51 | | | |

***************************
Flow Classification Summary
***************************

| Conduit | Adjusted /Actual Length | Dry | Up Dry | Down Dry | Sub Crit | Sup Crit | Up Crit | Down Crit | Norm Ltd | Inlet Ctrl |
|---|---|---|---|---|---|---|---|---|---|---|
| | | ---------- Fraction of Time in Flow Class ---------- | | | | | | | | |
| C1 | 1.00 | 0.01 | 0.00 | 0.00 | 0.34 | 0.65 | 0.00 | 0.00 | 0.64 | 0.00 |

*************************
Conduit Surcharge Summary
*************************

No conduits were surcharged.

Analysis begun on:  Tue Sep 22 16:35:39 2020
Analysis ended on:  Tue Sep 22 16:35:46 2020
Total elapsed time: 00:00:07

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.014)
------------------------------------------------------------

PROJECT NO: ROM-17026 (Beachwood Development, Wasaga Beach)
MODEL CREATED BY: The Jones Consulting Group Ltd.
DATE COMPLETED: May 2020 - PSWMR Submission

*************
Element Count
*************
Number of rain gages ...... 6
Number of subcatchments ... 5
Number of nodes ........... 4
Number of links ........... 5
Number of pollutants ...... 0
Number of land uses ....... 0

****************
Raingage Summary
****************

| Name | Data Source | Data Type | Recording Interval |
|---|---|---|---|
| 100y12h | 100y12h | INTENSITY | 6 min. |
| 100y24h | 100y24h | INTENSITY | 6 min. |
| 100y4h | 100y4h | INTENSITY | 5 min. |
| 100y6h | 100y6h | INTENSITY | 6 min. |
| Timmins | Timmins | INTENSITY | 60 min. |
| WQE | WQE | INTENSITY | 5 min. |

********************
Subcatchment Summary
********************

| Name | Area | Width | %Imperv | %Slope | Rain Gage | Outlet |
|---|---|---|---|---|---|---|
| 301 | 4.01 | 200.62 | 66.00 | 2.0000 | 100y24h | SWMF1 |
| 302 | 0.25 | 123.40 | 63.00 | 2.0000 | 100y24h | OF1 |
| 303 | 0.51 | 52.91 | 9.00 | 2.0000 | 100y24h | OF1 |
| 304 | 0.67 | 74.08 | 19.00 | 2.0000 | 100y24h | OF2 |
| 305 | 0.45 | 148.63 | 4.00 | 2.0000 | 100y24h | OF2 |

************
Node Summary
************

| Name | Type | Invert Elev. | Max. Depth | Ponded Area | External Inflow |
|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 179.30 | 2.20 | 0.0 | |
| OF1 | OUTFALL | 179.00 | 0.45 | 0.0 | |
| OF2 | OUTFALL | 180.00 | 0.00 | 0.0 | |
| SWMF1 | STORAGE | 179.75 | 1.95 | 0.0 | |

************
Link Summary
************

| Name | From Node | To Node | Type | Length | %Slope | Roughness |
|---|---|---|---|---|---|---|
| C1 | Cntrl_MH | OF1 | CONDUIT | 61.9 | 0.8078 | 0.0130 |
| OR1 | SWMF1 | Cntrl_MH | ORIFICE | | | |
| OR2 | SWMF1 | Cntrl_MH | ORIFICE | | | |
| EOW | SWMF1 | Cntrl_MH | WEIR | | | |
| Underdrains | SWMF1 | Cntrl_MH | OUTLET | | | |

*********************
Cross Section Summary
*********************

Full Full Hyd. Max. No. of Full

| Conduit | Shape | Depth | Area | Rad. | Width | Barrels | Flow |
|---|---|---|---|---|---|---|---|
| C1 | CIRCULAR | 0.45 | 0.16 | 0.11 | 0.45 | 1 | 0.26 |

*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************

Flow Units ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 04/24/2020 00:00:00
Ending Date .............. 04/27/2020 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 00:05:00
Routing Time Step ........ 5.00 sec
Variable Time Step ....... YES
Maximum Trials ........... 8
Number of Threads ........ 1

Head Tolerance .......... 0.001500 m

```
**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         0.711       121.000
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.323        54.986
Surface Runoff ...........         0.384        65.360
Final Storage ............         0.006         1.019
Continuity Error (%) .....        -0.302


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.385         3.846
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.375         3.751
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.009         0.095
Continuity Error (%) .....         0.006


***************************
Time-Step Critical Elements
***************************
None
```

*********************************
Highest Flow Instability Indexes
*********************************
All links are stable.

*************************
Routing Time Step Summary
*************************
Minimum Time Step : 4.50 sec
Average Time Step : 5.00 sec
Maximum Time Step : 5.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00
Percent Not Converging : 0.00

***************************
Subcatchment Runoff Summary
***************************

| Subcatchment | Total Precip | Total Runon mm | Total Evap mm | Total Infil mm | Imperv Runoff mm | Perv Runoff mm | Total Runoff mm | Total Runoff 10^6 ltr | Peak Runoff CMS | Runoff Coeff |
|---|---|---|---|---|---|---|---|---|---|---|
| 301 | 121.00 | 0.00 | 0.00 | 44.48 | 78.77 | 33.07 | 75.61 | 3.03 | 1.25 | 0.625 |
| 302 | 121.00 | 0.00 | 0.00 | 56.31 | 75.02 | 51.65 | 63.65 | 0.16 | 0.10 | 0.526 |
| 303 | 121.00 | 0.00 | 0.00 | 83.14 | 10.71 | 37.88 | 37.88 | 0.19 | 0.12 | 0.313 |
| 304 | 121.00 | 0.00 | 0.00 | 79.63 | 22.63 | 41.25 | 41.25 | 0.28 | 0.17 | 0.341 |
| 305 | 121.00 | 0.00 | 0.00 | 79.88 | 4.76 | 36.67 | 41.43 | 0.18 | 0.14 | 0.342 |

******************
Node Depth Summary
******************

| Node | Type | Average Depth Meters | Maximum Depth Meters | Maximum HGL Meters | Time of Max Occurrence days hr:min | Reported Max Depth Meters |
|---|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 0.24 | 0.88 | 180.18 | 0 12:18 | 0.88 |
| OF1 | OUTFALL | 0.05 | 0.39 | 179.39 | 0 12:18 | 0.39 |
| OF2 | OUTFALL | 0.00 | 0.00 | 180.00 | 0 00:00 | 0.00 |
| SWMF1 | STORAGE | 0.44 | 1.60 | 181.35 | 0 12:17 | 1.60 |

*******************
Node Inflow Summary
*******************

| Node | Type | Maximum Lateral Inflow CMS | Maximum Total Inflow CMS | Time of Max Occurrence days hr:min | Lateral Inflow Volume 10^6 ltr | Total Inflow Volume 10^6 ltr | Flow Balance Error Percent |
|---|---|---|---|---|---|---|---|
| Cntrl_MH | JUNCTION | 0.000 | 0.326 | 0 12:17 | 0 | 2.94 | 0.016 |
| OF1 | OUTFALL | 0.209 | 0.427 | 0 12:02 | 0.35 | 3.29 | 0.000 |
| OF2 | OUTFALL | 0.308 | 0.308 | 0 12:00 | 0.461 | 0.461 | 0.000 |
| SWMF1 | STORAGE | 1.252 | 1.252 | 0 11:59 | 3.04 | 3.04 | 0.000 |

**********************
Node Surcharge Summary
**********************

No nodes were surcharged.

*********************
Node Flooding Summary
*********************

No nodes were flooded.

```
**********************
Storage Volume Summary
**********************

--------------------------------------------------------------------------------------------------
                   Average     Avg  Evap Exfil       Maximum     Max    Time of Max    Maximum
                    Volume    Pcnt  Pcnt  Pcnt        Volume    Pcnt     Occurrence    Outflow
Storage Unit          1000 m3    Full  Loss  Loss       1000 m3    Full    days hr:min        CMS
--------------------------------------------------------------------------------------------------
SWMF1                  0.368      16     0     0        1.702      74       0  12:17      0.326


***********************
Outfall Loading Summary
***********************

-----------------------------------------------------------
                   Flow       Avg       Max       Total
                   Freq      Flow      Flow      Volume
Outfall Node          Pcnt       CMS       CMS    10^6 ltr
-----------------------------------------------------------
OF1                 97.78     0.013     0.427       3.290
OF2                 31.26     0.006     0.308       0.461
-----------------------------------------------------------
System              64.52     0.019     0.308       3.751


********************
Link Flow Summary
********************

-----------------------------------------------------------------------------
                          Maximum  Time of Max   Maximum    Max/    Max/
```

| Link | Type | \|Flow\| CMS | Occurrence days hr:min | \|Veloc\| m/sec | Full Flow | Full Depth |
|---|---|---|---|---|---|---|
| C1 | CONDUIT | 0.325 | 0 12:18 | 2.10 | 1.27 | 0.94 |
| OR1 | ORIFICE | 0.015 | 0 12:17 | | | 1.00 |
| OR2 | ORIFICE | 0.307 | 0 12:17 | | | 1.00 |
| EOW | WEIR | 0.002 | 0 12:17 | | | 0.01 |
| Underdrains | DUMMY | 0.002 | 0 01:43 | | | |

***************************
Flow Classification Summary
***************************

| Conduit | Adjusted /Actual Length | Dry | Up Dry | Down Dry | Sub Crit | Sup Crit | Up Crit | Down Crit | Norm Ltd | Inlet Ctrl |
|---|---|---|---|---|---|---|---|---|---|---|
| C1 | 1.00 | 0.02 | 0.00 | 0.00 | 0.01 | 0.96 | 0.00 | 0.00 | 0.67 | 0.00 |

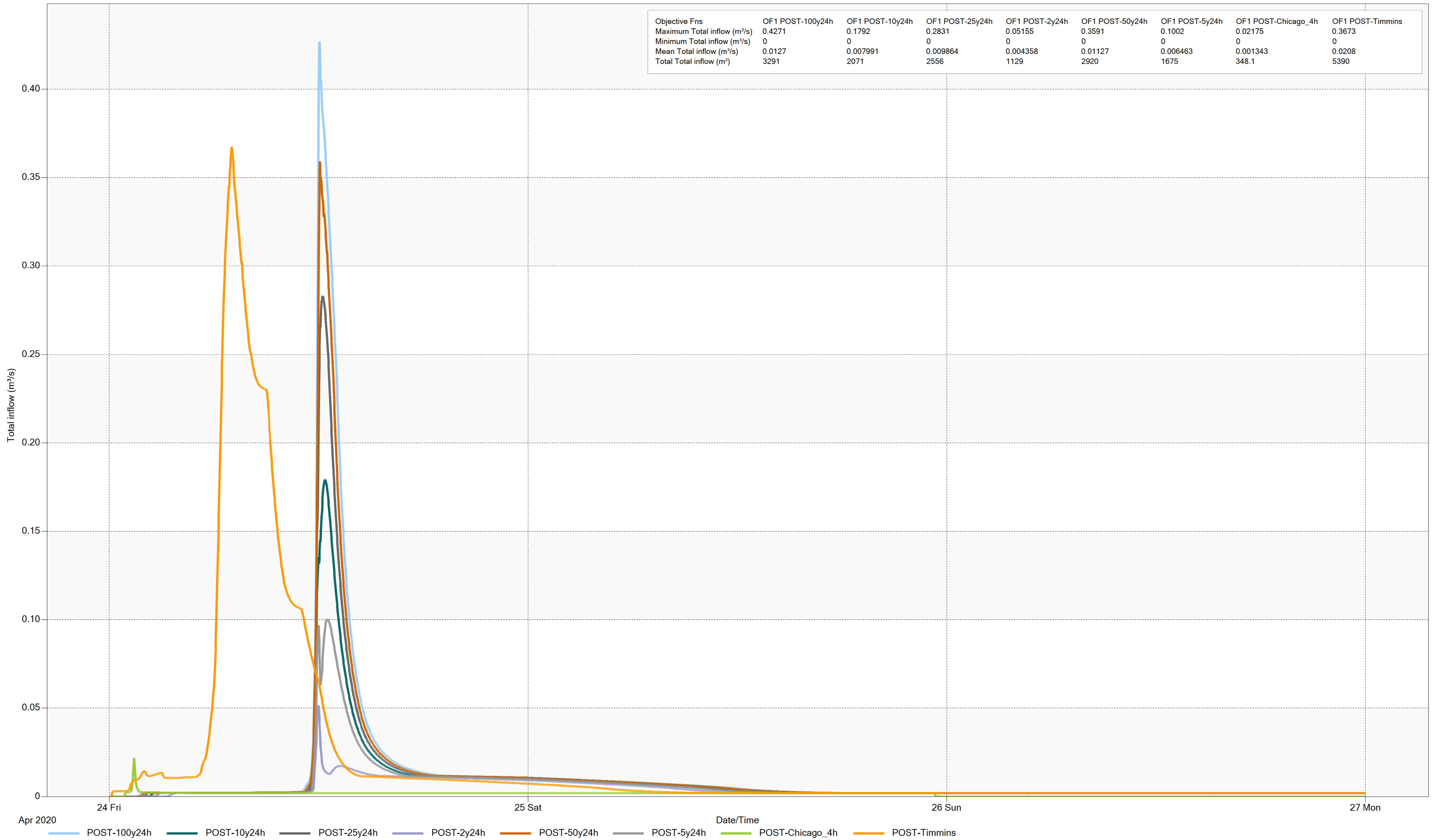
(---------- Fraction of Time in Flow Class ----------)

*************************
Conduit Surcharge Summary
*************************

| Conduit | Hours Full Both Ends | Hours Full Upstream | Hours Full Dnstream | Hours Above Full Normal Flow | Hours Capacity Limited |
|---|---|---|---|---|---|
| C1 | 0.01 | 0.67 | 0.01 | 0.84 | 0.01 |

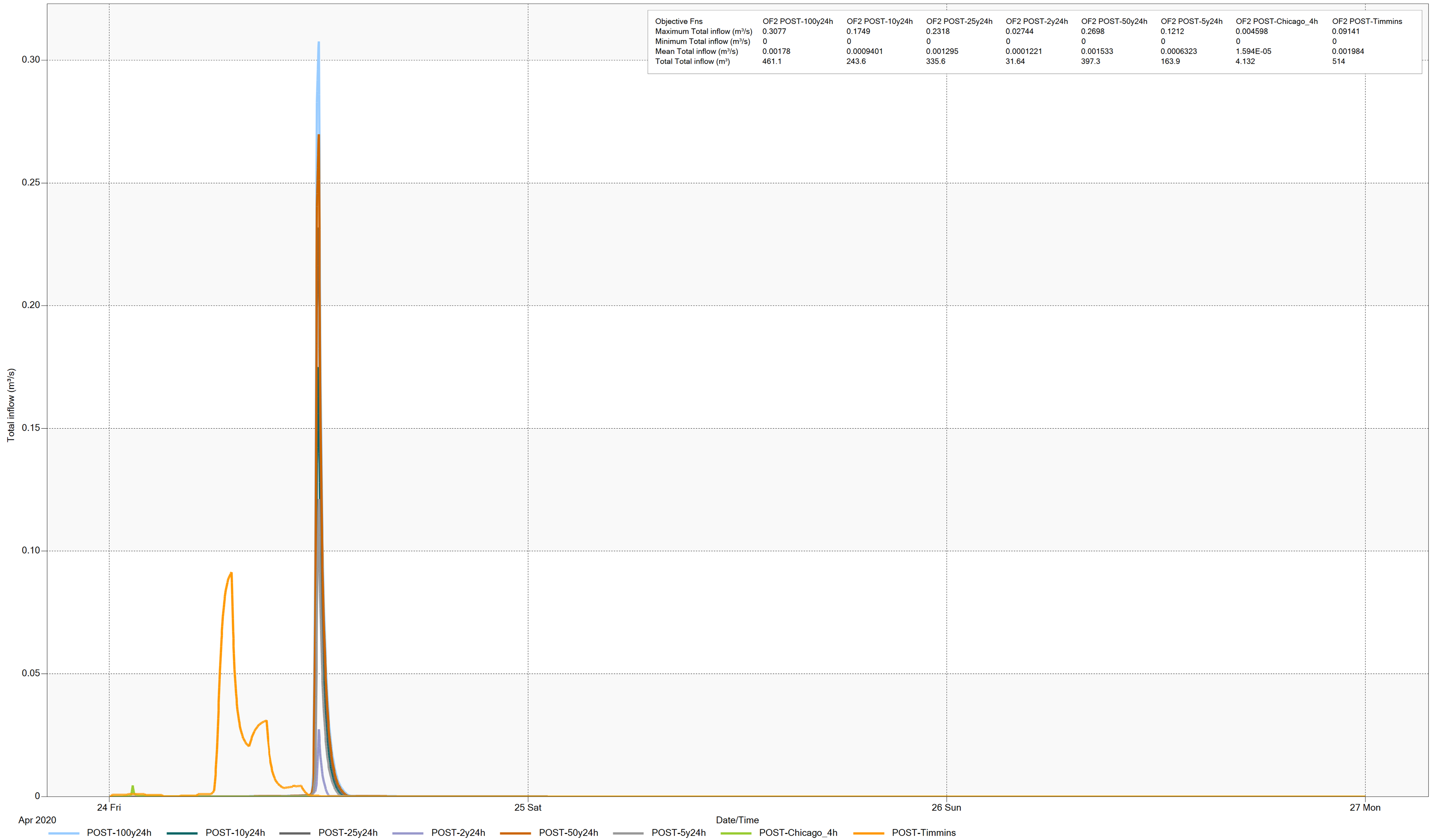
Analysis begun on: Tue Sep 22 16:34:57 2020
Analysis ended on: Tue Sep 22 16:34:58 2020
Total elapsed time: 00:00:01

# Node OF1



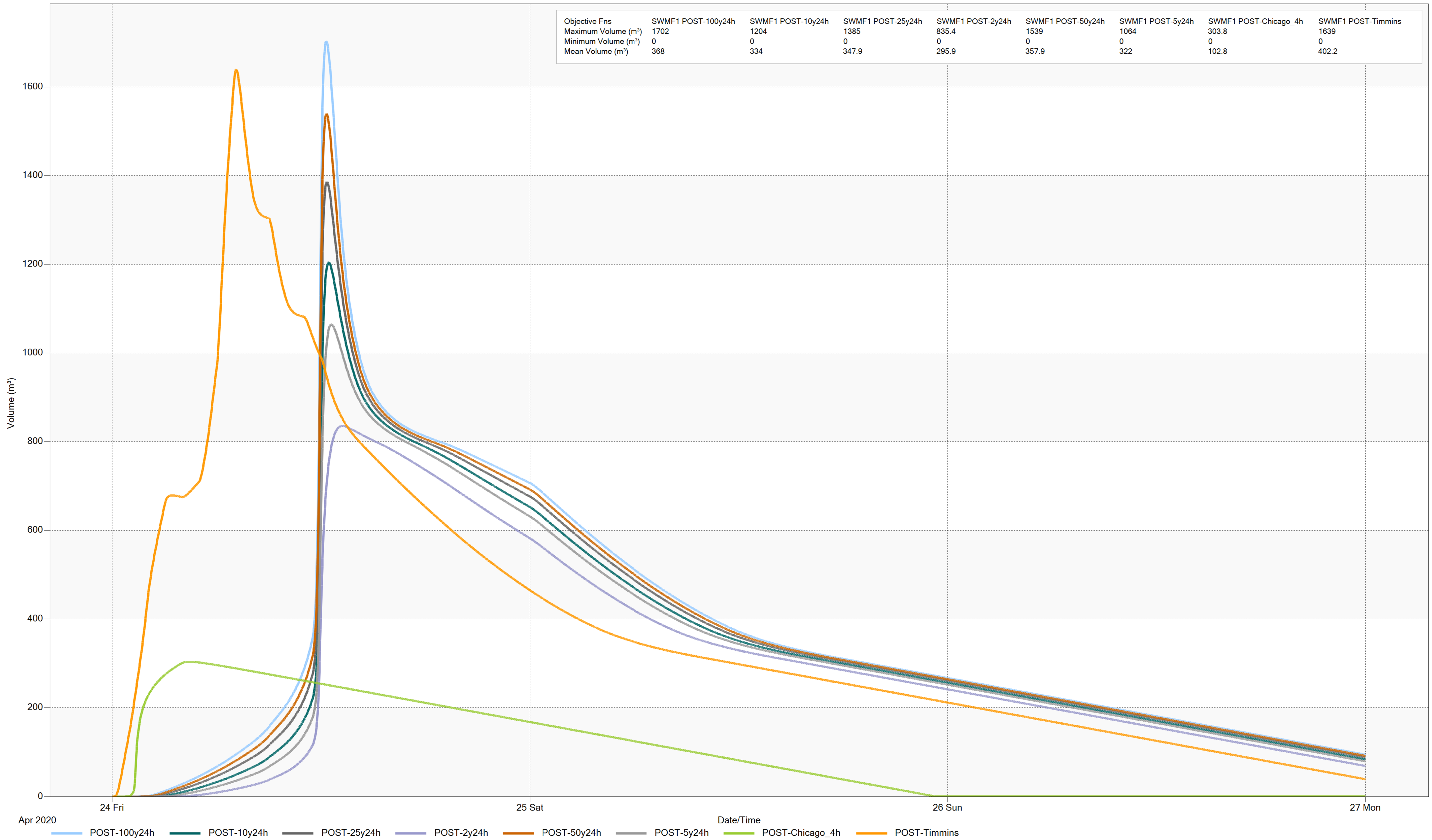
Objective Fns
OF1 POST-100y24h
OF1 POST-10y24h
OF1 POST-25y24h
OF1 POST-2y24h
OF1 POST-50y24h
OF1 POST-5y24h
OF1 POST-Chicago_4h
OF1 POST-Timmins
Maximum Total inflow (m³/s) 0.4271 0.1792 0.2831 0.05155 0.3591 0.1002 0.02175 0.3673
Minimum Total inflow (m³/s) 0 0 0 0 0 0 0 0
Mean Total inflow (m³/s) 0.0127 0.007991 0.009864 0.004358 0.01127 0.006463 0.001343 0.0208
Total Total inflow (m³) 3291 2071 2556 1129 2920 1675 348.1 5390
Total inflow (m³/s)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
24 Fri
25 Sat
26 Sun
27 Mon
Apr 2020
Date/Time
POST-100y24h
POST-10y24h
POST-25y24h
POST-2y24h
POST-50y24h
POST-5y24h
POST-Chicago_4h
POST-Timmins

# Node OF2




| Objective Fns | OF2 POST-100y24h | OF2 POST-10y24h | OF2 POST-25y24h | OF2 POST-2y24h | OF2 POST-50y24h | OF2 POST-5y24h | OF2 POST-Chicago_4h | OF2 POST-Timmins |
|---|---|---|---|---|---|---|---|---|
| Maximum Total inflow ($m^3/s$) | 0.3077 | 0.1749 | 0.2318 | 0.02744 | 0.2698 | 0.1212 | 0.004598 | 0.09141 |
| Minimum Total inflow ($m^3/s$) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Mean Total inflow ($m^3/s$) | 0.00178 | 0.0009401 | 0.001295 | 0.0001221 | 0.001533 | 0.0006323 | 1.594E-05 | 0.001984 |
| Total Total inflow ($m^3$) | 461.1 | 243.6 | 335.6 | 31.64 | 397.3 | 163.9 | 4.132 | 514 |

Node SWMF1



Objective Fns
SWMF1 POST-100y24h
SWMF1 POST-10y24h
SWMF1 POST-25y24h
SWMF1 POST-2y24h
SWMF1 POST-50y24h
SWMF1 POST-5y24h
SWMF1 POST-Chicago_4h
SWMF1 POST-Timmins
Maximum Volume (m³) 1702 1204 1385 835.4 1539 1064 303.8 1639
Minimum Volume (m³) 0 0 0 0 0 0 0 0
Mean Volume (m³) 368 334 347.9 295.9 357.9 322 102.8 402.2
Volume (m³)
1600
1400
1200
1000
800
600
400
200
0
24 Fri
25 Sat
26 Sun
27 Mon
Apr 2020
Date/Time
POST-100y24h
POST-10y24h
POST-25y24h
POST-2y24h
POST-50y24h
POST-5y24h
POST-Chicago_4h
POST-Timmins

# Appendix B

## Supporting Calculations – Sanitary Servicing

- Sanitary Sewer Design Sheet
- Sanitary Sewer Capacity Analysis (not yet available)

# SANITARY SEWER DESIGN SHEET - BEACHWOOD DEVELOPMENT LTD

**TOWN OF WASAGA BEACH**

DATE : August 2020
DESIGNED BY : VS
CHECKED BY : JB
FILE No : ROM-17026

**RESIDENTIAL POPULATION DENSITIES**

LOW DENSITY (SINGLE FAMILY/SEMI-DETACHED) = 2.6 PEOPLE/UNIT
MEDIUM DENSITY (TOWNHOUSES) = 2.6 PEOPLE/UNIT
HIGH DENSITY (APARTMENTS) = 2.6 PEOPLE/UNIT @65uph

FIRE/AMBULANCE = 35 m3/ha/d = 100 PEOPLE/HA
COMMERCIAL / INSTITUTIONAL = 35 m3/ha/d = 100 PEOPLE/HA
ELEMENTARY SCHOOL = 400 PEOPLE
SECONDARY SCHOOL = 1000 PEOPLE
SEWAGE = 350 L/DAY/CAP
INFILTRATION = 0.28l/s/ha
PEAKING FACTOR = HARMON FORMULA
DESIGN VELOCITY (MIN.) = 0.4 m/s (TOWN OF WASAGA BEACH ENGINEERING STANDARDS, MARCH 2015)

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

$$V_{ACT} > 0.4m/s$$

P= Population in thousands

PROJECT NAME: BEACHWOOD DEVELOPMENT

| LOCATION | | | | AREA (HECTARES) | | | | POPULATION | | | | | | SEWAGE FLOW | | | | PIPE DATA | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STREET | AREA No. | FROM M.H. | TO M.H. | NET OR GROSS | DIMENSIONS | DELTA AREA ha | TOTAL AREA ha | PER ha | PER LOT | DELTA LOTS | DELTA POP. | OTHER POP. | TOTAL POP. | M 2 -> 4 | SEWAGE l/s | INFILT. l/s | TOTAL l/s | **D** (mm) | S (%) | Q Full (L/S) | V Full (m/s) | V Design (m/s) | Qfull check | V check |
| St. 'A' | 16 | SAN MH 42 | SAN MH 41 | | | 0.12 | 0.12 | | 2.6 | 3 | 8 | 0 | 8 | 4.42 | 0.14 | 0.03 | 0.17 | 200 | 3.00 | 56.81 | 1.81 | 0.402 | OKAY | OKAY |
| St. 'C' | 14 | SAN MH 43 | SAN MH 44 | | | 0.13 | 0.13 | | 2.6 | 4 | 10 | 0 | 10 | 4.41 | 0.19 | 0.04 | 0.22 | 200 | 2.50 | 51.86 | 1.65 | 0.407 | OKAY | OKAY |
| St. 'A' | 15 | SAN MH 44 | SAN MH 41 | | | 0.22 | 0.35 | | 2.6 | 6 | 16 | 0 | 26 | 4.36 | 0.46 | 0.10 | 0.56 | 200 | 3.70 | 63.09 | 2.01 | 0.618 | OKAY | OKAY |
| St. 'B' | 11 | SAN MH 41 | SAN MH 40 | | | 0.10 | 0.57 | | 2.6 | 2 | 5 | 0 | 39 | 4.34 | 0.69 | 0.16 | 0.85 | 200 | 0.80 | 29.34 | 0.93 | 0.411 | OKAY | OKAY |
| St. 'B' | 10 | SAN MH 40 | SAN MH 39 | | | 0.28 | 0.85 | | 2.6 | 7 | 18 | 0 | 57 | 4.3 | 1.00 | 0.24 | 1.23 | 200 | 0.60 | 25.41 | 0.81 | 0.416 | OKAY | OKAY |
| St. 'B' | 9 | SAN MH 39 | SAN MH 38 | | | 0.20 | 1.05 | | 2.6 | 3 | 8 | 0 | 65 | 4.29 | 1.13 | 0.29 | 1.42 | 200 | 0.50 | 23.19 | 0.74 | 0.409 | OKAY | OKAY |
| St. 'B' | 8 | SAN MH 38 | SAN MH 37 | | | 0.10 | 1.15 | | 2.6 | 3 | 8 | 0 | 73 | 4.28 | 1.26 | 0.32 | 1.58 | 200 | 0.50 | 23.19 | 0.74 | 0.422 | OKAY | OKAY |
| St. 'C' | 13 | SAN MH 44 | SAN MH 47 | | | 0.37 | 0.37 | | 2.6 | 14 | 36 | 0 | 36 | 4.34 | 0.64 | 0.10 | 0.74 | 200 | 1.20 | 35.93 | 1.14 | 0.456 | OKAY | OKAY |
| St. 'C' | 12 | SAN MH 47 | SAN MH 46 | | | 0.43 | 0.8 | | 2.6 | 16 | 42 | 0 | 78 | 4.27 | 1.35 | 0.22 | 1.57 | 200 | 0.90 | 31.12 | 0.99 | 0.516 | OKAY | OKAY |
| St. 'C' | 17 | SAN MH 46 | SAN MH 45 | | | 0.07 | 0.87 | | 2.6 | 3 | 8 | 0 | 86 | 4.26 | 1.48 | 0.24 | 1.72 | 200 | 0.50 | 23.19 | 0.74 | 0.432 | OKAY | OKAY |
| St. 'C' | 7 | SAN MH 45 | SAN MH 37 | | | 0.19 | 1.06 | | 2.6 | 4 | 10 | 0 | 96 | 4.25 | 1.66 | 0.30 | 1.95 | 200 | 2.60 | 52.89 | 1.68 | 0.800 | OKAY | OKAY |
| St. 'B' | 6 | SAN MH 37 | SAN MH 36 | | | 0.18 | 2.39 | | 2.6 | 3 | 8 | 0 | 177 | 4.17 | 2.99 | 0.67 | 3.66 | 200 | 0.50 | 23.19 | 0.74 | 0.538 | OKAY | OKAY |
| St. 'B' | 5 | SAN MH 36 | SAN MH 35 | | | 0.26 | 2.65 | | 2.6 | 5 | 13 | 0 | 190 | 4.16 | 3.20 | 0.74 | 3.94 | 200 | 0.50 | 23.19 | 0.74 | 0.549 | OKAY | OKAY |
| St. 'D' | 1 | Block 42 | SAN MH 34 | | | 0.81 | 0.81 | | 2.6 | 67 | 174 | 0 | 174 | 4.17 | 2.94 | 0.23 | 3.17 | 200 | 0.50 | 23.19 | 0.74 | 0.516 | OKAY | OKAY |
| St. 'D' | 2 | Block 43 | SAN MH 34 | | | 1.29 | 1.29 | | 2.6 | 67 | 174 | 0 | 174 | 4.17 | 2.94 | 0.36 | 3.30 | 200 | 0.50 | 23.19 | 0.74 | 0.522 | OKAY | OKAY |
| St. 'D' | 3 | SAN MH 34 | SAN MH 35 | | | 0.15 | 2.25 | | 2.6 | 4 | 10 | 0 | 359 | 4.04 | 5.87 | 0.63 | 6.50 | 200 | 5.00 | 73.34 | 2.33 | 1.440 | OKAY | OKAY |
| St. 'D' | 4 | SAN MH 35 | SAN MH 76 | | | 0.17 | 5.07 | | 2.6 | 4 | 10 | 0 | 559 | 3.95 | 8.94 | 1.42 | 10.36 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | SAN MH 76 | SAN MH 84 | | | | 5.07 | | | | | | 559 | 3.95 | 8.94 | 1.42 | 10.36 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | SAN MH 84 | SAN MH 102 | | | | 5.07 | | | | | | 559 | 3.95 | 8.94 | 1.42 | 10.36 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| **Drains to Shore Lane** | | SAN MH 102 | SAN MH 407 | | | | 5.07 | | | | | | 559 | 3.95 | 8.94 | 1.42 | 10.36 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | | | | | | Total Equivalent Units | | | **215** | | | **215** | | | | | | | | | | | |

# Appendix C

## Figures and Engineering Drawings

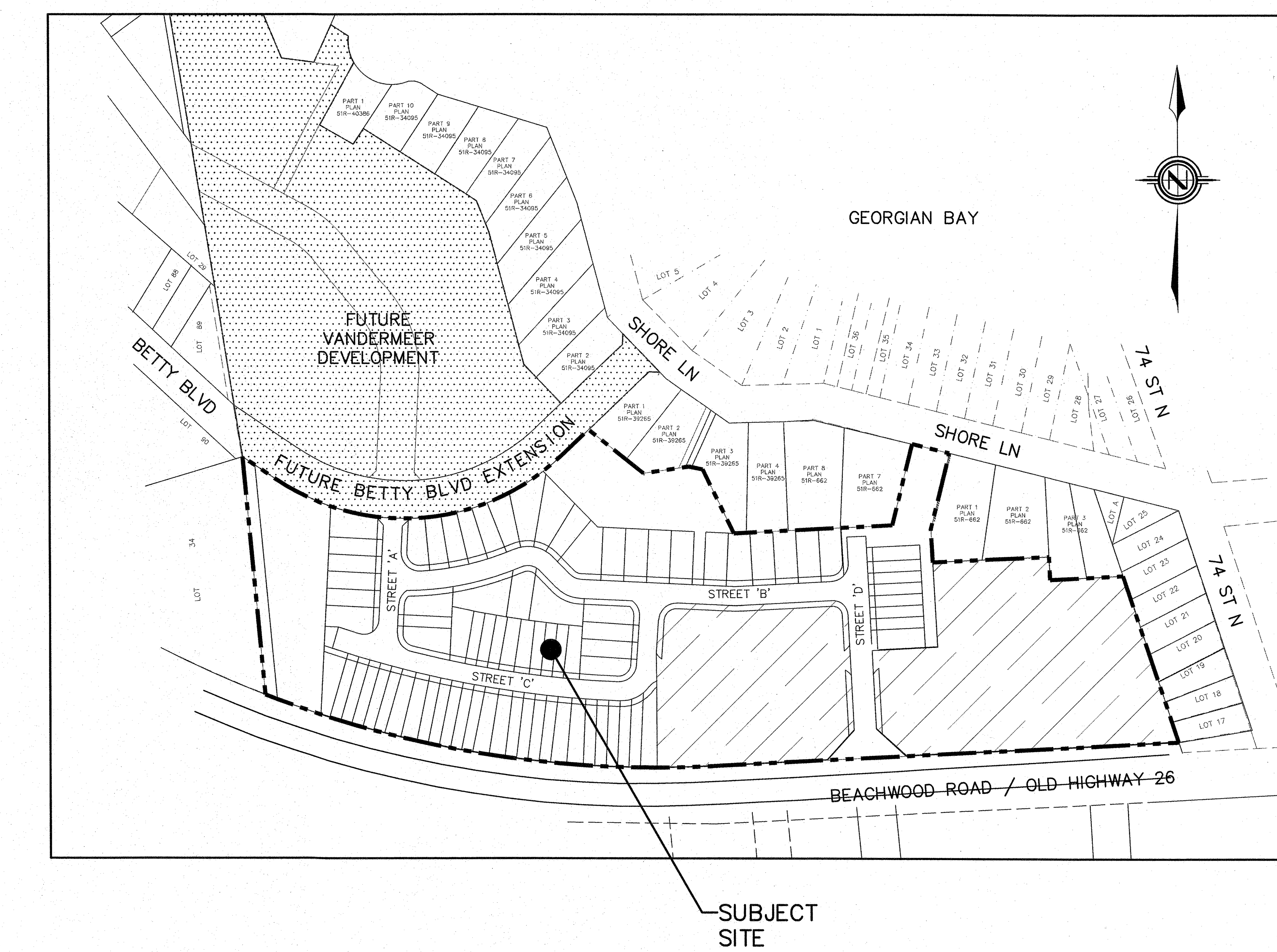
## (Reduced, NTS)

- SS-1 General Servicing Plan
- SS-2 General Servicing Plan
- SG-1 General Grading Plan
- SG-2 General Grading Plan
- STM-1 Storm Sewer Catchment Plan
- STM-2 Storm Sewer Sub-catchment Plan
- SAN-1 Internal Sanitary Drainage Area Plan
- DS-1 Storm Design Sheet
- DS-2 Sanitary Design Sheet
- SWM-1 Stormwater Management Plan Pre-Development Catchment Area
- SWM-2 Stormwater Management Plan Post-Development Catchment Area
- PND-1 Stormwater Management Facility Plan
- PND-2 Stormwater Management Facility Sections & Details

# BEACHWOOD DEVELOPMENT
# TOWN OF WASAGA BEACH

**DRAWING LEGEND**

| | |
|---|---|
| TITLE | TITLE PAGE |
| GS-1 | GENERAL SERVICING PLAN |
| GP-1 | GRADING PLAN |
| GP-2 | GRADING PLAN |
| PP-1 | PROFILE – STREETS 'A' & 'D', DRAINAGE EASEMENT |
| PP-2 | PROFILE – STREETS 'B' & 'C' |
| SAN-1 | SANITARY DRAINAGE SUB-CATCHMENT AREAS |
| STM-1 | STORMWATER SUB-CATCHMENT PLAN |
| DS-1 | STORM SEWER DESIGN SHEET |
| DS-2 | SANITARY SEWER DESIGN SHEET |
| SWM-1 | STORMWATER MANAGEMENT PRE-DEVELOPMENT AREAS |
| SWM-2 | OVERALL STORMWATER MANAGEMENT POST-DEVELOPMENT AREAS |
| WM-1 | WATERMAIN SWABBING PLAN |
| PND-1 | SWM FACILITY PLAN VIEW |
| PND-2 | SWM FACILITY SECTIONS & DETAILS |
| DE-1 | WESTERN DRAINAGE EASEMENT |
| DE-2 | EASTERN DRAINAGE EASEMENT |





**MUNICIPALITY:**

TOWN OF WASAGA BEACH
30 LEWIS STREET
WASAGA BEACH, ON L9Z 2K5
PH. 705.429.3844

**CLIENT:**

BEACHWOOD DEVELOPMENT INC.
60 PIPPIN ROAD, UNIT 46
VAUGHAN, ON L4K 4M8

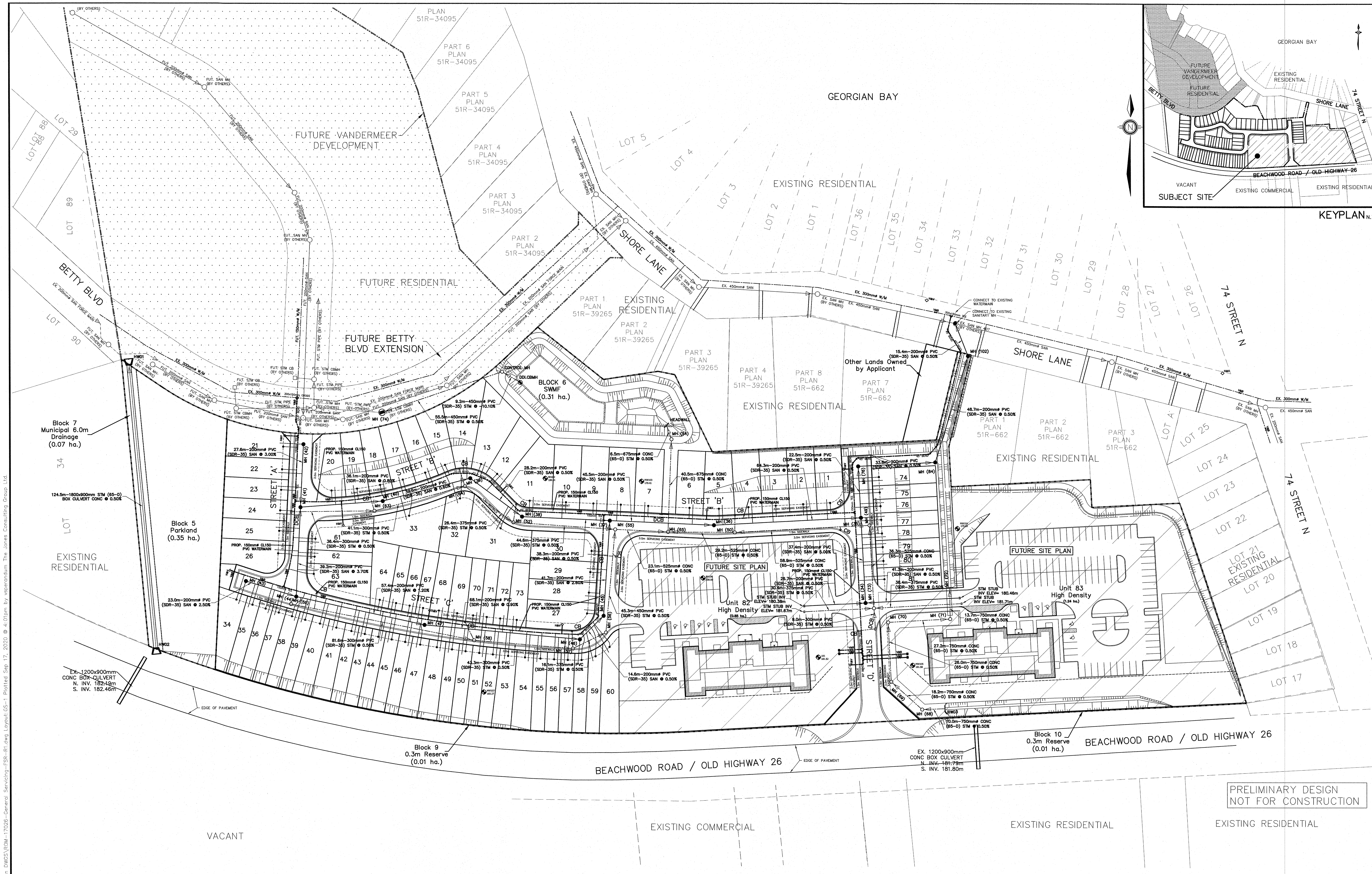
**DEVELOPER'S ENGINEER:**

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056


TJCG PROJECT # ROM-17026

PRELIMINARY DESIGN – FUNCTIONAL SERVICING REPORT SEP 2020

G:\Eng_30\ROM-17026\Production DWGS\ROM-17026-COVER.dwg Layout:Title Plotted Sep 17, 2020 @ 4:00pm by vsperandum The Jones Consulting Group Ltd.




**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE. N4924207.508 E569777.320 ELEV 184

BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL. N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

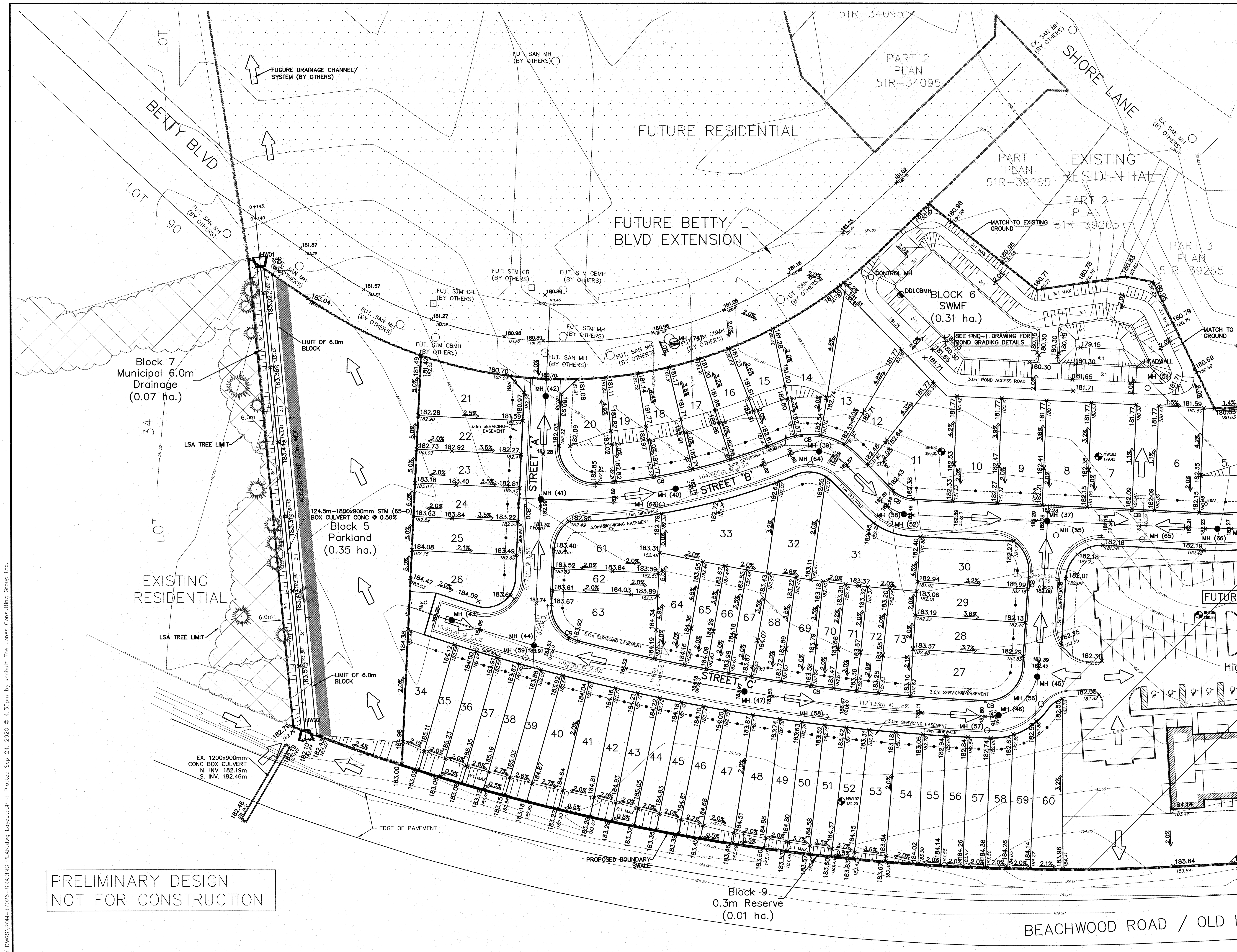
LICENSED PROFESSIONAL ENGINEER D. F. RICHARDSON 100112707 20-01-24 PROVINCE OF ONTARIO

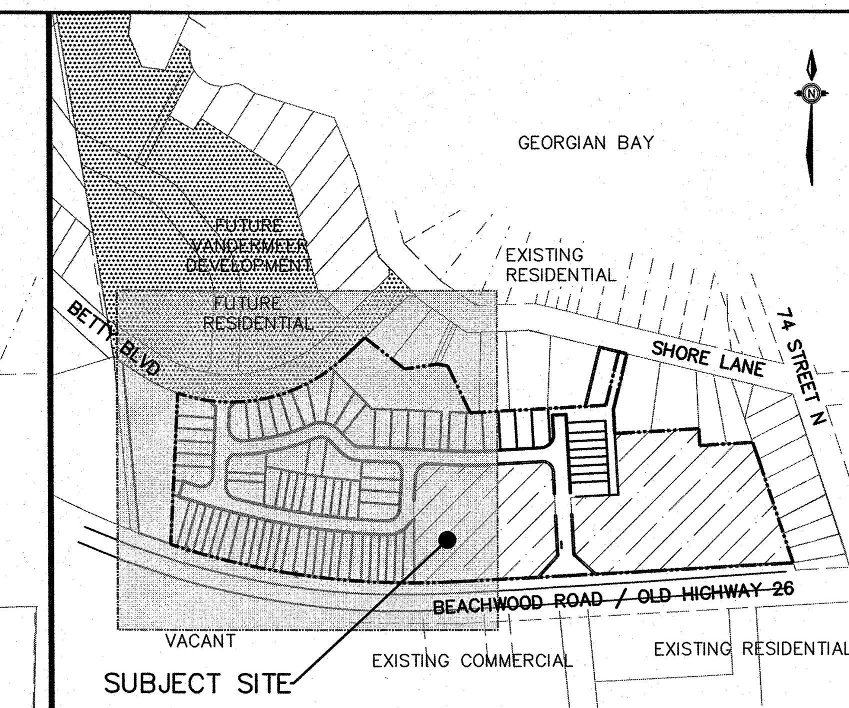
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

GENERAL SERVICING PLAN

JONES CONSULTING GROUP LTD. PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1 Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| DESIGN JT/VS | SCALE: 1:750 | DATE JUL 2020 |
|---|---|---|
| DRAWN JT/VS | PROJECT ROM-17026 | DWG. Nº GS-1 |
| CHECKED JB | | |




KEYPLAN
N.T.S

## LEGEND

LIMIT OF SUBDIVISION
EX. CONTOUR AND ELEVATION
SAN MANHOLE — MH (05)
STORM MANHOLE — MH (01)
CATCHBASIN — CB
DOUBLE CATCHBASIN — DCB
SLOPE (3:1 MAX.)
EX. BOREHOLE LOCATION ID# & GROUNDWATER ELEVATION
MAJOR OVERLAND FLOW ROUTE
DRAINAGE CHANNEL ℄
FINISHED GROUND ELEVATION — 183.53
EXISTING GROUND ELEVATION — 182.48
WATER VALVE AND BOX — V&B
HYDRANT AND VALVE — H&V
EXISTING TREES

BENCHMARK:

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE. N4924207.508 E569777.320 ELEV 184

BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL. N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

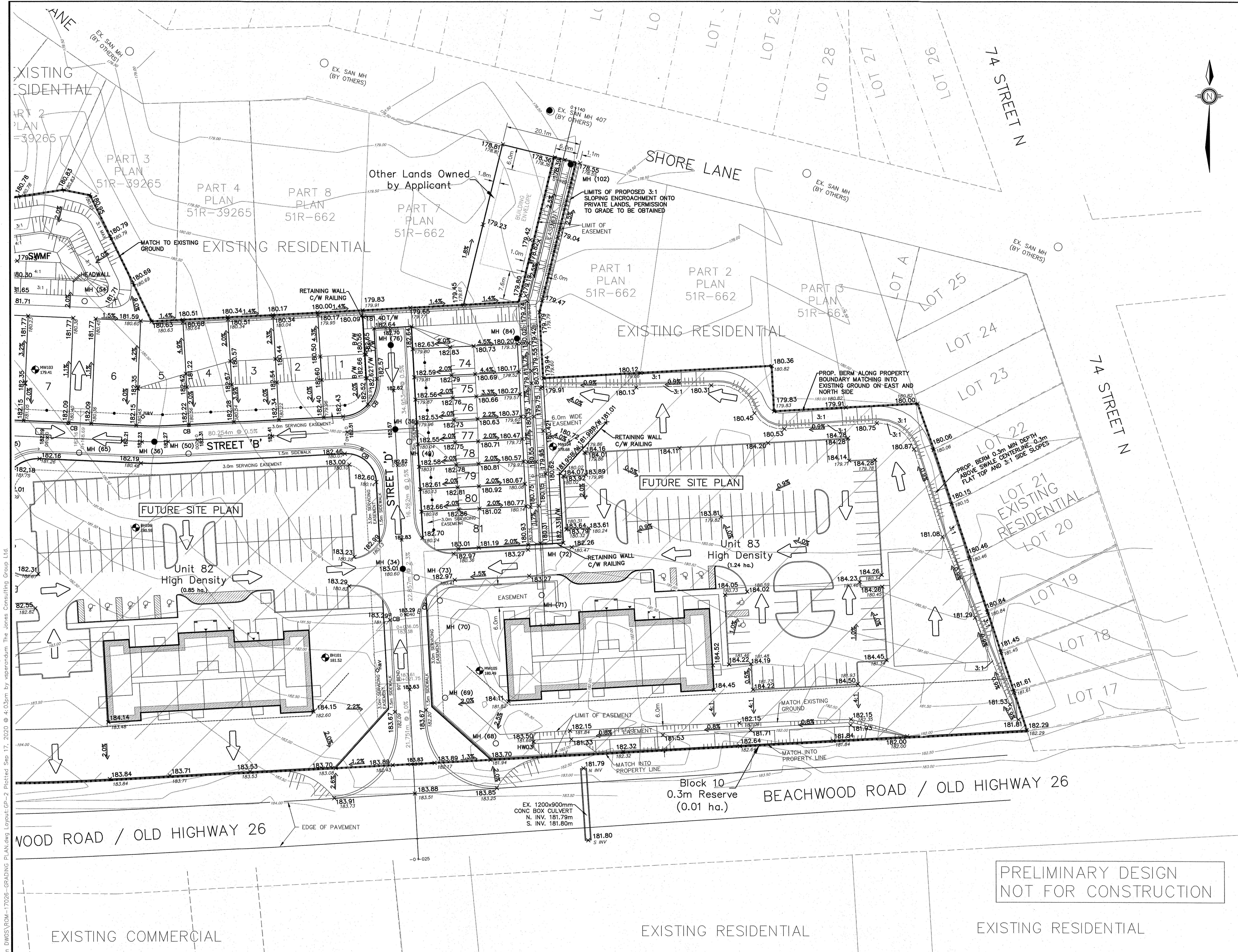
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

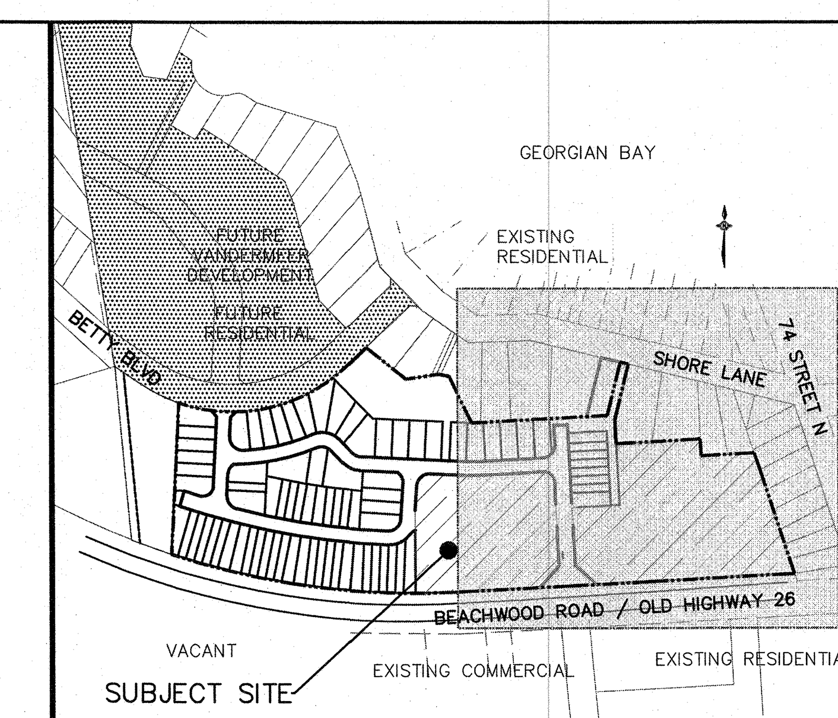
GRADING PLAN

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | | |
|---|---|---|---|
| DESIGN | JT/VS | SCALE: 1:500 | DATE JULY 2020 |
| DRAWN | JT/VS | PROJECT | DWG. Nº |
| CHECKED | JB | ROM-17026 | GP-1 |

G:\Eng_3D\ROM-17026\Production DWGS\ROM-17026-GRADING PLAN.dwg Layout:GP-1 Plotted Sep 24, 2020 @ 4:35pm by kschulz The Jones Consulting Group Ltd.




KEYPLAN N.T.S

## LEGEND

- LIMIT OF SUBDIVISION
- EX. CONTOUR AND ELEVATION
- SAN MANHOLE — MH (05)
- STORM MANHOLE — MH (01)
- CATCHBASIN — CB
- DOUBLE CATCHBASIN — DCB
- SLOPE (3:1 MAX.)
- EX. BOREHOLE LOCATION ID# & GROUNDWATER ELEVATION — BH7 254.60
- MAJOR OVERLAND FLOW ROUTE
- DRAINAGE CHANNEL ℄
- FINISHED GROUND ELEVATION — 183.53
- EXISTING GROUND ELEVATION — 182.48
- WATER VALVE AND BOX — V&B
- HYDRANT AND VALVE — H&V
- EXISTING TREES

**BENCHMARK:**

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N4924007.xxx E571589.xxx ELEV 180.020

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N4923771.xxx E569779.xxx ELEV. 183.592

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LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
24-09-20
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

GRADING PLAN

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
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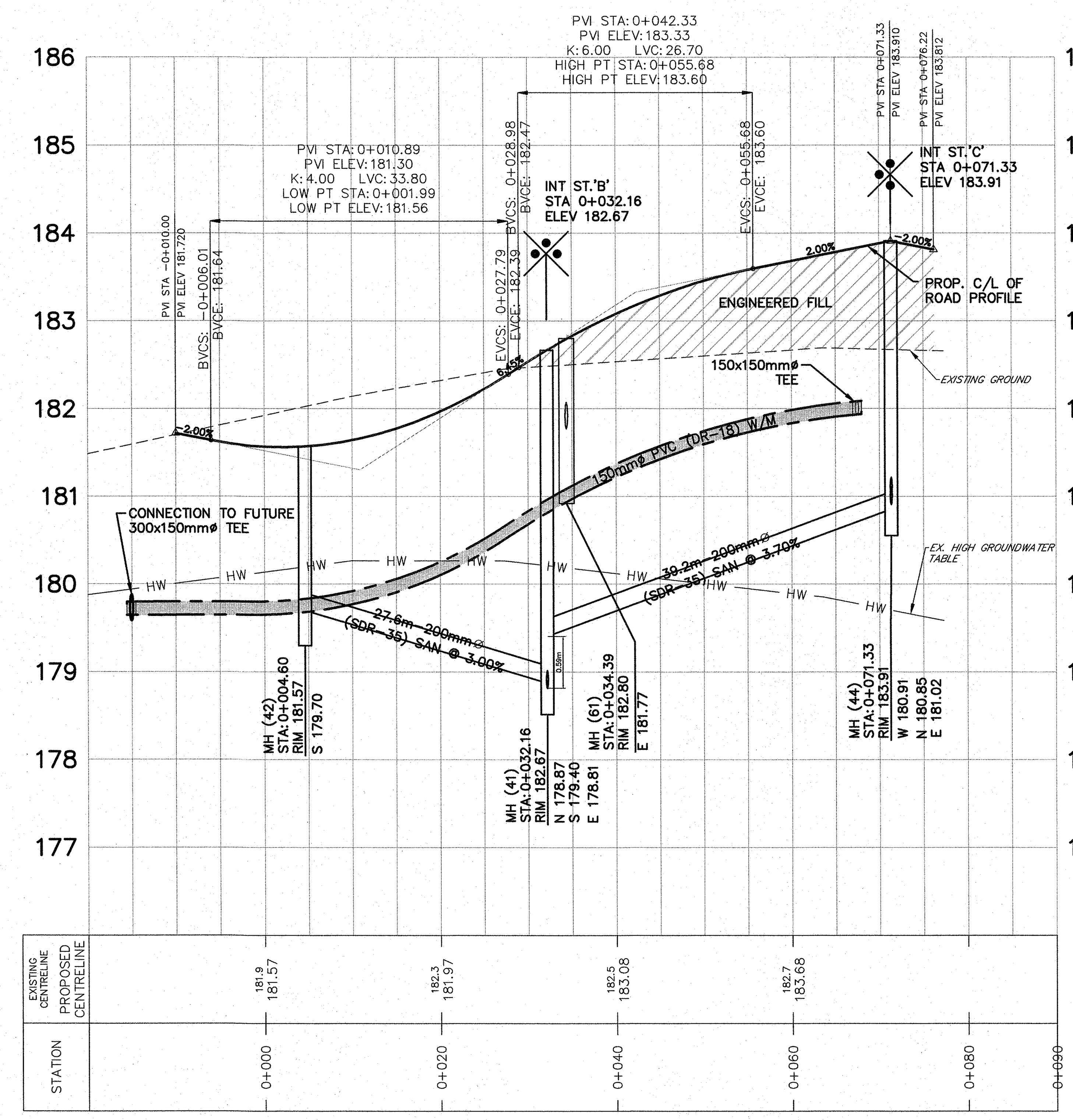
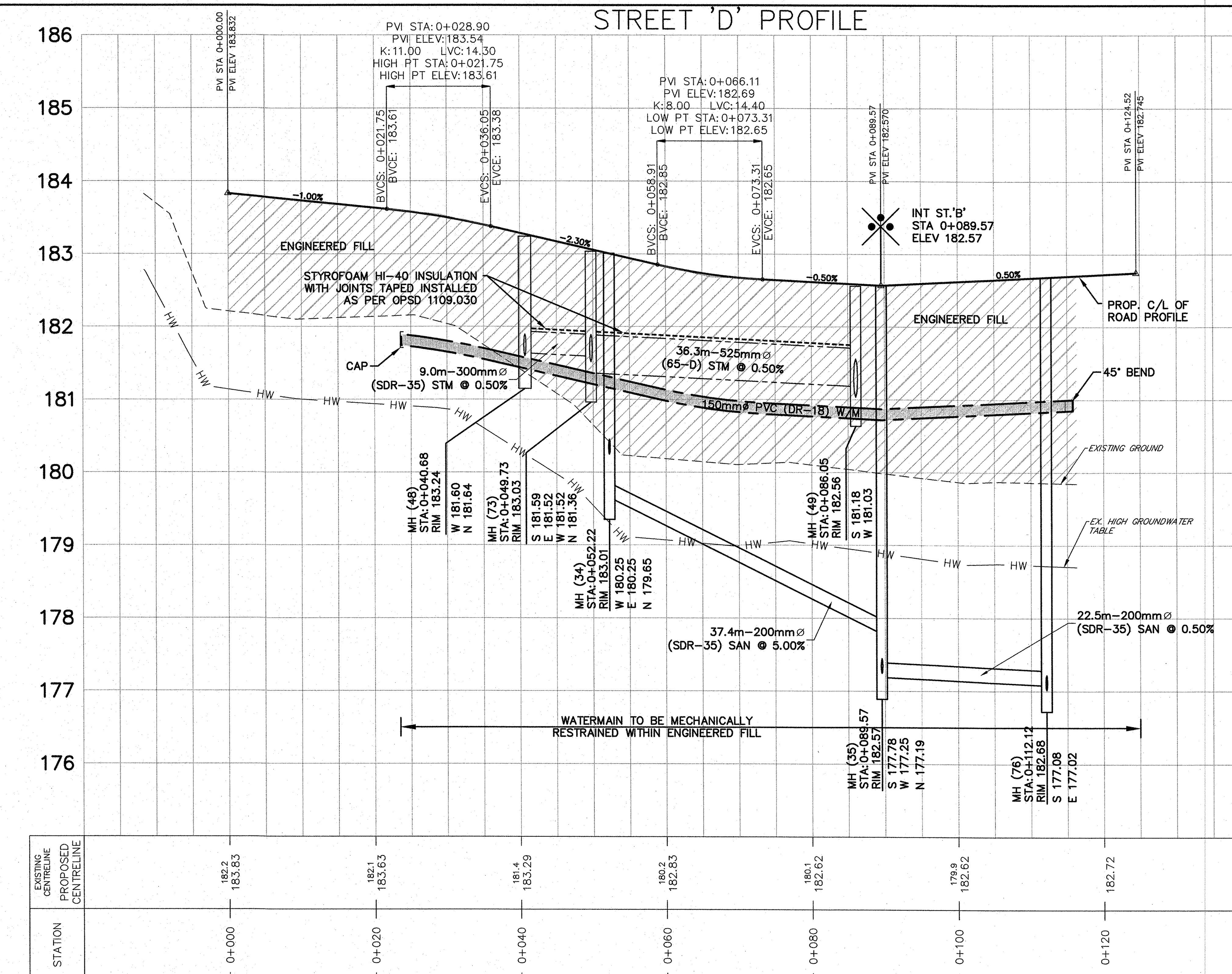
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN JT/VS | SCALE: 1:500 | DATE JULY 2020 |
| DRAWN JT/VS | PROJECT ROM-17026 | DWG. № GP-2 |
| CHECKED JB | | |

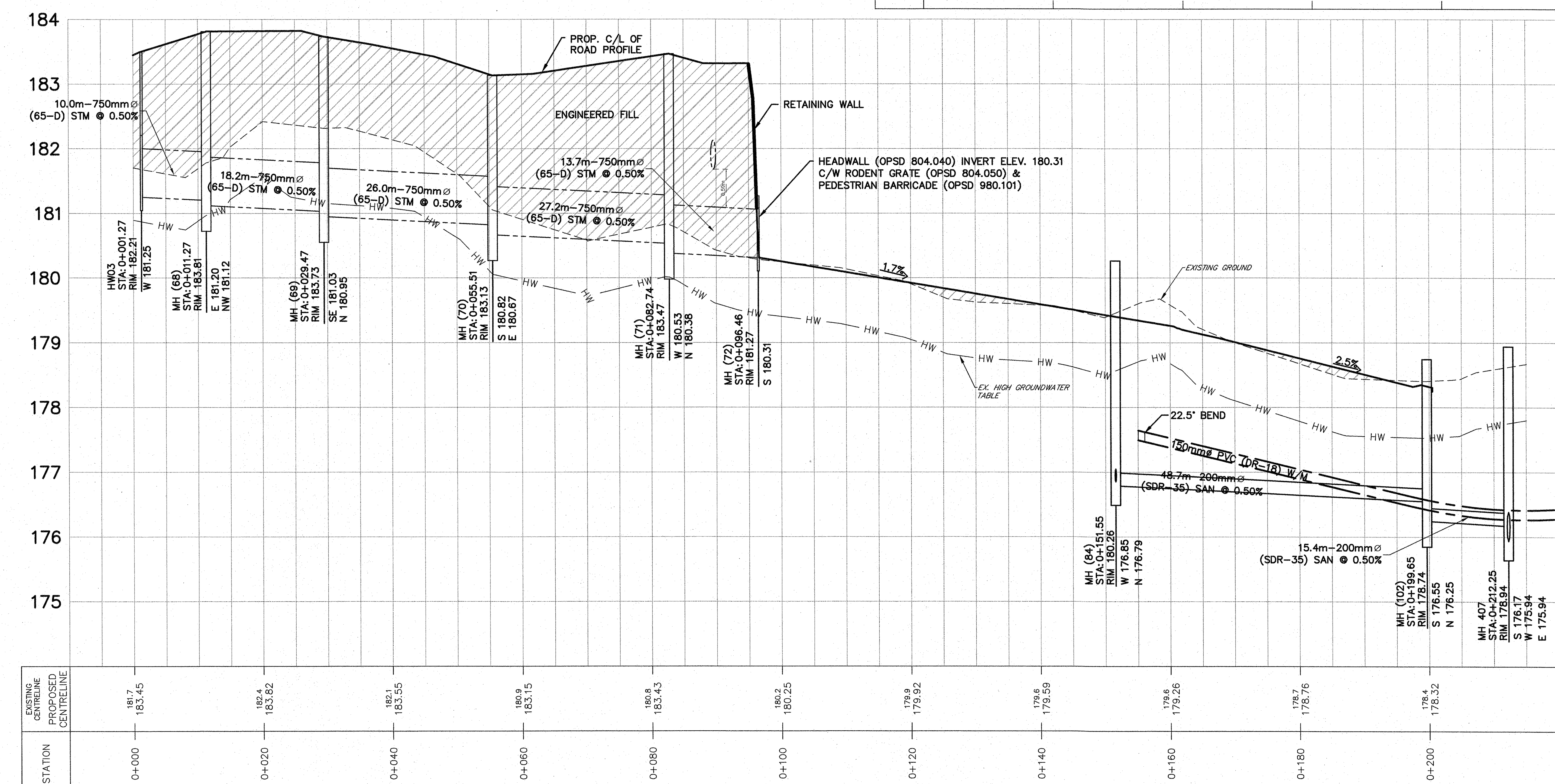
## STREET 'A' PROFILE




## STREET 'D' PROFILE




## DRAINAGE EASEMENT PROFILE




PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

**BENCHMARK:**

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

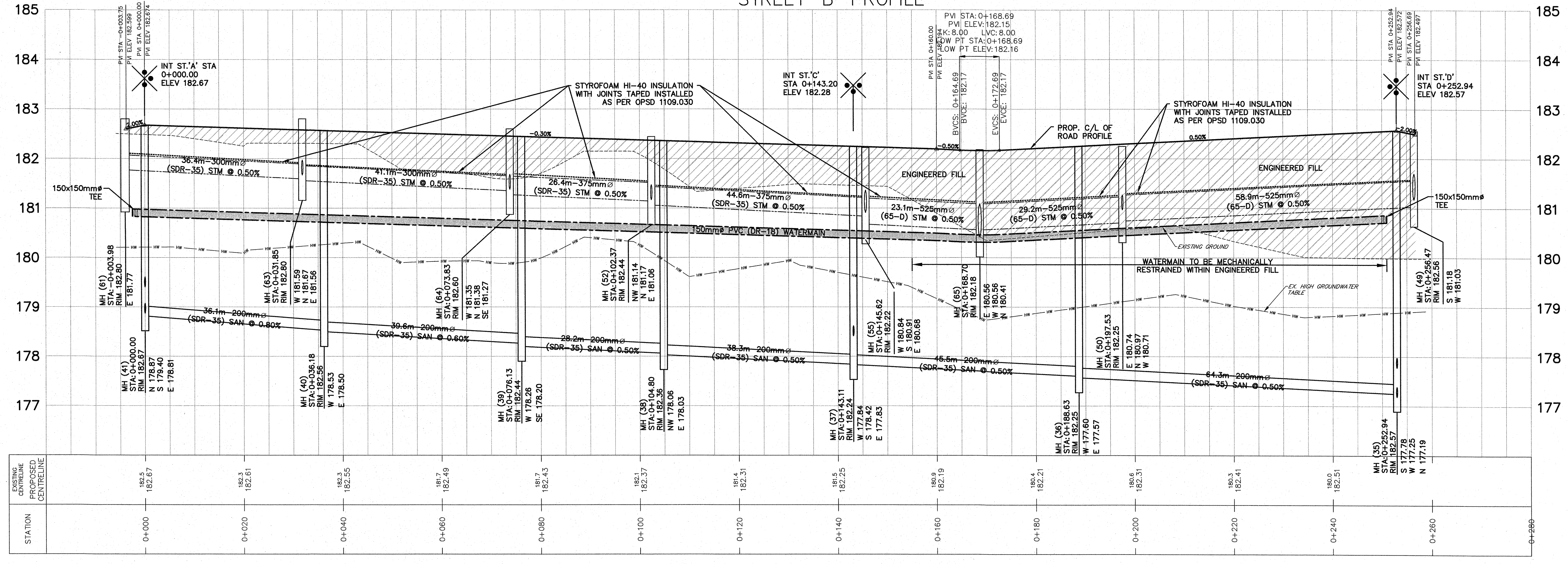
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

PROFILE
STREETS 'A' & 'D'
DRAINAGE EASEMENT

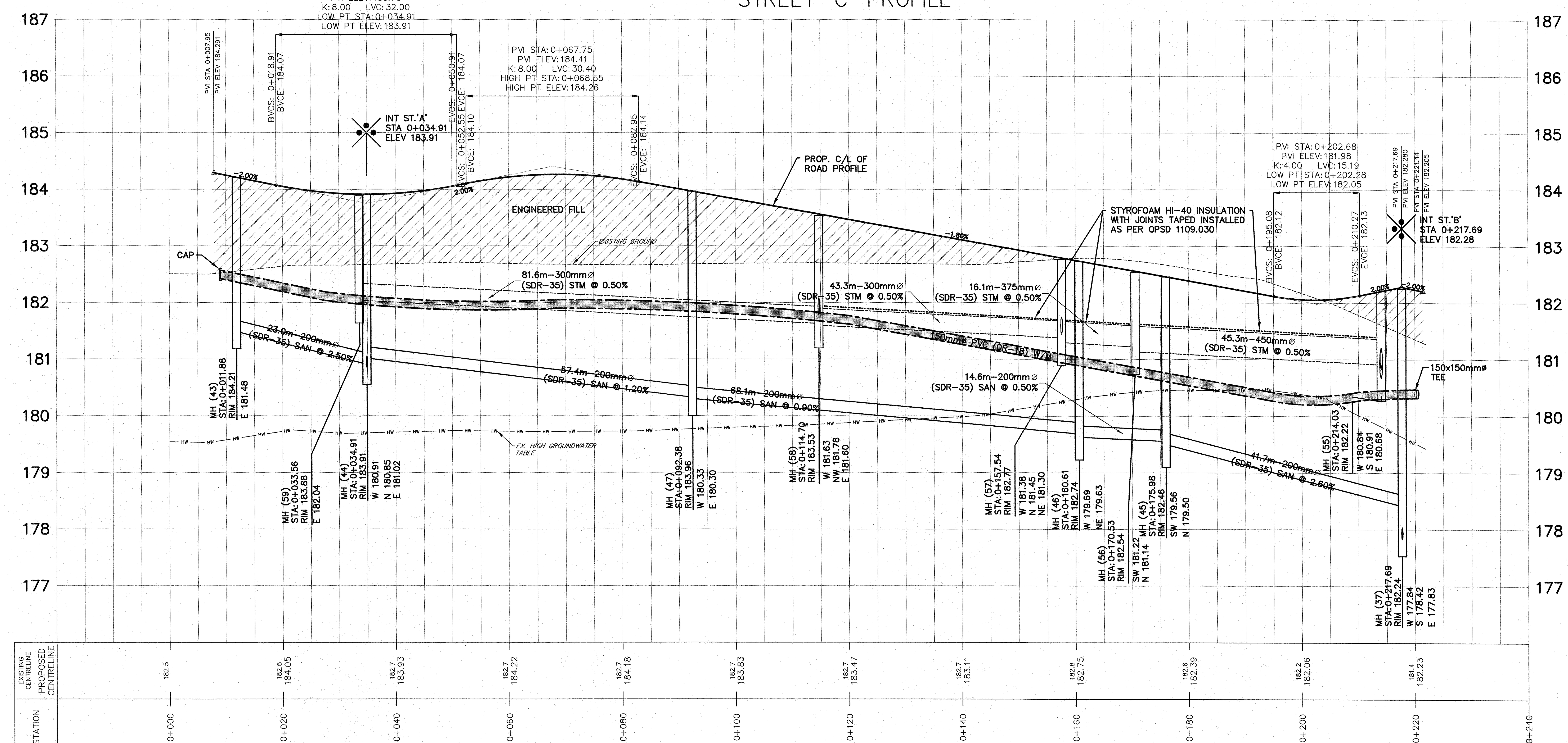
JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | | |
|---|---|---|---|
| DESIGN VS | SCALE: H 1:500 V 1:50 | DATE JULY 2020 | |
| DRAWN VS | PROJECT | DWG. № | |
| CHECKED JB | ROM–17026 | PP–1 | |

## STREET 'B' PROFILE




## STREET 'C' PROFILE




PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

**BENCHMARK:**

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|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

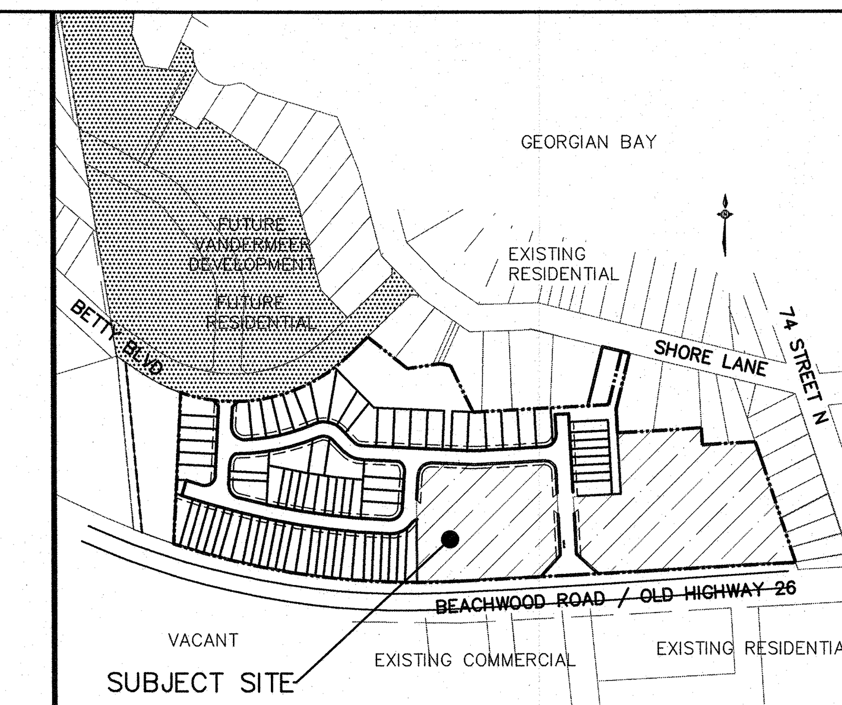
LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

PROFILE
STREETS 'B' & 'C'

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | | |
|---|---|---|---|
| DESIGN VS | SCALE: H 1:500 V 1:50 | DATE JULY 2020 | |
| DRAWN VS | PROJECT ROM-17026 | DWG. № PP-2 | |
| CHECKED JB | | | |


KEYPLAN N.T.S

## LEGEND

MH89A SANITARY MAINTENANCE HOLE

SANITARY SEWER & DIRECTION OF FLOW

SANITARY SERVICE

SUB CATCHMENT BOUNDARY

DIRECTION OF FLOW

AREA (ha.) 0.42 | PERSONS PER UNIT 3.13

No. OF UNITS 10 | CATCHMENT AREA ID NUMBER 13

PROPERTY LIMITS

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

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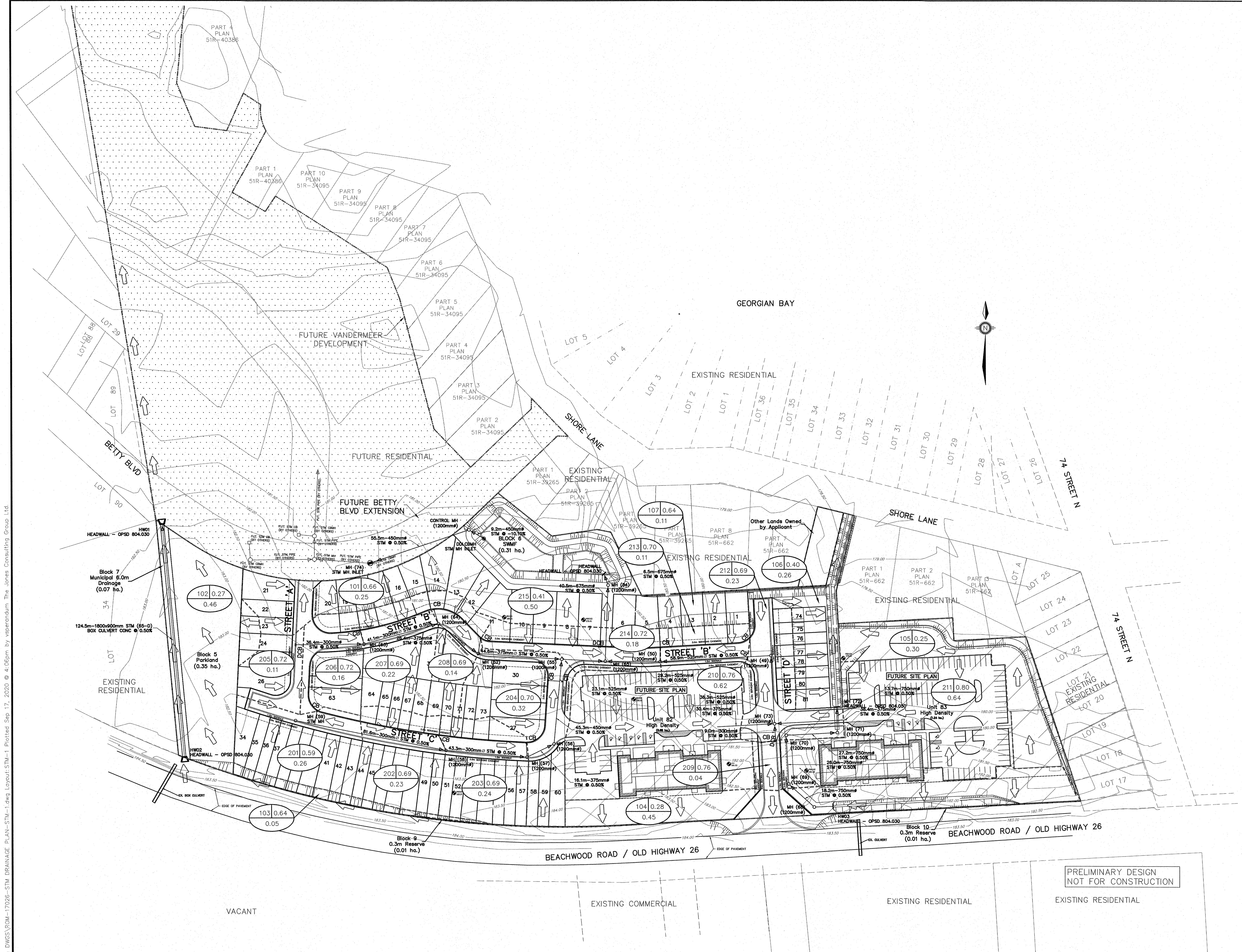
LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

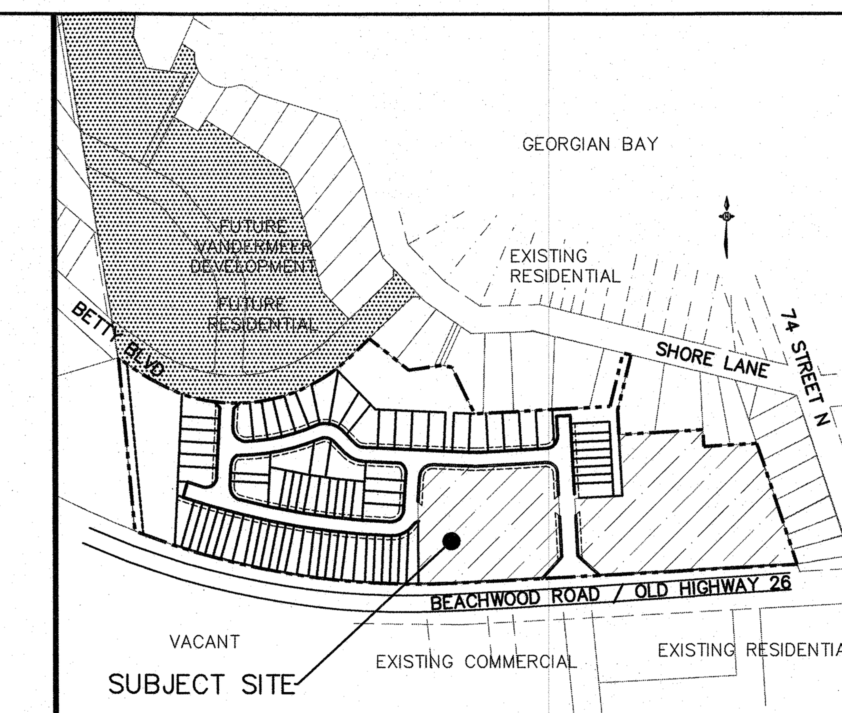
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

SANITARY DRAINAGE
SUB-CATCHMENT AREAS

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN VS | SCALE: 1:1000 | DATE APRIL 2020 |
| DRAWN KC/VS | PROJECT ROM-17026 | DWG. Nº SAN-1 |
| CHECKED JB | | |




KEYPLAN N.T.S

## LEGEND

- PROPERTY LIMITS
- EXISTING DITCH LINE
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- POST-DEVELOPMENT STORM SUB-CATCHMENT BOUNDARY
- STORM SEWER, MAINTENANCE HOLE AND DIRECTION OF FLOW
- STM SINGLE CATCHBASIN
- STORM DOUBLE CATCHBASIN
- RUN-OFF COEFFICIENT
- STORM CATCHMENT NUMBER
- AREA (HECTARES)
- DIRECTION OF STORM DRAINAGE

BENCHMARK:

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LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

STORMWATER
SUB-CATCHMENT PLAN

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN JB | SCALE: 1:1000 | DATE APR 2020 |
| DRAWN KC/VS | PROJECT ROM-17026 | DWG. Nº STM-1 |
| CHECKED JB | | |

## 5 Year Storm Sewer Sizing

| STREET | AREAS | MANHOLE | | LENGTH | INCREMENT | | | TOTAL | FLOW TIME (min) | | I | TOTAL Q | S | D | Q FULL | V FULL | % FULL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | FROM | TO | (m) | C | A | CA | CA | TO | IN | (mm/h) | (cms) | (%) | (mm) | (cms) | (m/s) | |
| St. 'C' | 201 | STM 59 | STM 58 | 81.6 | 0.59 | 0.26 | 0.15 | 0.15 | 15.00 | 1.41 | 74 | 0.031 | **0.50** | **300** | 0.068 | 1.0 | **45.9** |
| St. 'C' | 202 | STM 58 | STM 57 | 43.3 | 0.69 | 0.23 | 0.16 | 0.31 | 16.41 | 0.75 | 69 | 0.060 | **0.50** | **300** | 0.068 | 1.0 | **87.6** |
| St. 'C' | 203 | STM 57 | STM 56 | 16.1 | 0.69 | 0.24 | 0.17 | 0.48 | 17.15 | 0.24 | 67 | 0.089 | **0.50** | **375** | 0.124 | 1.1 | **71.7** |
| St. 'C' | 204 | STM 56 | STM 55 | 45.3 | 0.70 | 0.32 | 0.22 | 0.70 | 17.39 | 0.60 | 66 | 0.129 | **0.50** | **450** | 0.202 | 1.3 | **64.2** |
| | | | | | | | | | | | | | | | | | |
| St. 'B' | 205 | DCB | STM 63 | 36.4 | 0.72 | 0.11 | 0.08 | 0.08 | 15.00 | 0.63 | 74 | 0.016 | **0.50** | **300** | 0.068 | 1.0 | **23.7** |
| St. 'B' | 206 | STM 63 | STM 64 | 41.1 | 0.72 | 0.16 | 0.12 | 0.19 | 15.63 | 0.71 | 72 | 0.039 | **0.50** | **300** | 0.068 | 1.0 | **56.5** |
| St. 'B' | 207 | STM 64 | STM 52 | 26.4 | 0.69 | 0.22 | 0.15 | 0.35 | 16.34 | 0.39 | 69 | 0.067 | **0.50** | **375** | 0.124 | 1.1 | **53.8** |
| St. 'B' | 208 | STM 52 | STM 55 | 44.6 | 0.69 | 0.14 | 0.10 | 0.44 | 16.73 | 0.66 | 68 | 0.084 | **0.50** | **375** | 0.124 | 1.1 | **67.7** |
| St. 'B' | | STM 55 | STM 65 | 23.1 | | | | 1.14 | 17.99 | 0.27 | 65 | 0.206 | **0.50** | **525** | 0.304 | 1.4 | **67.8** |
| | | | | | | | | | | | | | | | | | |
| St. 'D' | 209 | DCB | STM 73 | 9.0 | 0.76 | 0.04 | 0.03 | 0.03 | 15.00 | 0.16 | 74 | 0.006 | **0.50** | **300** | 0.068 | 1.0 | **9.1** |
| St. 'D' | 210 | HD UNIT 82 | STM 73 | 30.4 | 0.76 | 0.62 | 0.47 | 0.47 | 15.00 | 0.45 | 74 | 0.096 | **0.50** | **375** | 0.124 | 1.1 | **77.7** |
| St. 'D' | 211 | HD UNIT 83 | STM 73 | 38.4 | 0.80 | 0.64 | 0.51 | 0.51 | 15.00 | 0.57 | 74 | 0.105 | **0.50** | **375** | 0.124 | 1.1 | **84.4** |
| St. 'D' | | STM 73 | STM 49 | 36.3 | | | | 1.01 | 15.57 | 0.43 | 72 | 0.202 | **0.50** | **525** | 0.304 | 1.4 | **66.4** |
| St. 'B' | 212 | STM 49 | STM 50 | 58.9 | 0.69 | 0.23 | 0.16 | 1.17 | 16.00 | 0.70 | 70 | 0.229 | **0.50** | **525** | 0.304 | 1.4 | **75.3** |
| St. 'B' | 213 | STM 50 | STM 65 | 29.2 | 0.70 | 0.11 | 0.08 | 1.25 | 16.70 | 0.35 | 68 | 0.237 | **0.50** | **525** | 0.304 | 1.4 | **77.9** |
| | 214 | STM 65 | STM 54 | 40.5 | 0.72 | 0.18 | 0.13 | 2.52 | 18.26 | 0.41 | 64 | 0.450 | **0.50** | **675** | 0.594 | 1.7 | **75.6** |
| | | STM 54 | HEADWALL | 6.5 | | | | 2.52 | 18.67 | 0.07 | 63 | 0.443 | **0.50** | **675** | 0.594 | 1.7 | **74.5** |
| | | | | | | | | | | | | | | | | | |
| | Total | | | | | 3.50 | 2.52 | | | | | | | | | | |
| | Q= 0.0028*C*I*A (cms); | | | | C=RUNOFF COEFFICIENT; | | | | I-RAINFALL INTENSITY (5 Year) =493.013/(T.C.+0.081)^0.701 | | | | | A=AREA (ha) | | | |

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

BENCHMARK:

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| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

STORM SEWER DESIGN SHEET

JONES
CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

DESIGN VS | DRAWN KC/VS | CHECKED JB
SCALE:
DATE APR 2020
PROJECT ROM-17026
DWG. Nº DS-1

# SANITARY SEWER DESIGN SHEET - BEACHWOOD DEVELOPMENT LTD

**RESIDENTIAL POPULATION DENSITIES**

LOW DENSITY (SINGLE FAMILY/SEMI-DETACHED) = 2.6 PEOPLE/UNIT
MEDIUM DENSITY (TOWNHOUSES) = 2.6 PEOPLE/UNIT
HIGH DENSITY (APARTMENTS) = 2.6 PEOPLE/UNIT @65uph

**TOWN OF WASAGA BEACH**

DATE : August 2020
DESIGNED BY : VS
CHECKED BY : JB
FILE No : ROM-17026

FIRE/AMBULANCE = 35 m3/ha/d = 100 PEOPLE/HA
COMMERCIAL / INSTITUTIONAL = 35 m3/ha/d = 100 PEOPLE/HA
ELEMENTARY SCHOOL = 400 PEOPLE
SECONDARY SCHOOL = 1000 PEOPLE
SEWAGE = 350 L/DAY/CAP
INFILTRATION = 0.28l/s/ha
PEAKING FACTOR = HARMON FORMULA
DESIGN VELOCITY (MIN.) = 0.4 m/s (TOWN OF WASAGA BEACH ENGINEERING STANDARDS, MARCH 2015)

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

$$V_{ACT} > 0.4 m/s$$

P= Population in thousands

PROJECT NAME: BEACHWOOD DEVELOPMENT

| LOCATION | | | | | AREA (HECTARES) | | | POPULATION | | | | | | | SEWAGE FLOW | | | | PIPE DATA | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STREET | AREA | FROM | TO | NET OR | DIMENSIONS | DELTA | TOTAL | PER | PER | DELTA | DELTA | OTHER | TOTAL | | M | SEWAGE | INFILT. | TOTAL | D | S | Q Full | V Full | V Design | Qfull check | V check |
| | No. | M.H. | M.H. | GROSS | | AREA ha | AREA ha | ha | LOT | LOTS | POP. | POP. | POP. | | 2 -> 4 | l/s | l/s | l/s | (mm) | (%) | (L/S) | (m/s) | (m/s) | | |
| St. 'A' | 16 | SAN MH 42 | SAN MH 41 | | | 0.12 | 0.12 | | 2.6 | 3 | 8 | 0 | 8 | | 4.42 | 0.14 | 0.03 | 0.17 | 200 | 3.00 | 56.81 | 1.81 | 0.402 | OKAY | OKAY |
| St. 'C' | 14 | SAN MH 43 | SAN MH 44 | | | 0.13 | 0.13 | | 2.6 | 4 | 10 | 0 | 10 | | 4.41 | 0.19 | 0.04 | 0.22 | 200 | 2.50 | 51.86 | 1.65 | 0.407 | OKAY | OKAY |
| St. 'A' | 15 | SAN MH 44 | SAN MH 41 | | | 0.22 | 0.35 | | 2.6 | 6 | 16 | 0 | 26 | | 4.36 | 0.46 | 0.10 | 0.56 | 200 | 3.70 | 63.09 | 2.01 | 0.618 | OKAY | OKAY |
| St. 'B' | 11 | SAN MH 41 | SAN MH 40 | | | 0.10 | 0.57 | | 2.6 | 2 | 5 | 0 | 39 | | 4.34 | 0.69 | 0.16 | 0.85 | 200 | 0.80 | 29.34 | 0.93 | 0.411 | OKAY | OKAY |
| St. 'B' | 10 | SAN MH 40 | SAN MH 39 | | | 0.28 | 0.85 | | 2.6 | 7 | 18 | 0 | 57 | | 4.3 | 1.00 | 0.24 | 1.23 | 200 | 0.60 | 25.41 | 0.81 | 0.416 | OKAY | OKAY |
| St. 'B' | 9 | SAN MH 39 | SAN MH 38 | | | 0.20 | 1.05 | | 2.6 | 3 | 8 | 0 | 65 | | 4.29 | 1.13 | 0.29 | 1.42 | 200 | 0.50 | 23.19 | 0.74 | 0.409 | OKAY | OKAY |
| St. 'B' | 8 | SAN MH 38 | SAN MH 37 | | | 0.10 | 1.15 | | 2.6 | 3 | 8 | 0 | 73 | | 4.28 | 1.26 | 0.32 | 1.58 | 200 | 0.50 | 23.19 | 0.74 | 0.422 | OKAY | OKAY |
| St. 'C' | 13 | SAN MH 44 | SAN MH 47 | | | 0.37 | 0.37 | | 2.6 | 14 | 36 | 0 | 36 | | 4.34 | 0.64 | 0.10 | 0.74 | 200 | 1.20 | 35.93 | 1.14 | 0.456 | OKAY | OKAY |
| St. 'C' | 12 | SAN MH 47 | SAN MH 46 | | | 0.41 | 0.78 | | 2.6 | 16 | 42 | 0 | 78 | | 4.27 | 1.35 | 0.22 | 1.57 | 200 | 0.90 | 31.12 | 0.99 | 0.516 | OKAY | OKAY |
| St. 'C' | 17 | SAN MH 46 | SAN MH 45 | | | 0.09 | 0.87 | | 2.6 | 3 | 8 | 0 | 86 | | 4.26 | 1.48 | 0.24 | 1.72 | 200 | 0.50 | 23.19 | 0.74 | 0.432 | OKAY | OKAY |
| St. 'C' | 7 | SAN MH 45 | SAN MH 37 | | | 0.19 | 1.06 | | 2.6 | 4 | 10 | 0 | 96 | | 4.25 | 1.66 | 0.30 | 1.95 | 200 | 2.60 | 52.89 | 1.68 | 0.800 | OKAY | OKAY |
| St. 'B' | 6 | SAN MH 37 | SAN MH 36 | | | 0.18 | 2.39 | | 2.6 | 3 | 8 | 0 | 177 | | 4.17 | 2.99 | 0.67 | 3.66 | 200 | 0.50 | 23.19 | 0.74 | 0.538 | OKAY | OKAY |
| St. 'B' | 5 | SAN MH 36 | SAN MH 35 | | | 0.25 | 2.64 | | 2.6 | 5 | 13 | 0 | 190 | | 4.16 | 3.20 | 0.74 | 3.94 | 200 | 0.50 | 23.19 | 0.74 | 0.549 | OKAY | OKAY |
| St. 'D' | 1 | HD UNIT 82 | SAN MH 34 | | | 0.82 | 0.82 | | 2.6 | 67 | 174 | 0 | 174 | | 4.17 | 2.94 | 0.23 | 3.17 | 200 | 0.50 | 23.19 | 0.74 | 0.516 | OKAY | OKAY |
| St. 'D' | 2 | HD UNIT 83 | SAN MH 34 | | | 1.29 | 1.29 | | 2.6 | 67 | 174 | 0 | 174 | | 4.17 | 2.94 | 0.36 | 3.30 | 200 | 0.50 | 23.19 | 0.74 | 0.522 | OKAY | OKAY |
| St. 'D' | 3 | SAN MH 34 | SAN MH 35 | | | 0.15 | 2.26 | | 2.6 | 4 | 10 | 0 | 359 | | 4.04 | 5.87 | 0.63 | 6.50 | 200 | 5.00 | 73.34 | 2.33 | 1.440 | OKAY | OKAY |
| St. 'D' | 4 | SAN MH 35 | SAN MH 76 | | | 0.19 | 5.09 | | 2.6 | 4 | 10 | 0 | 559 | | 3.95 | 8.94 | 1.43 | 10.37 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | SAN MH 76 | SAN MH 84 | | | | 5.09 | | | | | | 559 | | 3.95 | 8.94 | 1.43 | 10.37 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | SAN MH 84 | SAN MH 102 | | | | 5.09 | | | | | | 559 | | 3.95 | 8.94 | 1.43 | 10.37 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| Drains to Shore Lane | | SAN MH 102 | SAN MH 407 | | | | 5.09 | | | | | | 559 | | 3.95 | 8.94 | 1.43 | 10.37 | 200 | 0.50 | 23.19 | 0.74 | 0.717 | OKAY | OKAY |
| | | | | | | | Total Equivalent Units | | | 215 | | | 215 | | | | | | | | | | | | |

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

BENCHMARK:

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE. N4924207.508 E569777.320 ELEV 184

BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL. N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

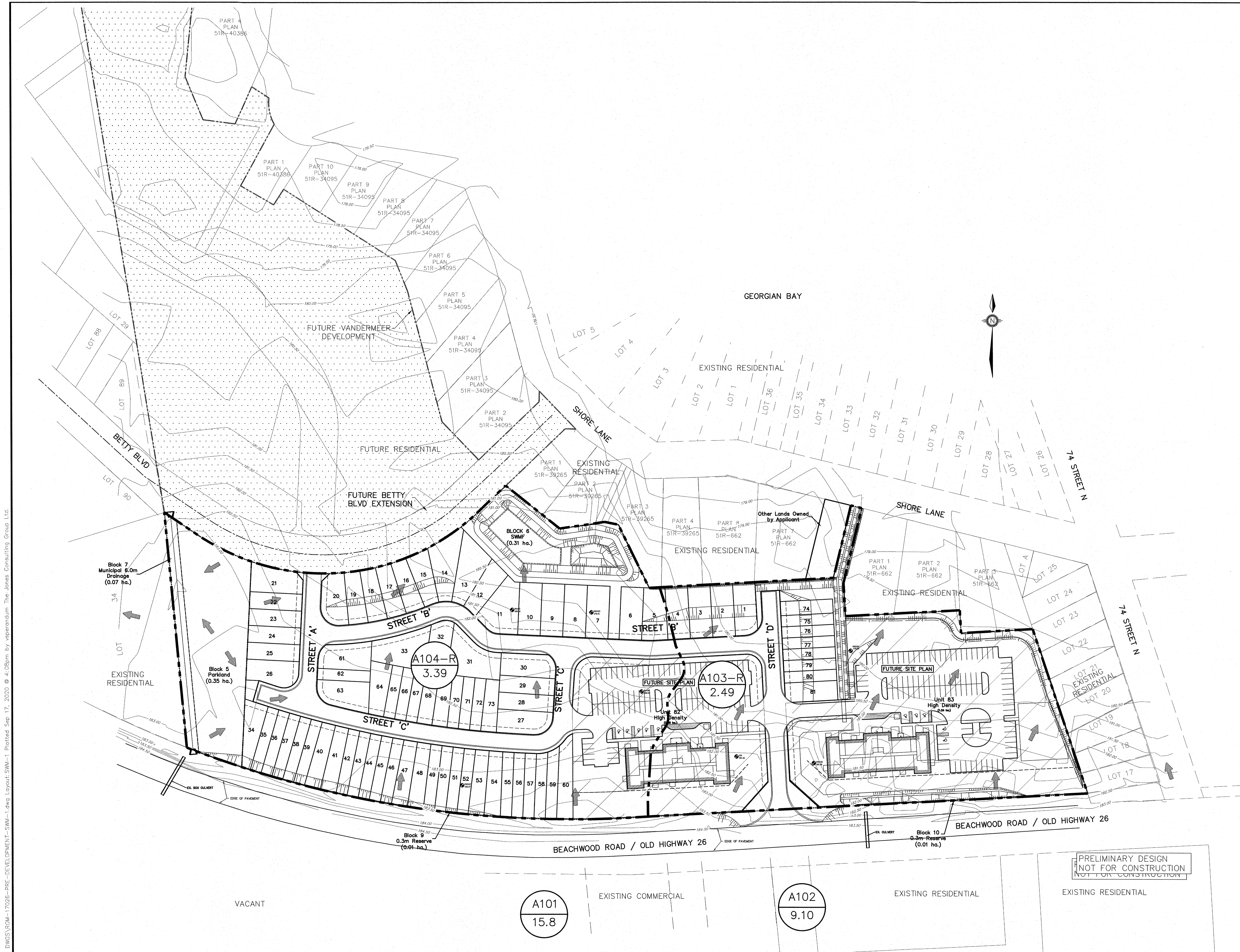
LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

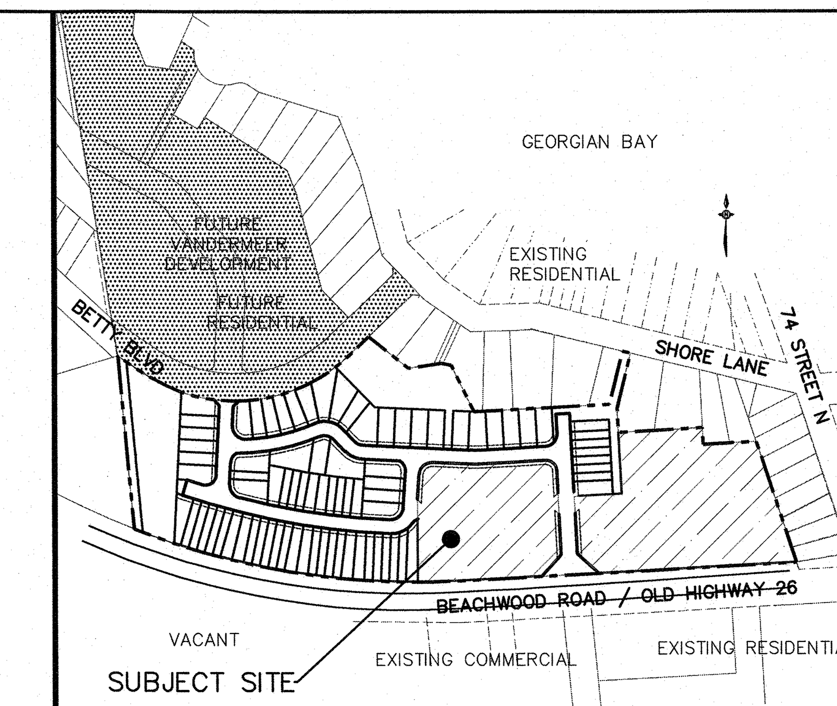
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

SANITARY SEWER DESIGN SHEET

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

DESIGN VS | SCALE: 1:1000 | DATE APRIL 2020
DRAWN KC/VS | PROJECT ROM-17026 | DWG. № DS-2
CHECKED JB




KEYPLAN N.T.S

## LEGEND

- PROPERTY LIMITS
- EXISTING DITCH LINE
- INTERNAL PRE-DEVELOPMENT STORM CATCHMENT BOUNDARY
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- EXTERNAL STORM CATCHMENT NUMBER — A101
- AREA (HECTARES) — 15.8
- DIRECTION OF EXISTING STORM DRAINAGE

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE. N4924207.508 E569777.320 ELEV 184

BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL. N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

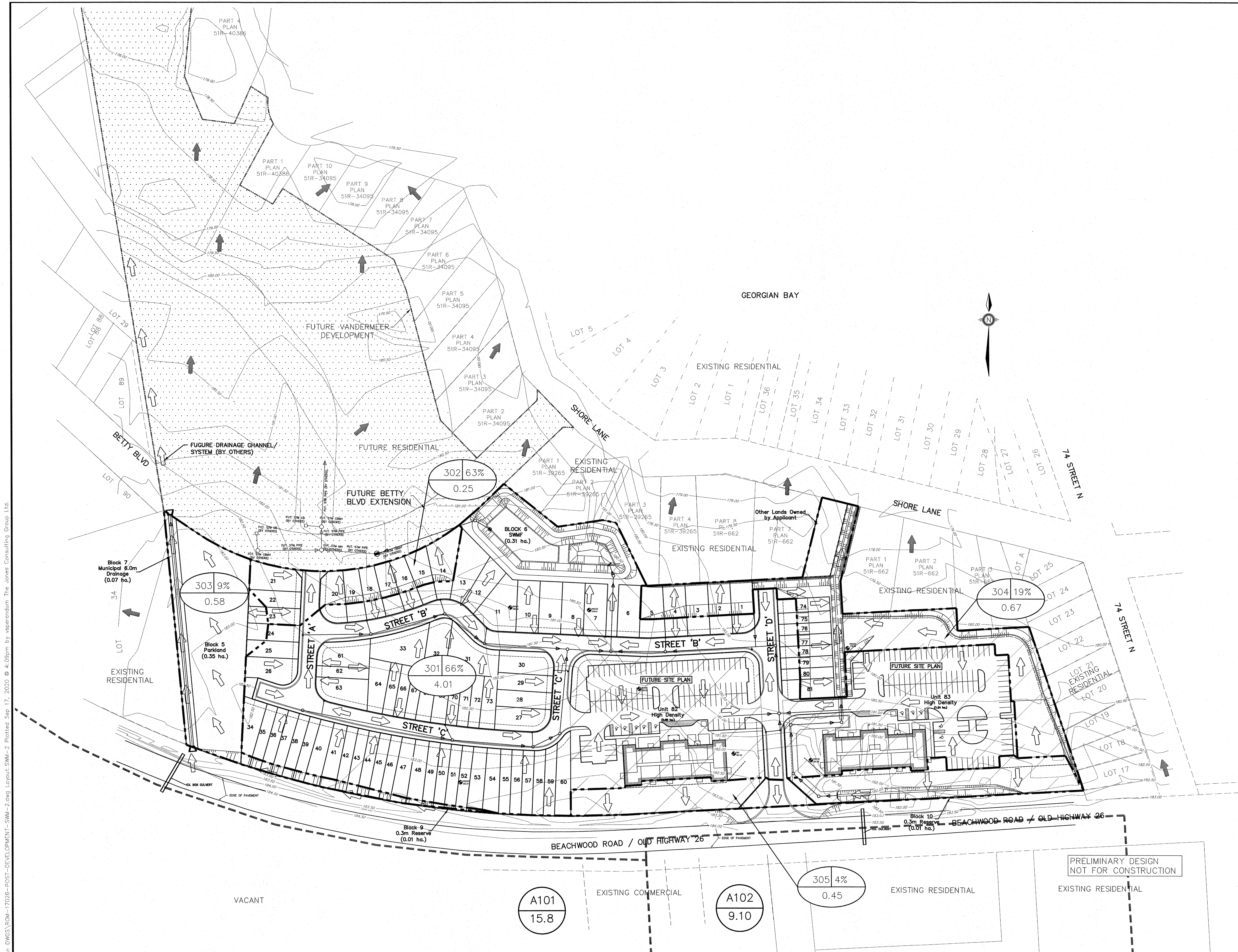
LICENSED PROFESSIONAL ENGINEER D. F. RICHARDSON 100112707 20-09-24 PROVINCE OF ONTARIO

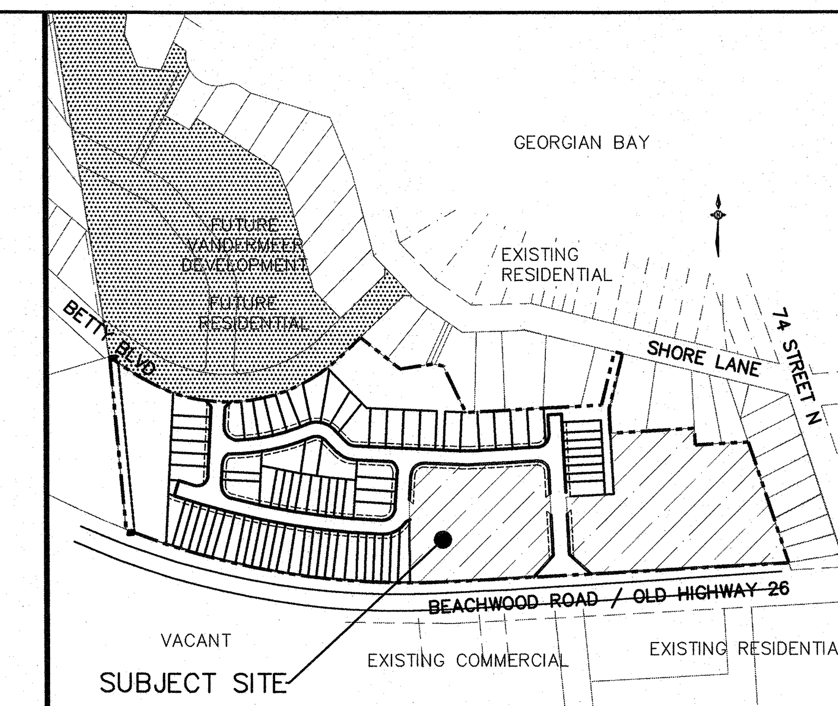
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

**STORMWATER MANAGEMENT PRE–DEVELOPMENT AREAS**

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN JB | SCALE: 1:1000 | DATE APR 2020 |
| DRAWN KC/VS | PROJECT ROM–17026 | DWG. № SWM–1 |
| CHECKED JB | | |




KEYPLAN N.T.S

## LEGEND

- PROPERTY LIMITS
- POST-DEVELOPMENT STORM CATCHMENT BOUNDARY
- EXISTING DITCH LINE
- STORM SEWER, MAINTENANCE HOLE AND DIRECTION OF FLOW
- STM SINGLE CATCHBASIN
- STORM DOUBLE CATCHBASIN
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- % IMPERVIOUSNESS
- STORM CATCHMENT NUMBER
- AREA (HECTARES)
- DIRECTION OF EXTERNAL STORM DRAINAGE
- DIRECTION OF STORM DRAINAGE
- PRE-DEVELOPMENT STORM CATCHMENT BOUNDARY

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

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BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL. N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

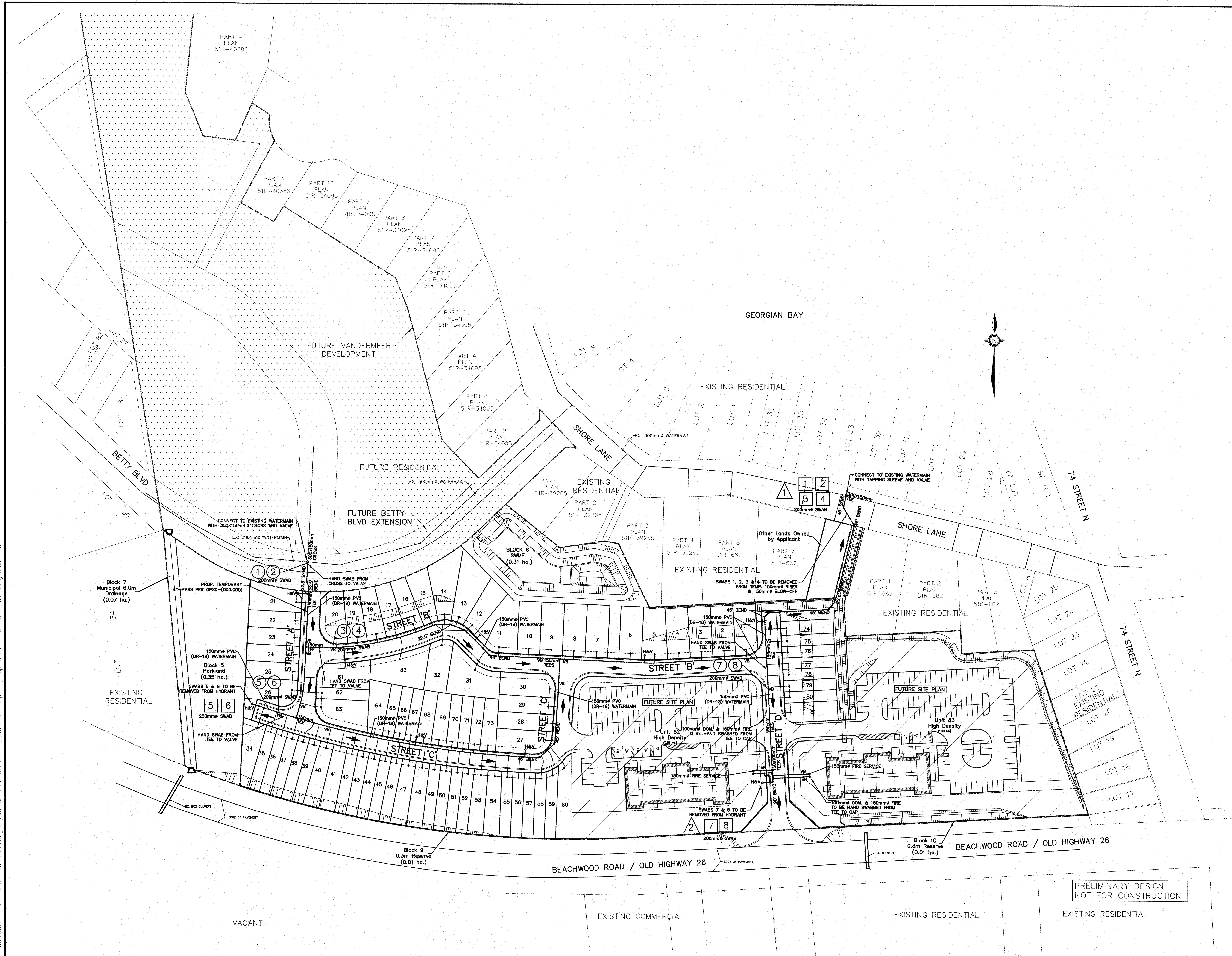
LICENSED PROFESSIONAL ENGINEER
D. F. RICHARDSON
100112707
25-09-24
PROVINCE OF ONTARIO

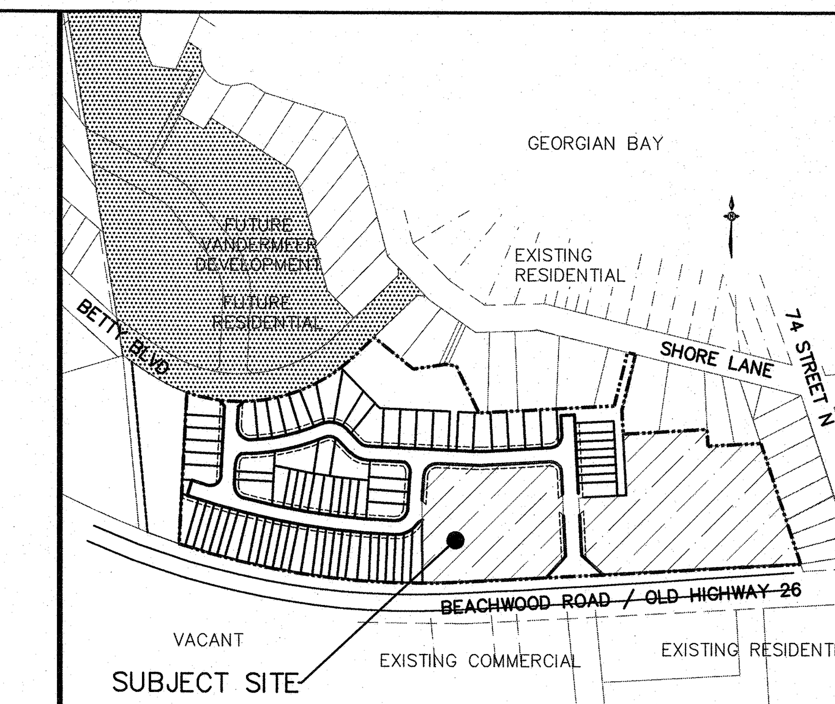
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

OVERALL STORMWATER MANAGEMENT
POST-DEVELOPMENT AREAS

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN JB | SCALE: 1:1000 | DATE APR 2020 |
| DRAWN KC/VS | PROJECT ROM-17026 | DWG. Nº SWM-2 |
| CHECKED JB | | |




KEYPLAN N.T.S

## LEGEND

| | |
|---|---|
| | EX. VALVE & BOX |
| | EX. HYDRANT & VALVE |
| | EX. WATERMAIN |
| H&V | PROP. HYDRANT AND VALVE |
| V&B | PROP. VALVE |
| | PROP. WATERMAIN |
| 200mm⌀ (1) | SWAB NUMBER, INSERTION LOCATION & SIZE |
| 200mm⌀ [1] | SWAB NUMBER, REMOVAL LOCATION & SIZE |
| △2 | WATER SAMPLE LOCATION AND NUMBER |
| | SWAB TRAVEL DIRECTION |
| | LIMIT OF SUBDIVISION |

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| | | | |
| | | | |
| | | | |
| | | | |
| 1 | FSR SUBMISSION | SEP 2020 | DR |

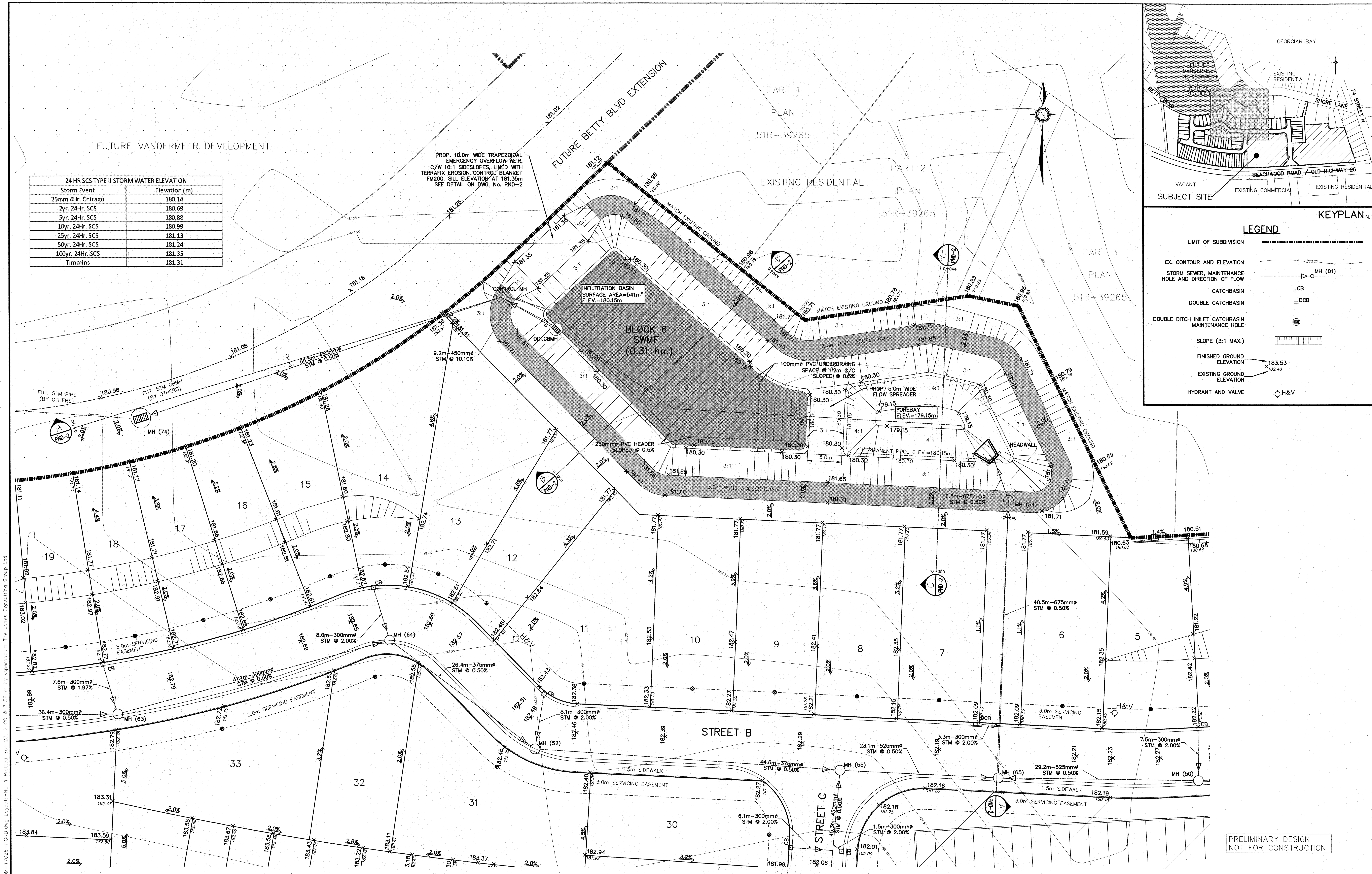
LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

WATER SWABBING PLAN

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | | |
|---|---|---|---|
| DESIGN VS | SCALE: 1:1000 | DATE JULY 2020 | |
| DRAWN VS | PROJECT ROM-17026 | DWG. № WM-1 | |
| CHECKED JB | | | |




| 24 HR SCS TYPE II STORM WATER ELEVATION | |
|---|---|
| Storm Event | Elevation (m) |
| 25mm 4Hr. Chicago | 180.14 |
| 2yr. 24Hr. SCS | 180.69 |
| 5yr. 24Hr. SCS | 180.88 |
| 10yr. 24Hr. SCS | 180.99 |
| 25yr. 24Hr. SCS | 181.13 |
| 50yr. 24Hr. SCS | 181.24 |
| 100yr. 24Hr. SCS | 181.35 |
| Timmins | 181.31 |

BENCHMARK:

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE. N4924207.508 E569777.320 ELEV 184

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

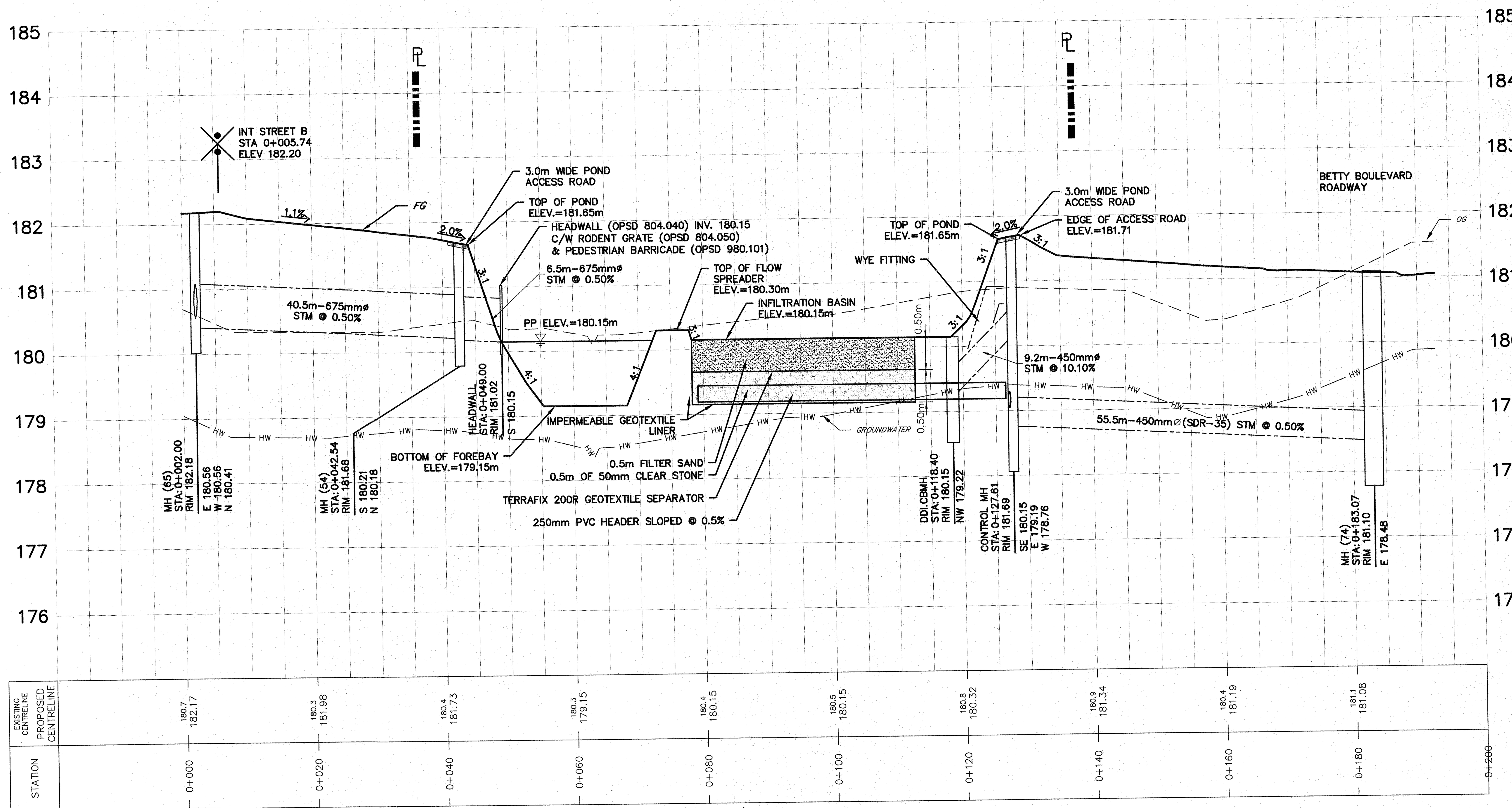
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

SWM FACILITY
PLAN VIEW

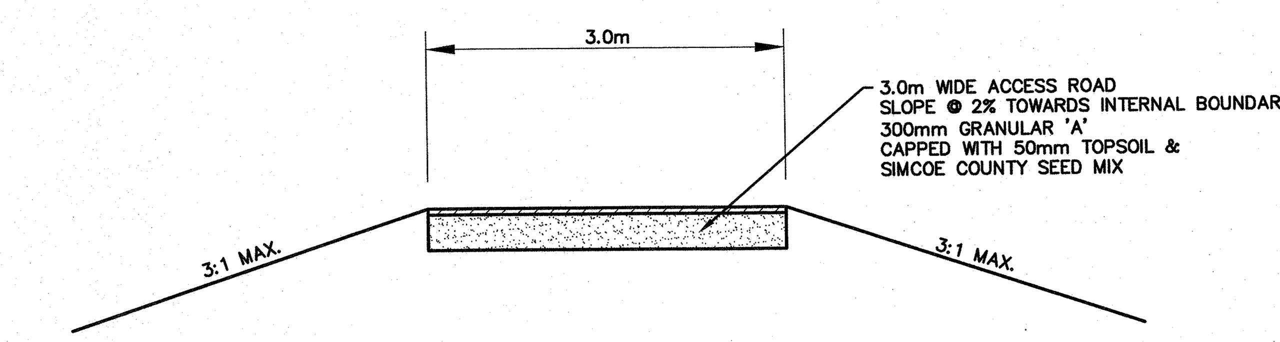
JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | |
|---|---|---|
| DESIGN JB | SCALE: 1:250 | DATE JUL 2020 |
| DRAWN VS | PROJECT ROM-17026 | DWG. № PND-1 |
| CHECKED JB | | |

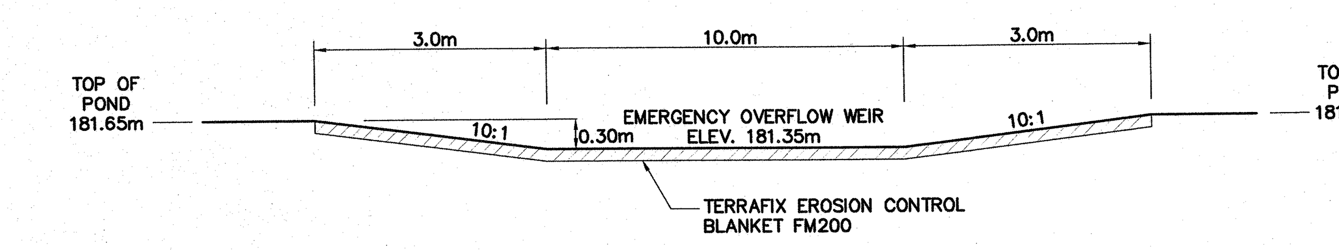
| 24 HR SCS TYPE II STORM WATER ELEVATION | |
|---|---|
| Storm Event | Elevation (m) |
| 25mm 4Hr. Chicago | 180.14 |
| 2yr. 24Hr. SCS | 180.69 |
| 5yr. 24Hr. SCS | 180.88 |
| 10yr. 24Hr. SCS | 180.99 |
| 25yr. 24Hr. SCS | 181.13 |
| 50yr. 24Hr. SCS | 181.24 |
| 100yr. 24Hr. SCS | 181.35 |
| Timmins | 181.31 |




SECTION A–A
PND-1
HOR. 1:500
VERT. 1:50



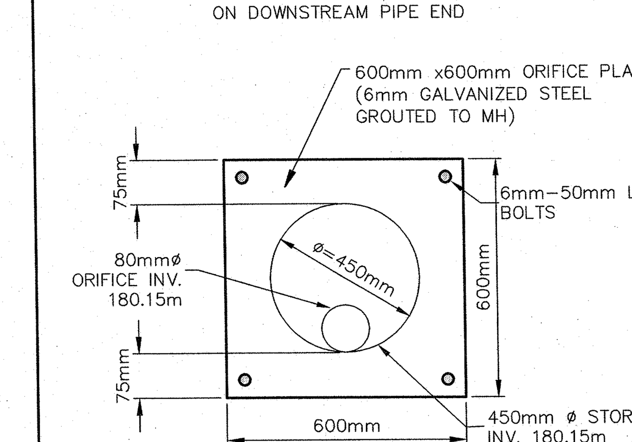

POND ACCESS ROAD TYP.
SCALE 1:50

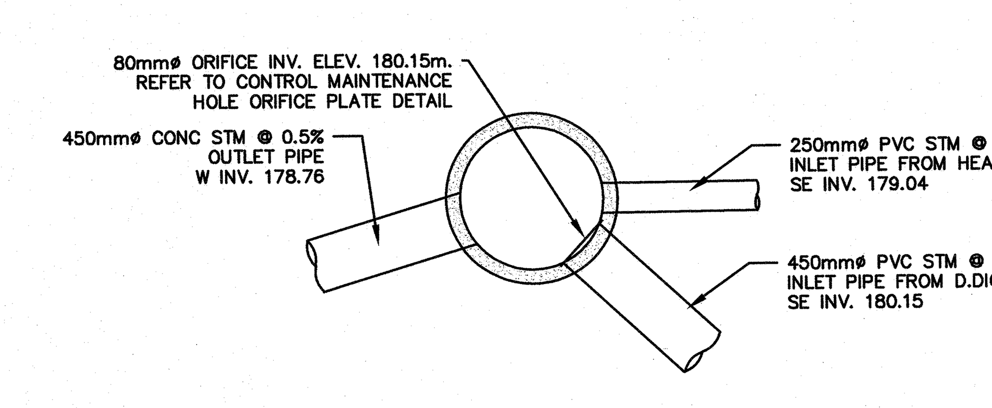



EMERGENCY OVERFLOW WEIR
NOT TO SCALE

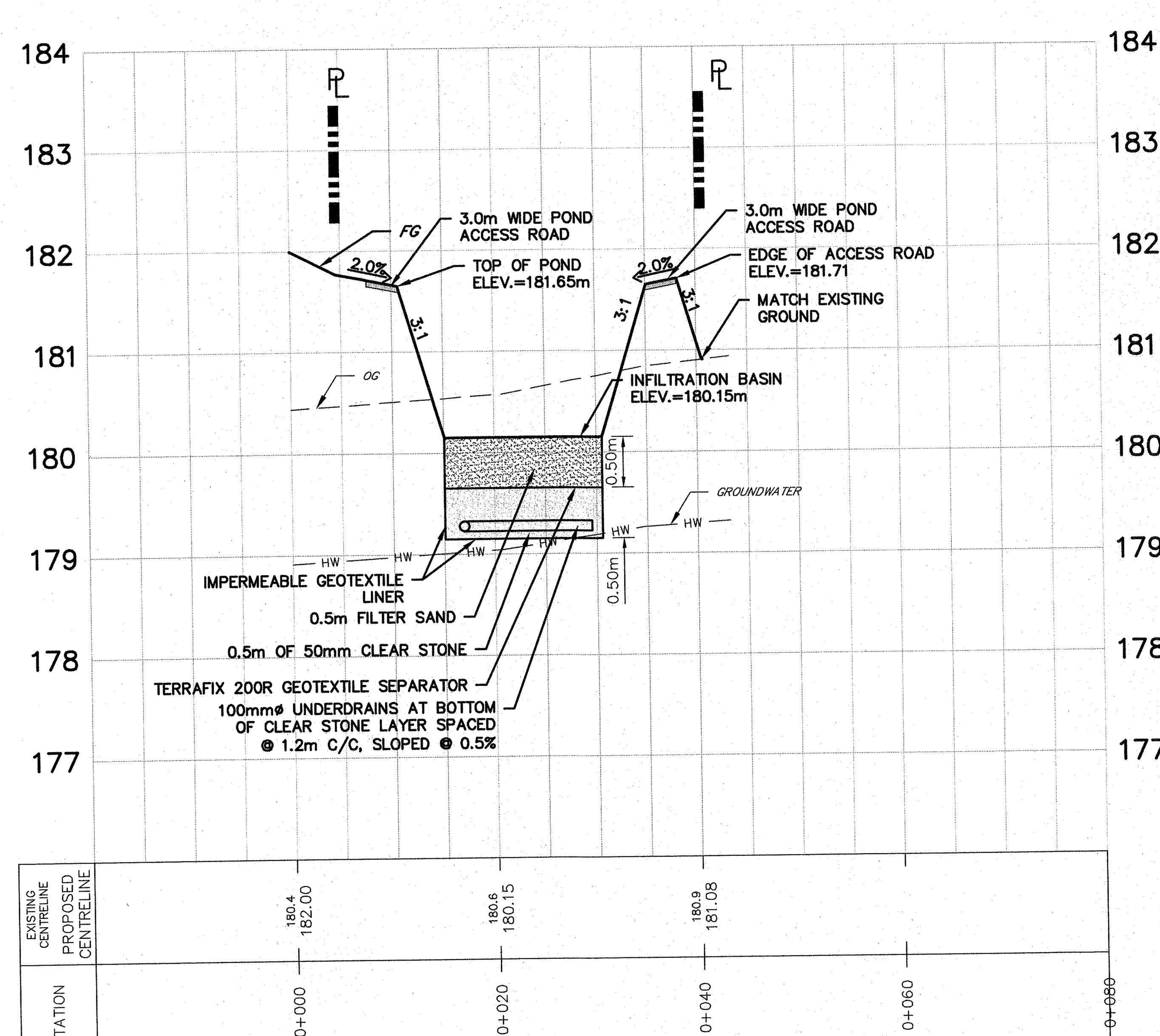
CONTROL MANHOLE
ORIFICE PLATE DETAIL

BOLTED TO INSIDE OF CONTROL MANHOLE ON DOWNSTREAM PIPE END
N.T.S.

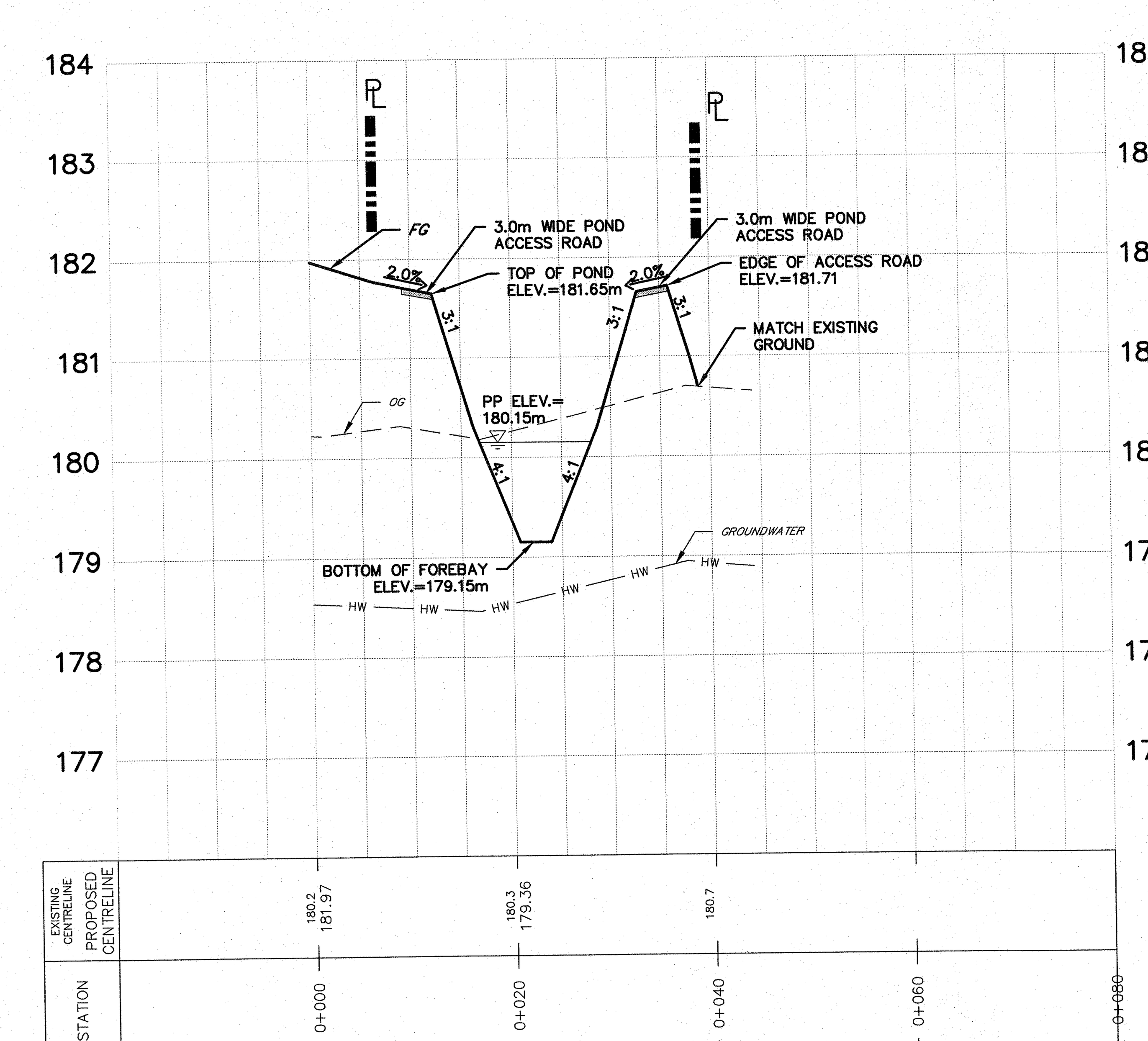



STM MH OUTLET CONTROL
PLAN VIEW SCALE 1:50




SECTION B–B
PND-1
HOR. 1:500
VERT. 1:50




SECTION C–C
PND-1
HOR. 1:500
VERT. 1:50

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE.
N4924007.xxx E571589.xxx ELEV 180.020

BENCHMARK NO. 00819880756. MONUMENT LOCATED ON THE SOUTH SIDE OF HIGHWAY 26, 0.5 KM WEST FROM 74TH STREET NORTH ALSO 0.8 KM EAST FROM MARILYN LANE.
N4924207.508 E569777.320 ELEV 184

BENCHMARK NO. 0011971U183. COLLINGWOOD SMALL CONCRETE CULVERT UNDER HIGHWAY NO. 26, 1.3 KM NORTHWEST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, TABLET IN TOP OF CULVERT, 24 CM FROM SOUTHWEST END AND 18 CM FROM SOUTHEAST EDGE OF CULVERT, 2 M BELOW ROAD LEVEL.
N4923771.xxx E569779.xxx ELEV. 183.592

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-01-24
PROVINCE OF ONTARIO

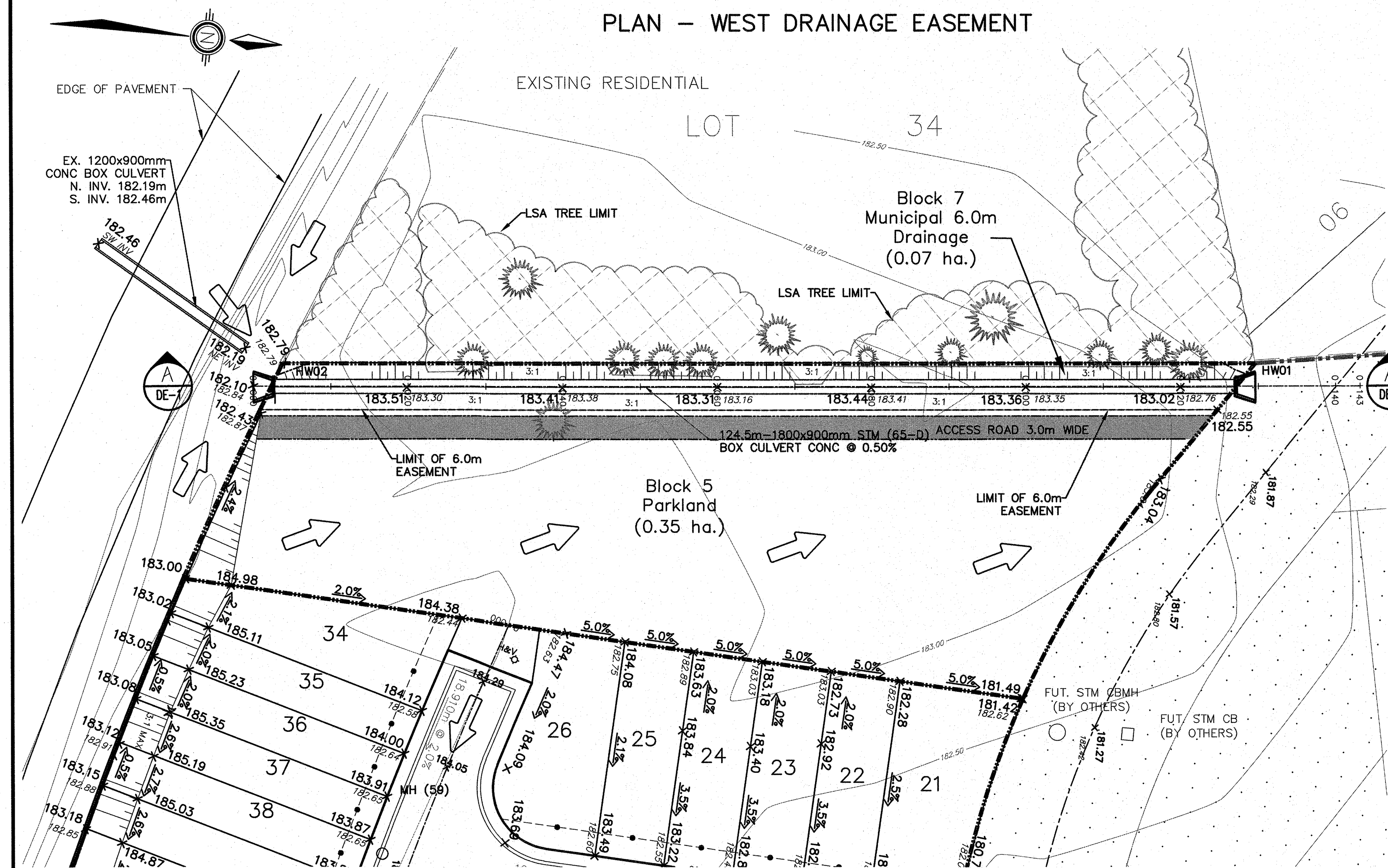
BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

SWM FACILITY
SECTIONS & DETAILS


JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056


| DESIGN JB | SCALE: 1:250 | DATE JUL 2020 |
|---|---|---|
| DRAWN VS | PROJECT ROM–17026 | DWG. № PND–2 |
| CHECKED JB | | |

PLAN – WEST DRAINAGE EASEMENT

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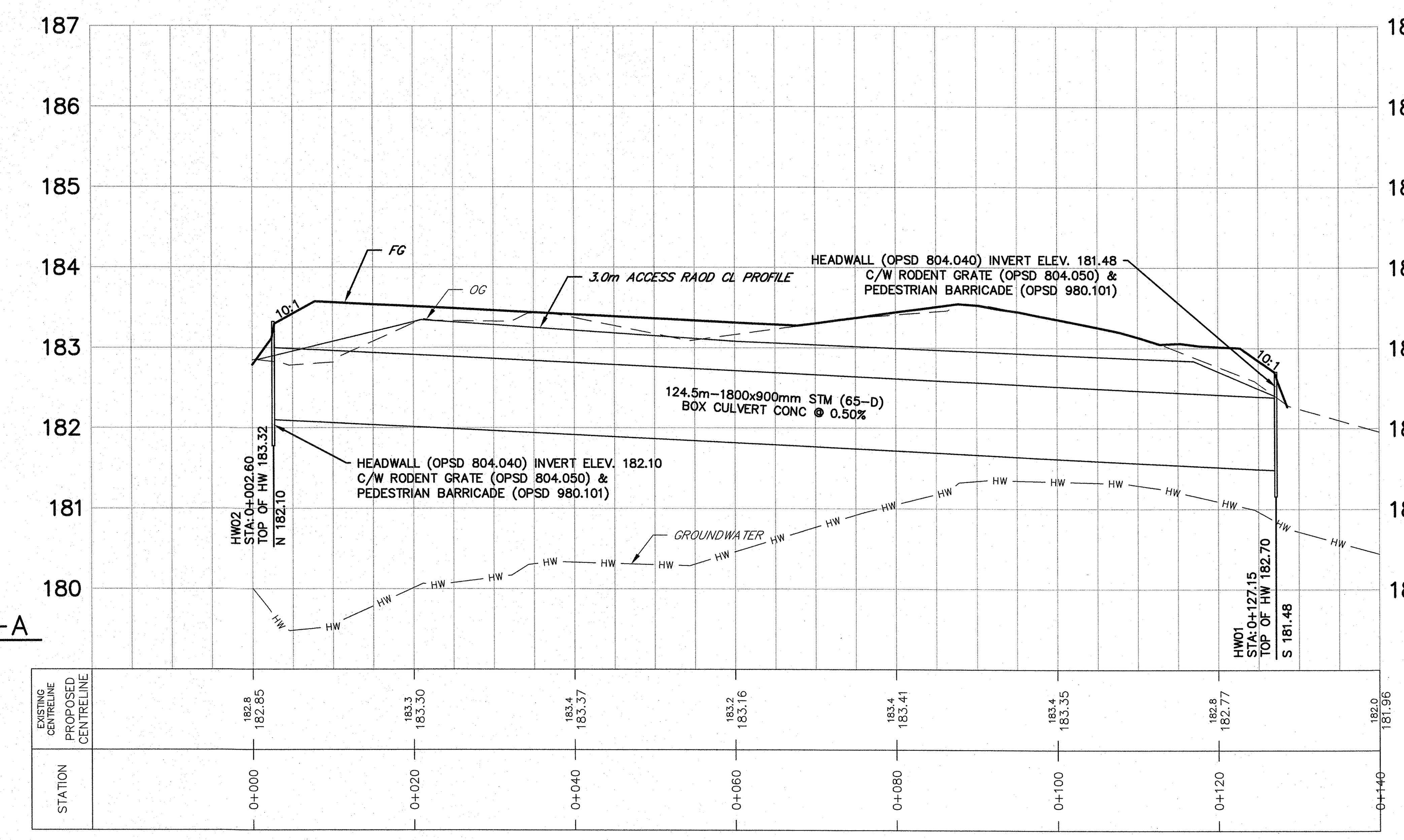


SECTION A–A
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VERT. 1:50

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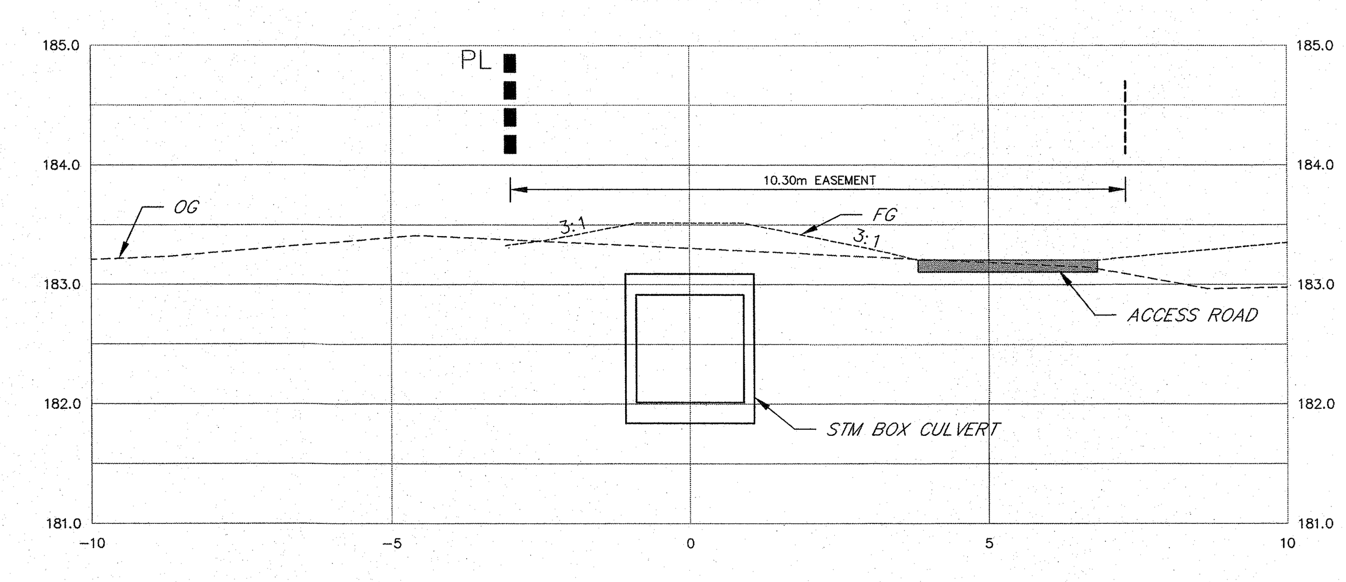


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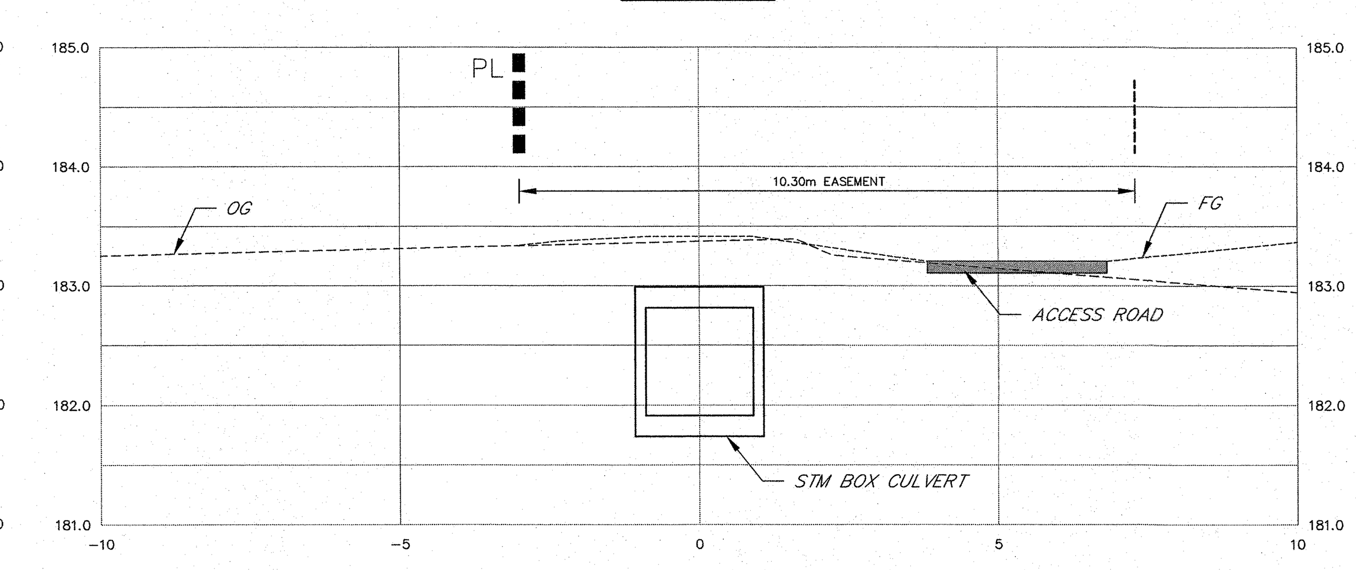


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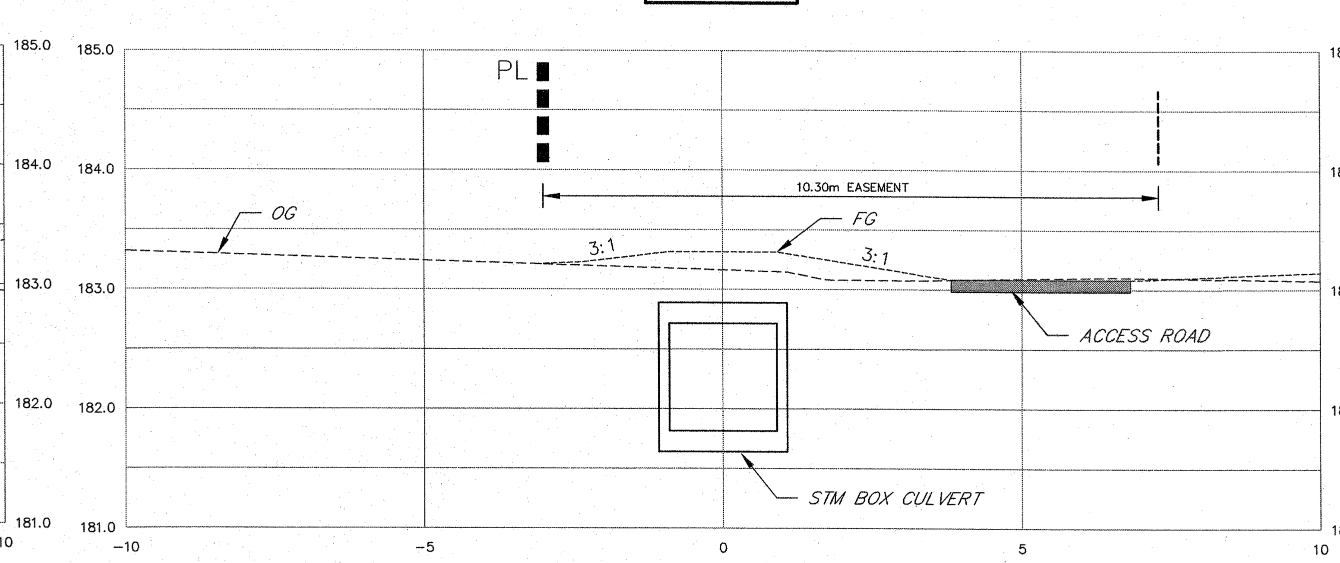


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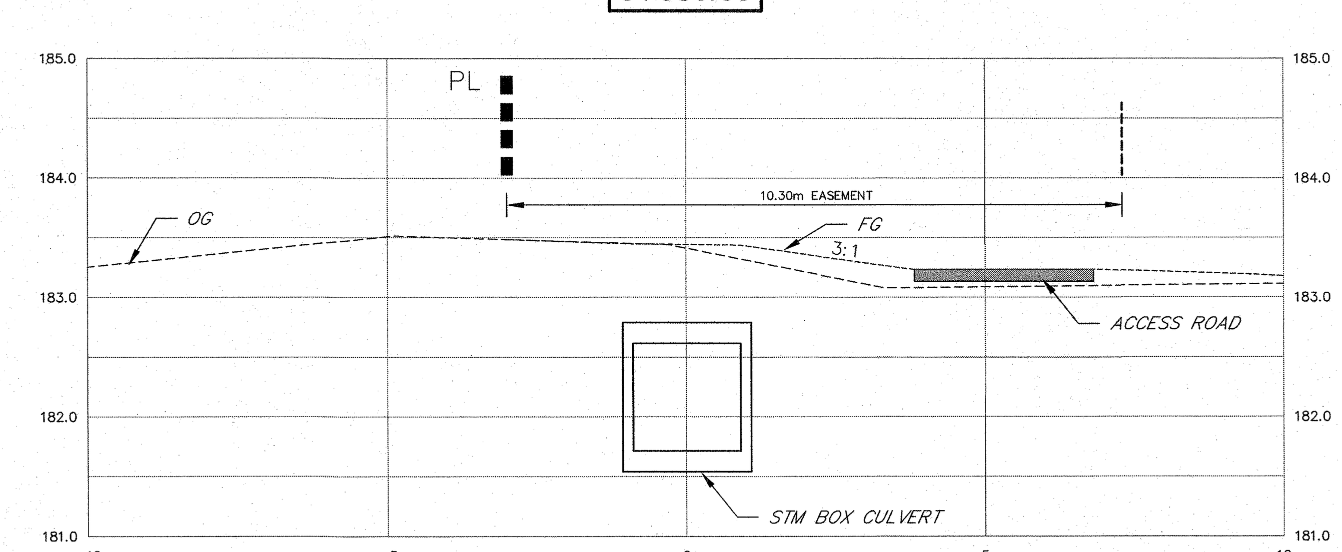


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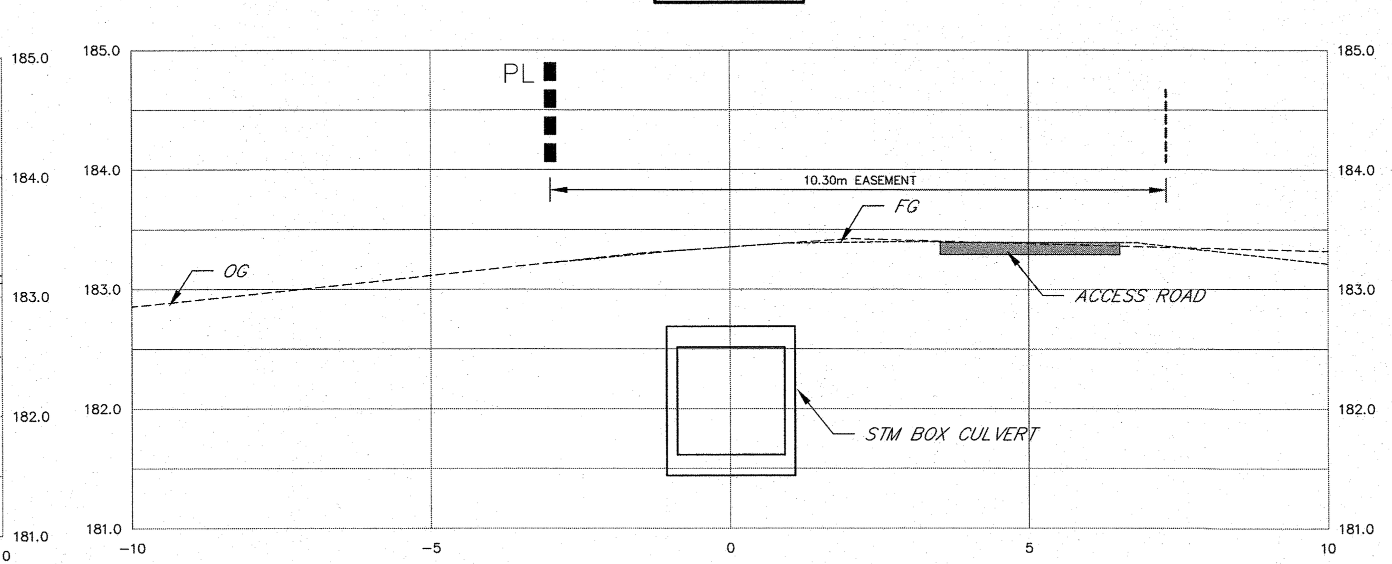


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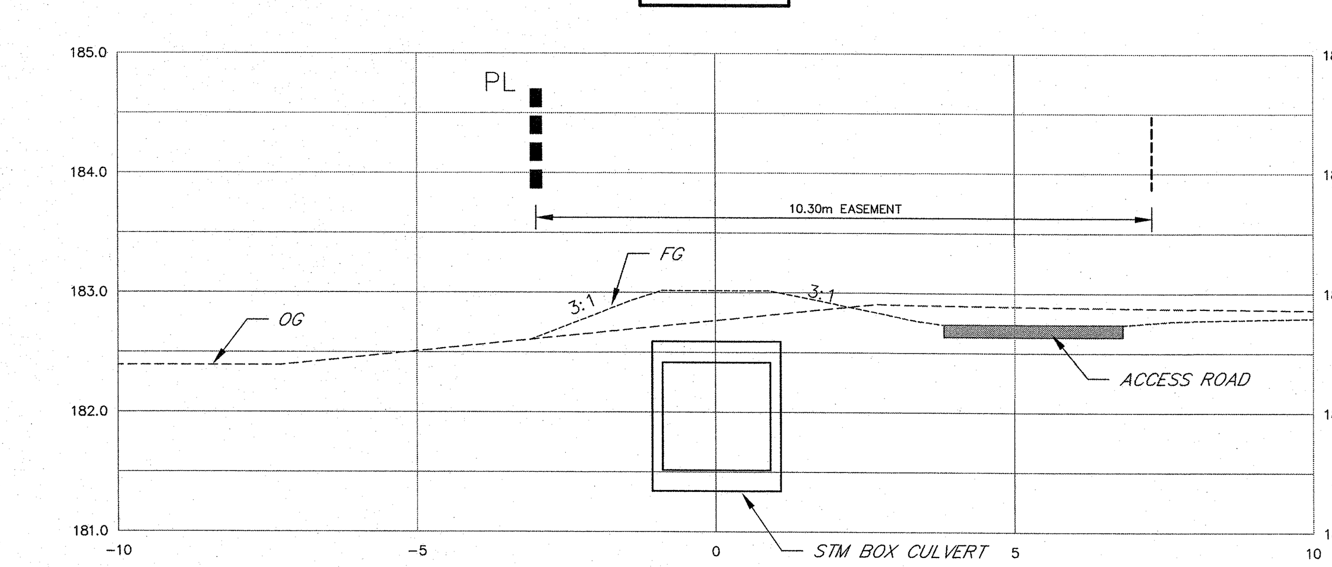


SCALE H 1:100 V 1:50

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

**BENCHMARK:**

BENCHMARK NO. 0011971U190 LOCATED AT WASAGA BEACH DEEP BENCH MARK IN MANHOLE IN GROUND OF BYRNES AVENUE PUBLIC SCHOOL, 0.7 KM NORTHEAST OF JUNCTION OF HIGHWAY NO. 26 AND SIMCOE ROAD NO. 7, IN LAWN, 28.3 M NORTHEAST OF NORTHEAST CORNER OF SCHOOL, 22.2M SOUTH OF CENTRE LINE OF SIMCOE ROAD NO. 7, 10.4 M EAST OF EAST LANEWAY TO SCHOOL, 5.8 M NORTH OF A PINE TREE. N4924007.xxx E571589.xxx ELEV 180.020

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER
D.F. RICHARDSON
100112707
20-09-24
PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

WESTERN DRAINAGE EASEMENT

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
Barrie, ON L4N 0W5
P. 705.734.2538
F. 705.734.1056

| | | | |
|---|---|---|---|
| DESIGN JT/VS | SCALE: AS NOTED | DATE JULY 2020 | |
| DRAWN JT/VS | PROJECT ROM–17026 | DWG. Nº DE–1 | |
| CHECKED JB | | | |

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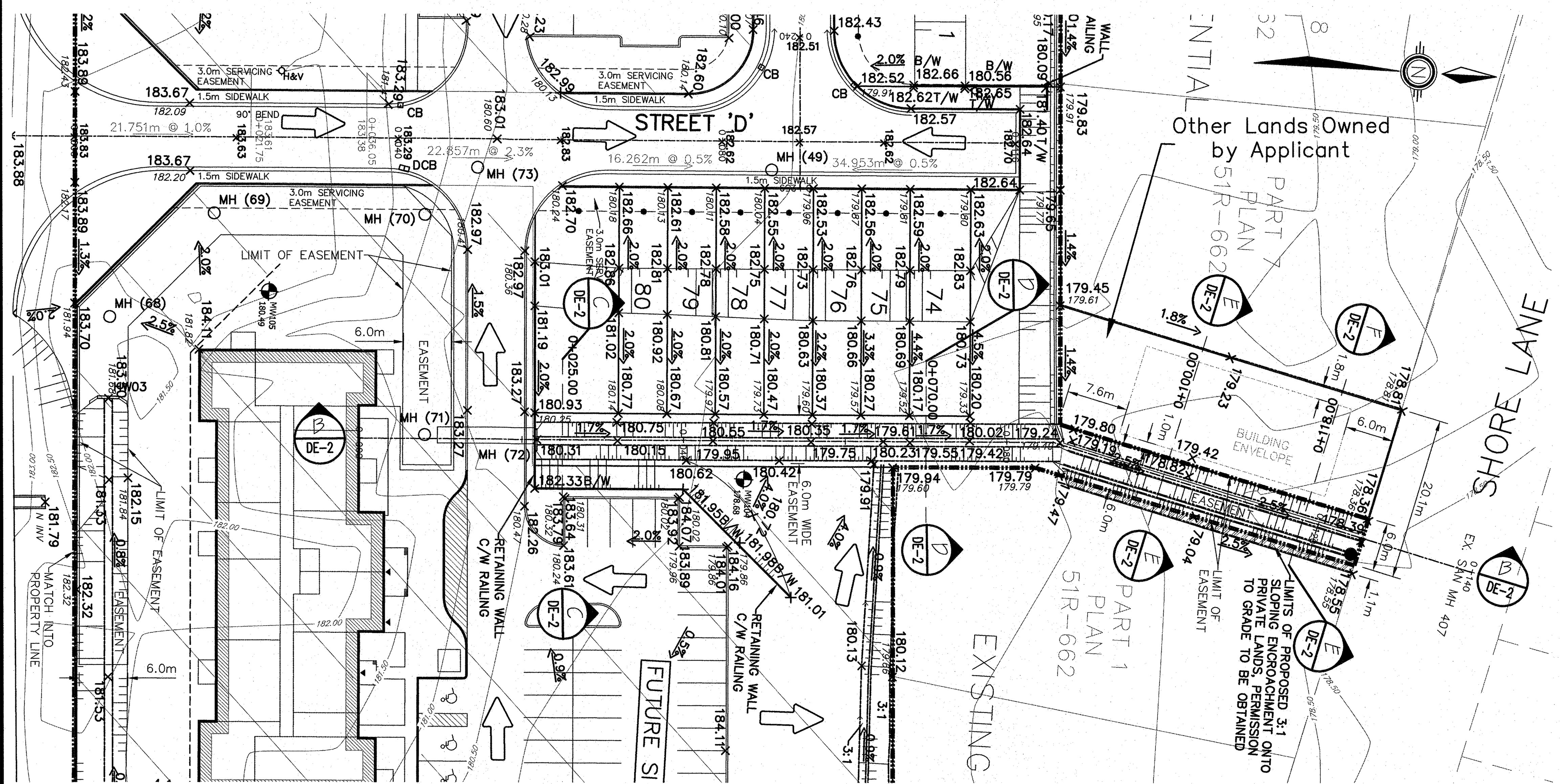
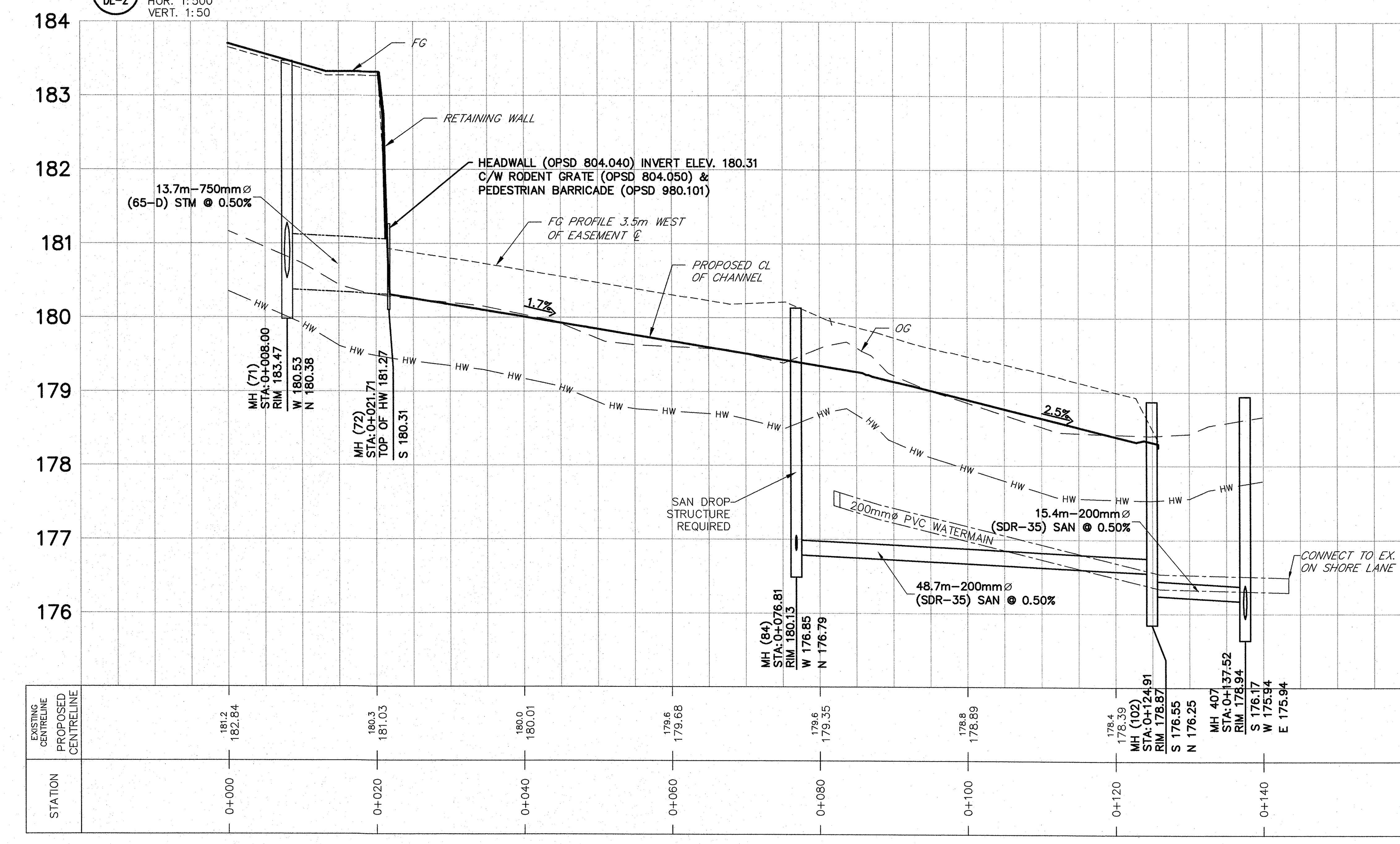



PLAN – EAST DRAINAGE EASEMENT SCALE 1:500

SECTION B–B (DE-2) HOR. 1:500 VERT. 1:50

PROFILE – EAST DRAINAGE EASEMENT

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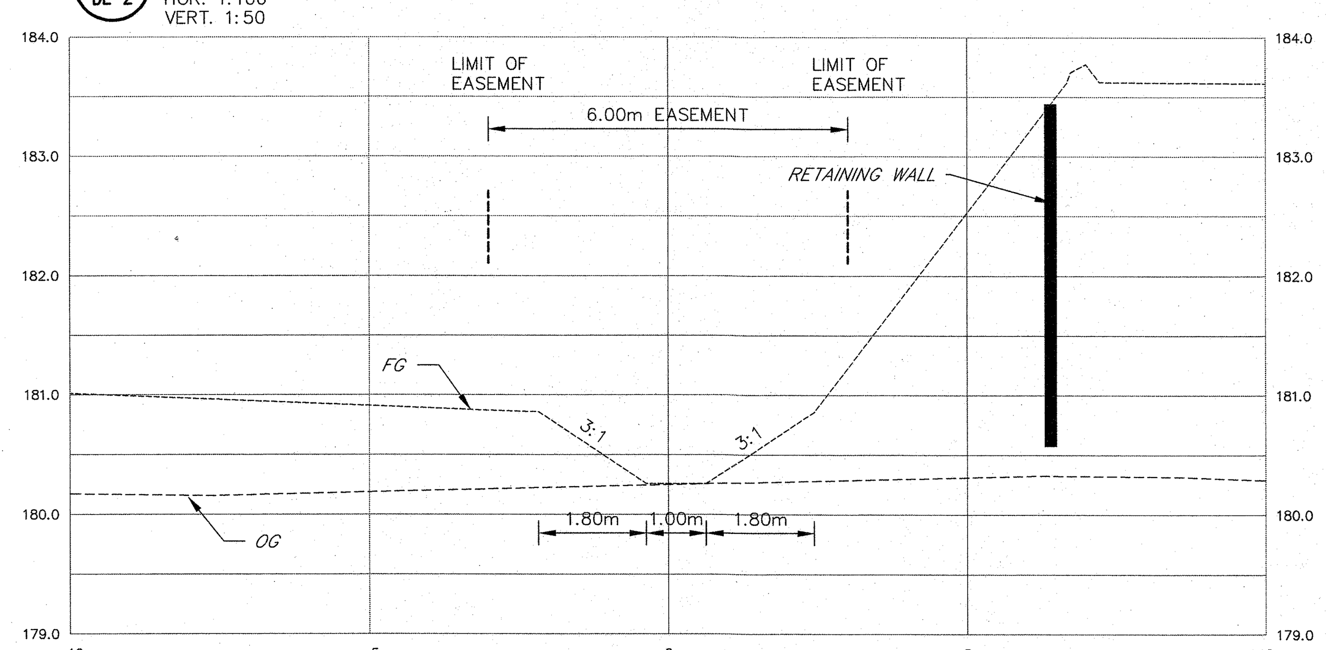


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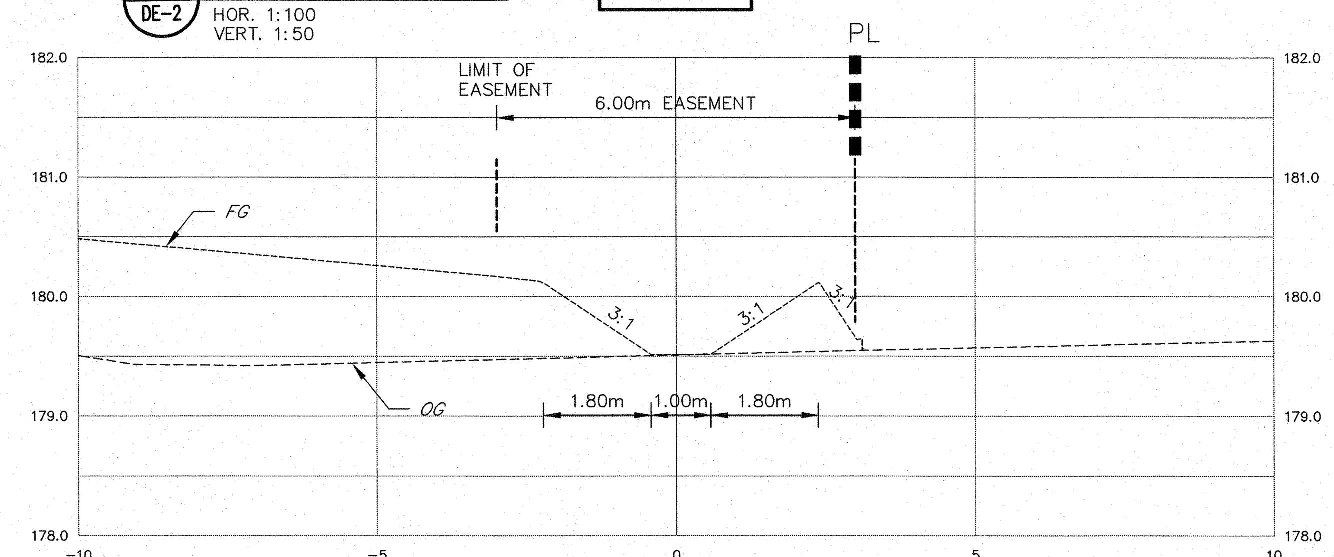
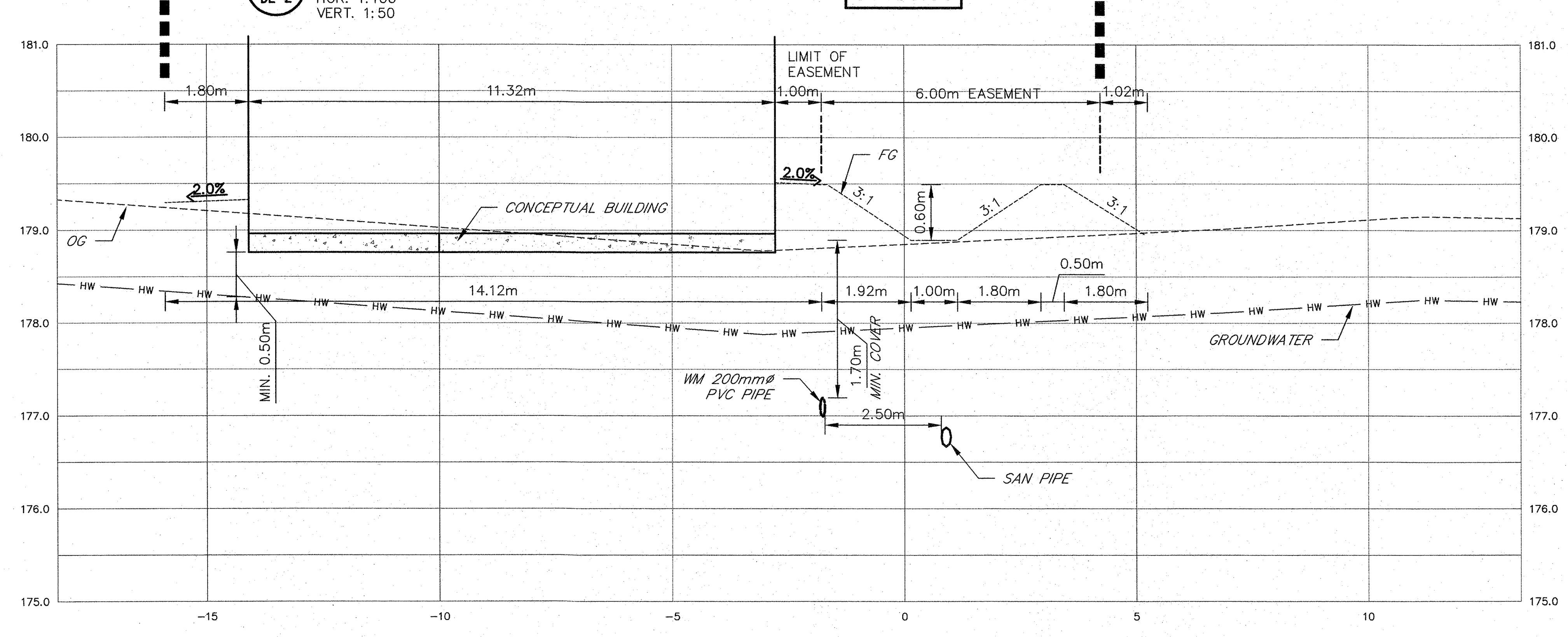



PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

SECTION E–E (DE-2) HOR. 1:100 VERT. 1:50 0+100.00

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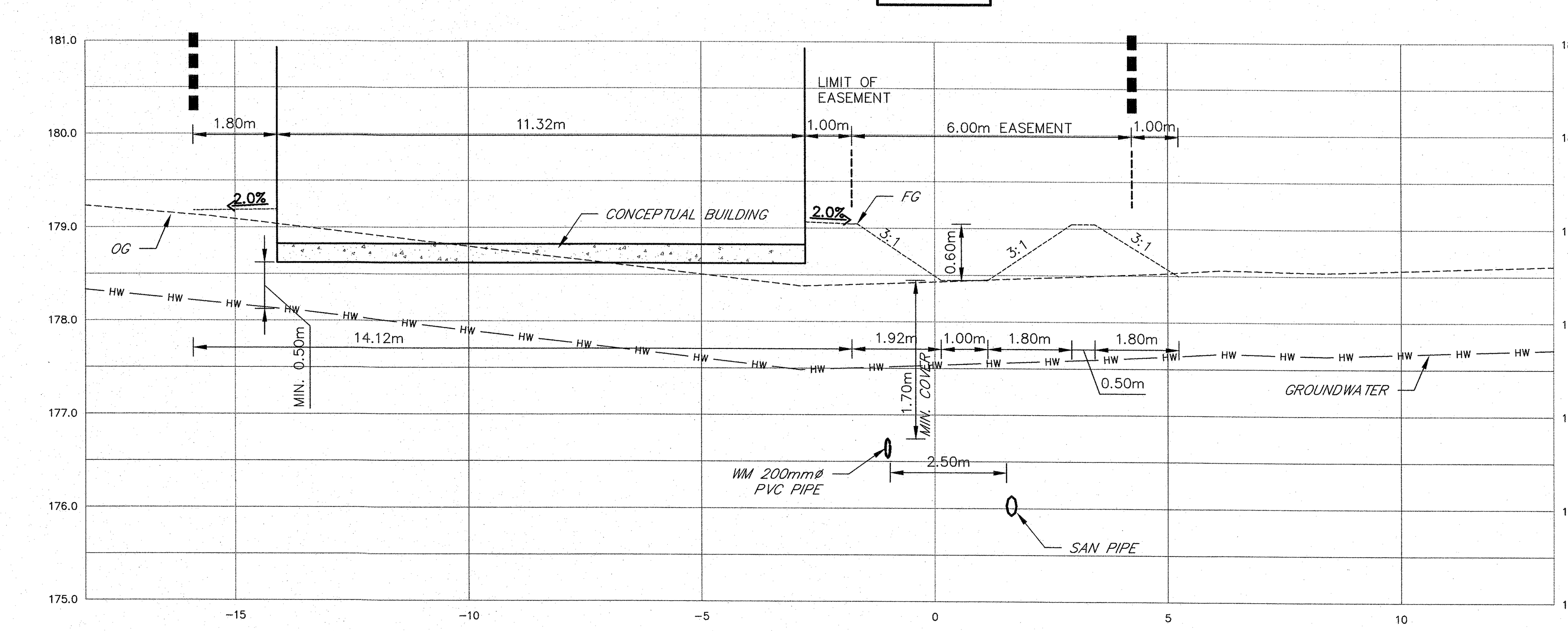


**BENCHMARK:**

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | FSR SUBMISSION | SEP 2020 | DR |

LICENSED PROFESSIONAL ENGINEER D.F. RICHARDSON 100112707 20-09-24 PROVINCE OF ONTARIO

BEACHWOOD DEVELOPMENT INC.
BEACHWOOD DEVELOPMENT
TOWN OF WASAGA BEACH

WESTERN DRAINAGE EASEMENT

JONES CONSULTING GROUP LTD. PLANNERS & ENGINEERS
229 Mapleview Dr. E, Unit 1
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P. 705.734.2538
F. 705.734.1056

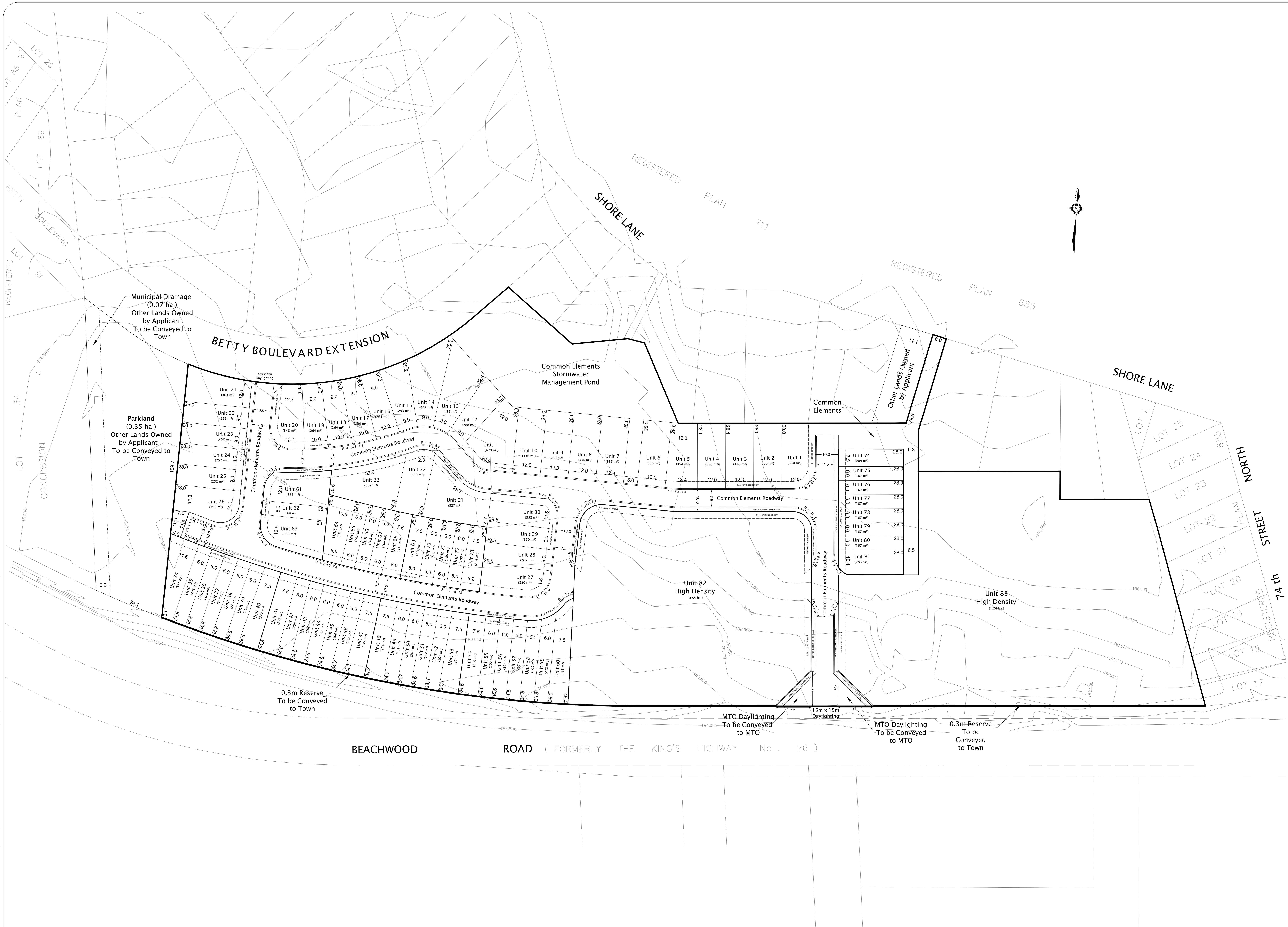
| | | | |
|---|---|---|---|
| DESIGN JT/VS | SCALE: AS NOTED | DATE JULY 2020 | |
| DRAWN JT/VS | PROJECT ROM–17026 | DWG. Nº DE–2 | |
| CHECKED JB | | | |

# Appendix D

## External Information

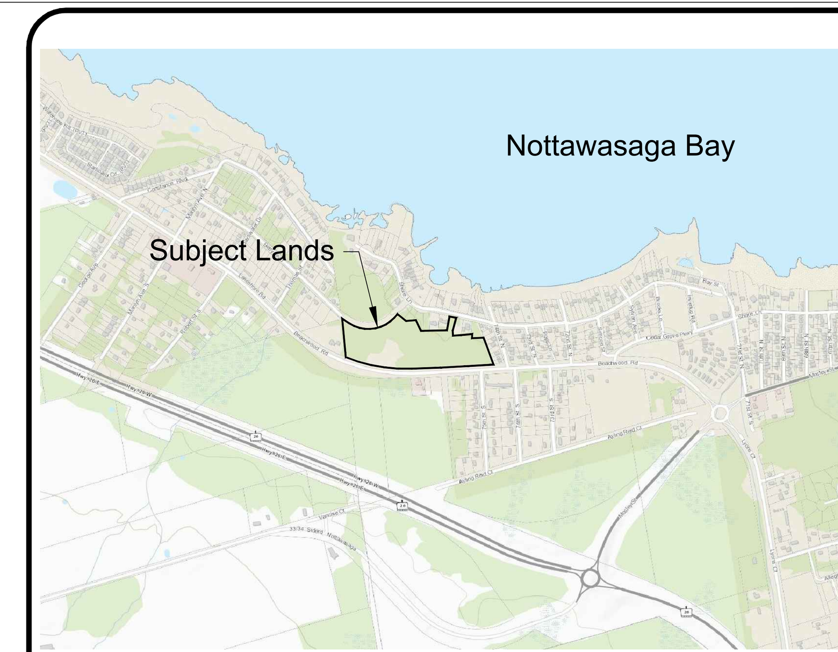
## Planning and External Reference Information

- Proposed Draft Plan of Vacant Land Condominium by TJCG – July 27, 2020
- Town of Wasaga Beach Comprehensive Zoning By-law 2003-60 excerpts
- Town of Wasaga Beach Official Plan Schedules B, C, D and H
- Chapman Property Shore Lane Plan and Profile – PP1
- Town of Wasaga Beach Record Drawing – 105120-SW10-RD
- Soils Map Excerpt
- Nottawasaga Valley Conservation Authority online Mappings
- County of Simcoe Online Mapping
- Exceprts from West End Drainage Study, Town of Wasaga Beach, Ainley – August 2019




# BEACHWOOD DEVELOPMENTS INC.

## TOWN OF WASAGA BEACH




KEY PLAN SCALE: 20,000

## Proposed Draft Plan of Vacant Land Condominium
Part of Lot 34, Concession 3
Town of Wasaga Beach
County of Simcoe
2020

OWNERS' CERTIFICATE

WE, THE UNDERSIGNED, BEING THE REGISTERED OWNERS OF THE SUBJECT LANDS, HEREBY AUTHORIZE THE JONES CONSULTING GROUP LTD., TO PREPARE THIS DRAFT PLAN OF SUBDIVISION AND TO SUBMIT SAME TO THE TOWN OF WASAGA BEACH FOR APPROVAL.

DATE ____________ M.ROMANIN CONTRACTING LIMITED

SURVEYOR'S CERTIFICATE

I CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AND THEIR RELATIONSHIP TO ADJACENT LANDS ARE ACCURATELY AND CORRECTLY SHOWN.

DATE ____________ SASA KRCMAR OLS, KRCMAR SURVERYORS

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT

a) SHOWN ON DRAFT PLAN
b) SHOWN ON DRAFT PLAN
c) SHOWN ON KEY PLAN
d) RESIDENTIAL, SWM & DRAINAGE
e) SHOWN ON DRAFT PLAN
f) SHOWN ON DRAFT PLAN
g) SHOWN ON DRAFT PLAN
h) MUNICIPAL PIPED WATER TO BE PROVIDED
i) SAND
j) SHOWN ON DRAFT PLAN
k) ALL MUNICIPAL SERVICES TO BE PROVIDED
l) NONE

| SUBDIVISION STATISTICS | AREA (ha.) | UNITS |
|---|---|---|
| Residential - Singles (Units 1 - 33) | 1.10 ha. | 33 units |
| Residential - Townhomes (Units 34 - 81) | 1.07 ha. | 48 units |
| Residential - High Density (Units 82 & 83) | 2.09 ha. | 134 units |
| Stormwater Management | 0.31 ha. | |
| Roadway | 0.68 ha. | |
| Common Elements | 0.12 ha. | |
| County Daylighting | 0.02 ha. | |
| 0.3m Reserves | 0.01 ha. | |
| SUBDIVISION TOTAL | 5.40 ha. | |
| Parkland to be Conveyed | 0.35 ha. | |
| Municipal Drain to be Conveyed | 0.07 ha. | |
| Other Lands Owned by Applicant | 0.06 ha. | |
| Other Lands Total | 0.48 ha. | |
| Total Land Holdings | 5.88 ha. | 215 units |

0 5 10 50M

SCALE 1 : 750 (A1)

BEACHWOOD DEVELOPMENTS INC.
WASAGA BEACH

Proposed Draft Plan of Vacant Land Condominium

Brandi L. Clement — REGISTERED PROFESSIONAL PLANNER — R.P.P. — ONTARIO PROFESSIONAL PLANNERS INSTITUTE — INSTITUT DES PLANIFICATEURS PROFESSIONNELS DE L'ONTARIO

| | |
|---|---|
| Date Issued: | JULY 27, 2020 |
| Checked By: | BC |
| Project No.: | ROM-17026 |
| Drawn By: | m.c.r. |
| Drawing Name: | ROM-17026-DP-VLC-20-07-27.dwg |

| SCHEDULE OF REVISIONS | | |
|---|---|---|
| DATE | DESCRIPTION | DRAWN |

JONES CONSULTING GROUP LTD.
PLANNERS & ENGINEERS
229 Mapleview Drive East, Unit 1, Barrie, Ontario, L4N 0W5
Phone: 705-734-2538 Fax: 705-734-1056
www.jonesconsulting.com

## SECTION 4 - RESIDENTIAL TYPE 1 (R1) ZONE

**4.1** No person shall within any Residential Type 1 (R1) Zone use any lot or erect, alter, enlarge, maintain or use any building or structure for any purpose or use other than as permitted for one or more of the following uses and in accordance with the following provisions or requirements as set out herein:

**4.2 PERMITTED USES**

**4.2.1 Residential Uses:**

a) single detached dwelling unit
b) one (1) attached accessory dwelling unit, subject to Subsection 3.28

**4.2.2 Non-Residential Uses:**

a) accessory use directly related to the uses permitted in the R1 Zone
b) home occupation
c) public use
d) bed and breakfast

**4.3 ZONE PROVISIONS**

**4.3.1 Lot Area (minimum):**

| | | |
|---|---|---|
| a) | lot served by a public water system and a sanitary sewer system | 464.5sq.m |
| b) | lot served by a public water system | 1,400 sq.m |
| c) | other lots | 1,860 sq.m |

**4.3.2 Lot Frontage (minimum):**

| | | |
|---|---|---|
| a) | lot served by a public water system and a sanitary sewer system | 12 m |
| b) | lot served by a public water system | 22m |
| c) | other lots | 30m |

| | | |
|---|---|---|
| **4.3.3** | **Front Yard Depth (minimum):** | 6m |
| **4.3.4** | **Exterior Side Yard Width (minimum):** | 4.5m |
| **4.3.5** | **Interior Side Yard Width (minimum):** | 1.8m |
| **4.3.6** | **Rear Yard Depth (minimum):** | 7.6m |
| **4.3.7** | **Dwelling Unit Area (minimum):** | 93 sq.m |

| | | |
|---|---|---|
| **4.3.8** | **Landscaped Open Space (minimum):** | 30% |
| **4.3.9** | **Lot Coverage (maximum):** | 35% |
| **4.3.10** | **Height of Building (maximum):** | 10m |

**4.3.10.1** *(2007-17)* A single detached dwelling will be restricted to two storeys but may have a loft or living space located within the pitched roof area/attic of the said dwelling.

**4.3.11 Dwelling Units per Lot (maximum):**

| | | |
|---|---|---|
| **a)** | single-detached dwelling | 1 |
| **b)** | attached accessory dwelling unit | 1 |

**4.3.12 Accessory Uses see Section 3.1**

**4.3.13 Parking Provisions see Section 3.38**

**4.3.14 Attached Accessory Dwelling in Residential Dwellings, see Subsection 3.28**

## 4.4 ZONE EXCEPTIONS

The following Zone categories shall have the same permitted uses and zone provisions as the regular R1 Zone except as noted.

### 4.4.1 R1-1 Zone

### 4.4.2 R1-2 Zone Schedule "B"

The front yard depth minimum shall be 5.7 m.

### 4.4.3 R1-3 Zone

A home medical office shall also be permitted.

### 4.4.4 R1-4 Zone Schedule "H"

The front yard depth minimum shall be 1.2 m; the north interior side yard width minimum shall be 0.6 m; the rear yard depth minimum shall be 1.2 m.

#### 4.4.5 R1-5 Zone Schedule “H”

The rear yard depth minimum shall be 10.6 m.

#### 4.4.6 R1-6 Zone Schedule “H”

A bed and breakfast shall also be permitted and the exterior side yard width minimum shall be 1.8 m.

#### 4.4.7 R1-7 Zone Schedule “I”

The front lot line shall be the north lot line for the purpose of yards only and the Rear Yard Depth (minimum), as it applies to the south rear yard, shall be 12 m.

#### 4.4.8 R1-8 Zone Schedule “P”

The minimum lot frontage shall be 15.2 m.

#### 4.4.9 R1-9 Zone Schedule “P”

The minimum lot frontage shall be 18.2 m.

#### 4.4.10 R1-10 Zone Schedule “I”

The minimum lot area shall be 380 square metres.

#### 4.4.11 R1-11 Zone Schedule “I”

The minimum lot frontage shall be 7.6 metres; the minimum interior side yard depth, as it relates to the north and south interior side yards only, shall be 7.6 metres; and, the minimum rear yard depth shall be 1.8 metres.

#### 4.4.12 R1-12 Zone Schedule “I”

The minimum front yard depth shall be 2.4, the minimum interior side yard depth, as it relates to the north and south interior side yards only, shall be 7.6 metres; and, the minimum rear yard depth shall be 1.8 metres.

#### 4.4.13 R1-13 Zone Schedule “A”

The lot area minimum shall be 410 square metres; the lot frontage minimum shall be 15 metres; the front yard depth minimum shall be 4.5 metres and 6 metres from any attached garage; the exterior side yard width minimum shall be 3 metres and lot coverage maximum shall be

43%. For the purpose of this section a private street shall be deemed to be a public street. For those lots with two side lot lines backing onto the shoreline of Nottawasaga Bay, no buildings or structures shall be located within 7.6 metres of a rear lot line; no fence shall be permitted along a rear lot line and no fence shall encroach within 6 metres of a rear lot line. In addition to the list of permitted Non-Residential Uses, a private clubhouse shall be permitted.

**4.4.14 R1-14 Zone Schedule "H"**

A Bed and Breakfast use and an accessory building with the installation of plumbing shall be permitted. Such accessory building shall not exceed 28 square metres in area and shall not be used or human habitation.

**4.4.15 R1-15 Zone Schedule "T"**

The front yard depth minimum shall be 12.0 metres.

**4.4.16 R1-16 Zone Schedule "C"**

The minimum front yard depth shall be 10 m and the front wall of an attached garage shall project no more than 1m beyond the front wall of the habitable area of the dwelling***.***

**4.4.17 R1-17 Zone Schedule "E"**

A rest home shall also be permitted. The maximum number of residents shall be eight (8).

**4.4.18 R1-18 Zone Schedule**

A bed and breakfast use shall also be permitted and the maximum driveway width shall be 8.4 metres.

**4.4.19 R1-19 Zone Schedule "T"**

The minimum separation requirement of Zoning By-law 2003-60 for a driveway setback from a street line shall be 7.0 metres.

**4.4.20 R1-20 Zone Schedule "T"**

The minimum interior side yard width shall be 1.35 metres.

**4.4.21 R1-21 Zone**

The use permitted shall be non-residential and shall be restricted to a

Private Community Centre.

### 4.4.22 R1-22 Zone Schedule "F"

The minimum lot area shall be 585 sq.m. The Minimum Lot Frontage shall be 18 metres. The minimum Exterior Side Yard shall be 4.0 metres, except where the front elevation of a garage faces a street-line, in which case the Exterior Side Yard to the garage shall be 6 metres. The maximum distance from the main wall of the dwelling unit to the front elevation of a garage shall be 1.5 metres. The minimum Interior Side Yard shall be 1.5 metres. The maximum Lot Coverage shall be 40%.

### 4.4.23 R1-23 Zone Schedule "T"

The minimum front yard setback requirement shall be 5.3 metres, the minimum exterior side yard setback requirement shall be 2.1 metres, the minimum interior side yard setback shall be .8 metres.

### 4.4.24 R1-24 Zone Schedule "S"

A 2.0 metre wide planting strip consisting of existing vegetation shall be required along the east interior side lot line.

### 4.4.25 R1-25 Zone Schedule "R"

Permitted residential uses shall be limited to single-detached dwelling units;

Section 4.3, Zone Provisions, applies to the lands zoned R1-25, except where specifically identified below:

Minimum lot depth shall be 30 metres; Minimum lot area shall be 360 square metres; Minimum front yard depth shall be 4.0 metres to the building face and side of an attached garage and 6.0 metres to the front of an attached garage; Where a garage is attached to a residential dwelling unit, the maximum projection of the face of the attached garage from the building face shall be 2.4 metres; Minimum exterior side yard width shall be 4.0 metres to the building face and 6.0 metres to the front of an attached garage; Minimum interior side yard width shall be 1.2 metres to the building face and to the rear or side of an attached garage and 8.0 metres to the front of an attached garage when garage is facing the side yard; Minimum rear yard depth shall be 7.5 metres to the building face and 0.6 metres to the garage wall of a detached garage in the rear lot; Minimum landscaped open space shall be 30%;Maximum lot coverage shall be 55%; Maximum driveway width for single-detached dwelling units shall be 6.0 metres; Maximum height of building for single-detached dwelling units shall be 11.0 metres; Minimum outdoor amenity space excluding the driveway for single-detached dwelling units shall be 90 square metres per unit and having no dimension less than 6.0 metres; and one (1) attached accessory dwelling unit per lot, subject to Subsection 3.28.

Notwithstanding the General Provisions, Section 3.3.4 and 3.3.6 to the contrary, the following provisions shall apply: porches/verandas shall be permitted to encroach by sixty (60%) percent and steps by eighty (80%) percent into any required front or exterior side yard setback depth. Porch/verandas shall not be subject to nor restricted to a maximum area size. Maximum height of porch/verandas and steps within permitted encroachment shall be 1.6 metres.

**4.4.26 R1-26 Zone Schedule "O"**

Notwithstanding Subsection 3.18 of the Zoning By-law, the existing single detached dwelling may be enlarged.

**4.4.27 R1-27 Zone Schedule "G"**

Notwithstanding Sections 3.1.2 and 3.18, the maximum lot area that shall be used by detached accessory buildings is 98 square metres , and the maximum horizontal dimension of a detached accessory building shall be 9.2 metres. A building or structure may be constructed on a lot which does not have frontage on a municipal street.

**4.4.28 R1-28 Zone Schedule "F"**

The minimum lot area shall be 585 square metres; the minimum lot frontage shall be 18 metres; the minimum exterior side yard shall be 4 metres, except where the front elevation of a garage faces a street-line, in which case the exterior side yard to the garage shall be 6 metres; the maximum distance from the main wall of the dwelling unit to the front elevation of a garage shall be 1.5 metres; the minimum interior side yard shall be 1.5 metres; the maximum lot coverage shall be 40%; notwithstanding General Provision 3.9.1 and the exterior side yard provisions of the R1.28 Zone, for corner lots at the intersection of a collector road and a local road where the conveyance of a 10 metre by 4 metre daylighting triangle affects the proposed building envelope, the exterior side yard setback shall be measured from a line created by projecting the lot line to the point of intersection of the two street lines, rather than the new lot line created by the dedication of the daylighting triangle.

**4.4.29 R1-29 Zone Schedule "F"**

The minimum lot area shall be 585 square metres; the minimum lot frontage shall be 18 metres; the minimum exterior side yard shall be 4 metres, except where the front elevation of a garage faces a street-line, in which case the exterior side yard to the garage shall be 6 metres; the maximum distance from the main wall of the dwelling unit to the front elevation of a garage shall be 1.5 metres; the minimum interior side yard shall be 1.5 metres; the maximum lot coverage shall be 40%.

**4.4.30 R1-30 Zone Schedule "G"**

In addition to Section 4.3, Zone Provisions, the following applies to the lands zoned R1-30:

A maximum of four existing accessory structures are permitted; the minimum side yard setback from the north interior lot line for existing accessory structures shall be .44 metres and the minimum interior side yard setback from the southern interior lot line for existing accessory structures shall be .9 metres; the minimum rear yard setback for existing accessory structures shall be .86 metres; the accessory structures shall not be connected to any private or municipal water and sewer services, shall not contain plumbing fixtures, may be used for storage or sleeping quarters only and shall not be used as a tourist establishment.

**4.4.31 R1-31 Zone Schedule "H"**

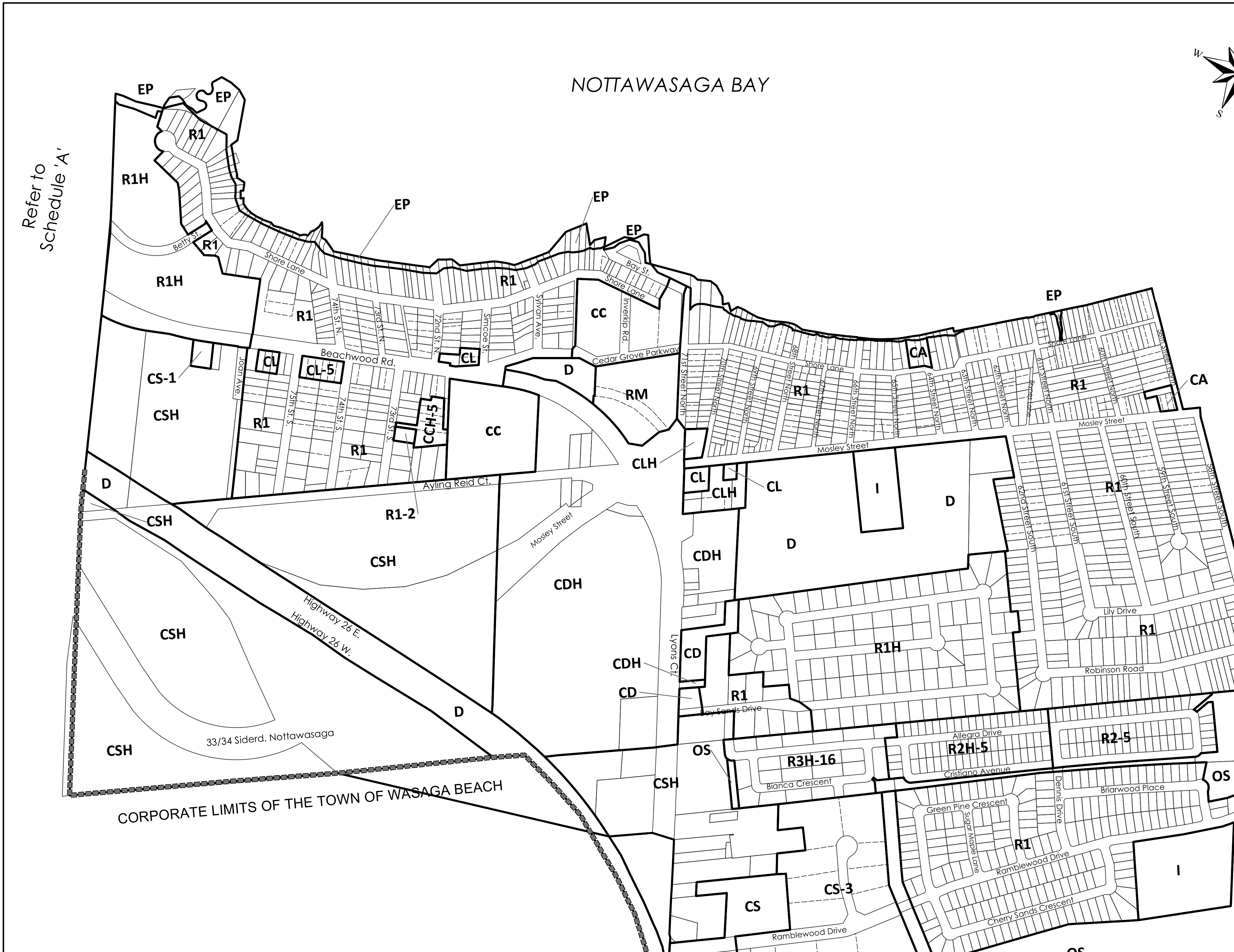
Notwithstanding Subsection 3.18 Street Requirements of the Zoning By-law, buildings and structures may be erected on lands that do not front onto a municipal street.

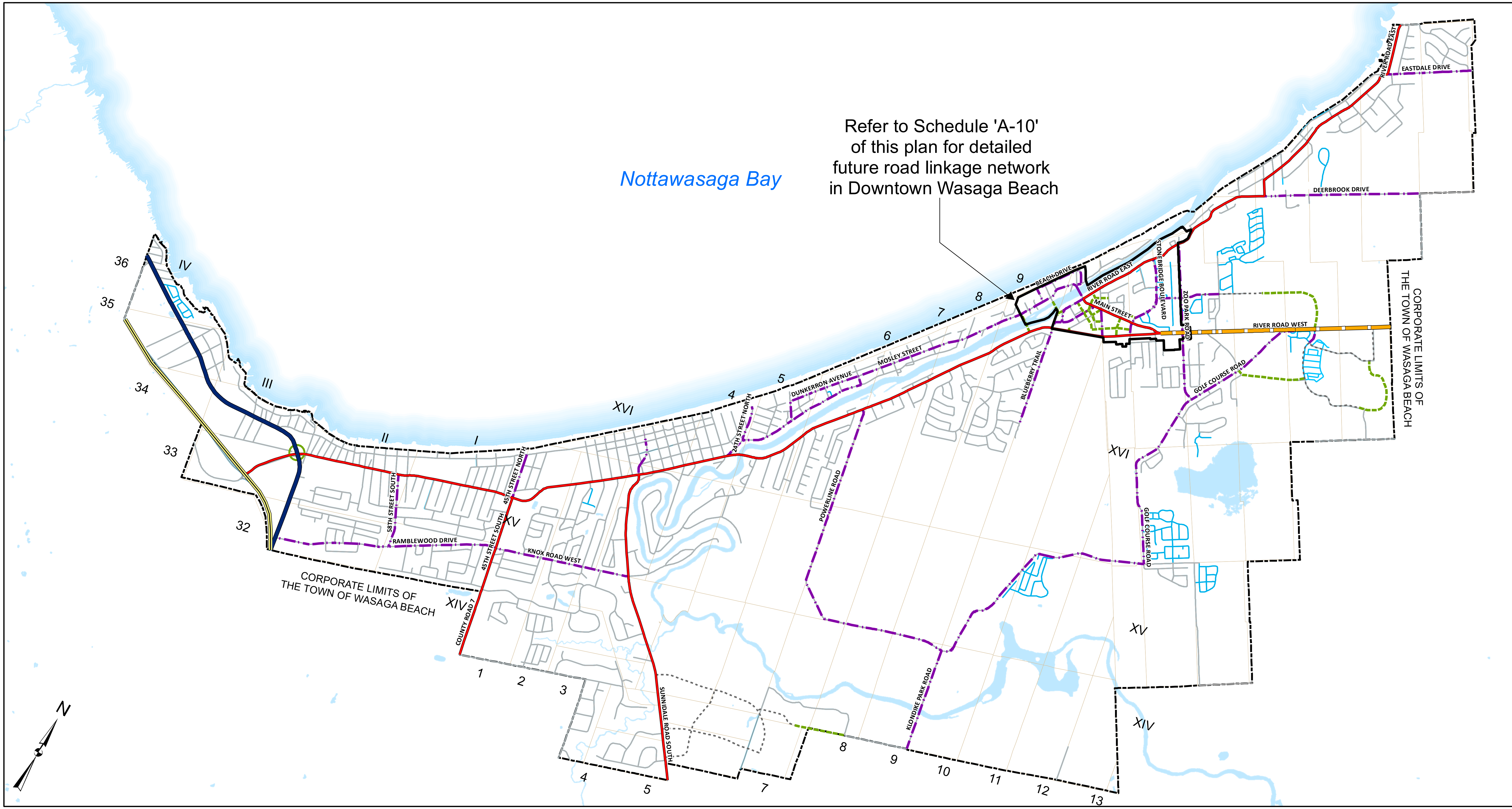
**4.4.32**

**4.4.33**

**4.4.34 R1-34 Zone Schedule "T"**

Notwithstanding the provisions of Section 4 the minimum lot area requirement shall be 400 square metres. Notwithstanding the provisions of Section 3.3 steps may project a maximum distance of 2.4 meres into a required front yard and a deck greater than 0.6 meres in height may project 1.2 metres into a required front yard.


NOTTAWASAGA BAY
Refer to Schedule 'A'
CORPORATE LIMITS OF THE TOWN OF WASAGA BEACH
EP
R1
R1H
CS-1
CSH
CL
CL-5
CCH-5
CC
D
RM
CLH
R1-2
CDH
CD
OS
R3H-16
R2H-5
R2-5
CS-3
CS
I
CA
Beachwood Rd.
Shore Lane
Betty St.
Joan Ave.
75th St. S.
74th St. N.
74th St. S.
73rd St. N.
73rd St. S.
72nd St. N.
Simcoe St.
Sylvan Ave.
Inverkip Rd.
Bay St.
Cedar Grove Parkway
71st Street North
70th Street North
69th Street North
68th Street North
67th Street North
66th Street North
65th Street North
64th Street North
63rd Street North
62th Street North
61st Street North
60th Street North
58th Street North
Brynes Lane
Mosley Street
62nd Street South
61st Street South
60th Street South
59th Street South
58th Street South
Lily Drive
Robinson Road
Ayling Reid Ct.
Highway 26 E.
Highway 26 W.
33/34 Siderd. Nottawasaga
Lyons Ct.
Bay Sands Drive
Allegra Drive
Cristiano Avenue
Bianca Crescent
Dennis Drive
Briarwood Place
Green Pine Crescent
Sugar Maple Lane
Ramblewood Drive
Cherry Sands Crescent
W
S




Adopted by Council: September 9, 2003

Approved by the County of Simcoe: June 22, 2004

Consolidated: January 2020

NOTE: The lot lines depicted on this map are for reference only and may not reflect accurately property boundaries in all instances.

0 300 600 1,200 1,800 2,400 3,000
Metres

**Legend**

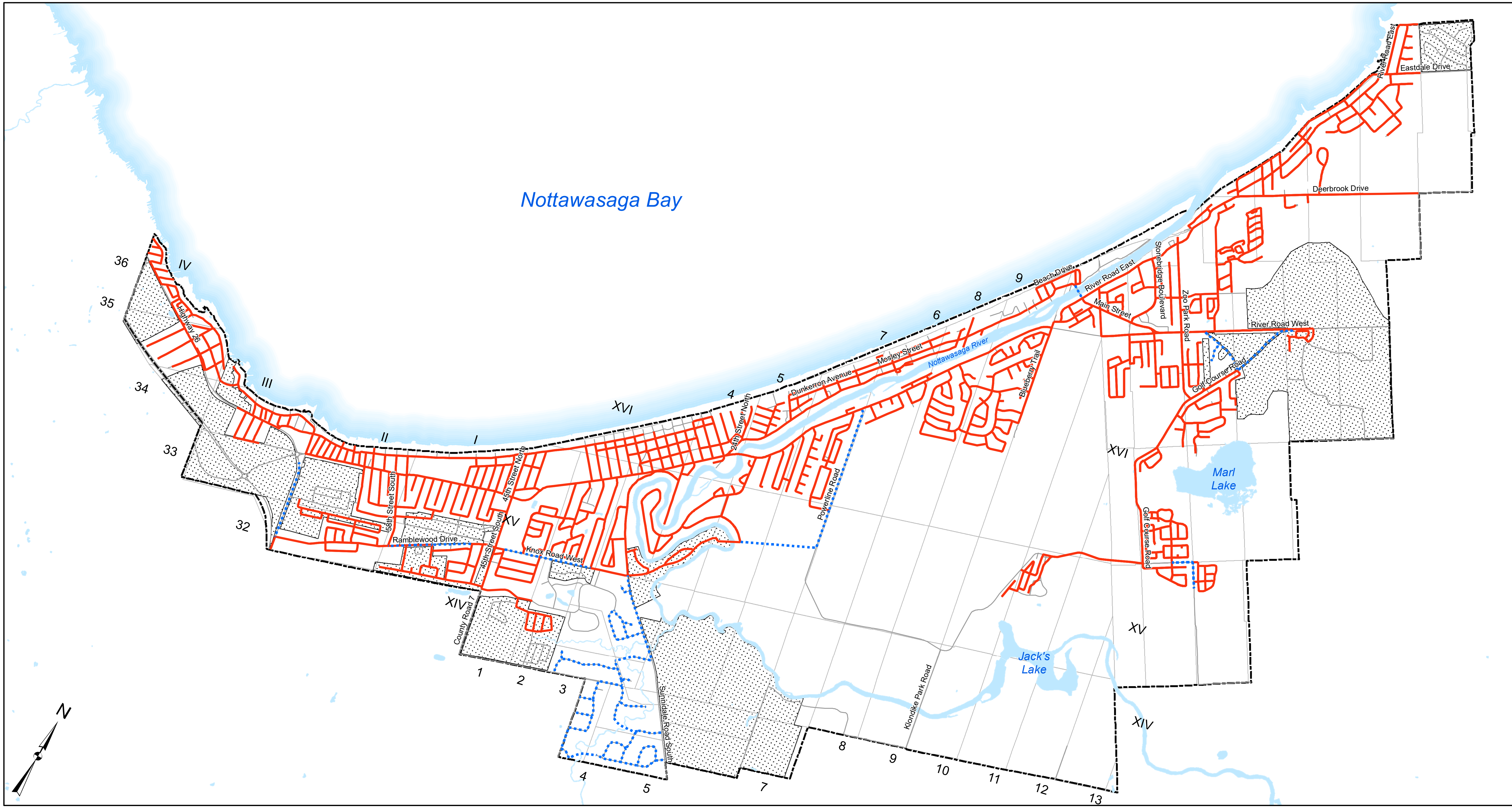
- Provincial Highway/Future Collector Road
- Provincial Highway 26
- Planned Interchange
- Controlled Access
- Arterial Road
- Collector Road
- Planned Collector Road (Not Built)
- Future Road Linkage
- Local Municipal Road
- Private Local Road

TOWN OF WASAGA BEACH INCORPORATED 1974

# Schedule 'B'

## Transportation System

Official Plan for theTown of Wasaga Beach




Adopted by Council: September 9, 2003

Approved by the County of Simcoe: June 22, 2004

Consolidated: January 2020

NOTE: The lot lines depicted on this map are for reference only and may not reflect accurately property boundaries in all instances.

0 300 600 1,200 1,800 2,400 3,000
Metres

**Water Servicing Only**

**Water and Sewer Servicing**

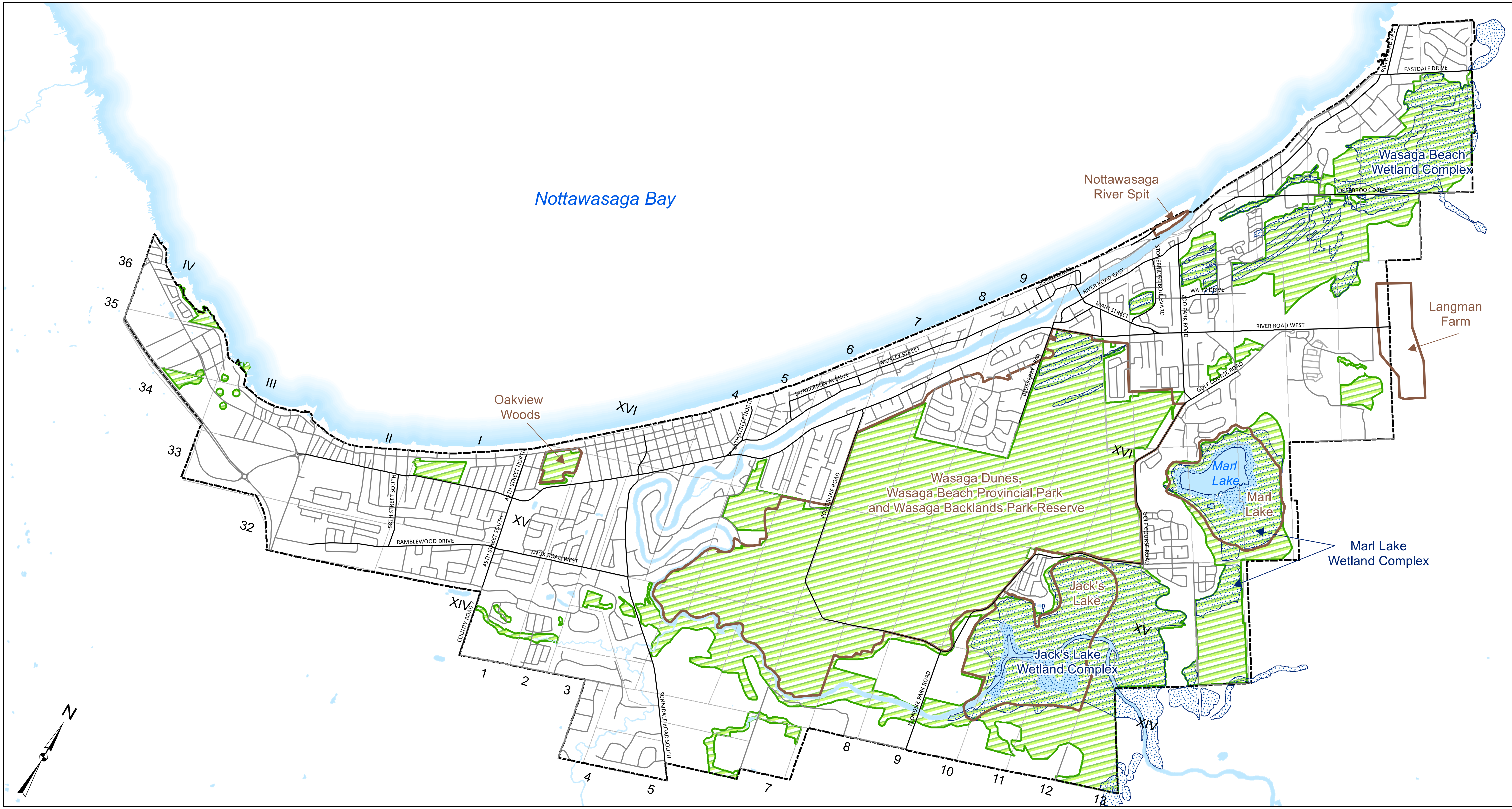
**Future Staging Areas**

TOWN OF WASAGA BEACH INCORPORATED 1974

# Schedule 'C'

## Municipal Servicing Area

Official Plan for theTown of Wasaga Beach




Adopted by Council: September 9, 2003

Approved by the County of Simcoe: June 22, 2004

Consolidated: January 2020

NOTE: The lot lines depicted on this map are for reference only and may not reflect accurately property boundaries in all instances.

0 300 600 1,200 1,800 2,400 3,000
Metres

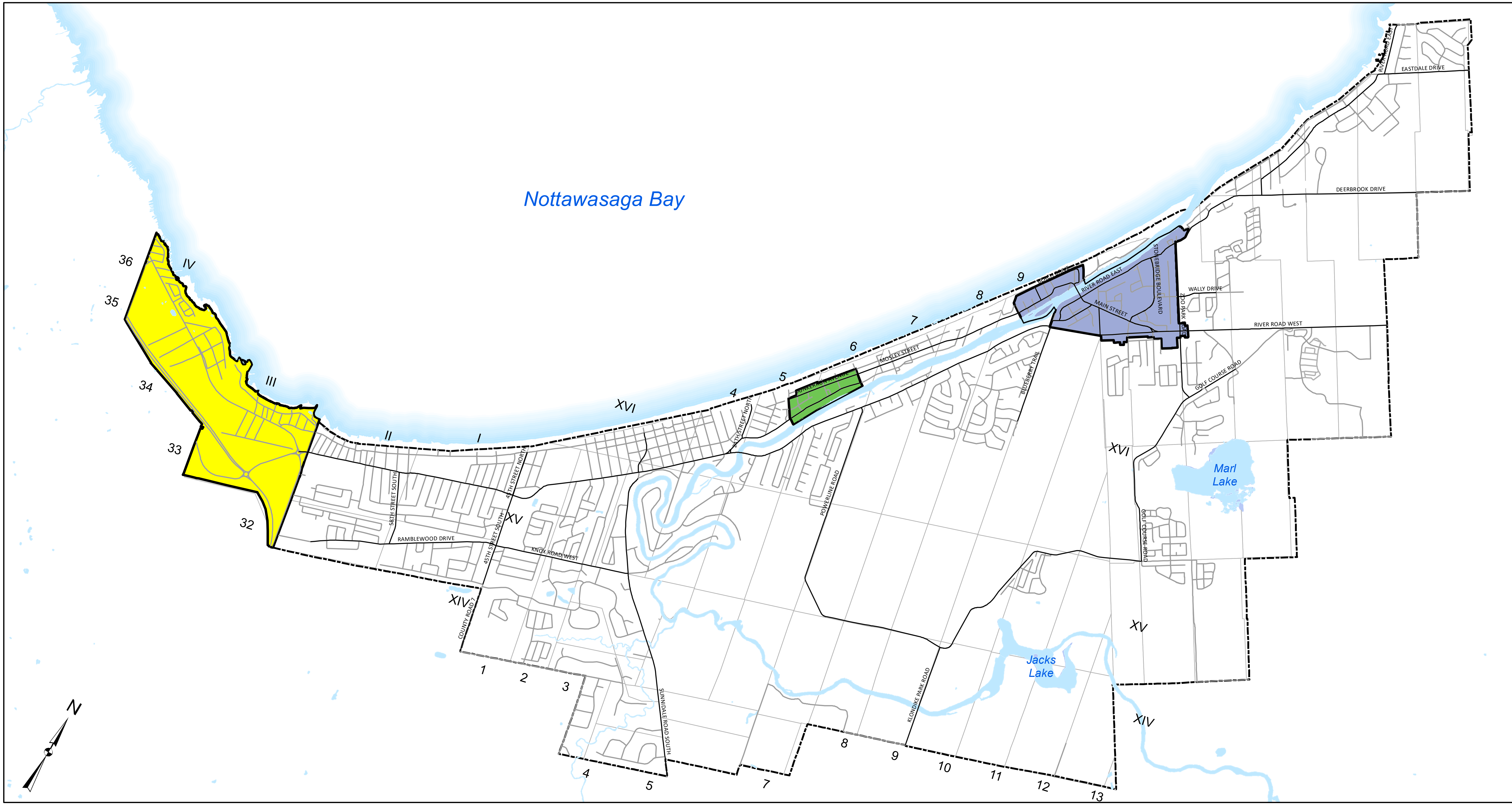
- ANSI (Area of Natural and Scientific Interest)
- Provincially Significant Wetland Complex
- Natural Heritage System Category 1 and 2 Lands

TOWN OF WASAGA BEACH INCORPORATED 1974

# Schedule 'D'

## Natural Heritage System

Official Plan of theTown of Wasaga Beach




Adopted by Council: September 9, 2003

Approved by the County of Simcoe: June 22, 2004

Consolidated: January 2020

NOTE: The lot lines depicted on this map are for reference only and may not reflect accurately property boundaries in all instances.

0 300 600 1,200 1,800 2,400 3,000

Metres

Legend

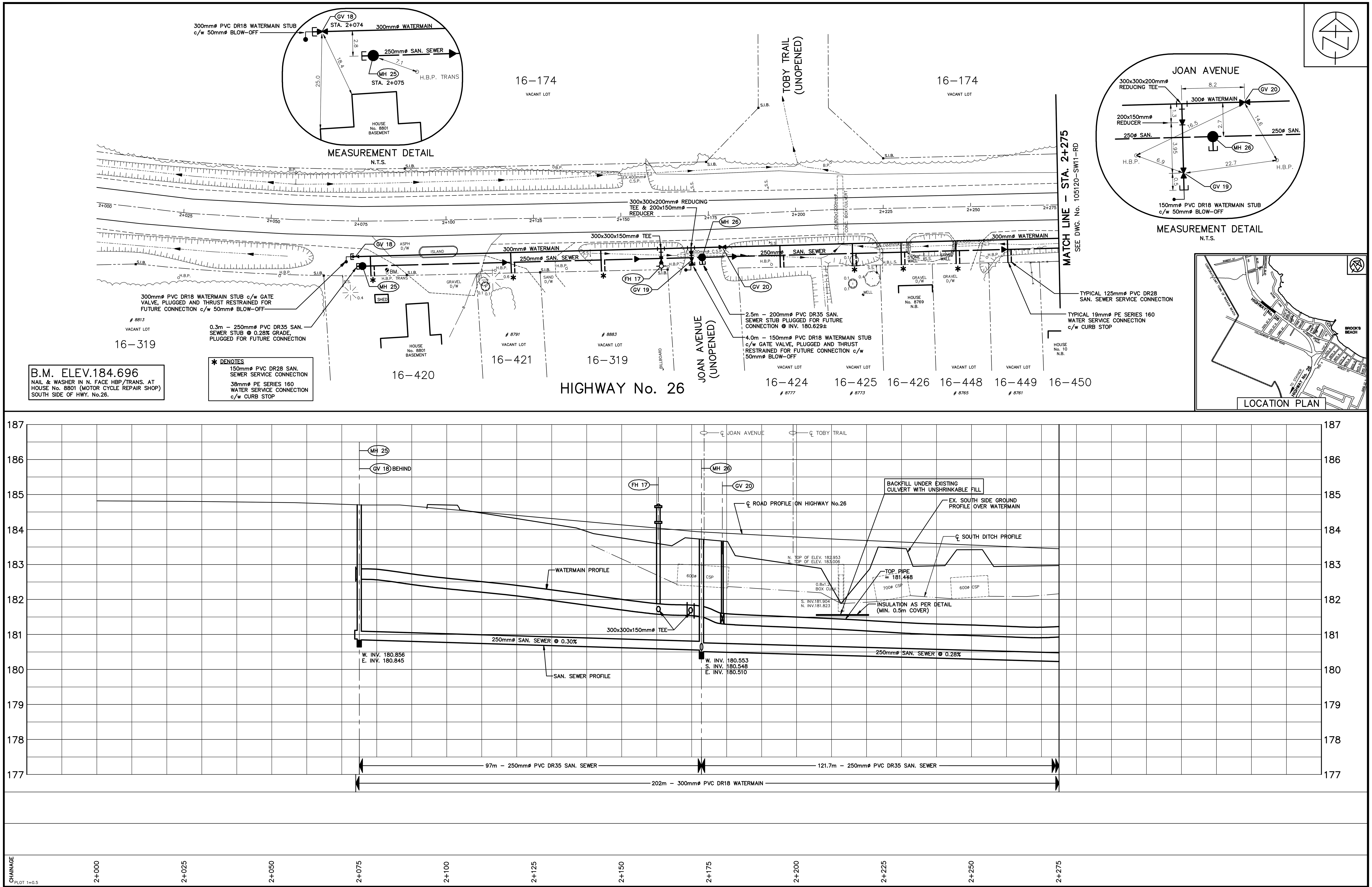
- Downtown Wasaga Community Improvement Project Area
- Mosley Village Community Improvement Project Study Area
- West End Community Improvement Project Study Area

TOWN OF WASAGA BEACH INCORPORATED 1974

# Schedule 'H'

## Community Improvement Project Areas

Official Plan for theTown of Wasaga Beach




## LEGEND

- PROPERTY LINE
- B — BURIED BELL CABLE
- G — BURIED GAS MAIN
- CTV — BURIED CABLE TV
- EXISTING WATER MAIN AND VALVE
- EX. SANITARY SEWER & M.H.
- PROPOSED WATER MAIN AND VALVE
- PROPOSED SANITARY SEWER
- PROPOSED SANITARY FORCEMAIN
- MH — PROPOSED SANITARY M.H.
- GV — PROPOSED GATE VALVE
- FH — PROPOSED FIRE HYDRANT, VALVE & HYDRANT LEAD

NOTE:
1. LOCATIONS OF UTILITIES ARE APPROXIMATE.
2. THE MINIMUM CLEAR SEPARATION BETWEEN WATERMAINS AND SANITARY SEWERS SHALL BE 2.5m HORIZONTALLY OR 0.5m VERTICALLY.

## RECORD DRAWING

| | |
|---|---|
| COMPILED BY: D.H. | DATE: JAN. 2009 |
| CHECKED BY: D.B.E. | DATE: JAN. 2009 |
| DRAWN BY: P.C.S. | DATE: MAY 2010 |
| CHECKED BY: D.B.E. | DATE: MAY 2010 |

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| | REVISED AS RECORD DRAWING | MAY 2010 | D.B.E. |

**RECORD DRAWING**
NOTICE TO USERS

Information contained on this drawing may have changed since the completion of construction. Further, this drawing may contain information provided by others. While Ainley & Associates Limited believes this information to be reliable and correct as of the completion of construction, no warranty is provided as to its accuracy and/or completeness. As such, Ainley & Associates Limited shall not be responsible for any errors or omissions which may result from incorporation of the information herein.

| | |
|---|---|
| SCALE: | HORIZ. 1=500 VERT. 1=50 |
| DESIGN: | D.B.E. |
| DRAWN: | P.C.S. |
| CHECKED: | D.B.E. |
| DATE: | AUG. 2007 |

TOWN OF WASAGA BEACH

WEST END SERVICING

PLAN & PROFILE
HIGHWAY No. 26
STA. 2+000 TO STA. 2+275

Ainley GROUP — CONSULTING ENGINEERS PLANNERS

CONTRACT No. 15 | DWG. 105120-SW10-RD

## Bottom Land

Low lying soils adjoining stream courses and subject to flooding are designated as Bottom Land. These soils occupy 4,900 acres in the County. They are immature soils showing none of the horizon differentiation common to other mineral soils. The profile commonly consists of layers of sand, silt or gravel which are grey in colour except when organic material has been mixed with them, in which case, the layer is dark in colour. The drainage varies from imperfect to poor but is usually poor.

Bottom Land is used largely for pasture.

## Marsh

Areas that are covered by water for the entire year are mapped and designated as Marsh. These areas, occupying 2,800 acres, are covered by water-loving plants such as cattails and sedges and by scrub vegetation, the chief of which is willow. The soil surface may consist of sand, clay or thin muck deposits. These areas are unsuitable for agricultural purposes.

## Eastport Series

The Eastport series consists of one type, the sand, and it occurs along parts of the shore of Nottawasaga Bay. Here it occurs in the form of dunes and the topography is rough and slopes are often short and steep. The soil is rapidly drained, with small depressional areas of poor drainage being present. Vegetative cover is scanty but where it is present, it consists mainly of pine, poplar and some white birch.

There is practically no horizon development in the profile. In most cases, the profile consists simply of a grey calcareous sand. Where the sand has been stabilized by vegetation, the profile has a thin organic layer underlain by a pale yellow horizon which in turn rests on the grey sand.

This sand is subject to wind erosion and has no value as agricultural land. It is used for recreational purposes and as building sites for summer cottages.

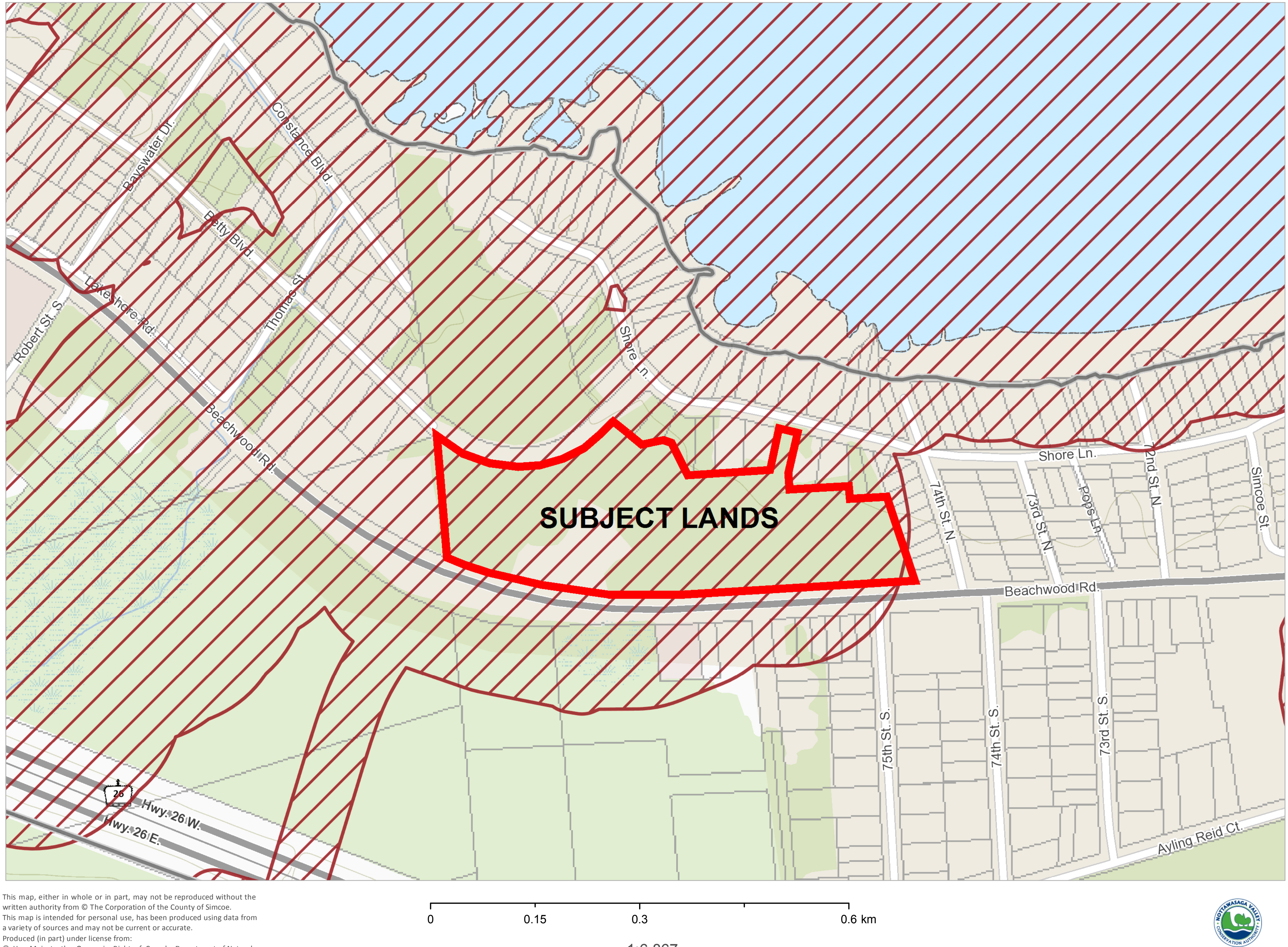
## Rock

Areas of bare rock with little or no soil cover have been designated as Rock. Rock occupies 1,100 acres and most of it is found in Matchedash Township. It has no agricultural value.

## Soil Complexes

Soil complexes are mapping units that represent a combination of two or more soil types that cannot be separated individually on a soil map. In naming each complexed area, the names of the two dominant types are used. Thus, an area in which Simcoe silty clay loam and Berrien sandy loam were dominant would be called Simcoe silty clay loam — Berrien sandy loam. In each case, the soil mentioned first occupies the largest acreage. Soils other than those mentioned in the soil complex may occur. These are com-

# Shore Lane - NVCA Regulated Area






April 17, 2020

# Shore Lane - Site Location




April 17, 2020

# Shore Lane - Forested Area

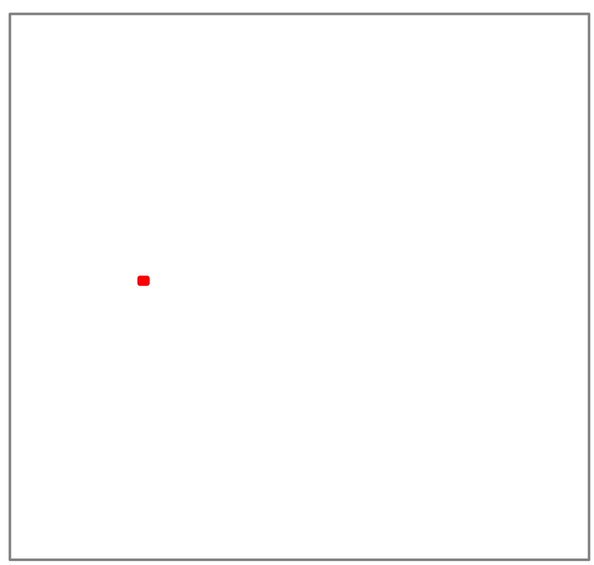

Municipal Border
Contour - 2 Meter
Trail
RailLine
Highway
Major Road
Ramp
Local Roads
Unassumed and Seasonal Roads

Water Line, Intermittent
Water Line, Permanent
Building Footprint
Provincial Park
Water Area, Permanent
Wetland Area, Permanent
Wooded Area
Cartographic Urban Area
Cemetery - Polygon

First Nations Land
Municipal Park

**Landuse**

Amusement Park
Commercial
Educational
Golf Course
Government
Industrial
Medical





0 0.125 0.25 0.5 km

1:8,953

April 17, 2020

# Shore Lane - Site Location Image


- Municipal Border
- Contour - 2 Meter
- Trail
- RailLine
- Highway
- Major Road
- Ramp
- Local Roads
- Unassumed and Seasonal Roads
- Water Line, Intermittent
- Water Line, Permanent
- Building Footprint
- Provincial Park
- Water Area, Permanent
- Wetland Area, Permanent
- Wooded Area
- Cartographic Urban Area
- Cemetery - Polygon
- First Nations Land
- Municipal Park

**Landuse**

- Amusement Park
- Commercial
- Educational
- Golf Course
- Government
- Industrial
- Medical



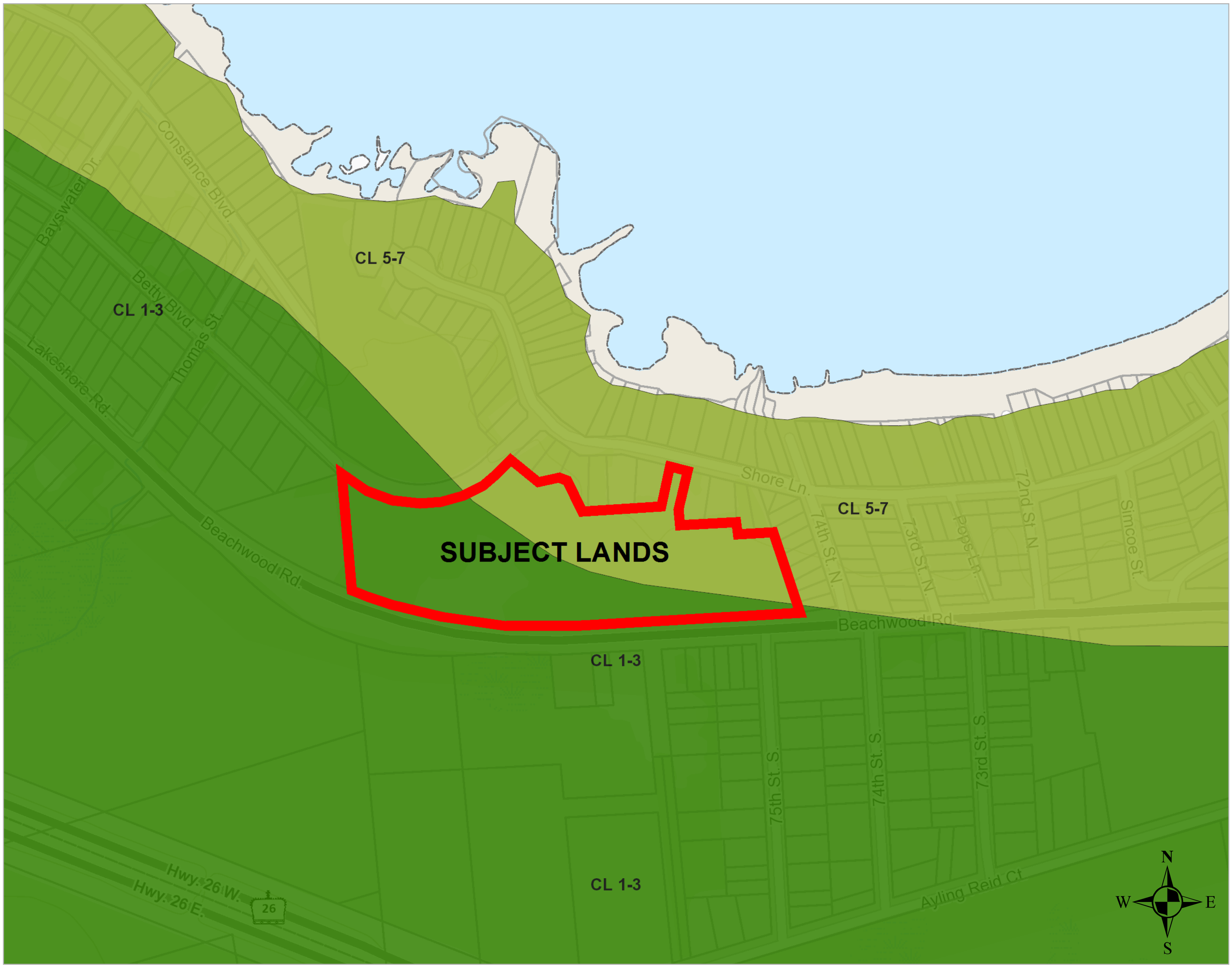
0 0.05 0.1 0.2 km

1:4,477

COUNTY OF SIMCOE

April 20, 2020

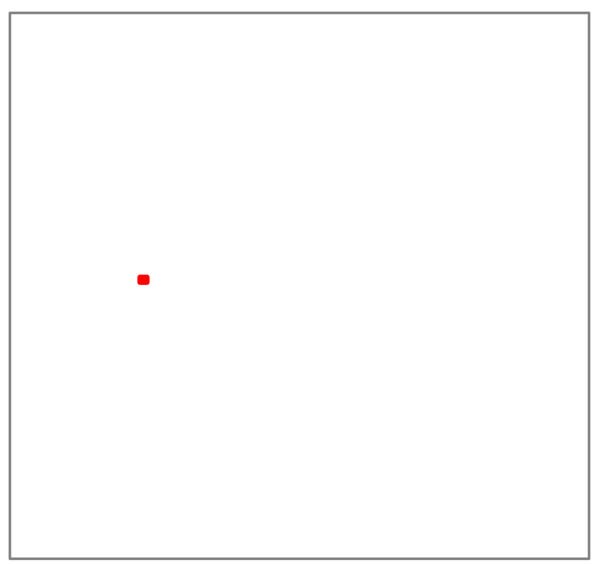
# Shore Lane - Soil Class




Municipal Border
Contour - 2 Meter
Trail
RailLine
Highway
Major Road
Ramp
Local Roads
Unassumed and Seasonal Roads
Water Line, Intermittent
Water Line, Permanent
Building Footprint
Provincial Park
Water Area, Permanent
Wetland Area, Permanent
Wooded Area
Cartographic Urban Area
Cemetery - Polygon
First Nations Land
Municipal Park

**Landuse**

Amusement Park
Commercial
Educational
Golf Course
Government
Industrial
Medical

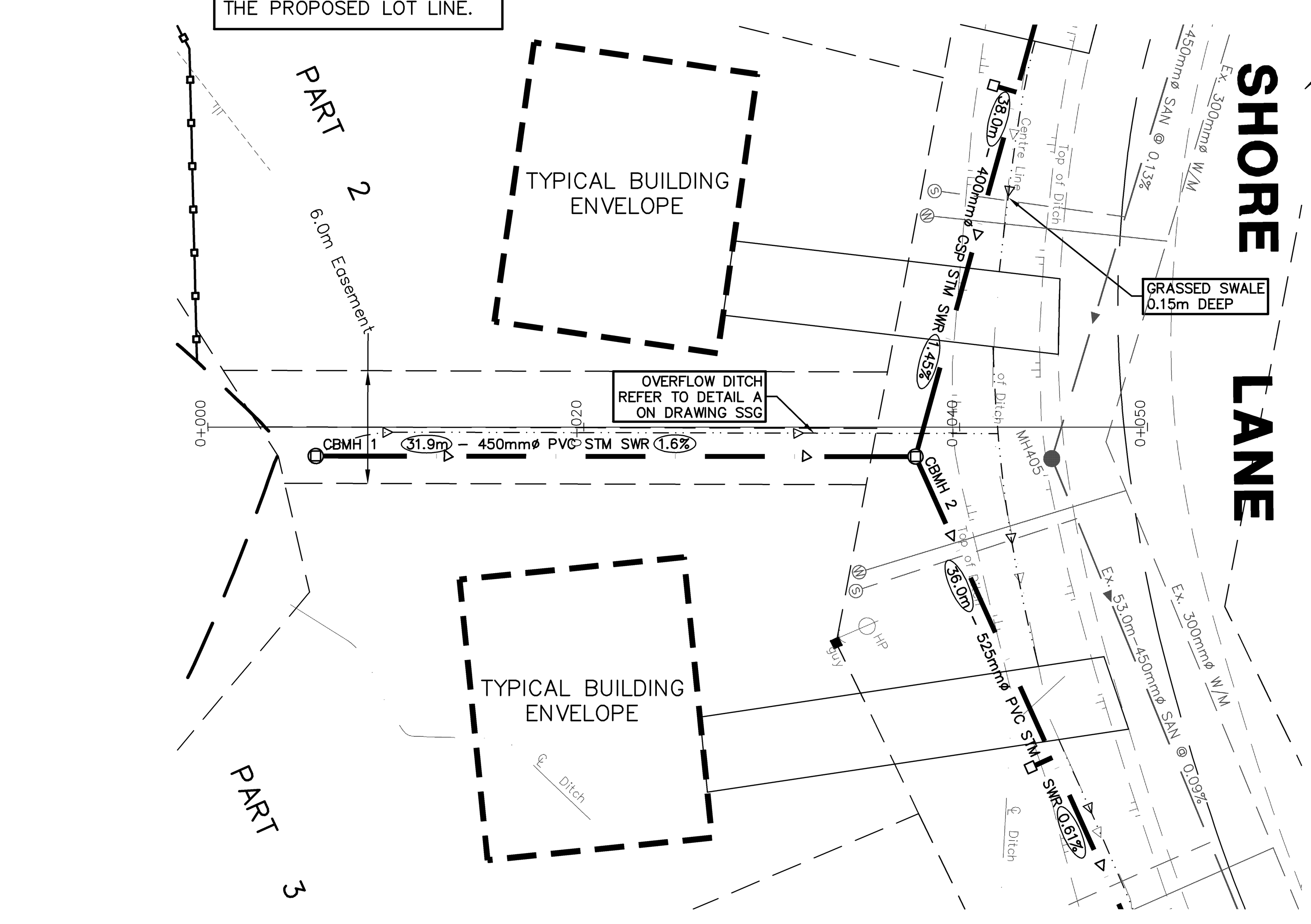


0 0.125 0.25 0.5 km

1:8,953

April 17, 2020

NOTE:
BASELINE ESTABLISHED AS THE PROPOSED LOT LINE.




**CAUTION:**

THE LOCATION OF ALL UNDERGROUND AND ABOVE GROUND UTILITIES AND STRUCTURES ARE NOT NECESSARILY SHOWN ON THE DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE LOCATION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK THE CONTRACTOR SHALL CONFIRM THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND SHALL ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

## RECORD INFORMATION

CHAPMAN PROPERTY

SHORE LANE, TOWN OF WASAGA BEACH

NOTE:

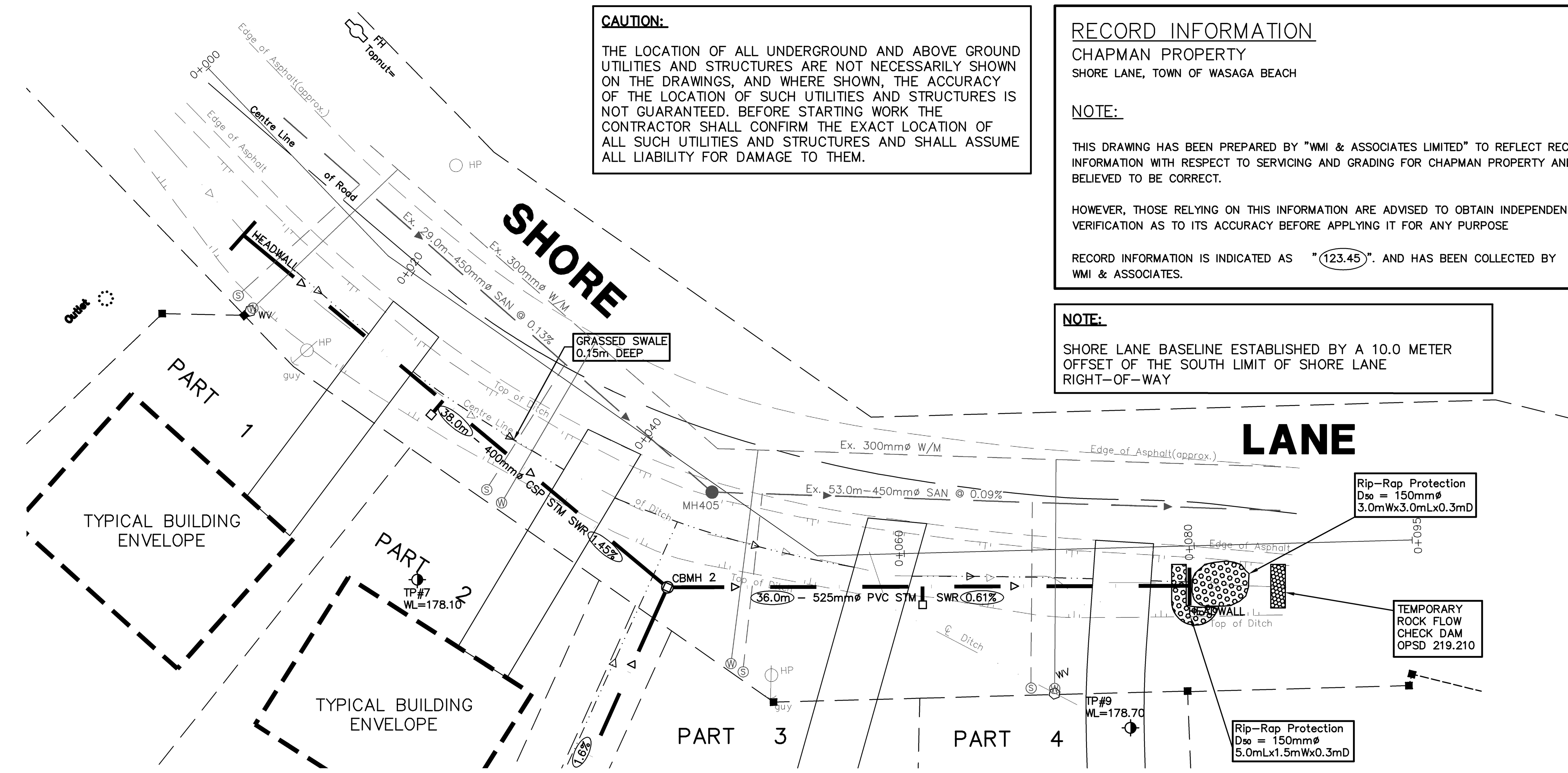
THIS DRAWING HAS BEEN PREPARED BY "WMI & ASSOCIATES LIMITED" TO REFLECT RECORD INFORMATION WITH RESPECT TO SERVICING AND GRADING FOR CHAPMAN PROPERTY AND IS BELIEVED TO BE CORRECT.

HOWEVER, THOSE RELYING ON THIS INFORMATION ARE ADVISED TO OBTAIN INDEPENDENT VERIFICATION AS TO ITS ACCURACY BEFORE APPLYING IT FOR ANY PURPOSE

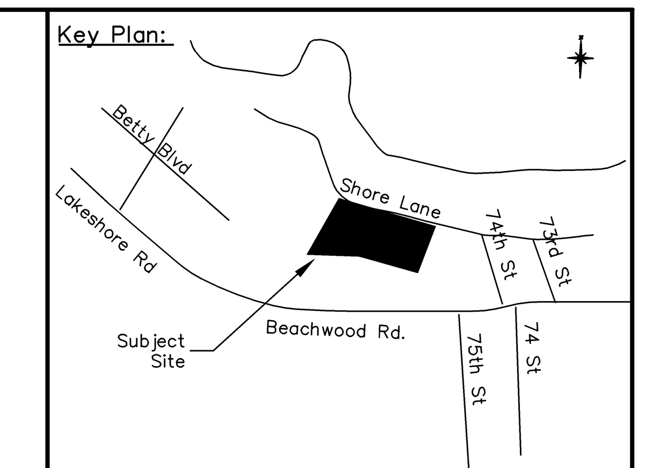
RECORD INFORMATION IS INDICATED AS "123.45". AND HAS BEEN COLLECTED BY WMI & ASSOCIATES.

**NOTE:**

SHORE LANE BASELINE ESTABLISHED BY A 10.0 METER OFFSET OF THE SOUTH LIMIT OF SHORE LANE RIGHT–OF–WAY




Key Plan:



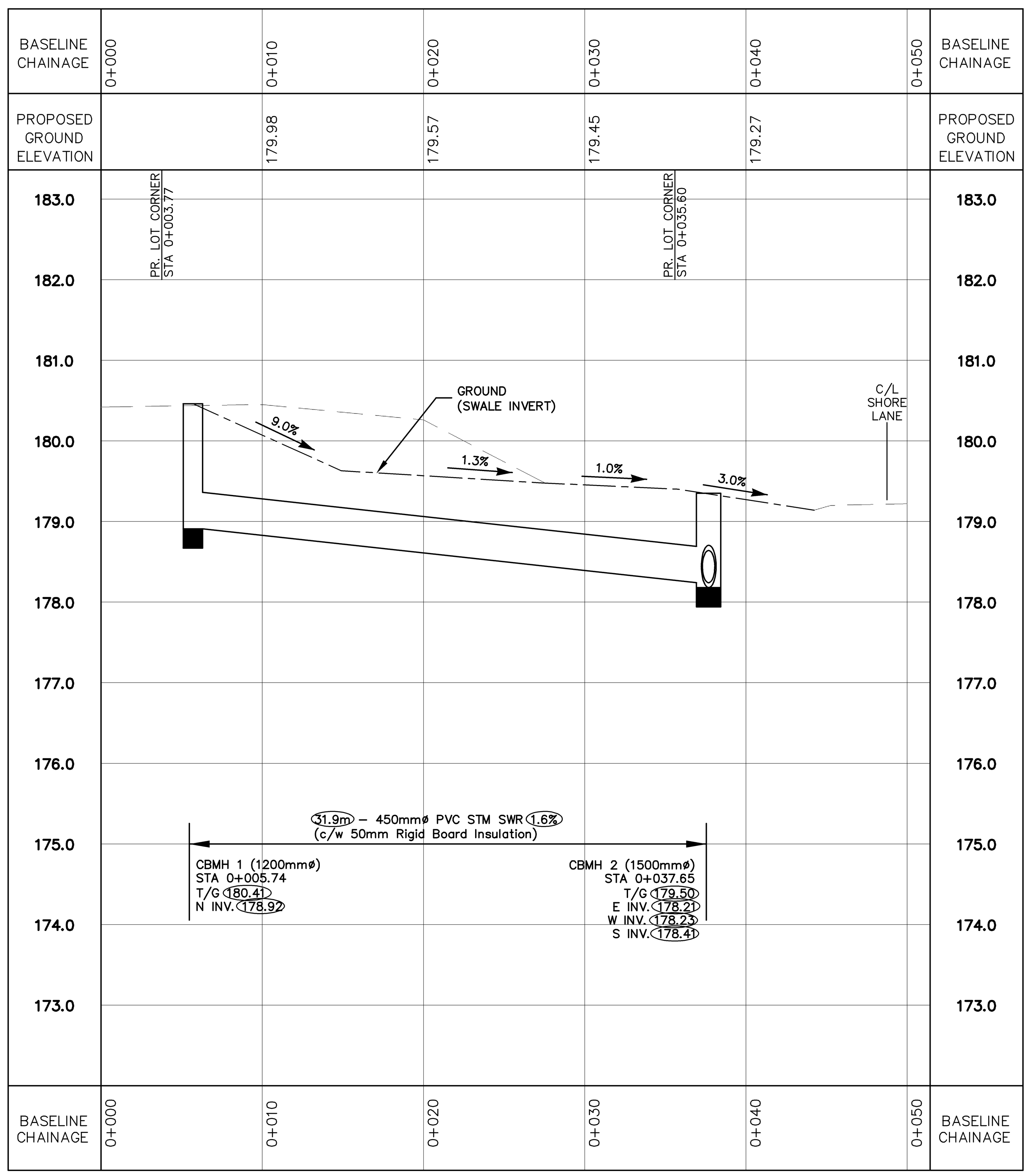

Legend:

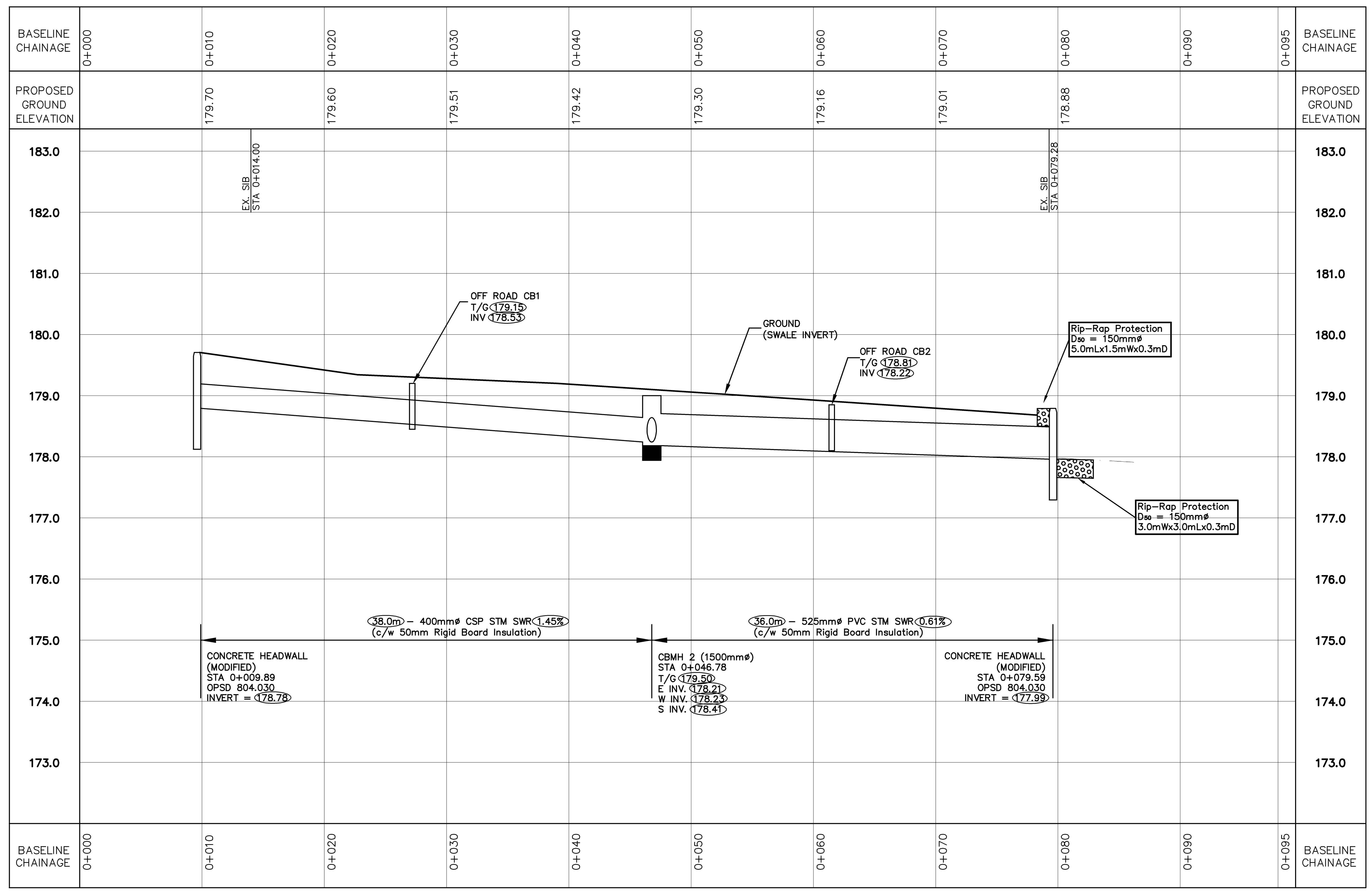
EXISTING FEATURES (EX)

- EX STD IRON BAR
- EX IRON BAR
- EX UTILITY POLE
- EX BELL PED
- EX ELEVATION
- EX WATER SERVICE
- EX FIRE HYD.
- EX ST NAME SIGN
- EX STOP SIGN
- EX ELEVATION (TO REMAIN)
- EX FENCE
- EX U/G GASMAIN
- EX U/G BELL
- EX WATERMAIN & VALVE
- EX SAN SEWER & MH
- EX STM SEWER & MH

PROPOSED FEATURES (PR)

- PR STREET NAME SIGN
- PR STOP SIGN
- PR FENCE
- PR STREET LIGHT
- PR WATER SERVICE
- PR SAN SERVICE
- PR HYDRO TRANSFORMER
- PR WATERMAIN & VALVE
- PR FIRE HYDRANT
- PR WATER VALVE
- PR SAN SEWER
- MH K — PR SANITARY MANHOLE
- PR STM SEWER
- MH 4 — PR CATCHBASIN MANHOLE
- MH 4 — PR MANHOLE
- PR CATCHBASIN
- 000.00 — PROPOSED ELEVATION
- PR DRAINAGE DIRECTION
- PR SWALE
- FFF — PR. FINISHED FIRST FLOOR ELEVATION
- TF — PR. TOP OF FOUNDATION ELEVATION
- FBF — PR. FINISHED BASEMENT FLOOR ELEVATION
- USF — PR. UNDERSIDE OF FOOTING ELEVATION
- AWL — APPROXIMATE WATER LEVEL (SEE NOTE 4.)
- PR. BUILDING FOOTPRINT (APPROXIMATE)
- TP# WL=123.45 — TEST PIT# WATER LEVEL (SEE NOTE 4.)




Notes:

1. BASEPLAN INFORMATION (PROPERTY LINES, EXISTING ELEVATIONS AND INFORMATION) ARE BASED ON THE DRAWING PREPARED BY PATTEN & THOMSEN.

2. THE BUILDING FOOTPRINTS SHOWN ON EACH LOT ARE APPROXIMATE ONLY AND ARE SUBJECT TO CHANGE IN ACCORDANCE WITH THE FINAL ARCHITECTURAL PLANS.

3. THE PROPOSED GRADING ON EACH LOT IS BASED ON THE APPROXIMATE BUILDING FOOTPRINT AND IS SUBJECT TO CHANGE IN ACCORDANCE WITH THE FINAL ARCHITECTURAL PLANS.

4. ALL TEST PIT INFORMATION AND WATER LEVELS ARE BASED ON THE SUBSURFACE INVESTIGATION PREPARED BY TERRAPROBE, DATED DECEMBER 17, 2013. THE TERRPROBE INVESTIGATION SUMMARIZES WATER LEVELS AT EACH TEST PIT AND APPROXIMATE WATER LEVELS FOR THE BUILDING AREA ON EACH LOT.

5. AT SPECIFIC LOCATIONS, THE UNDERSIDE OF FOOTING ELEVATION MAY BE ADJUSTED (STEPPED UP). HOWEVER, IT SHOULD BE NOTED THAT ALL FOOTINGS SHALL HAVE FROST COVER IN ACCORDANCE WITH THE ONTARIO BUILDING CODE.

6. ALL FOOTINGS TO BE CONSTRUCTED IN ACCORDANCE WITH THE SUBSURFACE INVESTIGATION PREPARED BY TERRAPROBE, DATED DECEMBER 17, 2013.

Notes:

1. Unless noted otherwise, the measurements and distances shown on this drawing are shown in meters.
2. Do not scale drawings.
3. It is the contractor's responsibility to verify all dimensions, levels and datums on site and report any discrepancies or omissions to WMI & Associates Ltd. prior to construction.
4. This drawing is to be read and understood in conjunction with all other relevant documents applicable to this project.
5. This drawing is the exclusive property of WMI & Associates Ltd. and the reproduction of any part of this document without prior written consent is strictly prohibited.

Benchmark: 180.36m

Temporary benchmark is the top nut of the existing fire hydrant on the north side of Shore Lane west of Part 1.

| No. | Issue / Revision | Date |
|---|---|---|
| 1 | MOE Submission | Nov. 25, 2013 |
| 2 | First Submission | Feb. 24, 2014 |
| 3 | Second Submission | March 19, 2014 |
| 4 | Record Information (Storm System) | Oct. 14, 2014 |
| 5 | Record Information (Storm System) | Jan. 11, 2016 |

Chapman Property
Shore Lane, Town of Wasaga Beach

SHORE LANE PLAN AND PROFILE

Client:
B. Chapman
7458 Highway No.26
Stayner, Ontario
L0M 1S0


WMI & Associates Limited
119 Collier Street
Barrie, Ontario
L4M 1H5
Ph 705-797-2027
www.wmiengineering.ca


| Drawn By | Checked By | Drawing No. |
|---|---|---|
| TG | RDW | P1 |
| Scale H 1:250 V 1:50 | Project No. 13–232 | |


\\WMI-SERVER\wmi-server\Data\Projects\2013\13-232\CAD\WMI\Record\160111-baseplan-Issue5_Record.dwg, 2016-01-27 3:22:25 PM, 1:24.9999

Simcoe County Soils mapping, soils in the area are characterized as good drainage soil, represented by Gravelly Sandy Loam (Sargent); however; the area along Georgian Bay contains an excessive drainage soil which is Eastport Sand based on Simcoe County soils Mapping. Land use in the area is dominated by woodland with the present of community along Shore Lane land and 75th Street South. A listing of the information sources for the current study is provided in *Table 1*.

**Table 1: Data and sources used for the model preparation**

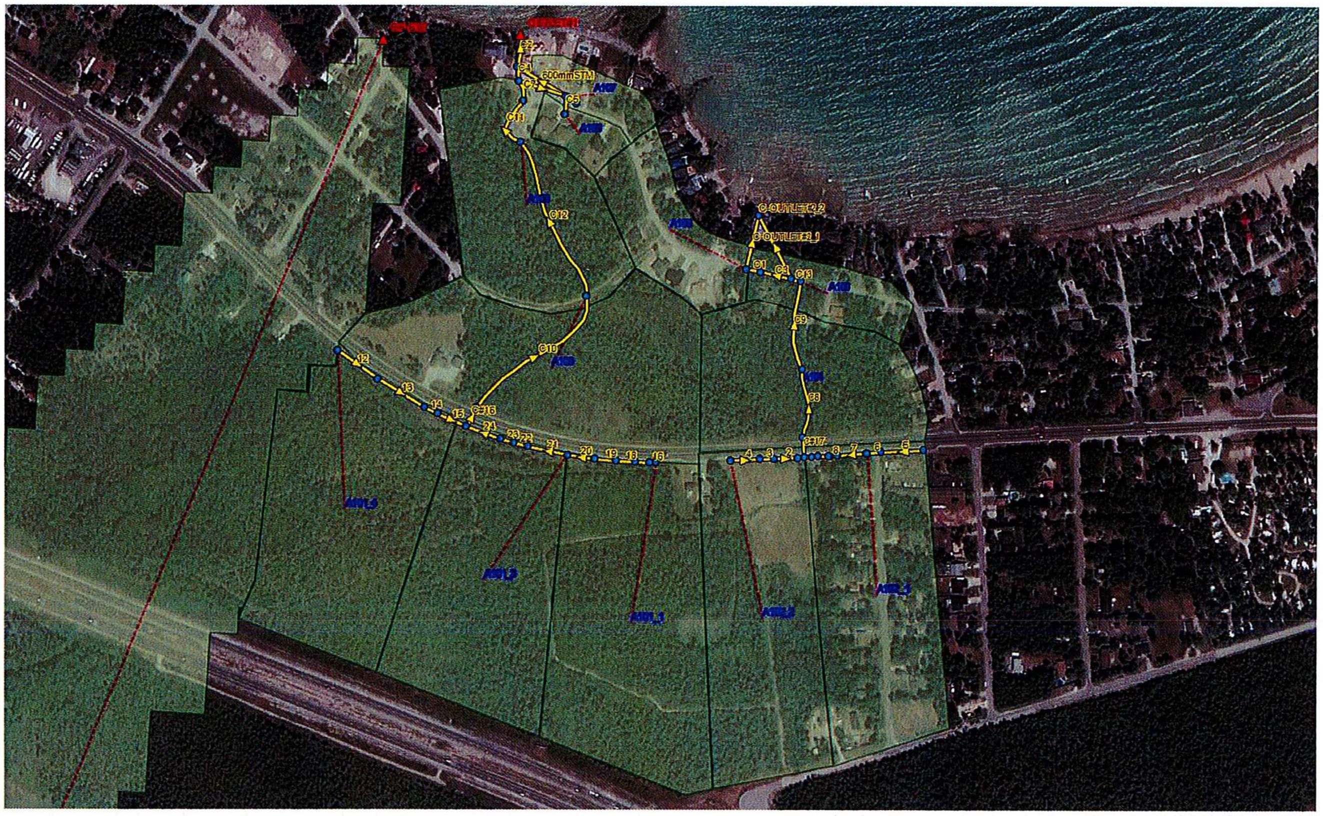
| Date | Sources |
|---|---|
| Soil | Simcoe County Soils Mapping |
| Land Use | GIS Simcoe map |
| Topography | Topographic survey conducted Aug,2018 and (1m) contour map from the county of Wasaga Beach |

A site investigation of the subject property and surrounding drainage areas was conducted on June 29th, 2018 to gain an understanding of the available drainage pathways required to accommodate the development of the proposed site, and provide a cursory assessment of the existing infrastructure. This investigation was also beneficial to identify key elements to be captured in the topographic survey of the area. Culverts, outlets and drainage pathways were identified and confirmed as required to be surveyed where suitable access could be obtained to update our existing condition model. Photos for each culvert and ditches were taken and appended in Appendix A.

A topographic survey of the area was undertaken from June 11 to October 31, 2018 on the subject property as well as adjacent lands. Additional survey was obtained within the Municipal right- of- ways, and select private properties along Ayling Reid Court, 75th Street South, Beachwood Road, 74th Street North, and Shore Lane as shown in Figure 1. Base plans were created from this topographic data and utilized as the basis for the development of a PC-SWMM model of existing conditions. The model was used to establish the expected flows from the tributary areas and provide a preliminary assessment of the available drainage outlets. It is noted that, due to the extensive wood cover on the private properties north of Beachwood Road, the area could not be fully surveyed without extensive clearing of the area. It was agreed with the Nottawsaga Conservation Authority that small portions of the lands could be cleared to establish the key areas such that the existing conditions model can be completed and drainage solutions to accommodate development can be investigated, with supplementary data obtained from the Town's GIS database.

Drainage for this area is directed towards two main outlets. The area draining to Outlet 1, located at the end of the cul-de-sac of Shore Lane, as shown on **Figure 1,** consists of a mix of open field, woodland, and small portion of low to medium intensity development. The total drainage area to Outlet 1 is approximately 26.3 ha. Outlet 2 is located at 2222 Shore Lane, and consists of an 800mm diameter CSP storm sewer and a designed overland flow route constructed in 2013. The total drainage area to outlet 2 is 15.3 ha and the outlet 2 configuration was obtained from ***2222 Shore Lane Overland Drainage Route*** As-Built Drawing #C1 prepared by RJ Burnside & Associates Ltd., Revision 6 on November 27, 2013, and sealed on December 18, 2013, which is scheduled in Appendix B.

## Figure 1: PC-SWMM Model map and elements



## Table 3: Peak flows ($m^3/s$) at Outlets - Scenario 3

| Storms | Outlets | Area (ha) | 2 year | 5 year | 10 year | 25 year | 50 year | 100 year |
|---|---|---|---|---|---|---|---|---|
| 12-hr-SCS | OUTLET# 1 | 26.3 | 0.266 | 0.382 | 0.460 | 0.562 | 0.657 | 0.762 |
| | OUTLET# 2 | 15.3 | 0.322 | 0.399 | 0.504 | 0.667 | 0.787 | 0.908 |
| 4-hr Chicago | OUTLET# 1 | 26.3 | 0.26 | 0.372 | 0.449 | 0.553 | 0.636 | 0.715 |
| | OUTLET# 2 | 15.3 | 0.333 | 0.413 | 0.486 | 0.656 | 0.776 | 0.884 |
| 24-hr-SCS | OUTLET# 1 | 26.3 | 0.276 | 0.400 | 0.485 | 0.626 | 0.770 | 0.936 |
| | OUTLET# 2 | 15.3 | 0.346 | 0.430 | 0.575 | 0.782 | 0.95 | 1.154 |

the impact of improvements at each outlet since the 2009 and to account for any reported issues within these two watersheds. Culvert crossings in the model are numbered C16 and C17 as shown in ***Figure 1***. Three scenarios were considered in the PC-SWMM model as follows:

1. For Option 1, the hydraulic characteristics of the ditches and culverts on public lands were determined based on the results of the topographic survey, Record Drawings, where available, and field observations. Cross-Section drawing provides a comparison of the cross sections generated through a combination of GIS data with local topographic survey and the sections as modelled in the PC-SWMM simulation.
2. As per Option 1, but with the assumption that Culverts #16 and #17, under Beachwood Road, are 50% blocked; and
3. The same as Option 1 with an assumed cross section of adequate capacity (0.6 m deep V-ditches at most locations, and a 1m wide flat bottom ditch through one section) was utilized to establish the expected flow depths throughout each watershed and establish the capacity of the downstream infrastructure to accommodate a range of expected flows.

## 2.2 Existing Conditions Assessment

The purpose of the current study is two-fold: to evaluate any existing drainage issues within the drainage areas which could be impacted by the future development of the Public Works property and Joan Avenue; and to identify the drainage strategy to provide the requisite controls for the proposed development while determining an appropriate route for conveyance of surface flows from the site to the Nottawasaga Bay.

As identified in Section 2, there are two main drainage pathways which are investigated under this drainage study. In order to provide a summary of the existing issues along these routes and to provide a basis for comparison for post development scenarios, the following subsections provide a summary of the results at each location with a summary of the existing conditions or potential implications in the post development scenario. The results of the PC-SWMM model are included in Appendix C.

### 2.2.1 Outlet 1 – 8801 Beachwood Road (Conduit C16)

Under existing conditions, frequent ponding of this property and the low-lying area to the west occurs annually during the spring thaw in the area and in flash flood conditions during the mid-winter or spring thaw conditions water levels can increase to a point affecting the residential structures. The model results under Scenario 1 indicate that the reported ponding at 8801 Beachwood Road occur during events of the 10-year magnitude or greater. During the 100-year event, ponding in this area rises to approximately 184.72m, about 5cm above the finished floor elevation at this location. The results indicate that the ditch at this location is operating under surcharge conditions during the 2-year event. However, given that this scenario has been completed under the assumption that no flows leave the system, even in the event of ponding, the results indicate that if this issue can be resolved locally, then there is no indication that flooding would occur downstream, subject to confirmation of the flow pathway through the properties north of Beachwood Road.

Due to the overgrown conditions at Culvert #16 as observed during the field visits, Scenario 2 was developed to assess the potential impact of significant blockage of these culverts. For the purpose of this assessment, it was assumed that 50% of the total culvert area was blocked from the invert upwards. This change had no impact on the ponding results at 8801 Beachwood Road. The blockage of Culvert 17 as assumed in this scenario will result in lack of capacity at the driveway culvert on the ditch south of Beachwood Road upstream of the culvert and the results shows concerns along the east side of this ditch.

Scenario 3 demonstrates that if sufficient capacity can be achieved through the private properties to the north, then both Outlets can convey the expected flows at each location and also demonstrate the potential to accommodate additional flow for the return period events. It is suggested that while deriving design options for the proposed works that the Regional event be simulated to assess the performance of these outlets under extreme conditions before additional flow is directed to these areas, since the model results indicate surcharging conditions at each outlet.

### 2.2.2 Outlet 1 – Culvert #16 (Conduit C16) Outlet 2 0- Culvert #17 (Conduit C17)

These two culverts crossing Beachwood Road, operated and maintained by MTO, are key components for each outlet. Culvert #16 is a 1220mmx1220mm concrete box culvert which is located east of Robert Street South, and conveys drainage from a 15.8 ha area, which includes the subject property of the future location of the Works Depot. The drainage area to this structure was significantly reduced as a result of the realignment of Highway 26 in accordance with the "*New Construction Detailed Design Drawings"* for Highway 26 prepared by Delcan in March 2010. As a result the existing flows at this culvert are significantly below the capacity of the crossing. Culvert #17 is a 1220mmx910mm concrete box culvert which is located west of 75$^{th}$ Street conveys drainage from a 9.10 ha area, which includes the area where Joan Avenue will be constructed. Expected flows at each of these culverts, along with a comparison to their full capacity based on MTO Highway Drainage Design Standards are summarized in Table **7 and Table 8.**

**Table 7: MTO Crossing Culvert Descriptions and Key Water Levels**

| Culvert # | Shape | Size | Material | U/S Invert (m) | D/S Invert (m) | 10-yr HGL (m) | 100 yr HGL (m) | Edge of pavement (m) |
|---|---|---|---|---|---|---|---|---|
| 16 | Box | 1220mmx1220mm | Concrete | 182.47 | 182.24 | 182.64 | 182.75 | 185.12 |
| 17 | Box | 1220mmx910mm | Concrete | 181.84 | 181.80 | 182.08 | 182.27 | 183.44 |

**Table 8: Flows and Capacity for MTO Culverts**

| | Culvert # | | 2-yr | 5-yr | 10-yr | 25-yr | 50-yr | 100-yr | Regional |
|---|---|---|---|---|---|---|---|---|---|
| 24-hr-SCS | 16 | Flow ($m^3/s$) | 0.291 | 0.387 | 0.445 | 0.539 | 0.645 | 0.776 | 0.835 |
| | | % of Capacity | 5% | 7% | 8% | 10% | 12% | 14% | 15% |
| | 17 | Flow ($m^3/s$) | 0.202 | 0.267 | 0.312 | 0.399 | 0.501 | 0.665 | 0.566 |
| | | % of Capacity | 16% | 21% | 24% | 31% | 39% | 51% | 44% |