Geotechnical Desktop Study – Smithville 3A/Block Plan Area 9

Townline and Port Davidson Road,
Smithville, ON

July 2, 2024

Prepared for:
Lockbridge Development Inc.

Prepared by:

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Project No. 161414473

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Revision	Description	Author		Quality Check		Independent Review	
0	Draft	Essa Nimer	2024/04/08	Raid Khamis	2024/04/08	Hassan Gilani	2024/04/08
0	Final	Essa Nimer	2024/07/02	Raid Khamis	2024/07/02	Hassan Gilani	2024/07/02

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July 2, 2024

1.0 INTRODUCTION

Stantec Consulting Ltd., (Stantec) has been engaged by Lockbridge Development Inc, hereafter referred to as “the client” (Client) to conduct a geotechnical study for an existing approximately 44 hectares (100 acres) agricultural land. The site is currently forming parts of Lot 31 and Lot 32, Concession 6, located in Smithville, Municipality of West Lincoln, ON. The approximately 40 hectares parcel of land is bounded by Port Davidson Road to the West, Townline Road to the North, agricultural lands to the south and existing residential and agricultural lands to the East. Stantec has conducted a geotechnical investigation on its approximately 29-hecatres, excluding 11-hecatres located in its northwest corner on **Drawing No. 1 –** Site Plan bounded by a thick red border.

Stantec has provided the findings of the geotechnical investigations and geotechnical design and construction recommendations based thereon through its geotechnical investigation report dated April 10, 2024. The reader is referred to the Stantec geotechnical report for the details.

The 11-hectares land located in the northwest corner, hereafter referred to as the ‘Site’ was excluded from the geotechnical investigation by the Client. The purpose of the desktop study was to review the available factual geotechnical, and geological information available for Site, and to provide a summary of the geotechnical subsurface soil and groundwater conditions anticipated to be encountered based thereon. The published factual geotechnical, and geological information was also interpreted to provide preliminary geotechnical design and construction recommendations for the proposed development for the planning stage works only. For detailed design, subsurface conditions must be confirmed through an intrusive geotechnical investigation.

Limitations associated with this report and its contents are provided in the statement included in **Appendix A**.

2.0 PROJECT AND SITE DESCRIPTION

It is understood that the Client may acquire the approximately 11.01 ha rectangular Site along with the approximately 29-hectares land abutting it on its east and south sides and develop it together as a subdivision comprised of low to medium density residential units, commercial and institutional facilities, and stormwater management (SWM) ponds serviced by paved roads, and underground and possibly overhead utilities.

The Site has a few agricultural single-family commercial, and residential structures along Townline road and Port Davidson Road (**Photo 1C to Photo 7C**), which form the norther and western Site boundaries, respectively. It was observed that a gas easement owned by Westover Express Pipeline Ltd. (WEX) is marked on the western boundary of the Site at 2483 Port Davidson Road, and likely continues east into the Site (**Photo 4C**). A review of the aerial photos available on the Niagara Navigator dating back to 1934 show that the Site has been under agricultural usage since year 1934. The Site is generally flat with ground elevation ranging from 187 m along its southern limits to 190 m in its northern portion with the higher contour generally in the middle of the Site.

The conceptual development plan spread over the Site and the adjacent 4-sided polygon shaped 33-hectare land shows that the development will comprise low to medium density residential units, commercial and institutional facilities, stormwater management pond (SWM), serviced by paved roads, and underground and possibly overhead utilities. The conceptual development plan is shown on **Drawing No. 2 – Conceptual Development Plan**.

3.0 REGIONAL GEOLOGY

The Site is located in the approximately 3,500 km^2 physiographic region of the Haldimand Clay Plain, which occupies the Niagara Peninsula between the Niagara Escarpment and Lake Erie. It is bounded by the Niagara River in the east and extends past Highway 6 connecting Hamilton on the Niagara Escarpment to Port Dover on the Lake Erie. The plain was submerged in the waters of the proglacial lakes and is covered with deep water low-permeability glaciolacustrine deposits comprised of silts and clays interspersed by morainic ridges generally in the north along the Niagara Escarpment.

The Quaternary geology map shows that the general Site area is covered with glaciolacustrine silt and clay deposits associated with pro-glacial lakes. These deposits occur as interstratified layers. The glaciolacustrine deposits generally thicken in a north to south direction becoming approximately 20 m thick in the Welland-Caistor Centre corridor, then again decreasing in thickness up to the Onondaga Escarpment. The glaciolacustrine silts sand clays have been described as poorly draining with low infiltration rates, resulting in surface ponding in poor drainage areas and moisture from surface infiltration occupying upper subsoil horizons for lengthy periods of time. Although soils have high water holding capacities, can be droughty during dry periods because of their inability to release sufficient moisture for plant use. With change in moisture these types of heavy clays can undergo significant volume changes, which manifests itself as expansion and desiccation during wet and dry weather conditions, respectively. As the footings are typically placed at depths exceeding 1.2 m for protection against frost, swelling clays are not a significant geotechnical issue, although there are instances where structures have developed wall cracks due to this very issue.

The dolostone bedrock at the Site is a part of the Guelph Formation. According to the bedrock topography map[1], the bedrock elevation ranges from 184 to 181 m across the site, sloping up from south to north, which indicates that the bedrock is approximately 7 m to 10 m below the existing grades across the site.

The logs of water wells in the general area shows that the groundwater is present within the dolostone bedrock. There are indications that the groundwater in the bedrock aquifer could be under 2 to 3 m of artesian pressure.

4.0 STRATIGRAPHIC SUMMARY

The predominant soils at the Site are likely to be the glaciolacustrine heavy clays interstratified with silt and sand seams. The following soil layers, in an increasing order of depth, can be expected to be encountered at the Site:

- Topsoil: The Site is a farmland, and therefore a surficial 0.3 to 1.1 m thick organic-heavy topsoil layer is expected across the Site.
- Brown Clay Crust: A 2 to 3 m thick desiccation-induced crust with a consistency of generally very stiff brown clays.

[1] Feenstra, B.H. (1981): Bedrock Topography of the Grimsby Area, Southern Ontario; Ontario geological Survey Preliminary Map P.2401, Scale 1:50,000.

- Grey Clay: Colloquially known as the 'Blue' clay, the clay underlying the crust is in its nonweathered state. It can be expected to be generally in a firm to stiff state and may continue down to the bedrock.
- Bedrock: Dolostone bedrock of the Guelph Formation is generally of high compressive strength (100MPa or more). Depth to the bedrock across the site can be expected to vary from 7 m to 10 m below the existing grades, with local variations due to troughs and ridges in the bedrock surface due to the potential karstic processes.
- Groundwater: The phreatic surface is likely to be at the interface of brown and grey clays i.e., at a depth of 2 m to 3 m below the ground surface. Free groundwater conditions representing an aquifer are not anticipated within the clayey overburden. The free groundwater is present within the dolostone bedrock.

5.0 DISCUSSION AND RECOMMENDATIONS

The proposed residential-commercial-institutional development is expected to comprise one to two storey structures constructed with one level of basement or without a basement. The Site is generally flat with a relief of approximately 4 m across the 11.01-hecatre Site, as such no major cut and fill operations are expected except for the topsoil stripping, and utility installations.

Based on the review of the available information listed above, the following section presents preliminary geotechnical discussion and recommendations for the abovementioned site. However, these recommendations are only considered preliminary, and a detailed geotechnical investigation by means of drilling boreholes and test pits on site should be conducted on site to confirm soil stratigraphy and soil condition for the site. once site plans become available to provide recommendations based on the proposed additions.

5.1 PRELIMINARY FOUNDATION DESIGN - BEARING CAPACITY

The one-to two storey residential-commercial-institutional structures are expected to be relatively lightly loaded and can be supported on conventional shallow strip and spread footings. Clay crust material is usually well over consolidated, desiccated, fissured, oxidized, and with partially air-filled voids, if the water table is or recently has been below the base of the footing. As it has a high shear strength than the underlying clays, the property which governs the allowable footing pressures is the deformation of the material which results in settlement of the footings. The deformation in the crust usually takes place rapidly upon loading and is composed of immediate deformation due to closing of fissures, compression of air voids, and possibly some porewater drainage.

Conventional spread footings when supported within the native undisturbed very stiff clay crust at a depth of 1 m below the existing grades may be designed for the Serviceability Limit State (SLS) geotechnical soil resistance of 150 kPa to limit the total allowable settlement to 25 mm and factored (Φ=0.5) at Ultimate Limit State (ULS) geotechnical soil resistance value of 225 kPa.

The footings' subgrade must be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost.

5.1.1 Slab-on-Grade

The native soils in their undisturbed state will provide adequate support to typical commercial slabs-on-grade when supported on a minimum 200 mm OPSS Granular 'A base course-compacted to at least 100% Standard Proctor Maximum Dry Density (SPMDD).

The basement floors below the groundwater table may have to be designed as structural slabs or provided with sub-floor drainage in addition to the perimeter subject to the findings of the detailed hydrogeological study.

5.1.2 Seismic Site Class

The seismic Site Class value, as defined in Section 4.1.8.4 of the 2012 Ontario Building Code (OBC), contains a seismic analysis and design methodology which uses a seismic site response and site classification system defined by the average shear stiffness of the upper thirty (30) meters of the ground below the foundation level. Based on the findings of the geotechnical desktop study, owing to the relatively shallow bedrock depth, a Seismic Site Class ‘D’ (stiff soil) can be considered for this Site for planning purposes.

For detailed design, it is recommended that geophysical methods such as multichannel analysis of surface waves (MASW) method, a seismic surface wave method for shear wave velocity evaluation, should be used for determining the Seismic Site Class.

5.1.3 Excavation and dewatering

Temporary excavations for the proposed development must be conducted in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). The native soil classification for excavation as per the following:

Table 5.1: Soil Types as per OHSA

Soil Type	Soil Type
Native Silty Clay	3

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the regulation requirements. The regulation stipulates safe excavation slopes by soil type as per table 6.2.

Table 5.2: Excavation Slopes for Each Soil Type as per OHSA

Soil Type	Base of Slope	Slope inclination
1	Within 1.2 meters of bottom of excavation	1 H:1V
2	Within 1.2 meters of bottom of excavation	1 H:1V
3	From Bottom of excavation	1 H:1V
4	From Bottom of excavation	3 H:1V

Any soft/loose soils or soils encountered below a free groundwater table should be classified as Type 4 soil. The maximum excavation side slope for a Type 4 soil is 3:1 (Horizontal: Vertical) in accordance with the OHSA regulation.

Stockpiling of any materials adjacent to excavations should be avoided. Similarly, traffic should not be permitted in proximity to open excavations. For this purpose, it is recommended that all storage of materials and traffic be restricted from a 3 m wide strip around the excavations, measured from the crest of the excavation designed and constructed in accordance with the OH&S Act.

If space is restricted such that the side slope cannot be safely cut back in accordance with the OH&S Act & Regulations, if sloughing and cave-in are encountered in the excavations, or if the excavations are to remain open for a longer period, an engineered shoring system should be used for the approximately up to 7 m deep bulk excavation for the proposed below grade levels. To avoid tiebacks, a soldier pile-lagging system would be suitable for up to 7 m deep excavation.

Based on the information revealed during the investigation, it is considered that conventional sump pumping should be applicable to control localized seepage that may occur for an excavation into the clayey fill soils up to 6 to 7 m deep.

5.2 SHORING SYSTEM – DESIGN PARAMETERS

The design of the shoring system should be the contractor's responsibility and should be designed by a licensed professional engineer. For the design, unfactored soil design parameters for foundation walls are listed in Table 7.3.

Table 5.3: Unfactored Soil Design Parameters for Foundation Walls

Parameter	Clay Crust	Grey Clay
Bulk Unit Weight, γ (kN/m^3)	20.0	20.0
Effective Friction Angle, ϕ'	32.0	27.0
At Rest Earth Pressure (K_o) (Static)	0.47	0.55
Active Earth Pressure (K_A) (Static)	0.31	0.38
Passive Earth Pressure, (K_P)	3.25	2.66

5.3 SITE SERVICING

The predominant subgrade soils beneath the service pipes will consist of firm to stiff clays, which would provide suitable supports to the proposed service utility pipes. If any very loose or soft areas are detected during inspection, they should be excavated and replaced with compacted granular material such as OPSS.MUNI 1010 Granular A or Granular B Type II.

The pipe bedding for the services should be conventional Class B pipe bedding comprising a minimum 150 mm thick layer of OPSS.MUNI 1010 Granular ‘A’ aggregate below the pipe invert. The bedding course may be thickened if portions of the subgrade become wet during excavation. OPSS.MUNI 1010 Granular A type aggregate should be provided around the pipe to at least 300 mm above the top, and the bedding should be compacted to 98% SPMDD. Service lines installed outside of heated areas should be provided with a minimum 1.2 m of soil cover or equivalent insulation for frost protection.

5.4 TRENCH BACKFILL

Bedding for services should consist of OPSS Granular ‘A’ material. In general, a minimum of 150 mm of bedding and 300 mm of cover material is recommended.

The bedding and cover material should be compacted to achieve a minimum of 98% of the material’s Standard Proctor Maximum Dry Density (SPMDD).

The bedding and cover on each side of the pipe should be completed simultaneously and at no time should the difference from one side of the pipe to the other exceed 200 mm.

These recommendations should be confirmed with the pipe manufacturer and care must be taken to avoid incurring damage to the services. Pipe manufactures may have additional/alternative requirements that should be reviewed by the Designer and Contractor prior to installation of the services.

The trenches above the specified pipe bedding should be backfilled with inorganic soils that are not excessively wet placed in 200 mm thick lifts and compacted to at least 98% SPMDD. Where the service trenches enter the building, the trench backfill must be compacted as structural fill to a minimum of 100% SPMDD. Any trench backfill below a pavement structure should be compacted to 100% SPMDD within 1 m from the top of subgrade level. Based on the results of in-situ moisture content tests conducted on the native overburden deposits, the materials may be suitable for reuse as trench backfill. Any overly wet material may require drying prior to reusing as backfill.

6.0 PAVEMENT DESIGN

6.1 SUBGRADE PREPARATION

The proposed residential-commercial-institutional development will have roads service all three types of facilities. Based on the geological literature, the pavement subgrade will likely consist of very stiff clays after stripping of the topsoil. A subgrade comprised of very stiff clays is considered suitable to support pavements subject to approval by a geotechnical engineer based on results of proof-rolling.

If new fill is required to raise the grade, selected on-site fill could be used, provided it is free of any organic material. The fill should be placed in large areas where it can be compacted by a heavy sheep-foot type roller. Any fill placed to increase or level the grade must be compacted to a minimum 98 percent SPMDD in lifts not exceeding 200 mm. In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is also recommended.

The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of sub-base fills, restricted construction lanes, and half-loads during paving may be required, especially if construction is conducted during wet weather conditions.

6.2 DRAINAGE

Control of surface water is a crucial factor in achieving a good pavement life. The subgrade must be free of depressions and sloped (preferably at a minimum grade of 3 percent) to provide effective drainage toward subgrade drains. Grading adjacent to parking area should be designed to ensure that water is not allowed to pond adjacent to the outside edges.

Continuous edge subdrains should be provided and connected to catch basins to facilitate drainage of granular materials. The subdrain invert should be maintained at least 0.3 metres below subgrade level. To minimize the problems of differential movement between the pavement and catch basins/manhole due to frost action, the backfill around the structures should consist of free-draining granular material.

6.3 RECOMMENDED PAVEMENT STRUCTURE

The following asphaltic concrete and granular pavement thickness may be used for the design of the potential driveways and parking areas. The pavement designs include a Heavy Duty for access routes and a Standard Duty for car parking areas and are based on providing a maximum design life of 20 years.

Table 6.1: Minimum Component Thickness - Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty Pavement Design (Main Roads)	Light Duty Pavement Design (Local Roads)
Surface Course Asphaltic Concrete HL3 HS (OPSS 1150)	97% Maximum Relative Density (OPSS 310)	50 mm	50 mm
Base Course Asphaltic Concrete HL8 HS (OPSS 1150)	97% Maximum Relative Density (OPSS 310)	75 mm	50 mm
Base Course: 19mm Crusher Run	100% Standard Proctor Maximum Dry Density (ASTM-D698)	380 mm	325 mm

If pavement construction occurs in wet inclement weather, it may be necessary to provide additional subgrade support for construction traffic by increasing the thickness of the granular subbase.

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in **Appendix A**. It is the responsibility of Lockbridge Development Inc., who is identified as the “Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report;
- Basis of the report;
- Standard of care;
- Interpretation of site conditions;
- Varying or unexpected site conditions; and,
- Planning, design or construction.

Yours truly,

STANTEC CONSULTING LTD.

APPENDIX A

A.1 STATEMENT OF GENERAL CONDITIONS

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This professional work product ("hereinafter referred to as the Report") has been prepared for the sole benefit of the Client in accordance with Stantec's contract with the Client. While the Report may be provided by the Client to applicable authorities having jurisdiction and to other third parties in connection with the project, Stantec disclaims any legal duty based upon warranty, reliance, or any other theory to any third party, and will not be liable to such third party for any damages or losses of any kind that may result.

BASIS OF THIS REPORT: This Report relates solely to the site-specific project for which Stantec was retained and the stated purpose for which the Report was prepared. The information, opinions, conclusions and/or recommendations made in this Report are in accordance with Stantec's present understanding of the site-specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time the scope of work was conducted and do not take into account any subsequent changes. If the proposed site-specific project differs or is modified from what is described in this Report or if the site conditions are altered, this Report is no longer valid unless Stantec is requested by the Client to review and revise the Report to reflect the differing or modified project specifics and/or the altered site conditions. This Report is not to be used or relied on for any variation or extension of the project, or for any other project or purpose or site, and any unauthorized use or reliance is at the recipient's own risk.

STANDARD OF CARE: Preparation of this Report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

PROVIDED INFORMATION: Stantec has assumed all information received from the Client and third parties in the preparation of this Report to be correct. While Stantec has exercised a customary level of judgment or due diligence in the use of such information, Stantec assumes no responsibility for the consequences of any error or omission contained therein.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this Report are based on site conditions encountered by Stantec at the time of the scope of work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behaviour. Extrapolation of in-situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

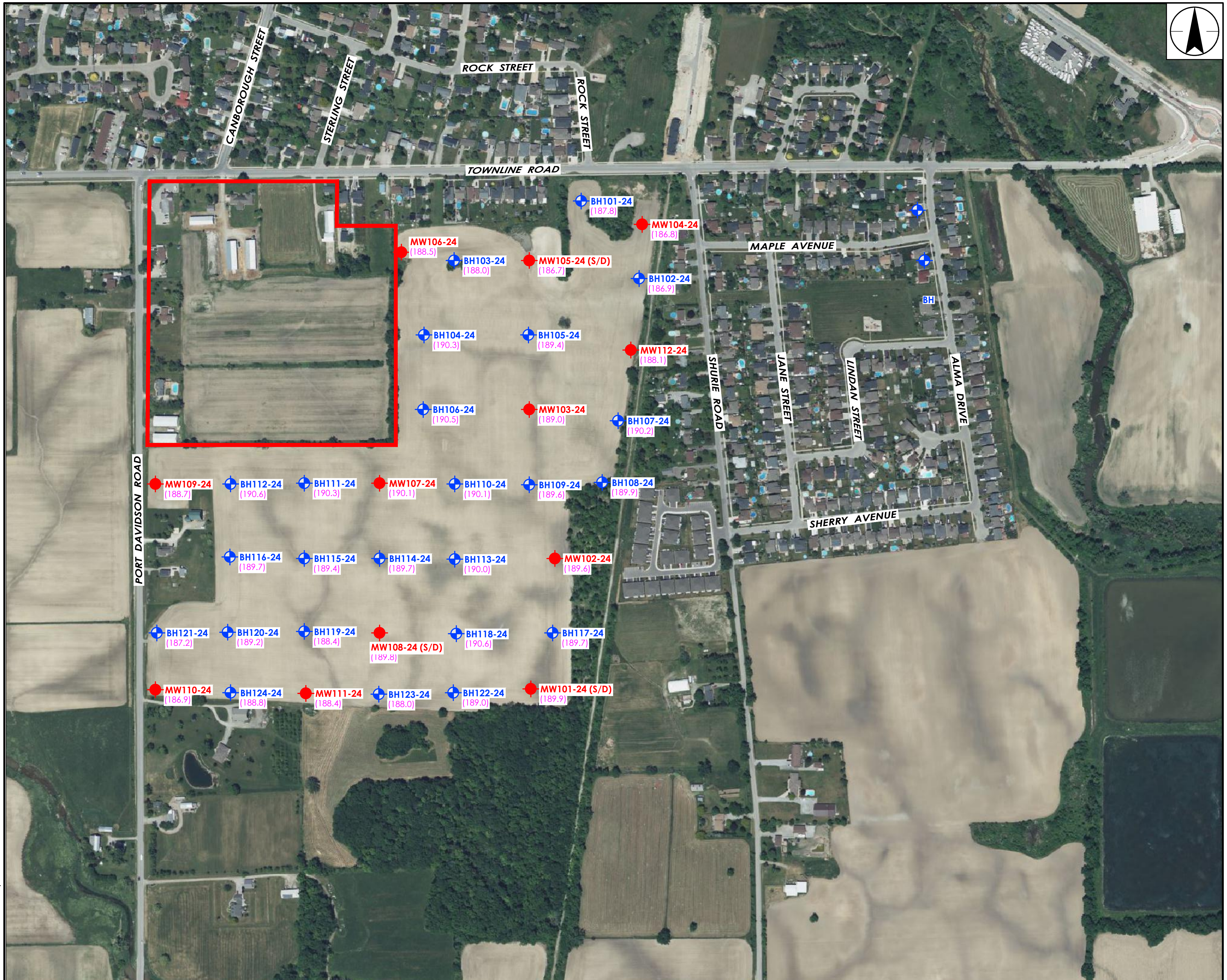
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this Report or encountered at the test and/or sample locations, Stantec must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the Report conclusions or recommendations are required. Stantec will not be responsible to any party for damages incurred as a result of failing to notify Stantec that differing site or subsurface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec geotechnical engineers, sufficiently ahead of initiating the next project stage (e.g., property acquisition, tender, construction, etc.), to confirm that this Report completely addresses the elaborated project specifics and that the contents of this Report have been properly interpreted. Specialty quality assurance services (e.g., field observations and testing) during construction are a necessary part of the evaluation of subsurface conditions and site work. Site work relating to the recommendations included in this Report should only be carried out in the presence of a qualified geotechnical engineer; Stantec cannot be responsible for site work carried out without being present.

APPENDIX B

B.1 SITE LOCATION PLAN

B.2 CONCEPTUAL DEVELOPMENT



Stantec

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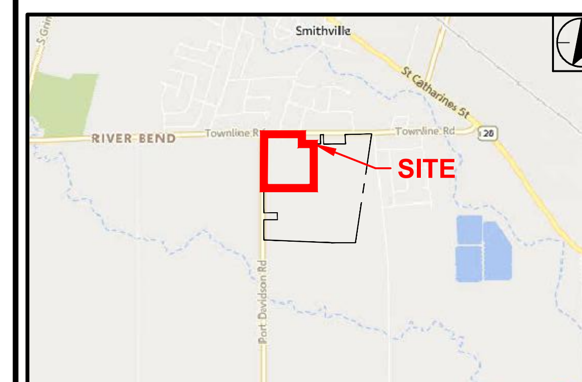
LEGEND

- BOREHOLE (STANTEC, 2024)
- MONITORING WELL (STANTEC, 2024)
- (186.9) BOREHOLE ELEVATION (METERS)
- APPROXIMATE BOUNDARY OF DESKTOP STUDY



KEY PLAN 1 : 50 000

NOTES

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 17.
2. IMAGERY:IMAGERY: © 2024 MICROSOFT CORPORATION © 2024 MAXAR © CNES (2024) DISTRIBUTION AIRBUS DS.

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MARCH 2024
Project No. 161414473

Client/Project
LOCKBRIDGE DEVELOPMENT INC
DESKTOP STUDY
BLOCK 9, SMITHVILLE, ONTARIO

Drawing No.
1

Title
SITE PLAN

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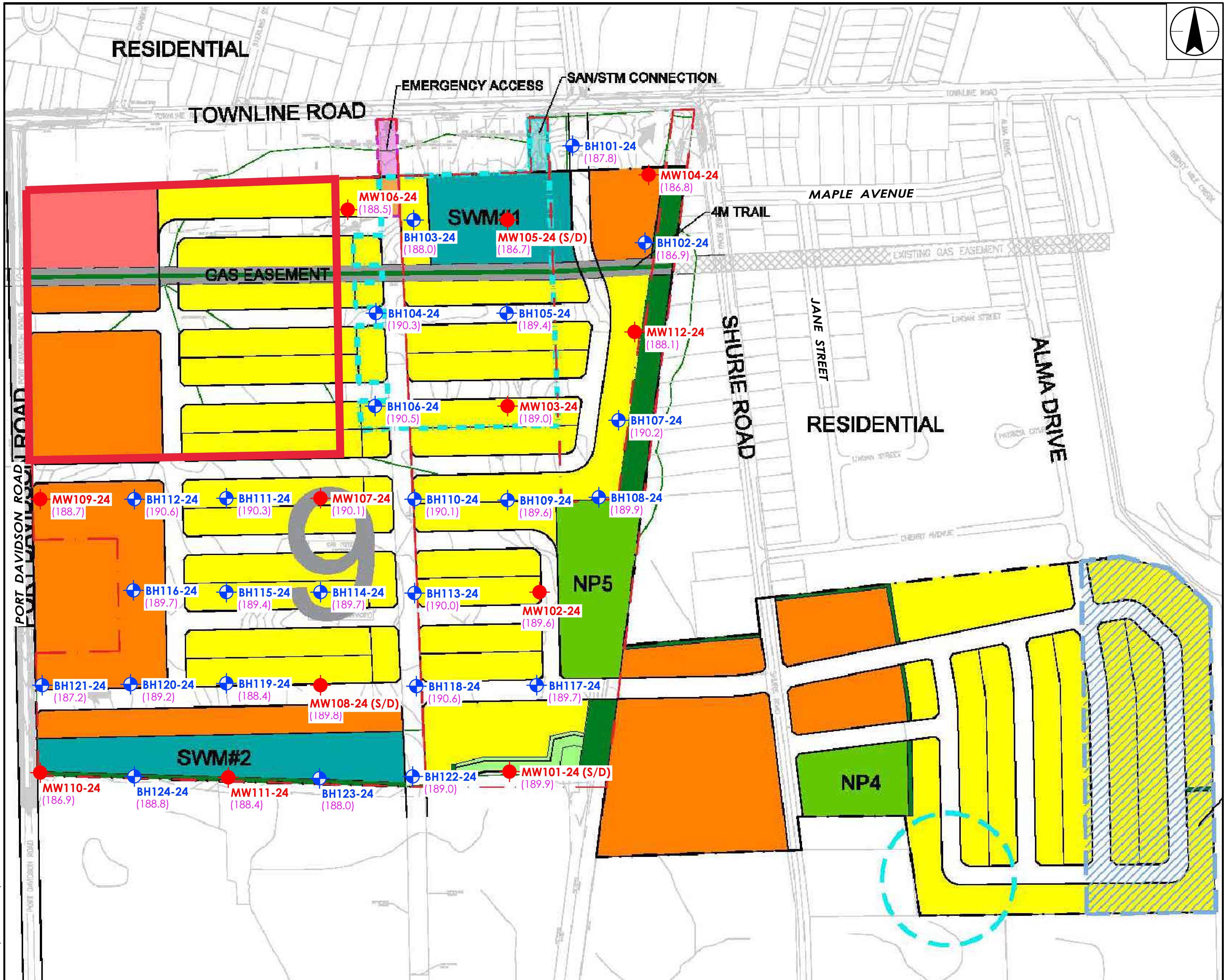
Printed: Mar 22, 2024 By: G. Briones



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Printed: Apr 05, 2024 By: G. Briones

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LEGEND

- BOREHOLE (STANTEC, 2024) MONITORING WELL
- (STANTEC, 2024)
- (186.9) BOREHOLE ELEVATION (METERS)
- APPROXIMATE BOUNDARY OF DESKTOP STUDY
- BLOCK PLAN AREA 9
- LOW DENSITY RESIDENTIAL
- MEDIUM DENSITY RESIDENTIAL
- COMMERCIAL
- PARK LAND / OPEN SPACE / NP4, NP5
- NATURAL HERITAGE SYSTEMS (NHS) - TRAILS
- NATURAL FEATURES AND 15m BUFFER
- PROPOSED S.W.M. FACILITY
- PHASE 1
- S.W.M. LOCATION
- LAGOON BUFFER

KEY PLAN 1 : 50 000

NOTES

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 17.
2. IMAGERY:IMAGERY: © 2024 MICROSOFT CORPORATION © 2024 MAXAR © CNES (2024) DISTRIBUTION AIRBUS DS.

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APRIL 2024
Project No. 161414473

Client/Project
LOCKBRIDGE DEVELOPMENT INC
DESKTOP STUDY
BLOCK 9, SMITHVILLE, ONTARIO

Drawing No.
2

Title
CONCEPTUAL DEVELOPMENT PLAN

APPENDIX C

C.1 SITE PHOTOS

Photo No. 1C: Photo of an existing facility (Smithville Animal Hospital) located on the intersection of Townline Road and Port Davidson Road, Looking East

Photo No. 2C: The western boundary of the site along Port Davidson Road, looking east

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Photo No. 3C:	Along the Northern Boundary of the site, looking South

Photo No. 4C:	Gas easement along the Western boundary of the site, the alignment of the gas easement heads East into the site

Photo No. 5C:	Along the Southern boundary of the site, looking North

Photo No. 6C:	Along the eastern boundary of the site, looking west

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Photo No. 7C:	Along the eastern boundary of the site, looking west