

TECHNICAL MEMORANDUM

DATE June 13, 2022

Project No. 21501071-1000

TO Brad Hewton P. Eng.
Morrison Hershfield

CC Andrew Eagen, P.Eng.

FROM Kenton Power, P.Eng.
Reviewed by Bill Cavers, P.Eng.

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**DRAFT DESKTOP FOUNDATION REVIEW
FOUNDATION SUPPORT FOR ENVIRONMENTAL ASSESSMENT
HIGHWAY 416 / BARNSDALE ROAD UNDERPASS
GWP 4057-20-00, AGREEMENT NO. 4019-E-0023
GEOCRES NO. 31G-289**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a desktop foundation review for a new interchange at the existing Highway 416 and Barnsdale Road underpass located near Manotick in Ontario, under Assignment No. 2 of the Mega 18 Retainer Assignment (4019-E-0023).

This technical memorandum presents a brief summary of the factual findings from a foundation review carried out for this project, together with preliminary geotechnical/foundation recommendations based on the existing subsurface information.

The terms of reference for the original scope of work are outlined in MTO's Work Item Order Form for Assignment 2, dated November 15, 2021. Golder's scope of work for the preliminary foundation engineering services associated for this assignment was provided in the Work Order for this assignment dated November 12, 2021.

The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

2.0 SITE DESCRIPTION

A new interchange is proposed around the existing Highway 416 and Barnsdale Road underpass located near Manotick in Ontario. At this location, Highway 416 is a divided highway with two travel lanes in each direction and Barnsdale Road has a single undivided lane in each direction. The location of the intersection of Highway 416 and Barnsdale Road is shown on the Key Plan on Figure 1.

The existing underpass, which carries two lanes of Barnsdale Road over Highway 416 (MTO Structure Site 3-552) is located within the outer boundaries of the City of Ottawa about 6 km south of Fallowfield Road, and 2 km north of Bankfield Road. The location of the bridge is shown on the Key Plan on Figure 1.

Barnsdale Road is an undivided road with a single travel lane in each direction with a rural cross section, gravel shoulders and drainage provided by ditching to the north and south. Steel beam guiderails and concrete barriers are provided along both sides of the Barnsdale Road in the vicinity of the underpass structure.

At this location, Highway 416 has a four-lane cross-section with two northbound and two southbound through lanes separated by a wide, vegetated median. Steel beam guiderails are also present along both sides of the highway in the vicinity of the underpass structure.

The original 1990 General Arrangement (GA) Drawing 1 and Footing Details Drawing 3 (provided in Appendix B) indicate that the footings for the abutments and pier are supported by 310x110 H-piles driven to bear directly on the dolostone bedrock, as follows:

Table 1: Existing Bridge Foundation Conditions

Foundation Element	Underside of Pile Cap Elevation	Pile Cap Size
West Abutment	101.2 m	4.0 m wide by 10.0 m long
Centre Pier	95.8 m	5.6 m by 5.6 m
East Abutment	101.2 m	4.0 m wide by 10.0 m long

3.0 SITE STRATIGRAPHY

3.1 Overview - Available Information

The current scope of work does not include additional borehole drilling or other intrusive field investigations and this summary is based solely on previously collected subsurface information pertinent to the proposed site.

The subsurface information used in the preparation of this report was obtained from historical data provided in the previous the Foundation Investigation Report for the site:

- Foundation Investigation Report for Barnsdale Road Interchange Underpass Structure # 24, W.P. 128-87-10, Site No. 3-552, Hwy. 416, District 9, Ottawa, dated July 14, 1989. (GEOCRES 31G-197).

A total of ten boreholes along with six dynamic cone penetration tests were advanced at the site as part of the original investigation along the then-proposed Barnsdale Road Interchange. Bedrock was cored at three boreholes at the structure foundation locations. In general, at the borehole locations, the subsurface conditions consist of fill material overlying native sand and silt deposit, underlain by glacial till, underlain by dolostone bedrock with shale interbeds. Based on existing construction drawings, the existing underpass structure is supported on steel piles driven to bedrock. Copies of the borehole records and Bore Hole Locations & Soil Strata Drawing 1288710-A are provided in Appendix A.

3.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 416 lies within the minor physiographic region known as the Edwardsburg Sand Plain, which lies within the major physiographic region of the Ottawa-St. Lawrence Lowland. The Edwardsburg Plain region is characterized by a slightly undulating sand plain that overlies boulder clay and bedrock. The sand is likely glaciofluvial in origin, deposited in the late stages of the Champlain Sea with a few morainic structures remaining.

The surficial geological mapping² produced by the Geological Survey of Canada (GSC) indicate that the study area is underlain by reworked glaciofluvial sands and silts overlying sandy silt to silty sand-textured till, see Figure 3. The published drift thickness mapping (depth to bedrock) indicates that the bedrock surface is generally located at depths ranging from 15 to 25 m; see Figure 5. The bedrock geology mapping³ indicates that the bedrock at study area is limestone and dolomite of the Oxford Formation; see Figure 4.

3.3 Site Stratigraphy Overview

Based on existing geological mapping and the results of previous investigation reports at the site, the subsurface conditions at the site are anticipated to generally consist of surficial fill deposits, underlain by native deposits of sand and silt, underlain by glacial till, which in turn underlain by dolostone bedrock;. Figure 3 illustrates the surficial geology mapping in the area surrounding the crossing.

A more detailed description of the overburden soil deposits, and bedrock geology conditions encountered during the 1989 field investigation is provided in the following sections.

3.3.1 Sand and Silt

A native sand and silt deposit exists at surface/near surface in all previous boreholes. Random zones of poorly graded sand and nodules of silt were also encountered at varying depths within the sand and silt deposit.

The sand and silt deposit was not fully penetrated in all of the previous boreholes, but proven to depths ranging from 8.1 m to 11.1 m below existing ground surface. Where fully penetrated, the native sand and silt deposit extends to depths of 11.6 to 14.0 m below existing ground surface (i.e., Elevations of 87.7 m to 85.1 m). Based on previous Standard Penetration Tests (SPT) N values, the sand and silt deposit are indicative of a soil in a loose to dense state, but generally compact to dense state of packing.

3.3.2 Glacial Till

A stratum of glacial till consisting of a homogeneous mixture of sand, gravel, and boulders underlies the native silt and sand deposit at Boreholes 24-1 to 24-6. The till layer was not fully penetrated in Boreholes 24-2, 24-3, and 24-6 but proven to extend to depths ranging from 15.2 m to 17.2 m below existing ground surface (Elevations of 83.9 m to 82.1 m). Where fully penetrated at Boreholes 24-1, 24-4 and 24-5, the till extends to depths ranging from 18.4 to 19.5 m below existing ground surface (Elevations 80.7 m to 79.6 m). Based on SPT N values, the till is in a generally loose to very dense, but generally dense to very dense state.

¹ Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

² Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release---Data 128-REV

³ Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1. ⁴ Idriss, I.M. and Boulanger, R.W. (2008) Soil Liquefaction during Earthquake. EERI Publication, Monograph MNO-12, Earthquake Engineering Research Institute, Oakland. <https://www.eeri.org/>

3.3.3 Bedrock Geology

Bedrock geology mapping indicates that the overburden materials at the site are underlain by dolostone of the Oxford Formation. Figure 4 illustrates the general bedrock geology in the area of the crossing.

Bedrock was encountered and cored at Boreholes 24-1, 24-4 and 24-5, and at depths ranging from 18.4 to 19.5 m below existing ground surface (Elevations 80.7 m to 79.6 m). The borehole records from the previous investigation indicate the bedrock was classified as sound dolostone bedrock.

3.4 Groundwater

Observation of the groundwater levels was carried out by measuring the water level in the open boreholes and monitoring the water level in piezometers installed within the native surficial deposit during the previous investigations. The water level was found at depths ranging from ground surface to 1.6 m below the existing ground surface at the time of investigation (or Elevations ranging from 98.3 m to 98.4 m). It should be noted the open borehole water level measurements are not reflective of stabilized conditions.

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

4.0 PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

4.1 General

The following sections provide preliminary foundation design recommendations based on our interpretation of the factual information obtained during the desktop study for the proposed Barnsdale Road superstructure rehabilitation/replacement project.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives for the functional design of the project.

The subsurface information used in the preparation of this technical memorandum was obtained from historical data provided in the previous Foundation Investigation Report for the design of the existing structure (GEOCRE 31G-197).

This discussion and recommendations are intended for the use of MTO and their designers for this assignment and shall not be used or relied upon for any other purpose or by any other parties, including the future construction contractor. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

4.2 Proposed Works

Based on MH's *Barnsdale Road Interchange Side Road Alternative Drawings*, dated January 2022, nine alternatives for the proposed interchange are being considered. It is understood that based on the GWP 4057-20-00 Highway 416 and Barnsdale Road Interchange Coarse Evaluation Matrix, also provided by MH, only four alternatives are being carried forward for further analysis and consideration (as shown on the drawings provided in Appendix C). The following is a summary of the proposed alternatives being carried forward based on these preliminary EA drawings:

- Alternative 2: Incorporates six new On and Off ramps, On and Off ramps at both north-west and south-east quadrants, as well as a new On-ramp to northbound Hwy 416 from westbound Barnsdale Road, and a new Hwy 416 On-ramp to southbound from eastbound Barnsdale Road at the Highway 416 and Barnsdale Road Structure. This alternative includes relocation of the Trail Road, Borrisokane Road and William McEwen Drive roadways.
- Alternative 3: Incorporates six new On and Off ramps: On and Off ramps at both the north-east and south-west quadrants; a new Off-ramp from northbound Hwy 416 to eastbound Barnsdale Road; and, a new Hwy 416 Off-ramp from southbound Hwy 416 to westbound Barnsdale Road. This alternative includes relocation of the Trail Road, Borrisokane Road and William McEwen Drive roadways.
- Alternative 5: Incorporates six new On and Off ramps, On and Off Ramps at the south-west and south-east quadrants, as well as a new Hwy 416 southbound Off-ramp to Barnsdale Road westbound, and a new Highway 416 On-ramp northbound from Barnsdale Road westbound at the Highway 416 and Barnsdale Road Structure. This alternative requires relocation of the existing Trail Road, Borrisokane Road and William McEwen Drive roadways.
- Alternative 8: Incorporates new On and Off Ramps from/to Highway 416 northbound to and from Barnsdale Road in the southeast quadrant, and new On and Off Ramps from/to Highway 416 southbound to and from William McEwen Drive. This alternative would require realignment of William McEwen Drive.

The proposed alternatives provided above may use the existing bridge, which it is understood can accommodate the above alternatives without replacement or significant rehabilitation. A replacement bridge may also be considered and the preliminary guidance below considers both retention of the existing structure and a replacement structure.

4.3 SEISMIC DESIGN

4.3.1 Seismic Hazard and Importance Category

Section 4.4.3 of the CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, it is anticipated that the bridge structure will be given an importance category of "Other" bridge.

4.3.2 Seismic Site Classification

In accordance with the Table 4.1 of the CHBDC, the selection of the seismic site classification is based on the soil and bedrock conditions encountered in the upper 30 m of the stratigraphy below the founding elevation. Based on the current understanding of the foundation conditions at the site as well as the soil conditions encountered in the upper 30 m of the stratigraphy below the founding elevation the site would be classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC. It may be possible to upgrade the classification from a Site Class D to a more favourable Site Class C, if shear wave velocity testing is carried out. Further discussion is provided in *Additional Investigation Works* Section below.

4.3.3 Liquefaction

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within silty or sandy soils under undrained conditions. In loose soil deposits, these stresses may have the potential to densify the soil (leading to potentially large surface settlements) and may generate excess pore pressures. The excess pore pressures can lead to sudden temporary losses in shear strength.

The site is underlain by deposits of silt and sand, over sand and gravel glacial till. SPT N values within these deposits generally range from loose (as low as 5) to very high (greater than 50) generally increasing with depth. Very low SPT test results in granular soil can often be caused by disturbance due to drilling, or unbalanced porewater pressures during testing. Similarly, very high SPT test results are often a result of cobbles and boulders, and not the density of the soil matrix itself. The average SPT N values for the site are in the range of 10 to 50 blows per 300 mm of penetration, with lower and higher values distributed both horizontally and vertically throughout the boreholes.

A preliminary seismic liquefaction assessment was completed for the site. The methodology used to assess liquefaction potential is consistent with the “simplified” approach outlined by Idriss and Boulanger (2008)⁴. It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction assessment was carried out using the in situ SPT data collected at both locations. The design groundwater level was based on the average recorded groundwater elevation in the 1989 investigation of 98.5 m. The CRR with depth was calculated using the distribution of SPT N values, estimated pore water pressure, and estimated fines content (based on visual observations and local experience). The assessment was based on a site specific peak ground acceleration of 0.295 g (corresponding to a seismic event with a 2% probability of exceedance in 50 years).

Based on the typical range of SPT N values, the site is likely not considered to be at large-scale risk of seismic liquefaction. Although there are low SPT N values recorded in boreholes 24-5 to 24-10, they appear to be relatively localized (i.e., they are not indicative of a particular zone or layer of very loose soil and are more likely indicative of drilling and testing disturbance or random variations in the soil).

4.4 FOUNDATION OPTIONS

4.4.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the existing underpass structure and foundation systems may be classified as having medium to large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a “typical” consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Section 3.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a “low” degree of site and prediction model understanding” for these sites. Accordingly, the appropriate corresponding ULS and SLS consequence factor, γ of 1.0, and geotechnical resistance factors from Table 6.2 of the CHBDC have been used for design, as indicated in the following sections.

⁴ Idriss, I.M. and Boulanger, R.W. (2008) Soil Liquefaction during Earthquake. EERI Publication, Monograph MNO-12, Earthquake Engineering Research Institute, Oakland. <https://www.eeri.org/>

As per Section 6.14.4 of the CHBDC for seismic design the consequence factor, Ψ , should be taken as 1.0 while the resistance factor, ϕ_{gu} , should be taken from Table 6.3 based on the structural design approach.

4.4.2 Frost Protection

The native subgrade soils at this site are considered to be frost susceptible. As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario), the design frost penetration depth at the site should be taken as 1.8 m below the existing ground surface. Footings constructed at this site or the underside of pile caps should have a minimum embedment depth of 1.8 m for frost protection purposes.

It is understood that the widening for the speed change ramps may require excavation of the embankment slope in front of the existing bridge abutments. Where a minimum of 1.8 m of frost protection (in any direction) cannot be provided for the existing pile caps, insulation will be required to protect the piles and any foundation elements. Insulation, such as high density polystyrene rigid foam insulation with a suitable protective covering, of the slope surface and abutment wall could be considered as an alternative to earth cover for frost protection. Additional guidance on insulation details can be provided if and when required but require further understanding of the proposed design.

4.4.3 Foundation Design Alternatives

The results of the desktop review indicate that the site soil stratigraphy consists of native sand and silt, overlying a layer of dense glacial till underlain by dolostone bedrock.

Key elevations are as follows:

- Existing ground surface elevation of Highway 416 is approximately Elevation 99 m
- Top of the sand and silt deposit is at Elevations ranging from 99.0 to 98.0 m
- Top of glacial till deposit is at Elevations ranging from 87.7 to 85.1 m
- Top of bedrock surface is at Elevations ranging from 80.7 m to 79.6 m

Based on the results of the measured SPT N values, the silt and sands are generally in a loose to dense state of compactness. The silt and sand deposit therefore has insufficient strength to support the anticipated foundation loads associated with the proposed abutments and piers if supported on shallow foundations.

The glacial till deposit generally consists of a generally dense layer of sand and gravel containing cobbles and boulders. The till would be generally feasible to support the proposed abutments and piers, however the relatively deep excavations required to reach the till surface make founding on the till impractical from a constructability standpoint.

Based on the foregoing, shallow foundations are not considered to be a feasible foundation option and therefore the foundations would need to be supported on the dolostone bedrock. Therefore, deep foundations, founded on or in the bedrock, are anticipated for this site. Piles will be required as the depth to the bedrock surface is between 18 and 20 m below the top of pavement elevation of Highway 416.

4.4.3.1 *Deep Foundations*

Conceptually, the following types of deep foundations could be used at this site to support foundations:

- Driven steel piles (typically H sections or pipe piles) to bedrock; or,
- Drilled, cast-in-place concrete piles with rock sockets.

Driven steel piles (either single piles or in groups) are often the most cost-effective in terms of supporting vertical loads and are commonly used in the area.

Drilled, cast-in-place piles are less common, but are used on a variety of projects. Drilled piles are not as ideally suited to the conditions at this site since the drilling conditions are not ideal (the fill and glacial till contains cobbles and boulders, high water table) and there is a need to deal with excess soil and groundwater during construction. Drilled piles are however feasible if properly designed and constructed and do have some advantages in that they can be designed to carry very large loads. This is particularly relevant if there are large lateral and uplift loads because the presence of a rock socket provides significant resistance compared to driven piles.

Piles driven to sound rock generate high ultimate geotechnical capacities, generally equal to or in excess of the structural capacity of the steel section (i.e., with increased loading or driving stresses, the steel section will become damaged and fail before the bedrock yields). For the purpose of design, the minimum factored ultimate geotechnical resistance may be assumed to be equal to the ultimate resistance of the steel section. This will exceed the factored structural capacity of the pile estimated by the structural engineers.

If drilled piles are used to support the new structure(s), they will be drilled through the overburden (fill and glacial till) into the underlying dolostone bedrock. Casing will be required to advance the piles through the soil. The casing should be extended so that it is “seated” a minimum of 500 mm into the bedrock.

Due to the difficulty in socketing liners into the limestone bedrock to completely cut off the water infiltration, it may not be feasible to dewater and clean the base of the caisson, or to inspect the base prior to concreting. As such, end-bearing support may not be fully developed and should be neglected in the design, unless additional measures are taken during construction to confirm the base of the socket has been adequately cleaned. The axial geotechnical resistance for rock-socketed caissons is therefore generally assumed to be based on the sidewall (shaft) resistance of the rock socket rather than end-bearing (there is a contribution to shaft resistance from the soil overburden, but it is small compared to the rock socket and it typically neglected). Rock-socketed caissons may therefore be designed based on a factored preliminary geotechnical shaft resistance at ULS of 1.0 MPa for piles socketed into sound rock. For preliminary design this condition can be assumed to be met at 0.5 m below the bedrock surface. This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

Settlements for piles driven to or socketed into sound rock are generally negligible, and the geotechnical resistance mobilized at 25 mm of settlement (a typical SLS condition) would be expected to exceed the factored axial resistance at ULS. Geotechnical SLS considerations therefore do not generally govern the design of pile driven to sound rock.

4.4.3.1.1 Lateral Resistance for New Construction

For new construction, the following preliminary guidance should be considered for lateral pile resistance.

The lateral resistance of a slender pile is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter that may be used to preliminarily determine lateral deflection of piles is the coefficient of horizontal subgrade reaction (k_h). For this site, k_h may be assumed to be:

$$k_h = \eta_h z$$

Where:

k_h = the modulus of subgrade reaction (kN/m³);

η_h = a coefficient based on soil type (use 4.4 MPa/m); and,

z = the depth under consideration

The value above is for a single pile group. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal reaction (k_h) by an appropriate factor as follows:

Table 2: Coefficient of Horizontal Subgrade Reaction Reduction Factors

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other spacings may be interpolated from the values above. No reduction is required for the first row of piles (i.e., the row which bears against undisturbed soil with no piles in front).

It should be noted that the method of applying a linear “spring” to represent the soil reaction to loading is a significant simplification of the soil/pile behaviour. If lateral load resistance governs the pile design, more rigorous, non-linear methods of analysing resistance exist, one common one being the method of p-y curves. These methods, however, require knowledge of the pile size, location, loading, pile cap construction, etc. and are therefore typically more suited to the detailed design phase when these items are known. Golder can provide additional assistance during detailed design, if required.

If the existing bridge is to be retained, the lateral resistance of the existing pile system should be evaluated based on the changes in loading. In particular, excavation of the embankment slope in front of the abutment walls will result in an increase in the net lateral earth pressure acting on the abutments and this additional loading will need to be evaluated against the available resistance.

4.4.3.2 Lateral Resistance of Existing Piles

It is understood that an inspection of the upper 4 m of the front piles will be carried out as part of the structural evaluation for the reuse of the existing piles. A review of the existing construction drawings was carried out to determine if the back row of piles would be fully laterally supported. Based on the existing General Arrangement and Footing Details drawings provided in Appendix B the existing foundation conditions are as follows:

- Existing abutment pile cap is 4.0 m wide
- Front 9 piles are battered to the front at 1H:4V with an approximate length of 23 m
- Spacing between the front piles is approximately 1.1 m
- Back 5 piles are battered at 1H:10V alternating back and front facing batters with an approximate length of 22 m
- Center-to-center distance between the front and back piles is 3.0 m. Meaning once the soil is removed from the in front of the front row of piles there will be 3 m of soil remaining in front of the back row of tiles
- Embankment was constructed with granular material with a maximum particle size of 75 mm

Based on the conditions outlined above and a maximum excavation depth of 4.0 m, there should be sufficient soil (a minimum of 3 pile diameters, or 1.5 m) in front of the back row of piles for them to be considered fully laterally supported when the front piles are exposed. However, this assumes that the soil between the front row of piles is not disturbed. The contract must ensure that sloughing of the soil out from between the piles does not occur. This will likely require shoring between the piles.

4.5 Re-use of Existing Foundations

Conceptually from a foundations perspective, the existing piles could be used to support a new structure on the same alignment. The General Arrangement and Footing Details drawings for the existing bridge indicate the bridge is supported on HP 310 x 110 piles driven to the surface of the bedrock. The assumed geotechnical resistance is not provided on the drawings but the structural loading on the piles is noted as 1,400 kN and 1,150 kN for ULS and SLS, respectively. The driving criteria for the piles has also not been provided.

Based on the type of rock (i.e., dolostone) and assuming the rock is fair quality or better and it can be confirmed that the piles were driven to a reasonable set on the bedrock surface, the geotechnical resistance should at least equal the structural capacity of the piles. The structural capacity would need to be confirmed, after suitable corrosion testing and/or field measurements (after exposing piles) to assess the potential section loss, by a structural engineer.

4.6 Embankment Settlement and Stability

New embankments will be required for the interchange ramps. Assuming the ramp embankments are no higher than the existing and are provided with side slopes no steeper than 2 horizontal to 1 vertical the following should apply:

- The embankments will likely have factors of safety against global instability under both static and seismic loading conditions of at least 1.5 and 1.1, respectively; and,

- The post-paving settlements upon completion of the construction should be meet MTO's requirements for non-freeways (e.g., less than 25 mm within 20 m of the bridge abutments) without the need for preloading or other mitigation measures.

The above guidance is preliminary since it is based on the available existing information (which does not include boreholes advanced at the ramp locations) and must be confirmed during detailed design.

4.7 Construction Considerations

4.7.1 Excavations

Excavations to depths of approximately 1.8 m below grade are anticipated to install founding elements below the frost penetration depth at the site.

The soils at this site would be generally classified as Type 3 soils (loose to compact sands and silts above groundwater level) in accordance with the OHSA. Accordingly, excavations should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Any native sand and silt which extends below the water table would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V, unless adequate groundwater control is in place to allow 1H:1V side slopes.

4.7.2 Temporary Protection Systems

If the required safe side slopes for open cut excavations cannot be accommodated, then temporary roadway protection systems (i.e., excavation shoring) will be required to facilitate excavation to the foundation level.

The design of the shoring will be entirely the responsibility of the contractor. Where a protection system is required, the support system should be designed and constructed in accordance with OPSS.PROV 539 (Construction Specification for Temporary Protection Systems). The lateral movement of the temporary protection system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any utilities that may be present in the area can tolerate this magnitude of deformation. Traffic loading should be included as a surcharge. Traffic loading does not account for construction equipment loadings, which may be higher; the contractor's shoring designer should confirm those load requirements.

A conventional shoring system consisting of interlocking steel sheet piling supported against lateral movement using walers, tie backs (into the glacial till or underlying bedrock, if applicable) and/or internal struts/braces is considered feasible.

4.7.3 Embankment Reinstatement

Embankment reinstatement should be carried out in accordance with OPSS.PROV 206 (*Construction Specification for Grading*) and should match the adjacent slope geometry. The new embankment material should consist of imported Granular B Type II material. Excavated granular fill may also be reused as embankment fill provided there is no organic material in the excavated fill and there is sufficient space to stockpile on site and control the moisture content within acceptable limits for compaction.

Granular fill should be placed and compacted in accordance with OPSS.PROV 501 (*Construction Specification for Compacting*). Where new embankment fill is placed against existing embankment slopes the existing earth or fill slope must be benched in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

4.8 Additional Investigation Works

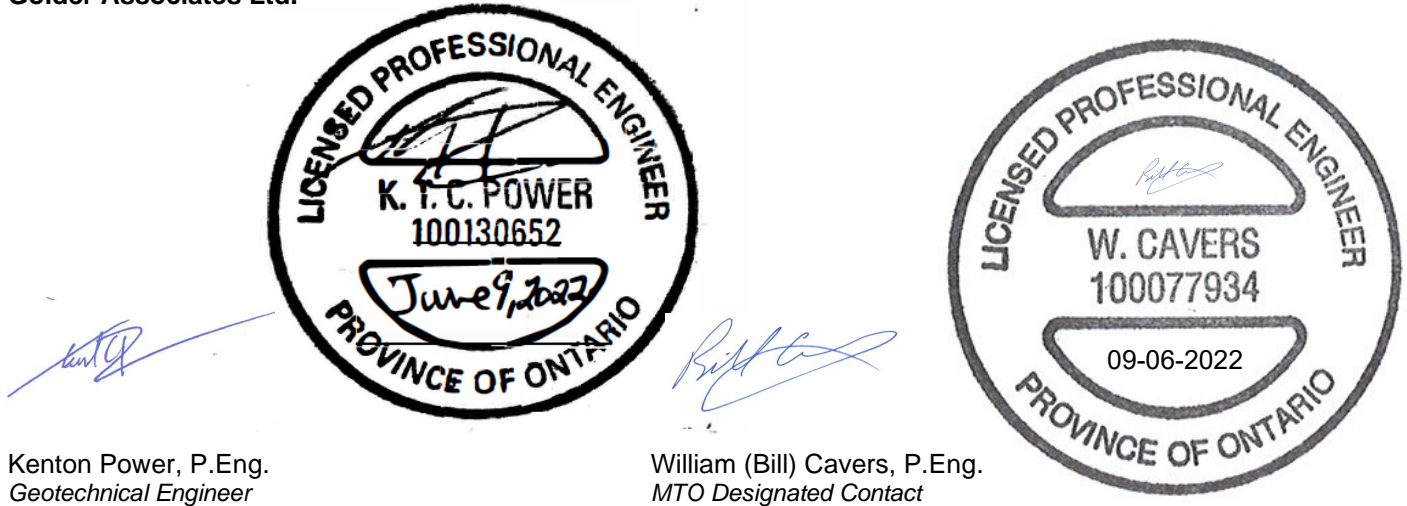
During the future detailed design, further foundation investigation and analysis will be warranted once the proposed rehabilitation plans have been finalized. It is recommended that the detailed geotechnical investigation include the following:

- An assessment of the thickness and geotechnical properties of the fill, native soil and bedrock by drilling a minimum of four boreholes on Barnsdale Road – two behind each abutment, and one borehole at the location of the pier on Highway 416 – to supplement the existing borehole information. Coring and laboratory testing to determine the quality and strength of the existing bedrock should also be carried out to confirm the geotechnical resistances.
- It may be beneficial depending on the proposed rehabilitation plan to carry out Multi-Channel Analysis of Surface Wave (MASW) testing to assess the average shear wave velocity of the 30 m of soil/bedrock beneath at one of the existing abutment foundations. The site specific shear wave velocity profile may make it possible to upgrade the seismic site class provided and to further assess the liquefaction potential.
- Should re-use of the existing pile foundations be considered, additional investigations to confirm the potential pile geotechnical resistance (such as geophysics to confirm the piles extend to rock) should be considered. The potential section loss of the existing piles should also be investigated by undertaking additional corrosion testing and/or exposing one or more piles for measurement of the sections.

5.0 CLOSURE

This memo was prepared by Kenton Power, P.Eng. and reviewed by Bill Cavers, P.Eng. a Senior Geotechnical Engineer with Golder and the Designated MTO Foundations Contact for this project.

Golder Associates Ltd.



Kenton Power, P.Eng.
Geotechnical Engineer

William (Bill) Cavers, P.Eng.
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KCP/WC/hdw

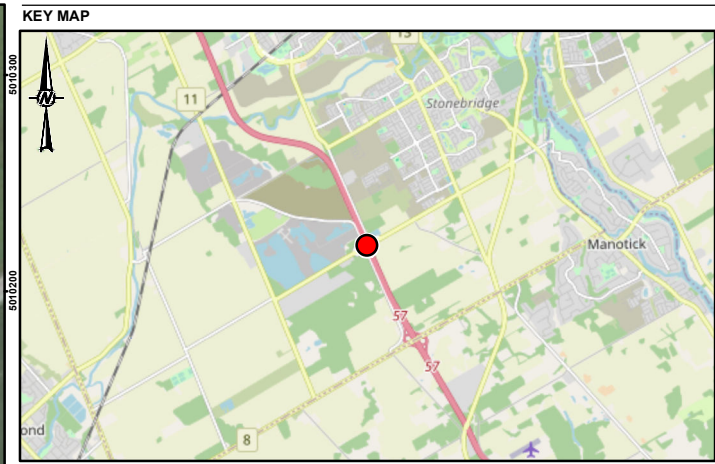
Distribution:

1. E-copy to Dillon Consulting Limited
2. E-copy to MTO
3. Golder Associates Ltd.

Attachments:


- Figure 1: Site Plan
- Figures 2 to 5 soil and bedrock geology figures
- Appendix A: 31G-197 Borehole Location and Soil Strata Drawing and Previous Record of Boreholes
- Appendix B: 31G-197 Construction Drawings
- Appendix C: EA Alternatives Highway 416 at Barnsdale Road


[https://golderassociates.sharepoint.com/sites/155319/project files/6 deliverables/1-desktop/3-final/21501071 rev0 hwy 416 barnsdale rd memo 2022-06-03.docx](https://golderassociates.sharepoint.com/sites/155319/project%20files/6%20deliverables/1-desktop/3-final/21501071%20rev0%20hwy%20416%20barnsdale%20rd%20memo%202022-06-03.docx)



SCALE 1:150,000

LEGEND

 EXISTING BOREHOLE LOCATION (GEOCRES No. 31G-197)

 APPROXIMATE SITE LOCATION

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

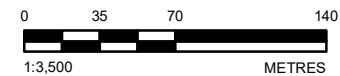
REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020

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SOURCE: ESRI, MAXAR, GEOEYE, EARTHSTAR GEOGRAPHICS, CNES/AIRBUS DS, USDA, USGS, AEROGRID, IGN, AND THE GIS USER COMMUNITY

3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28



CLIENT

MORRISON HERSHFIELD

PROJECT

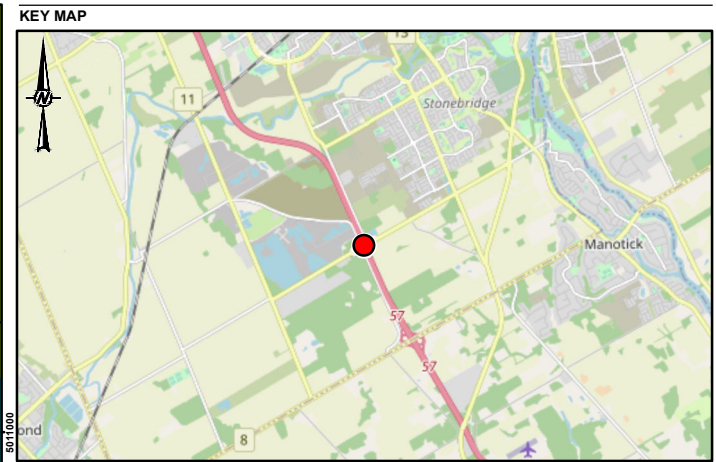
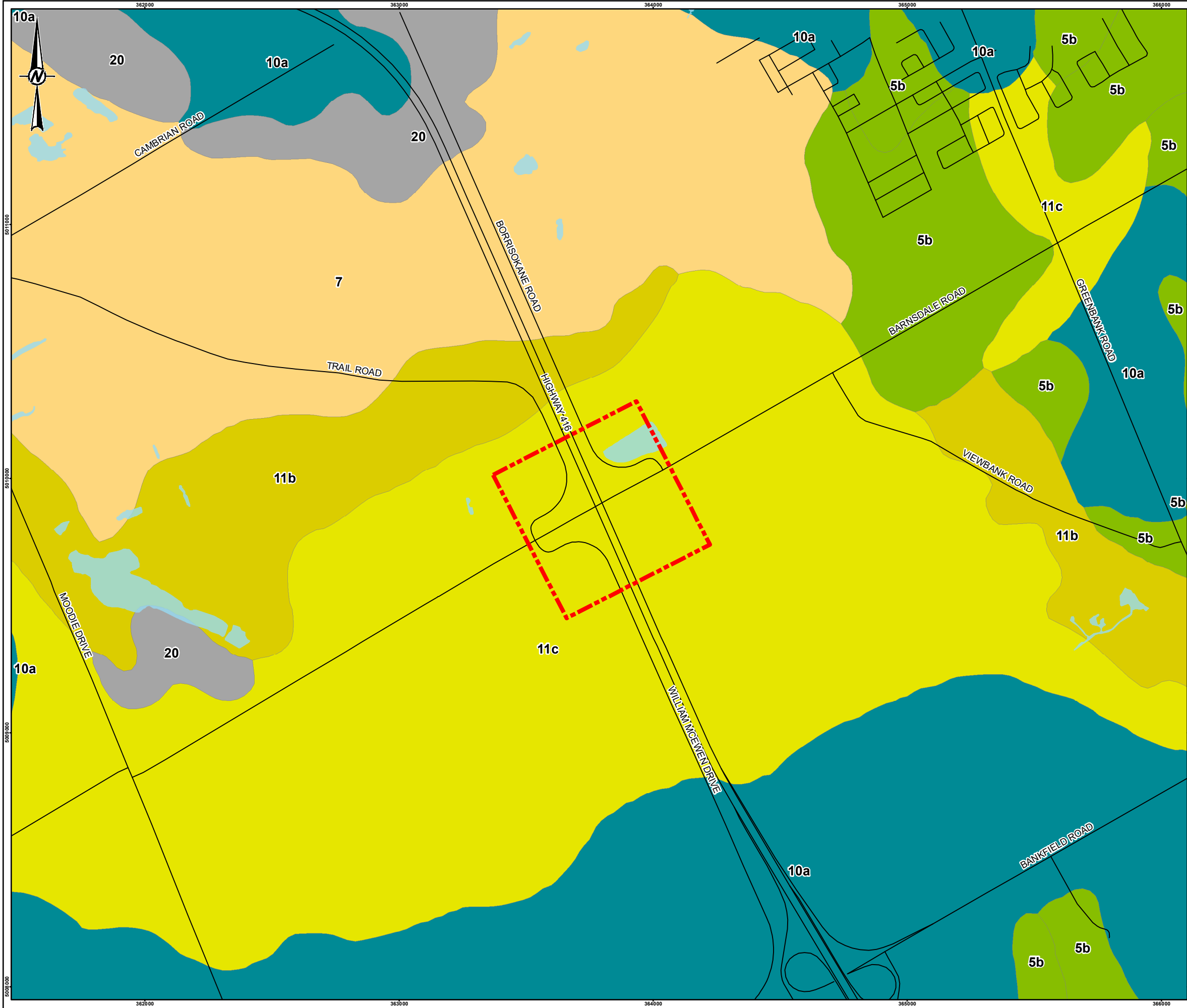
DESKTOP FOUNDATION REVIEW
BARNSDALE RD INTERCHANGE
HIGHWAY 416, NEPEAN, ONTARIO

TITLE

SITE PLAN

CONSULTANT	YYYY-MM-DD	2022-02-14
 GOLDER MEMBER OF WSP	DESIGNED	---
	PREPARED	MG
	REVIEWED	KM
	APPROVED	WC

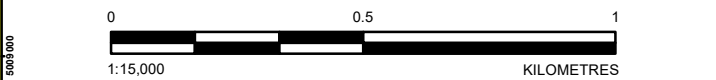
PROJECT NO. 21501071	CONTROL 0001	REV. 0	FIGURE 1
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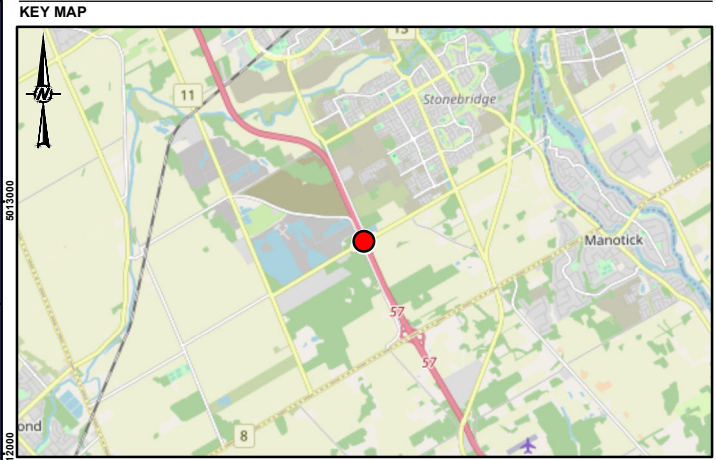
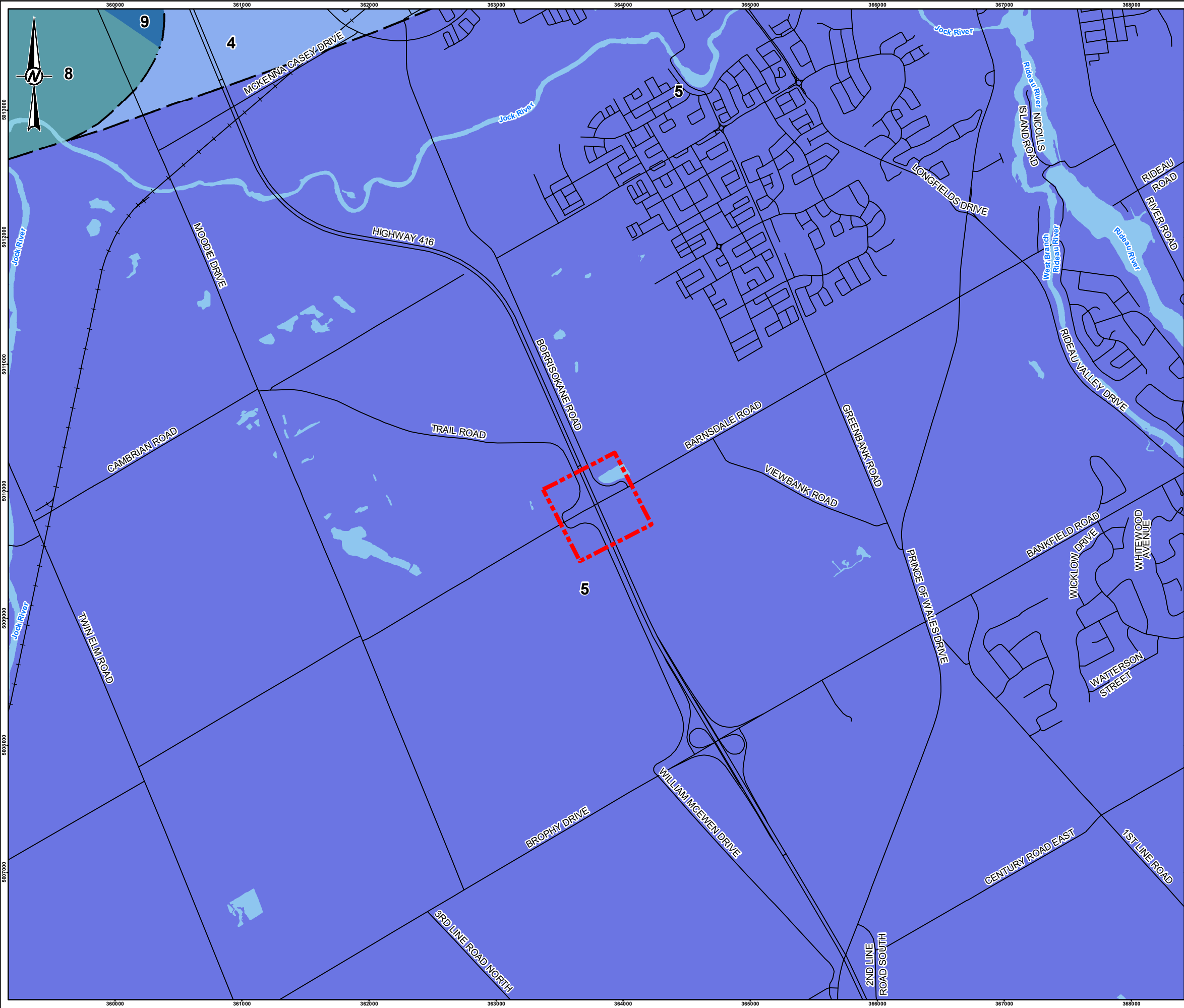
SCALE 1:150,000

- LEGEND**
- ROADWAY
 - WATERBODY
 - 5b: STONE-POOR, CARBONATE-DERIVED SILTY TO SANDY TILL
 - 7: GLACIOFLUVIAL DEPOSITS
 - 10a: MASSIVE-WELL LAMINATED
 - 11b: LITTORAL-FORESHORE DEPOSITS
 - 11c: FORESHORE-BASINAL DEPOSITS
 - 20: ORGANIC DEPOSITS
 - APPROXIMATE SITE LOCATION

- NOTE(S)**
1. ALL LOCATIONS ARE APPROXIMATE
- REFERENCE(S)**
1. ONTARIO GEOLOGICAL SURVEY 2010. SURFICIAL GEOLOGY OF SOUTHERN ONTARIO; ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE-DATA 128-REV
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
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4. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28



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MORRISON HERSHFIELD			
PROJECT			
DESKTOP FOUNDATION REVIEW BARNSDALE RD INTERCHANGE HIGHWAY 416, NEPEAN, ONTARIO			
TITLE			
SURFICIAL GEOLOGY			
CONSULTANT		YYYY-MM-DD	2022-02-04
 GOLDER MEMBER OF WSP		DESIGNED	---
		PREPARED	MG
		REVIEWED	KM
		APPROVED	WC
PROJECT NO.	CONTROL	REV.	FIGURE
21501071	0001	0	3



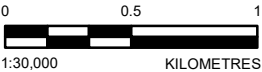
SCALE 1:150,000

LEGEND

- ROADWAY
- RAILWAY
- FAULT
- WATERBODY
- 9: BOBCAYGEON FORMATION
- 8: GULL RIVER FORMATION
- 5: OXFORD FORMATION
- 4: MARCH FORMATION
- APPROXIMATE SITE LOCATION

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)
1. ARMSTRONG, D.K. AND DODGE, J.E.P. 2007. PALEOZOIC GEOLOGY OF SOUTHERN ONTARIO; ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE--DATA 219
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEEN'S PRINTER 2020
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CLIENT
MORRISON HERSHFIELD

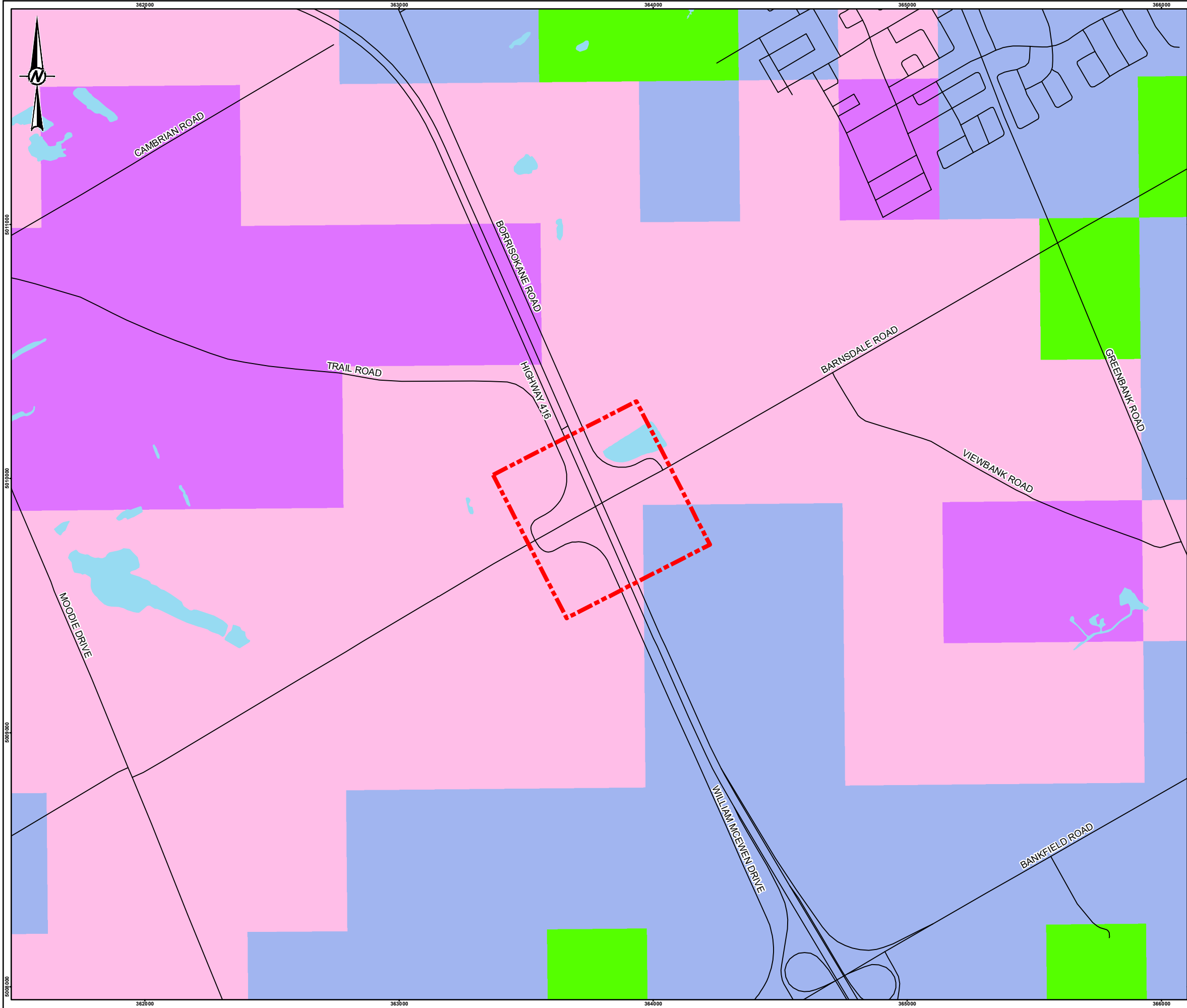
PROJECT
DESKTOP FOUNDATION REVIEW
BARNSDALE RD INTERCHANGE
HIGHWAY 416, NEPEAN, ONTARIO

TITLE
BEDROCK GEOLOGY

CONSULTANT	YYYY-MM-DD	2022-02-04
DESIGNED	---	
PREPARED	MG	
REVIEWED	KM	
APPROVED	WC	

GOLDER
MEMBER OF WSP

PROJECT NO. 21501071	CONTROL 0001	REV. 0	FIGURE 4
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KEY MAP

LEGEND

ROADWAY

WATERBODY

APPROXIMATE SITE LOCATION

TREND IN DEPTH TO BEDROCK (METRES)

- 5 - 10
- 10 - 15
- 15 - 25
- 25 - 50

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. BEDROCK TOPOGRAPHY AND OVERBURDEN THICKNESS MAPPING, SOUTHERN ONTARIO, ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE - DATA 207
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
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4. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 0.5 1
1:15,000 KILOMETRES

CLIENT
MORRISON HERSHFIELD

PROJECT
DESKTOP FOUNDATION REVIEW
BARNSDALE RD INTERCHANGE
HIGHWAY 416, NEPEAN, ONTARIO

TITLE
DRIFT THICKNESS

CONSULTANT	YYYY-MM-DD	2022-02-04
DESIGNED	---	
PREPARED	MG	
REVIEWED	KM	
APPROVED	WC	

GOLDER
MEMBER OF WSP

PROJECT NO.
21501071

CONTROL
0001

REV.
0

FIGURE
5

APPENDIX A

List of Abbreviations and Symbols (MTO)
Borehole Location and Soil Strata 31G-197
Record of Borehole Logs, 31G-197

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (i.e., SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (i.e., some sand)
≤ 10	trace (i.e., trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
E	void ratio
N	porosity
S	degree of saturation

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

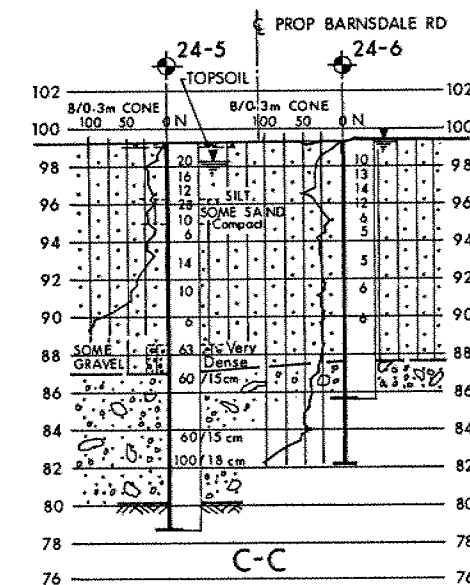
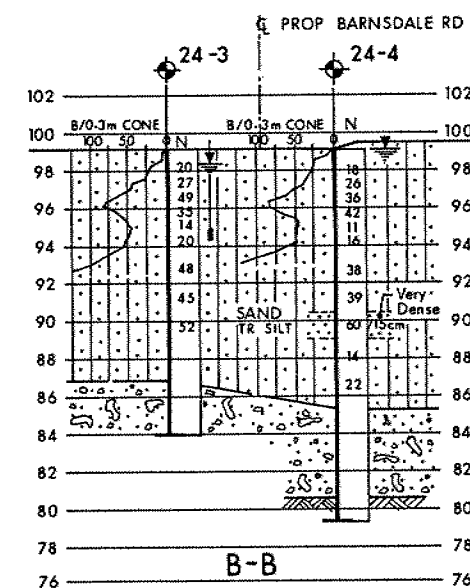
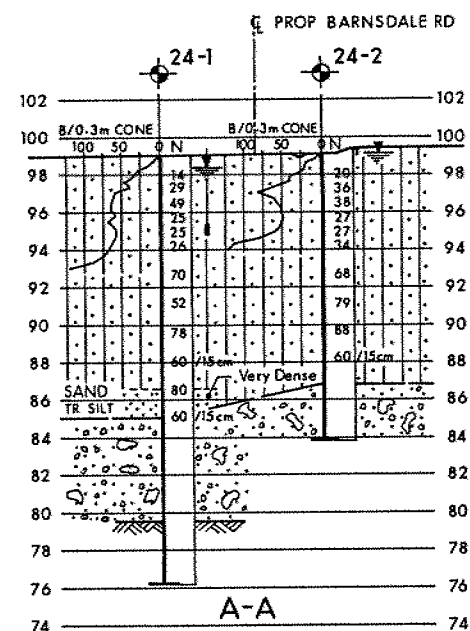
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

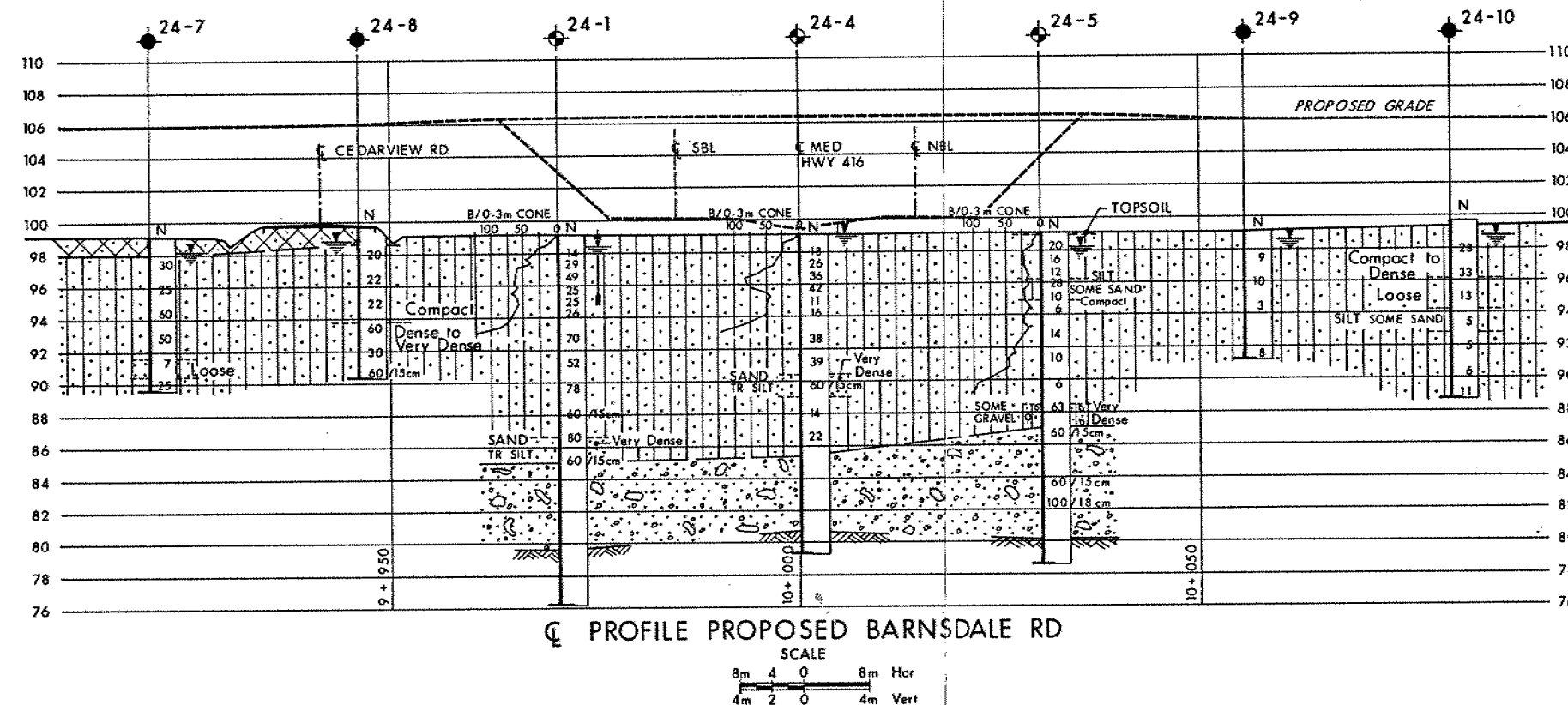
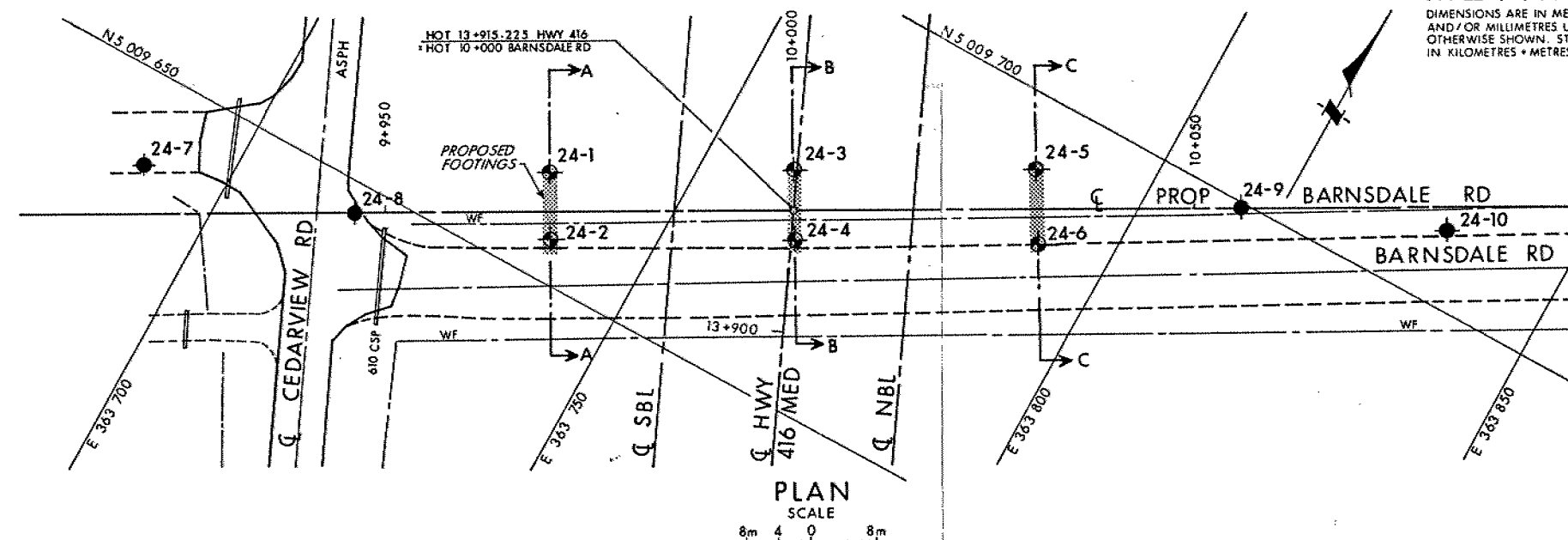
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

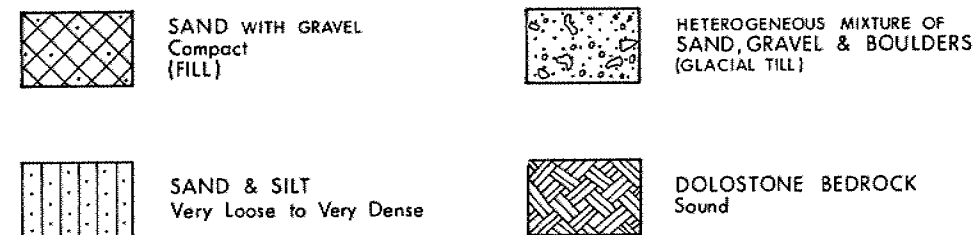
JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	



SCALE
4m 2 0 4m



SOIL STRATIGRAPHY LEGEND



METRIC

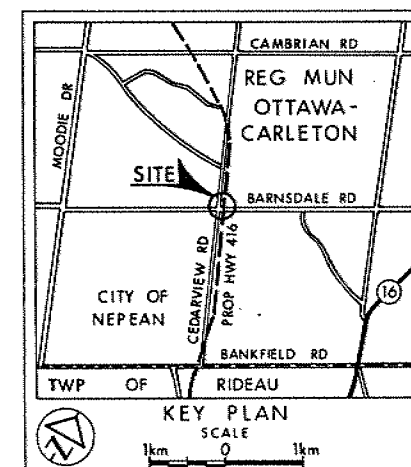
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 128-87-10

BARNSDALE RD UNDERPASS
(STRUCTURE -24)
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation 89 05
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
24-1	99.1	5 009 662.4	363 734.1
24-2	99.0	5 009 655.1	363 737.2
24-3	99.1	5 009 677.0	363 760.3
24-4	99.1	5 009 669.5	363 764.6
24-5	99.2	5 009 691.5	363 786.6
24-6	99.3	5 009 683.4	363 791.3
24-7	99.2	5 009 639.0	363 689.8
24-8	99.8	5 009 646.4	363 715.5
24-9	99.2	5 009 699.3	363 811.0
24-10	99.7	5 009 708.9	363 834.4

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
Geocres No 31G-197		
HWY No 416		DIST 9
SUBMD TS	CHECKED	DATE 89 07 04 SITE 3-552
DRAWN DT	CHECKED	APPROVED DWG 128 8710-A

RECORD OF BOREHOLE No 24-1

METRIC

W P 128-87-10

LOCATION Co-ords: N 5 009 662.4; E 363 734.1

ORIGINATED BY TS

DIST 9 HWY 416

BOREHOLE TYPE HS Auger, B-Casing, Rock Coring, Cone Test

COMPILED BY TS

DATUM Geodetic

DATE 89 05 12-13

CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
99.1	Ground Surface										
0.0			1	SS	14		98				
			2	SS	29						
			3	SS	49						
			4	SS	25		96				
			5	SS	25						
			6	SS	26						
	Sand and Silt Grey Compact to V. Dense		7	SS	70						
			8	SS	52						
			9	SS	78						
			10	SS	60	15 cm	88				
			11	SS	80						
	Sand, Tr. Silt V. Dense		12	SS	60	15 cm	86				
85.1			13	BXL RC	15% Rec		84				
14.0			14	BXL RC	18% Rec						
	Het. Mixt. of Sand, Gravel and Boulders (Glacial Till)		15	BXL RC	18% Rec		82				
			16	BXL RC	78% Rec		80				
79.6			17	BXL RC	100% Rec		78				
19.5			18	BXL RC	100% Rec						
	Dolostone Bedrock Sound										
76.2											
22.9	End of Borehole										

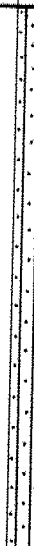
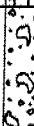

+3, x5: Numbers refer to
Sensitivity
20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 24-2

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 655.1; E 363 737.2 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, B-Casing, Rock Coring, Cone Test COMPILED BY TS
 DATUM Geodetic DATE 89 05 15-16 CHECKED BY _____

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L	WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
99.0	Ground Surface												
0.0	Sand and Silt Grey Compact to V. Dense		1	SS	20								
			2	SS	36								
			3	SS	38								
			4	SS	27								
			5	SS	27								
			6	SS	34								0 58 (42)
			7	SS	68								0 58 (42)
			8	SS	79								
			9	SS	88								0 54 (46)
			10	SS	60								
86.8	Het. Mixt. of Sand, Gravel and Boulders (Glacial Till)		11	BXL RC	9% Rec							RQD = 0%	
12.2			12	BXL RC	10% Rec								RQD = 0%
83.8	End of Borehole												
15.2													

RECORD OF BOREHOLE No 24-3

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 677.0; E 363 760.3 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BW-Casing, Rock Coring, Cone Test COMPILED BY TS
 DATUM Geodetic DATE 89 05 17 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
99.1	Ground Surface												
0.0	Sand and Silt Grey, Compact to V. Dense		1	SS	20								0 51 (49)
			2	SS	27								
			3	SS	49								
			4	SS	35								
			5	SS	14								
			6	SS	20								
			7	SS	48								
			8	SS	45								
			9	SS	52								
86.9	Het. Mixt. of Sand, Gravel and Boulders (Glacial Till)		10	BXL RC	20% Rec								RQD = 0%
12.2			11	BXL RC	5% Rec								
83.9													RQD = 0%
15.2	End of Borehole												

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 24-4

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 669.5; E 363 764.6 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BW-Casing, Rock Coring, Cone Test COMPILED BY TS
 DATUM Geodetic DATE 89 05 13-14 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT (%)			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _p W W _L	W _p W W _L	10 20 30			
99.1	Ground Surface												
0.0	Sand and Silt Grey, Compact to Dense	1	SS	18									
		2	SS	26									
		3	SS	36									
		4	SS	42									
		5	SS	11									0 55 (45)
		6	SS	16									0 53 (47)
		7	SS	38									
		8	SS	39									
		9	SS	60		15 cm							0 88 (12)
		Sand, Tr. Silt V. Dense	10	SS	14								
		11	SS	22									
85.4	Het. Mixt. of Sand, Gravel and Boulders (Glacial Till)	12	SS	*									
13.7		13	BXL RC	6% Rec									RQD = 0%
		14	BXL RC	5% Rec									RQD = 0%
		15	BXL RC	18% Rec									RQD = 0%
80.7	Dolostone Bedrock, Sound	16	BXL RC	98% Rec									RQD = 78%
18.4													
79.3													
19.8	End of Borehole												
	*Sampler Bouncing												

*3, *5: Numbers refer to Sensitivity
 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 24-5

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 691.5; E 363 786.6
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BW-Casing, Rock Coring, Cone Test
 DATUM Geodetic DATE 89 05 12-13
 ORIGINATED BY TS
 COMPILED BY TS
 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
99.2	Ground Surface													
0.0	Topsoil		1	SS	20									
	Brown Grey		2	SS	16									
			3	SS	12									
	Silt, Some Sand		4	SS	28									
	Compact		5	SS	10									
			6	SS	6									
	Sand and Silt		7	SS	14									
	Loose to Compact		8	SS	10									
			9	SS	6									
	Some Gravel		10	SS	63									
87.0	V. Dense		11	SS	60	15 cm								
12.2			12	BXL RC	38% Rec									
	Het. Mixt. of Sand, Gravel and Boulders		13	SS	60	15 cm								
	(Glacial Till)		14	SS	100	18 cm								
			15	BXL RC	22% Rec									
			16	BXL RC	48% Rec									
80.1	Dolostone Bedrock		17	BXL RC	94% Rec									
19.1	Sound													
78.6														
20.6	End of Borehole													

+3, x5: Numbers refer to Sensitivity
 20
 15 5 (%) STRAIN AT FAILURE
 10

RECORD OF BOREHOLE No 24-6

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 683.4; E 363 791.3
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BW-Casing, Washboring & Cone Test
 DATUM Geodetic DATE 89 05 16
 ORIGINATED BY TS
 COMPILED BY TS
 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
99.3	Ground Surface												
0.0	Sand and Silt Grey Loose to Compact		1	SS	10								
			2	SS	13								
			3	SS	14								
			4	SS	12								
			5	SS	6								
			6	SS	5								
			7	SS	5								
			8	SS	6								
			9	SS	6								
87.7	Het. Mixt. of Sand Gravel and Boulders (Glacial Till)		10	RC	23%	Rec							
11.6			11	BXL	10%	Rec							
85.6													
13.7	End of Borehole												
82.1													
17.2	End of Cone Test												

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 24-7

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 639.0; E 363 689.8 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, Washbore COMPILED BY TS
 DATUM Geodetic DATE 89 05 14 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80					
99.2	Ground Surface															
0.0	Sand with Gravel Brown, Compact (Fill)		1	AS	-											
98.0			2	SS	30											
1.2	Sand and Silt Grey Compact to V. Dense		3	SS	25											0 53 (47)
			4	SS	60											
			5	SS	50											
			6	SS	7											1 67 (32)
	Loose		7	SS	25											
89.6	End of Borehole															

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 24-8

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 646.4; E 363 715.5 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, Washbore COMPILED BY TS
 DATUM Geodetic DATE 89 05 14 CHECKED BY _____

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
99.8	Ground Surface													
0.0	Sand with Gravel Brown, Compact (Fill)		1	AS	-									
98.6			2	SS	20									
1.2	Sand and Silt Grey		3	SS	22									0 54 (46)
			4	SS	22									
	Compact Dense to V. Dense		5	SS	60									0 54 (46)
			6	SS	30									
90.3			7	SS	60	15 cm								
9.5	End of Borehole													

+3, x5: Numbers refer to Sensitivity
 20
 15 5 (%) STRAIN AT FAILURE
 10

RECORD OF BOREHOLE No 24-9

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 699.3; E 363 811.0 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, Washbore COMPILED BY TS
 DATUM Geodetic DATE 89 05 17 CHECKED BY _____

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60					
99.2	Ground Surface														
0.0	Sand and Silt V. Loose to Compact - Brown Grey		1	SS	9										0 30 66 4
			2	SS	10										
			3	SS	3										0 52 (48)
91.1			4	SS	8										
8.1	End of Borehole														

+³, x⁵: Numbers refer to Sensitivity
 20
 15 ϕ 5 (%) STRAIN AT FAILURE
 10

RECORD OF BOREHOLE No 24-10

METRIC

W P 128-87-10 LOCATION Co-ords: N 5 009 708.9; E 363 834.4 ORIGINATED BY TS
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, Washbore COMPILED BY TS
 DATUM Geodetic DATE 89 05 14 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60					
99.7	Ground Surface														
0.0															
	Sand and Silt		1	AS	-										
			2	SS	28										
	Compact to Dense		3	SS	33										
	Loose		4	SS	13										
	Silt, Some Sand		5	SS	5										
			6	SS	5										
			7	SS	6										
88.6			8	SS	11										
11.1	End of Borehole														

+3, x5: Numbers refer to Sensitivity
 20
 15 10-5 (%) STRAIN AT FAILURE
 10

APPENDIX B

Construction Drawings 1 and 3
(GEOCRES No. 31G-197)

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. 9
CONT No
WP No 128-87-10



BARNSDALE RD. UNDERPASS
GENERAL ARRANGEMENT

SHEET

LIST OF ABBREVIATIONS:
W.P.-WORKING POINT
T/P-TOP OF PAVEMENT

NOTES

CLASS OF CONCRETE

PIER AND DECK 35 MPa
REMAINDER 30 MPa

REINFORCING STEEL

GRADE 400 UNLESS OTHERWISE SPECIFIED.
BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.

CLEAR COVER TO REINFORCING STEEL

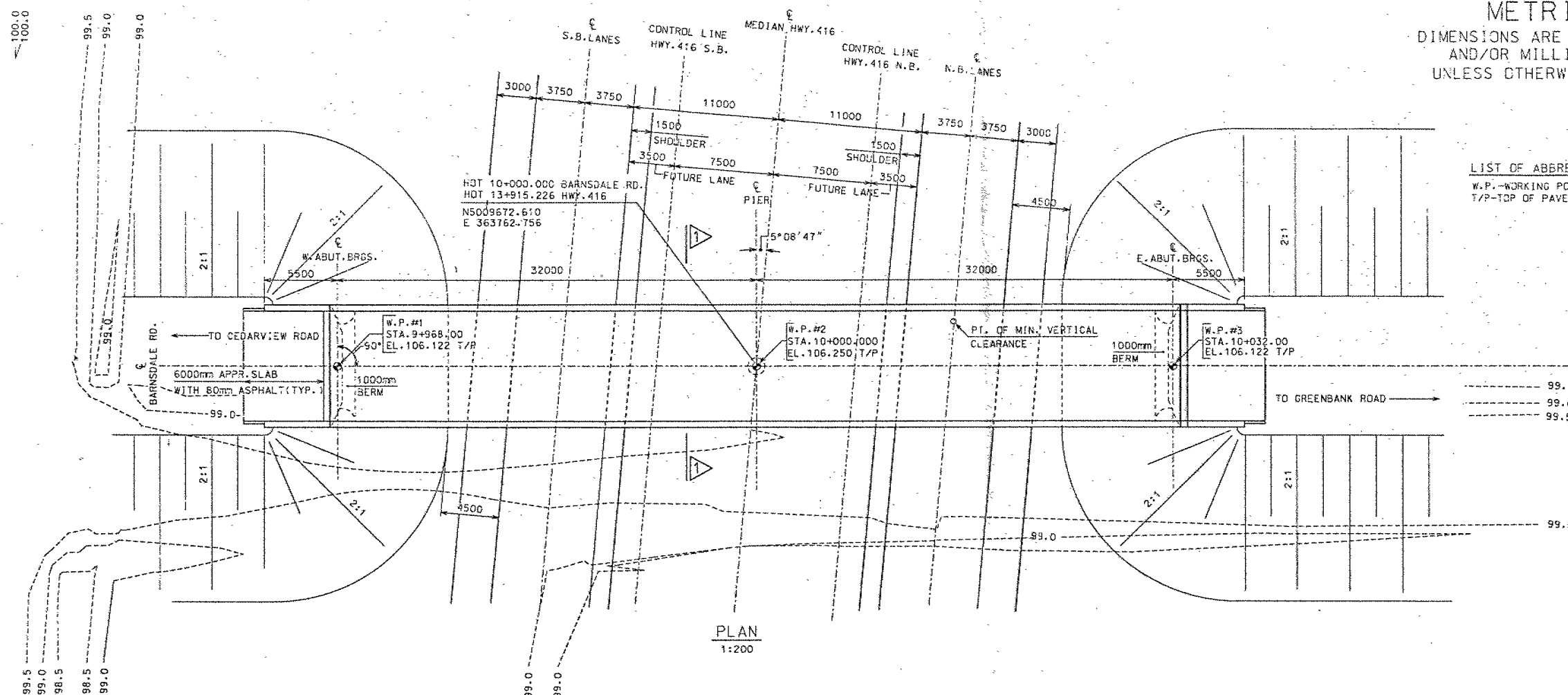
FOOTINGS 100 ± 25
ABUTMENTS & WINGWALLS:
FRONT FACE 80 ± 20
BACK FACE 70 ± 20
PIER 80 ± 20
DECK TOP AND ENDS 70 ± 20
BOTTOM AND SIDES 50 ± 10
REMAINDER 70 ± 20
UNLESS OTHERWISE NOTED

CONSTRUCTION NOTES

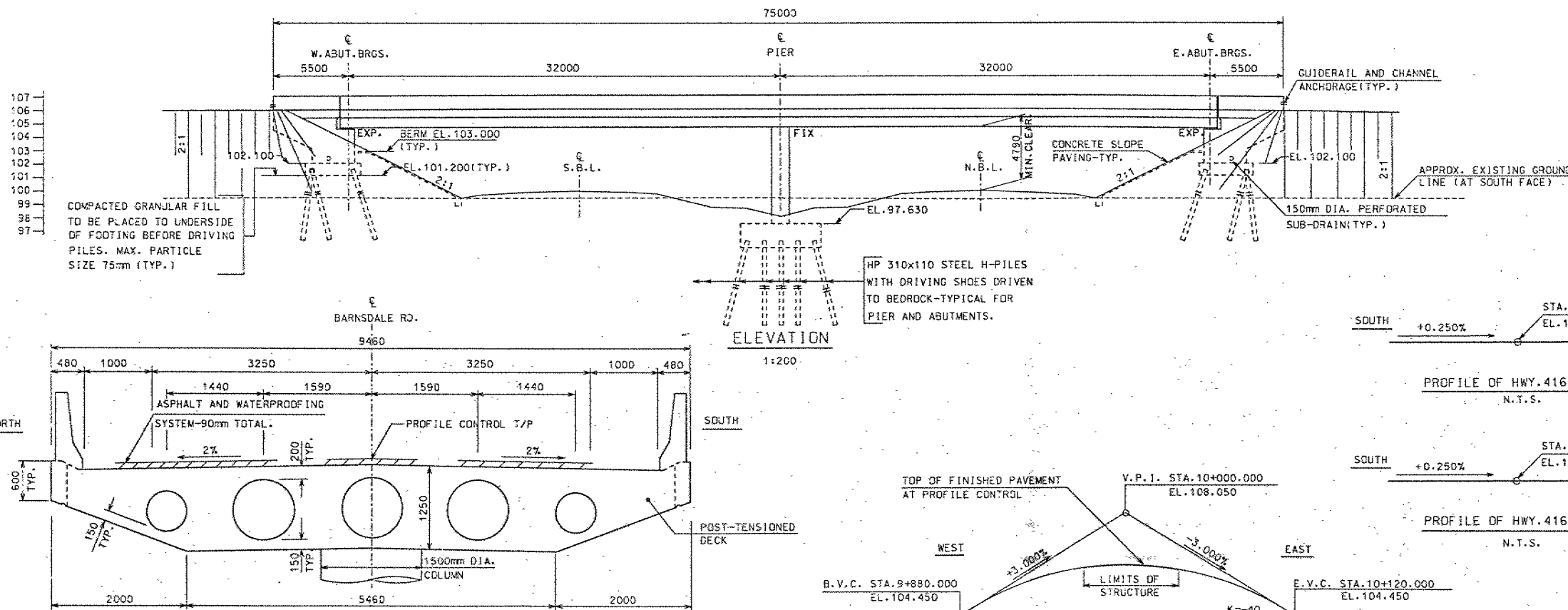
IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.

LIST OF DRAWINGS:

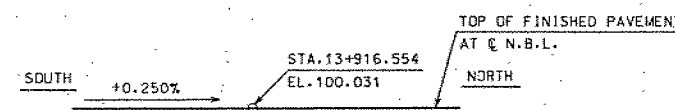
1. GENERAL ARRANGEMENT
2. BORE HOLE LOCATIONS AND SOIL STRATA
3. FOOTING DETAILS
4. ABUTMENTS AND WINGWALLS
5. PIER DETAILS AND BEARINGS
6. DECK DETAILS
7. LONGITUDINAL TENDON DETAILS I
8. LONGITUDINAL TENDON DETAILS II
9. TRANSVERSE TENDON DETAILS
10. DECK REINFORCING I
11. DECK REINFORCING II
12. BARRIER WALL
13. 6000mm APPROACH SLAB
14. DETAILS OF CONC. SLOPE PAVING
15. JOINT ANCHORAGE AND ARMOURING
16. AS CONSTRUCTED ELEV. & DIM.
17. BRIDGE DATE & SITE NUMBER DATA
18. PILE DRIVING-STEEL & DIESEL HAMMERS
19. STANDARD DETAILS
20. QUANTITIES I
21. QUANTITIES II



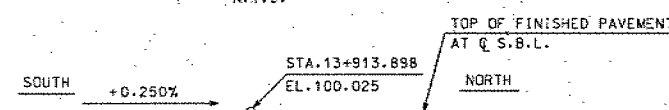
PLAN
1:200



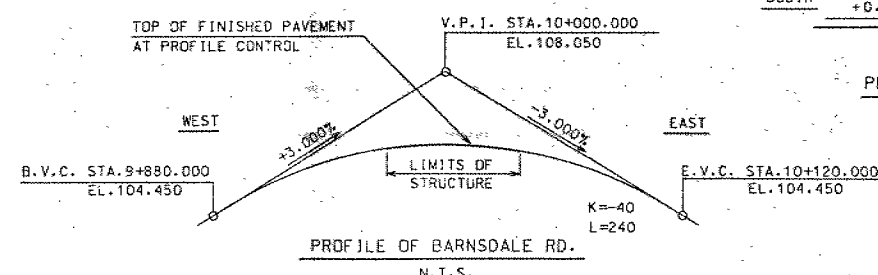
ELEVATION
1:200



PROFILE OF HWY. 416 N.B.L.
N.T.S.

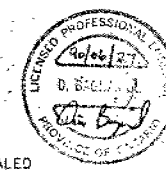


PROFILE OF HWY. 416 S.B.L.
N.T.S.



PROFILE OF BARNSDALE RD.
N.T.S.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

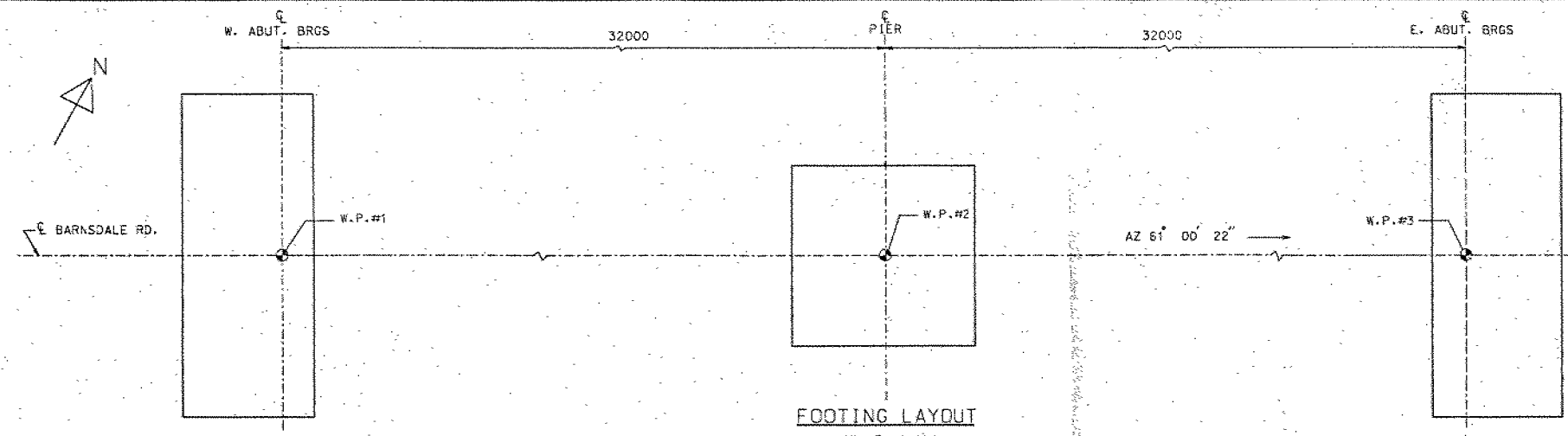


BN ELEV. 99.666
N&W IN ROOT OF 0.2 CEDAR
48.2 LT 13+981.6

APPLICABLE STANDARD DRAWINGS:

DD-3503 MIN. GRANULAR BACKFILL REQUIREMENTS

REVISIONS	DESCRIPTION
DESIGN D.B. CHK	CODE D-802-83 LOAD CLASS A DATE MAY 90
DRAWN J.D. CHK D.B. SITE 3-552 STRUCT	SCHEME DWG 1



METRIC
 DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES
 UNLESS OTHERWISE SHOWN

CONT No		SHEET
WP No 128-87-10		
BARNSDALE RD. UNDERPASS		
FOOTING DETAILS		

W.P.	STATIONS	CO-ORDINATES	
		NORTH	SOUTH
1	9+968.000	5009657.099	363734.767
2	10+000.000	5009672.610	363762.756
3	10+032.000	5009688.121	363790.746

PILE DATA - (HP 310 x 110)

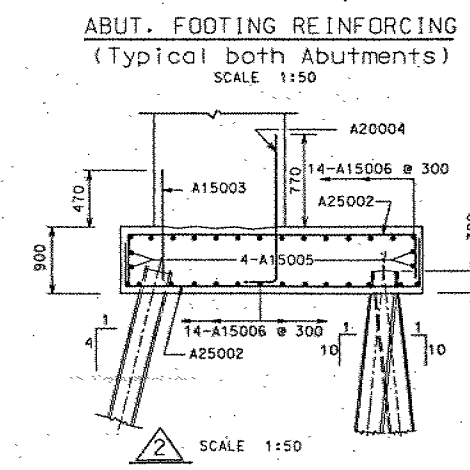
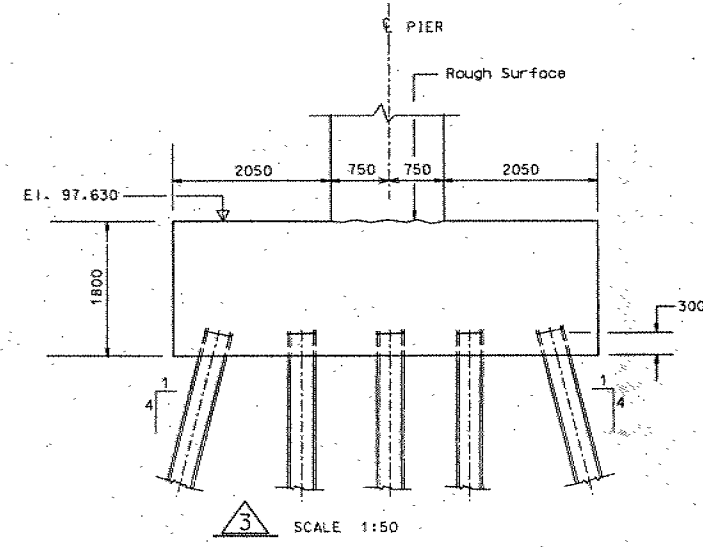
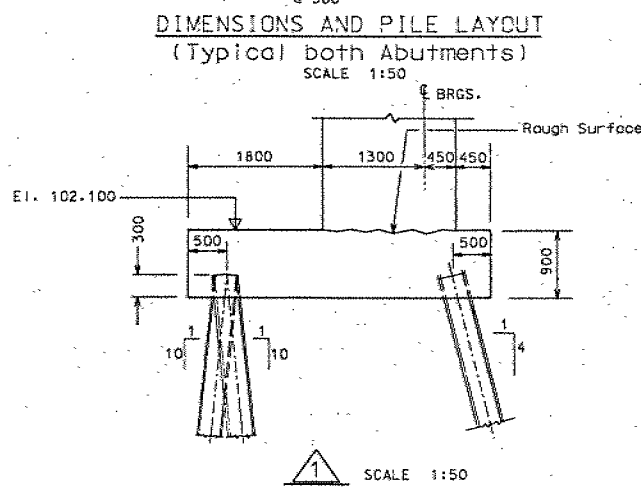
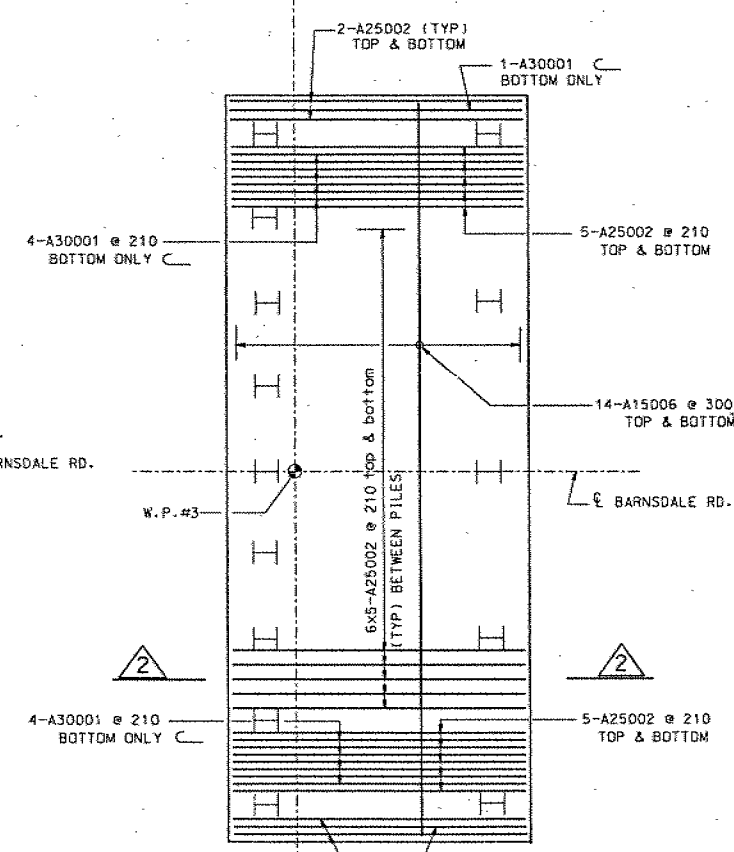
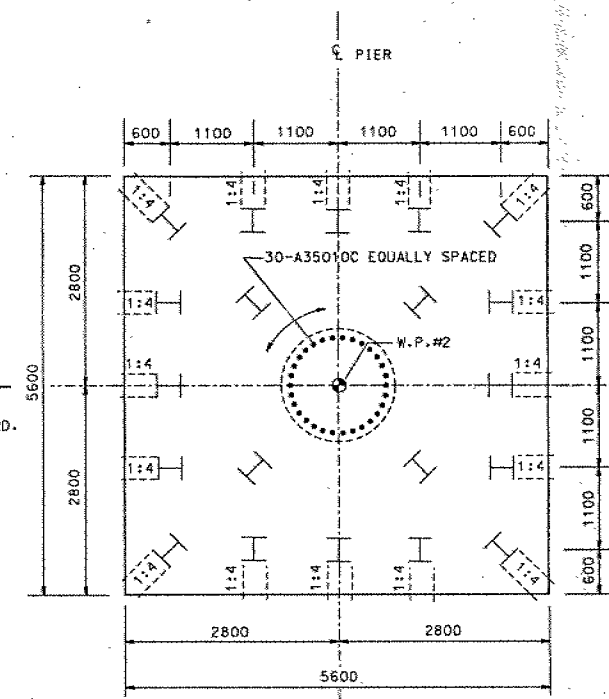
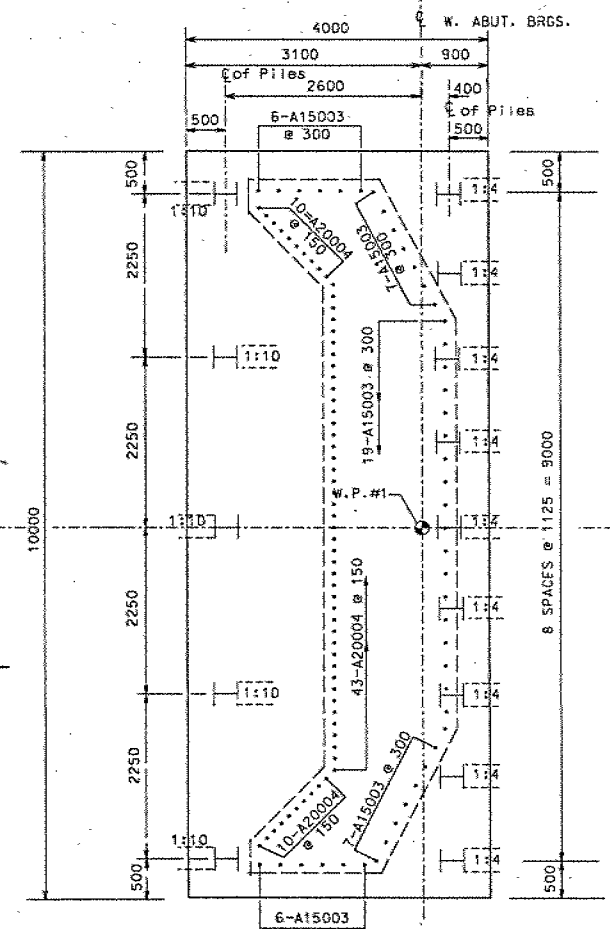
LOCATION	NO.	BATTER	LENGTH	CUT-OFF ELEV.
WEST ABUTMENT	9	1:4	23000	101.500
PIER	16	1:4	16000	96.130
EAST ABUTMENT	5	1:4	22000	101.500

PILE DESIGN DATA

Max. Combined Factored Loads : S L S II = 1150 Kn
 U L S = 1400 Kn

NOTES:

- ALL PILES TO BE HP 310 x 110
- PILES TO BE DRIVEN TO BEDROCK
- ALL PILES SHALL HAVE DRIVING SHOES
- PILE SPACING IS MEASURED AT THE UNDERSIDE OF FOOTINGS
- PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTH BELOW CUT-OFF



DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING

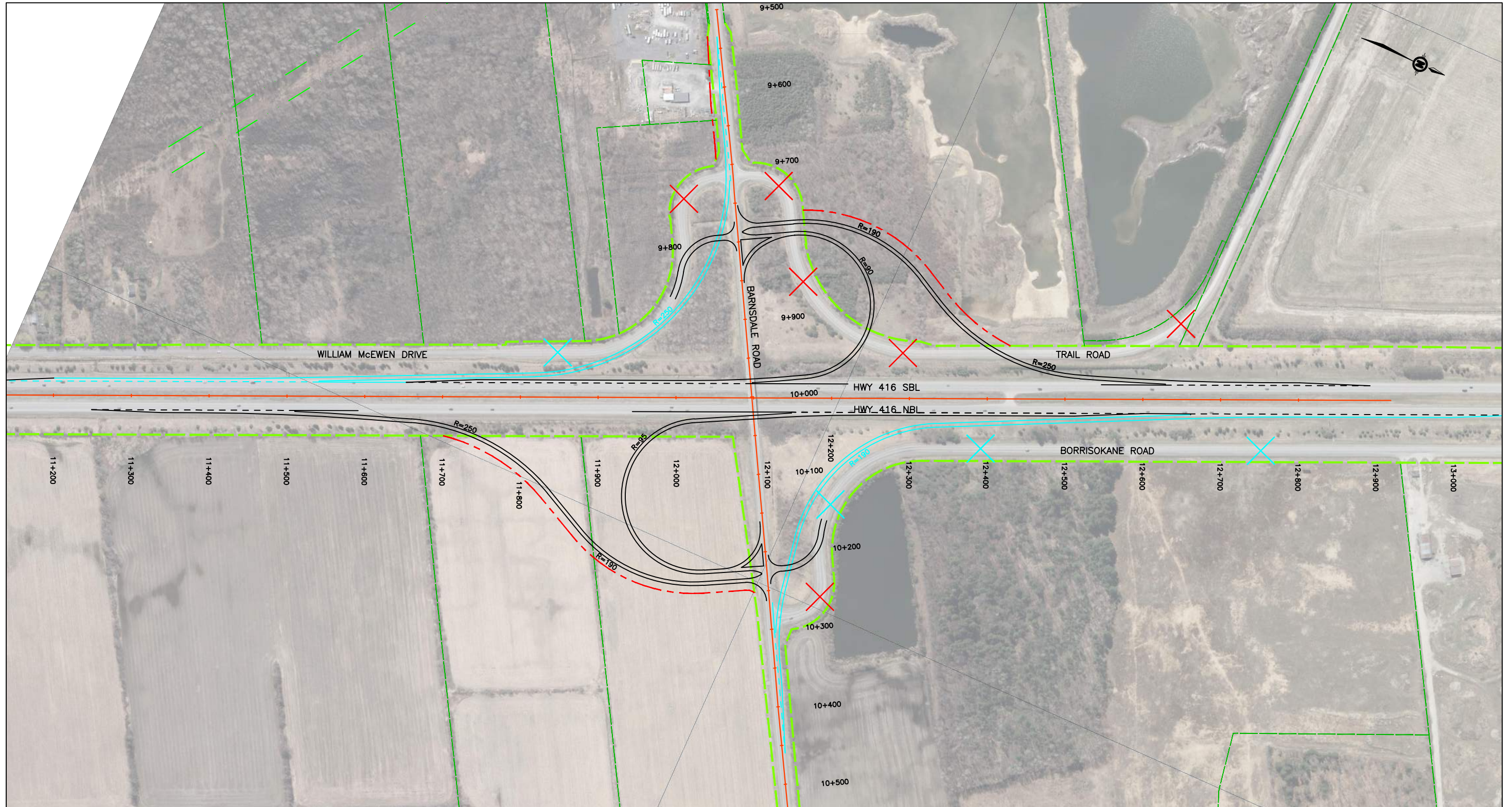
REVISIONS	DESCRIPTION
DESIGN M.G. CHK	CODE CHRC-63 LOAD CLASS A DATE
DRAWN J.E. CHK M.G. SITE 3-552 ISTRUCT	ISCHME DWG 3

APPENDIX C

Morrison Hershfield EA Alternatives Highway 416 at Barnsdale Road

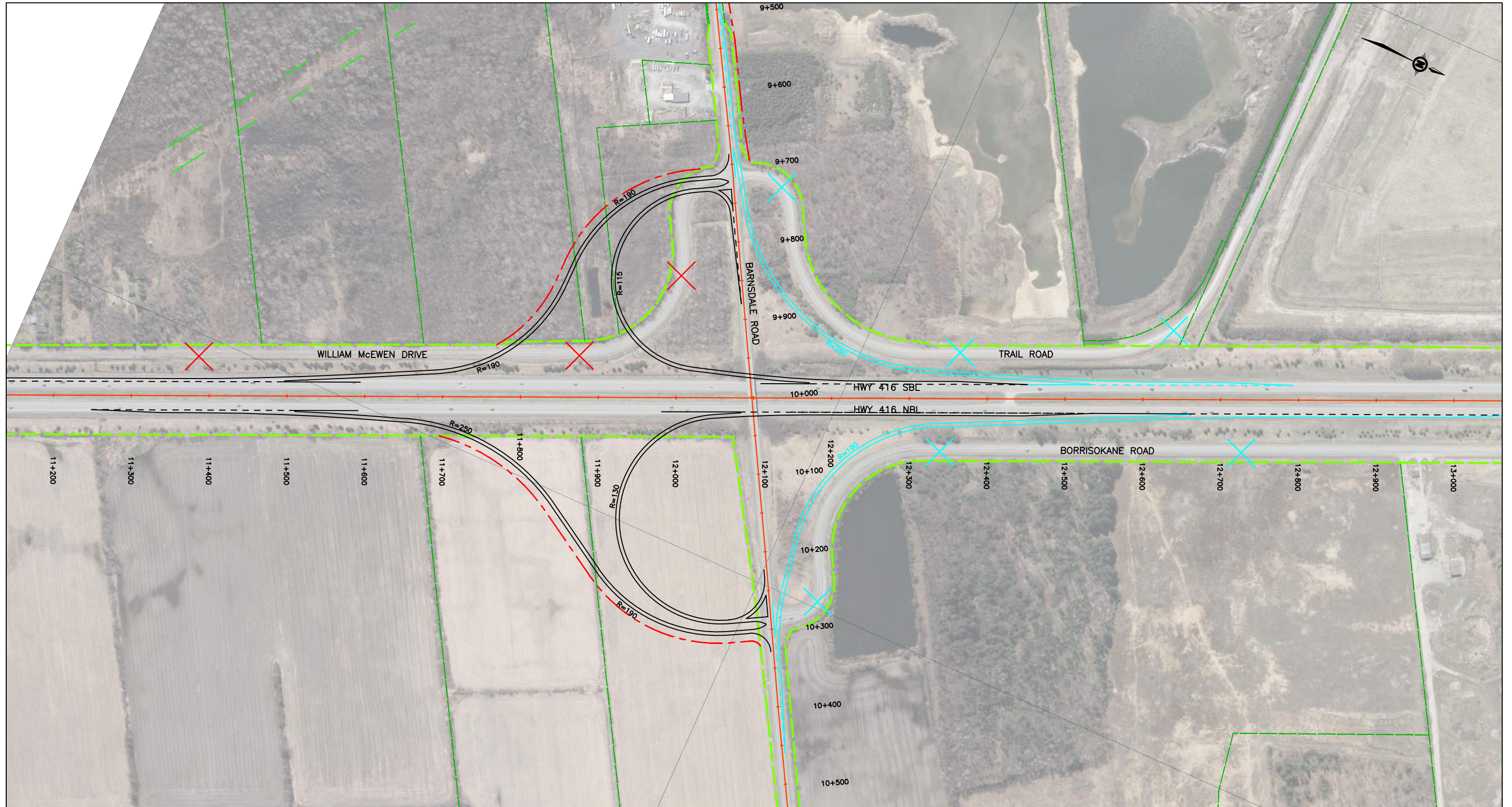
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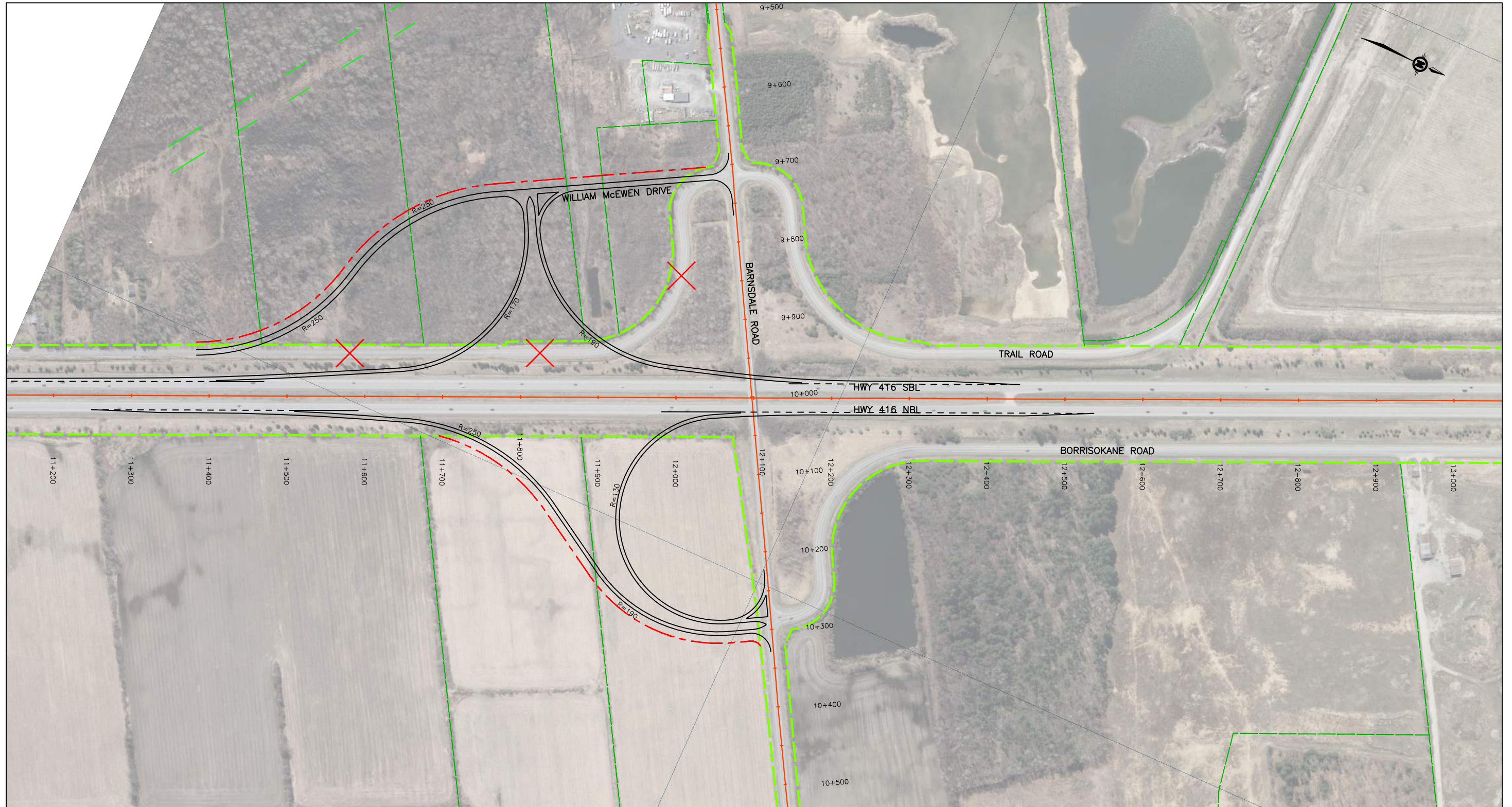
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Plot Date: January 25, 2022 10:26:12 AM

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