

**FUNCTIONAL SERVICING REPORT**

**WYMBOLWOOD BEACH HOUSES  
1 DUNSFORD LANE**

**TOWNSHIP OF TINY  
COUNTY OF SIMCOE**

**PREPARED FOR:**

**INTERRA LANDS INC.**

**PREPARED BY:**

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**DECEMBER 2025**

**CFCA FILE NO. 2251-6304**

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Revision Number	Date	Comments
Rev.1	December 2025	Issued for First Submission

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## 1.0 INTRODUCTION

C.F. Crozier & Associates inc. (Crozier) was retained by Interra Lands Incorporated (Interra) to prepare a Functional Servicing Report in support of the Zoning Bylaw Amendment and proposed severance for the Subject Lands located at 1 Dunsford Lane in the Township of Tiny. The property is legally described as Lot 19, Concession 17, Township of Tiny, Simcoe County. The triangular property is located on the east shore of Nottawasaga Bay, north of 7<sup>th</sup> Concession. The property is bounded by the shoreline and two local access roads. Refer to Figure 3 for the Site Location.

The following documents were reviewed as part of this engineering study:

- Preliminary Geotechnical Investigation Wymbolwood Beach Resort (SPL Consultants Ltd., April 2015)
- Natural Hazards Assessment Letter (Shoreplan Engineering Ltd., December 2025)

This report has been prepared to document details associated with the functional servicing and stormwater management design for the proposed development. Contained in this report is a description of the existing site (Section 2.0); the shoreline hazard assessment (Section 3.0), the sanitary servicing strategy (Section 4.0); the water servicing strategy (Section 5.0); the drainage & stormwater management strategy (Section 6.0); road design and driveway access (Section 7.0); and a concluding discussion (Section 8.0).

## 2.0 SITE DESCRIPTION

The property currently has 3 single-storey framed buildings used as a resort. Along the shoreline boundary of the site, the structures are fronted by a low stone wall that blends into the dunes near the edges of the site. The current water's edge of Nottawasaga Bay is approximately 20m from the 178m contour line.

The site is currently serviced by a private well and an onsite sewage system. The locations of any existing wells or sewage systems are unknown and there is no record of the size or function of the sewage system. There are no municipal water and wastewater systems available in the area. It has been assumed that the existing well will be located and decommissioned in accordance with O.Reg. 903.

According to the architectural plans prepared by Ian Maclaren Architect Inc. (November 20, 2025), the proposed development will include two residential lots with municipal road access to each unit, individual private onsite sewage systems and individual private drilled drinking water wells. Lot 1 refers to the north lot with the North Cottage and Lot 2 refers to the south lot with the South Cottage. Please refer to Figures 1 & 2 that show the north and south cottages respectively. The Severance Sketch prepared by Innovative Planning Solutions (IPS) describes the lands to be severed and the lands to be retained. Please refer to Figure 3 in the Figures Appendix.

The site is accessed on both sides from public road allowances with gravel driveways. The beachfront property features highly permeable sand and currently, there are no existing stormwater management controls on the site.

### 3.0 SHORELINE HAZARD ASSESSMENT

A Natural Hazards Assessment prepared in December 2025 by Shoreplan Engineering Limited (Shoreplan) documents the existing natural hazards and their implications on the proposed development. The natural hazards considered for the assessment include the erosion hazard, the flood hazard, and the dynamic beach hazard as defined in the Natural Hazards Policies (3.1) of the Provincial Policy Statement (PPS).

The location of the proposed buildings, wells, and septic units are to be located outside the development setback line. Please refer to the Natural Hazards Assessment prepared by Shoreplan in Appendix C for full details and recommendations.

### 4.0 SANITARY SERVICING – PRIVATE ONSITE SEWAGE SYSTEMS

Based on a review of the Township's available mapping there is no municipal sanitary infrastructure available in the area of the Site. Therefore, it is proposed that the North and South Cottages will each be serviced by an individual private onsite sewage system. The onsite sewage system for each lot will be designed and installed in accordance with the Ontario Building Code (OBC), Township of Tiny, and County of Simcoe guidelines.

#### 4.1 Sewage System Design Flows

Peak daily sewage flows for each lot were calculated using Table 8.2.1.3.A of the OBC for residential occupancies.

##### 4.1.1 North Cottage

The North Cottage consists of a 4-bedroom dwelling and has an approximate finished floor area of 242 m<sup>2</sup>. The North Cottage will have 3-bathroom groups and additional fixtures for a total of 35.5 fixture units. The total daily design sewage flow is shown in Table 1. Detailed calculations are provided in Appendix A.

**Table 1: Total Maximum Day Sewage Flow Calculation for the North Cottage**

Description	Number of Units	Additional Flow per Unit (L)	Total Daily Design Flow (L/day)
Base Flow			2,000
Additional Flow			
i) Each bedroom over 5, or	0	500	0
ii) Area over 200 m <sup>2</sup> .			
A) Each 10 m <sup>2</sup> over 200 m <sup>2</sup> to 400 m <sup>2</sup>	5	100	500
B) Each 10 m <sup>2</sup> over 400 m <sup>2</sup> to 600 m <sup>2</sup>	0	75	0
C) Each 10 m <sup>2</sup> over 600 m <sup>2</sup>	0	50	0
iii) Fixture Units over 20	15.5	50	775
Additional Flow (greatest of i, ii, iii)			775
<b>Total Daily Design Flow:</b>			<b>2,775</b>

Therefore, the total maximum day sanitary sewage flow for the North Cottage is 2,775 L/day.

#### 4.1.2 South Cottage

The South Cottage consists of a 9-bedroom dwelling and has an approximate finished floor area of 403 m<sup>2</sup>. The South Cottage will have 8-bathroom groups and additional fixtures for a maximum of 70 fixture units. The total daily design sewage flow is shown in Table 2. Detailed calculations are provided in Appendix A.

**Table 2: Total Maximum Day Sewage Flow Calculation for the South Cottage**

Description	Number of Units	Additional Flow per Unit (L)	Total Daily Design Flow (L/day)
Base Flow			2,500
Additional Flow			
i) Each bedroom over 5	4	500	2,000
ii) Area over 200 m <sup>2</sup>			
A) Each 10 m <sup>2</sup> over 200 m <sup>2</sup> to 400 m <sup>2</sup>	20	100	2,000
B) Each 10 m <sup>2</sup> over 400 m <sup>2</sup> to 600 m <sup>2</sup>	1	75	75
C) Each 10 m <sup>2</sup> over 600 m <sup>2</sup>	0	50	0
iii) Fixture Units over 20	50	50	2,500
Additional Flow (greatest of i, ii, iii)			2,500
<b>Total Daily Design Flow:</b>			<b>5,000</b>

Therefore, the total maximum day sanitary sewage flow for the South Cottage is 5,000 L/day.

Properties with a total daily design sanitary sewage flow not exceeding 10,000 L/day are regulated by the Ontario Building Code. As shown above, the total daily design sanitary sewage flows for each property are less than 10,000 L/day. Therefore, a building permit will be required for each onsite sewage system prior to construction.

#### 4.2 Soils Evaluation

SPL Consultants completed a test pitting program at the site on February 26, 2015. Based on the reported surficial geology of the area (OGS) and the field observations by SPL Consultants, the soils of the site were identified as sand with trace amounts of gravel deposits. Three (3) test pits were completed; refer to the Geotechnical Report prepared by SPL Consultants for the locations of TP15-01, TP15-02, and TP15-03 (provided in Appendix C).

The geotechnical report indicates that the following surficial geological materials were present in the test pits:

- The soils encountered in TP15-01 consisted of approximately 1.9 m of sand fill, underlain by approximately 0.3 m of fine light brown native sand. The test pit was excavated to a termination depth of 2.2 meters below ground surface (mbgs). No signs of groundwater were observed in TP15-01 and the test pit walls were dry and stable.
- TP15-02 consisted of approximately 0.1 m of topsoil, underlain by approximately 2.1 m of fine to medium grain light brown sand. The test pit was excavated to a termination depth of 2.2 mbgs. Groundwater was observed in TP15-02 at 1.0 mbgs and the sand was observed to be saturated at 1.0 mbgs. It is important to note that TP15-02 is located close to the 178 m contour line and higher groundwater elevations are expected near the shoreline.
- TP15-03 consisted of approximately 50 mm of topsoil, underlain by approximately 100 mm of sand fill with trace amounts of gravel, 400 mm of fine to medium grey sand, and 700 mm of compact grey sand. The test pit was excavated to a termination depth of 2.4 mbgs.

Groundwater was not found but a wet cave was observed from the surface of the termination depth at 2.4 mbgs.

Based on the results of the grain size analysis provided in the SPL report, we have classified the soil as an SP soil, which has a T-time range of 2 – 8 min/cm, as described in Supplementary Standard SB-6 of the OBC. A percolation time (T-time) of 6 min/cm has been assigned for design purposes.

### **4.3 Proposed Sanitary Servicing Strategy**

The Class 4 Sewage System proposed for each lot comprises of a Waterloo Biofilter Level IV treatment unit meeting the CAN/BNQ 3680-600 standard discharging to a leaching bed constructed as a filter bed for treatment and dispersal of the treated effluent. Each proposed filter bed will include a sand and stone layer sized in accordance with OBC Sentence 8.7. 5. Refer to Figure 2 for the Sanitary Servicing Plan and layout of the proposed onsite sewage systems.

#### **4.3.1 North Cottage**

The proposed treatment unit for the North Cottage is a Waterloo Biofilter basket tank configuration, Model AD-BA30, with a treatment capacity of 3,000 L/d. Sewage will flow from the North Cottage by gravity to the inlet of a 9,000 L Waterloo Biofilter anaerobic digester tank (ADIPC-9000) equipped with an innertube on the inlet and an internal pump chamber to dose the biofilter basket tank.

A 7,480 L Basket Biofilter Tank Model BT-7480 equipped with (2) basket filters filled with Biofilter medium is proposed. The effluent is distributed evenly over the surface of the biofilter medium, and effluent is treated as it infiltrates through the foam. All pumps and controls will be designed and supplied by Waterloo Biofilter. Treated effluent from the BT-7480 is pumped to the filter bed.

The filter bed has been designed to meet the total daily design flow of 2,775 L/d and will be located southeast of the proposed North Cottage. According to OBC 8.7.5.2.(3), the loading rate on the effective (stone) area is 100 L/m<sup>2</sup>/day for filter beds with Level IV Treatment. Considering OBC 8.7.5.2 (5), the minimum filter bed area at the surface must be 28 m<sup>2</sup>. An area of 30.0 m<sup>2</sup> is proposed.

The filter bed will be equipped with three (3) runs of 75 mm diameter perforated pipe, each pipe will be 6.0 m long, spaced 1.0 m apart, and will be constructed within the stone layer.

The stone area will be underlain with filter sand meeting the grading requirements of OBC 8.7.5.3 (3) and will be a minimum of 750 mm thick below the stone layer. With a percolation rate of T = 6 min/cm for the underlying native soil, a minimum contact area of 20 m<sup>2</sup> is required at the base of the filter bed in accordance with OBC 8.7.5.3 (6). This area will be increased to 30.0 m<sup>2</sup> to match the area of the stone.

#### **4.3.2 South Cottage**

The proposed treatment unit for the South Cottage is a Waterloo Biofilter basket tank configuration, Model AD-BA50, with a treatment capacity of 5,000 L/d. Sewage will flow from the South Cottage by gravity to the inlet of a 13,500 L Waterloo Biofilter anaerobic digester tank (ADIPC-13500) equipped with an innertube on the inlet and an internal pump chamber to dose the biofilter basket tank.

An 11,250 L Basket Biofilter Tank Model BT-11250 equipped with (2) basket filters filled with Biofilter medium is proposed. The effluent is distributed evenly over the surface of the biofilter medium, and effluent is treated as it infiltrates through the foam. All pumps and controls will be designed and supplied by Waterloo Biofilter. Treated effluent from the BT-11250 is pumped to the filter bed.

The filter bed has been designed to meet the total daily design flow of 5,000 L/d and will be located east of the proposed South Cottage. According to OBC 8.7.5.2.(3), the loading rate on the effective (stone) area is 100 L/m<sup>2</sup>/day for filter beds with Level IV Treatment. Considering OBC 8.7.5.2 (5), the minimum filter bed area at the surface must be 50 m<sup>2</sup>.

The filter bed will be equipped with seven (7) runs of 75 mm diameter perforated pipe, each pipe will be 6.0 m long and spaced 1.0 m apart and constructed within the stone layer.

The stone area will be underlain with filter sand meeting the grading requirements of OBC 8.7.5.3 (3) and will be a minimum of 750 mm thick below the stone layer. With a percolation rate of  $T = 6$  min/cm for the underlying native soil, a minimum contact area of 35 m<sup>2</sup> is required at the base of the filter bed in accordance with OBC 8.7.5.3 (6). An area of 50.0 m<sup>2</sup> will be provided to match the area of the stone.

## **5.0 WATER SERVICING – PRIVATE WELL**

Based on a review of the Township's available mapping, there is no municipal sanitary infrastructure available in the area of the Site. Therefore, potable drinking water will be provided to Lots 1 and 2 with a new individual drilled water well. The locations of the wells are shown in Figure 1 and each well shall be located to adhere to the minimum setback requirements as described in Ontario Regulation 903 and in the Ontario Building Code. Please note that a well with a watertight casing 6.0 m below grade will be required in order to achieve a 15 m setback to the onsite sewage system. It is assumed that there is a suitable groundwater regime to support the lots, which should be confirmed prior to construction. The new wells must be drilled by a licensed well contractor in accordance with O. Reg. 903.

## **6.0 STORMWATER MANAGEMENT**

Based on the site conditions and the nature of the proposed development, stormwater management design criteria are summarized below:

- Based on the existing sandy soils and the associated high infiltration rate, the proposed redevelopment is an ideal location to implement Low Impact Development practices to reduce peak runoff as well as provide quality control and water balance benefit.
- Quality control will be provided to an 'enhanced' level as defined by MOE's Stormwater Management Planning and Design Manual.
- Low Impact Development (LID) practices are to be employed in an effort to improve site hydrology and meet water balance initiatives.

Detailed LID design will be assessed at the SPA submission.

### **6.1 Drainage**

Under existing conditions, stormwater from the buildings and access roads is directed toward Nottawasaga Bay via uncontrolled sheet flow. The existing drainage patterns will be maintained upon development. Road drainage will be directed toward Nottawasaga Bay via roadside ditches.

### **6.2 Stormwater Quantity Control**

Table 3 presents a comparison between the overall site runoff coefficient in existing and proposed conditions. As shown in Table 3, the site's runoff coefficient increases from 0.40 to 0.53 in the proposed condition. As a result, a minor increase in peak stormwater runoff is expected upon development.

**Table 3: Runoff Coefficient Comparison**

Surface	Existing Conditions		Proposed Conditions (No Mitigation)	
	Area (m <sup>2</sup> )	Runoff Coefficient	Area (m <sup>2</sup> )	Runoff Coefficient
North Roof, South Roof, Decks	488	0.90	815	0.90
Gravel Driveway	138	0.80	178	0.80
Landscape Area	1582	0.25	1215	0.25
<b>Total</b>	<b>2208</b>	<b>0.40</b>	<b>2208</b>	<b>0.53</b>

It is our opinion that the small increase in peak stormwater runoff generated by this development will not impact the downstream Nottawasaga Bay due to the proximity and size of the water body. In an effort to provide water quantity control, Low Impact Development (LID) practices are proposed. Soakaway pits have been preliminarily sized to infiltrate runoff from the roof of the north and south cottages as indicated in Figure 1. Based on the Geotechnical Report prepared by SPL Consultants, the soakaway pits have been designed with a maximum drawdown time of 48 hours, a design infiltration rate of 24mm/hr and a clearance of 1m from the groundwater table. The proposed location of the soakaway pits meet the required setbacks from the wells, septic beds, property lines and buildings. The preliminary sizing of the soakaway pits is provided in Appendix B.

Road drainage will be collected within the roadside flat-bottomed infiltration swales and discharged downstream of the hammerhead via culverts. The swales have been designed with a longitudinal slope of 0.5% to promote infiltration and quality treatment.

## 7.0 ROAD DESIGN & DRIVEWAY ACCESS

Access to the site will be provided exclusively via Dunsford Lane, which will extend approximately 92 m east from Tiny Beaches Road South. Street A will not provide access to individual lots but will remain part of the overall site layout for connectivity and servicing purposes. Both Dunsford Lane and Street A will have the following parameters:

- 7.0 m road width;
- 1.2 m shoulders;
- Cross-fall of 2%;
- Flat-bottomed infiltration swales;
- Hammerhead turn-around along Dunsford Lane;
- Hammerhead turn-around along Street A (Subject to change pending Town comments).

Our preliminary road design and grading provides positive drainage toward Nottawasaga Bay. The road will be superelevated with a 2% cross-fall toward Georgian Bay to promote runoff and reduce ice buildup. The 7.0 m width exceeds the 6.0 m minimum required for emergency vehicle access under the Ontario Building Code. Detailed grading will be completed at the detailed design stage.

The proposed development will provide individual access to each lot via Dunsford Lane, which will remain a gravel road as noted acceptable by the Town. Each driveway will also be constructed with gravel within the property line and will include a minimum of two parking spaces.

## 7.1 Waste Management Considerations (Simcoe County)

Based on discussions with the County of Simcoe, curbside waste collection will be provided for all residential lots with frontage on publicly owned and maintained roads. Access to individual lots will be exclusively via Dunsford Lane, supporting curbside collection where feasible.

Should collection along Dunsford Lane not be practical due to operational constraints, a designated common collection point will be established at the intersection of Dunsford Lane and Tiny Beaches Road South. This approach ensures efficient waste management for the development while aligning with County guidance.

## **7.2 Winter Maintenance Considerations (Township of Tiny)**

The turnaround area at the end of Dunsford Lane has been designed to enhance winter maintenance functionality, with its length maximized within boundary limits to provide snow storage while reasonably minimizing impacts on manoeuvring space, development area, and the adjacent driveway.

Shifting the access further south was reviewed to improve offset and snow storage; however, the driveway location shown on the current plans has limited flexibility due to septic system sizing requirements. These measures, combined with the hammerhead turnaround and 7.0 m road width, aim to provide adequate space for snowplow manoeuvring and snow storage on a best-efforts basis. Please refer to Figure T300 in the Figures Appendix for the snow plow turning analysis.

Additional refinements may occur during detailed design.

## **8.0 CONCLUSIONS & RECOMMENDATIONS**

We conclude that the proposed development of the subject lands can be readily serviced and meet the objectives of the regulatory agencies.

1. A Natural Hazards Assessment prepared by Shoreplan Engineering Ltd. verified that the proposed location of the units does not encroach on any hazard limits.
2. On-site soils are primarily classified as sand with trace amounts of gravel deposits. Groundwater was observed to be 1.0 m to 2.2 m below ground surface (SPL Consultants, February 26, 2015). Additional soils and groundwater information is provided in the Geotechnical Investigation Report (SPL Consultants, April 2015).
3. Private onsite sewage systems will be provided for each lot. A Waterloo Biofilter treatment system with a filter bed is proposed for each cottage. The treatment units and filter bed have been located on each lot such that they meet or exceed the minimum distances from structures, property lines, and proposed wells.
4. A new onsite sewage system, rated for 2,775 L/day is proposed for the North Cottage. Effluent from the advanced treatment unit will be pumped to the proposed filter bed with a total surface area of 30.0 m<sup>2</sup>. The stone area will be equipped with three (3) runs of 75 mm diameter, 6.0 m long effluent distribution pipe, and spaced 1.0 m apart.
5. A new onsite sewage system, rated for 5,000 L/day is proposed for the South Cottage. Effluent from the advanced treatment unit will be pumped to the proposed filter bed with a total surface area of 50.0 m<sup>2</sup>. The stone area will be equipped with seven (7) runs of 75 mm diameter, 6.0 m long effluent distribution pipe, and spaced 1.0 m apart.
6. The north and south lots will each be serviced by an individual drilled well with a watertight casing to at least 6.0 m below grade.

7. Individual access to each lot has been provided along with a minimum of 2 parking spaces per lot.
8. Stormwater management controls will be provided via the proposed LID measures, including soakaway pits and roadside swales. Based on the native soil conditions, the proposed concept grading plan, and the small increase in peak flows expected from this development, it is anticipated that no detrimental downstream impacts will occur.
9. Dunsford Lane is proposed to be superelevated with a 2% cross-fall toward Georgian Bay. Dunsford Lane is to remain a gravel road, with curbside waste collection where feasible. The hammerhead turnaround and 7.0m road width provide adequate space for snow storage and snowplow maneuvering.

Therefore, we recommend approval of the Zoning Bylaw Amendment application and severance submission for the subject lands from the perspective of engineering servicing requirements.

Respectfully submitted,

**C.F. CROZIER & ASSOCIATES INC.**



Nicole O'Connor, P.Eng.  
Project Engineer

**C.F. CROZIER & ASSOCIATES INC.**



Raphael de Mesa, EIT  
Engineering Intern

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# APPENDIX A

## Sanitary Design Flow Calculations



## ONSITE SEWAGE SYSTEM RESIDENTIAL CALCULATION SHEET

**Project Name:** 1-Dunsford Lane  
**Project Number:** 2250-6304

**Date:** 2022.01.12  
**Designed By:** A.L,R.D.M.  
**Checked By:** K.R

#### input required

House Details: 4 bedroom  
 242 m<sup>2</sup>

References

Description	Number of Units	Additional Flow per Unit (L)	Total Flow (L/day)
Base Flow			2000
<b>Additional Flow</b>			
i) Each bedroom over 5	0	500	0
ii) Area over 200m <sup>2</sup>			
A) Each 10m <sup>2</sup> over 200m <sup>2</sup> to 400m <sup>2</sup>	5	100	500
B) Each 10m <sup>2</sup> over 400m <sup>2</sup> to 600m <sup>2</sup>	0	75	0
C) Each 10m <sup>2</sup> over 600m <sup>2</sup>	0	50	0
<b>Total Additional Sewage Flow from Area</b>			<b>500</b>
iii) Fixture Units over 20	16	50	775
<b>Addition flow (greatest of i,ii,iii)</b>			<b>775</b>
<b>Total Daily Design Sanitary Sewage Flow (L/day):</b>			<b>2775</b>

### Pre-Treatment Options

Required septic tank size = 5550 L minimum

Propose Level IV Treatment (Y/N): Y

Native Percolation time, T = 6 min/cm

Imported Sand Percolation time = 10 min/cm

### Option #1 - Filter Bed

Minimum required contact area = 28 m<sup>2</sup> 7.0m x 3.7m and 7.0m x 4.9m

Required extended contact area = 20 m<sup>2</sup>

Maximum loading rate = 10 L/m<sup>2</sup>/day

Minimum loading area = 278 m<sup>2</sup>

30 m<sup>2</sup> provided  
 3 runs of 6m at 1.0m O.C.



**ONSITE SEWAGE SYSTEM RESIDENTIAL CALCULATION SHEET**

Project Name: 1-Dunsford Lane  
 Project Number: 2251-6304

Date: 2022.01.12  
 Designed By: A.L.R.D.M.  
 Checked By: K.R

#### input required

Fixtures	Number of Fixtures			Fixture Units per Fixture	Total Fixture Units
	basement	1st floor	2nd floor		
Bathroom Group (flush tank)	1		2	6	18.0
2 Piece Bathroom		1		5.5	5.5
Basement Rough-in				6	0.0
Sinks (Domestic Lavatory w. 1/2" trap, kitchen sink)		4		1.5	6.0
Clothes Washer	1			1.5	1.5
Dishwasher (if not connected to kitchen sink)				1	0.0
Shower (from 1 head)				1.5	0.0
Floor drain	1			3	3.0
Laundry Tub	1			1.5	1.5
			<b>Total Fixture Units</b>		<b>35.5</b>



## ONSITE SEWAGE SYSTEM RESIDENTIAL CALCULATION SHEET

**Project Name:** 1-Dunsford Lane  
**Project Number:** 2250-6304

**Date:** 2022.01.12  
**Designed By:** A.L.R.D.M.  
**Checked By:** K.R

#### input required

House Details: 9 bedroom  
 403.36 m<sup>2</sup>

References

Description	Number of Units	Additional Flow per Unit (L)	Total Flow (L/day)
Base Flow			2500
<b>Additional Flow</b>			
i) Each bedroom over 5	4	500	2000
<b>ii) Area over 200m<sup>2</sup></b>			
A) Each 10m <sup>2</sup> over 200m <sup>2</sup> to 400m <sup>2</sup>	20	100	2000
B) Each 10m <sup>2</sup> over 400m <sup>2</sup> to 600m <sup>2</sup>	1	75	75
C) Each 10m <sup>2</sup> over 600m <sup>2</sup>	0	50	0
<b>Total Additional Sewage Flow from Area</b>			<b>2075</b>
iii) Fixture Units over 20	50	50	2500
<b>Addition flow (greatest of i,ii,iii)</b>			<b>2500</b>
<b>Total Daily Design Sanitary Sewage Flow (L/day):</b>			<b>5000</b>

**Pre-Treatment Options**

Required septic tank size = 10000 L minimum

Propose Level IV Treatment (Y/N): Y

Native Percolation time, T = 6 min/cm

Imported Sand Percolation time = 10 min/cm

**Option #3 - Filter Bed**

Minimum required contact area = 50.00 m<sup>2</sup>      7.04m x 7.1m      50.0 m<sup>2</sup>

Required extended contact area = 35 m<sup>2</sup>

Maximum loading rate = 10 L/m<sup>2</sup>/day

Minimum loading area = 500 m<sup>2</sup>



### ONSITE SEWAGE SYSTEM RESIDENTIAL CALCULATION SHEET

Project Name: 1-Dunsford Lane  
Project Number: 2251-6304

Date: 2022.01.12  
Designed By: A.L.R.D.M.  
Checked By: K.R

### input required

Fixtures	Number of Fixtures	Fixture Units per Fixture	Total Fixture Units
Bathroom Group	8	6.0	48.0
2 Piece Bathroom	2	5.5	11.0
Basement Rough-in	0	6.0	0.0
Extra Bathtub/Shower	1	1.5	1.5
Extra Bathroom Sink	2	1.5	3.0
Clothes Washer	1	1.5	1.5
Laundry Tub	1	1.5	2
Kitchen Sink	1	3.0	3
Dishwasher	0	1.0	0
		<b>Total Fixture Units</b>	<b>69.5</b>

# APPENDIX B

## Stormwater Management Calculations



### Stormwater Management Summary - Soakaway Pit Design

Storage Parameters	
Rooftop Area (m <sup>2</sup> ) <sup>(1)</sup>	315
Rainfall Depth (mm)	25
Runoff Coefficient	0.90
Required Storage Volume (m <sup>3</sup> )	7.0875
Void Ratio	0.4
Required Infiltration Trench Volume (m <sup>3</sup> )	17.72

Infiltration Parameters	
Native Soil Infiltration Rate (mm/hr) <sup>(2)</sup>	60
Safety Factor	2.5
Design Infiltration Rate (mm/hr)	24
Drawdown Time (hr)	48
Maximum Soakaway Depth (m)	1.15

Soakaway Pit Design - North Cottage	
Proposed Length of Soakaway Pit (m)	7.0
Proposed Width of Soakaway Pit (m)	3.0
Proposed Depth of Soakaway Pit (m) <sup>(3)</sup>	1.10
Proposed Soakaway Pit Volume (m <sup>3</sup> )	9.24

Soakaway Pit Design - South Cottage	
Proposed Length of Soakaway Pit (m)	5.1
Proposed Width of Soakaway Pit (m)	3.9
Proposed Depth of Soakaway Pit (m) <sup>(3)</sup>	1.1
Proposed Soakaway Pit Volume (m <sup>3</sup> )	8.7516

<b>Total Proposed Soakaway Pit Volume (m<sup>3</sup>)</b>	17.99
<b>Total Rainfall Depth Retained (mm)</b>	25

Notes:

(1) Half of roof from North and South Buildings proposed to drain to soakaway pits.

(2) Based on soils identified in Geotechnical Report by SPL Consultants

(3) A clearance of 1m between the groundwater table and the footprint of each LID will be maintained.

# APPENDIX C

## Background Reports

**REPORT ON  
PRELIMINARY GEOTECHNICAL INVESTIGATION  
WYMBOLWOOD BEACH RESORT  
1 DUNSFORD LANE, TOWNSHIP OF TINY, ONTARIO**

**Prepared For:**

**BILL ULICKI  
ROMSPEN INVESTMENT CORPORATION  
162 CUMBERLAND STREET, SUITE 399  
TORONTO, ONTARIO, M5R 3N5**

**SPL Project No.:10001448  
April 13, 2015**

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**APPENDIX A:** INDIVIDUAL TEST PIT LOGS

**APPENDIX B:** GRAIN SIZE ANALYSIS RESULTS

**APPENDIX C:** GENERAL REQUIREMENTS FOR ENGINEERED FILL

## 1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by Romspen Investment Corporation to undertake a geotechnical investigation for the proposed development of four residential buildings on the property currently identified as Wymbolwood Beach Resort, 1 Dunsford Lane, Township of Tiny, Ontario.

The municipal address of the site is 1 Dunsford Lane, Township of Tiny, Ontario. The site is located on the eastern shore of the Georgian Bay, as shown on **Drawing 1**.

It is our understanding that the existing hotel structures currently located on the site are to be demolished for the proposed development of four detached residential buildings, which will result in the sub-division of the current property and include the installation of separate septic and water supply systems for each of the four proposed building/properties.

The purpose of the geotechnical investigation was to characterize the subsurface soil and groundwater conditions, by means of test pits at three locations, and based on this information provide recommendations for the bearing capacity of the soils with regards to building foundations and roadway design.

The work plan included *in-situ* infiltration testing, using a double ring infiltrometer, to determine percolation rates for the soils but due to ground frost, and groundwater being encountered during the excavation of the test pits, this testing will be completed in the later portion of April 2015.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with the applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for Bill Ulicki at Romspen Investment Corporation. Third party use of this report without SPL consent is prohibited.

## 2. FIELD AND LABORATORY WORK

To characterize the type and depth of soil materials present at the site, three test pits (TP15-01 – TP15-03) were excavated on February 26, 2015. The test pits were excavated in accessible areas, the locations of the test pits are shown on **Drawing 1**.

Excavation of the test pits was completed by Charles Morden Construction using a CAT 420E IT front-end loader equipped with an excavator bucket. The test pits were excavated to depths ranging between 2.2 and 2.4 m below ground, and encountered soil materials consisting of topsoil, fill materials containing sand with trace gravel and rootlets, and sand extending beyond the final depth investigated. Groundwater was encountered in TP15-02 and TP15-03, at 1.0 m and 2.1 m below ground, respectively. Bedrock was not encountered in any of the test pits. A test pit log for each location is attached in **Appendix A**. Selected soil samples were collected from the test pits and delivered to SPL laboratory for testing of moisture content and grain size distribution analysis. The results of the detailed laboratory analysis are presented in **Appendix B**.

## 3. SITE AND SUBSURFACE CONDITIONS

### 3.1 Soil Conditions

The soils present at the site from the existing ground surface to the explored depth of the test pits predominantly consisted of fill over sand. More specifically, topsoil was encountered at the ground surface in TP15-02 and TP15-03 and ranged in thickness from 50 mm to 100 mm. Fill material was encountered in TP15-01 and TP15-03 and extended to depths of 0.4 m & 1.9 m below ground. The undisturbed native sand, underlying the fill (TP15-01, TP-03) and topsoil (TP-15-02) was observed to be in a compact to dense state.

During the excavation of the test pits, six representative grab samples were obtained at varying depths for moisture content calculations and grain size distribution via sieve analysis. The six tested samples of the soil material contained 0 to 6% gravel, 93 to 100% sand, and 0 to 1% silt and clay size soil particles. The grain size distribution curves for the samples are presented in **Appendix B**.

### 3.2 Groundwater Conditions

During the excavation and at the completion of the test pits, wet-cave was observed from surface in test pits TP15-02 and TP15-03. TP15-01 was open and dry upon completion. Observation of the groundwater conditions were made during the excavation of the test pits, and are summarized in **Table 1** below.

**Table 1.0 – Groundwater Observations in Test Pits**

TP No.	Date Excavated	Depth to Water (mbg* / elevation)
15-01	February 26, 2015	NA
15-02	February 26, 2015	1.0 / 177.6
15-03	February 26, 2015	2.1 / 178.1

\*mbg – Meters below grade

#### 4.0 DISCUSSION AND RECOMMENDATIONS

##### 4.1 General

Topsoil was encountered at ground surface in TP15-02 and TP15-03 and ranged in thickness from 50 mm to 100 mm. Sand was encountered underlying the topsoil to the final depth of the test pits, up to 2.4 mbg. Manual probing of the sand indicated that the material is in a compact state.

Groundwater was encountered in two of the test pits (TP15-02 @ 1.0 mbg & TP15-03 @ 2.1 mbg). It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

##### 4.2 Foundations

The footprint of the four proposed buildings will range between 185.76 m<sup>2</sup> and 198.67 m<sup>2</sup>. The native compact sand, at a minimum depth of 0.9 m below the original ground surface is considered capable of safely supporting the proposed building foundations with a design allowable bearing pressure of up to 75 kPa. Variations in the soil conditions are expected between the test pit locations. The bearing pressures should be confirmed by a qualified inspector from SPL Consultants prior to the placement of concrete.

##### 4.3 Other Comments on Foundations

Groundwater control will be required for any excavation below the water table, which was identified as high as 1.0 mbg during the test pits.

All footings exposed to seasonal freezing conditions should be provided at least 1.5 m of earth cover or equivalent thermal insulation against frost.

It should be noted that the recommended bearing capacities have been calculated by SPL from the test pit information for the design stage only. The investigation and comments are on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between test pits when foundation construction is underway. The

interpretation between test pits and the recommendations of this report must therefore be checked through field inspections provided by SPL to validate the information for use during the construction stage.

Depending on the depth of fill generated as a result of demolition, consideration may be given to the replacement of unsuitable existing material with an engineered fill where required. Detailed requirements for the construction of an approved engineered fill are provided in Appendix C.

Once the site has been adequately dewatered to a depth that will allow the removal of any unsuitable material the sub-excavation operation may begin. The type of dewatering system would depend on the ultimately exposed depth of unsuitable material. A vacuum well point system would effectively dewater the site if operated continuously until the engineered fill is completed. However, depending upon the depth of fill and season of construction, it may be possible to install a system of interceptor ditches and sump pumps at each corner of the required engineered fill pad to control seepage. Regardless, a dewatering specialist should be consulted.

In this regard, once the structurally sensitive areas of the structures have been adequately stripped of any organic material, disturbed soil and/or fill and the site dewatered the construction of the engineered fill pad would begin. The exposed subgrade must then be inspected by personnel from SPL Consultants Limited in order to assess the stability and compacted if necessary. Due to the sandy nature of the underlying soil, it is recommended that the stripping for and construction of engineered fill proceed during the dry summer months of the year in order to minimize subgrade instability.

Following subgrade inspection, the area may be raised to the floor slab subbase level with a well-graded, granular material with a maximum 8% silt content. In this regard, some on-site material may be assessed for reuse as engineered fill. Otherwise an imported Granular B type material or equivalent is recommended. All fill must be compacted in lifts not exceeding 20 cm in thickness, to 100% of the Standard Proctor Dry Density. The moisture content of the fill material placed should be within 2% of the Optimum Moisture Content in order to achieve optimum compactive effort.

Finally, the engineered fill must extend at least one meter beyond the proposed building envelope and slope down to the surrounding subexcavated level at 45°.

In order to ensure the above criteria are satisfied, the removal of the any organically included or disturbed soil as well as the placement of engineered fill must be supervised on a continuous basis by a qualified soil technologist from our office. In addition, all footings founded on engineered fill and the top of the poured concrete foundation walls must be reinforced with, at minimum, two 15 M continuous reinforcing bars in order to minimize the effects of variations in the degree of compaction of the engineered fill. Finally, for the purpose of frost protection, all exterior footings and footings exposed to frost action should be covered by at least 1.5 m of soil.

Building footings founded on an engineered fill constructed as described briefly above and detailed in Appendix C may be designed using an allowable design bearing pressure of 75 kPa (SLS).

#### 4.4 Pavement Design

The subgrade is expected to consist of native soils or clean earth fill materials. Approved earth fill materials encountered on the site may be utilized for subgrade preparation provided they are environmentally acceptable and do not contain excessive amounts of organics and deleterious materials, as well as their in-situ moisture content is within 2 percent of the optimum moisture content. The pavement subgrade should be proof-rolled; and any loose, soft, wet or unstable areas should be sub-excavated, and backfilled with clean earth fill placed in 150 mm thick lifts and compacted to a minimum of 98 percent SPMDD. Local sub-excavation may be required in areas where incompetent (loose/firm) subgrade conditions and significant organic/topsoil inclusions are encountered. Removal of all fill materials is not necessary, provided the subgrade is proof-rolled to detect soft areas. The entire pavement subgrade should be compacted to a minimum of 98 percent SPMDD prior to the granular sub-base placement.

Based on the above and assuming that traffic usage will be Urban Local Residential, Urban Local Divided Road or Urban Collector, the following minimum pavement thickness is recommended.

Table 2: Recommended Minimum Pavement Structure Thickness

Pavement Layer	Compaction Requirements	Minimum Thickness (Urban Local Residential, Urban Local Divided Road or Urban Collector)
Asphaltic Concrete	Minimum 97% BRD	50 mm OPSS HL 3 70 mm OPSS HL 8
OPSS Granular A Base	100% SPMDD**	150 mm
OPSS Granular B Sub-base	100% SPMDD	300 mm

\* Use high stability mix, 100 % virgin crushed aggregates for heavy duty parking areas

\*\* Denotes Standard Proctor Maximum Dry Density, ASTM-D698

The subgrade must be compacted to 98% SPMDD for at least the upper 300 mm unless accepted by SPL

The long term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved.

The site subgrade and weather conditions (i.e. if wet) at the time of construction may necessitate the placement of thicker granular sub-base layer in order to facilitate the construction. Furthermore, heavy

construction equipment may have to be kept off the newly constructed roads before the placement of asphalt and/or immediately thereafter, to avoid damaging the weak subgrade by heavy truck traffic.

The granular materials should be placed in lifts 150 mm thick or less and be compacted as referenced in Table 2. Asphalt materials should be rolled and compacted as per OPSS 310.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

**SPL CONSULTANTS LIMITED**



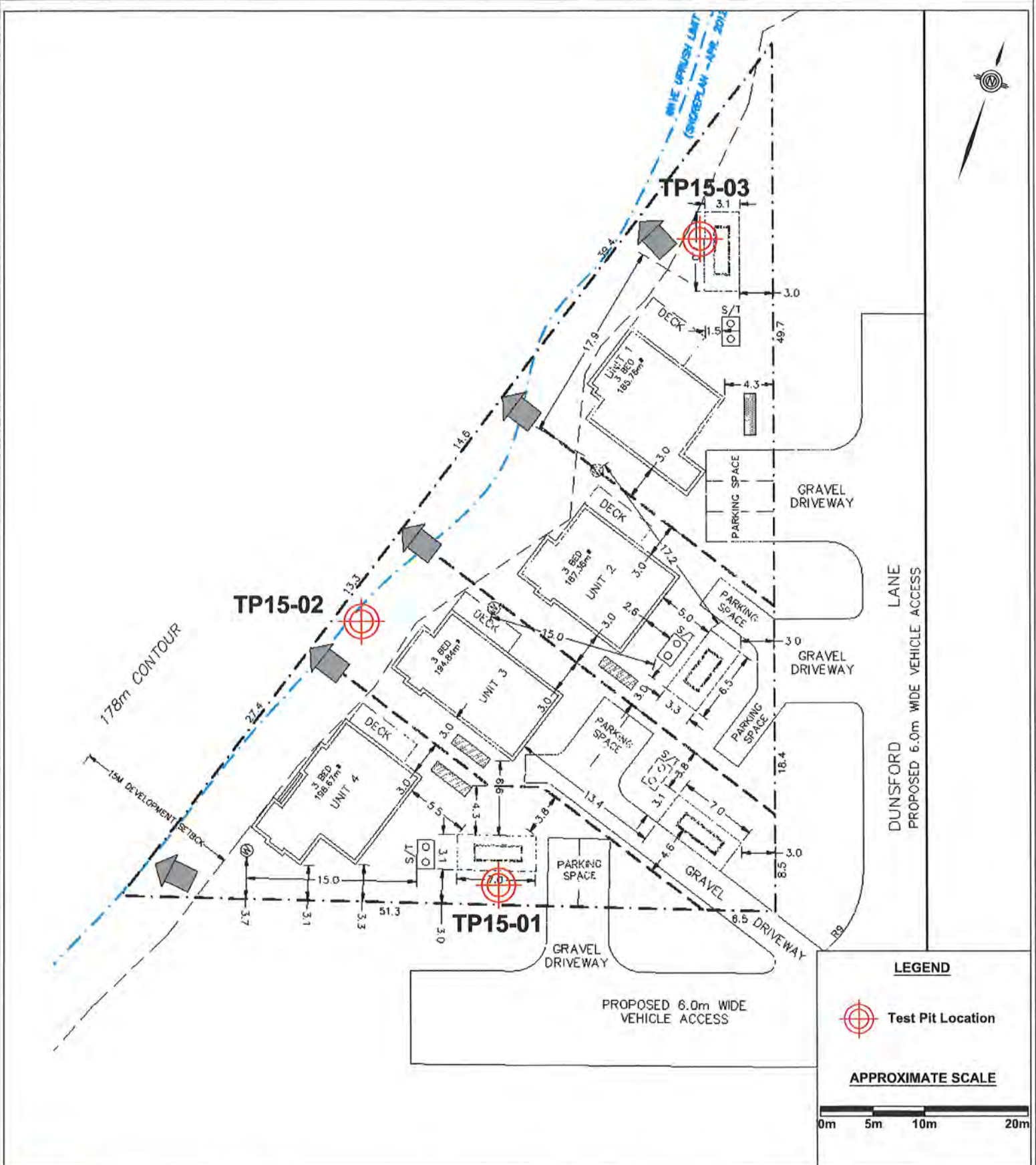
Gord Jarvis  
Project Manager

**REVIEWED BY**

Kent Malcolm, P.Eng.  
Senior Geotechnical Engineer

# Drawings

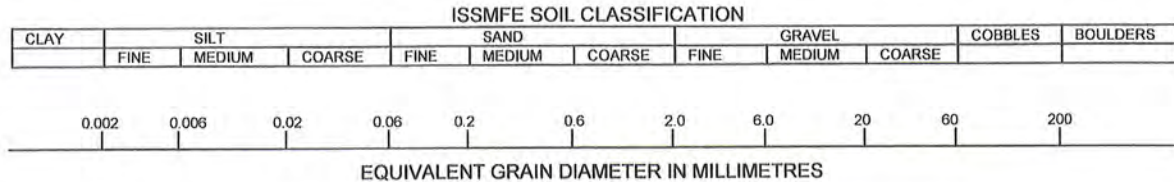




Client:	<b>ROMSPEN INVESTMENT CORPORATION</b>		Project No.:	<b>10001448</b>	Drawing No.:	<b>1</b>
Drawn:	<b>MV</b>	Approved:	<b>GJ</b>	Title: <b>TEST PIT LOCATION PLAN</b>		
Date:	<b>MARCH, 2015</b>	Scale:	<b>As Shown</b>	Project: <b>PRELIMINARY GEOTECHNICAL INVESTIGATION 1 DUNSFORD LANE, TOWNSHIP OF TINY, ONTARIO</b>		
Original Size:	<b>Letter</b>	Rev:	<b>N/A</b>			

## Drawing 1A: Notes On Sample Descriptions

- All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by SPL also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)					

UNIFIED SOIL CLASSIFICATION

- Fill:** Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.
- Till:** The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

# Appendix A

## Test Pit Logs



PROJECT: Preliminary Geotechnical Investigation  
 CLIENT: Romspen Investment Corporation  
 PROJECT LOCATION: 1 Dunsford Lane, Township of Tiny, Ontario  
 DATUM: Geodetic  
 BH LOCATION:

**DRILLING DATA**  
 Method: Excavator  
 Diameter:  
 Date: Feb/26/2015  
 REF. NO.: 10001448  
 ENCL NO.: 1

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					POCKET PEN. (C <sub>u</sub> ) (kPa)	NATURAL UNIT WT (Mg/m <sup>3</sup> )	REMARKS AND GRAIN SIZE DISTRIBUTION (%)	
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" BLOWS 0.3 m	SHEAR STRENGTH (kPa)							WATER CONTENT (%)
						20	40	60	80	100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
						○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE					w <sub>p</sub>	w	w <sub>l</sub>		
						50	100	150	200	250	10    20    30				GR SA SI CL
181.4 0.0	FILL: sand, fine, trace gravel, trace rootlets, brown, frozen at surface														
180.9 0.5	depth of frost														
179.5 1.9	SAND: sand, fine, layered, light brown, moist		1	GRAB											0 99 1 0
179.2 2.2	END OF TEST PIT: Notes: - test pit was open and dry upon completion - concrete mass encountered from surface to depth of 1.9m at south wall of test pit														

SPL SOIL TEST PIT - 10001448 - TEST PIT LOGS.GPJ SPL\_GDT 4/8/15

GRAPH NOTES: +, x, 3, 3. Numbers refer to Sensitivity      ○ = 3% Strain at Failure





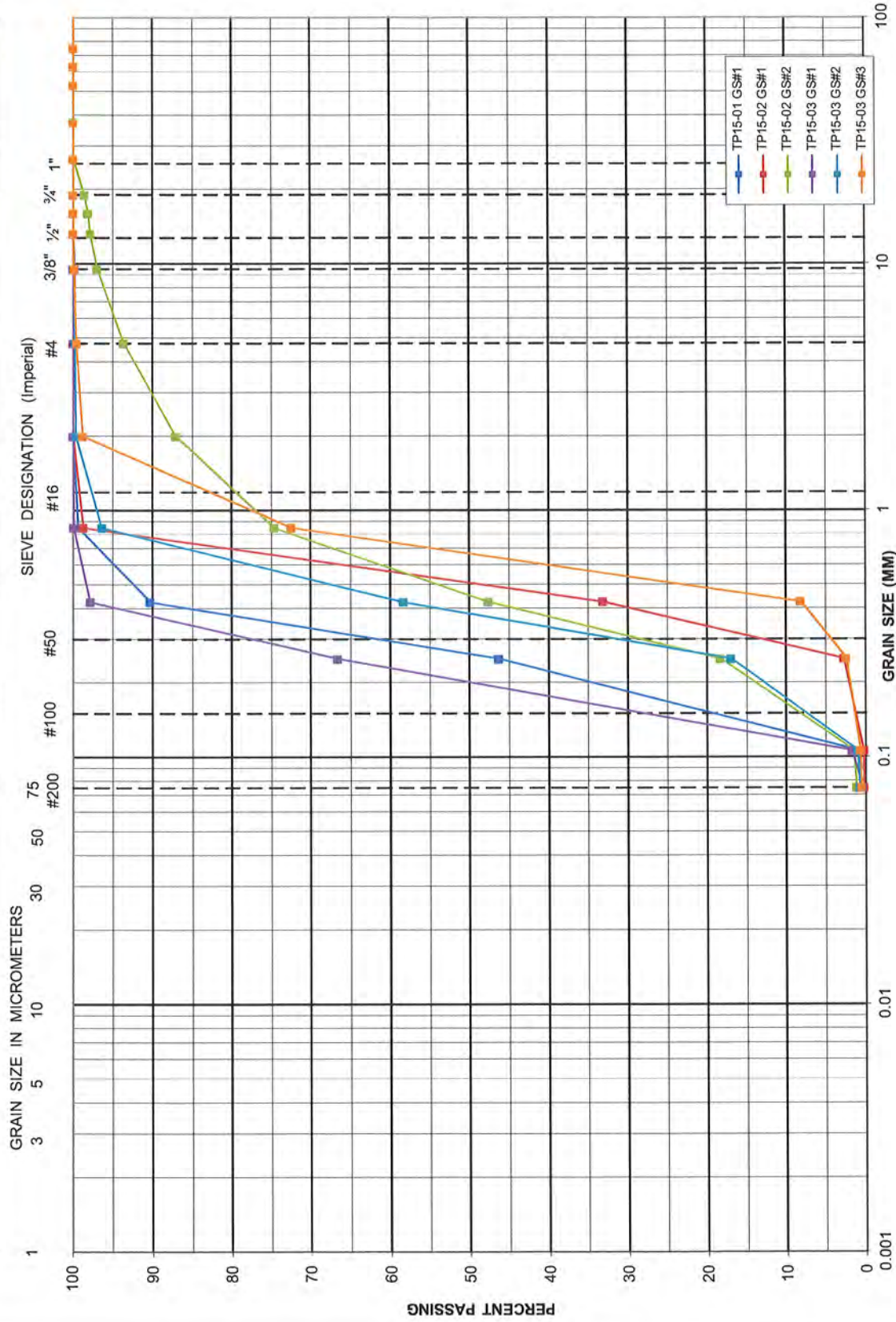
## **Appendix B**

### **Grain Size Analysis Results**



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



APPENDIX B  
 Project No. 10001448  
 Date: March 03, 2015

GRAIN SIZE DISTRIBUTION 10001448  
 Preliminary Geotechnical Investigation

**SPL Consultants Limited**  
 Geotechnical Environmental Materials Hydrogeology

## **Appendix C**

### **General Requirements For Engineered Fill**



## GENERAL REQUIREMENTS FOR ENGINEERED FILL

Compacted imported soil that meets specific engineering requirements and is free of organics and debris and that has been continually monitored on a full-time basis by a qualified geotechnical representative is classified as engineered fill. Engineered fill that meets these requirements and is bearing on suitable native subsoil can be used for the support of foundations.

Imported soil used as engineered fill can be removed from other portions of a site or can be brought in from other sites. In general, most of Ontario soils are too wet to achieve the 100% Standard Proctor Maximum Dry Density (SPMDD) and will require drying and careful site management if they are to be considered for engineered fill. Imported non-cohesive granular soil is preferred for all engineered fill. For engineered fill, we recommend use of OPSS Granular 'B' sand and gravel fill material.

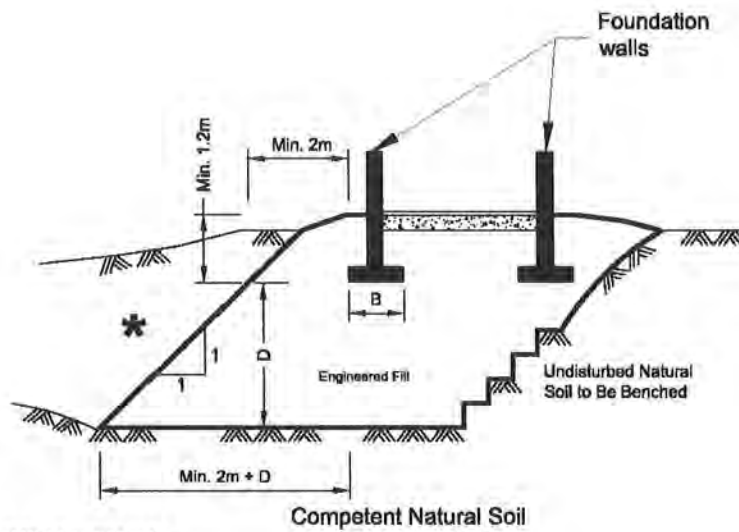
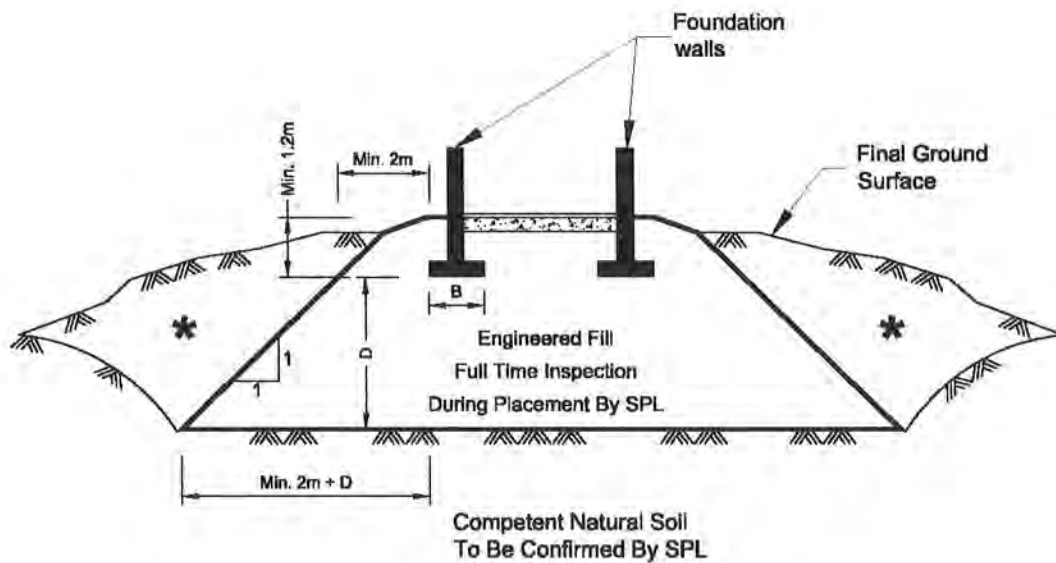
Adverse weather conditions such as rain make the placement of engineered fill to the required degree of density difficult or impossible; engineered fill cannot be placed during freezing conditions, i.e. normally not between December 15 and April 1 of each year.

The location of the foundations on the engineered fill pad is critical and certification by a qualified surveyor that the foundations are within the stipulated boundaries is mandatory. Since layout stakes are often damaged or removed during fill placement, offset stakes must be installed and maintained by the surveyors during the course of fill placement so that the contractor and engineering staff are continually aware of where the engineered fill limits lie. Excavations within the engineered fill pad must be backfilled with the same conditions and quality control as the original pad.

To perform satisfactorily, engineered fill requires the cooperation of the designers, engineers, contractors and all parties must be aware of the requirements. The minimum requirements are as follows, however, the geotechnical report must be reviewed for specific information and requirements.

1. Prior to site work involving engineered fill, a site meeting to discuss all aspects must be convened. The surveyor, contractor, design engineer and geotechnical engineer must attend the meeting. At this meeting, the limits of the engineered fill will be defined. The contractor must make known where all fill material will be obtained from and samples must be provided to the geotechnical engineer for review, and approval before filling begins.
2. Detailed drawings indicating the lower boundaries as well as the upper boundaries of the engineered fill must be available at the site meeting and be approved by the geotechnical engineer.
3. The building footprint and base of the pad, including basements, garages, etc. must be defined by offset stakes that remain in place until the footings and service connections are all constructed. Confirmation that the footings are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and SPL Consultants Limited. Without this confirmation no responsibility for the performance of the structure can be accepted by SPL Consultants Limited. Survey drawing of the pre and post fill location and elevations will also be required.
4. The area must be stripped of all topsoil and fill materials. Subgrade must be proof-rolled. Soft spots must be dug out. The stripped native subgrade must be examined and approved by a SPL Consultants Limited engineer prior to placement of fill.

5. The approved engineered fill material must be compacted to 100% Standard Proctor Maximum Dry Density throughout. Engineered fill should not be placed during the winter months. Engineered fill compacted to 100% SPMDD will settle under its own weight approximately 0.5% of the fill height and the structural engineer must be aware of this settlement. In addition to the settlement of the fill, additional settlement due to consolidation of the underlying soils from the structural and fill loads will occur and should be evaluated prior to placing the fill.
6. Full-time geotechnical inspection by SPL Consultants Limited during placement of engineered fill is required. Work cannot commence or continue without the presence of the SPL Consultants Limited representative.
7. The fill must be placed such that the specified geometry is achieved. Refer to the attached sketches for minimum requirements. Take careful note that the projection of the compacted pad beyond the footing at footing level is a minimum of 2 m. The base of the compacted pad extends 2 m plus the depth of excavation beyond the edge of the footing.
8. A bearing capacity of 75 kPa at SLS (125 kPa at ULS) can be used provided that all conditions outlined above are adhered to. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.
9. All excavations must be done in accordance with the Occupational Health and Safety Regulations of Ontario.
10. After completion of the engineered fill pad a second contractor may be selected to install footings. The prepared footing bases must be evaluated by engineering staff from SPL Consultants Limited prior to footing concrete placements. All excavations must be backfilled under full time supervision by SPL Consultants Limited to the same degree as the engineered fill pad. Surface water cannot be allowed to pond in excavations or to be trapped in clear stone backfill. Clear stone backfill can only be used with the approval of SPL Consultants Limited.
11. After completion of compaction, the surface of the engineered fill pad must be protected from disturbance from traffic, rain and frost. During the course of fill placement, the engineered fill must be smooth-graded, proof-rolled and sloped/crowned at the end of each day, prior to weekends and any stoppage in work in order to promote rapid runoff of rainwater and to avoid any ponding surface water. Any stockpiles of fill intended for use as engineered fill must also be smooth-bladed to promote runoff and/or protected from excessive moisture take up.
12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.
13. The geometry of the engineered fill as illustrated in these General Requirements is general in nature. Each project will have its own unique requirements. For example, if perimeter sidewalks are to be constructed around the building, then the projection of the engineered fill beyond the foundation wall may need to be greater.
14. These guidelines are to be read in conjunction with SPL Consultants Limited report attached.



\* Backfill in this area to be as per the SPL report.

December 12, 2025

**SHOREPLAN**

Mr. Richard Weldon  
Interra Lands Corporation  
7 Hoggs Lane  
Toronto ON M3B 2W5

Dear Sir,

**Re: Dynamic Beach Assessment  
1 Dunsford Lane, Tiny Township  
Our File: 11-1673**

This letter report describes an updated dynamic beach assessment completed for your Wymbolwood Beach property at 1 Dunsford Lane, Tiny. It was intended to determine the dynamic beach hazard limit along a section of the shore not included in a previous assessment, but it also confirmed our previous findings. The dynamic beach hazard is one of the three natural hazards defined by the Provincial Planning Statement (PPS) and the MNR (2001) Technical Guides prepared to support the PPS. It is our understanding that this letter will be submitted to Tiny Township in support of an application for site development to allow you to re-build closer to Georgian Bay than permitted by municipal bylaws.

### **Previous Assessments**

Shoreplan completed an initial natural hazards assessment for this property in 2013. That report, dated June 26, 2013, described the erosion hazard, the flooding hazard, and the dynamic beach hazard as defined in the MNR (2001) Technical Guides. There is no erosion hazard at this site. The flooding hazard was calculated from a wave uprush analysis using typical profiles derived a survey completed by Shoreplan staff. Existing retaining walls were determined to provide a physical barrier to beach profile adjustments and thus defined the dynamic beach limit.

A second assessment was completed on December 20, 2021 to update the flood hazard assessment following a period of record high Georgian Bay mean water levels. That assessment used a provided topographic survey prepared by J. D. Barnes Ltd and a design wave condition determined from an updated wave hindcast produced by the U.S. Army Corps of Engineers.

### **Existing Conditions**

The subject property is located on the east shore of Nottawasaga Bay immediately north of the 7<sup>th</sup> concession road allowance in the Township of Tiny. We originally reviewed the site on December 21, 2011 when the water level was approximately 176.0 metres. It was revisited on November 11,

2021 when the water level was approximately 176.6m, although that water level was measured during an approximately 0.4m setdown due to easterly wind conditions. A final site review was completed on June 30, 2025 when the water level was approximately 176.4m

Site details and grades were obtained from the topographic plan provided for our 2021 assessment (Plan of Survey Part of Lot 19 Concession 7 Township of Tiny County of Simcoe, prepared by J D Barnes Limited, dated 11/30/2021). Figure 1 of this report is based on that plan. All elevations discussed in this letter are relative to the geodetic datum CGVD1928:1978.

The property is triangular with a width in the order of 125 metres in the vicinity of the 178 metre contour line. Figure 1 shows a plan of the site. There are three existing building structures on the property, as shown in Figure 1. In the central part of the site, the structures are fronted by a low stone wall that blends into the dunes near the edges of the site. The land above the wall has been terraced with graded berms and stone and timber retaining walls. The dunes were once re-graded in front of the stone wall, but a new foredune has started to form.

Photo 1 to Photo 4 show matched pairs of 2021 and 2025 photographs with similar views in each matched pair. These show the extent of new dune formation that has occurred since our 2021 review.

### **Dynamic Beach Hazard Limit**

There is no doubt that this site meets the criteria for a dynamic beach as defined in the MNR 2001 Technical Guides. Dynamic beach limits can be established in the field by mapping the edge of the foredune on undisturbed beaches, but prior re-grading of the dunes precludes that option for this property. The technical guides' default definition of the dynamic beach allowance is a 30-metre setback from the limit of wave uprush under design conditions, but there is no scientific basis for that value, and it is always found to be excessive when detailed analyses are completed for sites like Wymbolwood Beach. The technical guides note that the default dynamic beach allowance is used in the absence of a site-specific study using accepted scientific and engineering methods. This report describes such a study and therefore supersedes the default allowance.

There are two key aspects to the dynamic beach allowance. One is to ensure that profile changes during design events do not damage infrastructure. The second is to ensure that new development does not encroach on the defined portion of the dynamic beach. The technical guides note that *“Defined portions of the dynamic beach means those portions of the dynamic beach which are highly unstable and/or critical to the natural protection and maintenance of the first dune feature and/or beach profile where development would create or aggravate flooding or erosion hazards, cause updrift and/or downdrift impacts and/or cause adverse environmental impacts.”*

The existing stone wall and timber retaining walls are a barrier between the beach where dunes can form and the existing buildings. Sand on the landward side of those walls will not play any role in the natural protection and maintenance of the first dune feature or beach profile. The walls form a physical limit to the dynamic beach allowance.

The current development plans show proposed buildings both to the north and south of the existing walls, so a more detailed assessment was required to determine the dynamic beach limit in those areas.

We have completed several peer reviewed dynamic beach hazard limit assessments on the Lake Huron shoreline within the jurisdiction of the St. Clair Region Conservation Authority (SCRCA) consistent with their prescribed methods. Their procedure is described in the following extract from correspondence with SCRCA:

**SCRCA General Policies and Guidelines for Determining the Dynamic Beach Hazard Limit and Re-Development within the Dynamic Beach Hazard**

**Determining the Dynamic Beach Hazard Limit**

*A proper study to further evaluate the dynamic beach limit typically involves a two-step process. The first step is an initial site reconnaissance to determine if further, more detailed analysis would be warranted. The initial reconnaissance would be accompanied by a review of existing data on the coastal processes and geomorphology of the area. The second step involves site specific field surveys of the nearshore and beach/dune profiles, compiling offshore bathymetric data, collecting sand samples and testing for grain size, determining design flood levels and wave conditions and numerical modeling of the likely limit of the dynamic beach hazard under storm and high water conditions using appropriate cross-shore beach profile models. (SCRCA SMP)*

We applied this same approach for 1 Dunsford Lane. Our site review showed that a more detailed analysis was required so we completed a numerical analysis of the dynamic beach hazard limit under storm and high water conditions using the DHI Litpack profile evolution model, which is an appropriate cross-shore beach profile model and is based on accepted scientific and engineering principles.

Storm conditions were selected from the results of the U.S. Army Corps of Engineers WIS hindcast for station 93383, which is located offshore of the site, where the water depth is 31m. The WIS data consists of hourly significant wave height, peak wave period and mean wave direction for the

45-year period from 1979 to 2023. The station 93383 WIS data was used for the wave uprush analysis described in our 2021 assessment report.

A storm selection process identified 10 storm events where the minimum peak event wave height was 3.75m and the event had at least 12 hours of wave heights greater than 1.0m. Figure 2 shows the wave height profiles for the top 5 of those storms, ranked by the peak wave height. Two of those storms were selected for the profile evolution modeling: Event 3 and Event 8. Event 3 had a duration of 45 hours. The storm peak wave condition had a significant wave height of 4.0m, a peak wave period of 9.3s and a mean wave back azimuth direction of 310 degrees at the hindcast location. Event 8 had a duration of 45 hours. The storm peak wave condition had a significant wave height of 4.4m, a peak wave period of 8.4s and a mean wave back azimuth direction of 300 degrees at the hindcast location.

Each of these storm events was transferred into a point on the 6m contour (chart datum), offshore of the site. Recorded water levels during the storm events were scaled up so that the peak of the storm was modeled at the 100-year instantaneous water level. Representative sediment parameters were taken from grain size distribution plots presented in SPL (2015). Those plots are presented in Figure 3. Sample TP15-03 GS#2, which was a medium sand with a median diameter of 0.45mm, was considered representative of the beach sand at this site. Sample TP15-03 GS#1, which was a fine sand with a median diameter of 0.2mm, was the finest sand sampled and was used for sensitivity testing during the profile evolution modeling.

Figure 1 shows the location of 6 profile lines labeled A to F. Profiles A to E were used during our 2021 wave uprush analysis and profile F was added for this assessment. These profiles were derived from the 2021 J.D. Barnes topographic survey and represent beach conditions following a period of high lake levels. This makes them well suited for profile evolution modeling under design conditions as no adjustment is necessary to represent a high water level equilibrium condition.

Baseline condition modeling was completed by considering both storm event 3 and storm event 8, described above, with the medium beach sand. The profile changes were similar, with each storm eroding away the beach both above and below the waterline and forming a nearshore bar. Figure 4 shows an example of the baseline condition modeling results for profile F.

Model sensitivity analyses were considered for both storm duration and sediment size. Storm duration sensitivity was tested by modeling storm events 3 and 8 back-to-back, which effectively ignores the beach rebuilding that occurs between major storm events. Sediment size sensitivity was tested by modeling the fine sand distribution rather than the medium sand representative of site conditions. Figure 5 and Figure 6 show the results of those analyses for profile F. The profile change modeling is more sensitive to storm duration than to sediment size.

Table 1 summarizes the distances that the 178m contour receded on each profile for the storm events modeled with the medium beach sand. The 178m contour was selected to provide a consistent benchmark for comparison. These values do not represent the full width over which the beach profile changed.

**Table 1 178m Contour Recession Distances**

<b>Profile</b>	<b>Storm Event 3</b>	<b>Storm Event 8</b>	<b>Storm Events 3 &amp; 8</b>
<b>A</b>	9.8	10.2	11.9
<b>B</b>	12.2	12.3	14.1
<b>C</b>	17.4	17.3	19.5
<b>D</b>	11.0	11.0	12.8
<b>E</b>	13.8	14.0	15.4
<b>F</b>	13.2	13.5	14.5
<b>minimum</b>	9.8	10.2	11.9
<b>maximum</b>	17.4	17.3	19.5
<b>average</b>	12.9	13.0	14.7

Using the same approach as applied for similar studies on Lake Huron, the dynamic beach hazard limit was defined using the results of the worst-case sensitivity analysis profile evolution modeling. That produced the dynamic beach hazard limit shown on Figure 7.

### **Development Plans**

A GIS shapefile of the dynamic beach hazard limit shown on Figure 7 has been provided to the project team to assist other aspects of the project. Development that takes place landward of this limit will not impact the dynamic beach. It will not create or aggravate flooding or erosion hazards, cause updrift and/or downdrift impacts, or cause adverse environmental impacts.

Cantilevering second storey floors or decks beyond the dynamic beach hazard limit will have no impact on the dynamic beach hazard. Foundation walls constructed close to the hazard limit must have buried scoured protection as an addition safeguard should design conditions be exceeded. It is important that maintenance access to that scour protection not be obstructed.

### **Conclusions**

This report describes a site-specific analysis of the dynamic beach hazard limit using accepted scientific and engineering methods. The dynamic beach hazard is the governing natural hazard at this site. Its location is shown on

Figure 7. Development that takes place landward of this limit will not impact the dynamic beach. It will not create or aggravate flooding or erosion hazards, cause updrift and/or downdrift impacts, or cause adverse environmental impacts.

**SHOREPLAN**

**Closing Comments**

We trust that these comments will assist you in your dealings with Tiny Township. Do not hesitate to call should you have any questions regarding this letter report.

Yours truly

Shoreplan Engineering Limited



Bruce Pinchin, P.Eng.



**References**

MNR, 2001. Great Lakes - St. Lawrence River System and Large Inland Lakes. Technical Guides for flooding, erosion and dynamic beaches in support of natural hazards policies 3.1 of the provincial policy statement. Watershed Science Centre. ISBN: 9780968819616

SPL (2015). Report on Preliminary Geotechnical, Investigation Wymbolwood Beach Resort, 1 Dunsford Lane, Township of Tiny, Ontario. Unpublished report prepared by SPL Consultants Limited for Romspen Investment Corporation. SPL Project No. 10001448. April 13, 2015

**Photo 1, November 2021**



**SHOREPLAN**

**Photo 2, June 2025**



Photo 3, November 2021



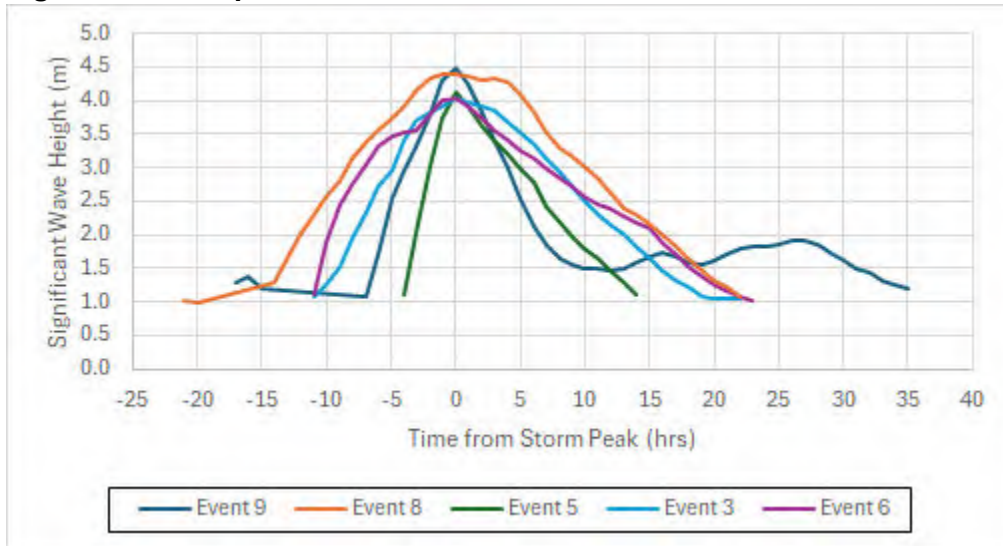
SHOREPLAN

Photo 4, June 2025





**Figure 2 Top Storm Events from WIS Hindcast**

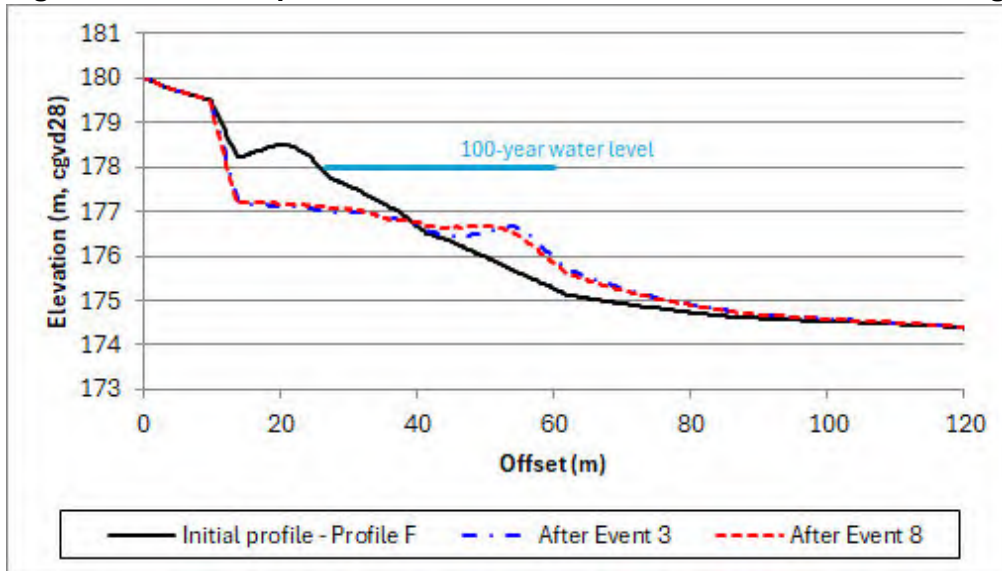


**SHOREPLAN**

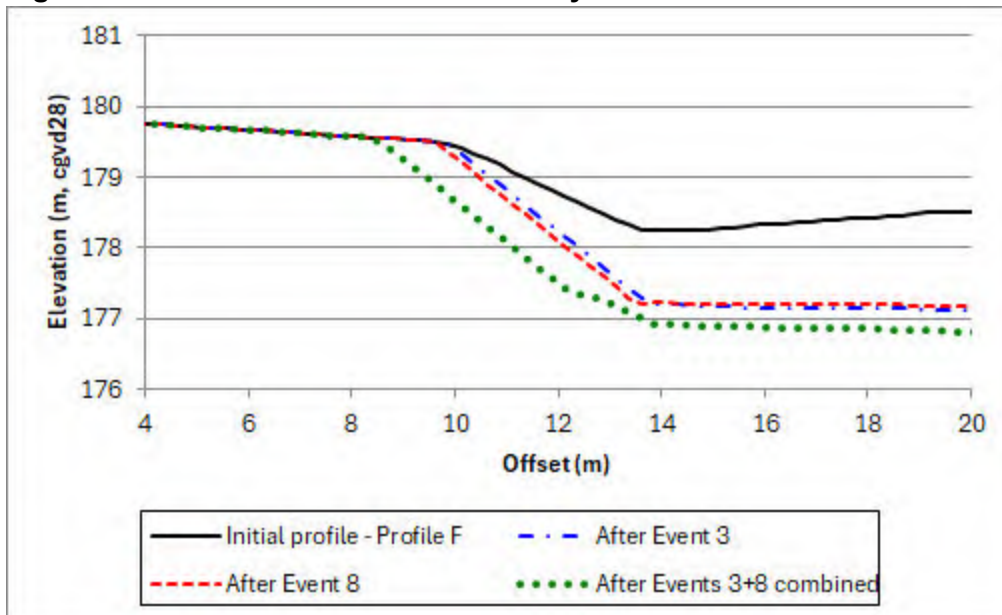
**Figure 3 Grain Size Distribution Plots**



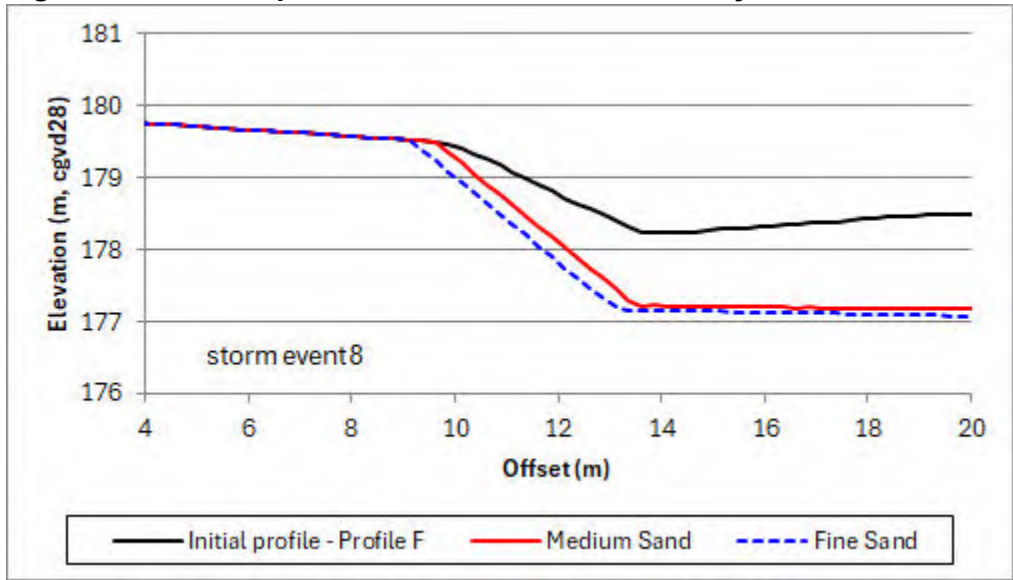
**Figure 4 Example of Baseline Condition Profile Evolution Modeling**



**Figure 5 Profile Evolution Sensitivity to Storm Duration**



**Figure 6 Example of Profile Evolution Sensitivity to Sand Size**

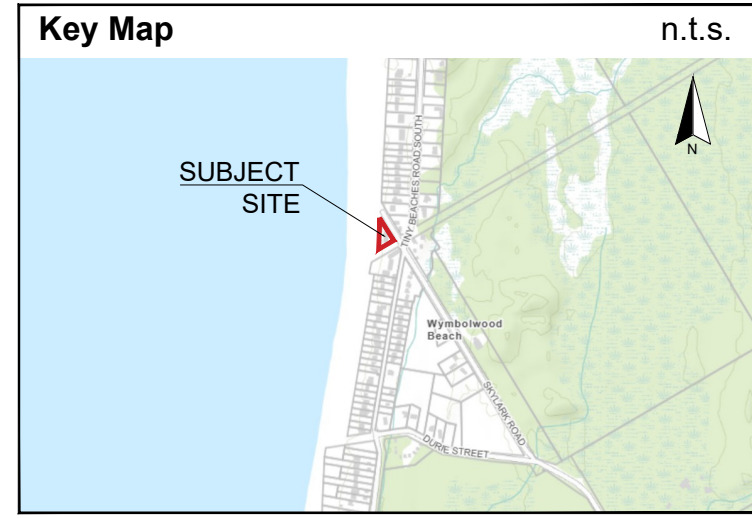
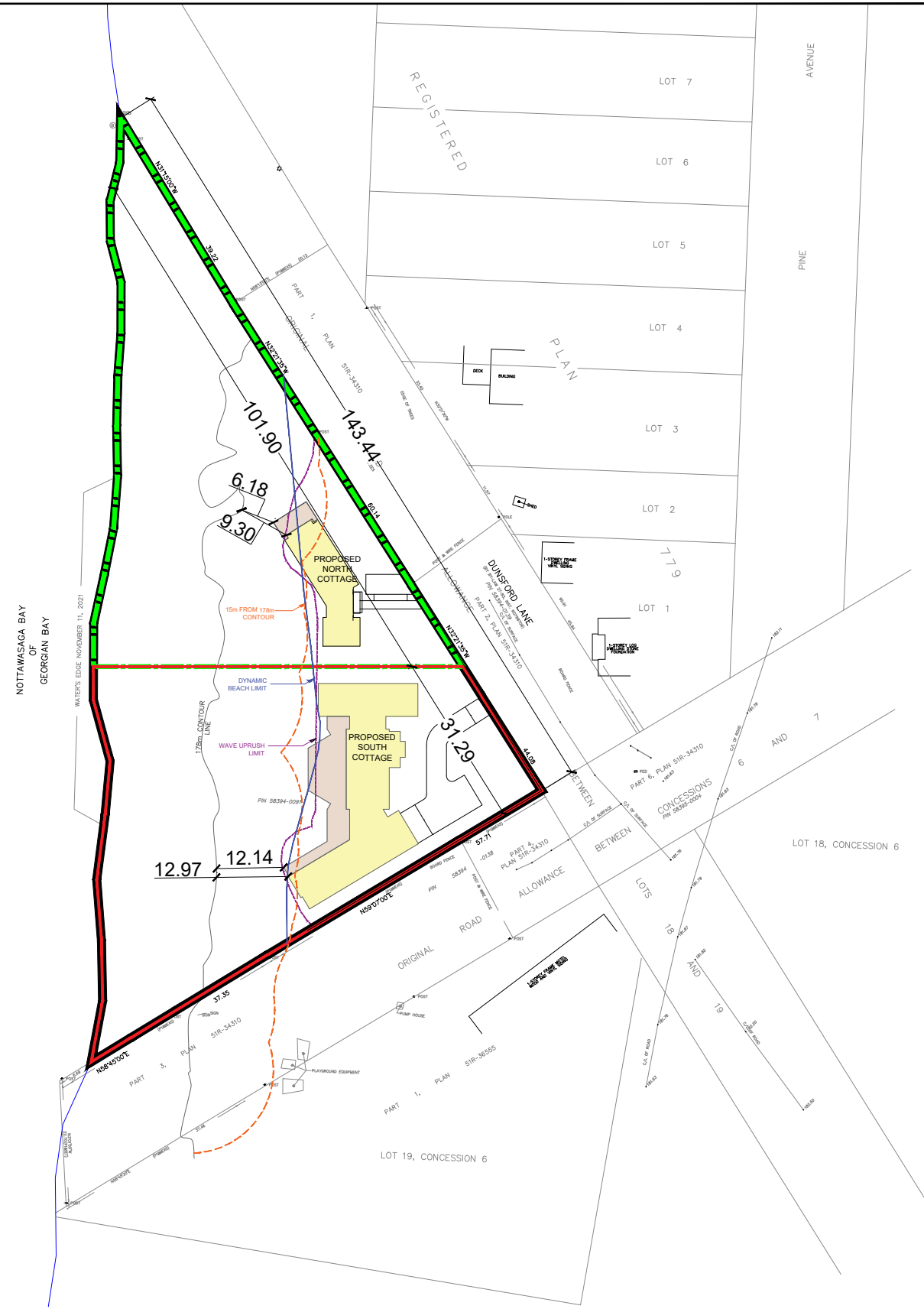


**SHOREPLAN**

**Figure 7 Dynamic Beach Hazard Limit**

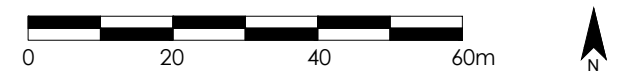


# FIGURES



- LEGEND**
- Subject Site (Area: 6,719.04m<sup>2</sup> / 0.37 ha.)
  - Lands to be Severed:
    - Area: 3,186.44m<sup>2</sup>
    - Frontage: 101.90m
  - Lands to be Retained:
    - Area: 31.29m
    - Frontage: 3,532.60m<sup>2</sup>

Source: Township of Tiny Zoning By-Law 22-075  
 Note: This drawing is for discussion purposes only. The information shown is approximate and subject to change.



Date: Dec. 15, 2025	Drawn By: A.S.
File: 21 - 1161	Drawn By: N.S.

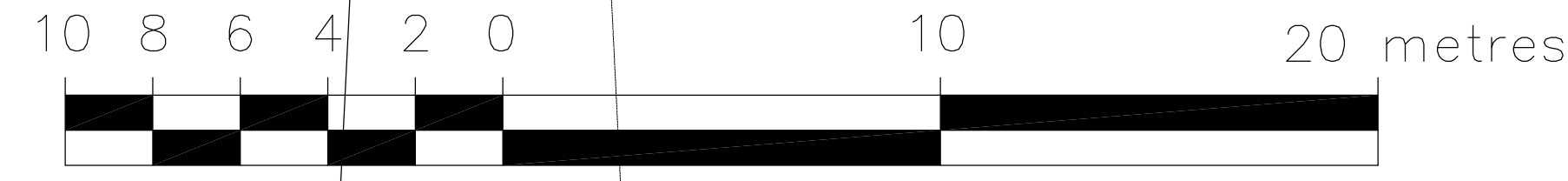
# SEVERANCE SKETCH

## 1 DUNSFORD LANE, TOWNSHIP OF TINY

SCHEDULE OF REVISIONS			
No.	Date	Description	By

**INNOVATIVE PLANNING SOLUTIONS**  
PLANNERS • PROJECT MANAGERS • LAND DEVELOPMENT  
 647 WELHAM ROAD, UNIT 9, BARRIE, ON, L4N 0B7 Tel: 705-812-3281  
 3800 STEELS AVE. W, SUITE 200W, VAUGHAN, ON, L4L 4G9 Tel: 905-291-7525  
info@ipsconsultinginc.com / www.ipsconsultinginc.com

PLAN OF SURVEY OF  
PART OF LOT 19  
CONCESSION 7  
TOWNSHIP OF TINY  
COUNTY OF SIMCOE



METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

LEGEND

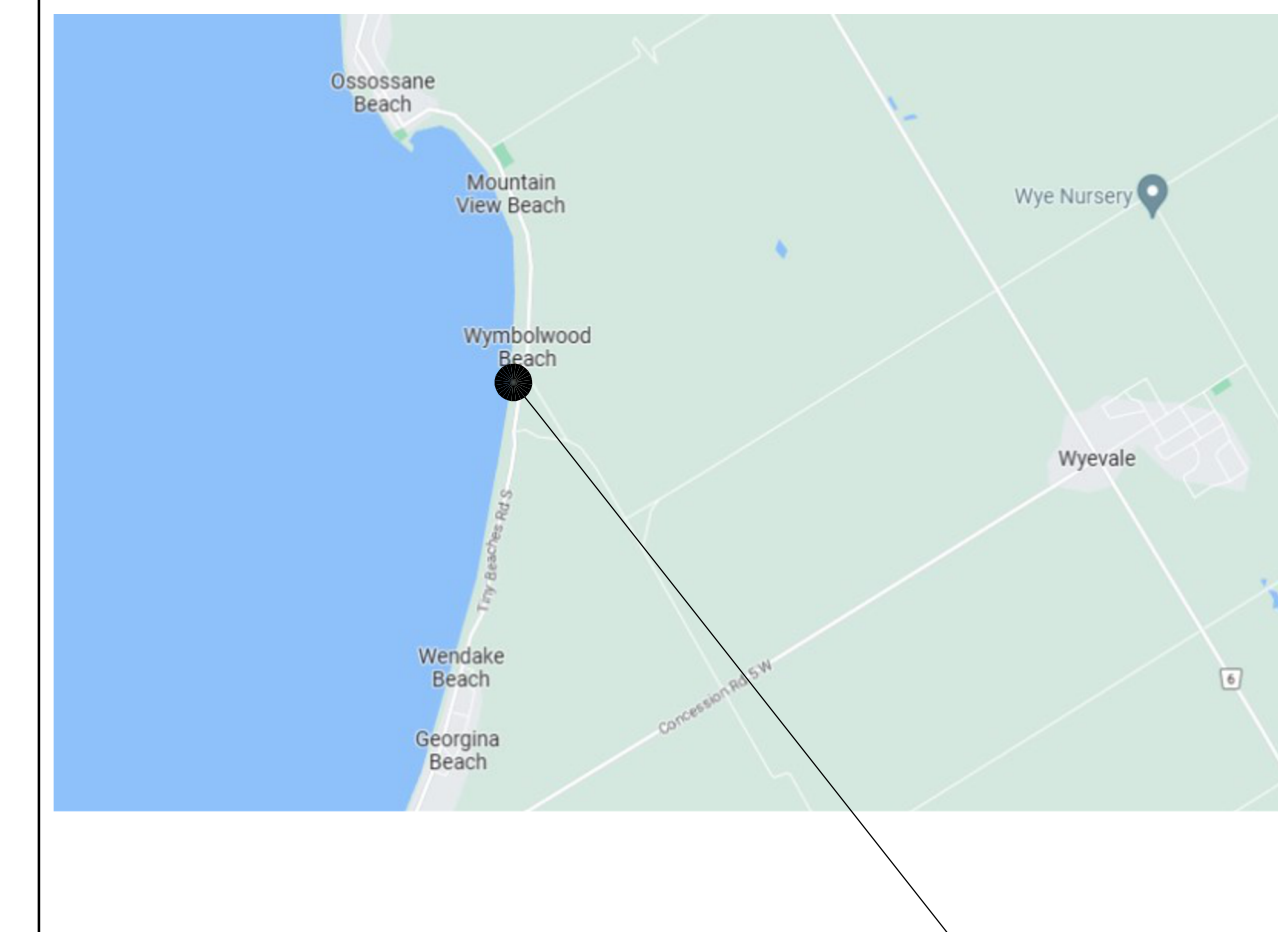
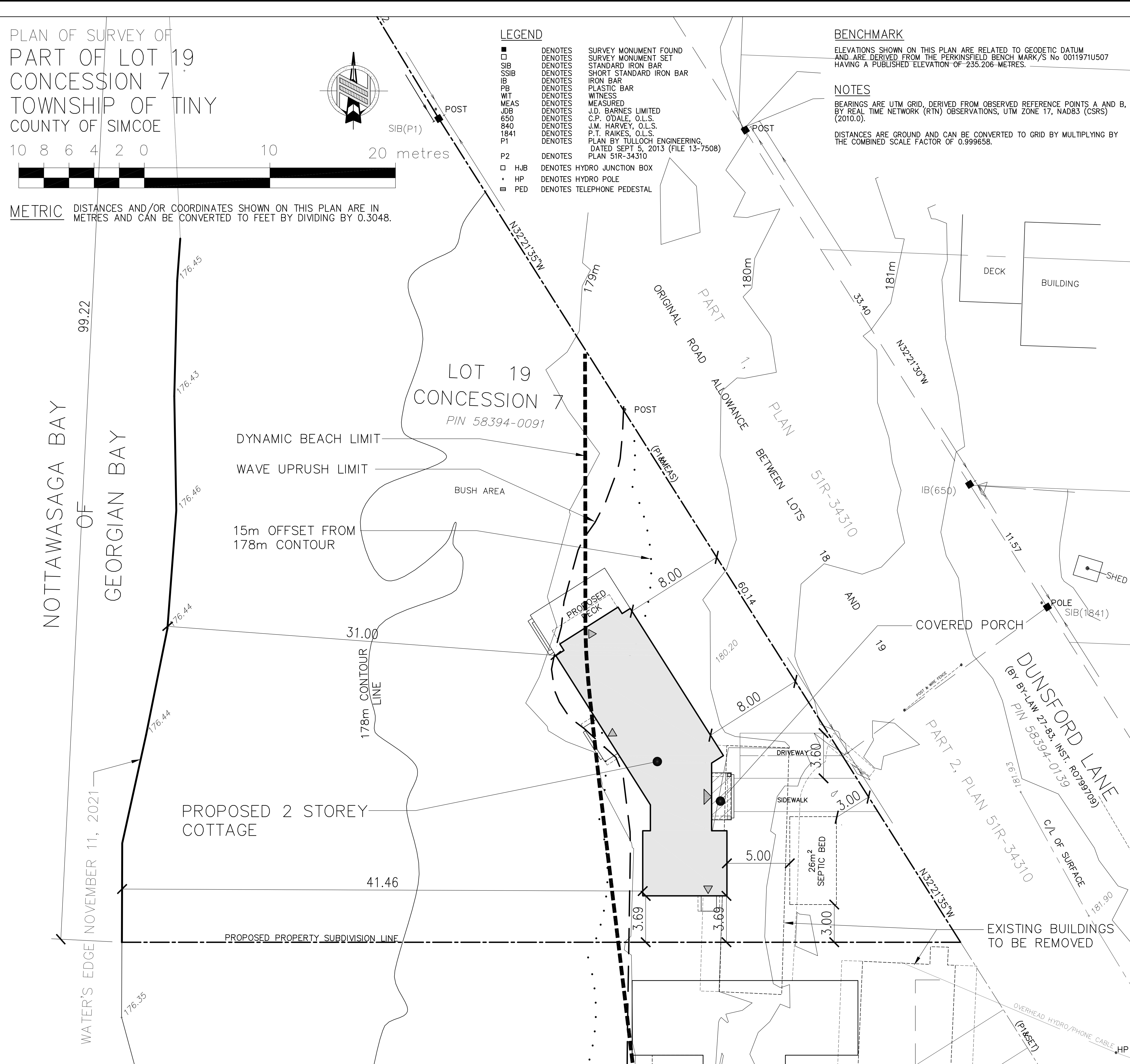
- DENOTES SURVEY MONUMENT FOUND
- DENOTES SURVEY MONUMENT SET
- SIB DENOTES STANDARD IRON BAR
- SSIB DENOTES SHORT STANDARD IRON BAR
- IB DENOTES IRON BAR
- PB DENOTES PLASTIC BAR
- WT DENOTES WITNESS
- MEAS DENOTES MEASURED
- JOB DENOTES J.D. BARNES LIMITED
- 650 DENOTES C.P. O'DALE, O.L.S.
- 840 DENOTES J.M. HARVEY, O.L.S.
- 1841 DENOTES P.T. RAIKES, O.L.S.
- P1 DENOTES PLAN BY TULLOCH ENGINEERING, DATED SEPT 5, 2013 (FILE 13-7508)
- P2 DENOTES PLAN 51R-34310
- HJB DENOTES HYDRO JUNCTION BOX
- HP DENOTES HYDRO POLE
- PED DENOTES TELEPHONE PEDESTAL

BENCHMARK

ELEVATIONS SHOWN ON THIS PLAN ARE RELATED TO GEODETIC DATUM AND ARE DERIVED FROM THE PERKINSFIELD BENCH MARK/S No 0011971U507 HAVING A PUBLISHED ELEVATION OF -235.206 METRES.

NOTES

BEARINGS ARE UTM GRID, DERIVED FROM OBSERVED REFERENCE POINTS A AND B, BY REAL TIME NETWORK (RTN) OBSERVATIONS, UTM ZONE 17, NAD83 (CSRS) (2010.0).  
DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999658.



1 KEY PLAN  
A1-2 SCALE: DNS

OWNER: Interra Lands Inc. 7 Hoggs Lane Toronto ON, M5B 2W5  
ARCHITECT: IAN MACLAREN ARCHITECT INC. 295 ROBINSON STREET OAKVILLE, ONTARIO L6J 1G7 ATTENTION: IAN MACLAREN PH: 905-339-1219 EXT. 225

APPLICANT: OWNER'S AGENT

SITE STATISTICS

	%	METRIC	IMPERIAL
LOT AREA: FULL LOT		3,186.44	34,298.55
LOT FRONTSIDE:		99.22	325.52
LOT COVERAGE: FULL LOT			
PROPOSED COVERAGE:			
PROPOSED COTTAGE		154.08	1,658.46
PROPOSED COVERED PORCH		5.57	60.00
PROPOSED LOT COVERAGE: (FULL LOT)	5.01%	159.65	1718.46

COTTAGE GROSS FLOOR AREAS:

GROUND FLOOR	154.08	1,658.46
SECOND FLOOR	105.00	1,130.22
PROPOSED TOTAL COTTAGE FLOOR AREA:	259.08	2788.68

Drawings must NOT be scaled. Contractor must check and verify all dimensions, specifications and drawings on site and report any discrepancies to the architect prior to proceeding with any of the work.



SITE ACCREDITATION:

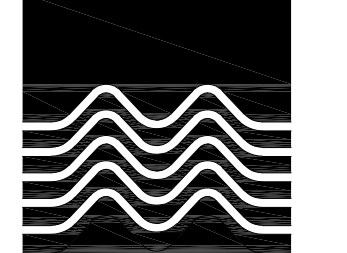
PART OF:  
LOT 19, CONC 7  
GEOGRAPHIC TOWNSHIP OF TINY  
COUNTY OF SIMCOE  
DISTRICT MUNICIPALITY OF PARRY SOUND  
INFORMATION TAKEN FROM A SURVEY PREPARED BY:  
LAURENCE J. KUELLING  
ONTARIO LAND SURVEYORS  
2022

METRIC DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

SITE LEGEND:

- PROPERTY LINE
- EXISTING GRADE
- FINISHED GRADE
- F.F.E. FINISHED FLOOR ELEVATION
- F.B.E. FINISHED BASEMENT ELEVATION
- F.D.E. FINISHED DECK ELEVATION
- ▲ MAIN ENTRANCE
- ▲ SECONDARY ENTRANCE
- PROPOSED ADDITION AREA
- TREE HOARDING
- EXISTING TREE TO REMAIN
- EXISTING TREE TO BE REMOVED
- SURVEY MONUMENT FOUND
- SURVEY MONUMENT SET
- SIB STANDARD IRON BAR
- IB IRON BAR

2	10.24.25	DYNAMIC BEACH LINE ADDED
1	03.07.23	PRELIMINARY REVIEW
REF.	DATE:	DESCRIPTION:
REVISION / ISSUANCE:		



ian MACLAREN ARCHITECT inc  
905.339.1219 www.ianmclarenarchitect.ca  
295 ROBINSON ST., SUITE 300, OAKVILLE, ON L6J 1G7

CLIENT:  
**WYMBOLWOOD COTTAGE NORTH**

ADDRESS: DUNSFORD LANE  
MUNICIPALITY: TINY, ON.

DRAWING TITLE:  
**SITE PLAN & SITE STATISTICS**

DRAWN: A.B.  
DATE: MAR.2023 SCALE: AS NOTED  
JOB NUMBER: 20130 SHEET NUMBER: A1-2

3 SITE PLAN  
A1-2 SCALE: 1:150

2 SITE STATISTICS  
A1-2 SCALE: DNS

I:\02\2023\CONSULTING\DOCUMENTS\SERVICES\1130\_WYMBOLWOOD\_COTTAGE\DRAWINGS\2023\WYMBOLWOOD\_SITE\_PLAN.DWG

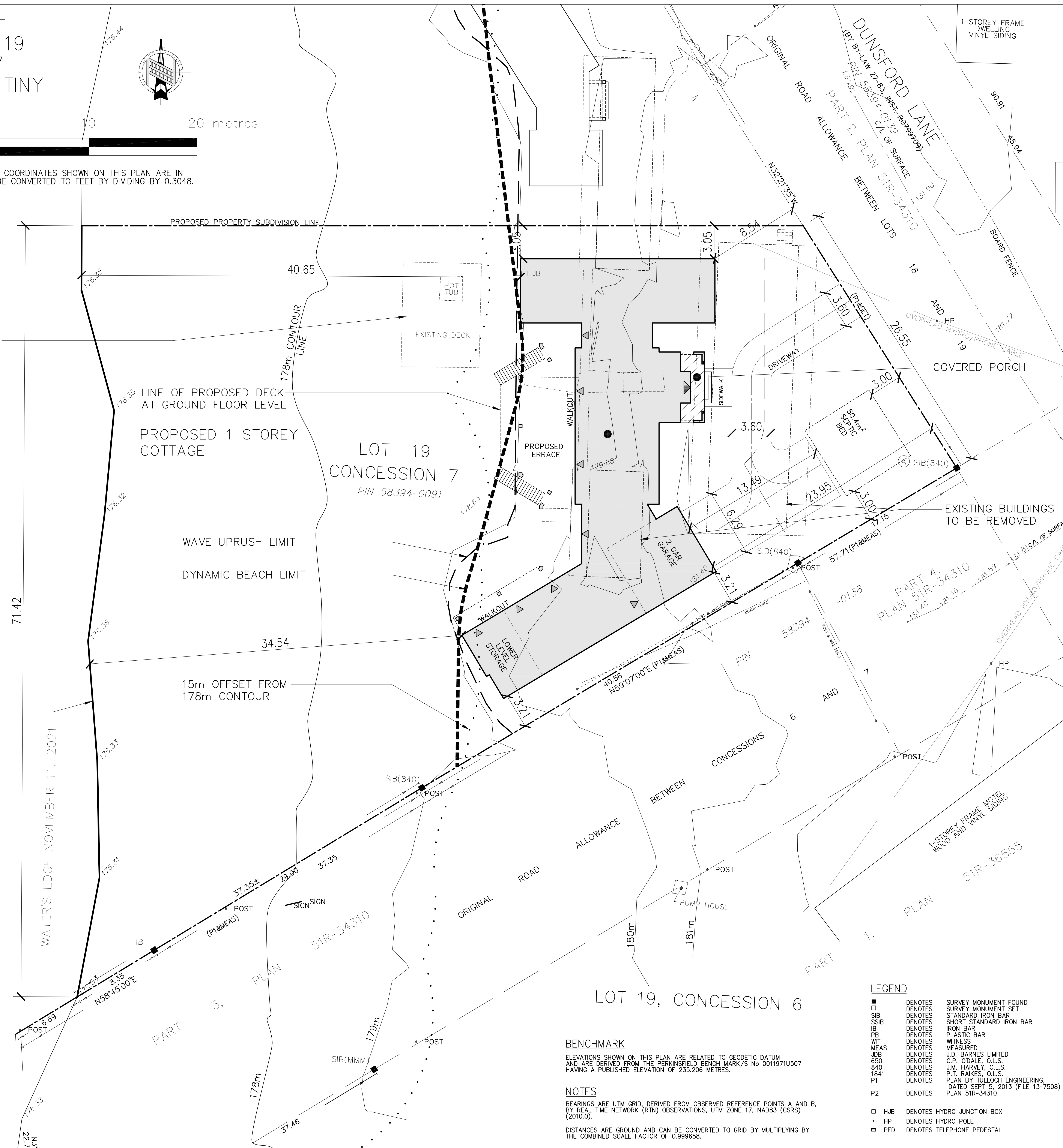
PLAN OF SURVEY OF  
PART OF LOT 19  
CONCESSION 7  
TOWNSHIP OF TINY  
COUNTY OF SIMCOE

10 8 6 4 2 0 10 20 metres

METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

EXISTING DECK & HOT TUB TO BE REMOVED

NOTTAWASAGA BAY  
OF  
GEORGIAN BAY



**BENCHMARK**

ELEVATIONS SHOWN ON THIS PLAN ARE RELATED TO GEODEIC DATUM AND ARE DERIVED FROM THE PERKINSFIELD BENCH MARK/S No 0011971U507 HAVING A PUBLISHED ELEVATION OF 235.206 METRES.

**NOTES**

BEARINGS ARE UTM GRID, DERIVED FROM OBSERVED REFERENCE POINTS A AND B, BY REAL TIME NETWORK (RTN) OBSERVATIONS, UTM ZONE 17, NAD83 (CSRS) (2010.0).  
DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999638.

**LEGEND**

- DENOTES SURVEY MONUMENT FOUND
- DENOTES SURVEY MONUMENT SET
- SIB DENOTES STANDARD IRON BAR
- SSIB DENOTES SHORT STANDARD IRON BAR
- IB DENOTES IRON BAR
- PB DENOTES PLASTIC BAR
- WIT DENOTES WITNESS
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- JOB DENOTES J.D. BARNES LIMITED
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- HP DENOTES HYDRO POLE
- PED DENOTES TELEPHONE PEDESTAL

**1 KEY PLAN**

SCALE: DNS

**OWNER**  
Interra Lands Inc.  
7 Hogs Lane  
Toronto ON  
M8B 2W5

**ARCHITECT**  
IAN MACLAREN ARCHITECT INC.  
295 ROBINSON STREET  
OAKVILLE, ONTARIO L6J 1G7  
ATTENTION: IAN MACLAREN  
PH: 905-339-1219 EXT. 225

**APPLICANT:**  
OWNER'S AGENT

**SITE STATISTICS**

**ZONING:**

	%	METRIC	IMPERIAL
LOT AREA: FULL LOT		3,532.60	38,024.59
LOT FRONTAGE:		71.42	234.32
LOT COVERAGE: FULL LOT			
PROPOSED COVERAGE:			
PROPOSED COTTAGE		435.46	4,687.24
PROPOSED COVERED PORCH		11.83	127.38
PROPOSED LOT COVERAGE: (FULL LOT)	12.66%	447.29	4814.62

**COTTAGE GROSS FLOOR AREAS:**

GROUND FLOOR (EXCLUDES GARAGE)	381.19	4,103.10
PARTIAL FINISHED BASEMENT	322.27	3,488.88
PROPOSED TOTAL COTTAGE FLOOR AREA:	703.46	7571.98

**2 SITE STATISTICS**

SCALE: DNS

20130

A1-2

Drawings must NOT be scaled. Contractor must check and verify all dimensions, specifications and drawings on site and report any discrepancies to the architect prior to proceeding with any of the work.



**SITE ACCREDITATION:**

PART OF:  
LOT 19, CONC 7

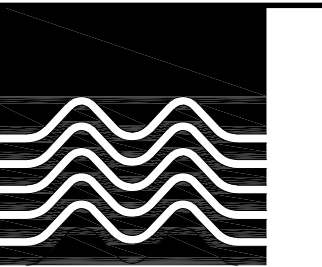
GEOGRAPHIC TOWNSHIP OF TINY  
COUNTY OF SIMCOE  
DISTRICT MUNICIPALITY OF PARRY SOUND  
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ONTARIO LAND SURVEYORS  
2022

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- ▭ PROPOSED ADDITION AREA
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- EXISTING TREE TO REMAIN
- EXISTING TREE TO BE REMOVED
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- SURVEY MONUMENT SET
- SIB STANDARD IRON BAR
- IB IRON BAR

2	10.24.25	DYNAMIC BEACH LINE ADDED
1	03.07.23	PRELIMINARY REVIEW
REF.	DATE:	DESCRIPTION:
REVISION / ISSUANCE:		



CLIENT:  
WYMBOLWOOD  
COTTAGE  
SOUTH

ADDRESS: DUNSFORD LANE  
MUNICIPALITY: TINY, ON.

DRAWING TITLE:  
**SITE PLAN &  
SITE STATISTICS**

DRAWN: A.B.  
DATE: MAR 2023 SCALE: AS NOTED  
JOB NUMBER: SHEET NUMBER:

20130

A1-2

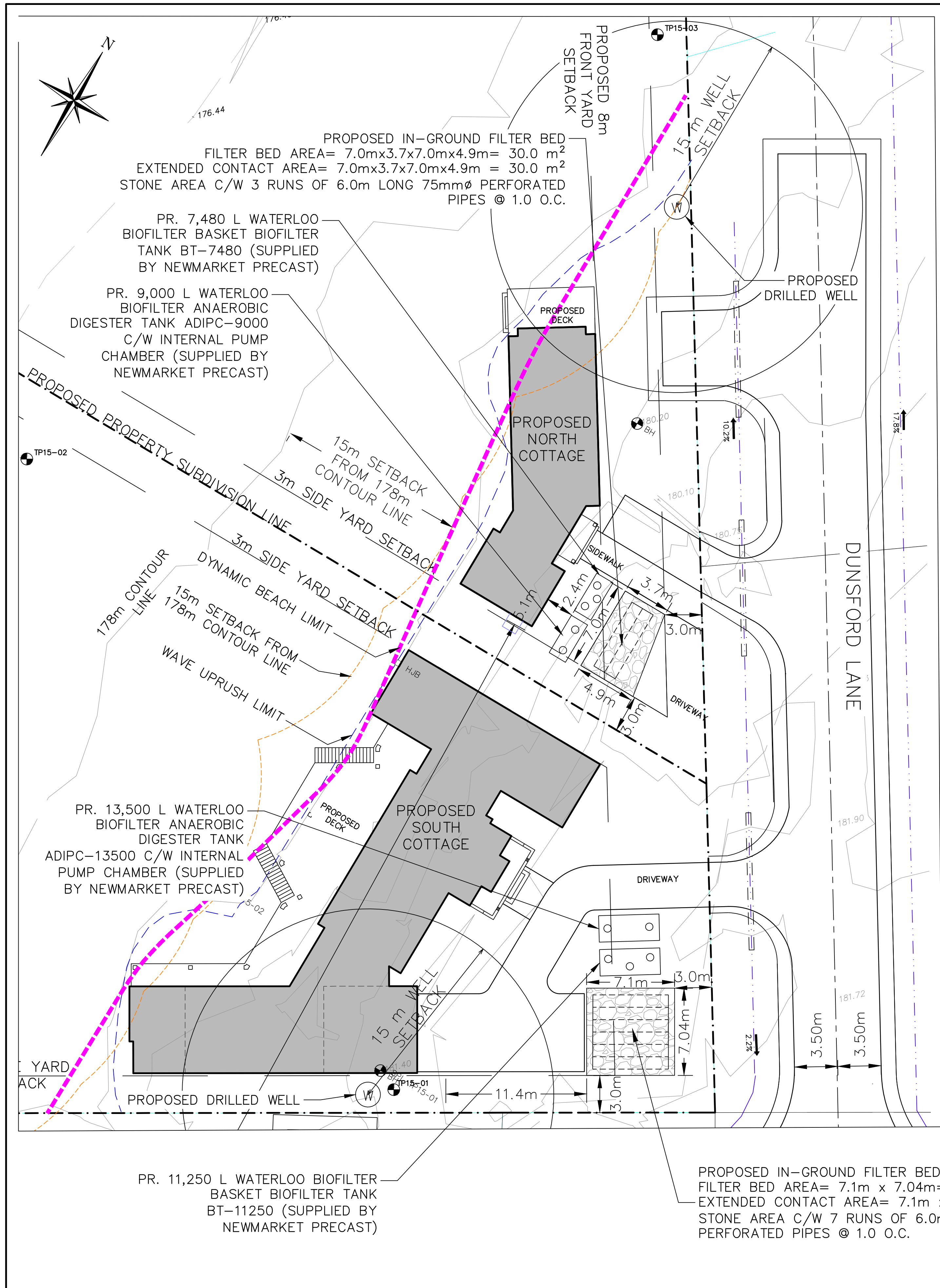
**3 SITE PLAN**

SCALE: 1:150

20130

A1-2





**ONSITE SEWAGE SYSTEM OBC AND TINY TOWNSHIP SETBACKS NOTES:**

- 5m SETBACK FROM SEWAGE SYSTEM TO BUILDING
- 1.5m SETBACK FROM TANKS TO BUILDING
- 3m SETBACK TO PROPERTY LINE
- 15m DRILLED WELL SETBACK FROM SEWAGE SYSTEM
- 15m SETBACK FROM 178 masl CONTOUR LINE (HIGH WATER ELEVATION OF GEORGIAN BAY) PER TINY TOWNSHIP REQUIREMENTS

**ON-SITE SEWAGE SYSTEM DESIGN NOTES – NORTH COTTAGE**

PROPOSED 4 BEDROOM, 242 m <sup>2</sup> COTTAGE WITH THIRTY-SIX (35.5) FIXTURE UNITS	BASE FLOW (4 BEDROOMS) = 2,000 L/DAY ADDITIONAL FLOOR AREA (42 m <sup>2</sup> ) = 500 L/DAY ADDITIONAL FIXTURE UNITS (15.5) = 775 L/DAY Q TOTAL (2,000+775) = 2,775 L/DAY
SOIL PERCOLATION RATE	T = 6 min/cm (ESTIMATED BY C.F. CROZIER)
PROPOSED TREATMENT UNIT	WATERLOO BIOFILTER AD-BA30
PROPOSED FILTER BED STONE AREA	MINIMUM SIZE = Q/100 = 2,775/100 = 28.0 m <sup>2</sup> PROVIDED = 30.0 m <sup>2</sup>
PROPOSED FILTER BED EXTENDED CONTACT AREA	MINIMUM SIZE = QT/850 = 2,775*6/850 = 20m <sup>2</sup> PROVIDED = 30.0 m <sup>2</sup>

**ON-SITE SEWAGE SYSTEM DESIGN NOTES – SOUTH COTTAGE**

PROPOSED 9 BEDROOM, 403 m <sup>2</sup> COTTAGE WITH SEVENTY (70) FIXTURE UNITS	BASE FLOW (9 BEDROOMS) = 2,500 L/DAY ADDITIONAL BEDROOMS (4) = 2,000 L/DAY ADDITIONAL FLOOR AREA (203 m <sup>2</sup> ) = 2,075 L/DAY ADDITIONAL FIXTURE UNITS (50) = 2,500 L/DAY Q TOTAL (2,500+2,500) = 5,000 L/DAY
SOIL PERCOLATION RATE	T = 6 min/cm (ESTIMATED BY C.F. CROZIER)
PROPOSED TREATMENT UNIT	WATERLOO BIOFILTER AD-BA50
PROPOSED FILTER BED STONE AREA	MINIMUM SIZE = Q/100 = 5,000/100 = 50.0 m <sup>2</sup> PROVIDED = 50.0 m <sup>2</sup>
PROPOSED FILTER BED EXTENDED CONTACT AREA	MINIMUM SIZE = QT/850 = 5,000*6/850 = 35 m <sup>2</sup> PROVIDED = 50.0 m <sup>2</sup>



**LEGEND**

- PROPERTY LINE
- EXISTING GRADE
- TESTPIT LOCATION (SPL CONSULTANTS, APRIL 13, 2015)
- PROPOSED FILTER BED
- PROPOSED TREATMENT TANK
- PROPOSED COTTAGE BUILDING

- THIS DRAWING IS THE EXCLUSIVE PROPERTY OF C.F. CROZIER & ASSOCIATES INC. AND THE REPRODUCTION OF ANY PART WITHOUT PRIOR WRITTEN CONSENT OF THIS OFFICE IS STRICTLY PROHIBITED.
- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, LEVELS, AND DATUMS ON SITE AND REPORT ANY DISCREPANCIES OR OMISSIONS TO THIS OFFICE PRIOR TO CONSTRUCTION.
- THIS DRAWING IS TO BE READ AND UNDERSTOOD IN CONJUNCTION WITH ALL OTHER PLANS AND DOCUMENTS APPLICABLE TO THIS PROJECT.
- DO NOT SCALE THE DRAWINGS.
- ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

**SURVEY NOTE:**  
 ORIGINAL SURVEY WAS COMPLETED BY J. D. BARNES LTD. DATED MARCH 6, 2015.

**BENCH MARK:**  
 ELEVATIONS HEREON ARE GEODETIC AND ARE DERIVED FROM THE PERKINSFIELD MARK/S NO. 001971U507 HAVING A PUBLISHED ELEVATION OF 235.206 METRES.

No.	ISSUE	DATE: MMM/DD/YYYY
1	ISSUED FOR 1ST SUBMISSION	DEC/17/2025

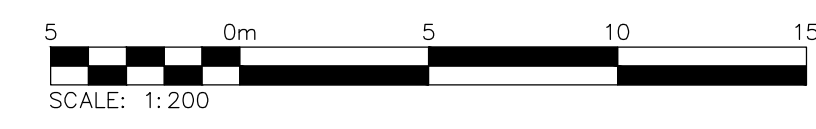
**PRELIMINARY**  
 NOT TO BE USED FOR CONSTRUCTION

Project: 1 DUNSFORD LANE TOWNSHIP OF TINY

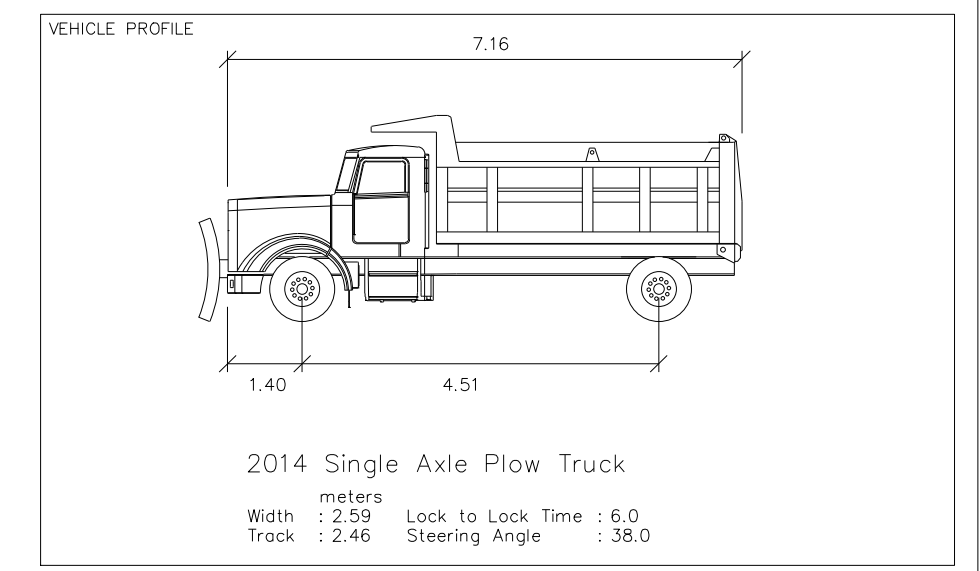
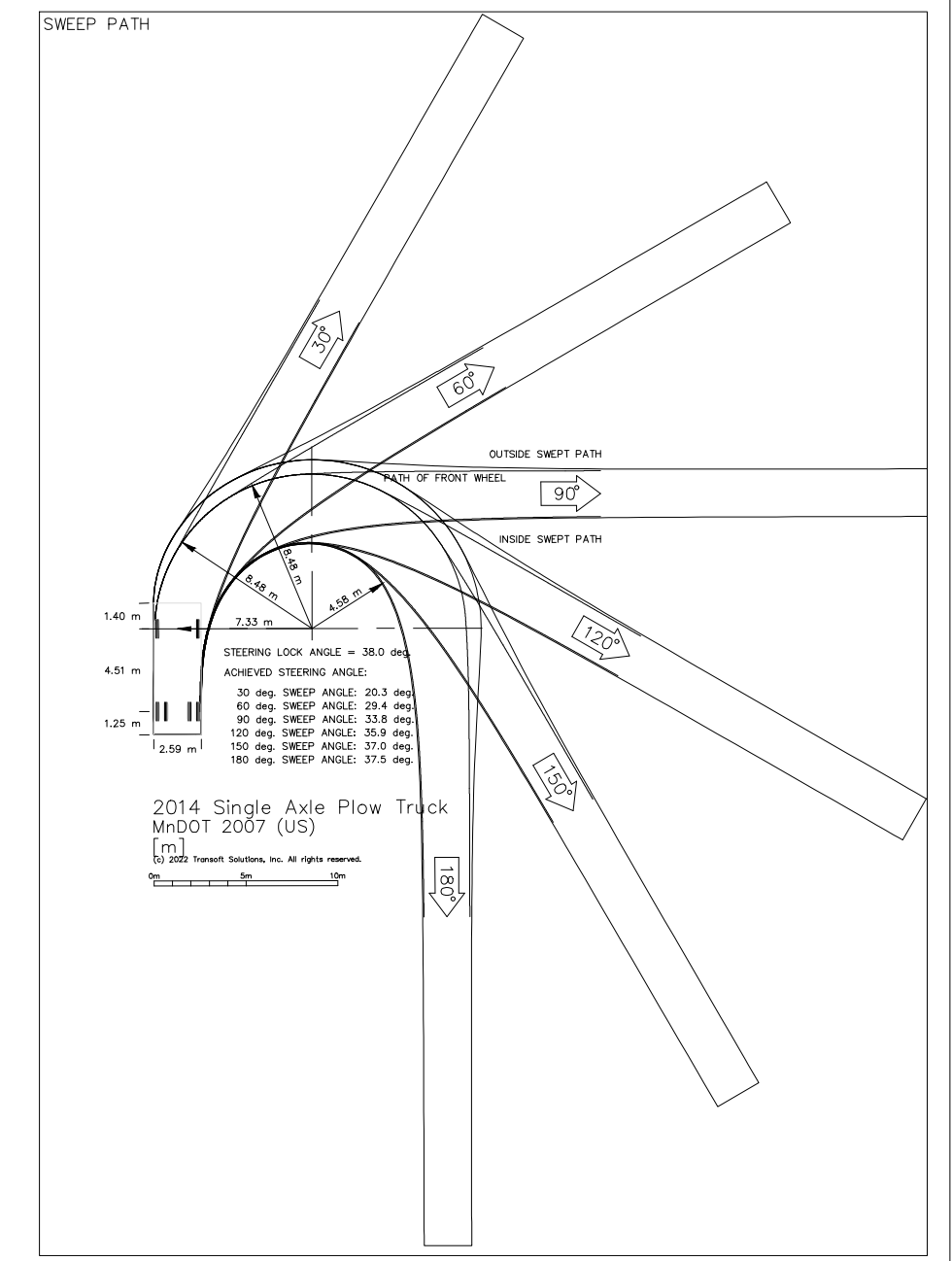
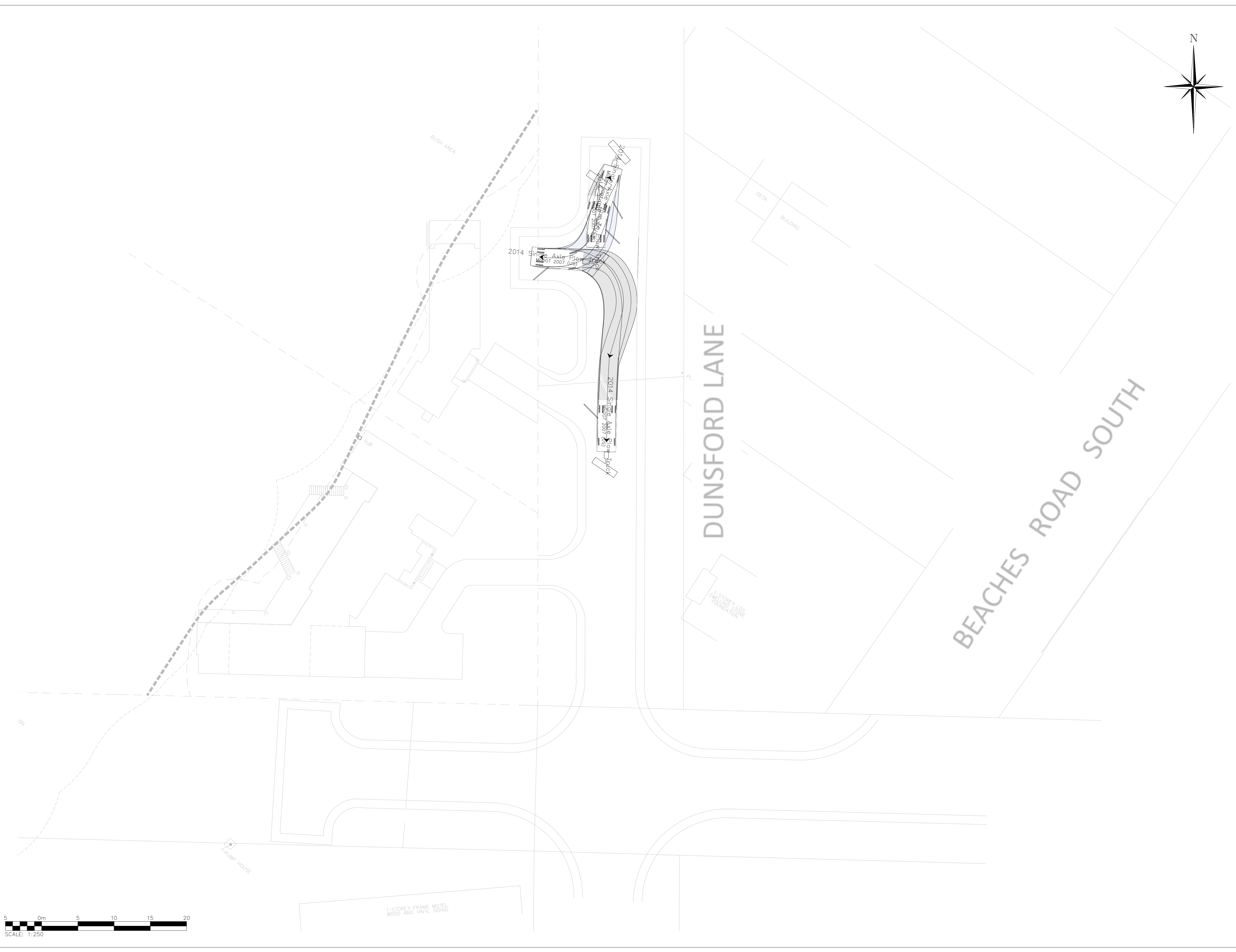
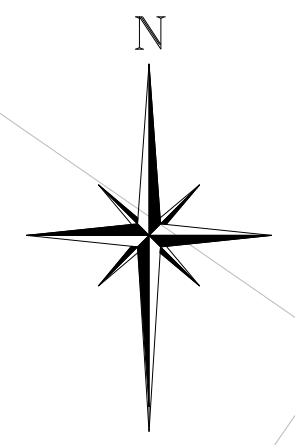
Drawing: SANITARY SERVICING PLAN

**CROZIER CONSULTING ENGINEERS**  
 ADMIRAL BUILDING  
 1 FIRST STREET, SUITE 200  
 COLLINGWOOD, ON, L9Y 1A1  
 705-446-3510 T  
 705-446-3520 F  
 WWW.CFCROZIER.CA  
 INFO@CFCROZIER.CA

Drawn By: R.D.M./A.L.	Design By: R.D.M./A.L.	Project: 2251-6304
Check By: K.S./K.R.	Scale: 1:200	Drawing: FIGURE 5



**FOR REVIEW**  
NOT TO BE USED FOR CONSTRUCTION



No.	ISSUE	DATE: MM/DD/YYYY
1	ISSUED FOR REVIEW	08/12/2022
2	ISSUED FOR 1st SUBMISSION	12/12/2025

Project  
1 DUNSFORD LANE  
TOWNSHIP OF TINY

Drawing  
SNOW PLOW  
VEHICLE MANEUVERING ANALYSIS

**CROZIER**  
CONSULTING ENGINEERS

2800 HIGH POINT DRIVE  
SUITE 100  
MILTON, ON, L9T 6P4  
905-875-0026 T  
905-875-4915 F  
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Drawn By	M.F.	Design By	Project	2251-6304
Check By	R.M.	Check By	Scale	1:250
			Drawing	T300

