Appendix C:
Geotechnical Considerations

TECHNICAL MEMORANDUM

DATE March 24, 2015 **PROJECT No.** 1404896

TO Sheldon Dattenberger
Parsons Inc.

FROM Erin O'Neill, P.Eng.
Mike Cunningham, P. Eng.

EMAIL eoneill@golder.com
mcunningham@golder.com

OVERVIEW OF SUBSURFACE CONDITIONS AND POTENTIAL ISSUES/CONSTRAINTS
DOWNTOWN OTTAWA TRUCK TUNNEL FEASIBILITY STUDY
HIGHWAY 417 AT NICHOLAS STREET TO THE MACDONALD CARTIER BRIDGE

This memo provides a brief overview of the subsurface conditions within the study area for the Downtown Ottawa Truck Tunnel, as well as a preliminary assessment of issues/constraints, as input to the ongoing feasibility study.

BACKGROUND

The purpose of the current study is to assess the feasibility of a truck (or mixed-traffic) tunnel connecting the Macdonald-Cartier Bridge to Nicholas Street/Highway 417 in Ottawa, Ontario. The study area is bounded by Nicholas Street and Sussex Drive to the west, Nelson Street to the east, and the Macdonald Cartier Bridge to the north. The southern extent will depend on the connection details with Highway 417, and may extend to Mann Avenue, or possibly to the south side of Highway 417, depending on the onramp/offramp requirements.

The alignment and profile of the tunnel developed as part of this study will be driven by a number of factors, including: traffic considerations, community impacts, subsurface conditions, conflicts with existing buried infrastructure (e.g., deep sewers, transit tunnels, etc.) and the potential for impacts to adjacent structures within the surrounding high-density urban environment resulting from construction and post-construction drainage of settlement-sensitive clay. It is understood that a number of north-south corridors are being considered at this time, based primarily on traffic considerations (i.e., independent of geology).

The purpose of this memorandum is to provide the designers with a broad summary of the general subsurface conditions (soil, bedrock and groundwater) based on available geologic mapping and Golder Associates' extensive experience with the ground conditions in the Ottawa area. Future memos will focus on specific selected routes and will provide general guidelines on the geotechnical and hydrogeologic conditions associated with excavation and tunnel construction and identify important issues or challenges related to the anticipated soil and bedrock conditions along the selected alignment.

Although a part of Golder's overall scope for input to this feasibility study, an assessment of the environmental, archaeological and cultural heritage aspects of the project are provided separately.

SUBSURFACE CONDITIONS

Overburden Conditions

The study area is located in the minor physiographic region referred to as the Ottawa Valley Clay Plain (Chapman and Putnam, 1984) and is characterized by relatively thick deposits of sensitive marine clay overlying relatively thin, commonly reworked glacial till and glaciofluvial deposits that in turn overlie bedrock.

Where the bedrock or glacial till are present at shallow depth, the overlying clay deposits can be absent. More recent deposits of alluvial sand locally overlie the marine clay. Organic soils (such as peat) have also developed in some poorly drained areas. The published surficial geology map and trends in drift thickness (i.e., depth to bedrock) mapping from the Geological Survey of Canada (Belanger, 2008) are provided in Figures 1 and 2. The mapping does not always accurately reflect the contacts between different stratigraphic units as encountered during actual borehole drilling, but provides a general overview of the regional soil stratigraphy. More recent fill deposits are also not shown on the geological mapping but are common within the urban part of Ottawa being considered for this study.

Near-surface Fills and Alluvial Sand Cap

The study area is located in an urban setting, and near-surface fills beneath roadways and undeveloped areas, and within existing utility trenches, should be anticipated throughout the project area. The fill should be expected to be highly variable in composition, ranging from reworked silty clay to silty sand, to imported crushed stone. Based on select borehole data in the project area, it is expected that the fill typically ranges from about 2 to 4 metres in thickness, but thicker deposits should be anticipated along the Nicholas Street corridor adjacent to the Rideau Canal and at the Highway 417 ramps/approach embankments. Former landfills are known to be present at Highway 417, near King Edward Avenue and Templeton Street, and next to the Rideau River at Bordeleau Park.

A relatively thin alluvial sand cap is also indicated to be present along the central portion of the study area, in the neighbourhood known as Sandy Hill.

Champlain Sea Clay

The fill and alluvial sand cap, where present, are underlain within much of the study area by silty clay, clay and silt which were deposited within the post-glacial Champlain Sea basin. Often the upper 3 to 5 metres of the clay have been weathered and form a stiffer crust. Along the Nicholas Street corridor between about Mann Avenue and Laurier Avenue, the silty clay deposits are up to 20 metres (but more typically about 10 metres) thick and of firm to stiff consistency. Along Cumberland Avenue north of Laurier Avenue, the deposits are about 3 to 5 metres in thickness, are stiff to very stiff, and are expected to thicken to the east.

Known as Champlain Sea clay or Leda clay, these deposits are characterized by tiny, flat particles of mineral matter which are arranged like a house of cards with the cards at irregular angles but holding each other up; the spaces in between the particles are water filled. The particles are weakly cemented at the particle contact points. This cementation, together with erosion, has led to some overconsolidation of these deposits, resulting in a typically firm to stiff consistency. When the sensitive, unweathered silty clay soils are disturbed, or the loading is increased beyond its apparent preconsolidation pressure (e.g., through lowering of the groundwater table and/or additional building or embankment loading), the house of cards structure collapses and results in significant consolidation (i.e., compression) of the deposit.

Glacial Till and Glaciofluvial Deposits

The underlying glacial till deposits typically consist of a heterogeneous mixture of gravel, cobbles and boulders in matrix of silt and sand. The glacial till within the project area is anticipated to vary from about 1 to 10 metres in thickness. Locally, these deposits can be much thicker within infilled bedrock valleys, such as near the intersection of Laurier Avenue and Nicholas Street, and on Rideau Street between Sussex Drive and Nicholas Street. The glacial till and overlying marine clays are often separated by a thin transitional unit comprised of clayey silt to silt or fine sand, which is typically less than 2 metres in thickness.

Bedrock Conditions

Bedrock Geology

The study area is underlain by a series of conformable sedimentary rocks, consisting of Ordovician shales, limestones, dolostones and sandstones that are, in turn, underlain unconformably by igneous and metamorphic rock of the Precambrian Shield. Based on the surface bedrock map published by the Ontario Geological Survey (refer to Figure 3) and previous geotechnical drilling, including recent boreholes for the Ottawa Light Rail Transit (OLRT) and Combined Sewage Storage Tunnel (CSST) projects, the main geological formations found within the study area anticipated to be, from oldest to youngest, Verulam, Lindsay, Billings and Carlsbad formations.

The Verulam formation is characterized by thinly to medium bedded, fine to medium grained, grey, crystalline, moderately strong to strong limestone, with thin interbeds and seams of weak, black shale. The overlying Lindsay formation is characterized as an argillaceous, nodular, fossiliferous, grey, fine to coarse grained, medium strong limestone, with varying thickness of shale interbeds, with the percentage of shale increasing upwards in the sequence. The upper portion of the Lindsay formation contains a higher percentage of black, calcareous, fossiliferous, weak shale (>60%) with grey limestone interbeds of varying thickness. Conformably overlying the Lindsay formation is the Billings formation, which consists of a dark brown to black, moderately fossiliferous, slightly to non-calcareous, weak, slake susceptible shale. Fossils in the Billings formation are often pyritized. Gradationally superseding the Billings formation is the Carlsbad formation which consists of interbedded shale, fossiliferous calcareous siltstone, and silty limestone. Based on recent OLRT and CSST drilling, it is anticipated that Billings formations shales should be anticipated in the south portion of the study area (e.g., along Nicholas Street south of about Waller Street), and that limestones (Lindsay over Verulam formation) are present to the north. Note that the Ontario Geological Survey bedrock geology map (modified from Armstrong and Dodge, 2007) does not always accurately reflect the rock formations as encountered during drilling for previous projects.

The bedrock surface varies throughout the study area, but is anticipated to be closest to ground surface along the escarpment adjacent to the Ottawa River at the far north end of the study area, and becoming deeper to the southeast. Published mapping (see Figure 2) indicates that the bedrock is as deep as 10 to 15 metres below ground surface along most of Nelson Street and much of the study area south of Laurier Street, although bedrock was encountered at depths of 15 to 20 metres below ground surface along much of Nicholas Street.

Regional Tectonic and Seismic Setting

The downtown Ottawa area is dissected by the NW-SE trending Gloucester Fault and a series of secondary splay faults. Significant variation in the locations of inferred faulting is apparent when the older published mapping is compared with the more recent publications, partially due primarily to naming changes in geologic formations, and partly from the fact that the faults themselves are generally concealed beneath the overburden or downtown infrastructure, such that their plan positions have only been inferred. Within the study area, published mapping indicates that a fault is located south of Laurier Street East, crossing the study area between about the MacKenzie King Bridge and Osgoode Street. Drilling carried out during previous investigations for the CSST, OLRT and Somerset Wastewater Storage Facility (SWSF) suggests that the main throws of the fault may run more parallel to the Rideau Canal (roughly along the Nicholas Street corridor east of the Canal), with several secondary faults forming graben-type structures.

While faults within the study area are not considered to be seismically active, vertical offsets of adjacent rock blocks can range from one to more than 25 metres. At depth, faults encountered in previous project borings typically comprise less and about 1 metre (thickness) of broken or crushed rock or gouge material, and are frequently calcite healed. Faults within the Ottawa area are sometimes associated with deep overburden zones, areas of poorer quality rock and higher groundwater inflows, particularly where faults intersect the soil/rock interface. Deep erosional features within the bedrock surface are known to exist east of the Rideau Canal near the intersection of Rideau and Sussex Streets (mapped at over 33 metres depth), at the intersection of Laurier Street East and Nicholas Street (about 23 metres deep), and at Nicholas Street and Waller Street (about 26 metres deep).

Groundwater and Hydrologic Setting

Major water features within the study area include the Ottawa River to the north of the project area and the Rideau River to the east. The Rideau Canal is located just west of the study area; however, it is a man-made seasonal watercourse and is understood to have minimal connectivity to the bedrock groundwater flow system. Regional groundwater flow in downtown Ottawa is generally north towards the Ottawa River and east towards the Rideau River.

Extensive deposits of coarse and permeable overburden, capable of supplying sufficient quantities of groundwater for domestic use, are not present in the study area. Aquifers within the Billings, Lindsay and Verulam formations are typically poor producers, where flow primarily occurs along horizontal fractures along bedding planes and vertical fractures caused by bedrock jointing. These formations are considered low to moderate in terms of available yield. The hydraulic conductivity of the bedrock typically varies from about 10^{-6} to 10^{-8} metres per second (decreasing with depth).

Geotechnical Issues for Consideration

The comments provided below are intended only to assist with evaluating the general feasibility of tunnel construction, and to assist in evaluating the advantages, disadvantages, risks and relative costs associated with different alignments. At the time of writing, no information was available on the tunnel profile (i.e., grade), size/diameter of tunnel, or proposed construction methods (e.g., cut and cover or bored tunnel).

These comments are based on available geologic mapping and our general knowledge of the study area. Additional review of available information (and potentially field investigations) will be required to confirm the overall feasibility of the selected/shortlisted options.

- The presence of existing tunnels within the downtown core may restrict the feasible profile and alignment of the truck tunnel. As a general guideline, a minimum (horizontal and vertical) separation of at least two tunnel diameters should be maintained from existing tunnels, including the IOS, OLRT and CSST. If tunnel construction under private property or along narrow right-of-ways is proposed, there may be potential conflicts with existing or proposed basements/foundations.
- The profile of any fully-tunneled option should ideally be picked to keep the tunnel fully within either soil or bedrock (i.e., to avoid mixed faced or transitional conditions). Ideally the tunnel should have two to three diameters of crown cover to maintain adequate stability. Depending on the size of the proposed tunnel, this may mean that sections of the tunnel may be more cost-effectively constructed as cut and cover, rather than as a bored tunnel.

- In areas of significant overburden thickness, shaft excavations (or those for cut-and-cover construction) may require costly excavation shoring. Impacts on adjacent structures resulting from ground movements due to shoring or tunnel construction will need to be considered. The selected excavation and support methods will need to consider the presence of boulders in the glacial till, as well as the potential for flowing ground conditions below the groundwater table in sandier zones (as frequently encountered at the transition between the Champlain Sea Clay and underlying glacial till).

- In areas overlain by (or in close proximity to) Champlain Sea Clay deposits, the extent of groundwater level drawdown resulting from excavation activities will need to be evaluated, including the potential for widespread underdrainage of the clay via the bedrock. Widespread lowering of the piezometric pressure in the clay deposits, particularly in areas of the city with heavy structures founded on raft slabs on clay, would result in increased loading of the sensitive silty clay which, if stressed close to or beyond its pre-consolidation pressure, could result in damaging consolidation settlements.

- Depending on the selected alignment and profile of the tunnel, excavations may extend into or be tunneled entirely within bedrock. In the Verulam and Lindsay limestones (expected along the northern portion of the tunnel alignments), discontinuity controlled wedges and blocks and the intrinsic hardness of the formation will constitute the main geotechnical concerns, as the rock is expected to be competent and strong to very strong. In the south part of the study area, the tunnel could encounter weak to medium strong Billings shale, which consists entirely of black shale. The main issues in this rock would be time-dependent deformation behaviour, swelling, and slaking that could have an impact on the design and construction of the tunnel lining. Depending on the size, depth, and means of construction, feasible methods of excavation within the bedrock could include drill and blast, sequential excavation (SEM is currently being undertaken for the OLRT project), or tunnel boring (as is planned for the CSST project). Blasting or other ground vibrations associated with rock excavation can likely be controlled, but impacts to surrounding utilities and structures should be considered.

- Known faults within the Ottawa area can sometimes be associated with deep overburden zones and potentially areas of poor quality rock. It is expected that Nicholas Avenue parallels a large offset fault along much of its length (between at least Laurier Avenue East and Somerset Street East) and a deep buried valley is known to exist along Nicholas Street between Waller to north of Laurier Avenue East. A similar deep erosional valley is known to exist east of the Rideau Canal near the intersection of Rideau and Sussex Streets with the bedrock surface mapped at over 33 metres depth. The existing Interceptor Outfall Sewer was designed to skirt these features. The OLRT Rideau Station excavation passes through this buried valley.

- Depending on the expected tunnel profiles, alignments may encounter soil or groundwater contamination. The tunnel alignments are located in areas of past industrial activity, so it is possible that any groundwater alevel lowering could mobilize contaminated groundwater from sites above and adjacent to the tunnel alignment and draw it towards/into the tunnel. Areas of potential concern identified at this early stage include: a former Gas Plant west of King Edward Avenue between York and George Streets; poor fill quality along the Nicholas Street alignment (which follows old railway lines); and three former landfills, one near Highway 417, one east of King Edward Avenue and south of Templeton Street, and one northeast of Rose and Bruyere Streets. There are also several gas stations and dry cleaners along King Edward Avenue and in the market area.

Closure

We trust that the summary of general ground conditions and preliminary geotechnical issues related to tunnel construction meet with your current needs. We look forward to providing more site-specific information and input as the project continues. Should you require further information at this time, please do not hesitate to contact us.

Yours truly,

GOLDER ASSOCIATES LTD.

Erin O'Neill, P. Eng.
Associate, Geotechnical Engineer

Michael Cunningham, P. Eng.
Principal, Geotechnical Engineer

ESO/MIC/md
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Attachments: Important Information and Limitations of this Report
Figure 1 – Surficial Geology
Figure 2 – Drift Thickness (Depth to Bedrock)
Figure 3 – Bedrock Geology

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 practicing 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.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, **Parsons Inc.** The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

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.

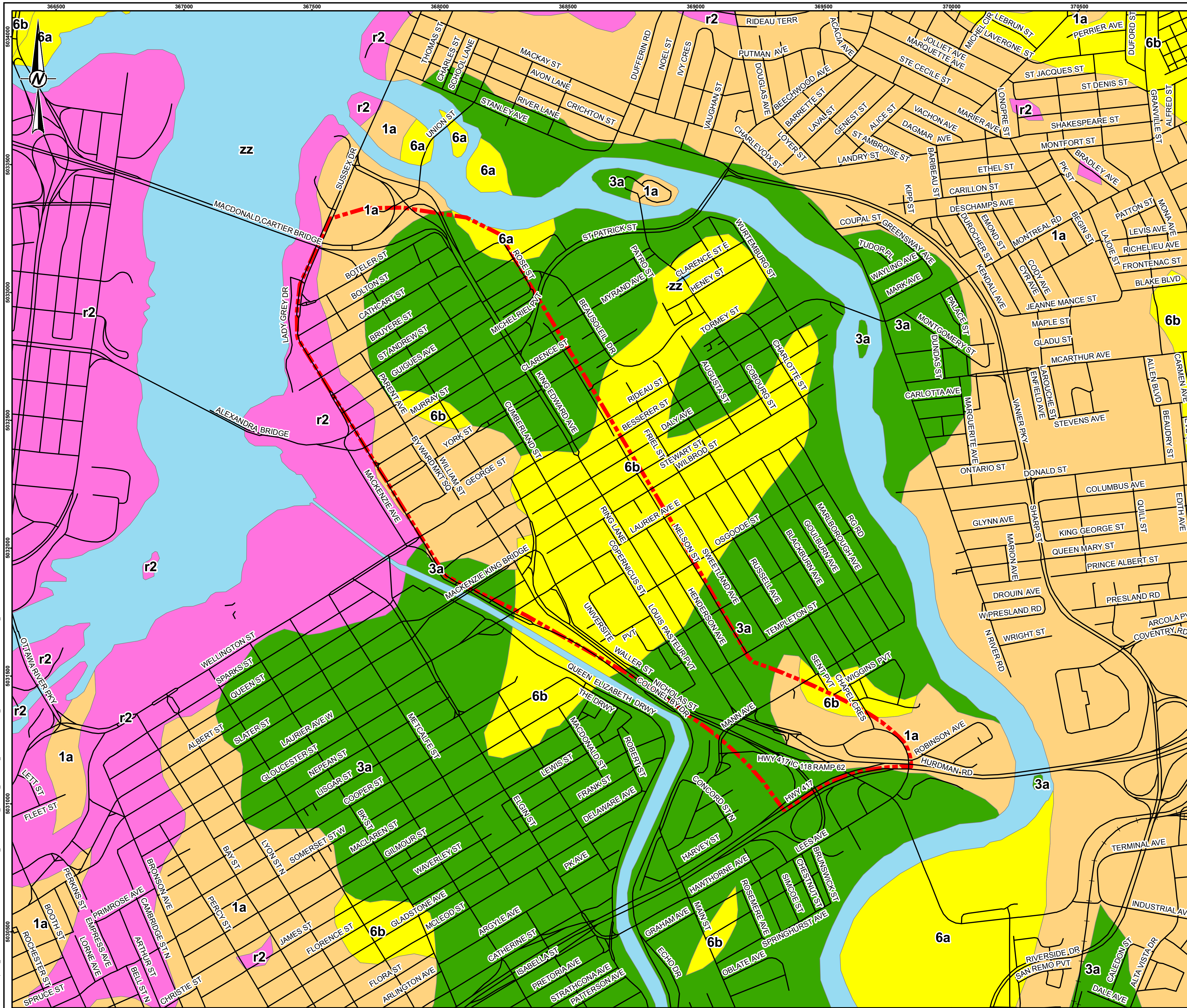
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.



LEGEND

- ROADWAY
- RAILWAY
- STUDY AREA
- 6a: ALLUVIAL DEPOSITS: SILTY SAND, SILT, SAND & CLAY
- 6b: ALLUVIAL DEPOSITS: MEDIUM GRAINED STRATIFIED SAND WITH SOME SILT
- 3a. OFFSHORE MARINE DEPOSITS: CLAY, SILT UNDERLYING EROSIONAL TERRACES
- 1a. TILL, PLAIN WITH LOCAL RELIEF <5 m
- r2. BEDROCK: LIMESTONE, DOLOMITE, SANDSTONE & LOCAL SHALE
- zz. WATERBODY

0 150 300 600

1:15,000 METRES

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

REFERENCE(S)

1. BÉLANGER, J. R. 2008 URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE 5311, 1 DVD.
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
CITY OF OTTAWA

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
SURFICIAL GEOLOGY

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-03-24
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:

25mm

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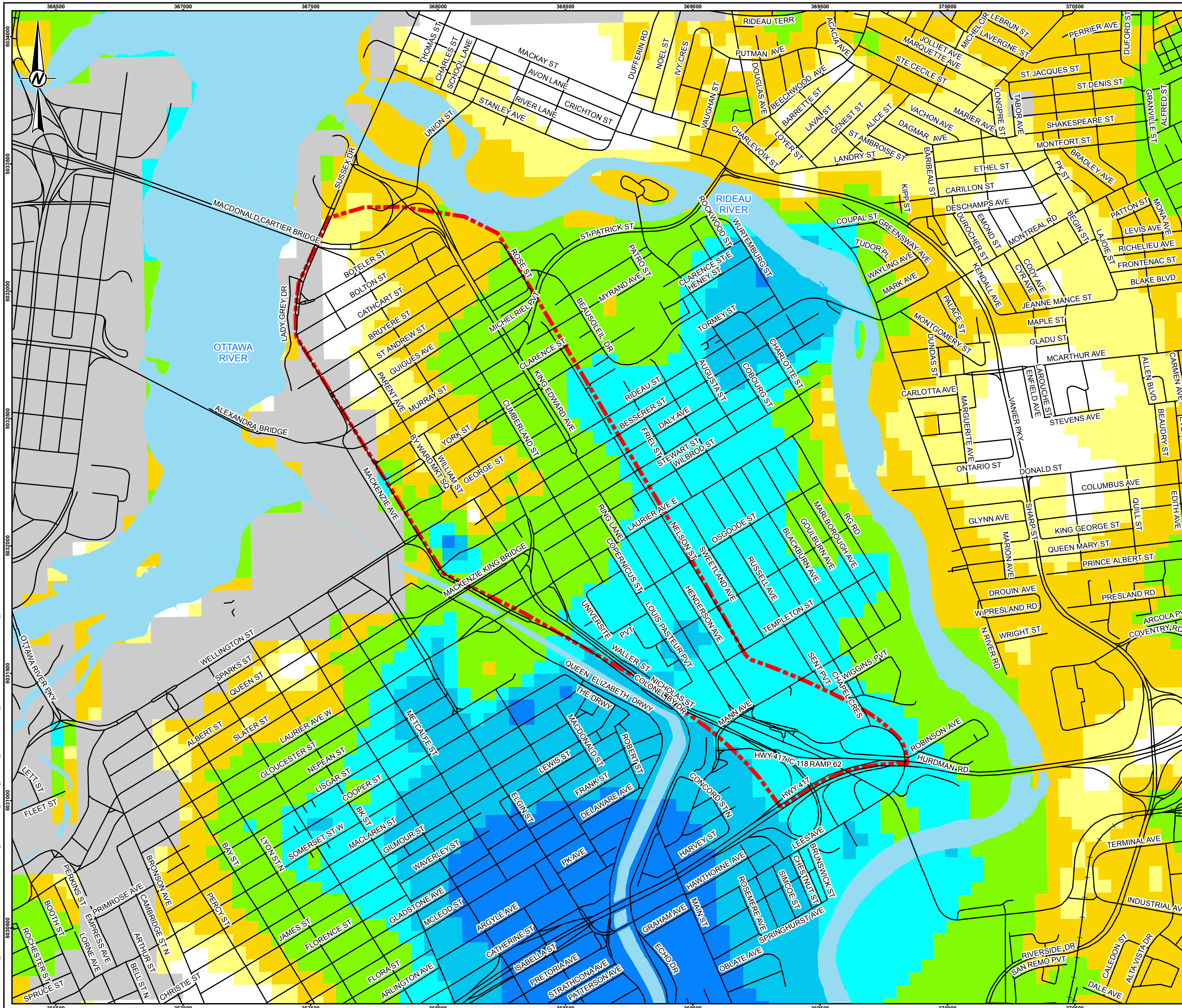

LEGEND

- ROADWAY
- RAILWAY
- WATERBODY
- STUDY AREA

TREND IN DEPTH TO BEDROCK (METRES)

- 0 to 1
- 1 to 2
- 2 to 3
- 3 to 5
- 5 to 10
- 10 to 15
- 15 to 25
- 25 to 50
- 50 to 100
- 100 to 200

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

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 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
CITY OF OTTAWA

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
DRIFT THICKNESS

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-03-24
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
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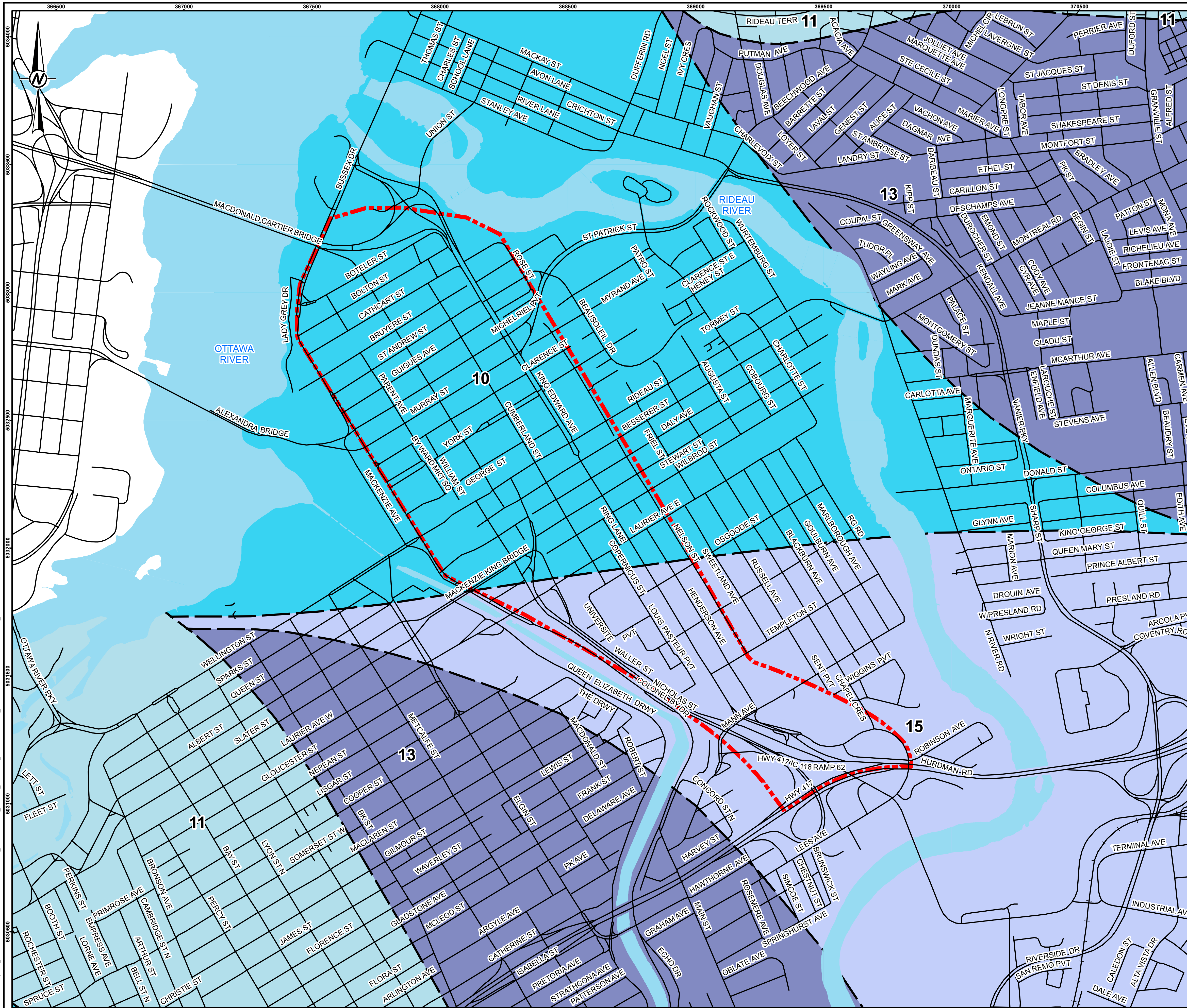

LEGEND

- ROADWAY
- RAILWAY
- WATERBODY
- STUDY AREA
- 15: CARLSBAD FORMATION - SHALE AND LIMESTONE
- 13: BILLINGS FORMATION - SHALE, MINOR LIMESTONE
- 11: LINDSAY FORMATION - LIMESTONE; NODULAR TO BLACK LAMINATED
- 10: VERULAM FORMATION - LIMESTONE AND SHALE

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

REFERENCE(S)

1. ARMSTRONG, D.K. AND DODGE, J.E.P. PALEOZOIC GEOLOGY MAP 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, © QUEENS PRINTER 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
BEDROCK GEOLOGY

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-03-24
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:

25mm

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TECHNICAL MEMORANDUM

DATE	October 27, 2015	**PROJECT No.**	1404896/1000/1500
TO	Mr. Chris Gordon, P. Eng. Parsons Inc.	**CC**	Moreno Pescara GEODATA Engineering S.p.A.
FROM	Erin O'Neill, P.Eng. Mike Cunningham, P.Eng.	**EMAIL**	eoneill@golder.com

GEOTECHNICAL AND HYDROGEOLOGICAL OVERVIEW ASSESSMENT
PREFERRED TUNNEL ALIGNMENT
DOWNTOWN OTTAWA TRUCK TUNNEL
OTTAWA, ONTARIO

Golder Associates Ltd. (Golder) was retained by Parsons Inc. (Parson) to carry out a geotechnical overview assesement for the proposed Downtown Ottawa Truck Tunnel project. This memo provides a brief overview of the subsurface conditions along the preferred alignment (as determined by the study team), as well as a preliminary assessment of issues/constraints, as input to the ongoing feasibility study.

1.0 BACKGROUND

The overall purpose of the current study is to assess the feasibility of a truck (or mixed-traffic) tunnel connecting the Macdonald-Cartier Bridge to Nicholas Street/Highway 417 in Ottawa, Ontario. The original study area was bounded by Nicholas Street and Sussex Drive to the west, Nelson Street to the east, the Macdonald-Cartier Bridge to the north, and Highway 417 to the south. During the course of the assessment, the eastern limit of the study area was later expanded to include alignments as far east as the Vanier Parkway.

A number of north-south corridors were considered by the larger study team (Parsons, GEODATA, Golder, and various stakeholders including the MTO, MTQ, NCC, City of Ottawa and others) as part of the study to connect the Macdonald-Cartier Bridge at the north end of the alignment to either Nicholas Street or the Vanier Parkway near Highway 417. The preferred alignment and profile of the tunnel developed by the team as part of the study was driven by a number of factors, including: traffic considerations, community impacts, subsurface conditions, conflicts with existing buried infrastructure (e.g., deep sewers, transit tunnels, etc.) and the potential for impacts to adjacent structures within the surrounding high-density urban environment resulting from construction and post-construction drainage of settlement-sensitive clay.

The preferred tunnel alignment and arrangement selected by the study team is as follows:

- Three kilometer long twin tunnels are planned, with excavated diameters of approximately 13 metres each, and invert levels (not including the 5 percent grade portal approaches) at between about 24 and 35 metres elevation. Tunnels are roughly 25 metres apart (centreline to centerline) with 4 metre by 4 metre cross passages connecting the two tunnels at roughly 250 metre spacings. No shafts or surface access points are planned.

- The preferred alignment is provided on Figures 1 through 3. Most of the tunnel alignment follows roughly subparallel to (and some 50 and 400 metres west of) the Rideau River, passing under the Sandy Hill and Lowertown neighbourhoods of Ottawa. The area is a mixture of older residential neighbourhoods, with some taller residential and commercial buildings located along the Rideau River, and along streets such as Rideau Street, Laurier Avenue, Stewart Street, and others. A number of embassies are also located in close proximity to the alignment. The alignment generally does not follow road right-of-ways, but as much as possible avoids being directly under these structures.

- Two tunnel portals would be required; one located at North River Road at Coventry Road near the Vanier Parkway and Highway 417 interchange (south portal), and a second portal located between King Edward Avenue and the Macdonald-Cartier Bridge (north portal). The portals would likely involve cut-and-cover construction to transition from the at-grade streets to the twin tunnels.

The purpose of this memorandum is to:

- provide the designers with a summary of the general subsurface conditions (soil, bedrock and groundwater) based on available geologic mapping, boreholes advanced as part of previous studies in the vicinity of the tunnel alignment, and Golder Associates' experience with the ground conditions in the Ottawa area; and,

- based on the anticipated subsurface conditions, provide preliminary, planning-level guidelines on the geotechnical and hydrogeologic conditions associated with excavation and tunnel construction and identify important issues or challenges related to the anticipated soil, bedrock and groundwater conditions along the selected alignment.

Although a part of Golder's overall scope for input to this feasibility study, assessment of the environmental, archaeological and cultural heritage aspects of the project are provided separately.

2.0 ANTICIPATED SUBSURFACE CONDITIONS

The published surficial geology map and trends in drift thickness (i.e., depth to bedrock) mapping from the Geological Survey of Canada (Belanger, 2008) are provided in Figures 1 and 2, respectively. Published bedrock geology mapping by the Geologic Survey of Canada and Ontario Geological Survey are provided on Figures 3A and 3B, respectively. The mapping does not always accurately reflect the contacts between different stratigraphic units or bedrock formations as has been encountered during drilling for previous projects, but provides a general overview of the regional soil and bedrock stratigraphy. More recent fill deposits are also not shown on the geological mapping but are common within the urban part of Ottawa being considered by this study.

A simplified geologic profile is provided on Figure 4 for the southbound tunnel. Given the proximity of the two tunnels, the subsurface conditions are not expected to vary significantly between the northbound and southbound lanes.

The geologic profile is based on available subsurface information (i.e., boreholes advanced as part of previous studies along with published geologic mapping) and our experience with the ground conditions in the Ottawa area. Only a limited number of boreholes have been advanced along the alignment as part of previous studies, and most of those are shallow (i.e., do not extend to the tunnel horizon nor, in many cases, to the rock surface). In the Sandy Hill area, which is a topographic high, the deepest boreholes typically do not extend more than about 5 or 10 metres depth below the ground surface, tens of metres above the anticipated tunnel horizon. Where boreholes do extend to the bedrock surface, most do not penetrate more than a metre or two into bedrock. As such, in the absence of study-specific drilling, the profile shown and discussions below provide our best estimate/interpretation of the ground conditions along the tunnel alignment. Additional site-specific subsurface investigation will be required as part of feasibility, functional, preliminary and detailed design phases of the project.

2.1 Overburden Conditions

The study area is located in the minor physiographic region referred to as the Ottawa Valley Clay Plain (Chapman and Putnam, 1984) and is characterized by relatively thick deposits of sensitive marine clay overlying relatively thin, commonly reworked glacial till and glaciofluvial deposits that in turn overlie bedrock. Where the bedrock or glacial till are present at shallow depth, the overlying clay deposits can be absent. More recent deposits of alluvial sand locally overlie the marine clay. Organic soils (such as peat) have also developed in some poorly drained areas.

2.1.1 Near-Surface Fills and Alluvial Sand Cap

The tunnel alignment is located in an urban setting, and near-surface fills beneath roadways, parking areas, existing utility trenches and undeveloped areas (e.g. parklands), should be anticipated throughout the project area. The fill should be expected to be highly variable in composition, ranging from reworked silty clay to silty sand, to imported crushed stone. Based on select borehole data in the area of the portals, it is expected that the fill at the north portal is 1 to 3 metres thick and consists of silty sand fill with metal and concrete debris and cobbles. Thicker fills would be encountered if the portal excavations extended into the existing 9 metre high bridge embankment. At the south portal, fills are expected to consist of sand with varying amounts of silt, clay, and gravel, and are also expected to be about 1 to 3 metres thick.

Former landfills are known to be present next to the Rideau River (e.g., at Bordeleau Park, at the northeast corner of Rose Street and Bruyere Street).

Thin alluvial deposits should also be anticipated along the Rideau River and may be present at both portal locations. A relatively thin alluvial sand cap is also indicated to be present along the central portion of the study area, in the neighbourhood known as Sandy Hill.

2.1.2 Champlain Sea Clay

The fill and alluvial sand cap, where present, are underlain within much of the study area by silty clay, clay and silt which were deposited within the post-glacial Champlain Sea basin. Often the upper 3 to 5 metres of the clay have been weathered/dessicated and form a stiffer crust. Along the central part of the alignment between Laurier Avenue and Beausoleil Drive in the area known as Sandy Hill, the silty clay deposits are about 10 to 15 metres thick and, below the depth of weathering, are of firm to stiff consistency. North of Beausoleil Drive, the deposits are typically less than about 5 metres in thickness and are stiff to very stiff. Clay deposits are not anticipated at either of the portal locations.

Known as Champlain Sea clay or Leda clay, these deposits are characterized by tiny, flat particles of mineral matter which are arranged like a “house of cards” with the cards at irregular angles but holding each other up; the spaces in between the particles are water filled. The particles are weakly cemented at the particle contact points. This cementation, together with erosion, has led to some overconsolidation of these deposits, resulting in a typically firm to stiff consistency (at least in the Central portions of Ottawa). When the sensitive, unweathered silty clay soil is loaded beyond its apparent preconsolidation pressure (e.g., through lowering of the groundwater table and/or additional building foundation or embankment loading), the house of cards structure collapses and results in significant consolidation (i.e., compression) of the deposit.

2.1.3 Glacial Till and Glaciofluvial Deposits

The underlying glacial till deposits typically consist of a heterogeneous mixture of gravel, cobbles and boulders in matrix of silt and sand. Cobbles and boulders of various origins (sedimentary, igneous, and metamorphic rocks) are all commonly encountered. Boulders up to 3 metres in maximum dimension are not uncommon, although boulders up to a maximum size of 1.5 metres are more typical. The glacial till and overlying marine clays are often separated by a thin transitional unit comprised of clayey silt to silt or fine sand, which is typically less than 2 metres in thickness.

The glacial till along the tunnel alignment is anticipated to vary from about 1 to 6 metres in thickness. At the north portal and approach, a thin layer of till (i.e., less than 0.5 metres thick) is expected. At the south portal and approach, the glacial till is expected to be about 1 to 5 metres thick.

2.2 Bedrock Conditions

As mentioned previously, only a limited number of boreholes have been advanced in the general vicinity the tunnel alignments as part of previous studies, and many of those do not extend to the rock surface, particularly in the Sandy Hill area. The top of rock profile shown on Figure 3 provides our "best guess" of the bedrock surface along the tunnel alignment based on available data, and is primarily based on boreholes advanced south of about Laurier Avenue and north of about Murray Avenue. A relatively large data gap exists in the Sandy Hill area and the bedrock surface along this part of the alignment is based primarily on boreholes advanced several hundred metres away from the alignment. Additional site-specific subsurface investigation will be particularly important for this section of the alignment as part of subsequent phases of the project to confirm the rock elevations, particularly in this area.

2.2.1 Bedrock Surface

Based on available information, the bedrock surface along the tunnel alignments is anticipated to be at between about 48 and 55 metres elevation. Because of the sedimentary nature of the bedrock, the bedrock surface is expected to be relatively flat-lying. However, some steps and/or erosional features such as buried stream channels in the bedrock surface should be expected. For example, it is not uncommon for the bedrock surface in the central parts of Ottawa to step up and down by 1 to 3 metres over relatively short distances.

The depth of the bedrock surface below ground surface varies significantly along the tunnel alignment, predominantly because of changes in ground surface topography. At the south portal, the rock surface is expected to be approximately 4 to 5 metres below the existing ground surface. Between the river crossing and the north portal, the bedrock surface is anticipated to be between 1 and 20 metres below ground surface. North and south of the proposed river crossing location, bedrock is visible within the river bed when river levels are at their lowest. At the north portal, the depth to the bedrock surface is expected to be between about 1 and 3 metres beneath the general ground surface (i.e., not including beneath the embankment). The bedrock is exposed at the extreme north end of the alignment below the south abutment of the Macdonald-Cartier Bridge.

2.2.2 Bedrock Geology

The study area is underlain by a series of conformable sedimentary rocks, consisting of Ordovician shales, limestones, dolostones and sandstones that are, in turn, underlain unconformably by igneous and metamorphic rock of the Precambrian Shield. Based on the surface bedrock maps published by the Ontario Geological Survey (refer to Figure 3A) and the Geological Survey of Canada (Figure 3B) the main geological formations found at the bedrock surface within the study area are anticipated to be, from oldest to youngest, the Verulam, Lindsay, Billings and Carlsbad Formations.

The Verulam Formation is characterized by thinly to medium bedded, fine to medium grained, grey, crystalline, moderately strong to strong limestone, with thin interbeds and seams of weak, black shale. The overlying Lindsay Formation is characterized as an argillaceous, nodular, fossiliferous, grey, fine to coarse grained, medium strong limestone, with varying thickness of shale interbeds, with the percentage of shale increasing upwards in the sequence. The upper portion of the Lindsay Formation contains a higher percentage of black, calcareous, fossiliferous, weak shale (i.e., typically greater than 60 percent) with grey limestone interbeds of varying thickness. Conformably overlying the Lindsay Formation is the Billings Formation, which consists of a dark brown to black, moderately fossiliferous, slightly to non-calcareous, weak, slake susceptible shale. Gradationally superseding the Billings Formation is the Carlsbad Formation which consists of interbedded shale, fossiliferous calcareous siltstone, and silty limestone.

Along the tunnel alignment, very few previous/existing boreholes had been advanced more than a metre or two below the “top of rock” surface and very few are logged with sufficient geologic detail to determine with any certainty the corresponding bedrock formation. The deepest boreholes within the general study area are from the recent Ottawa Light Rail Transit (OLRT) project, Combined Sewage Storage Tunnel (CSST) project and Somerset Combined Sewage Outfall (Somerset CSO) tunnel drilling investigations as well as investigations for the rehabilitation of the Macdonald-Cartier Bridge. At the Macdonald-Cartier Bridge (north of the far north end of the alignment), a deep borehole indicates primarily Verulam Formation limestones extending down to 10 metres elevation. The CSST boreholes along Cumberland Street and in Bordeleau Park indicate that middle and lower Lindsay Formation limestones are present and extend to at least 35 metres elevation, and thus Lindsay or Verulam Formation limestones are also expected along the tunnel horizon in the central portion of the alignment. At Somerset Street east of Range Road, a deep borehole was advanced to approximately 12 metres elevation. At this location, the bedrock at the tunnel horizon (i.e., between elevation 31 and 43 metres) is indicated to be Lindsay and Verulam Formation limestone. At the south end of the alignment, the bedrock consists of shale bedrock, which is indicated in the published mapping to be Carlsbad Formation.

2.2.3 Regional Tectonic Setting and Faulting

The downtown Ottawa area is dissected by the NW-SE trending Gloucester Fault and a series of secondary splay faults. Significant variation in the locations of inferred faulting is apparent when the older published mapping is compared with the more recent publications, partially due to naming changes in geologic formations, and partly from the fact that the faults themselves are generally concealed beneath the overburden or downtown infrastructure, such that their plan positions have only been inferred. Along the proposed alignment, current published mapping indicates that a roughly east-west trending fault may be present at the south end of the study area, intersecting the tunnel alignment somewhere between Mann Avenue and Somerset Street. The mapping suggests significant vertical offsets on either side of the fault (i.e., Carlsbad Formation south of the fault and Verulam Formation to the north). Older published mapping indicates a lithology change, rather than a fault (i.e., Billings Formation shale on the east side of the river, and Lindsay Formation limestone on the west side of the river).

While faults within the study area are not considered to be seismically active, vertical offsets of adjacent rock blocks have been observed to range from one to more than 25 metres as part of previous subsurface investigations in the downtown core. At depth, faults encountered in previous project borings typically comprise less than about 1 to 2 metres (thickness) of broken or crushed rock or gouge material, and are frequently calcite healed. Faults within the Ottawa area are sometimes associated with deep overburden zones, areas of poorer quality rock and higher groundwater inflows, particularly where faults intersect the soil/rock interface. Deep erosional features within the bedrock surface are known to exist east of the Rideau Canal near the intersection of Rideau and Sussex Streets (mapped at over 33 metres depth), at the intersection of Laurier Street East and Nicholas Street (about 23 metres deep), and at Nicholas Street and Waller Street (about 26 metres deep). Linear trends in bedrock lows in the vicinity of Bruyere and Rose Streets and in the area of Beausoleil Drive may also be indicative of possible faults in these areas.

2.3 Groundwater and Hydrologic Setting

Major water features within the study area include the Ottawa River to the north of the project area and the Rideau River to the east. The Rideau Canal is located just west of the study area; however, it is a man-made seasonal watercourse and is understood to have minimal connectivity to the bedrock groundwater flow system. Regional groundwater flow in downtown Ottawa is generally north towards the Ottawa River and east towards the Rideau River.

Extensive deposits of coarse and permeable overburden, capable of supplying sufficient quantities of groundwater for domestic use, are not present in the study area. Aquifers within the Billings, Lindsay and Verulam Formations are typically poor producers, where flow primarily occurs along horizontal fractures along bedding planes and vertical fractures caused by bedrock jointing. These formations are considered low to moderate in terms of available yield. The hydraulic conductivity of the bedrock typically varies from about 10^{-6} to 10^{-8} metres per second (decreasing with depth).

3.0 GEOTECHNICAL CONSIDERATIONS

The comments provided below are intended only to assist with evaluating the general feasibility of tunnel construction along the preferred alignment and profile. These comments are based on available geologic mapping and our general knowledge of the study area, as well as a limited number of available boreholes along the alignment. Additional study and investigation will be required to confirm the overall feasibility of the selected alignment and portal locations at the EA and functional design stages.

3.1 Feasibility Constraints and Considerations

It is understood that, to accommodate two lanes of traffic in each direction, the twin tunnels with excavations on the order of 13 metres in diameter are planned to be constructed at between about 24 and 35 metres elevation, as shown on the profiles provided by Parsons (Figure 4). The profile of the preferred alignment has been partially dictated at the north end to avoid direct conflicts with both the Interceptor Outfall Sewer (IOS) and planned CSST gravity flow sewers, while maintaining a sufficiently low-gradient entry/exit slope along the transition from the surface road to the tunnel. Both the IOS and CSST sewers are (or will be) tunneled in rock and have invert elevations of about 41 and 42 metres, respectively. Similar constraints also exist at the south end of the alignment, at Somerset Street and along the west bank of the Rideau River, where main trunk sewers are located.

It is understood that portal construction would likely involve cut-and-cover construction to transition from the at-grade streets to the twin tunnels. No shafts or surface access points between the north and south portal areas are currently planned.

Based on the above design constraints and the proposed alignment and profile of the preferred alignment, the following key geotechnical issues will need to be considered for tunnel construction:

- Rock Cover: The potential for encountering reduced or minimal rock crown cover, and the potential for ‘mixed face’ tunnelling conditions if there are significant changes in the “top of rock” elevation;
- Rock Mass Quality: The potential for rock conditions which could impact on the feasibility or costs of tunnel construction (e.g., poor working face stability, increased support requirements, etc.);
- Significant Geological Features such as Major Faults: Major fault zones could result in construction delays and increased costs associated with highly broken/fractured bedrock resulting in increased rock support and with higher groundwater inflows;
- Groundwater Management: The potential for excessive groundwater inflow and the impacts of groundwater level lowering (e.g., clay consolidation settlement); and,
- Impacts to Adjacent Structures: The potential for ground movements resulting from settlement, vibration or uncontrolled loss of ground that may impact on roadways, utilities or structures.

Construction of the portals and approaches can also present a significant challenge/constraint. The elevation of the bedrock surface and the overlying soil and groundwater conditions will dictate the construction methods at these locations. Some potential issues of concern relate to the following:

- The shoring requirements for overburden excavations, such as may be required due to: portal and approach depths; the proximity of excavations to structures, roadways, utilities, or waterways; and, the presence of weak or permeable ground conditions.
- The potential for excessive groundwater inflow if the approaches will be constructed through permeable overburden or bedrock, which is important in regards to: the significant pumping that could be required; the need for a dewatering discharge of suitable capacity (which meets MOECC and City requirements for water quality); and the associated potential for settlement of surrounding structures due to the groundwater level lowering.

Additional considerations which will need to be considered in preliminary/detailed design, including comments on spoils management and disposal, shale heave, etc. are also provided.

3.2 Local Tunnelling Experience

The Ottawa area has had numerous tunnel projects over the past 40 years, almost exclusively for water and wastewater conveyance, which have ranged in size from 2 to 4 metres in diameter. A recent exception is the OLRT tunnel, which is currently under construction in the downtown core, and averages 10 metres by 7 metres in cross section. Tunnels have been constructed within the local sedimentary bedrock formations using cut and cover, tunnel boring machine (TBM), road headers, hoe-ramming, hand mining or drill and blast methods. Bedrock tunnels constructed by TBM are often less problematic than those constructed by drill and blast which sometimes require additional post-blast scaling and increased temporary support.

Tunnels constructed by TBM within these sedimentary rocks generally only require temporary support in the form of rock bolts, wire mesh and pins and/or shotcreting. In some cases, additional temporary rock support or liners may be required in areas with less rock cover or where poorer quality rock, faulted rock, or highly weathered rock is encountered. TBM tunnels in the local limestone bedrock have had support issues at the tunnel haunches and crown depending on the tunnel alignment in relation to the orientation of the near vertical and orthogonal jointing common for these formations. In such cases, there can be a tendency for localized blocks or wedges of rock at the haunches, and/or delamination of slabs in the crown defined by the jointing, to fall into the tunnel after excavation. Where these conditions are expected to exist or be encountered, additional rock bolts with or without wire mesh are typically used to increase stability. Tunnels in the local shale bedrock are prone to roof failures due to delamination and shearing of shale layers along open bedding planes bounded by vertical jointing in the crown. The shale is also prone to ravelling failures over time as the shale degrades. The stand-up time of the shales can be very short in some cases and immediate support close to the face can be critical. Additional temporary support measures (e.g. steel ribs, lattice girders, liner plates, or shotcrete) have been required to maintain stability prior to installation of the final tunnel liner.

The OLRT tunnel is being constructed using the sequential excavation method (SEM) and is being excavated with roadheaders. SEM excavation allows for optimization of the ground support to mobilise the rock or soil strength and is typically employed for tunnels with non-circular cross-sections, such as for the current OLRT tunnel. The ground conditions along the proposed tunnel are similar to those along the OLRT tunnel, with the exception of the shale bedrock, and are similarly favourable for SEM tunnelling. The final lining is typically (although not always) installed after completion of excavation along the full length of the tunnel (as is being done for the watertight section of the OLRT tunnel) which can result in significant periods of groundwater lowering during construction (i.e., before installation of the watertight lining).

In areas overlain by, or adjacent to, compressible clays, rapid lining of the tunnel excavations has been mandated on the CSST project to limit underdrainage of the bedrock and the resulting potential for consolidation settlements of the overlying clays.

3.3 Geotechnical Considerations Related to Tunnel Construction

Given the depth and length of excavations and the urban setting, it is likely that the tunnels would be constructed using either conventional excavation methods (e.g. sequential excavation methods using a roadheader, or potentially drill and blast construction) or a large diameter tunnel boring machine (TBM). Available geotechnical information indicates that the tunnels will be constructed primarily within limestone or interbedded limestone and shale bedrock, although predominantly shale bedrock may potentially be encountered between the south portal and close to Somerset Street.

3.3.1 Rock Cover

The tunnels are planned for construction at an invert elevation of between about 24 and 35 metres elevation, as shown on the profiles provided by Parsons. As shown on the attached profile drawing (Figure 4), the main tunnel alignments are expected to be entirely within rock. Mixed face conditions (i.e., a mixture of soil and rock within the tunnel excavation) are not expected within the tunnel excavations, but should be expected within the cut and cover portions of the alignment. Available subsurface information indicates that the tunnels will generally have between about 8 and 14 metres of rock crown cover along the main tunnel alignment (i.e., along the 0.4% grade portions of the alignment), with lesser amount of cover indicated by previous drilling in some areas (e.g., on the west side of, and possibly at, the Rideau River, and near Old St. Patrick Street). At the north end of the alignment, before the start of the cut and cover section where the tunnel is at a 5.6 percent gradient, the rock crown cover diminishes from about 8 metres at King Edward Avenue to less than 2 metres of cover at the start of the “artificial” cut and cover tunnel.

It is understood that the assessment of the feasibility/adequacy of the rock cover/profile is being carried out by GEODATA; however, the following general guidelines are provided based on general practice. For bedrock tunnels to have adequate crown stability, they typically require a minimum thickness of competent rock in the zone above the tunnel (ideally 1.5 to 2 tunnel diameters above the top of the tunnel, as a rough guideline, but potentially less depending on the tunnel excavation method, tunnel diameter/cross section geometry, and the bedrock quality). In areas with limited rock cover, there is an increased likelihood that additional rock reinforcement in the crown will be required prior to or during excavation (e.g., grouted rock anchors, spiles, steel ribs, lattice girders or liner plates, reinforced shotcrete or pre-cast concrete liners or pre-cast concrete segments installed behind a double-shield TBM) to maintain stability. At and near the Rideau River, where there is potentially only 6 to 8 metres of rock cover, there is an added risk of having high water flow into the tunnel due to a direct hydraulic connection between the river via fractures in the bedrock (see discussions in Sections 3.3.4).

Areas of low rock cover identified above, particularly those in close proximity to the Rideau River, should be selected for further study early on in the EA and/or functional design investigation program to determine the depth to bedrock and bedrock quality at and above the tunnel horizon in these areas. Also, because of the relatively large data gap in Sandy Hill between about Laurier Avenue and Murray Avenue, additional site-specific subsurface investigation should be undertaken in this area early in subsequent phases of the project to confirm rock elevations. It is possible that more “buried valleys” exist along the tunnel alignment like the ones indicated on Laurier Avenue East and on Rideau Street between Sussex Drive and Nicholas Street, however the risk of encountering an unexpected valley along the tunnel alignments will be reduced by advancing boreholes at regular intervals, as will be required as in future design phases of the project.

3.3.2 Rock Mass Quality

Based on available information and the proposed tunnel elevations, it is expected that the bedrock encountered along most of the tunnel alignment will be Lindsay and Verulam Formation limestone. Carlsbad and/or Billing Formation shales should be anticipated in the southern portion of the alignment, beginning somewhere between Somerset Street and Mann Avenue.

Drilling carried out as part of previous tunnel projects, including the OLRT, CSST and CSO Somerset investigations, indicates that the limestone and interbedded limestone and shale of the Lindsay and Verulam Formations is typically fresh to slightly weathered, horizontally and thinly to thickly bedded, moderately strong to strong (unconfined compressive strengths typically in the range of 40 to 150 megapascals). Near vertical jointing is common, at a spacing that is typically in the range of 0.5 to 2 metres. Approximately orthogonal joint sets are common, resulting in rectangular rock blocks. Testing carried out on samples of limestone collected as part of the OLRT and CSST projects from the Lindsay and Verulam formation limestones indicate that they have very low to low abrasivity, as would be expected for these types of rocks. Billings Formation shale is typically fresh to moderately weathered, laminated to thinly bedded, weak to medium strong (unconfined compressive strengths typically in the range of 10 to 40 megapascals). Little testing has been carried out in the Carlsbad formation interbedded shale, fossiliferous calcareous siltstone, and silty limestone.

Little is known about the quality of the deep bedrock at the tunnel horizon along the preferred alignment, which is located much deeper than most of the previous investigations in the area. Boreholes advanced into the bedrock along most of the alignment often do not extend more than a metre or to below the top of rock, and are frequently part of investigations carried out some time ago (i.e., many dating back to the 1960's and 1970's) and contain very little information on the quality of the rock encountered. Data collected for the CSO Somerset tunnel (Somerset Street between the Rideau River and King Edward Avenue), however, indicates good to excellent rock quality (RMR 60 to 100) for the sewer tunnel horizon located between about 26 and 30 metres elevation, which is in both Lindsay and Verulam Formation limestones. Further to the west, where the CSO Somerset tunnel passes through Billings Formation shales (west of the Rideau Canal), the rock quality was indicated to be poor to fair (RMR 20 to 60).

As discussed above, the rock quality is generally expected to be good to excellent within the Verulam and Lindsay formation limestones anticipated along most of the tunnel alignment. It is envisaged that, where adequate rock cover exists, a regular pattern of rock bolts with mesh installed immediately behind the cutter head or excavation face as the tunnel advances would be sufficient to maintain the safety of the tunnel for workers over the course of construction. The final tunnel liner will need to be designed to withstand the full hydrostatic head above the tunnel and some additional vertical loading from unsupported rock blocks in the tunnel crown (temporary steel support elements, such as rock bolts, will corrode over time and cannot be relied upon for long term reinforcement).

In zones of poor to fair quality rock, additional rock reinforcement may be required prior to or during excavation to maintain crown stability. The final concrete liner in these zones may also need to be strengthened with steel reinforcement. Tunnels in the shale bedrock are prone to roof failures due to delamination and shearing of shale layers along open bedding planes bounded by vertical jointing in the crown, and to ravelling failures over time as the shale degrades. As such, if shale is encountered along the tunnel alignment (i.e., south of about Somerset Street), additional support measures (e.g. steel ribs, lattice girders, liner plates, or shotcrete) may be required to maintain stability prior to installation of the final tunnel liner. Zones of very poor to fair quality rock should be expected within the top 1 to 2 metres below the soil/rock interface (i.e., at the north portal approaches, where the rock can be slightly to moderately weathered, and tends to be more highly fractured). Zones of very poor to fair rock quality may also be encountered immediately adjacent to buried valleys and at localized fault zones as discussed in Section 3.3.3 below.

Three or four deep boreholes extending to below the tunnel horizon should be advanced early investigation to confirm the rock quality and rock formations along the preferred tunnel alignment.

3.3.3 Fault Zones

Published geological literature indicates that several faults cross the study area. Faults are zones of highly disturbed rock that were created when large areas, or blocks, of rock were displaced relative to each other during historical geologic events. Faults in the Ottawa area are considered to be inactive (i.e., no movement in Holocene period). The locations of these 'known' faults are not mapped accurately in the published geology mapping and the actual location may be tens of metres distant from that shown on the mapping. Additional faults and tributary faults likely also lie within the study area. Recent investigations along the OLRT, CSST and CSO Somerset tunnel alignments through the downtown core identified numerous faults with vertical shifts in the stratigraphic sequence (vertical 'throw') of up to approximately 30 metres. Most of the significant faulting is located east of about Nicholas Street, although other faults with smaller throws were inferred along the CSO Somerset alignment at Blackburn Avenue and at King Edward Avenue, and along the CSST alignment on Cumberland Street at Wilbrod Street, George Street and Guigues Avenue.

Based on the results of these and other previous investigations in the downtown area, these fault zones are often steeply inclined to near vertical and are typically in the order of 1 to 2 metres in width; although faults with much larger widths have been mapped in the Ottawa area. The fault zones can sometimes be associated with deep overburden valleys. Faults often contain highly fractured rock which has been healed with calcite veins; however, voids and fault gouge (finely ground rock that is soil-like in consistency and behaviour) have also been encountered. The rock quality in fault zones through shale bedrock tend to be of very poor to poor quality, while those in limestone tend to be of poor to fair quality. Some fault zones can have higher hydraulic conductivity than the surrounding rock mass, while other faults (i.e., healed faults) have similar hydraulic conductivity to the surrounding rockmass. Bedding planes within the rock adjacent to the fault can be distorted and folded or rotated from their typically near horizontal orientation.

At the south end of the alignment, published mapping suggests that the fault between Somerset Street and Mann Avenue trends in an E-W direction, roughly orthogonal to the tunnel alignment at this location. As such, the apparent width of the fault (or set of faults), as measured along the tunnel alignment, is generally expected to be limited and the fault zone is unlikely to significantly impact the overall feasibility of the tunnel. Faulted ground will, however, impact the rate of progression of the tunnelling work. While some faults are expected to present little difficulty in crossing, others may require additional effort to cross due to their width and increased permeability. Faulted zones with poor to fair quality rock and/or higher hydraulic conductivity will likely require additional temporary rock support or liners, or possibly grouting ahead of the tunnel face during construction.

Faults in the downtown area generally trend in the NW-SE direction, roughly parallel to the preferred tunnel alignment north of about Stewart Street. While unlikely (and not indicated in any published mapping), it is possible that because the tunnel alignment runs roughly parallel to the general trend in faulting, the apparent width of a fault along this portion of the alignment may be large and that the rock along a significant section of the tunnel may be fractured, folded, or otherwise altered, which could in turn impact on the tunnelling conditions (e.g. poor rock quality, higher groundwater infiltration, soft ground) and would have a significant impact on the tunnelling costs and schedule.

It will likely not be feasible or cost effective to determine the ground conditions at every fault that crosses the tunnel horizon. However, in areas where these faults and structurally disturbed zones are expected, angled core drilling and borehole geophysics (including natural gamma, conductivity and acoustic/optical televiewer surveys) will be necessary to obtain adequate baseline data for determining representative discontinuity fabric characteristics of the rock mass, for checking the lithology of the rock, to examine in situ rock quality, and to allow checks to be made for the presence of gouge zones and high permeability zones. Further investigation to evaluate the possible presence of a fault south of Somerset Street and (although a low likelihood) north of Stewart Street, should be undertaken early in the design process to confirm the alignment selection.

3.3.4 Groundwater Management

The groundwater inflow from the limestone bedrock is generally not expected to be so great as to impact on the feasibility of tunnel construction. Hydraulic conductivities within the local sedimentary rocks are generally low to moderate and are typically controlled by fracture flow. The hydraulic conductivity for the Verulam and Lindsay Formation limestones encountered along the OLRT and CSST alignments ranged from 10^{-6} to 10^{-9} metres per second.

Higher groundwater inflows will likely be associated with some of the more permeable fault zones that the tunnel alignments will cross, and also with areas with limited rock cover such as at the Rideau River crossing, where fractures in the rock may be hydraulically connected to the river. It is expected that these higher groundwater inflows can often be controlled by grouting ahead of the tunnel face. A crossing of the Rideau River (by the IOS) was successfully completed using drill and blast techniques without any known significant issues, however this was a much smaller diameter tunnel and is located more than 2 kilometres north of the planned crossing location for this tunnel.

A detailed hydrogeological assessment will ultimately be needed to assess the expected rate of groundwater pumping to be required during construction at the portals, cut and cover tunnels, and along the tunnel alignments (particularly at the river crossing), and to assess the potential for dewatering-induced settlements, as part of a Permit-to-Take Water (PTTW) application. For the anticipated rates of pumping and the size of the project, a Category 3 PTTW from the Ministry of Environment and Climate Change will be required.

As part of the hydrogeological assessment, the off-site effects of possible groundwater lowering must be considered for both during construction (from active dewatering/depressurization within the tunnels and at shaft locations) and in the long term (as a result of leakage into the tunnel or portals). This issue is especially important in areas where sensitive Champlain Sea clays exist within the zone of influence of the dewatering. Tunnels under construction and constructed tunnels that are not watertight could act as subsurface drains. Uncontrolled dewatering of the bedrock along the tunnels and at the portals would result in drainage of the surrounding bedrock, which would then ‘under drain’ a surrounding area, and could lead to drawdowns in the bottom portion of the overlying soils. Where a property/structure within the zone of groundwater level lowering in underlain by clays, some settlement of adjacent raft/footing foundations for structures could result from the clay being stressed close to or beyond its pre-consolidation pressure. This issue is particularly relevant to the portion of the tunnel alignments beneath Sandy Hill (i.e., between about Laurier Avenue and Beausoleil Drive), where the tunnel is overlain by some 10 to 15 metres of sensitive Champlain See Clay. In this area a large number of multi-storey buildings have been constructed which may be supported on raft slabs on clay, as discussed in Section 3.3.5.1 below. The northern portion of the alignments (between Rideau Street and Cathcart Street) and the portions of the alignment between Somerset Avenue and the west side of the Rideau River, which are also overlain by clay, and will also, need to be checked.

3.3.4.1 Hydrogeological Modelling

Based on the preliminary project details and the subsurface information collected from previous projects, a very preliminary hydrogeological assessment was carried out to provide initial estimates of the potential extent of temporary groundwater level drawdown that could occur during construction of the tunnels and portal excavations. The model used was that developed for the CSST and OLRT projects located further to the west of the study area and was updated for this project to include additional historical borehole data closer to the alignment. These historical data were compiled into a geological database that was used to develop the various geological surface for the groundwater flow model. The main geological surfaces used in the model were: base of fill; base of Champlain Sea clay; base of glacial till (i.e., top of bedrock); base of shallow bedrock (i.e., 5 metres below the base of glacial till / top of bedrock surface); and base of intermediate bedrock (i.e., 10 metres below the base of glacial till / top of bedrock surface) for the reasons described below.

Constant head model boundaries were used to represent the Ottawa River and Rideau River. The Rideau Canal was not treated as a constant head boundary because the canal is an artificially excavated feature that is not directly linked (hydogeologically) to the bedrock. For the forecast simulations, seepage node boundaries were used to represent the dewatering required for the underground workings.

Surficial recharge was applied to the uppermost surface of the model as a constant flux. The recharge rates applied in the groundwater flow model were based on those specified in previous groundwater flow models (e.g., the OLRT and CSST models), and were ultimately refined through a calibration process. Recharge rates applied to the upper model boundary are summarized as follows: 15 millimetres per year (mm/year) was applied in the downtown area and clay zone; 50 mm/year was assigned in all other areas. Throughout the simulation, a condition was specified such that recharge would be applied only to model elements where the pressure head was less than zero (i.e., the groundwater level was below ground surface).

The hydraulic parameters assigned in the model are provided in Table 1 below, and are consistent with those used in the previous CSST model. In the absence of site specific testing, these hydraulic parameters are considered reasonable for this feasibility-level assessment, will need to be confirmed with additional testing in the future stages of the project. The results of packer testing data from the OLRT and CSST projects indicate that the hydraulic conductivity of the bedrock is controlled more by the depth below the rock surface rather than by bedrock formation. As such, the hydraulic conductivities of the bedrock were assigned in the model based on depth, and not bedrock formation, with the upper 5 and 10 metres of bedrock typically being more permeable. For the glacial till and rock units, the hydraulic conductivity values were based on the median hydraulic conductivity determined from the results of in-situ hydraulic conductivity tests performed as part of previous projects in the downtown Ottawa area. The hydraulic conductivity value assigned to the clay unit was based on results from previous Golder projects in the Ottawa area. The hydraulic conductivity of the fill unit was selected based on previous modelling, and refined through the calibration process. A ratio of 10 Horizontal to 1 Vertical was used for the clay and bedrock units.

Specific storage represents the volume of water that is released from the aquifer due to compression of the matrix following depressurization. The specific storage value assigned to the clay unit (0.001 m^{-1}) was selected based on the results of consolidation tests completed as a part of the CSST project and other projects completed in downtown Ottawa. The specific storage values for the other overburden strata were based on literature values and assumed to be 0.001 m^{-1} for the fill unit and $1\text{x}10^{-4}$ m^{-1} for the glacial till unit. Based on the results of pumping tests on the CSST project, the specific storage for the rock units was assigned a value of $1\text{x}10^{-5}$ m^{-1} (Note: specific storage = storativity ÷ aquifer thickness).

Table 1: Regional Model Hydraulic Parameters

Material	Hydraulic Conductivity (m/s)		Specific Storage (m^{-1})
	Horizontal - K_H	Vertical - K_v	
Fill	$2x10^{-5}$	$2x10^{-5}$	$1x10^{-3}$
Clay	$1x10^{-8}$	$1x10^{-9}$	$1x10^{-3}$
Glacial till	$2x10^{-6}$	$2x10^{-6}$	$1x10^{-4}$
Shallow bedrock (0 to 5 metres)	$5x10^{-6}$	$5x10^{-7}$	$1x10^{-5}$
Intermediate bedrock (5 to 10 metres)	$3x10^{-7}$	$3x10^{-8}$	$1x10^{-5}$
Deep bedrock (> 10 metres)	$4x10^{-8}$	$4x10^{-9}$	$1x10^{-5}$

The hydrogeological modelling was carried out stages, as the project details were refined, as described below.

- Scenario 1: Full Tunnel Length Open for 4 years (Long-term drainage)

  1) The full lengths of the tunnels and portals would be unlined, and therefore act as groundwater drains, for an extended period. As a simplification, the full lengths of the tunnels were modelled as being built instantaneously.
  2) A simulation length of 4 years was assigned, which approximates steady state conditions.
  3) Cross-passages have not been included in the model, for simplicity.

At the request of the project team, additional modeling was then carried out to simulate both conventional and mechanized tunnel construction. Refinements were made to the drainage conditions simulated in regional hydrogeologic model to reflect these new details on the construction methodology and timing developed by the study team. These analyses included the following additional details:

- Scenario 2: Excavation, draining and subsequent installation of watertight lining of the tunnels advanced *progressively* using conventional tunneling techniques:

  1) Tunnel excavation advances progressively at a rate of 20 metres per week, from the south portal to the north portal.
  2) Excavation of the northbound tunnel is advanced 50 metres ahead of the southbound tunnel.
  3) Any single 150 metre section of tunnel is only open (i.e., draining) for 53 days before being permanently lined and made watertight. For practical purposes, the tunnels were specified in the model as a series of seepage nodes (one at the crown and one at the invert of each tunnel) to represent the open length of tunnel on a 150 m length basis, such that a 53 days' worth of tunneling is "switched on" in the model, and then sealed ("switched off") at the same time that the subsequent 150 m length boundaries are "switched on". This approach is considered conservative, as the length of open tunnel is overestimated for any given time.
  4) The west portal and cut and cover section were included as a part of the model boundaries representing the tunnel. The east portal is located outside of the model domain (east of the Rideau River). Cross-passages have not been included in the model, for simplicity.

To evaluate the potential changes in piezometric head at the top of the rock assuming that the tunnel was being constructed with a TBM with pre-cast concrete tunnel liner segments installed immediately behind the TBM shield but no pressurization of the tunnel face (e.g., with a double shield TBM), the drainage conditions simulated in the regional model were modified to include the following:

- Scenario 3: Excavation, draining and rapid installation of watertight lining of the tunnels advanced using mechanized tunneling techniques:
  1) The rate of advance provided by the designers was 15 metres per day, with only a single 15 metre length in each tunnel draining at any time.
  2) For practical purposes, the tunnels were specified in the model as a series of seepage nodes that were progressively "switched on and off" to represent the open length of tunnel on a weekly basis. It was assumed that tunneling would occur 5 days of the week (therefore a 75 m length of tunnel would be completed per week).
  3) Cross-passages have not been included in the model, for simplicity.

A fourth scenario was also proposed by the study team, which involves construction of the tunnels using a fully pressurized-face TBM, which would result in no changes in groundwater pressures resulting from tunnel construction. In this scenario, the only features that would be draining along the tunnel alignments would be the portal and cut and cover tunnel areas, and the 4 metre by 4 metre access cross passages which would be installed to connect the two tunnels at roughly 250 metre intervals along the tunnel alignment.

3.3.4.2 Predicted Groundwater Drawdowns

The resulting predictions of groundwater drawdown within the project area for Scenario 1 (at the end of the 4 years of draining) are shown on the attached Figure 5. The contours shown on the figure correspond to changes in the groundwater level (piezometric head) in the upper portions of the bedrock at the end of 4 years of unmitigated drainage. The results of that modelling show significant and far-reaching drawdowns. As indicated in the figures, the predicted drawdowns are up to approximately 10 metres in magnitude directly above the tunnels and extend up to about 1 kilometre from the alignment (based on the 0.5 metre drawdown contour).

The results of the modelling of Scenario 2 are illustrated on Figure 6 for a representative section of the tunnel. A review of the model results indicates that the simulated groundwater drawdown at the top of the bedrock, for this particular section of simulated tunnel construction, extends up to approximately 400 m towards the southwest and 325 m towards the northeast of the proposed alignment (as defined by the 0.5 m contour).

The results from the Scenario 3 simulation are attached Figure 7. In this simulation the TBM advances 15 metres per day. With the limited duration of drainage, drawdown at the top of rock was predicted to be less than 0.5 metres for the duration of this simulation (i.e., a plot showing drawdown contours for the top of rock would show no drawdown). Instead, the attached image in Figure 7 shows drawdown at the tunnel horizon.

Scenario 4 has not yet been modelled, however groundwater drawdowns are expected to be less than those for Scenario 3, except possibly in the areas of the cross-passages, where some, more localized, drawdown would be expected.

The following provides a list of the predicted drawdowns at some of the taller buildings, which are present along, or within the general proximity of, the preferred alignment through the Sandy Hill neighbourhood, which are underlain by the thickest deposits of compressible clays, and which are potentially supported on raft foundations.

Table 2: Predicted Drawdowns at Select Building Locations

Building Address	Building Height (number of stories)	Maximum Predicted Drawdown at the Top of Rock (m)		
		Scenario 1	Scenario 2	Scenario 3
423 Rideau Street	22	6	0.1	<0.5
161 Augusta Street	9	11	0.6	<0.5
10 Tormey Street	14	7.5	1.4	<0.5
345 Laurier Avenue	10	2.5	0.5	<0.5
400 Laurier Avenue	11	3	0.5	<0.5
373 Laurier Avenue	11	3	0.6	<0.5
404 Laurier Avenue	5	4	0.8	<0.5

A review of building records has not been carried out (to confirm their founding conditions). Other potentially vulnerable buildings may be present along the alignment.

3.3.5 Impacts to Adjacent Structures

The potential for impacts to adjacent structures due to ground movements resulting from tunnel construction would need to be addressed on a case-by-case basis, based on the ground conditions, the tunnelling equipment, and on the type and proximity of nearby foundations. Potential sources of ground movement during tunneling include settlement due to under-drainage, ground movement due to general construction or uncontrolled loss of ground, and vibration.

3.3.5.1 Underdrainage and Clay Consolidation Settlement

A detailed assessment of the impacts of construction on adjacent buildings (particularly those supported on raft slabs supported on the clay deposits) will need to be carried out based on the predicted amount of settlement due to under-drainage from the bedrock tunnels and portal construction. Areas where under-drainage will be of particular importance include the Sandy Hill area (between about Laurier Avenue and Beausoleil Drive), and to a lesser extent along the portions of the alignment north to Cathcart Street and between Mann Avenue and Somerset Street, were the tunnel profile is also overlain by silty clay deposits. Buildings supported on raft foundations on these sensitive clay deposits have increased risk of experiencing consolidation settlements due to the higher existing stress levels in the clay. In these areas, a detailed hydrogeological assessment will be required and, depending on the results of this assessment, water-tight tunnelling methods and portals could be required to limit under-drainage during construction. To assist in the analysis, an inventory of foundation systems within the potential zone of influence of the construction dewatering in these areas will be required.

In theory, significant consolidation settlement will only occur when the vertical effective stress in the clay deposit exceeds the preconsolidation pressure. However, secondary compression (i.e., creep) settlements can occur even when the stress level approaches the preconsolidation pressure. There is also some inherent uncertainty to all of the parameters involved in stress calculations, including the preconsolidation pressure, the groundwater levels, the unit weights of the soils, and the imposed loads from foundations. It is therefore general practice to not permit stress increases that exceed 80 percent of the available ‘overconsolidation’ (i.e., of the stress difference between the preconsolidation pressure and the original natural effective stress level).

The predicted groundwater drawdowns for the various scenarios were reviewed, and the following general geotechnical comments can be made based on a conceptual/screening level assessment using only available information (i.e., desktop review) and estimated/assumed material properties:

- In the absence of additional stress due to imposed loads (such as from foundations or embankments), it is generally expected that the resulting final effective stress level in the clay for Scenarios 1, 2 and 3 will remain below the interpreted preconsolidation pressure of the clay, which is expected to be in the range of 200 to 300 kilopascals.
- Where heavy buildings are founded on the clay (e.g., on raft slabs), the additional load imposed by the buildings raises the existing effective stress levels in the clay. For example, without any changes in the existing groundwater levels, the stress level in the clay beneath some of the higher buildings (e.g., more than about 10 storeys high) is likely already essentially at or exceeding a level which is more than the original/natural stress level plus 80 percent of the available "overconsolidation".
- Piezometric changes in the clay via underdrainage from the bedrock (i.e., groundwater drawdowns at the top of rock) will further increase the effective stresses in the clay. Under Scenario 1 (fully draining tunnel, Figure 5), drawdowns of between 3 and 11 metres are anticipated beneath all of the high/heavy buildings listed in Table 2 (Section 3.3.4.2). Based on our assessment using estimated soil properties and building loads, these drawdowns would likely result in increases in stresses in the underlying clay beneath these buildings to stress levels which exceed the preconsolidation pressure.
- Under Scenario 2 (which simulates conventional excavation, with 150 metre long sections of tunnel left to freely drain for 53 days, Figure 6), the predicted drawdowns at these same buildings are generally in the 0.5 to 1 metre range. Preliminary analyses indicate that groundwater drawdowns of 0.5 to 1 metre beneath the taller buildings could raise the stress level in the underlying clays to stess levels which at or just in excess of 80 percent of the available "overconsolidation" beyond the original/natural stress level. For shorter buildings, or lesser drawdowns, there would be lesser associated risks.
- Under Scenario 3, which models rapid lining of the tunnel with pre-cast concrete segments which are installed immediately behind the TBM, groundwater drawdowns of less than 0.5 metres at the top of rock are anticipated along and adjacent to the tunnels, which would result in very little change to the existing stress conditions in the clay. While higher levels of drawdown are anticipated at the tunnel horizon (as shown on Figure 7), these piezometric head changes are temporary and transient, and, as indicated by the hydrogeologic modelling, are not likely to affect the piezometric pressures in the clay.

Based on the predicted drawdowns under Scenario 1, it is considered that the resulting changes in piezometric levels in the clay could result in some settlement of some heavier structures along or near to the alignment. As such, it is recommended that the finished tunnel and all access shafts (cross-passages) and portal excavations along the alignment be made permanently water tight to avoid long term drainage into the tunnels, cross-tunnels, and portal areas. The finished tunnel liner will need to be designed for the full hydrostatic head and should be sealed against water infiltration.

The results of hydrogeological modelling Scenario 2 indicate that the groundwater drawdown extends up to 400 metres from the proposed alignment (as defined by the 0.5 m contour). The predicted drawdowns beneath the buildings listed in Table 2 are generally low (in the range of 0.5 to 1 metre) but, given the height of the buildings, could still pose a risk of inducing some settlements (particularly for higher/heavier buildings which may have already experienced significant initial post-construction settlement). The results of Scenario 3 suggest that rapidly installing a segmental concrete liner immediately behind the TBM as tunnel excavation advances could minimize transitory groundwater drawdowns to negligible levels which would likely not propagate to the clay. Scenario 4 had not been modelled, but it is expected to show even less (and more localized) drawdown, since only the cross tunnels would be temporarily draining and for a relatively short duration.

Since the results of this conceptual/screening level analysis indicate that, for Scenario 1 and possible Scenario 2, actual overstressing of the clay (i.e., raising the stress level to *above* the preconsolidation pressure) would be limited to only those locations with significant imposed existing loading, which might be fairly limited in number, it might initially be concluded that localized mitigation measures could conceivably be sufficient to mitigate the risk of causing damage (e.g., by installing a recharge system adjacent to those critical structures). However, it is considered that the assessed drawdowns would represent a significant and larger-scale risk for the project (i.e., not only to just a few isolated structures), based on the following risk-factors:

- Previous experience with sustained dewatering of the clay deposits for construction in the downtown area has shown that, even in the absence of significant additional loading that would lead to stressing beyond the preconsolidation pressure, the recompression settlements (i.e., those occurring at stress levels below the preconsolidation pressure) can be sufficient to impact on utilities and surface features (e.g., in the order of 25 millimetres). It should also be noted that these recompression settlements have been measured at values that exceed what would be predicted based on the rather small recompression indices measured for the deposit (possibly because of both higher actual recompression index values as well as secondary compression effects). The settlements can also be entirely *differential* in nature when they occur adjacent to pile-supported structures, and can therefore have a greater impact that would otherwise be anticipated.

- The significant calculated extent of the drawdowns under Scenarios 1 and 2 presents a challenge in terms of assessing the potential risk to so many properties/structures. Due to the very large size of the drawdown zone, it would be impractical (or at least very challenging) to inventory and assess the potential impacts to every structure within the drawdown zone. For example, it is expected that drawings and/or foundation loading information would be available for only a fraction of the structures within the drawdown zone. In addition, an extensive geotechnical investigation would be required to evaluate the ground conditions (clay properties) within this full zone, so that the impacts could be evaluated; the available geotechnical data (which is currently based only on a "desktop" collection of available information) is not sufficient to provide a confident assessment of the ground conditions within this full area.

- Even if it were feasible to collect sufficient geotechnical data within the drawdown limits, and to assess the foundation loads for the numerous structures within this zone, it would likely not be feasible to inventory the specific vulnerabilities of so many structures, based on their current conditions. For example, based on other project experience by Golder, there are existing structures within the general study area that:

  1) Have probably been constructed on raft foundations and, for heavier buildings founded on clay, have quite probably already experienced more than 50 millimetres of post-construction settlement.

  2) Have already been damaged by previous settlements due to shrinkage of the underlying clay due to desiccation resulting from the water demand of tree roots.

  There are, therefore, already structures within the zone of anticipated drawdowns with little to no remaining tolerance to accept further differential settlements without experiencing at least cosmetic damage, and possibly serviceability or structural damage. Similarly, there may be structures for which part of the structure is supported on deep foundations and part on shallow foundations, and which therefore have very limited tolerance to experience differential settlement without there being an immediate impact on the appearance and serviceability of the structure.

- Given how close the calculated *existing* stresses are at some locations to the interpreted preconsolidation pressure (e.g., as is expected to be the case beneath the 10 to 11 storey tall buildings on Laurier Avenue) even slight variations from the parameters used in this assessment (including both the consolidation properties of the clay and the hydrogeologic properties used in the modelling, which are only based on a desktop review, and not site-specific information) could greatly increase the anticipated impacts. That is, there is very little apparent 'factor of safety'. And, given the sensitive nature of the clay deposit, with a high compression index (i.e., with a highly non-linear behaviour at stress levels at and above the preconsolidation pressure), even a minor overstressing of the clay can result in a disproportionately large increase in settlements.
- Should delays occur during construction (as a result of a variety of possible reasons), the drawdowns could potentially be even more significant than indicated by the analyses, both in terms of the geographic spread as well as the magnitude of the piezometric pressure reductions in the clay. In that regard, it should also be noted that the contractor's construction and dewatering schedule could differ from that used in this assessment, which might impact on the results.
- Should the groundwater levels not recover as quickly as predicted by the modelling (which is quite conceivable given the likely variations in recharge across the study area); the settlements experienced in some areas could also be larger than predicted.
- The large area of potential impacts represents a significant risk of damage claims, even if movements are not caused by the project (such as the common seasonal damage in this part of Ottawa due to foundation settlement resulting from the water demand of trees causing soil shrinkage).
- It should also be recognized that there is a potential risk of overlap with other construction projects which may be underway at the time of construction of the truck tunnels (or before), and those projects could be causing simultaneous or pre-construction overlapping groundwater level lowering.

Based on the above assessment, it is considered that:

- The final tunnel will need to be made water-tight.
- The duration and length of tunnel left draining during construction will need to be limited so as to minimize the potential for underdrainage and settlements of the overlying settlement sensitive clay soils.
- It may be feasible to construct the tunnel using conventional tunneling methods (e.g., sequential excavation method with cast-in-place watertight lining), provided that the length of tunnel left to drain freely is less than about 150 metres for about 50 days (based on the modelling results for Scenario 2). The resulting drawdowns do, however, increase the effective stress in the clays to a level which, beneath heavier buildings, is expected to be at, or just in excess of 80 percent of the clay's available overconsolidation. As such, it is concluded that further study (including extensive site investigations) and evaluation of specific buildings would need to be undertaken before this method of excavation could be clearly demonstrated to have a sufficiently low risk of inducing unwanted impacts.
- Scenario 3 suggests that rapidly lining the tunnels immediately behing a TBM (e.g., a double shield TBM and pre-cast concrete segments) would likely be sufficient to limit drawdowns such that there would be relatively little impact on the clay or risk of settlement of overlying buildings.

3.3.5.2 Ground Movement due to General Construction or Uncontrolled Loss of Ground

Where tunnelling in bedrock with at least two diameters of bedrock cover above the tunnel crown, it is expected that ground movement due to stress changes associated with the actual tunnel excavation should not be an issue. The current profile for the preferred alignment typically has less than one diameter of crown cover and, as such, the potential for ground movement will need to be assessed.

Wherever possible, tunnelling within the zone of influence of structural foundations should be avoided. The preferred alignment is a “cross-country” alignment which passes beneath several buildings. While most of these buildings are small and relatively lightly loaded, the tunnels will pass directly beneath, or in close proximity to, a number of larger/heavier buildings, including a 10 storey apartment building on Wilbrod Street, a 10 storey apartment building on Templeton Street, a 6 storey apartment complex on Beausoleil Drive and an 11 storey apartment building south of Murray Street/Myrand Avenue. Where the tunnels pass beneath and in close proximity to the foundations, the influence of the tunnel excavation on the existing foundations, and the added loading from the foundations on the tunnel, will need to be considered in the subsequent phases of design, and this condition may require additional temporary and permanent rock support. The absolute minimum clearance between the tunnel and the adjacent foundations (e.g., from the underside of deep foundations or footings) should be no less than 3 to 4 metres to allow for installation of rock bolts.

To reduce impacts on existing buried infrastructure, a minimum clear separation of at least 2 tunnel diameters is generally recommended from existing tunnels. At locations where the proposed tunnels cross perpendicular/orthogonal to existing tunnels (such as where the tunnels pass beneath the CSST and IOS tunnels), less vertical clearance can be used, provided that sufficient additional temporary and permanent rock support is installed within the new tunnel. Both the IOS and CSST sewers are (or will be) tunneled in rock and have an invert elevations of about 41 and 42 metres, respectively. The preferred alignments pass under and are roughly orthogonal to these two existing sewers. The limited thickness of rock crown cover (about 2 to 3 metres between the anticipated tunnel crown and the sewer inverts) poses some inherent risk to damaging the overlying sewers. This is particularly true beneath the IOS (which was constructed as a drill and blast tunnel) where the surrounding rock mass may have been damaged/fractured as part of the original tunnel construction. At the detailed design stage, the risk of loss of support from beneath the IOS as well as adequacy of the IOS liner to withstand the increase in loads resulting from the redistribution of stresses within the rock mass due to excavation of the tunnels will need to be considered. Additional rock support or underpinning of the existing sewers will likely be required at these crossings. Depending on the construction methodology selected, it may be challenging to install additional rock support from within the tunnel, and reinforcing of the rock beneath the sewers may be necessary in advance of tunneling. A minimum vertical clearance between the two tunnels of no less than 1.5 metres is recommended at this stage to allow for installation of rock bolts and/or to avoid intersecting rock bolts which may have been installed within the existing tunnel.

The risk of uncontrolled ground movements during tunnelling is greatly increased where mixed ground conditions are present at the tunnel face or in areas of very low rock crown cover such as north of King Edward Avenue at the north portal approach, where the tunnel crown rises towards the bedrock surface. Depending on the location of the bedrock surface and the quality (or absence) of rock immediately above the tunnel, significant loss of ground (i.e., sinkholes) could form above the tunnel unless measures are implemented to support/stabilize the rock. Careful control of the excavation face and crown will be needed to limit the potential for ground loss and potential settlements at ground surface. Open-cut (cut and cover) excavations may need to be lengthened to include this relatively short section.

3.3.5.3 Vibrations and Monitoring

While not expected to be a significant issue along much of the alignment given the depth of the tunnel, the vibrations induced during construction will need to be considered. Vibrations from portal construction (with possible pile installation and/or blasting) will likely be a greater issue than vibrations from the tunnelling operations, given the tunnel's depth (and assuming that it is excavatied with a TBM or roadheader); however non-damaging "nuisance" vibrations may also be experienced from the tunnel excavations. Additional assessment of this issue will need to be made at subsequent stages of the design, since the preferred alignment passes directly under several residences, particularly if drilling and blasting will be permitted. It is understood that others on the study team are reviewing the potential impacts due to vibration issues for this study.

A very large baseline pre-construction vibration, elevation, and groundwater survey with readings taken over the course of at least 1 year prior to construction and extending several blocks back (not just at adjacent buildings), and monitoring of groundwater levels and building elevations during construction is recommended.

3.3.6 Potential for Reuse of Tunnel Spoil

It is anticipated that the tunnel spoil/muck generated will be primarily from deposits of limestone or interbedded limestone and shale (i.e., from the Lindsay Formation and Verulam Formations, respectively), and to a lesser extent from shale bedrock of the Carlsbad and Billings Formations (which may be encountered at the tunnel horizon and portal excavations south of Somerset Street). It is anticipated that the tunnel muck can potentially be reused as Select Subgrade Material (SSM) and non-select earth fill. Use of limestone as Granular A and Granular B (Type II) could require crushing and/or screening, and it would need to be tested for durability for reuse in some applications (e.g., as pavement base/subbase). Depending on the percentage of shale, the interbedded limestone and shale may not meet the Granular A or B durability specifications. As such, the suitability of the interbedded limestone and shale deposits for reuse as Granular A or B would be location specific. Higher shale content muck may be prone to swelling/heave, which could limit its suitability for reuse as engineered/compacted fill beneath structures, but it could potentially still be used as general grading fill.

It would likely not be considered economically feasible to use any of the excavated tunnel muck as concrete aggregate due to the shale content. The shales in the Ottawa area contain sulphides which oxidize when exposed to air and water and generate sulphate or gypsum which can cause swelling of the aggregate. The flatter and elongated particles cut by a TBM could further limit their use as Portland cement concrete or Hot Mix Asphalt concrete aggregate.

3.3.7 Considerations Regarding the Management and Off-Site Disposal of Tunnel Spoil

The applicable regulations concerning the offsite disposal of tunnel spoil ("muck") are dependent on whether the muck is considered by the Ministry of Environment and Climate Change, Ontario (MOECC) to be a "soil" or "rock". As per the definition outlined in O.Reg. 153/04, "soil" means *"except for the purposes of shallow soil property as defined in section 43.1, unconsolidated naturally occurring mineral particles and other naturally occurring material resulting from the natural breakdown of rock or organic matter by physical, chemical or biological processes that are smaller than 2 millimetres in size or that pass the US #10 sieve".* Materials meeting this definition would be subject to the soil classification standards used for excess soil management (e.g., MOE Tables 1 and 3). There is no equivalent standard for disposal of excavated "rock" muck but, under the Environmental Protection Act (EPA), "rock" muck would need to be evaluated for "adverse effects" before being disposed of. Some of the items to consider when determining adverse effects for the excavated rock include, but are not limited to, the rock's propensity to leach chemicals, generate solids in runoff (applicable to receiving sites where surface water may become effected) and the rock's ability to generate extreme pH (acidic or basic) when mixed with water.

The gradation of the tunnel muck, and thus the applicable standards for disposal and reuse, would be dependent on the excavation method (e.g. conventional or mechanized excavation). Consultation with the MOECC may be required to determine if it would be treated as a rock or soil. If the gradation of the muck was such that it would be considered a "soil", then it should be noted that naturally occurring compounds may be present within the excavated tunnel spoils which would need to be considered with respect to potential off-Site disposal and re-use options. For example, the limestone bedrock which is present in the north and central sections of the tunnel alignment has the potential to contain naturally occurring metal concentrations that may exceed MOE Table 3 Standards. The shale bedrock which is present in the south section of the site has the potential to contain metals and/or organic compounds in concentrations exceeding MOE Table 3 Standards and, when exposed to air and water, the shale is potentially acid generating.

3.4 Geotechnical Considerations Related to Portal Construction

Excavations for portals and cut and cover tunnels will likely/generally require excavation shoring. Impacts on adjacent structures resulting from ground movements due to shoring or cut and cover tunnel construction will need to be considered. The selected excavation and support methods will need to consider the presence of boulders in the glacial till, as well as the potential for flowing ground conditions below the groundwater table in sandier zones (as frequently encountered at the transition between the Champlain Sea Clay and underlying glacial till) or high groundwater inflows from fills and sandier soils, particularly close to the Rideau River.

3.5 Additional Considerations

Additional considerations which will need to be addressed in subsequent phases of design are provided below.

- The tunnel alignments will be advanced though layered sedimentary bedrock formations. Bedding of these formations is near-horizontal but can be gently sloped. Given the relatively flat tunnel gradients, very strict grade control will be needed to maintain the vertical alignment of the tunnels and avoid drift along the bedding planes.

- The local shale (Billings and Carlsbad Formations) is prone to time dependant deformation (swelling expansion). There are two separate swelling mechanisms to be considered; the first is due to the creation of osmotic pressures in the shale when "fresh" groundwater dilutes the saline pore water, and the second occurs during periods of prolonged exposure to oxygen and warm air which results in oxidation of the pyrite in the shale. If any structure or lining is constructed directly in contact with the rock shortly after construction, the time-dependent deformation (swelling) of the rock would cause pressure to build up with time at the rock-structure interface and this pressure should be considered in design. The magnitude of the pressure depends on the rigidity of the structure, the time schedule of construction, the swelling characteristics of the rock and the initial stresses in the rock formation. The first swelling mechanism usually occurs at depth, due to stress relief, and is usually dealt with by delaying the placement of concrete against the shale by about 3 to 4 months to allow for a large part of the swelling to occur, or by introducing a compressible material between any concrete structures and the rock. For the second mechanism, local practice for temporary excavations is to keep the shale wet or to seal it from exposure to air with shotcrete as a temporary measure. To prevent swelling over the long term, the groundwater levels should be maintained at pre-construction levels (or at lease well above the tunnel crown level) in order to keep the shale submerged. Additional testing will be required where shale would be encountered along the tunnel alignment or at portal/surface structure locations so that the swelling characteristics of the shale can be taken into consideration.

- The shale is also potentially rich in sulphides and sulphate. As such, final concrete liners and concrete structures installed within shale-rich deposits must be made with sulphate resistant cement.

- Combustible gases (e.g. methane) are known to be potentially present in the shale bedrock of the Billings and Carlsbad Formations, and should be further assessed at the investigation stage. Construction monitoring and suitable ventilation may be required to mitigate the potential hazards during excavation and construction work.
- Depending on the expected tunnel profiles, the proposed alignments may encounter soil or groundwater contamination. The tunnel alignments are located in areas of past industrial activity, so it is possible that any groundwater level lowering could mobilize contaminated groundwater from sites above and adjacent to the tunnel alignments and draw it towards/into the tunnel. Areas of potential concern identified at this early stage are outlined in the environmental overview memo and include former landfills, retail fuel outlets, and auto garages with underground storage tanks and/or dry cleaning facilities. Making the finished tunnel and all permanent structures water tight will also limit the potential for migration of contaminants in the long term.
- Naturally occurring heavy metals and organic compounds in the limestone and shale bedrock may also impact groundwater quality pumped from the site. Before water can be discharged to the City sewer, pre-construction testing (and an ESA) will be required to check if the discharge quality would meet the City of Ottawa Sewer Use by-laws. Monitoring during construction will also be required, and excess water may need to be treated before it is disposed of.

4.0 SUMMARY AND RECOMMENDATIONS FOR FURTHER STUDY

The preferred twin truck tunnel alignments are considered generally feasibly for construction, however the adequacy of the rock cover at the proposed invert elevations should be reviewed by GEODATA based on the anticipated ground conditions and tunneling methodology. Along most of the alignment, the anticipated rock quality is good to excellent, but may be lower in the shale deposits identified south of about Somerset Street. There are expected to be localized areas with faults, low rock cover and/or poor rock quality along the alignment along with some risks with respect to clay consolidation settlement, groundwater and muck management, but it is generally anticipated that these risks can be managed, provided that sufficient and suitable subsurface information is collected at the design stage and mitigation measures implemented. For tunnelling projects, which are inherently of higher risk than above-grade construction, collection of comprehensive and detailed subsurface information is critical. Without this information, the risk of high cost construction claims and schedule delays is significantly higher.

The following key geotechnical and hydrogeologic considerations will need to be addressed at the EA, functional, preliminary and detailed design stages for the truck tunnels:

1) **General bedrock conditions**: The investigation will need to identify/evaluate:

   a) the general bedrock profile along the alignment, to confirm the adequacy of the bedrock cover over the tunnel crown, since low rock crown cover could compromise the crown stability.

   b) the bedrock quality (i.e., bedding and jointing) which could impact on the roof support requirements, overall tunnelling methodology, and productivity.

   c) the bedrock strength, hardness, abrasivity, and geochemistry which could impact on machine selection and performance, ventilation and muck management.

2) **Fault zones:** Several faults are known to cross the tunnel alignments. Those faults could represent zones of poor rock quality and/or high groundwater inflow which could impact on the local stability of the tunnel and machine performance. Special roof support systems, grouting and/or dewatering may be required in advance of the tunnelling to minimize or address some of the risks. These issues all impact on the cost and productivity of tunnelling through these zones. To limit the risk of claims and delays, it will be important to properly characterize the bedrock quality, fault conditions (including hydraulic conductivity) and, to the extent possible and apparent width of these faulted zones. This is of particular importance south of Somerset Street, where a major fault is mapped, and north of Stewart Street, where the proposed tunnel alignment runs roughly parallel to the general orientation of faulting in the area.

3) **Zones of low bedrock cover:** There are sections of the tunnel alignment where there is low rock cover. Within these sections, the adequacy of the bedrock cover needs to be confirmed/defined in greater detail. More closely spaced boreholes would be needed in these areas, such as at the Rideau River crossing, and just south of the north portal area.

4) **Zones of poor rock quality:** Zones of poor rock quality include fault zones and zones of low bedrock cover (as discussed above). However, shale bedrock, particularly when combined with faulted areas and areas with low bedrock cover (such as in the area of the Rideau River) can also represent zones of poor rock quality which may require additional rock support, and should be investigated.

5) **Zones overlain by clay:** Portions of the preferred tunnel alignment are overlain by clay. Proper characterization of the clay (including key parameters), and assessment of the impacts of groundwater drawdowns, will need to be completed in order to evaluate the risks associated with clay consolidation settlement (and building damage) resulting from tunnel drainage and to confirm any constraints on tunnelling methodology which could influence the overall costs and construction requirements for the project. Key clay parameters which should be obtained for a thorough geotechnical assessment include thickness, undrained shear strength, unit weight and preconsolidation pressure with depth. Pumping tests should also be carried out to better assess the hydraulic properties of the overburden and bedrock along the tunnel alignment.

6) **Portals and Cut and Cover Tunnels:** Additional investigation will be required in the areas where portals and cut and cover tunnels are planned. The overburden, bedrock, and groundwater conditions need to be defined so that the designers and contractor can evaluate the excavation conditions, shoring requirements, water inflow and cut-off requirements, and the potential for impacts on surrounding structures (e.g., settlements and vibrations).

It is generally not feasible to fully investigate a tunnelling project, and address all of the above issues, in a single phase of investigation; a staged approach is generally required so that the scope of the subsequent more focused investigations can be developed.

5.0 CLOSURE

This memo provides a brief overview of the subsurface conditions along the preferred alignment (as determined by the study team), as well as a preliminary assessment of issues/constraints, as input to the ongoing feasibility study for the Downtown Ottawa Truck Tunnel project. The input provided has been based solely on a review and interpretation of limited available data as described in this technical memorandum and as reported herein. No assurance is made regarding the accuracy and completeness of these data. No site visit was undertaken in preparing this technical memorandum and no site-specific subsurface investigations were conducted as part of this assessment.

This technical memorandum was prepared for the exclusive use of Parsons Inc., GEODATA, and the City of Ottawa and is intended to provide these parties with an assessment of the current subsurface conditions for this feasibility study. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. Should additional parties require reliance on this report, written authorization from Golder will be required.

We trust that this memorandum provides sufficient information for your present requirements. If you have any questions concerning this memorandum, please don't hesitate to contact us.

Yours truly,

GOLDER ASSOCIATES LTD.

Erin O'Neill, P.Eng.
Associate, Senior Geotechnical Engineer

Mike Cunningham, P.Eng.
Principal, Senior Geotechnical Engineer

SG/NB/WC/ESO/MJT/MIC/ob
n:\active\2014\1121 - geotechnical\1404896 parsons downtown ottawa truck tunnel\geotechnical\1404896 tm-002 geotechnical assessment preferred route.docx

Attachments: Importan Information and Limitations of this Report
Figure 1 – Surficial Geology
Figure 2 – Trends in Depth to Bedrock (Drift Thickness)
Figure 3a – Bedrock Geology (OGS)
Figure 3b – Bedrock Geology (GSC)
Figure 4 – Inferred Soil and Bedrock Profile along Preferred Tunnel Alignment
Figure 5 – Simulated Groundwater Drawdown Contours (Scenario 1)
Figure 6 – Simulated Groundwater Drawdown Contours (Scenario 2)
Figures 7 – Simulated Groundwater Drawdown Contours (Scenario 3)

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 practicing 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.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, **Parsons Inc.** The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

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.

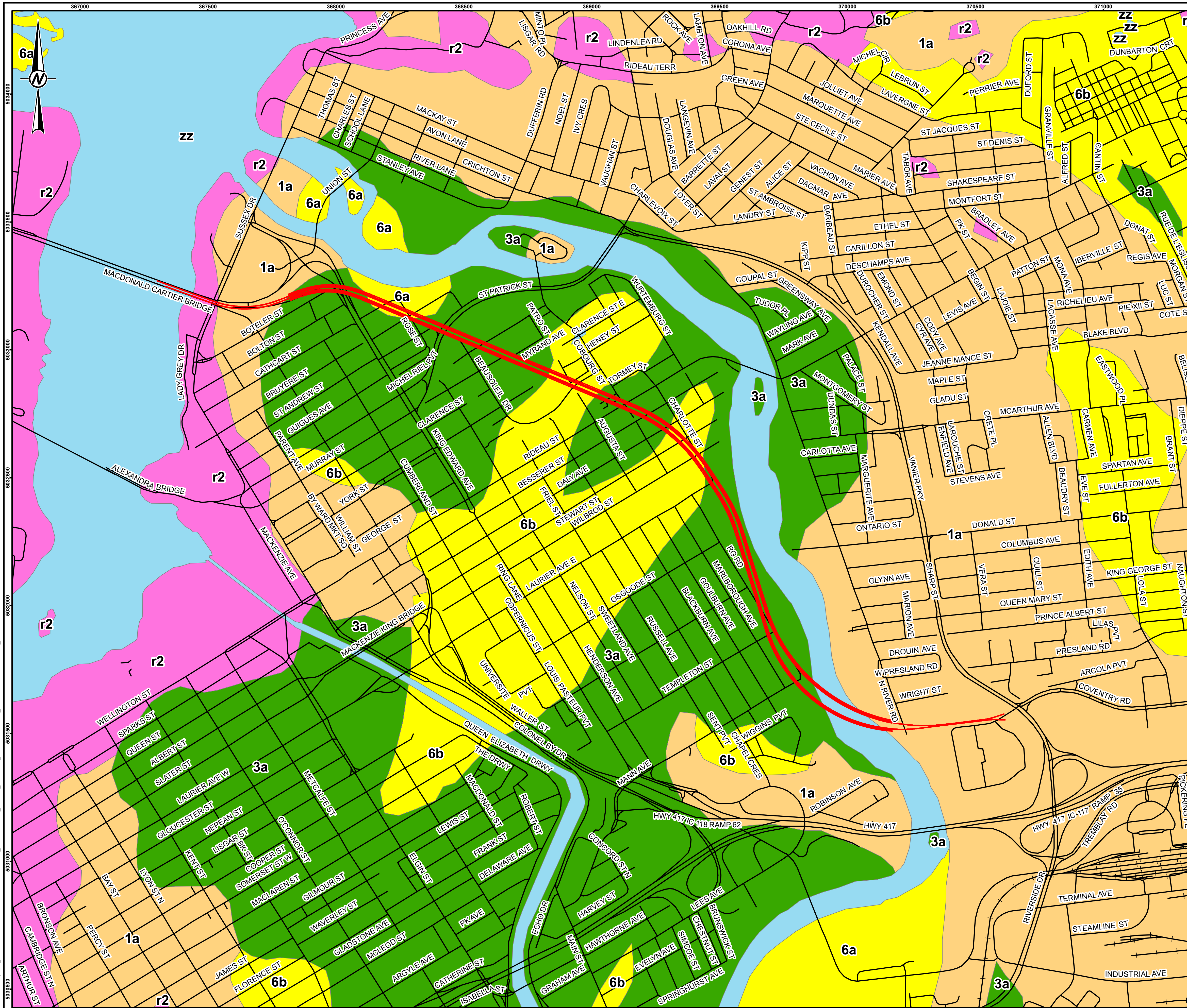
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.



LEGEND

- PREFERRED ALIGNMENT
- ROADWAY
- RAILWAY
- 6a: ALLUVIAL DEPOSITS: SILTY SAND, SILT, SAND & CLAY
- 6b: ALLUVIAL DEPOSITS: MEDIUM GRAINED STRATIFIED SAND WITH SOME SILT
- 3a. OFFSHORE MARINE DEPOSITS: CLAY, SILT UNDERLYING EROSIONAL TERRACES
- 1a. TILL, PLAIN WITH LOCAL RELIEF <5 m
- r2. BEDROCK: LIMESTONE, DOLOMITE, SANDSTONE & LOCAL SHALE
- zz. WATERBODY

0 150 300 600

1:15,000 METRES

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

REFERENCE(S)

1. BÉLANGER, J. R. 2008 URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE 5311, 1 DVD.
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
SURFICIAL GEOLOGY

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-10-22
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	1

25mm

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:

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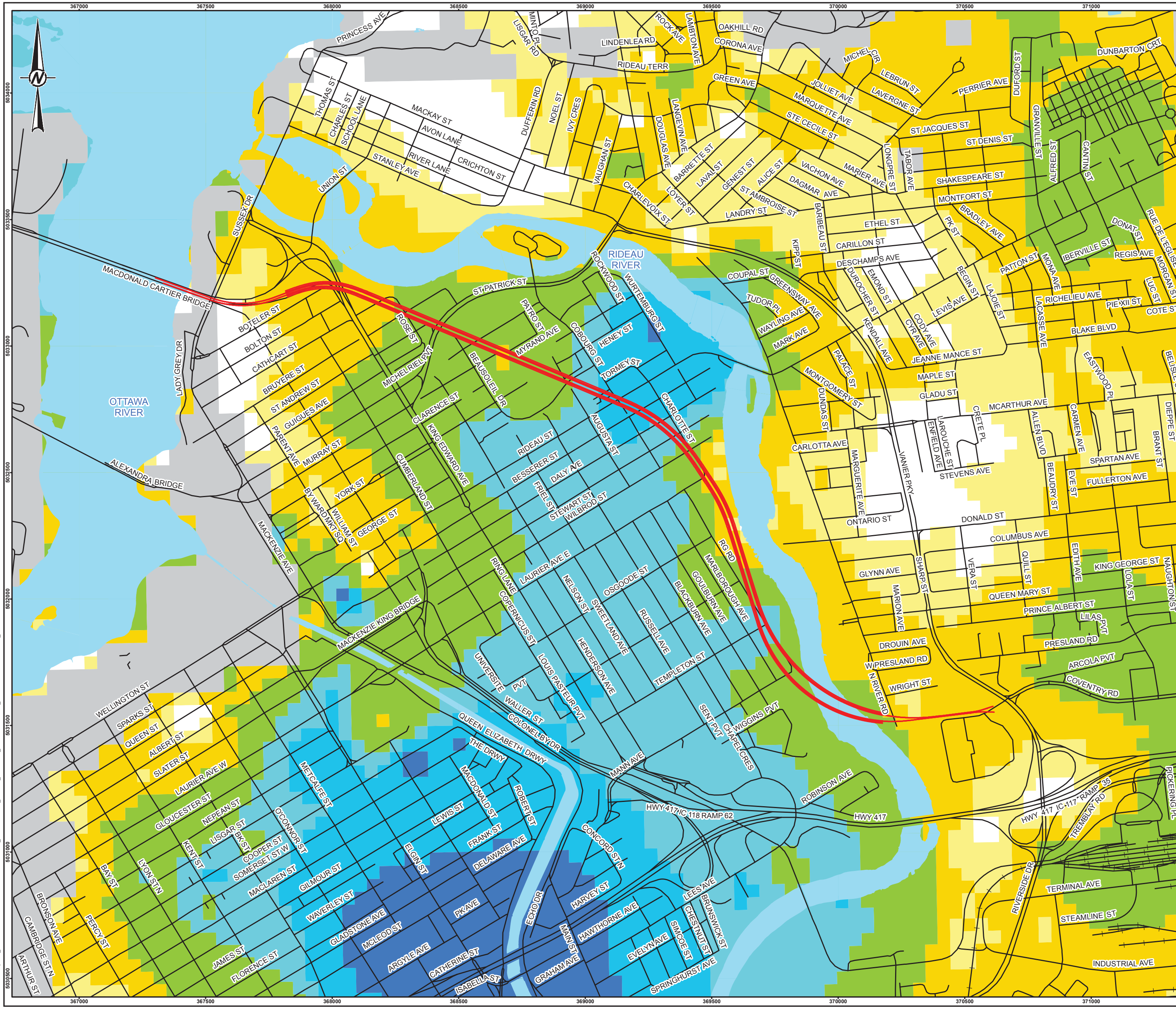

LEGEND

- PREFERRED ALIGNMENT
- ROADWAY
- RAILWAY
- WATERBODY

TREND IN DEPTH TO BEDROCK (METRES)

- 0 to 1
- 1 to 2
- 2 to 3
- 3 to 5
- 5 to 10
- 10 to 15
- 15 to 25
- 25 to 50
- 50 to 100
- 100 to 200

0 150 300 600
1:15,000 METRES

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

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 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
DRIFT THICKNESS

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-22	
DESIGNED	----	
PREPARED	JEM	
REVIEWED	ESO	
APPROVED	MIC	

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:
25mm

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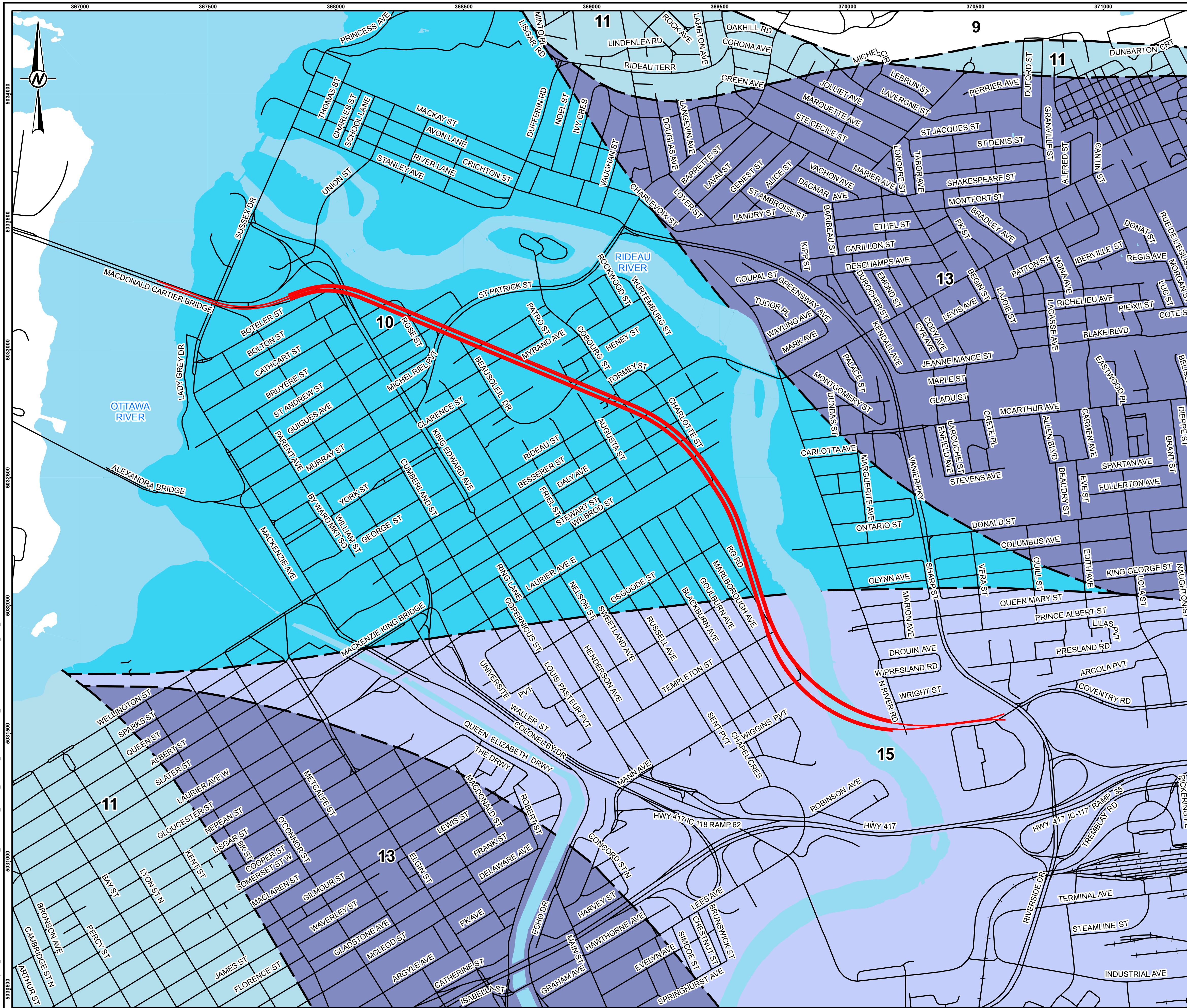

LEGEND

- PREFERRED ALIGNMENT
- ROADWAY
- RAILWAY
- WATERBODY
- 15: CARLSBAD FORMATION - SHALE AND LIMESTONE
- 13: BILLINGS FORMATION - SHALE, MINOR LIMESTONE
- 11: LINDSAY FORMATION - LIMESTONE; NODULAR TO BLACK LAMINATED
- 10: VERULAM FORMATION - LIMESTONE AND SHALE

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

REFERENCE(S)

1. ARMSTRONG, D.K. AND DODGE, J.E.P. PALEOZOIC GEOLOGY MAP 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, © QUEENS PRINTER 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
BEDROCK GEOLOGY

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-22
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
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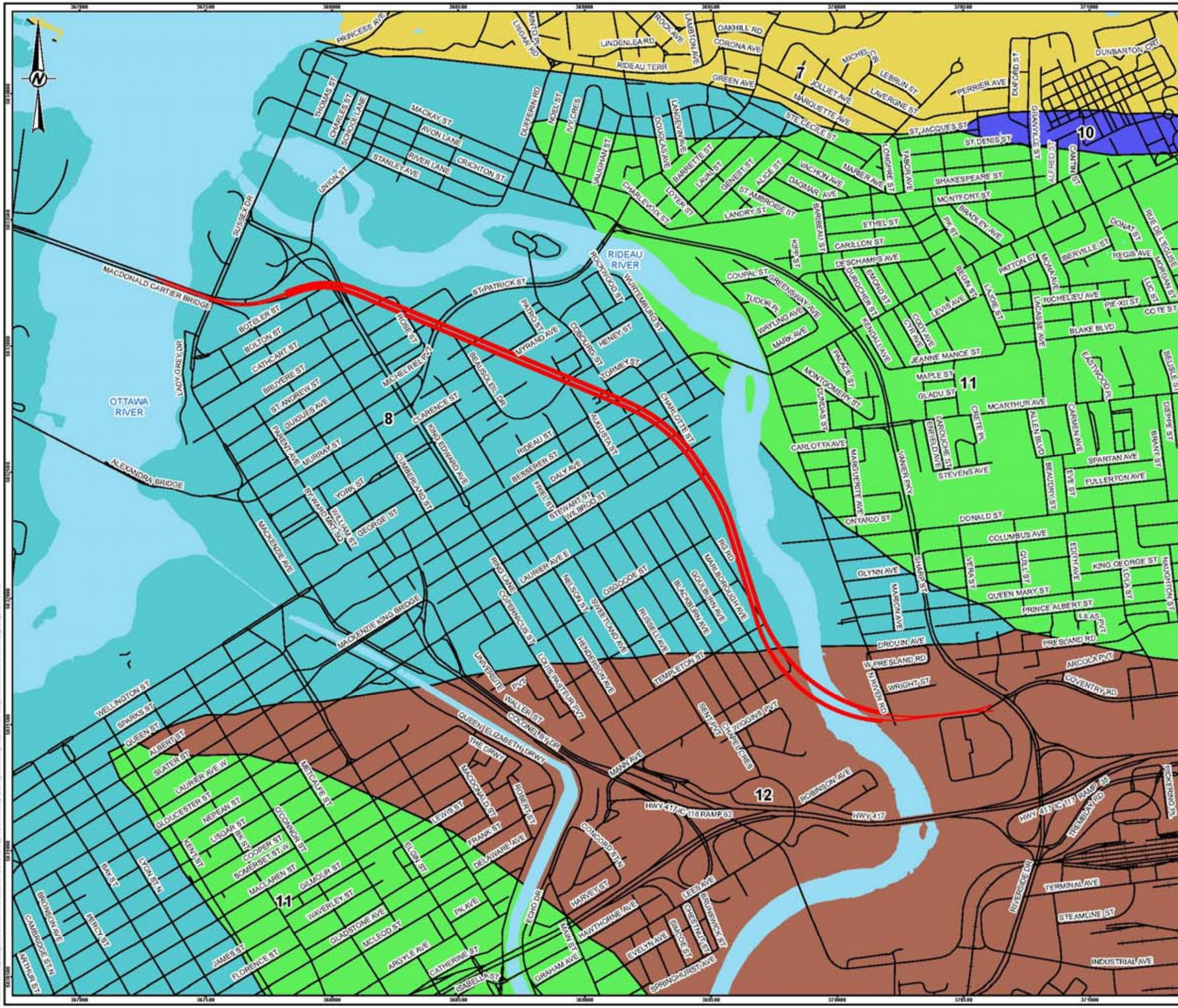


LEGEND

- PREFERRED ALIGNMENT
- ROADWAY
- RAILWAY
- WATERBODY
- 12. CARLSBAD FORMATION: INTERBEDDED DARK GRAY SHALE, FOSSILIFEROUS CALCAREOUS SILTSTONE, AND SILTY BIOCLASTIC LIMESTONE
- 11. BILLINGS FORMATION: DARK BROWN TO BLACK SHALE, WITH LAMINATIONS OF CALCAREOUS SILTSTONE
- 10. EASTVIEW FORMATION: INTERBEDDED SUBLITHOGRAPHIC TO FINE CRYSTALLINE LIMESTONE AND DARK BROWN TO DARK GREY SHALE
- 8. VERULAM FORMATION: INTERBEDDED BIOCLASTIC LIMESTONE, SUBLITHOGRAPHIC TO FINE CRYSTALLINE LIMESTONE
- 7. BOBCAYGEON FORMATION: INTERBEDDED SILTY DOLOMITE, LITHOGRAPHIC TO FINE CRYSTALLINE LIMESTONE, OOLITIC LIMESTONE, SHALE, AND FINE-GRAINED CALCAREOUS QUARTZ SANDSTONE

0 150 300 600
1:15,000 METRES

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1404896-1000.

REFERENCE(S)

1. BELANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001.
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2014
3. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9 VERTICAL DATUM: CGVD28

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
BEDROCK GEOLOGY

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-22
DESIGNED	----
PREPARED	JEM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	3B

LEGEND

APPROXIMATE FAULT LOCATION

SUBSURFACE STRATIGRAPHY

FILL
SAND
CHAMPLAIN SEA CLAY
GLACIAL TILL
LIMESTONE BEDROCK
SHALE BEDROCK

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. MEMO No. 1404896/1000/1500.
2. STRATIGRAPHIC BOUNDARIES SHOWN ON PROFILES WERE DEVELOPED BASED ON AVAILABLE SUBSURFACE INFORMATION AND ARE APPROXIMATE.
3. MAJOR FAULT LOCATION SHOWN ON PROFILE IS INFERRED FROM PUBLISHED MAPPING.

REFERENCE(S)

1. PROFILE SUPPLIED IN ELECTRONIC FORMAT BY PARSONS ON SEPTEMBER 25, 2015

CLIENT
PARSONS

PROJECT
DOWNTOWN OTTAWA TRUCK TUNNEL

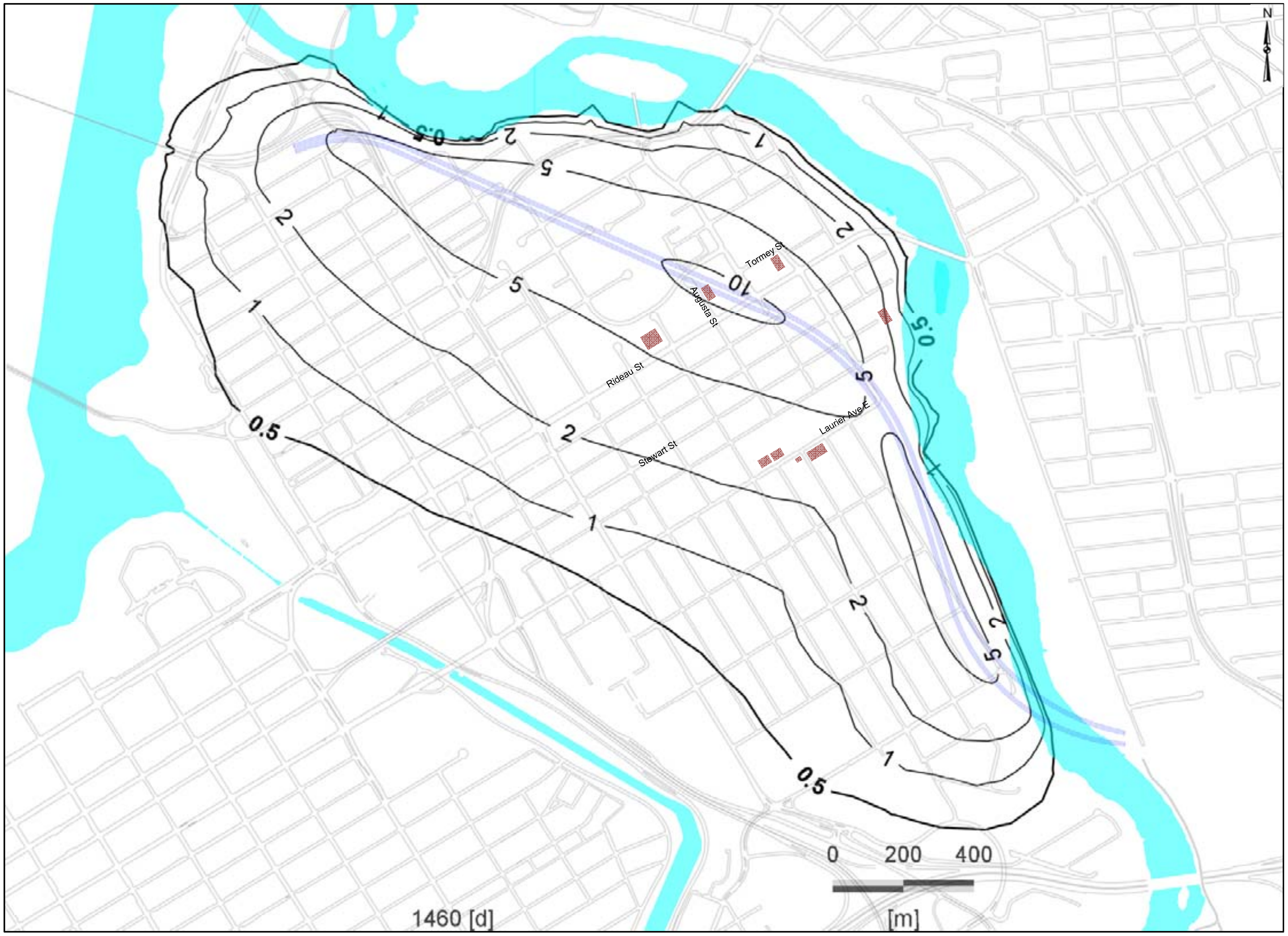
TITLE
STRATIGRAPHIC PROFILE

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-23
DESIGNED	----
PREPARED	JM
REVIEWED	ESO
APPROVED	MIC

PROJECT NO.	PHASE	REV.	FIGURE
1404896	1000	0	4

25 mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B



LEGEND

- Simulated Groundwater Drawdown at Top of Bedrock(m)
- Approximate Footprint of Buildings Considered in Consolidation Analysis
- Roads
- Proposed Tunnel Alignment
- Waterbody

CLIENT
Parsons

CONSULTANT

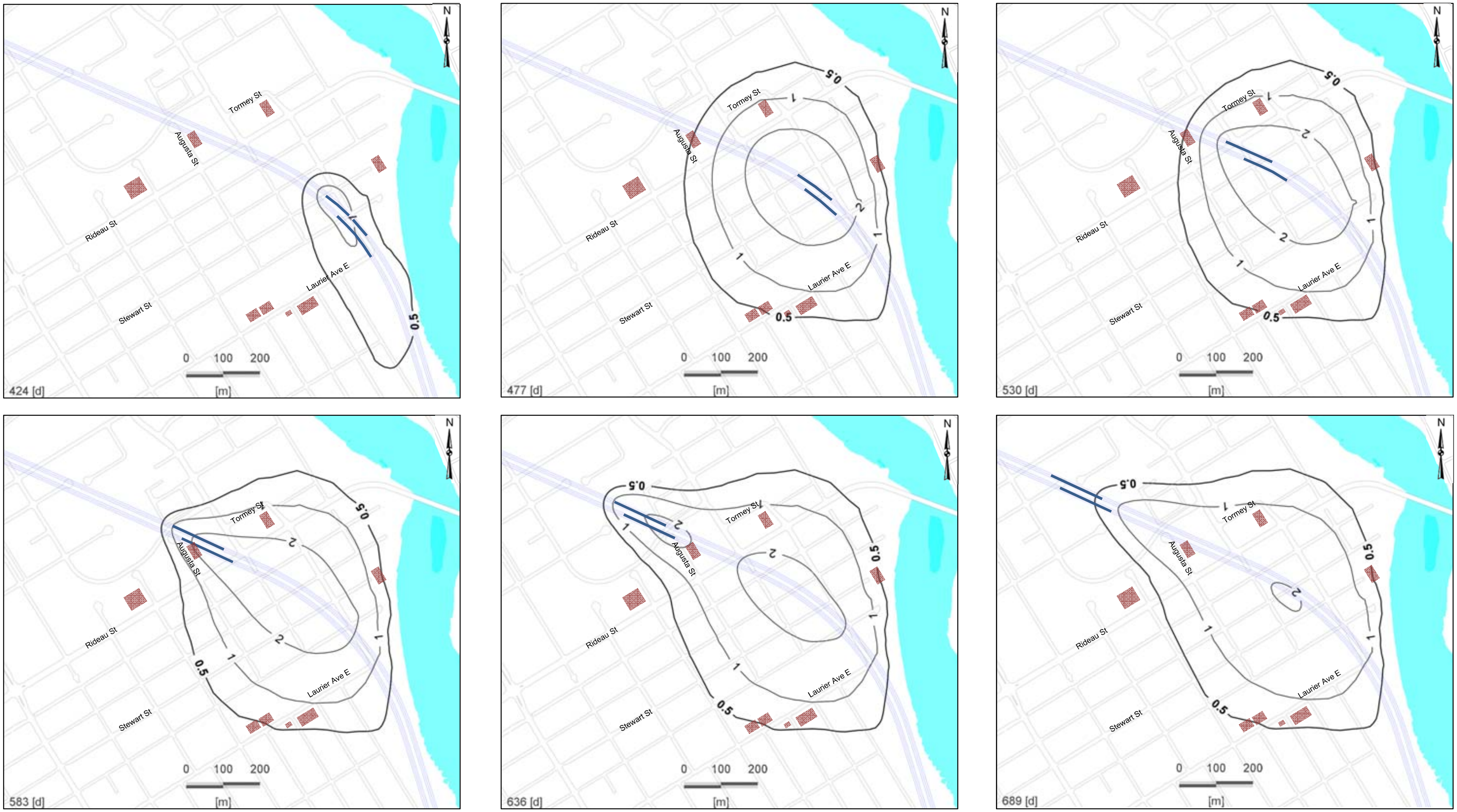
Golder Associates

YYYY-MM-DD	2015-10-22
PREPARED	NB
DESIGN	NB
REVIEW	ESO
APPROVED	MIC

PROJECT
GEOTECHNICAL AND HYDROGEOLOGICAL OVERVIEW ASSESSMENT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
SIMULATED GROUNDWATER DRAWDOWN CONTOURS (m)
SCENARIO 1 (FULL TUNNEL LENGTH OPEN FOR 4 YEARS)

PROJECT No.	Rev.	FIGURE
1404896	0	5



LEGEND

- Simulated Groundwater Drawdown at Top of Bedrock(m)
- Approximate Location of Open Tunnel Sections
- Approximate Footprint of Buildings Considered in Consolidation Analysis
- Roads
- Proposed Tunnel Alignment
- Waterbody

CLIENT
Parsons

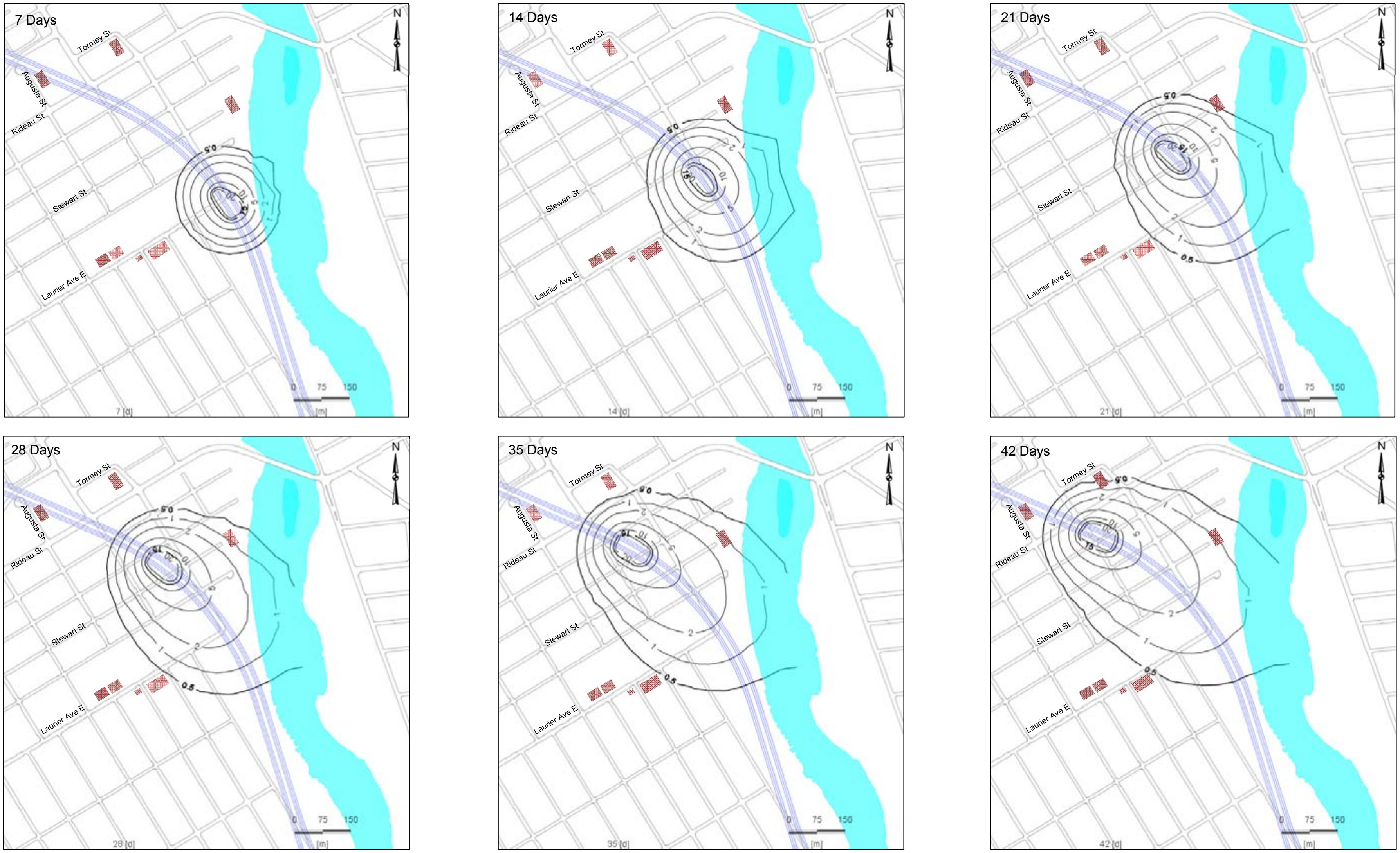
CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-22
PREPARED	NB
DESIGN	NB
REVIEW	ESO
APPROVED	MIC

PROJECT
GEOTECHNICAL AND HYDROGEOLOGICAL OVERVIEW ASSESSMENT
DOWNTOWN OTTAWA TRUCK TUNNEL

TITLE
SIMULATED GROUNDWATER DRAWDOWN CONTOURS (m)
SCENARIO 2 (150m OPEN TUNNEL LENGTH)

PROJECT No.	Rev.	FIGURE
1404896	0	6



LEGEND

- Simulated Groundwater Drawdown at Tunnel Horizon(m)
- Approximate Footprint of Buildings Considered in Consolidation Analysis
- Roads
- Proposed Tunnel Alignment
- Waterbody

NOTE: The images above show the simulated drawdown at the tunnel horizon. Drawdown at the top of rock was less than 0.5 m for the duration of the simulation.

CLIENT	Parsons
CONSULTANT	Golder Associates
YYYY-MM-DD	2015-10-22
PREPARED	NB
DESIGN	NB
REVIEW	ESO
APPROVED	MIC
PROJECT	GEOTECHNICAL AND HYDROGEOLOGICAL OVERVIEW ASSESSMENT DOWNTOWN OTTAWA TRUCK TUNNEL
TITLE	SIMULATED GROUNDWATER DRAWDOWN CONTOURS AT THE TUNNEL HORIZON (m) - SCENARIO 3 (TBM WITH PRE-CAST CONCRETE LINER SEGMENTS)
PROJECT No.	1404896
Rev.	0
FIGURE	7