APPENDIX F

GEOTECHNICAL INVESTIGATION REPORT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

GEOTECHNICAL INVESTIGATION
BRIDGE 24 TOWNSHIP ROAD 12 OVER NITH RIVER
TOWNSHIP OF BLANDFORD-BLENHEIM, ONTARIO

for

TOWNSHIP OF BLANDFORD-BLENHEIM
c/o K. SMART ASSOCIATES LIMITED

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PML Ref.: 23KF019
Report: 1
August 19, 2026

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Report: 1

Township of Blandford-Blenheim
c/o Mr. Trevor Hoard, C.E.T.
K. Smart Associates Limited
85 McIntyre Drive
Kitchener, Ontario

Dear Mr. Hoard

Geotechnical Investigation
Bridge 24, Township Road 12 Over Nith River
Township of Blandford-Blenheim, Ontario

Peto MacCallum Ltd (PML) is pleased to present the results of the Geotechnical investigation completed at the above noted project site. Authorization for the work was provided in an email from Mr. T. Hoard of K. Smart Associates Limited in an email dated May 31, 2023 and by Mr. J. Borton with a signed Engineering Services Agreement dated June 15, 2023.

In general, the project involves the planned replacement of Bridge 24 located on Township Road 12 over the Nith River between Oxford Road 3 and Blenheim Road in the Township of Blandford-Blenheim, County of Oxford, Ontario. The existing bridge is a single lane road bridge which comprises an approximately 45 m single-span steel truss type structure. Details of the proposed new bridge design or rehabilitation works have not been provided, but it is envisaged that the proposed new bridge design will maintain a similar span and grades at the site.

The purpose of this geotechnical investigation was to explore the subsurface soil and ground water conditions at the site. Based on the findings, we have prepared this engineering report with geotechnical design and construction recommendations for the bridge replacement, including shallow and deep foundations, engineered fill, excavations, groundwater control, and pavement design.

A total of four boreholes were advanced at the site to between 6.6 and 17.8 m depth. The general subsurface conditions comprised surficial pavement structure and fill, over buried topsoil, over sandy gravel / sand and gravel over silt till. One borehole was terminated in water bearing soil deposits under artesian pressure. The artesian ground water conditions will prohibit the installation of driven piles. As such, we are providing design recommendations for footings or caissons to support the new bridge.

We trust the information in this report is sufficient for your present purpose. If you have any questions, please do not hesitate to call our office.

Sincerely

Peto MacCallum Ltd.

William Loghrin, P.Eng.
Manager, Engineering Services

WL:cs

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1. INTRODUCTION

Peto MacCallum Ltd (PML) is pleased to present the results of the geotechnical investigation completed at the above noted project site. Authorization for the work was provided by Mr. T. Hoard of K. Smart Associates Limited on behalf of the Township of Blandford-Blenheim, in an email dated May 31, 2023 and a signed Engineering Services Agreement by Mr. J. Bolton on June 15, 2023.

A geotechnical investigation has been requested by the Township of Blandford-Blenheim as part of a Schedule "B" Municipal Class Environmental Assessment for the Bridge 24 located on Township Road 12 over the Nith River between Oxford Road 3 and Blenheim Road in the County of Oxford, Ontario. The existing bridge is a single lane bridge of approximately 45 m single-span steel truss supported on concrete abutments. Details of the proposed replacement bridge have not been provided, but it is envisaged that the proposed bridge will be widened to accommodate two lanes of traffic but maintain a similar span and grades at the site.

The purpose of this geotechnical investigation is to explore the subsurface soil and ground water conditions at the proposed site. Based on the findings, we have prepared this engineering report with geotechnical design and construction recommendations for the proposed bridge, including shallow and deep foundations, engineered fill, excavations, groundwater control, and pavement design.

A geoenvironmental investigation was also completed by PML as part of the project, and the results will be addressed under separate cover.

The comments and recommendations provided in this report are based on the site conditions at the time of the investigation and are applicable only to the proposed works as addressed in the report. Any changes in the proposed plans will require review by PML to re-assess the validity of the report, and may require modified recommendations, additional investigation and/or analysis.

This report is subject to the Statement of Limitations, which is included in Appendix A and must be read in conjunction with the report.

2. INVESTIGATION

The field work for the recent foundation investigation was carried out from June 20 to 26, 2023 and consisted of four boreholes. Two boreholes were to advanced to a depth of 6.6 m at the bridge approaches, and two deeper boreholes were advanced to 17.6 to 17.8 m at the bridge abutments. Reference is given to the attached Borehole Location Plan, Drawing 1 for particulars of the borehole locations.

The borehole locations were programmed through consultation with the client and established in the field by PML and surveyed by PML using a Sokkia GCX3 GNSS Receiver. The vertical and horizontal accuracies of this unit are 0.1 and 0.5 m, respectively. All elevations in this report are geodetic and expressed in metres.

Co-ordination for clearances of underground utilities was provided by PML. The boreholes were drilled cognizant of the underground and overhead utilities.

Traffic control was provided in accordance with Ontario Traffic Manual, Book 7, where required.

The boreholes were advanced using continuous flight hollow stem augers and diamond tipped casing, powered by a truck mounted CME-75 drill rig, equipped with an automatic hammer, supplied, and operated by a specialist drilling contractor, working under the full-time supervision of a member of PML's engineering staff.

The existing pavement component thicknesses were measured, and samples of the granular material were collected.

Representative samples of the subsurface soils were recovered at regular depth intervals for identification purposes using a conventional 51 mm OD split spoon sampler. The sampler excluded particles larger than 38 mm. Standard Penetration Tests (SPT), were carried out simultaneously with the sampling operations to assess the strength characteristics of the subsoil. The ground water conditions in the boreholes were assessed during drilling by visual examination of the soil samples, the sampler, and drill rods as the samples were retrieved, and measurement of the water level in the open boreholes, if any.

The boreholes were sealed and backfilled in accordance with O.Reg. 903 and capped with cold mix asphalt.

All recovered samples were returned to our laboratory for detailed examination and moisture content determinations. Grain size analyses were carried out on eight samples of the major soil units encountered with results presented on Figures 1 to 4, appended. Two of the recovered soil samples were submitted for Atterberg Limits tests, and two samples were also submitted for organic content determination using the ignition method, with results presented on the appended Log of Borehole sheets.

A limited chemical testing program was conducted concurrently with the geotechnical investigation, and the results of the chemical testing have been provided under separate cover.

3. SITE DESCRIPTION AND GEOLOGY

The site currently consists of a two-lane gravel road with paved approaches to the existing bridge. The current bridge is a 45 m, single-span steel truss structure, and is one traffic lane wide. The bridge spans over the Nith River, which lies about 4.4 m below the bridge road surface, approximate Elevation 290.7. The river is about 1 to 2 m deep. Two photographs of the site are appended.

3.1 Site Features, Physiography and Mapping

The site lies within a spillway region bordering kame moraines in a physiographic region known as the Oxford till plain (Chatman and Putnam, 1984). The physiographic landforms in the area are former glacial spillways and adjacent kame moraines.

The available Paleozoic information and bedrock stratigraphy is the area is presented on the Department of Mines Map 2254 and Map No. P.168. The bedrock underlying the Site is in the Salina Formation, which typically consists of limestone, dolostone, shale, sandstone, gypsum, salt from the upper Silurian era. Ontario Well Records were studied to check the bedrock depths near the site. Multiple wells within a 3 km radius of site noted bedrock at depth of 59 to 81 m with brown to grey limestone and occasional shale were noted in the well records.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions encountered in the boreholes, including: pavement component thicknesses, soil classifications and inferred stratigraphy and thicknesses, SPT "N" values, ground water levels, and the results of laboratory water content determinations and Atterberg Limits testing.

Due to the soil sampling procedures and the limited size of samples, the depth / elevation demarcations on the borehole logs must be viewed as "transitional" zones, and cannot be construed as exact geologic boundaries between layers. PML should be retained to assist in defining the geological boundaries in the field during construction, if required.

The subsurface soil stratigraphy encountered in Boreholes 1 to 4 advanced at the site comprised a surficial pavement structure over fill, over buried topsoil over sandy gravel / sand and gravel overlying a major silt till deposit which is interlayered with sand and gravel layers.

4.1 Subsurface Stratigraphy

4.1.1 Pavement Structure

A surficial pavement structure was encountered in Boreholes 1 to 4, and ranged between 0.6 and 1.0 m thickness. The pavement components comprised 25 to 40 mm of asphalt surface treatment, over 160 to 220 mm of granular base, over 360 to 740 mm of granular subbase. The pavement granular was typically moist with laboratory moisture contents ranging from 2 to 5%. Reference is given to the appended Figure 1 for the results of a particle size distribution analysis conducted on a sample of the granular base / subbase. The granular sample did not meet the Ontario Provincial Standard Specification (OPSS) for Granular B due to an elevated silt fraction.

4.1.2 Fill

Fill was contacted below the pavement in all four boreholes, extending to between 1.8 and 3.8 m depth (Elevation 291.3 to 292.9). The fill comprised of gravelly sand, silt, clayey silt and silty sand, and also contained organics in the form of occasional rootlets. The fill had SPT N values ranging between 1 to 9 blows per 0.3 m penetration of the split spoon sampler, corresponding to very loose to loose conditions, as well as indicating a highly variable level of compaction during placement. The fill was moist to wet with moisture contents typically ranging from 3 to 26%.

4.1.3 Topsoil

A 0.5 to 1.0 m thick buried topsoil deposit was encountered beneath the fill in all boreholes, extending to depth of 2.3 to 4.8 m (Elevation 290.3 to 292.4). The buried topsoil comprised sandy silt with varying amounts of gravel, clay and organics with localized peat pockets. Two samples of the topsoil were submitted to the laboratory for particle size distribution analysis with the results presented on Figure 2, appended. The two samples were also submitted for organic content testing, using the ignition method, and results ranged between 2 and 12%. The topsoil was typically very loose with N values of 2 and locally compact in Borehole 1. The buried topsoil was generally moist to wet with moisture contents of 25 and 114%.

4.1.4 Sandy Silt / Sandy Gravel / Sand and Gravel / Silty Sand

Deposits of sandy silt / sandy gravel / sand and gravel / silty sand were encountered below the topsoil and extended to between 3.5 to 6.6 m depth (Elevation 288.3 to 291.2) within the boreholes. These layered deposits of cohesionless soil also contained occasional sand and clayey silt layers along with occasional to numerous cobbles and occasional boulders. The sandy silt / sandy gravel / sand and gravel / silty sand was loose to very dense with N values ranging from 8 to greater than 50 blows per 0.3 m penetration of the split spoon sampler. Two samples of the sandy gravel / sand and gravel were submitted for particle size distribution analysis and reference is given to the appended Figure 3. The sandy silt / sandy gravel / sand and gravel / silty sand was moist to wet with laboratory moisture contents ranging from 6 to 21%.

4.1.5 Silt Till

A silt till deposit was encountered below the sandy silt / sand and gravel / sandy gravel soils. The silt till extended to 16.6 m depth in Borehole 2 and to the termination depth of 17.6 m in Borehole 3 (Elevation 278.5 and 277.5), respectively. Three samples of the till were submitted to the laboratory for particle size distribution analyses with the results presented on Figure 4. The till materials were compact to very dense with N values ranging from 10 to greater than 100. The soil was moist to wet with moisture contents ranging from 9 to 21%.

4.1.6 Deeper Sand and Gravel / Silty Sand

Deposits of sand and gravel / silty sand were encountered in Borehole 2 underlying the silt till and extending to the 17.8 m borehole depth of exploration. The sand was very dense with an N value greater than 100 blows per 0.3 m. The sand and gravel / silt sand soils were wet/saturated with a moisture content of 11%.

4.2 Ground Water

Reference is given to the appended Log of Borehole sheets for details of the ground water conditions encountered during the field work. The Nith River water level at the site was at approximate Elevation 290.7 (4.4 m below bridge deck) in August 2023.

During drilling, initial ground water strikes were observed in the boreholes between 3.0 and 4.8 m depths (Elevation 290.6 to 291.7) which is close to the Nith River water level. Upon completion of drilling in Boreholes 1 and 4, wet cave and free water was measured at 3.0 and 4.2 m depths (Elevation 291.7 and 290.7), respectively. Upon completion of drilling Borehole 2, which penetrated the deeper sand and gravel / silty sand layer, a flowing artesian condition was observed, with a pressure head of 5.4 m above road grade corresponding to about 9.8 m above the river level (Elevation 300.5).

The artesian flow observed from deeper sand and gravel / silty sand deposits represent a deeper ground water condition confined below the silt till.

Ground water levels will fluctuate seasonally, in response to variations in precipitation along with changes in the Nith River water level.

5. DISCUSSION AND RECOMMENDATIONS

The Township of Blandford-Blenheim is proposing to replace or rehabilitate Bridge 24 on Township Road 12 over the Nith River. No detailed plans for the bridge replacement been provided, but it is envisaged that the design will have two lanes, a typical 45 m span, and maintain the current grades at the site.

5.1 Foundations

It is understood that in general, an integral abutment foundation comprised of H-piles is generally envisaged for the new bridge. However, bedrock was not encountered within the depths explored, and more significantly a water bearing layer under artesian pressure was also encountered in the deeper sand and gravel deposits at the west abutment site (Borehole 3). The use of driven piles is not recommended since the required pile founding depths would be at or below the level artesian aquifer. If a pile were to encounter the soils under artesian pressures, then upward seepage of ground water between the driven pile and soil should be expected. A flowing ground water condition would be subject to scrutiny from the Ministry of Environment, Conservation and Parks (MECP) and if not adequately controlled could compromise the pile foundation integrity. Therefore, cognizant of the subsurface conditions at the site, it is recommended that the bridge be supported on footings with conventional abutments, or on cast-in-place concrete caissons which could be tailored to function as integral abutments. The native sandy gravel / sand and gravel, and silt till found at the site are adequate for support of footings or cast-in-place concrete caissons.

5.1.1 Footings

The existing fill and underlying buried topsoil are not suitable for supporting footings, however the underlying compact to dense native sandy gravel / sand and gravel are adequate for support of conventional abutment strip or pad footings. The following table summarizes the recommended design bearing resistances for footings founded on the native soils and the minimum depth that the bearing capacity can be achieved:

BOREHOLE	DEPTH / ELEVATION (m)	SUBGRADE SOIL	GEOTECHNICAL BEARING RESISTANCE AT SLS (kPa)	FACTORED BEARING RESISTANCE AT ULS (kPa)
2	5.0 / 290.1	Sandy Gravel	200	300
3	4.3 / 290.8	Sand and Gravel	200	300

SLS = Serviceability Limit State
ULS = Ultimate Limit State

The bearing resistance at Serviceability Limit State (SLS) is based on 25 mm of total settlement in the bearing stratum, with differential settlement of 75% of this value.

Footings subject to frost action must be provided with a minimum 1.5 m of earth cover or equivalent insulation. Scour protection must also be provided for the footings.

Prior to placement of structural concrete, all founding surfaces must be inspected by qualified geotechnical personnel to verify the design bearing capacity is available, or to reassess the design parameters based on the actual conditions.

For design, a coefficient of friction of 0.5 may be used between a cast-in-place concrete footing and the native sandy gravel / sand and gravel. An appropriate factor of safety must be used for design.

5.1.2 Caissons

The dense to very dense silt till underlying the site is adequate for support of cast-in-place concrete caissons. The following table summarizes the recommended bearing resistances and minimum founding depths for caissons.

BOREHOLE	DEPTH / ELEVATION (m)	SUBGRADE SOIL	GEOTECHNICAL BEARING RESISTANCE AT SLS (kPa)	FACTORED BEARING RESISTANCE AT ULS (kPa)
2	9.5 / 285.6	Silt Till	400	600
3	7.2 / 287.9	Silt Till	400	600

The caisson integrity should be verified in the field by SHAFT QUANTITATIVE INSPECTION DEVICE (SQUID™), to check if a suitable founding subgrade has been achieved. The caisson bearing surface should be verified by precise measurement of the bearing surface at the base with the SQUID device.

In order to maintain stability of caisson founding soils, installation of a deep well or well point dewatering system may also be required to adequately dewater and depressurize the deeper water bearing sand and gravel deposits, which are under artesian pressure and underly the silt till at the site. Further commentary is provided in the Excavation and Ground Water Control section below.

Cobbles and boulders should be expected within the overburden deposits.

It is anticipated that the working platforms to facilitate caisson installation might be required for support of the caisson drilling equipment. This may require excavation of any deleterious fill or organic deposits followed by placement of engineering fill to support the pile driving equipment. Any additional fill that may be required at these locations should comprise an approved granular material such as OPSS Granular B material placed in lifts of 300 mm and compacted to 98% Standard Proctor Maximum Dry Density (SPMDD). Granular material such as Granular B can be employed provided the maximum particle size does not exceed 75 mm.

The caissons piles should be installed and monitored in accordance with the requirements of OPSS.MUNI 903. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity, and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

5.1.2.1 Integral Abutments

It is understood that in general, an integral abutment design is preferred over conventional abutments for the new bridge. It is possible to construct a caisson foundation to function as an integral abutment system. The envisaged caisson pile system would require drilling of the caisson shaft, placement of the HP section fitted with the necessary shear connections, followed by placement of the concrete to the desired free pile level.

For the integral abutment design, the free pile elements of H-piles set in cast-in-place concrete caissons may be designed in the typical manner, as noted below. The typical minimum 3 m long free pile length below the abutment stem which should be incorporated in the design is not a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type 1. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown below maybe used.

SIEVE DESIGNATION		PERCENT PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 - 100
425 µm	#40	40 - 80
250 µm	#60	5 - 25
150 µm	#100	0 - 6

5.1.2.2 Lateral Resistance

The caisson's resistance to lateral loads may be provided by mobilization of passive resistance along the concrete shaft below the annular space. The resistance of this deep foundation system to support lateral loads may be assessed using horizontal subgrade reaction soil parameters. The modulus of horizontal subgrade reaction k_s, MN/m^3, for the typical near surface cohesionless soils of this site may be estimated using the following equation:

$$k_s = N_h \frac{z}{d}$$

where N_h = coefficient related to soil density
= 5 MN/m^3 for loose to compact cohesionless sand and gravel soil
= 15 MN/m^3 for silt till
z = depth, m
d = pile / caisson diameter, m

The lateral resistance of the upper section of the concrete caissons (above potential scour depths) should be neglected.

Group action for lateral loading should be considered based on the reduction factors given in the following table.

CAISSON SPACING PERPENDICULAR TO THE LOAD	SUBGRADE REACTION REDUCTION FACTOR
≥ 4D	1.00
3D	0.83
2D	0.67
1D	0.50

NOTE:

1. D is caisson diameter
2. Spacing is considered the distance from centre to centre

5.1.3 Seismic Considerations

Based on the N values in the stratigraphic profile revealed in the boreholes, and in accordance with the Canadian Highway Bridge Design Code (CHBDC), July 2025, CSA-S6-25, Clause 4.5 – Table 4.2, Site Classification for Seismic Site Response, Class D is applicable for the stratigraphic profile.

5.2 Abutment Wall Design

Abutment walls must be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the abutment. The lateral earth and water pressure, P (kPa), may be computed using the equivalent fluid pressure method presented in Section 6.12 of the CHBDC, or employing the following equation:

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

Where
- P = total lateral pressure at depth h (m) below ground surface (kPa)
- K = lateral earth pressure coefficient of compacted backfill
- h = depth below grade (m) at which lateral pressure is calculated
- γ = bulk unit weight of backfill
- q = vertical stress at depth h due to surcharge loads (kPa)
- C_p = compaction pressure (refer to clause 6.12 of CHBDC)
- C_s = earth pressure induced by seismic events (refer to clause 4.6 of CHBDC)

Lateral Earth Pressure Lands induced by seismic criteria must also be considered, as per CHBDC Section 6.1.4.

An appropriate factor of safety should also be applied.

Free draining granular material should be used as backfill behind the abutments comprising OPSS Granular A or Granular B, placed in thin lifts compacted to a minimum 95% SPMDD. Site soils are not suitable for use as free draining backfill. Over compaction close to the abutment wall should be avoided as this could generate excessive pressure on the abutment wall. The following parameters are recommended for design:

PARAMETERS	OPSS GRANULAR A	OPSS GRANULAR B TYPE I OR TYPE II
Angle of Internal Friction (degrees)	35°	32°
Unit Weight, γ (kN/m^3)	22.5 ± 0.3	21.5 ± 0.3
Rankin Active Earth Pressure Coefficient (K_a)	0.27	0.31
At Rest Earth Pressure Coefficient (K_o)	0.43	0.53
Rankin Passive Earth Pressure Coefficient (K_p)	3.69	3.13

A weeping tile system and/or weeping holes should be installed to minimize the build-up of hydrostatic pressure behind the abutments. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

5.3 Excavation and Ground Water Control

Excavation for bridge foundations is expected to extend to about Elevation 290 at the abutment locations, and will encounter the pavement structure and fill, buried topsoil material, and water bearing sandy gravel / sand and gravel deposits with cobbles and boulders.

The excavations are expected to typically extend through variable loose to compact fill and cohesionless soil, which are classified as Type 3 materials as defined in the Occupational Health and Safety Act (OHSA), providing adequate ground water control is achieved. Trench excavations within Type 3 soil that are to be entered by workers, may not be steeper than one horizontal to one vertical (1H:1V) from the base of the excavation. Workers should not enter an unprotected excavation if there is evidence of ongoing ground water seepage in the banks.

It envisioned that water tight shoring (a combination steel sheet piling or concrete caissons) would be required to complete excavations for footing and abutment construction. The shoring will be required to support the excavations and prevent ground and surface water inflow. The sheeting should be driven into the relatively impervious silt till layer contacted below Elevations 289.9 and 288.9, in Borehole 2 and 3, for the west and east abutments respectively. For design of temporary shoring for excavations, the following parameters may be assumed:

PARAMETER	FILL	SAND AND GRAVEL	SILT TILL
Angle of Internal Friction, ϕ, (degrees)	26	30	30
Shear Strength, c, (kPa)	--	--	--
Bulk Unit Weight (kN/m^3)	17	22	20
Rankin Active Earth Pressure Coefficient (K_a)	0.39	0.33	0.33
At Rest Earth Pressure Coefficient (K_o)	0.56	0.50	0.50
Rankin Passive Earth Pressure Coefficient (K_p)	2.56	3.00	3.00

Difficult driving conditions will be encountered in the sand and gravel and till deposits due to the presence of cobbles and boulders. Shoring should be designed and installed by specialist in this field.

For construction staging, cofferdam and minor river diversion will likely be required for the bridge abutments. It is recommended the work be scheduled following periods of prolonged dry weather, and when the ground water table and river flow are usually at their lowest, in order to minimize the quantity of water to be handled.

Excavations to the planned abutment levels will require rigorous dewatering to maintain a safe and sufficiently dry excavation and the use of keg wells or well point dewatering are envisaged.

Significantly less dewatering would be required if water tight sheet pile cofferdam / ground water cut-off structures are employed, but some dewatering will still be required given the heterogeneous nature of the on-site soils, which are interlayered with saturated sand and gravel deposits.

It is also noted that the silt till soils at the site are underlain by deposits of sand and gravel which are under significant artesian pressure with a head about 9.8 m above the river level. For any excavation planed below the current river bottom and into the till it will also be necessary to install a deep well or well point dewatering system to draw down and depressurize the underlying artesian ground water conditions. For excavations (including caisson drilling) below the current river level, there is a significant risk of base instability and sudden blow out failure of the excavation base if the artesian pressure head is not adequately controlled prior to excavation.

Regardless of the dewatering method chosen, the hydraulic head and ground water inflow must be properly controlled to ensure stable and safe excavation and to facilitate construction. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control ground water at least 0.30 m below the excavation base level, in order to provide a stable excavation base throughout construction.

It is recommended that test pits be carried out during the tendering stage of the project in order that prospective contractors may familiarize themselves with soil and ground water conditions. Also, the dewatering requirements should also be established by the contractor in the context of a performance specification.

It should be noted that, under the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, a Permit to Take Water (PTTW) from the Ministry of Environment Conservation and Parks (MECP) is required if the dewatering discharge is greater than 50,000 L/day. In accordance with the above noted regulatory requirements and in compliance with the MECP's PTTW Manual (April 2005) an application should be filed to the MECP for the subject property construction dewatering PTTW, if the dewatering discharge is greater than 400,000 L/day, or about 4.6 L/S. If the dewatering discharge is between 50,000 L/day (or about 0.6 L/S) and 400,000 L/day (or about 4.6 L/S) dewatering activities need to be registered on the Environmental Activity and Sector Registry (EASR).

For open cut excavations, dewatering volumes are expected to exceed 50,000 L/day cognizant of the ground water level and time of year the work is carried out. Dewatering from water tight sheeted abutment excavations will be significantly less than the open cut excavations. Regardless, construction of engineered fill pads for piling equipment, reconstruction of approach embankments, and ancillary work associated with the bridge reconstruction might still require excavation outside of the cut off / cofferdams, and therefore construction dewatering rates in excess of 50,000 L/day should still be anticipated. Therefore, an ESAR / PTTW and supporting hydrogeological assessment will likely be required. A detailed review of the final foundation and excavation levels will be required to determine the extent of the dewatering and the requirements for a hydrogeological investigation.

5.4 Pavement Design

It is assumed that the road grades along Township Road 12 will remain at their current elevations. Based on the proposed pavement usage, frost susceptibility, and strength of the expected subgrade soils, the following pavement component thicknesses are considered suitable for roadway reinstatement of Township Road 12.

PAVEMENT COMPONENT	THICKNESS
HL 3 Surface Asphalt	40 mm
HL 8 Binder Asphalt	60 mm
Granular A Base	150 mm
Granular B Subbase	450 mm
Total Thickness	**700 mm**

The pavement design considers that construction will be carried out during the drier time of the year. Preparation of the subgrade should involve removal of deleterious topsoil and organic fill materials and ensuring that the subgrade is stable and compacted to at least 95% SPMDD, as determined by proofrolling and inspection by qualified geotechnical personnel. If the subgrade is wet and unstable, subexcavation and placement of additional granular subbase material will be required.

The pavement materials should conform to current OPS and municipal specifications. The Granular A base and Granular B subbase courses should be placed in thin lifts and compacted to a minimum of 100% SPMDD, and asphalt should be placed to a minimum of 92% of the material's Maximum Relative Density (MRD) and reference is made to OPS Specification 310.

It should be noted that the subgrade will lose its strength if allowed to become wet due to surface water or during freezing and thawing periods. Therefore, drainage of the granular courses and subgrade becomes very essential. Drainage should be provided by extending the granular courses out to the face of the embankment slopes.

It is recommended that at the transition zones, the subgrade level of the new pavement sections and existing pavement section should match, if possible, to avoid any problems associated with differential frost heaving of the subgrade. Alternatively, frost tapering of the subgrade at 10H:1V would be recommended.

During construction, testing should be conducted to confirm the gradation and compactibility characteristics of the granular base and subbase materials.

Proofrolling procedures and the placement and compaction of all the fill and granular materials for the pavement construction and backfilling at the site should be inspected on a continuous basis by PML technicians.

5.5 Geotechnical Review and Construction Inspection and Testing

It is recommended that the final design drawings be submitted to PML for Geotechnical review for compatibility with site conditions and recommendations of this report.

Earthworks operations should be carried out under the supervision of PML to approve subgrade preparation, backfill materials, placement and compaction procedures and check the specified degree of compaction is achieved throughout.

Any caisson installation operations should be reviewed on a full-time basis by qualified Geotechnical personnel to check that the required capacity is achieved, and to document founding elevation, alignment and plumbness.

The comments and recommendations provided in the report are based on information revealed in the boreholes. Conditions away from and between boreholes may vary. Geotechnical review during construction should be ongoing to confirm the subsurface conditions are substantially similar to those encountered in the boreholes, which may otherwise require modification to the original recommendations.

6. CLOSURE

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to call our office.

Sincerely

Peto MacCallum Ltd.

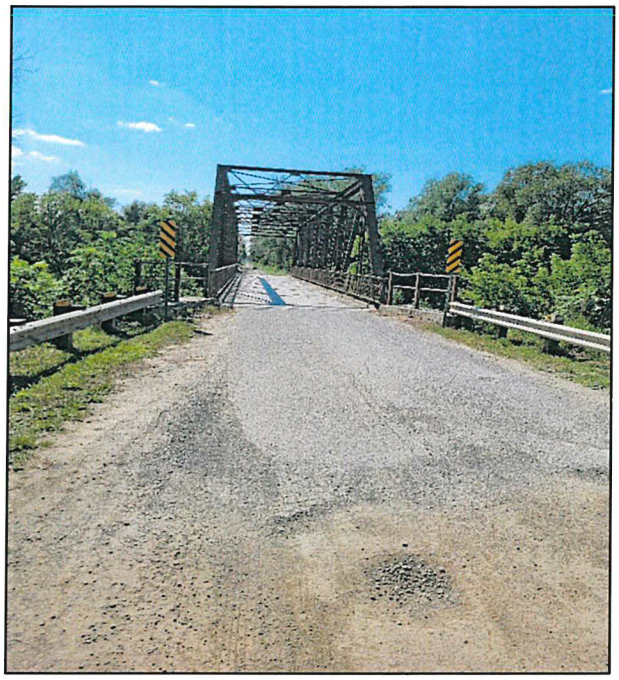


William Loghrin, P.Eng.
Manager, Engineering Services

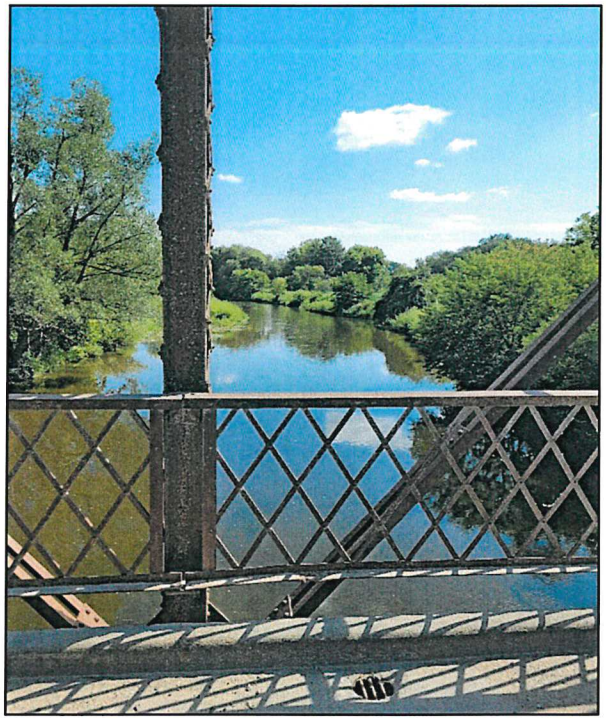


Gerry Mitchell, MEng, P.Eng.
Senior Consultant

WL/GM:cs



Photograph 1: Township Road 12, Bridge 24 facing west

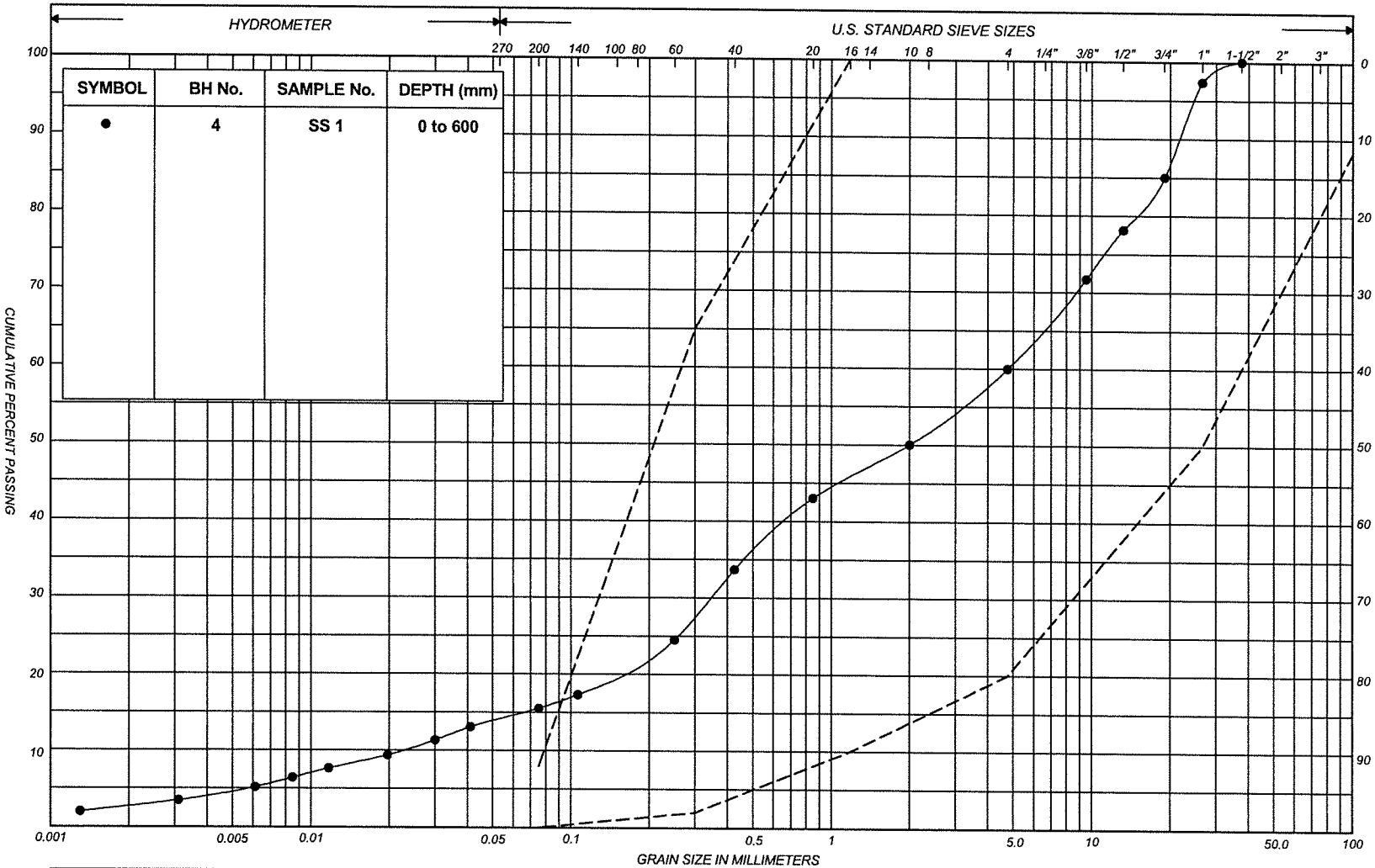


Photograph 2: Facing south from the bridge

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PARTICLE SIZE DISTRIBUTION CHART

PML REF.: 23KF019
FIGURE NO.: 1



SILT & CLAY			FINE	MEDIUM	COARSE	GRAVEL		COBBLES	UNIFIED		
			SAND								
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL		COBBLES	M.I.T.	
	SILT			SAND							
CLAY	SILT		V. FINE	FINE	MED.	COARSE	GRAVEL			U.S. BUREAU	
			SAND								

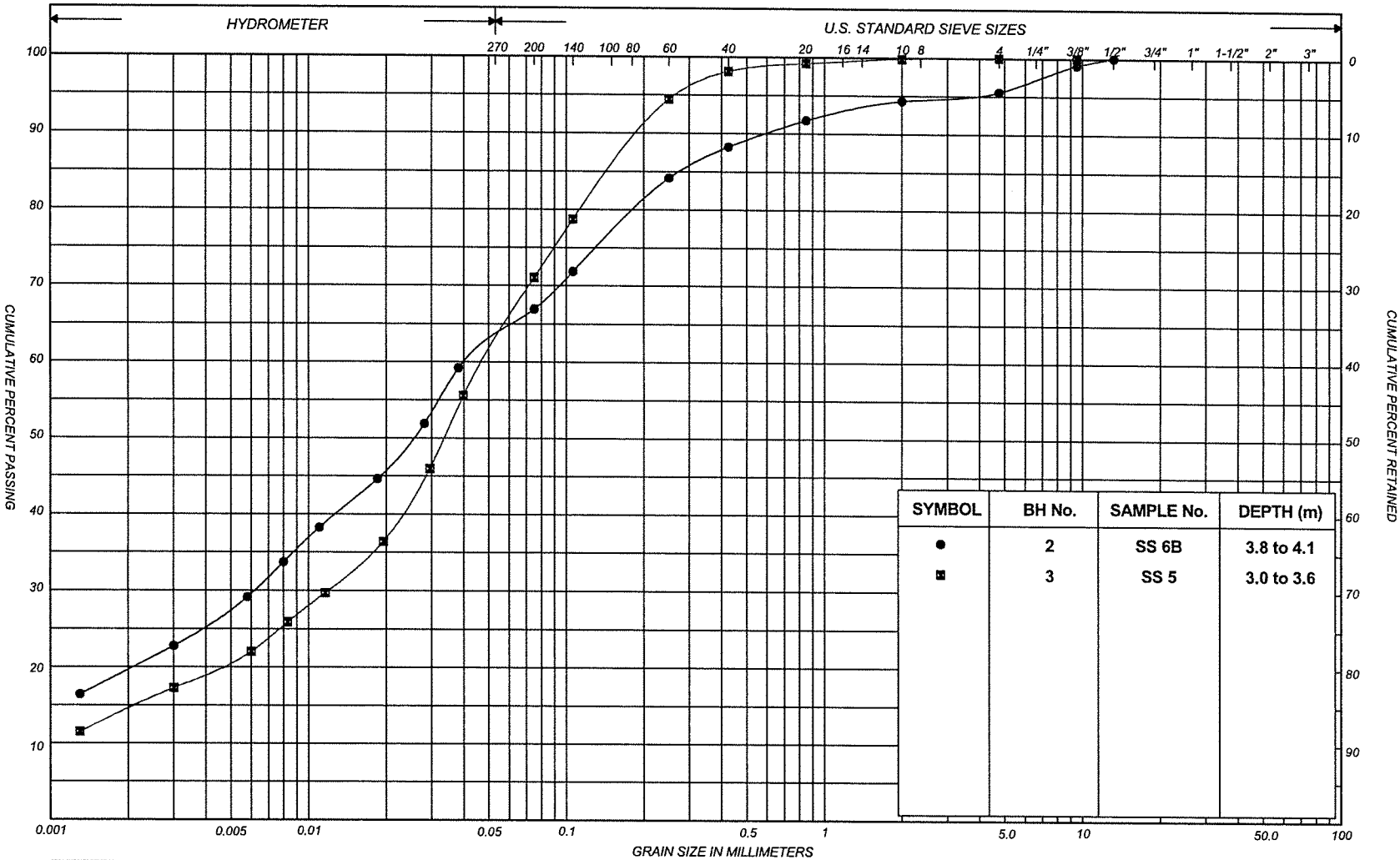
REMARKS: Granular Base / Subbase

COMPARED TO OPSS GRANULAR B TYPE I SPECIFICATONS

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PARTICLE SIZE DISTRIBUTION CHART

PML REF.: 23KF019
FIGURE NO. 2



SYMBOL	BH No.	SAMPLE No.	DEPTH (m)
●	2	SS 6B	3.8 to 4.1
⊠	3	SS 5	3.0 to 3.6

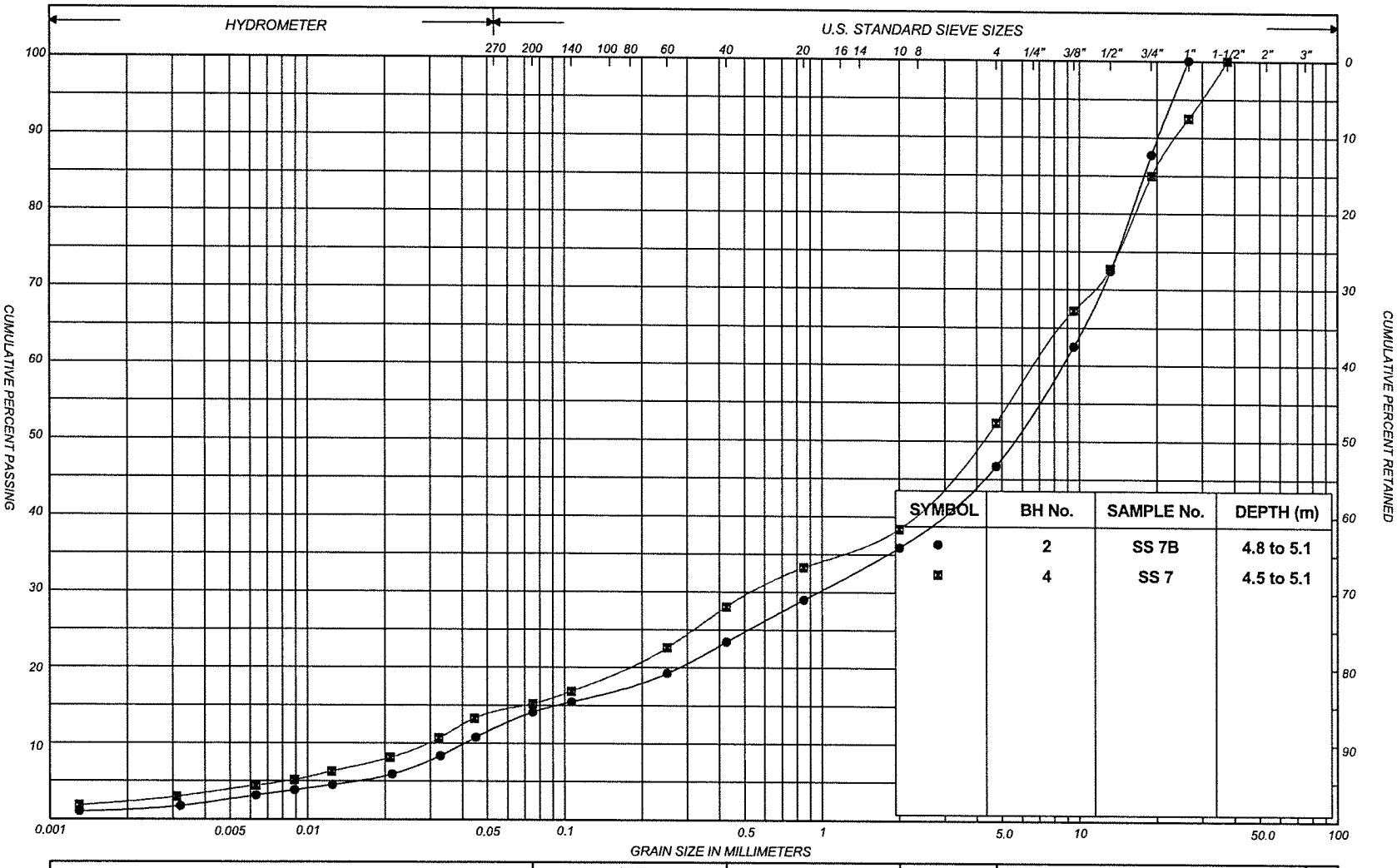
SILT & CLAY			FINE	MEDIUM	COARSE	GRAVEL		COBBLES	UNIFIED		
			SAND								
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL		COBBLES	M.I.T.	
	SILT			SAND							
CLAY	SILT		V. FINE	FINE	MED.	COARSE	GRAVEL			U.S. BUREAU	
			SAND								

REMARKS: Topsoil

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PARTICLE SIZE DISTRIBUTION CHART

PML REF.: 23KF019
FIGURE NO. 3



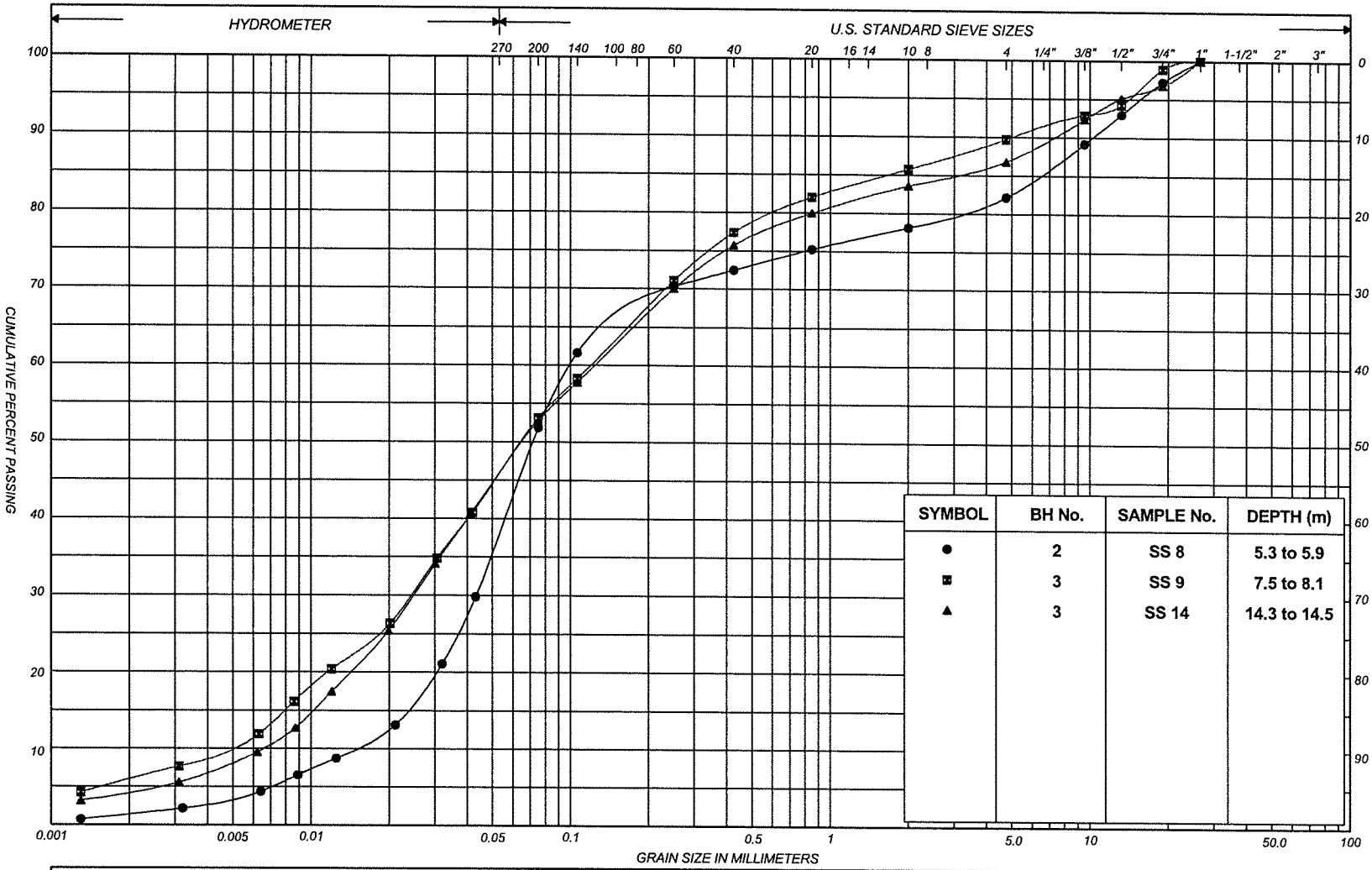
SILT & CLAY			FINE	MEDIUM	COARSE	GRAVEL			COBBLES	UNIFIED	
			SAND								
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL			COBBLES	M.I.T.
	SILT			SAND							
CLAY	SILT		V. FINE	FINE	MED.	COARSE	GRAVEL				U.S. BUREAU
			SAND								

REMARKS: Sandy Gravel / Sand and Gravel

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PARTICLE SIZE DISTRIBUTION CHART

PML REF.: 23KF019
FIGURE NO. 4



SYMBOL	BH No.	SAMPLE No.	DEPTH (m)
●	2	SS 8	5.3 to 5.9
■	3	SS 9	7.5 to 8.1
▲	3	SS 14	14.3 to 14.5

SILT & CLAY				FINE	MEDIUM	COARSE	GRAVEL			COBBLES	UNIFIED
				SAND							
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL			COBBLES	M.I.T.
	SILT			SAND							
CLAY	SILT			V. FINE	FINE	MED.	COARSE	GRAVEL			U.S. BUREAU
				SAND							

REMARKS: Silt Till

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. - Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow.

DESCRIPTION OF SOIL

The consistency of cohesive soils and the relative density or denseness of cohesionless soils are described in the following terms:

CONSISTENCY	N (blows/0.3 m)	c (kPa)	DENSENESS	N (blows/0.3 m)
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	> 50
Hard	> 30	> 200		

WTPL	Wetter Than Plastic Limit
APL	About Plastic Limit
DTPL	Drier Than Plastic Limit

TYPE OF SAMPLE

SS	Split Spoon	TW	Thinwall Open
WS	Washed Sample	TP	Thinwall Piston
SB	Scraper Bucket Sample	OS	Oesterberg Sample
AS	Auger Sample	FS	Foil Sample
CS	Chunk Sample	RC	Rock Core
ST	Slotted Tube Sample	USS	Undisturbed Shear Strength
PH	Sample Advanced Hydraulically	RSS	Remoulded Shear Strength
PM	Sample Advanced Manually		

SOIL TESTS

Qu	Unconfined Compression	LV	Laboratory Vane
Q	Undrained Triaxial	FV	Field Vane
Qcu	Consolidated Undrained Triaxial	C	Consolidation
Qd	Drained Triaxial		

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 1

17T 536684E 536684N

PROJECT Bridge 24 Township Road 12 Over Nith River
PML REF. 23KF019
LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario
BORING DATE 06-26-2023
ENGINEER WL
BORING METHOD Continuous Flight Hollow Stem Augers
TECHNICIAN SR

DEPTH (metres)	ELEV	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	294.7	SURFACE ELEVATION 294.7			
0.0	294.10	PAVEMENT STRUCTURE: 25 mm surface treatment asphalt, over 160 mm granular base, over 415 mm of granular subbase, mosit	1	SS	26
0.60					
		FILL: Loose light brown, gravelly sand, trace silt, moist	2[1]	SS	7
1.0					
1.3	293.4	occasional dark brown silt inclusions, moist	3A	SS	14
1.8					
	292.9	TOPSOIL: Black sandy silt, some clay, trace gravel, wet, occasional roots, occasional wood pieces	3B		
2.0					
2.3			4A		
	292.4	SANDY SILT: Loose, brown, sandy silt, trace to some clay, trace gravel, trace organics, moist to wet	4B	SS	9
2.9					
3.0	291.8	SANDY GRAVEL: Compact, brown, sandy gravel, trace silt, wet	5A	SS	16
3.5					
	291.2	SILT TILL: Compact, brown to greyish brown, sandy silt, trace to some gravel, trace clay, wet	5B		
4.0			6	SS	15
4.3					
	290.4	becomming moist, occasional fine sand seams	7	SS	15
5.0					
6.0					
6.4			8	SS	24
6.6		becoming grey			
7.0	288.1	BOREHOLE TERMINATED AT 6.6 m.			
8.0					
9.0					
10.0					
11.0					

ELEVATION SCALE: 294, 293, 292, 291, 290, 289

SHEAR STRENGTH (kPa)
+FIELD VANE △TORVANE ○ Qu
▲POCKET PENETROMETER ○ Q
50 100 150 200
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
20 40 60 80

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS
GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL

▼ El. 291.7

Upon completion of augering wet cave at 3.0 m

NOTES [1]Sample submitted for chemical analysis.

▼ GROUND WATER STRIKE

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 2

17T 536701E 536701N

PROJECT Bridge 24 Township Road 12 Over Nith River — **PML REF.** 23KF019

LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario — **BORING DATE** 06-26-2023 — **ENGINEER** WL

BORING METHOD Continious Flight Hollow Stem Augers and HQ Diamond Coring — **TECHNICIAN** SR

DEPTH (metres)	ELEV	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	295.1	SURFACE ELEVATION 295.1			
0.0	295.1	PAVEMENT STRUCTURE: 40 mm surface treatment asphalt, over 220 mm granular base, over 740 mm silty granular subbase, moist	1	SS	17
1.00	294.10	FILL: Soft brown clayey silt, some sand, trace gravel, APL	2A 2B[1]	SS	3
2.2	292.9	becoming very loose sandy silt, some gravel, some clay, occasional rootlets, moist	3 4	SS SS	4 1
3.0	292.1	becoming wet	5	SS	2
3.8	291.3	TOPSOIL: Very loose dark brown to black sandy silt, trace gravel, trace clay, wet, occasional rootlets, occasional wood pieces	6A 6B	SS	2
4.8	290.3	SANDY GRAVEL: Dense greyish brown sandy gravel, some silt, trace clay, wet, numerous cobbles, occasional boulder	7A 7B	SS	32
5.2	289.9	SILT TILL: Compact to dense greyish brown sandy silt, some gravel, trace clay, very moist to wet, occasional cobble, occasional boulder	8	SS	14
7.0	288.1	becoming grey	9	SS	13
8.2		becoming gravelly sand layer, wet	10	SS	32
8.5	286.6	becoming compact grey silt, some sand, some gravel, trace clay, moist to wet, occasional clayey silt layers, occasional cobble, occasional boulder	11A 11B	SS	21
		CONTINUED	12	SS	21

SHEAR STRENGTH (kPa)
+FIELD VANE △TORVANE ○ Qu
▲POCKET PENETROMETER ● Q
50 100 150 200
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
20 40 60 80

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30 40

UNIT WEIGHT kN/m³

114
60

GROUND WATER OBSERVATIONS AND REMARKS

El. 291.3

GRAIN SIZE DISTRIBUTION (%)
GR SA SI&CL

4 29 47 20
Organic Content 12%

53 33 13 1

18 30 51 1

NOTES [1]Sample submitted for chemical analysis.

GROUND WATER STRIKE

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 2

17T 536701E 536701N

PROJECT Bridge 24 Township Road 12 Over Nith River
PML REF. 23KF019
LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario
BORING DATE 06-26-2023
ENGINEER WL
BORING METHOD Continious Flight Hollow Stem Augers and HQ Diamond Coring
TECHNICIAN SR

DEPTH ELEV (metres)	DESCRIPTION	NUMBER	TYPE	"N" VALUES
11.5	CONTINUED FROM PREVIOUS PAGE			
		13	SS	19
		14	SS	20
		15	SS	10
15.2 / 279.9	becoming very dense, moist			
		16	SS	100/125mm
16.6 / 278.5	SAND AND GRAVEL: Very dense grey sand and gravel, trace to some silt, wet / saturated			
17.6				
17.8	SILTY SAND: Very dense grey silty sand, wet	17	SS	100/125mm
	BOREHOLE TERMINATED AT 17.8 m.			

SHEAR STRENGTH (kPa): +FIELD VANE △TORVANE ○ Qu ▲POCKET PENETROMETER ○ Q (50 100 150 200)
DYNAMIC CONE PENETRATION × STANDARD PENETRATION TEST ● (20 40 60 80)
ELEVATION SCALE: 283, 282, 281, 280, 279, 278
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30 40)
UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS
GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL

Upon completion of drilling artesian flow from cased borehole, hydraulic head at 5.4 m above road grade (elevation 300.5). First attempt to artesian seal flow failed, with hydraulic head at 3.1 m above road grade (elevation 292.0).

NOTES [1]Sample submitted for chemical analysis.

▼ GROUND WATER STRIKE

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 3

17T 536763E 536763N

PROJECT Bridge 24 Township Road 12 Over Nith River — **PML REF.** 23KF019

LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario — **BORING DATE** 06-20-2023 — **ENGINEER** WL

BORING METHOD Continious Flight Hollow Stem Augers and HQ Diamond Coring — **TECHNICIAN** SR

DEPTH (metres) / ELEV	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	SURFACE ELEVATION 295.1			
0.0 / 295.1	PAVEMENT STRUCTURE: 40 mm surface treatment asphalt, over 195 mm granular base, over	1A	SS	34
0.60 / 294.50	365 mm granular subbase, moist FILL: Loose to very loose brown silty sand, some gravel, trace clay, moist, occasional rootlets	1B		
1.0		2[1]	SS	7
2.0		3	SS	1
3.0 / 292.1		4	SS	5
3.0 / 292.1	TOPSOIL: Very loose dark brown sandy silt, some clay, moist to wet, occasional rootlets	5	SS	2
3.7 / 291.4	SANDY SILT: Loose, grey, sandy silt, some clay to clayey, trace gravel, very moist to wet	6A	SS	8
4.1 / 291.0	SAND AND GRAVEL: Compact brown sand and gravel, some silt, occasional cobbles, wet	6B		
5.0		7	SS	25
6.2 / 288.9	SILT TILL: Compact grey silt, some sand, some gravel, trace clay, occasional clayey silt layers, occasional sand layers, occasional cobble, occasional boulder, wet	8A 8B	SS	10
7.0		9	SS	24
8.2 / 286.9	becoming silt, trace to some sand, trace clay, moist to wet	10	SS	30
9.0				
10.0		11	SS	19
10.8 / 284.3	becoming very dense, moist			
11.0	CONTINUED			

SHEAR STRENGTH (kPa): +FIELD VANE △TORVANE ○ Qu; ▲POCKET PENETROMETER ○ Q; 50 100 150 200

DYNAMIC CONE PENETRATION × STANDARD PENETRATION TEST ●; 20 40 60 80

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS

GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL

Organic Content 2%, non-plastic 0 29 57 14

▼ El. 290.6

non-plastic 10 37 47 6

NOTES [1]Sample submitted for chemical analysis.

▼ GROUND WATER STRIKE

Peto MacCallum Ltd
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 3

17T 536763E 536763N

PROJECT Bridge 24 Township Road 12 Over Nith River — **PML REF.** 23KF019

LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario — **BORING DATE** 06-20-2023 — **ENGINEER** WL

BORING METHOD Continious Flight Hollow Stem Augers and HQ Diamond Coring — **TECHNICIAN** SR

DEPTH ELEV (metres)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
11.5	CONTINUED FROM PREVIOUS PAGE				
		12	SS	84	283
		13	SS	50/100mm	282
13.7					
281.4	becoming sandy silt, some gravel, trace clay, moist to wet, occasional clayey silt layers DTPL to APL	14	SS	50/100mm	281
		15	SS	100/75mm	280
					279
					278
17.6		16	SS	102/75mm	
277.5	BOREHOLE TERMINATED AT 17.6 m.				

SHEAR STRENGTH (kPa)
+FIELD VANE △TORVANE ○ Qu
▲POCKET PENETROMETER ○ Q
50 100 150 200
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
20 40 60 80

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS

GRAIN SIZE DISTRIBUTION (%)
GR SA SI&CL

13 34 49 4

Upon completion of drilling wet cave at 5.9 m

NOTES [1]Sample submitted for chemical analysis.

▼ GROUND WATER STRIKE

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 4

17T 536781E 536781N

PROJECT Bridge 24 Township Road 12 Over Nith River — **PML REF.** 23KF019

LOCATION Township of Blanford - Blenheim, County of Oxford, Ontario — **BORING DATE** 06-20-2023 — **ENGINEER** WL

BORING METHOD Continuous Flight Hollow Stem Augers — **TECHNICIAN** SR

DEPTH (metres)	ELEV	DESCRIPTION	NUMBER	TYPE	"N" VALUES
		SURFACE ELEVATION 294.9			
0.0	294.9	PAVEMENT STRUCTURE: 40 mm surface treatment asphalt, over 200 mm granular base, over 360 mm granular subbase, moist	1	SS	32
0.60	294.30	FILL: Loose dark brown silty sand, some gravel, trace clay, moist	2[1]	SS	9
1.0					
			3A	SS	8
2.0					
			4	SS	9
2.7	292.2	occasional layers of coarse sand, moist			
3.0			5	SS	5
3.4	291.5	TOPSOIL: Very loose dark brown sandy silt, trace to some clay, moist to wet			
3.9	291.0	becoming grey, wet	6A	SS	2
4.0			6B		
4.2	290.7	SAND AND GRAVEL: Compact brown sand and gravel, some silt, trace clay, occasional cobble, occasional boulder, wet	7	SS	12
5.0					
6.0					
6.3			8A	SS	69
6.6	288.3	SILTY SAND: Very dense silty sand, some gravel, trace clay, wet	8B		
		BOREHOLE TERMINATED AT 6.6 m.			
7.0					
8.0					
9.0					
10.0					
11.0					

ELEVATION SCALE: 294, 293, 292, 291, 290, 289

SHEAR STRENGTH (kPa)
+FIELD VANE ΔTORVANE O Qu
▲POCKET PENETROMETER O Q
50 100 150 200
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
20 40 60 80

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS

GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL

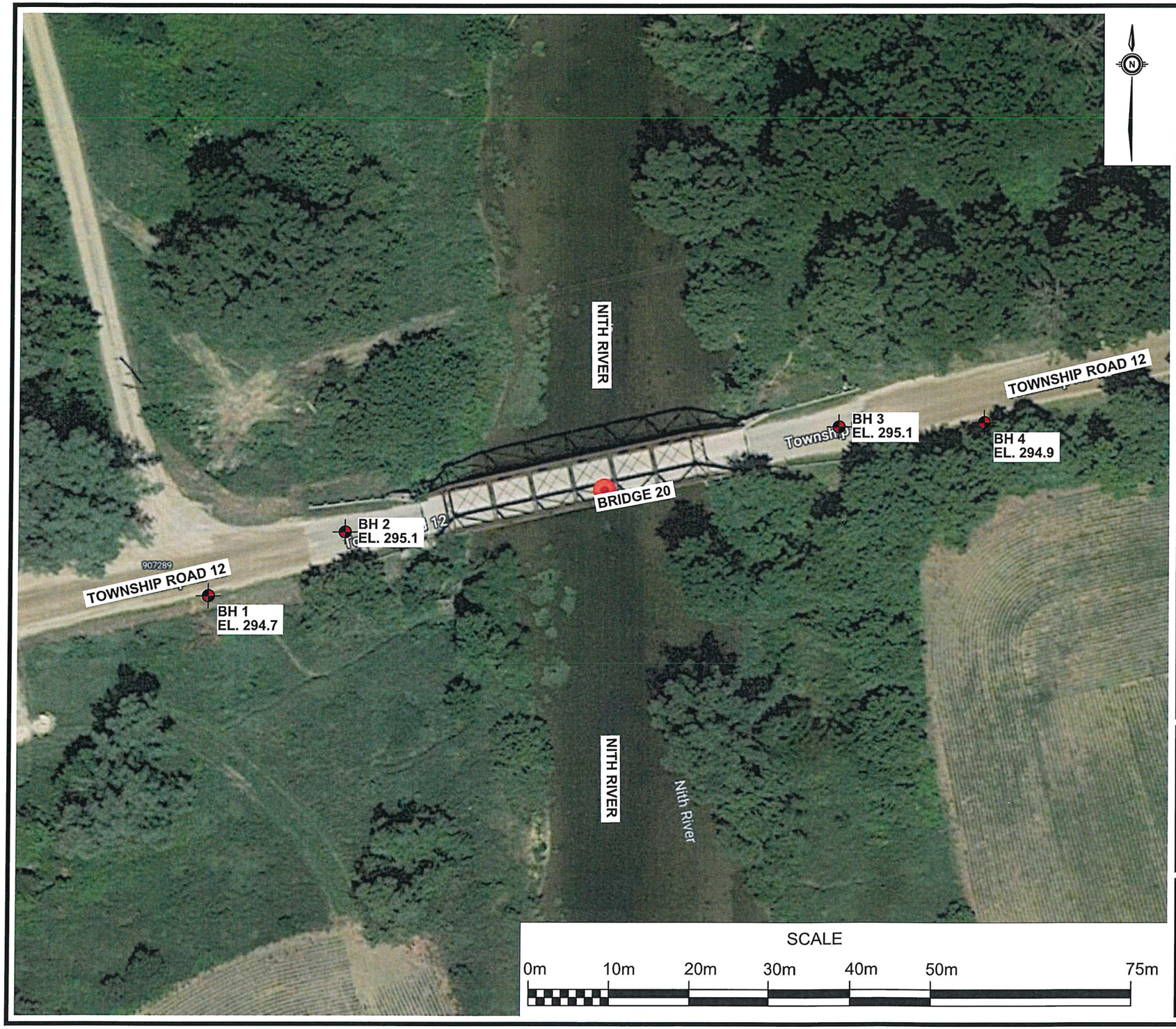
40 44 13 3

El. 290.1 48 37 13 2

Upon completion of augering free water at 4.2 m cave at 4.8 m

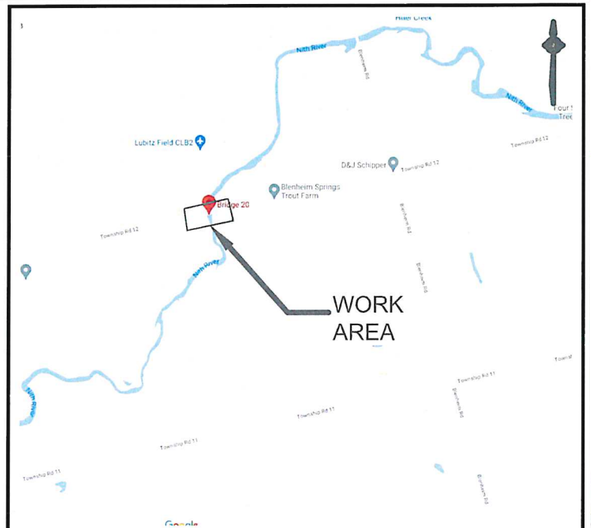
NOTES [1]Sample submitted for chemical analysis.

GROUND WATER STRIKE



SCALE

0m 10m 20m 30m 40m 50m 75m



KEY PLAN

TOWNSHIP OF BLANDFORD-BLENHEIM, ONTARIO

LEGEND:

BH 4 EL. 294.9 — PETO MACCALLUMLTD. (PML) BOREHOLE (BH) LOCATION ELEVATION (METRIC, GEODETIC)

REFERENCE:

BOREHOLE LOCATION PLAN REPRODUCED FROM GOOGLE MAPS.

NOTES:

1. THE INFERRED STRATIGRAPHY REFERRED TO IN THE REPORT IS BASED ON THE DATA FROM THESE BOREHOLES SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY BETWEEN THE BOREHOLES MAY VARY.
2. GEODETIC GROUND SURFACE ELEVATIONS AND UTM COORDINATES AT THE BOREHOLE LOCATIONS WERE DETERMINED BY PML USING A GLOBAL NAVIGATION SATELLITE SYSTEM (GNSS). THE SURVEY EQUIPMENT COMPRISED A SOKKIA CANADA GCX-3 NETWORK REAL TIME KINEMATIC (RTK) ROVER SYSTEM.

K. SMART ASSOCIATES LIMITED.

GEOTECHNICAL INVESTIGATION

BRIDGE 24 TOWNSHIP ROAD 12 OVER NITH RIVER
TOWNSHIP OF BLANDFORD-BLENHEIM
COUNTY OF OXFORD, ONTARIO

DRAFT BOREHOLE LOCATION PLAN

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN	S. REGI	DATE	SCALE	PML REF.	DWG. NO.
CHECKED	W. LOGHRIN	AUG. 2026	AS SHOWN	23KF019	1
APPROVED	W. LOGHRIN				

APPENDIX A

STATEMENT OF LIMITATIONS

STATEMENT OF LIMITATIONS

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