
APPENDIX F

SUBSURFACE CONDITIONS REPORT



July 2009

REPORT ON

**GEOTECHNICAL OVERVIEW
WEST TRANSITWAY EXTENSION
BAYSHORE STATION TO WEST OF MOODIE DRIVE
OTTAWA, ONTARIO**

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REPORT



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1.0 INTRODUCTION

This report provides an overview of the geotechnical issues relating to the proposed extension of the West Transitway from the Bayshore Transit Station to about 750 metres west of Moodie Drive in Ottawa, Ontario.

The purpose of this assignment was to collect and collate the existing available subsurface information along the proposed alignment and, based on an interpretation of that information, to provide some preliminary geotechnical design guidelines and to identify key geotechnical constraints that are likely to influence planning and design decisions.

The guidelines provided in this report are intended solely for the preliminary planning and route selection for the proposed extension. More detailed geotechnical input will be required at the design stage. Additional geotechnical investigation will also be required before geotechnical input for the functional and detailed design can be provided.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text of this report but forms an integral part of this document.



2.0 DESCRIPTION OF PROJECT AND SITE

Consideration is being given to extending the West Transitway from the Bayshore Transit Station to about 750 metres west of Moodie Drive (for approximate location, see Key Plan in Figure 1).

It is understood that several corridor alignments, which include both north and south of Highway 417, are currently being considered. However, for this geotechnical overview, and given that the subsurface conditions are considered to be somewhat similar on both sides of the highway, and that access to the site was available, the alignment which is proposed to extend *north* of Highway 417 was assessed. If the corridor is ultimately chosen to extend *south* of Highway 417, it is considered that the challenges and preliminary design guidelines presented in this report would still be generally appropriate. Regardless of which corridor is selected, further investigation will be required during the functional and detailed design stages.

At this time, it is understood that this section of the Transitway will be an exclusive grade-separated corridor which *may* be located somewhat adjacent to, but north of, Highway 417 (as discussed above). At approximately 500 metres west of Moodie Drive, the Transitway may cross over Highway 417 and connect with the existing eastbound Transitway lanes. The entire length of alignment selected for this assessment is about 4 kilometres and includes the following possible/proposed grade separations and/or structures (from east to west):

- Retaining wall adjacent to Highway 417 as part of the Holly Acres Road structure East approach embankment, between Bayshore Transit Station to Holly Acres Road;
- Grade separation structure over Holly Acres Road and the westbound access ramp to Highway 417;
- Tributary of Stillwater Creek culvert extension, west of the westbound access ramp to Highway 417 from Holly Acres Road;
- Retaining walls or open-cut side slopes through the fill berm west of Stillwater Creek;
- Possibly one culvert for the Stillwater Creek realignment – location not known at this time;
- Moodie Drive Transit Station;
- Moodie Drive grade separation structure; and,
- Highway 417 crossing, west of Moodie Drive.

The planning and proposed alignments for this section of the Transitway extension, including discussions on the above structures and realignments, are presented in the following Environmental Assessment reports:

- Report to the Regional Municipality of Ottawa-Carleton by Delcan Corporation titled “West Urban Community, Transit Integration Study and Environmental Assessment”, dated July 1992 (project number 04-1831-A00).
- Report by the Regional Municipality of Ottawa-Carleton titled “West Transitway Extension, Woodroffe Avenue to West of Holly Acres Road, Environmental Assessment Report”, Volume 1 dated March 1994.



Published Geologic Survey of Canada (GSC) mapping, provided on Figures 2 to 4, indicates that the area of the proposed alignment is underlain by silty clay. The depth to the bedrock surface is indicated to be about 3 to 15 metres within the western portion of the alignment and decreases to about 2 to 5 metres in the area of Moodie Drive. The depth to the bedrock surface deepens again to about 10 to 15 metres within the central portion of the alignment and deepens further to up to about 25 metres at the eastern end of the alignment.

The bedrock underlying the western and central portions of the alignment is indicated to consist of sandstone of the March formation. The bedrock underlying the eastern portion of the alignment is indicated to consist of sandstone, limestone, and shale of the Rockcliffe formation.



3.0 SUBSURFACE CONDITIONS

3.1 Available Information

For this assessment, information on the subsurface conditions was obtained from the following sources:

- Published Geologic Survey of Canada (GSC) mapping.
- Reports in Golder Associates' records, which include those of McRostie Genest St-Louis and Associates.
- Ministry of Transportation's library ('Geocres').
- Reports provided by the City of Ottawa and McCormick Rankin Corporation.

The following reports were obtained and used for this assessment. The reports are listed in generally reverse chronological order by date of issue and are as follows:

1. Report to Totten Sims Hubicki by Jacques Whitford titled "Foundation Investigation, WBL Retaining Wall, Highway 417/Moodie Drive Interchange – Ottawa Area, G.W.P. 302-89-00", dated March 28, 2007 (project number NO11674, Geocres number 31G5-210).
2. Report to Totten Sims Hubicki by Jacques Whitford titled "Foundation Investigation, Jack and Bore Crossings and Overhead Sign Foundations, G.W.P. 302-89-00, Highway 417 from Eagleson Road to Highway 416, Ottawa, ON", dated November 2006 (project number NO11674, Geocres number 31G5-208).
3. Report to Totten Sims Hubicki by Jacques Whitford titled "Foundation Investigation, EBL Retaining Wall, Highway 417/Moodie Drive Interchange – Ottawa Area, G.W.P. 302-89-00", dated December 2006 (project number NO11674, Geocres number 31G5-203).
4. Report to Totten Sims Hubicki by Jacques Whitford titled "Foundation Investigation for High Mast Lights, G.W.P. 302-89-00, Highway 417 from Eagleson Road to Moodie Drive, Ottawa, ON", dated November 2006 (project number NO11674, Geocres number 31G5-205).
5. Report to Stantec Consulting Ltd. by Golder Associates Ltd. titled "Geotechnical Investigation, New Overhead Sign Foundations, Sign Number 5 (West Sign), Station 14+650 and Sign Number 6 (East Sign), Station 15+080, Highway 417 Reserved Bus Lane, Region of Ottawa-Carleton, Ontario" dated March 21, 2000 (project number 991-2062-2).
6. Report to Stantec Consulting Ltd. by Golder Associates Ltd. titled "Proposed Highway 417, Reserved Bus Lane, Acres Road to Moodie Drive, Region of Ottawa-Carleton, Ontario" dated March 15, 2000 (project number 991-2062-1).
7. Report to Stanley Consulting Group Ltd. by Golder Associates Ltd. titled "Geotechnical Investigation, Proposed West Transitway Extension and South-West Ramp, West of Bayshore Station (Station 6+400) to West of Acres Road (Station 15+210), Nepean, Ontario" dated March 30, 1998 (project number 971-2058).



8. Report by the Ministry of Transportation of Ontario, Engineering Materials Office, Foundation Design Section titled "SE Ramp Over Hwy. 417 West to Acres Road Ramp (Structure #11)" dated June 12, 1991 (Geocres number 31G5-178).
9. Report by the Ministry of Transportation of Ontario, Engineering Materials Office, Foundation Design Section titled "Hwy. 417 W.B. Overpass Structures at Acres Road, Structure No. 3" dated December 12, 1988 (Geocres number 31G5-146).
10. Report by the Ministry of Transportation of Ontario, Engineering Materials Office, Foundation Design Section titled "Hwy. 417 W.B. Overpass Structure at Acres Road and Associated Ramps" dated October 26, 1988 (Geocres number 31G5-145).
11. Report to J.L. Richards & Associates Limited by Golder Associates Ltd. titled "Geotechnical Investigation, Proposed Watts Creek Relief System, Regional Municipality of Ottawa-Carleton, Ontario" dated January 15, 1988 (project number 851-2546-1).
12. Report to City of Nepean by Golder Associates Ltd. titled "Soil Investigation, Proposed Mud Creek Storm Sewer – Phase II, Nepean, Ontario" dated March 3, 1981 (project number 791-2309-1).
13. Report to City of Nepean by Golder Associates Ltd. titled "Soil Investigation, Proposed Mud Creek Storm Sewer – Phase I, Nepean, Ontario" dated March June 27, 1981 (project number 791-2309).
14. Addendum Report to Cumming-Cockburn & Associates Ltd. by Golder Associates Ltd. titled "Soil Investigation, Proposed Queensway Sub-Collector Sewer, Township of Nepean, Ontario" dated January 26, 1973 (project number 72766-1).
15. Report to Cumming-Cockburn & Associates Ltd. by Golder Associates Ltd. titled "Soil Investigation, Proposed Queensway Sub-Collector Sewer, Township of Nepean, Ontario" dated June 20, 1972 (project number 72766).
16. Report by the Foundation Section, Materials & Testing Div., Department of Highways Ontario titled "Foundation Investigation Report for Existing Sewer and Water Main at the Crossing of New Moodie Drive and Existing Corkstown Road and the Existing Sewer at Station 126+97 of Hwy. 417 (Queensway), Twp. Of Nepean, Lots 10&8, Con.II&I, Dist. 9 (Ottawa)" dated November 3, 1967 (Project numbers W.J. 67-F-98-W.P. 429-64).
17. Report by the Foundation Section, Materials & Testing Div., Department of Highways Ontario titled "Foundation Investigation Report for The Proposed Crossing of Moodie Drive (Line 'A') and Ottawa Queensway Extension (Line 'D'), District #9 (Ottawa)" dated January 30, 1967 (Project numbers W.J. 66-F-108 (R) -- W.P. 430-64).
18. Report by the Foundation Section, Materials & Testing Div., Department of Highways Ontario titled "Preliminary Foundation Investigation Report for The Proposed Underpass at the Ottawa Queensway and County Road 40 Interchange – District #9 (Ottawa)" dated May 24, 1966 (Project numbers W.J. 66-F-16-W.P. 431-64).



19. Report by the Foundation Section, Materials & Testing Div., Department of Highways Ontario titled "Preliminary Foundation Investigation Report for The Proposed Underpass at the Ottawa Queensway and Moody Drive Interchange, District #9 (Ottawa) Hwy: Queensway" dated April 21, 1966 (Project numbers W.J. 66-F-18 -- W.P. 430-64).

The approximate locations of the test holes (i.e., boreholes, test pits, etc.) which were used from Golder Associates and McRostie Genest St-Louis and Associates reports are shown on Figure 5. The locations of the test holes from reports obtained from the City, MRC, and the Ministry of Transportation's libraries (i.e., investigations not carried out by Golder Associates) are not included in our GIS database and are therefore not shown on Figure 5. The locations of relevant boreholes from those other sources will be included on a plan in the geotechnical functional design report. It should also be noted that the locations of the test holes shown on Figure 5 are approximate only. In many cases the site features indicated on the test hole location plans in the reports were sparse, or have changed significantly, and the locations of some of the test holes therefore had to be approximated.

3.2 Summary of Subsurface Conditions

Based on the factual data compiled from the existing reports, the following sections provide a summary of the expected/indicated subsurface conditions along the proposed Transitway extension alignment.

The test hole data obtained from the previous investigation reports relate to the subsurface conditions at the time of investigation. The shallow subsurface conditions, particularly with regard to fill depths, may have changed since the time that the reports were prepared. The groundwater levels may have also changed significantly due to surrounding construction and development.

The subsurface conditions vary considerably along the proposed Transitway alignment. The western portion of the proposed alignment (i.e., west of Moodie Drive) is underlain by sensitive silty clay and glacial till extending to the surface of the bedrock at depths of about 5 to 15 metres. In the vicinity of Moodie Drive, limited thicknesses (i.e., in the order of 3 to 5 metres) of silty clay and glacial till overlie the bedrock. The depth to the bedrock surface deepens to about 10 to 15 metres within the central portion of the alignment and then deepens further to about 25 metres at the eastern limit end of the project (i.e., around Bayshore Station). The overburden material within the eastern portion of the project is indicated to consist of deposits of clay, silt, and sand.

The bedrock underlying the western and central portions of the alignment is indicated to consist of sandstone of the March formation. The bedrock underlying the eastern portion of the project is indicated to consist of sandstone, limestone, and shale of the Rockcliffe formation.

The following sections provide a more detailed summary of the subsurface conditions along the proposed alignment, focusing on areas in which structures or grade separations are proposed.



3.2.1 Holly Acres Road Structure and Approach Embankments

The subsurface conditions in the area of the proposed Holly Acres Road structure and approach embankments consist of up to about 4 to 8 metres of surficial random fill material overlying silty clay and clayey silt, sand, glacial till, and dolostone bedrock. The depth to the bedrock surface is indicated to be in the order of 25 to 27 metres below ground surface.

In the area of the east approach embankment, the surficial fill materials generally consist of silty clay and/or sand; these materials are likely associated with the construction of the existing Holly Acres Road structure and Highway 417. The fill materials in the area of the west approach embankment (forming a berm) reportedly consist of construction debris.

The fill materials are underlain by deposits of silty clay and clayey silt. These deposits generally extend to depths ranging from about 5 to 11 metres below ground surface. The existing subsurface information indicates that the silty clay is thinnest east of Holly Acres Road (and even absent at some locations) and increases in thickness towards to the west. The results of undrained shear strength testing carried out in the silty clay and clayey silt ranging from about 40 to 100 kilopascals indicate these deposits to have a firm to very stiff consistency.

The silty clay and clayey silt overlie a deposit of sand. The sand deposit extends to depths ranging from about 21 and 25 metres below ground surface. Standard penetration testing carried out within the sand deposit indicates it to be in a very loose to very dense state of packing. The looser material was generally located within the shallower portions of this deposit.

The sand deposits are underlain by glacial till, which varies between about 1 and 4 metres in thickness. The glacial till contains a mixture of sand, gravel, cobbles, and boulders.

Dolostone bedrock underlies the glacial till. The surface of the bedrock was encountered at depths ranging from about 25 to 27 metres below ground surface.

The recorded groundwater levels typically ranged from about 2 to 4 metres depth below ground surface.

3.2.2 Stillwater Creek Culvert

Surficial fill materials exist in berms to the east and west of the Stillwater Creek culvert. Current topographical mapping indicates that the fill berms extend up to about 6 metres in height to the east of the culvert and up to about 10 metres in height to the west of the culvert. It is reported that these berms generally consist of construction debris.

The fill materials are underlain by up to about 7 to 11 metres of silty clay and clayey silt. The upper 2 to 4 metres of the silty clay deposit has generally been weathered to a very stiff crust. Results of undrained shear strength testing carried in the underlying unweathered silty clay, ranging from about 40 to 90 kilopascals, indicate this deposit to have a firm to stiff consistency.



The silty clay and clayey silt deposits overlie a very loose to loose deposit of sand. The sand deposit extends to depths about 12 to 15 metres, and is underlain by glacial till. Although bedrock was not encountered in any of the boreholes put down in this area, published geological mapping indicates that the depth to the bedrock surface is in the range of 15 to 25 metres; this depth generally agrees with the adjacent boreholes, at the Holly Acres Road structure.

3.2.3 West of Stillwater Creek Culvert to East Moodie Drive Transit Station

This section extends parallel to Highway 417 for a distance of about 1.5 kilometres. As previously discussed, a fill berm exists west of the Stillwater Creek culvert and extends to about 600 metres west.

The depth to the bedrock surface decreases significantly from the east to the west. The depth to the bedrock surface is at about 15 to 25 metres near Stillwater Creek but only about 1 to 2 metres just east of the proposed Moodie Drive Transit Station.

The overlying native soils in the eastern portion of this area (i.e., west of Stillwater Creek) consist of silty clay that extends to about 7 to 11 metres depth (below the native ground surface). The thickness of silty clay decreases towards the west (in conjunction with the rise of the bedrock surface) and the deposit is generally absent in the area of the proposed Moodie Drive Transit station. The upper 2 to 4 metres of the silty clay deposit (where present) have generally been weathered to a very stiff crust. The undrained shear strengths in the underlying unweathered silty clay deposits, ranging from about 40 to 80 kilopascals, indicate that this deposit has a firm to stiff consistency.

The silty clay deposits, at least in the eastern portion of this section, overlie a very loose to loose deposit of sand. The sand extends to about 12 to 13 metres depth.

The deposit of sand (where present) is underlain by glacial till. The glacial till was not fully penetrated by any of the boreholes advanced in the eastern portion of this area but was proven to extend to a depth of about 13 metres below the ground surface. The depth to the glacial till, as well as the thickness of this deposit, decreases towards the west.

The recorded groundwater levels in this area ranged from approximately 1 to 2 metres below ground surface.

3.2.4 Moodie Drive Transit Station

The subsurface conditions in the vicinity of the proposed Moodie Drive Transit station consist of a very thin layer of sand or glacial till with the surface of the sandstone bedrock being at depths in the order of 1 to 2 metres below ground surface.

The groundwater levels are also in the range of 1 to 2 metres below ground surface.



3.2.5 Moodie Drive Structure and Approach Embankments

The subsurface conditions in the vicinity of the Moodie Drive structure and approach embankments are indicated to consist of about 1 to 2 metres of loose silty sand, underlain by up to about 4 to 5 metres of firm to stiff silty clay (undrained shear strengths ranging from about 40 to 80 kilopascals), overlying up to about 1 metre of glacial till.

The surface of the sandstone bedrock is expected at about 5 to 8 metres depth.

The groundwater levels ranged from approximately 3 to 5 metres below the ground surface.

3.2.6 Highway 417 Structure and Approaches

Although only conceptual sketches are available for this crossing, it is understood that the structure would be located about 500 to 550 metres west of Moodie Drive.

The subsurface conditions in the vicinity of the proposed Highway 417 overpass structure and approach embankments generally consist of embankment fill materials overlying sensitive silty clay and glacial till.

The silty clay in this area is about 6 to 10 metres thick (or more). The clay deposit thins to the west. The upper 2 to 4 metres of the silty clay have generally been weathered to a stiff brown crust. The underlying unweathered silty clay is typically softer and has undrained shear strengths of about 50 to 80 kilopascals, which indicate a firm to stiff consistency.

The silty clay deposit is underlain by glacial till. The existing boreholes advanced in this area were not advanced to sufficient depth to fully penetrate the glacial till deposit. Therefore the thickness of the glacial till deposit and the depth to the bedrock is not known. Published geological mapping does however indicate that the depth to the bedrock surface is in the range of 10 to 15 metres. Bedrock rises towards the west (in conjunction with the rise in the topography) and is exposed beside Highway 417, west of the Ottawa Central Railway (OCR) structure.

The recorded groundwater in this area ranges from about 2 to 3 metres depth below ground surface.

Between the proposed crossing area and the OCR structure, there appears to be an area where the silty clay is absent (i.e., the subsurface conditions consist of glacial till overlying bedrock). This area is located about 1 kilometre west of Moodie Drive.



4.0 DISCUSSION

4.1 General

This section of the report discusses, at a planning level, geotechnical engineering issues relating to the proposed Transitway extension. The guidelines in this section of the report are provided at a level of detail appropriate for planning, but not functional or detailed design. The functional and detailed design of the Transitway extension will require more detailed input and additional investigations.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text of this report but forms an integral part of this document.

4.2 Holly Acres Road Structure and Approach Embankments

The subsurface conditions in the vicinity of the proposed Holly Acres Road structure and approach embankments are indicated to consist of up to about 4 to 8 metres of surficial random fill material overlying firm to stiff silty clay and clayey silt, very loose to dense sandy soils, glacial till, and then dolostone bedrock. The bedrock surface is at about 25 to 27 metres depth.

4.2.1 Foundations for Structure

Given the large loads typically imposed by a bridge structure, the limited capacity of the compressible silty clay to support load, and the low bearing resistances offered by the loose sandy soils and fill materials, it is not considered feasible to support the bridge on shallow foundations. The structure will therefore have to be founded on deep foundations.

Two commonly used deep foundation types for supporting bridge structures are driven steel H-piles (typically end bearing on rock) and drilled cast-in-place caissons end bearing on, or socketed in, the bedrock. The surface of the bedrock is at about 25 to 27 metres depth. Cast-in-place concrete caissons, although technically feasible, are unlikely to be economic considering the significant depth to bedrock and the challenges associated with dewatering and cleaning of the sockets. Therefore, at this stage of the design, it should be planned to support the structure on driven steel H-piles.

The Ultimate Limit States (ULS) factored geotechnical axial resistance of a conventional sized H-pile (HP 310x110), if driven to the bedrock, would be in the order of 2,000 kilonewtons. The axial resistance at Serviceability Limit States (SLS) for 25 millimetres of settlement would be higher, and the SLS resistance would not therefore apply.

Some of the piles could have difficulty penetrating to the bedrock surface and could “hang up” on cobbles and boulders within the glacial till which overlies the bedrock. These piles might not therefore achieve the full design capacity. Where this does occur, the ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved, but could be somewhat less than if founded on bedrock.



4.2.2 Approach Embankments

Preliminary analyses indicate that the silty clay in the area of the approach embankments has sufficient capacity to support up to about 4 metres of total filling without undergoing significant consolidation settlement. Although the profile of the Transitway is not known at this time, it is expected that the approach fills will be more than about 5 metres in height and, as such, significant consolidation settlement could be expected. The magnitude of settlement will have to be assessed at the functional design stage, once of Transitway alignment and profile have been established. However it is not considered unreasonable to expect settlements of greater than 100 millimetres if the approach embankments are greater than 5 metres in height. Those settlements would be entirely differential relative to the bridge.

If it is determined that the estimated settlements cannot be tolerated, consideration could be given to the following options:

- Option 1.** Pre-load the site (probably with a surcharge) and allow the settlements to occur prior to paving. The settlements would need to be monitored to confirm the time at which sufficient settlement had occurred and that construction could commence. If required, the settlements could be accelerated by providing wick drains.
- Option 2.** Use light weight fill for the embankment construction, thereby reducing the stress increase on the compressible clay deposit to a level so that the settlements would be within acceptable tolerances. Two commonly used light weight fill materials are expanded polystyrene and blast surface slag.

Subexcavation of the silty clay would not be feasible due to its thickness. The excavation would likely be too large and too deep, at least in some locations, to be feasible or practical. Keeping the profile grade to less than about 4 metres above the existing grade is also probably not feasible for this project as the approach embankments will have to be of sufficient height to carry the Transitway over Holly Acres Road.

From a geotechnical perspective, if the construction schedule permits, Option 1 without wick drains would be preferred in that it would be the least expensive. However, depending on the actual alignment, the design of the embankments must also consider the proximity of the adjacent Highway 417 and the implications that the settlements may have on that roadway; this option may in fact not be feasible. Option 1 *with* wick drains would have less impact on the construction schedule but would be significantly more expensive than without the wick drains. Option 2, although practical and commonly used, would also be significantly more expensive than Option 1, but may be required depending on the proximity of the Transitway embankment to the adjacent highway.

It is also understood that, due to space restrictions, the east approach embankment may be contained within a retaining wall or possibly constructed as a mechanically stabilized slope. The feasibility of the type of wall or stabilized slope will be dependent on the height of the approach embankment, the magnitude of the settlements, and the location (i.e., proximity to the adjacent highway), all of which are unknown at this time. However, the following alternatives may be considered.

- **Mechanically reinforced slope.** A mechanically reinforced slope may be feasible, although some large settlements could be potentially expected. To reduce the magnitude of the settlements, the area of the



approach embankment could be pre-loaded or the embankment could be constructed using light weight fills.

- **Retained soil system (RSS) wall.** These systems are generally only feasible where the settlements are expected to be small. Therefore as a preliminary guideline, a RSS wall system is probably not suitable for this site, except maybe at the extreme east end where the clay is thinner and stiffer.
- **Concrete retaining walls on shallow footings.** Concrete retaining walls on shallow spread footings may be feasible for lower height walls and in areas where the compressible clay is thinner and stiffer (i.e., possibly towards the Bayshore Transit station). This type of wall can also only be considered if the differential settlements along the wall are less than about 25 to 30 millimetres; this may also require preloading the site or constructing the embankment using light weight fills.
- **Concrete retaining walls on rammed aggregate piers (geopier).** The rammed aggregate piers would have to be installed to sufficient depth to fully penetrate the clay deposit, and found in the underlying sand. Geopier foundations will likely be feasible in areas where the clay is relatively thin.
- **Concrete retaining walls on deep foundations.** It is considered feasible to support a concrete retaining wall on deep foundations. The deep foundations should consist of driven steel H-piles, end bearing on the bedrock surface. Although feasible, this option would be quite expensive given the depth to the bedrock surface.

If the location of the Transitway alignment is such that space restrictions are not an issue, then approach embankments with side slopes of 2 horizontal to 1 vertical could be considered. The stability of embankments against deep-seated shear failure through the silty clay will however have to be assessed once the height and location of the embankments are known. As a preliminary guideline, it is expected that up to a 6 metre high embankment with a 2 horizontal to 1 vertical side slope should be feasible.

Similarly, the stability of any proposed retaining wall system will also have to be assessed once the height and location of the walls are known. As a preliminary guideline, it is expected that up to a 4 metre high retaining wall should be feasible.

4.2.3 Seismic Design Issues

The existing subsurface information indicates that portions of the sand deposits which underlie this site (at depth) are in a very loose to loose state. There is therefore a potential for seismic liquefaction to occur.

Seismic liquefaction occurs when earthquake vibrations cause an increase in pore water pressure within the soil. The presence of excess pore water pressures reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as "lateral spreading". This strength loss can also result in instability of the approach embankments and retaining structures (i.e., shear failure though the underlying soil).



- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding of shallow foundations. This may impact on the shallow foundation design for low height retaining structures.
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading. This may result in additional piles being required so that adequate lateral resistance is provided during a seismic event.

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements. If seismic settlements occur, drag-loads will be induced on deep foundation elements; the design of the foundations would have to consider this additional load.

The available in-situ testing data indicates that portions of the sand deposit may be potentially liquefiable. However the results of the in-situ testing reflect a variety of in-situ testing equipment and methods used over more than 20 years of investigations. The accuracy of some of the results will need to be verified (by site-specific additional investigation) before the liquefaction potential can be confirmed.

If the loose sand is deemed to be liquefiable, consideration may have to be given to improving (densifying) a portion of, or the full depth of, the potentially liquefiable zones. This densification would help reduce the seismic settlements and lateral movements (i.e., increase the factor of safety against embankment instability). Ground improvement techniques which may be feasible for this site include vibro-compaction and jet grouting.

4.3 Stillwater Creek Culvert

It is understood that the existing culvert that crosses Stillwater Creek under Highway 417 would be extended beneath the Transitway alignment.

Surficial fill materials exist to the east and west of the Stillwater Creek culvert. Current topographical mapping indicates that the fill materials extend up to about 6 metres in height to the east of the culvert and up to about 10 metres in height to the west of the culvert. The fill materials are underlain by up to about 7 to 11 metres of firm to stiff silty clay and clayey silt, overlying sand.

Based on the subsurface and fill conditions, the main concern for the design and construction of the culvert extension is the settlement of the underlying compressible clayey soils and the impacts that those settlements could have on the performance of the culvert. Preliminary analyses indicate that the silty clay in the area of the Stillwater Creek culvert has sufficient capacity to support up to only about 4 to 5 metres of total filling without undergoing significant consolidation settlement. As such, it is very likely that the weight of the existing fill material has caused, and is still causing, consolidation settlement of the silty clay. Further filling, over the culvert extension, to the raise the grade up to the Transitway level would cause further settlement.

Notwithstanding the importance of limiting settlements induced by the weight of the fill material, it will also be important to limit the magnitude of the foundation stresses from the culvert, as not induce additional consolidation settlement.



The foundation design and settlement assessments for the culvert extension cannot be carried out until the Transitway profile and alignment have been selected. However, for planning purposes, the following should be considered:

- The Transitway profile should ideally be selected so that the grade raise in the area of the culvert would be no more than about 4 or 5 metres (above original/natural ground level). This restriction will limit the magnitude of the consolidation settlement of the silty clay and corresponding movement of the culvert.
- From a foundation design perspective, the culvert should consist of a closed box culvert rather than a rigid frame open box. A closed box culvert provides a greater distribution of the foundation loads and therefore lower foundation stress levels. The settlements for a rigid frame open box culvert would likely be larger due to the higher concentration of bearing stresses at the footings.
- The box culvert should be cambered and articulated. This design would make the culvert more tolerant to settlements.

Supporting the culvert on deep foundations (such as piles) would avoid culvert settlements, but could result in distortion of the overlying pavement and large vertical loading on the culvert, and is therefore not recommended.

4.4 Fill Berm West of Stillwater Creek Culvert

Current topographical mapping indicates that the fill berm extends up to about 10 to 12 metres in height and to about 600 metres west of Stillwater Creek. The fill material reportedly consists of construction debris, including large pieces of concrete.

The underlying native soils consist of firm to stiff silty clay that extends to about 7 to 11 metres depth (below the native ground surface). The silty clay deposit generally overlies loose deposits of sandy soils.

It is understood that the Transitway profile will likely include a “cut” through the fill berm.

Based on the current subsurface and fill information, some concerns for design and construction of the cut are:

- The settlement of the underlying compressible clayey soils and the impacts that those settlements could have on the performance of earth retaining systems: Preliminary analyses indicate that the silty clay in the area of the fill berm has sufficient capacity to support up to only about 4 to 5 metres of total filling without undergoing significant consolidation settlement. As such, it is very likely that the weight of the existing fill material has caused, and is still causing, significant consolidation settlement of the silty clay. Significant differential settlements could also be expected along the berm given the varying thickness of fill materials and compressible silty clay. Most retaining structures cannot tolerate the potential significant differential settlements. Therefore, given the expected large and differential settlements, open cut side slopes would be preferred, from a geotechnical perspective. As a preliminary guideline, it is considered that side slopes cut in the fill berm at an inclination of 2 horizontal to 1 vertical, up to 5 metres in height, should be feasible. Consideration could also be given to using concrete retaining walls founded on deep foundations, but the deep foundations will likely have difficulty penetrating through the fill material and, if the ground is settling, the wall settlements would not be consistent with the pavement settlements.



- The composition of the berm fill and the difficulty which may be encountered during excavation: Depending on the actual composition and size of the debris, large excavating equipment could be required and the debris may have to be broken up using mechanical methods (i.e., hoe-ramming). Depending on its chemical composition, the fill material may have to be disposed of at a landfill if removed from the site.
- The resistance of the sandy soils to seismic liquefaction: The existing subsurface information indicates that portions of the sand deposits which underlie the silty clay are in a very loose to loose state. There is therefore a potential for seismic liquefaction to occur. Seismic liquefaction may impact on the global stability of the soil retaining systems or cut slopes. A discussion on seismic liquefaction was provided in Section 4.2.3.

4.5 Moodie Drive Transit Station

The subsurface conditions in the vicinity of the proposed Moodie Drive Transit station are expected to consist of a very thin layer of sand or glacial till, with the surface of the sandstone bedrock being at depths in the order of 1 to 2 metres below ground surface.

4.5.1 Foundations

Given the shallow depth to the bedrock surface, the Transit station will likely be supported on spread footing foundations founded on or with the sandstone bedrock.

The factored bearing resistance at ULS for spread footing foundations founded on or within competent sandstone bedrock will likely be at least 2,000 kilopascals. The settlement of footings at the corresponding service (unfactored) load levels would be less than 25 millimetres and therefore SLS need not be considered in the foundation design.

It is not known if the bedrock contains joints or seams which are in-filled with frost susceptible soils, but is a possibility for the sedimentary bedrock at this site. Therefore, at this stage of the design, the following two options should be considered for foundation design with regard to frost protection:

- Option 1.** Found the footings directly on or in the bedrock and provide at least 1.5 metres of earth cover for frost protection purposes; or,
- Option 2.** Where 1.5 metres of earth cover cannot be provided without removing bedrock and/or extensively raising the surrounding grade, the required frost protection can be achieved by insulating the footing bearing surfaces with high density rigid insulation.

4.5.2 Excavation for Site Servicing

Excavation for the site services for the Transit station will be through limited thicknesses of overburden soils and into the bedrock. It is expected that bedrock removal could be carried out using drill and blast techniques or, for shallow excavations, using mechanical methods such as hoe ramming. However, the sandstone bedrock is hard and abrasive and greater than typical equipment wear should be expected.



4.6 Moodie Drive Structure and Approaches

The subsurface conditions in the vicinity of the Moodie Drive structure and approach embankments consist of about 1 to 2 metres of loose silty sand, underlain by firm to stiff silty clay and glacial till, with the surface of the bedrock at about 5 to 8 metres depth.

The Environmental Assessment report indicates that it will not be feasible to establish the Transitway profile *beneath* Moodie Drive due to drainage issues, and that it will instead be necessary to carry the Transitway over Moodie Drive.

If however it is ultimately decided to establish the Transitway profile below Moodie Drive, consideration must be given to the following:

- The construction of the structure could require mass excavation of sandstone bedrock. The sandstone bedrock in this area is very hard and abrasive, and will likely be difficult to excavate.
- The lowest portion of the Transitway would likely be below the water table and, as such, a permanent groundwater drainage system would be required. The outlet would need to have sufficient capacity to convey those flows. The permanent drainage could also result in long term groundwater level lowering in the area. The areal extent of that lowering would depend on the hydraulic conductivity of the bedrock, but could be quite large. Sensitive clay soils within that area (known to exist to the west) could undergo consolidation or shrinkage, resulting in ground settlements. At this time, there are not considered to be structures within the area of likely groundwater level lowering, but further evaluation of the potential impact would be required.

Consistent with the Environmental Assessment report, the following sections provide preliminary geotechnical guidelines on the basis that the Transitway will be carried over Moodie Drive and not under.

4.6.1 Foundations for Structure

Given the large loads expected for the bridge structure, and the limited capacity of the compressible silty clay on this site to support those loads, it is considered that shallow foundations in the overburden will not be feasible. The bedrock surface in this area is also likely too deep to make footings on bedrock a feasible option. It is therefore expected that the structure will have to be founded on deep foundations. Two suitable deep foundation types would be driven steel H-piles (end-bearing on bedrock) or rock-socketed cast-in-place caissons. The bedrock surface is estimated/indicated to be at about 5 to 8 metres depth.

The Ultimate Limit States (ULS) factored geotechnical axial resistance of a conventional sized H-pile (HP 310x110), if driven to the bedrock, would be in the order of 2,000 kilonewtons. The axial resistance at Serviceability Limit States (SLS) for 25 millimetres of settlement would be higher, and the SLS resistance would not therefore apply.

Caissons socketed into the bedrock could probably be designed using a factored shaft resistance of 1 megapascal, would generally be about 0.9 to 1.5 metres in diameter, and with a socket length at least twice the caisson diameter.



4.6.2 Approach Embankments

Preliminary analyses indicate that the silty clay in the area of the approach embankments (primarily the west embankment) has sufficient capacity to support up to about 4 to 5 metres of total filling without undergoing significant consolidation settlement. Although the profile of the Transitway is not known at this time, it is expected that the approach fills will be more than 5 metres in height, and as such, consolidation settlement should be expected. The magnitude of settlement will have to be assessed at the functional design stage once of Transitway profile has been established. However, it is not considered unreasonable to expect settlements of greater than 100 millimetres, if the approach embankments are greater than 4 or 5 metres in height. Those settlements would be entirely differential relative to the bridge.

If it is determined that the magnitude of settlement cannot be tolerated, consideration could be given to the following options (as previously discussed in detail in Section 4.2.2):

- Option 1.** Pre-load the site, probably with a surcharge, and allow the settlements to occur prior to paving. Wick drains could be used to accelerate the settlements.
- Option 2.** Use light weight fill for the embankment construction.
- Option 3.** Excavate the silty clay deposit from the embankment area and replace it with compacted embankment fill. This option would only be practical for the east approach where the clay is thin.
- Option 4.** Keep the profile grade to less than about 4 metres above the existing grade. This option is likely not feasible since the approach embankments will have to be of sufficient height to carry the Transitway over Moodie Drive.

From a geotechnical perspective, if the construction schedule permits, Option 1 *without* wick drains would be preferred in that it would be the least expensive. Option 1 with wick drains will have a less impact on the construction schedule but will be significantly more expensive than without the wick drains. Option 2, although practical and commonly used, would also be significantly more expensive than Option 1.

4.7 Highway 417 Structure and Approaches

Although only conceptual sketches are available for this crossing, it is understood that the structure would be located about 500 to 550 metres west of Moodie Drive.

The subsurface conditions in the vicinity of the proposed Highway 417 overpass structure and approach embankments generally consist of fill material (associated with the construction of Highway 417) overlying firm to stiff silty clay and glacial till. The silty clay in this area is about 6 to 10 metres thick (or more). The clay deposit thins to the west.

The existing boreholes advanced in this area were not advanced to sufficient depth to fully penetrate the glacial till deposit. Therefore the thickness of the glacial till deposit and the depth to the bedrock is not known. Published geological mapping does however indicate that the depth to the bedrock surface is in the range of 10 to 15 metres. Bedrock rises towards the west (in conjunction with the rise in the topography) and is exposed beside Highway 417, west of the Ottawa Central Railway (OCR) structure.



Between the proposed crossing area and the OCR structure, there appears to be an area where the silty clay is absent (i.e., the subsurface conditions consist of glacial till overlying bedrock). This area is located about 1 kilometre west of Moodie Drive.

4.7.1 Foundations for Structure

As discussed for the Moodie Drive structure, given the large loads expected for the bridge structure and the limited capacity of the compressible silty clay at the proposed location to support these loads, shallow foundations will not be feasible and the structure will have to be founded on deep foundations. However, the existing subsurface information does indicate that the silty clay becomes thinner and the depth to the glacial till is relatively shallow at about 1 kilometre west of Moodie Drive. Consideration could therefore be given to shifting the bridge, to about a 1 kilometre to the west of Moodie Drive, and possibly supporting the structure on spread footing foundations founded in the glacial till. The depth to and the compactness of the glacial till would however have to be assessed to determine if the alternative is feasible. Therefore, at this stage of design, it should still be planned to support the structure on deep foundations.

Two suitable deep foundation types would be driven steel H-piles (end-bearing on bedrock) or rock socketed cast-in-place caissons. Although the depth to the bedrock surface was not encountered during the previous investigations, the bedrock surface is estimated to be at about 10 to 15 metres depth (based on published geology mapping).

The Ultimate Limit States (ULS) factored geotechnical axial resistance of a conventional sized H-pile (HP 310x110), if driven to the bedrock, would be in the order of 2,000 kilonewtons. The axial resistance at Serviceability Limit States (SLS) for 25 millimetres of settlement would be higher, and the SLS resistance would not therefore apply.

Caissons socketed into the bedrock could probably be designed using a factored shaft resistance of 1 megapascal, would generally be 0.9 to 1.5 metres in diameter, and with a socket length at least twice the caisson diameter.

4.7.2 Approach Embankments

Preliminary analyses indicate that the silty clay in the proposed area of the approach embankments has sufficient capacity to support up to about 4 to 5 metres of total filling without undergoing significant consolidation settlement. Although the profile of the Transitway is not known at this time, it is expected that the approach fills will be more than 4 or 5 metres in height, to carry the Transitway over Highway 417 and, as such, consolidation settlement should be expected. The magnitude of settlement will have to be assessed at the functional design stage once of Transitway profile has been established. The settlements would be entirely differential relative to the bridge.

If it is determined that the magnitude of settlements cannot be tolerated, consideration could be given to the following options:

- Option 1.** Pre-load the site, probably with a surcharge and possibly wick drains, and allow the settlements to occur prior to paving.



Option 2. Use light weight fill for the embankment construction.

Keeping the profile grade to less than about 4 metres above the existing grade is likely not feasible since the approach embankments will have to be of sufficient height to carry the Transitway over Highway 417. The clay deposit is probably also too thick to sub-excavate.

Option 1 without wick drains would be the preferred option in that it would be the least expensive. Option 1 with wick drains will have a less impact on the construction schedule but will be significantly more expensive than without wick drains. Option 2 would be significantly more expensive than Option 1 and likely more expensive than Option 1 (with wick drains), but would not impact the schedule.

Shifting the bridge to the west where the silty clay is thinner (as previously discussed for the bridge foundations) might also result in better conditions for supporting the approach embankments.

4.8 Excavations

Excavations for sewers along the general Transitway alignment would extend through fill materials (at least on the eastern portion of the site) and into clayey soils, permeable sandy soils, and bedrock (in the area of the proposed Moodie Drive Transitway Station). Excavations deeper than about 3 to 4 metres would likely extend below the groundwater level.

No unusual problems are anticipated in excavating the native overburden using conventional hydraulic excavating equipment although some boulder removal could be required. There may however be difficulty in excavating through the fill berm in the area of the Stillwater Creek culvert. Mechanical methods (such as hoe-ramping) may be required to break up some of the larger objects (i.e., concrete slabs).

The significant issue with respect to excavations will be groundwater control. For excavations carried out within clayey soils, it should be possible to handle the groundwater inflow using sumps established in the bottom of the excavations. However, excavations below the groundwater level in the sandy soils could however experience significant groundwater inflow, depending on the depth of the excavation, and an active de-watering system could be required. Similar measures could be required if excavating in bedrock (Moodie Drive area). Depending on the rate of pumping, a Permit-To-Take-Water may need to be obtained from the Ministry of the Environment. An evaluation of the impacts of the groundwater level lowering on the settlement of surrounding structures would also be required.



4.9 Pavement Design

It is expected that this section of the Transitway will have a flexible pavement structure, but with a rigid pavement structure being used in the Moodie Drive Station.

Although traffic data has not been provided, it is expected that the composition of the Transitway pavement will consist of the following:

<u>Pavement Component</u>	<u>Thickness (millimetres)</u>
Asphaltic Concrete	140
OPSS Granular O Base	200
OPSS Granular B Type II Subbase	600 to 800



5.0 LIMITATIONS

The guidelines provided in this report are based on a limited amount of factual information and are intended solely for the planning of the proposed Transitway extension. The functional and detailed design of the proposed extension will require site-specific geotechnical investigations and design guidance.



REPORT SIGNATURE PAGE

GOLDER ASSOCIATES LTD.

T.M. Skinner, P.Eng.
Geotechnical Engineer

M.I. Cunningham, P.Eng.
Associate

TMS/MIC/cg

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

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Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

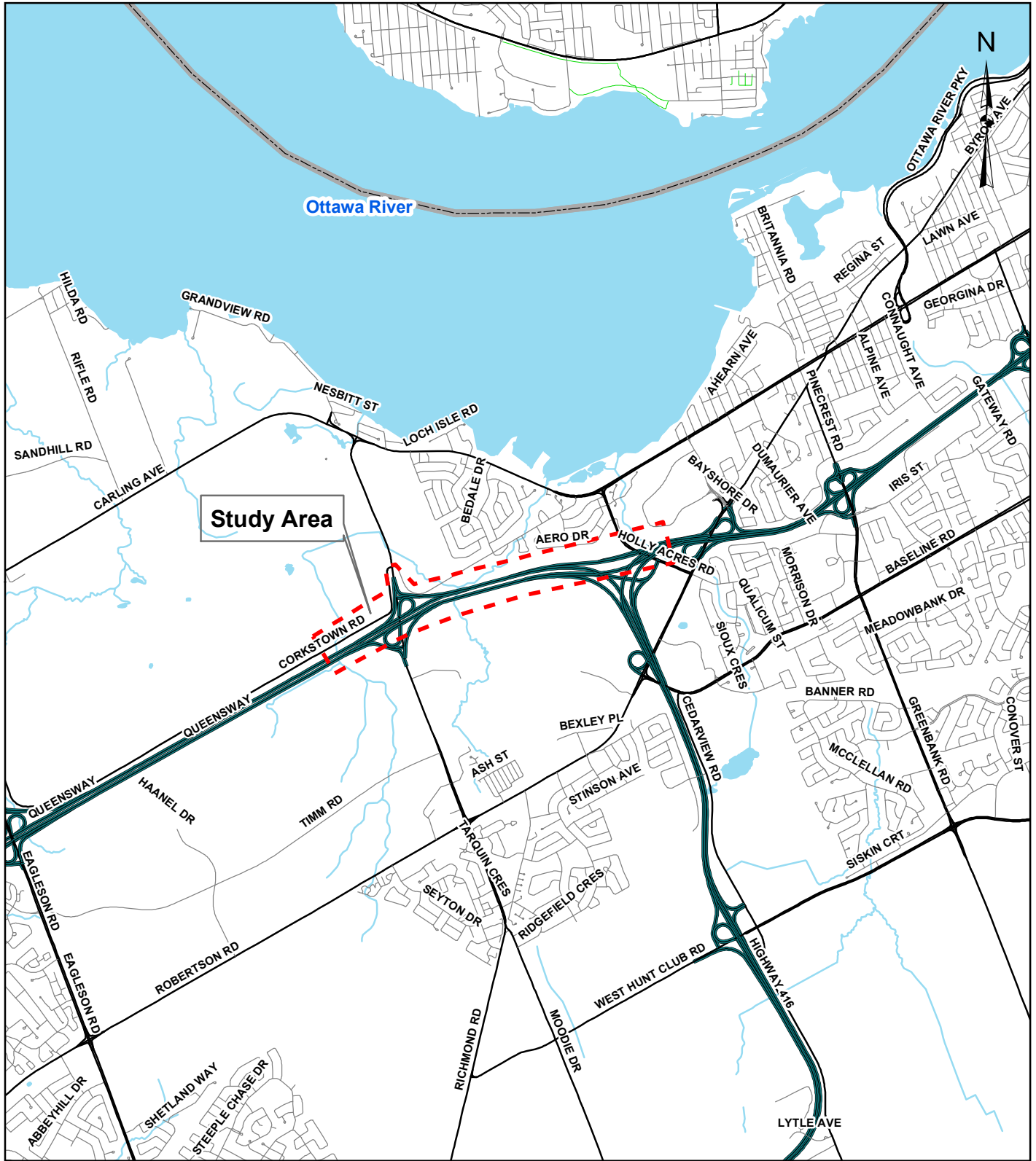
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

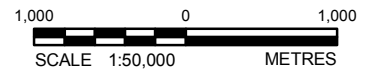
Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.


Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



REFERENCE

Digital base map data supplied by DMTI Spatial Inc. CANMAP, 2007
 Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 18



 <p>Golder Associates Ottawa, Ontario</p>	DATE	Feb. 10 2009	TITLE	<p>KEY PLAN</p>
	DESIGN	TMS	AB	
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SCALE	AS SHOWN	REV.	0	
				<p>FIGURE: 1</p>



LEGEND

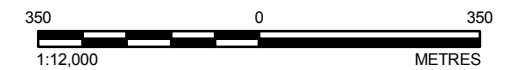
- ROADWAY
- RIVER OR STREAM
- WATERBODY
- SITE
- SURFICIAL GEOLOGY**
- 1a TILL, PLAIN WITH LOCAL RELIEF <5m
- 3 OFFSHORE MARINE DEPOSITS: CLAY, SILTY CLAY & SILT
- 3a_g OFFSHORE MARINE DEPOSITS: CLAY & SILT UNDERLYING EROSIONAL TERRACES (GULLIES & RAVINES)
- 5b NEARSHORE SEDIMENTS: FINE TO MEDIUM GRAINED SAND
- 6a ALLUVIAL DEPOSITS: SILTY SAND, SILT, SAND & CLAY
- 6a_g ALLUVIAL DEPOSITS: SILTY SAND, SILT, SAND & CLAY (GULLIES & RAVINES)
- 6b ALLUVIAL DEPOSITS: MEDIUM GRAINED STRATIFIED SAND WITH SOME SILT
- 7 ORGANIC DEPOSITS: MUCK & PEAT
- r2 BEDROCK: LIMESTONE, DOLOMITE, SANDSTONE & LOCAL SHALE
- zz WATER


SPECIAL NOTE

THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

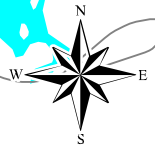
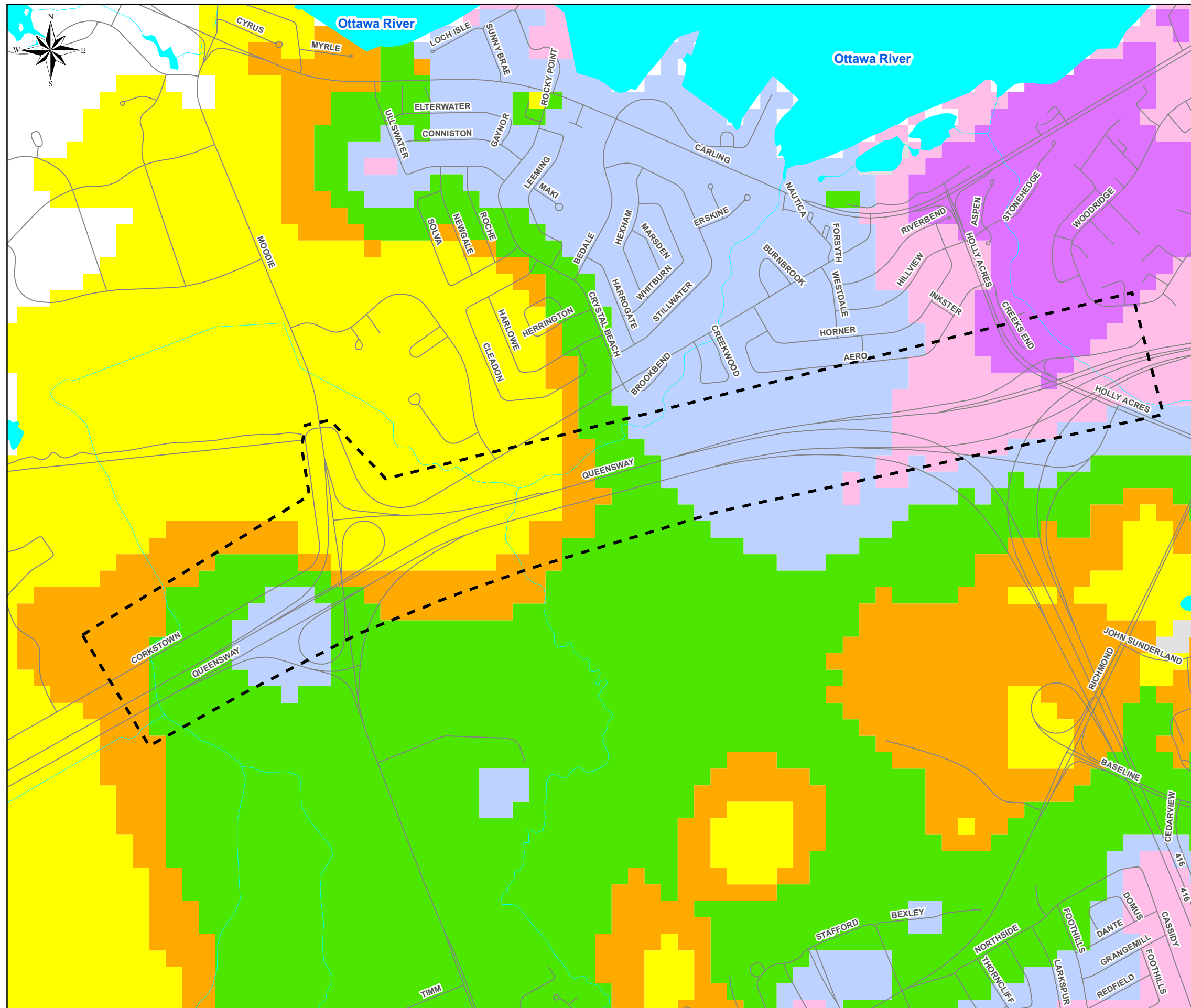
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BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001
 Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 18



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TITLE	SURFICIAL GEOLOGY		
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REVIEW	MIC 09 Jul. 2009		

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LEGEND

- ROADWAY
- RIVER OR STREAM
- WATERBODY
- SITE

TREND IN DEPTH TO BEDROCK METRES

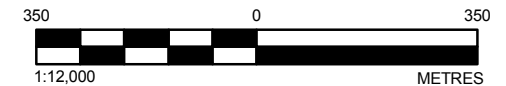
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- 5 to 10
- 10 to 15
- 15 to 25
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- 50 to 150


SPECIAL NOTE

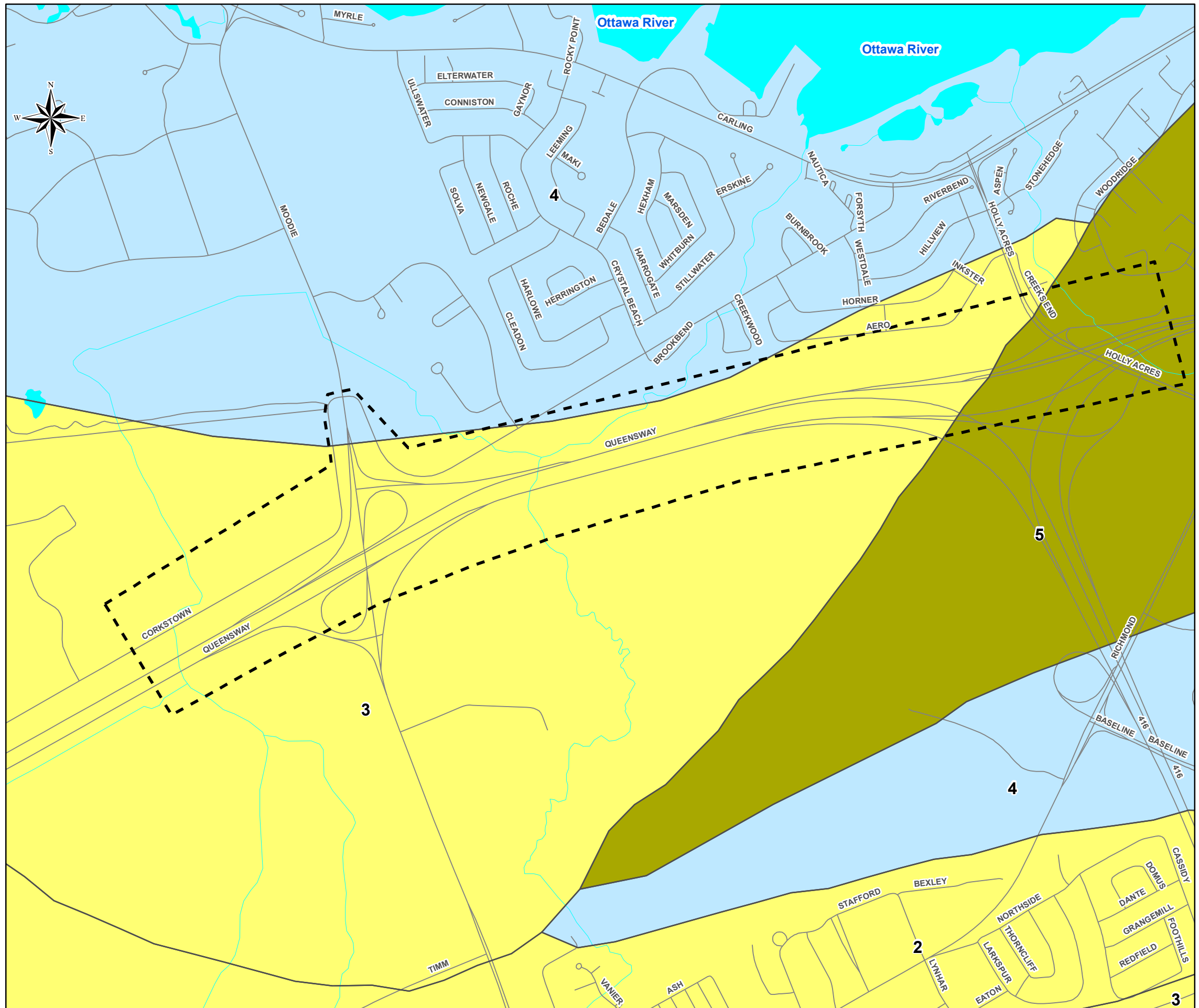
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REFERENCE

BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001
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PROJECT	WEST TRANSITWAY EXTENSION OTTAWA, ONTARIO		
TITLE	TREND IN DEPTH TO BEDROCK		
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	DESIGN	TMS 10 Feb. 2009	REV. 0
	GIS	AB 10 Feb. 2009	FIGURE: 3
	CHECK	TMS 09 Jul. 2009	
REVIEW	MIC 09 Jul. 2009		



LEGEND

- ROADWAY
- RIVER OR STREAM
- WATERBODY
- SITE

MIDDLE TO UPPER ORDOVICIAN

5 ROCKCLIFFE FORMATION: INTERBEDDED FINE-GRAINED LIGHT GREENISH GREY QUARTZ SANDSTONE, SHALEY LIMESTONE AND SHALE. LOCALLY CONGLOMERATE AT BASE, INTERBEDS OF CALCARENITE (ST. MARTIN MEMBER, 5A) AND SILTY DOLOSTONE IN UPPER PART

LOWER ORDOVICIAN

4 OXFORD FORMATION: SUBLITHOGRAPHIC TO FINE CRYSTALLINE DOLOSTONE

4* ALTERED FROM PUBLISHED MAPPING

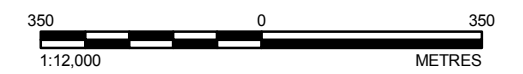
3 MARCH FORMATION: INTERBEDDED QUARTZ SANDSTONE, SANDY DOLOSTONE, AND DOLOSTONE

CAMBRIO ORDOVICIAN

2 NEPEAN FORMATION: FINE TO COARSE GRAINED QUARTZ SANDSTONE, PARTIALLY CALCAREOUS IN UPPER PART

REFERENCE

BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001
 Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 18



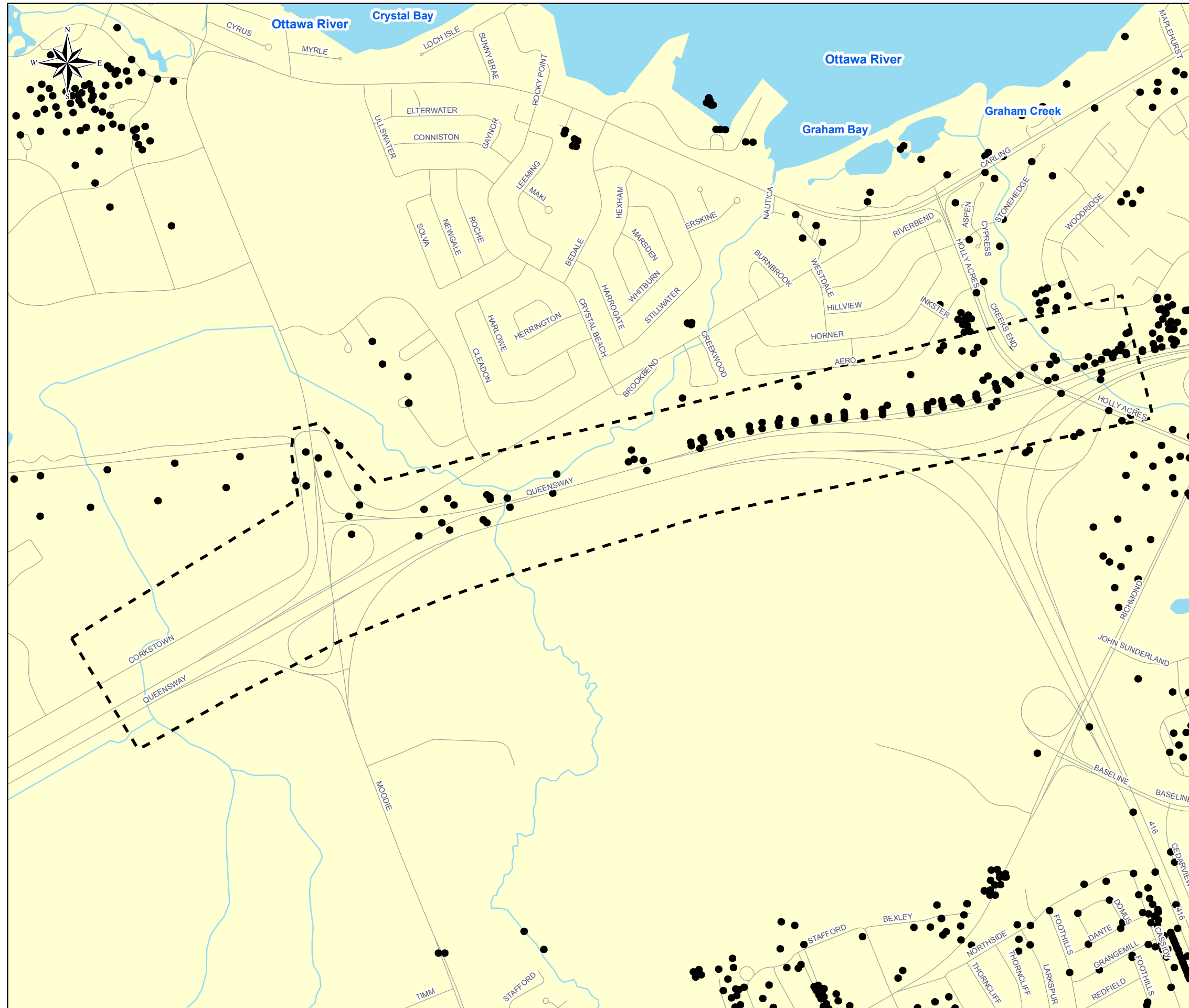
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 OTTAWA, ONTARIO

TITLE: **BEDROCK GEOLOGY**

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FIGURE: 4

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LEGEND

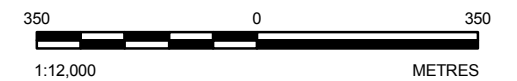
- TEST HOLE LOCATION
- ROADWAY
- RIVER OR STREAM
- WATERBODY
- ⋯ SITE


SPECIAL NOTE

THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

REFERENCE

BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001
 Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 18

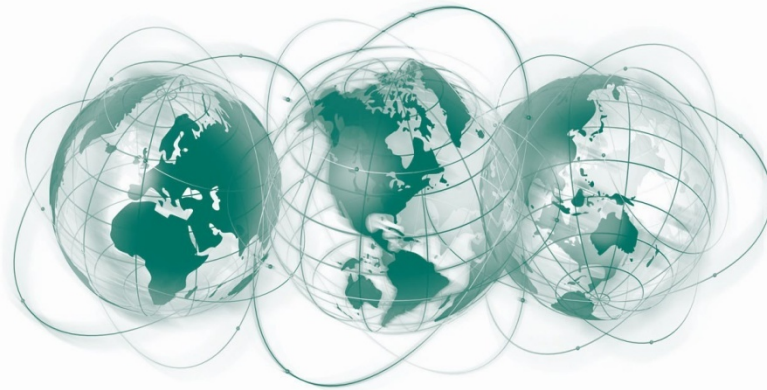


PROJECT	WEST TRANSITWAY EXTENSION OTTAWA, ONTARIO		
TITLE	EXISTING GOLDER GIS DATA		
	PROJECT No.	09-1121-0008	SCALE AS SHOWN
	DESIGN	TMS 10 Feb. 2009	REV. 0
	GIS	AB 10 Feb. 2009	FIGURE: 5
	CHECK	TMS 09 Jul. 2009	
REVIEW	MIC 09 Jul. 2009		

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