April 10, 2019
File: GE-00237

Mr. Julian Novick
Wastell Homes
5-1895 Blue Heron Drive
London, Ontario N6H 5L9

Reference: **RESIDENTIAL SUBDIVISION**
EAST STREET & DEXTER LINE
PORT STANLEY, ONTARIO

1.0 Introduction

It is understood that the site located northwest of the intersection of East Street and Dexter Line will be developed as a residential subdivision, accessed via a local road network connected to East Street and Beamish Street.

To assist in the design and construction of the proposed residential development, LDS has carried out a field program of test holes to characterize the soil and shallow groundwater conditions at the site. This letter provides a summary of the information collected onsite, and recommendations.

1.1 Field Program

On March 27, 2019, LDS staff advanced a three shallow test holes at the site using a backhoe operated by a local excavation contractor, to assess the soil and shallow groundwater conditions at the site. The test pits (denoted as TP1 to TP3) were advanced throughout the site, at the locations shown on the attached Site Plan. The test holes were advanced to depths of 3.9 to 4.5 m below grade.

Ground surface elevations at the test hole and borehole locations were surveyed by LDS using a Trimble R10 GPS rover, and the locations are shown on the attached plan.

Location	Easting, m E	Northing, m N	Ground Surface Elevation (m)
TP1	483339.13	4725324.83	214.991
TP2	483115.78	4725236.77	214.138
TP3	483055.36	4725302.23	214.029

1.2 Summarized Observations

Topsoil was contacted at ground surface in all of the test pits. Below the topsoil, natural silt and silt till soils were encountered within each test pit, extending throughout excavation depths. Details are available on the attached logs, and summarized in the following table:

Location	Soil Description		
	Topsoil	Silt	Silt Till
TP1	0.00 – 0.45 m	0.45 – 1.52 m	1.52 – 4.72 m
TP2	0.00 – 0.45 m	0.45 – 1.52 m	1.52 – 3.96 m
TP3	0.00 – 0.30 m	0.30 – 1.98 m	1.98 – 4.57 m

Within test pit TP3, a wet sand seam was encountered from 2.4 to 2.6 m below ground surface, and contributed minor sidewall caving and groundwater seepage into the open test pit. Test pits TP1 and TP2 remained open and dry throughout exploration depths.

Generally, the silt and silt till soils were found to be in a stiff to very stiff and moist state (by visual / tactile observations, and observed excavator resistance).

1.3 Local Well Record Review

A review of local well records available through the Ministry of Environment, Conservation, and Parks (MECP) for this area was carried out to review the water levels recorded in the nearby wells. Drawing 2 in Appendix C shows the location of the wells (with corresponding Well Registration No.) which are in close proximity to the site. The water supply wells are summarized in Appendix C, for reference.

The water supply wells are generally set into deep overburden or bedrock aquifers, all of which are 20 to 60 metres below existing grade.

2.0 Geotechnical Comments and Recommendations

Current layout plans for the proposed residential development are being prepared for the site, and is expected to include a condominium development accessed with a private roadway and serviced with municipal water and sewer services.

2.1 Site Preparation

It is expected that some site grading activities will be required, particularly where there are low areas present. Topsoil stripping and brush clearing is anticipated throughout the area to be developed. In general, this is expected to require the removal of about 300 to 450 mm of surficial topsoil. Thicker topsoil areas may be present in proximity to existing wooded areas, and where local depressions are present at the site.

Surficial topsoil may be stockpiled on site for possible re-use as landscaping fill. In the event that material is disposed of offsite, testing of the material for transport should conform to Ministry of Environment Conservation and Parks (MECP) Guidelines and requirements.

Where exposed subgrade soils are approved by the geotechnical consultant, and grades need to be raised to reach design elevations, it is anticipated that grades will be restored using structural / engineered fill. Engineered fill should consist of suitable, compactable, inorganic soils, which are free of topsoil, organics and miscellaneous debris. For best compaction results, the fill material should have a moisture content within about 3 percent of optimum, as determined by Standard Proctor testing.

Placement of engineered fill at the site should be monitored by the geotechnical consultant to verify that suitable materials are used, and to confirm that suitable levels of compaction are achieved. Engineered fill material should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent

Standard Proctor Maximum Dry Density (SPMDD). Additional notes regarding engineered fill placement are provided in Appendix A.

2.2 Excavations

Once the cut-fill activities are complete, excavations for the proposed buildings and site services are generally expected to extend through and will terminate within engineered fill material or the natural subgrade soils.

All work associated with design and construction relative to excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). Based on the results of the geotechnical investigation and in accordance with Section 226 of Ontario Regulation 213/91, the natural silt and silt till soils are classified at Type 2 soil, temporary excavation side slope may be cut vertical up to 1.2 m from the base of the excavation and must be cut back at a maximum inclination of 1H:1V above that point. Should saturated or saturated soils be encountered they would be classified as Type 3 soil. For excavations, which extend through or terminate in Type 3 soil, temporary excavation side slopes must be cut back at a maximum inclination of 1H:1V from the base of the excavation.

In the event that construction occurs in seasonally wet conditions or when frozen soil conditions are present, care will be required to maintain safe excavation side slopes, and suitable excavation bases. The contractor should use a reasonable effort to direct surface run-off away from open excavations.

2.3 Groundwater Control

Based on the results of the shallow test pit program and a review of MECP Well Records in the area of the site, it is not anticipated that shallow groundwater will be encountered. However, some limited seepage was encountered within test pits TP3 at a depth of 2.4 and may be anticipated in proximity to this test pit location.

Conventional groundwater control methods are expected to be suitable for excavations at the site, to address surface water infiltration and shallow groundwater seepage for excavations which do not extend below the stabilized groundwater table.

Although not anticipated for this project, projects requiring positive groundwater control with a removal rate in excess of 50,000 liters per day require a Permit to Take Water (PTTW) or a submission to the Environmental Activity and Sector Registry (EASR). PTTW and EASR applications are submitted to the Ministry of Environment according to Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and the Water Taking and Transfer Regulation O. Reg. 387/04 for their review and approval.

2.4 Foundation Design

For design of footings on the natural subgrade soils 1.2 m below existing grades or supported on engineered fill, the following allowable bearing pressures (net stress increase) can be used for design of footings:

- Serviceability Limit States (SLS) 145 kPa (~3000 psf)
- Ultimate Limit States (ULS) 190 kPa (~4000 psf)

All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m (4 feet) of soil cover or equivalent insulation.

Footings at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the closest edge of the lower footing. It is important that servicing excavations which encroach on the building foundations are checked to ensure that they do not undermine the building foundations.

Verification of footing base conditions should be undertaken by the geotechnical engineer at the time of excavation. Provided that the stability of the soils exposed at the founding level is not compromised as a result of construction activity, precipitation, cold weather conditions, etc., and the design bearing pressures are not exceeded, the total and differential settlements of footings are expected to be less than 25 mm and 19 mm, respectively.

It should be noted that the recommended bearing capacities have been calculated based on observations of the soil and groundwater conditions within the test pit program at the site. Where variations occur between the test pit locations, and during construction of the new building, site verification by LDS' geotechnical engineer is recommended to confirm soil conditions and verify soil bearing capacity.

2.5 Basement Construction

Residential buildings with basements at this site are not expected to encounter the stabilized groundwater table. The basement floors can be constructed using cast slab-on-grade techniques provided that the subgrade is stripped of unsuitable material. It is recommended that a minimum 200 mm (8 inch) thick compacted layer of 19 mm (¾ inch) clear stone be placed between the prepared subgrade and the floor slab to serve as a moisture barrier.

The portion of exterior basement walls below finished groundwater level should be damp-proofed and designed to resist a horizontal earth pressure 'P' at any depth 'h' below the surface as given by the following expression:

$$P = K (\gamma h + q)$$

where,
P = lateral earth pressure in kPa acting at depth h;
γ = natural unit weight, a value of 19.5 kN/m^3 may be assumed;
h = depth of point of interest in m;
q = equivalent value of any surcharge on the ground surface in kPa.
K = earth pressure coefficient, assumed to be 0.4

The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall.

In general, the excavated soils from the building footprints, which are free of topsoil and organics are generally expected to be suitable for re-use as foundation wall backfill. Some soil conditioning may be required in wet or winter conditions.

2.6 Site Services

Depending on final design grades, the subgrade soils within site serving trenches (constructed at conventional depths) are generally expected to comprise of natural subgrade soils. Although no bearing problems are anticipated for flexible or rigid pipes founded on natural deposits or approved fill, localized base improvement along the trench bottom may be required for excavations which terminate in wet subgrade soils. The extent of base improvement or stabilization is best determined in the field during construction, with consultation from a geotechnical engineer.

For services supported on the native deposits or engineered fill, the bedding should conform to OPS Standards. The bedding aggregate should be compacted to a minimum 95 percent SPMDD. Water and sewer lines installed outside of heated areas should be provided with a minimum 1.2 m of soil cover for frost protection.

A well graded stone layer may be used in the service trenches as bedding below the spring line of the pipe if necessary to provide stabilization to the excavation base in wet subgrade soils, where encountered. The use of geotextile may be considered for wrapping the pipe and limit the movement of fines from the surrounding soils into the bedding material. The potential locations for use of stone bedding can be identified through site inspection during construction, and will vary across the site due to seasonal conditions and variations in perched groundwater conditions.

Requirements for backfill in service trenches, etc. should also conform to Municipal and OPS Standards. A program of in-situ density testing should be set up to ensure that satisfactory levels of compaction are achieved. The inorganic material excavated from the service trenches may be used for construction backfill provided that reasonable care is exercised in handling the material. In this regard, the material should be within 3 percent of the optimum moisture as determined by the Standard Proctor density test. Stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet, adverse weather.

Soils excavated from below the stabilized groundwater table may be too wet for re-use as backfill, unless adequate time is allowed for drying, or if the material is blended with approved dry fill; otherwise, it may be suitable for reuse as landscape fill.

Backfill above the bedding aggregate can consist of the excavated (inorganic) soils, compacted in maximum 300 mm thick lifts to a minimum of 95 percent SPMDD. A program of in-situ density testing should be set up to ensure that satisfactory levels of compaction are achieved.

2.7 Pavement Design

The site is expected to be accessed from local roads connecting to East Street and Beamish Street. The exposed subgrade soils within the roadways are expected to be comprised of re-compacted soils comprised of natural silt. The road subgrade should be thoroughly proof-rolled and reviewed by the geotechnical consultant. In the event that loose or soft areas are noted, additional work may be required to sub excavate and replace unstable soils with suitable compactable material. In general terms, subgrade soils supporting site pavements should be compacted to a minimum level of 98 percent SPMDD.

The recommended pavement structure provided in this report is based on the natural subgrade soils encountered in the boreholes or suitably re-compacted soils, as described previously. Provided that the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated subgrade conditions and traffic loading on the internal network of local roads.

Pavement Component for Local Roads	Minimum Thickness	Compaction Requirements
Asphaltic Concrete	40 mm HL 3, 50 mm HL 8	92.0 – 96.5 % MRD

Pavement Component for Local Roads	Minimum Thickness	Compaction Requirements
Granular 'A' Base	150 mm	100% SPMDD
Granular 'B' Subbase	300 mm	100% SPMDD

Other granular configurations may be possible provided the granular base equivalency (GBE) thickness is maintained. These recommendations on thickness design are not intended to support heavy and concentrated construction traffic, particularly where only a portion of the pavement section is installed. If frequent construction traffic is anticipated while only a portion of the site pavements are in place, or if construction is undertaken in poor weather conditions, thickening of the granular subbase may be appropriate and can be reviewed during construction, by the geotechnical consultant.

Where local roads connect to existing pavements, subgrade levels and pavement components should be tapered to match / tie-into existing pavement structures to minimize differential settlements at the transition from existing to new pavement.

It is recommended that a program of inspection and materials testing (including laboratory analyses and compaction testing) be carried out during construction to confirm that geotechnical requirements are satisfied.

- Samples of both the Granular 'A' and Granular 'B' aggregates should be checked for conformance to OPSS 1010 prior to use on site, and during construction.
- The asphaltic concrete paving materials should conform to the requirements of OPSS 1150. The asphalt should be placed in accordance with OPSS 310.
- Specified compaction levels are identified in the table, above.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

3.0 Closing

We trust this satisfies your present requirements. If you have any questions or require anything further, please feel free to contact our office.

Respectfully,

LDS CONSULTANTS INC.

LICENSED PROFESSIONAL ENGINEER
Apr 10, 2019
R.A. WALKER
PROVINCE OF ONTARIO

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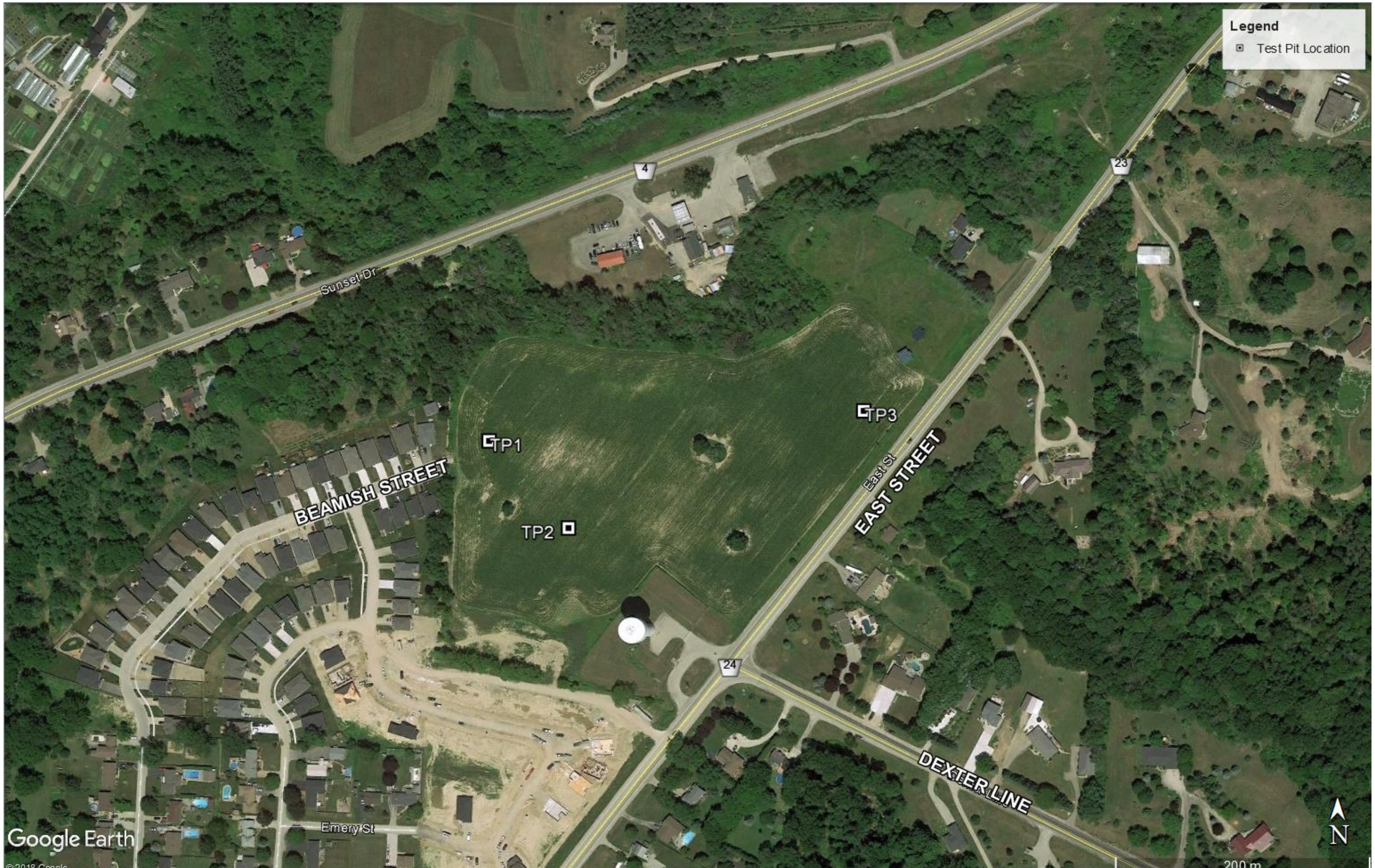
Attachment:

Drawing 1: Test Pit Location Plan
Test Pit Summary



SOURCE:
Google Earth Pro, Version 7.1.8.3036, Coordinates 17T 482728 m E 4725325m N, Image date 7/2/2018

LDS

PROJECT NAME
Wastell Homes
Proposed Subdivision

PROJECT LOCATION
East Street & Dexter Line
Port Stanley

DRAWING NAME
Test Pit Location Plan

SCALE	PROJECT NO.
As Shown	GE-00237
DATE	**DRAWING NO.**
April 2019	1

Test Pit Summary – East St, Port Stanley
March 27, 2019

TP1

0.00 – 0.45 m	Topsoil – dark brown, silty loam
0.45 – 1.52 m	Silt – brown, weathered, trace fine gravel, trace fine sand, stiff, moist
1.52 – 4.72 m	Silt Till - brown, stiff, moist
	- becoming grey below 2.13 m
	- becoming very stiff below 2.74m
4.72 m	*TP Terminated – open and dry upon completion.*

TP2

0.00 – 0.45 m	Topsoil – dark brown, silty loam
0.45 – 1.52 m	Silt – brown-grey, mottled, weathered, stiff, moist to damp
1.52 – 3.96 m	Silt Till - brown, stiff, moist
	- becoming grey and very stiff below 2.44 m
3.96 m	*TP Terminated – open and dry upon completion.*

TP3

0.00 – 0.30 m	Topsoil – dark brown, silty loam
0.30 – 1.98 m	Silt – brown, weathered, trace fine gravel, trace fine sand, stiff, moist
1.98 – 4.57 m	Silt Till - brown, stiff, moist
	- sand seam observed at 2.44 m
	- becoming very stiff below 2.59 m
	- becoming grey below 3.05 m
4.57 m	*TP Terminated – open upon completion*
	minor seepage and caving encountered at 2.43 m