



May 2014

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**ATMS Pole, Site 33-828-W-S**

**Reconstruction and Widening of Highway 401**

**From 0.5 KM West of Regional Road 8/King Street Easterly  
to 0.5 KM East of Regional Road 24/Hespeler Road - 5.5 KM**

**GWP 4-00-00**

**Ministry of Transportation, Ontario - West Region**

**Submitted to:**

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REPORT



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## FOUNDATION INVESTIGATION AND DESIGN REPORT ATMS POLE - SITE 33-828-W-S

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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEET

FIGURE 1 - Key Plan

DRAWING 1 - Borehole Locations

### APPENDICES

#### APPENDIX A

Rock Core Photographs



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**FOUNDATION INVESTIGATION AND DESIGN REPORT  
ATMS POLE - SITE 33-828-W-S**

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**PART A**  
**FOUNDATION INVESTIGATION REPORT**

**ATMS POLE - SITE 33-828-W-S  
RECONSTRUCTION AND WIDENING OF HIGHWAY 401  
FROM 0.5 KM WEST OF REGIONAL ROAD 8/KING STREET EASTERLY  
TO 0.5 KM EAST OF REGIONAL ROAD 24/HESPELER ROAD – 5.5 KM  
GWP 4-00-00  
MINISTRY OF TRANSPORTATION - WEST REGION**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Parsons Corporation (formerly Delcan Corporation) (Parsons) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 4-00-00. The project involves the detail design for the reconstruction and widening of Highway 401 from 0.5 kilometres west of King Street (Waterloo Regional Road 8) easterly to 0.5 kilometres east of Hespeler Road (Waterloo Regional Road 24).

This report addresses the foundation engineering requirements for a changeable or variable message sign (VMS) which will be installed as part of the Advanced Traffic Management System (ATMS). The pole will be located in the westbound shoulder at approximately Station 17+450 left (Lt).

The purpose of the foundation investigation is to explore the subsurface conditions at the proposed sign location by drilling a single borehole in close proximity to the sign location and carrying out in situ testing and laboratory testing on selected samples of the subsurface materials. The terms of reference for the scope of work are outlined in the MTO's Request for Proposals and in Golder Associates' proposal P0-1132-0056 dated July 23, 2010 and the updated scope of work as described in our letter 10-1132-0056-2000-L02, dated May 28, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated September 2010.

Parsons provided Golder Associates with drawings for this project in digital format.



## **2.0 SITE DESCRIPTION**

### **2.1 General**

The reconstruction and widening of Highway 401 to be undertaken as GWP 4-00-00 extends from west of King Street East (Regional Road 8) to east of Hespeler Road (Regional Road 24) in the City of Cambridge, Region of Waterloo. The ATMS pole is located at approximately Station 17+450 as shown in the Key Plan, Figure 1.

This section of Highway 401 is currently a six lane divided highway oriented generally east-west. Two underpass structures for Fountain Street North and Speedsville Road, two bridges for the east and west channels of the Speed River, as well as two overhead structures for the Grand River Electric Railway tracks and the Canadian National Railway tracks are situated within the project limits.

The ATMS pole is located immediately east of Speedsville Road. The surrounding lands are primarily undeveloped with some commercial/industrial properties to the north. The adjacent topography is relatively flat.

### **2.2 Site Geology**

This project lies within the physiographic region of southwestern Ontario known as the Waterloo Hills which primarily comprises sandy glacial till ridges or glacial kame moraines with outwash sands in the lower areas. The physiographic mapping indicates that the ATMS pole is situated in a former glaciofluvial spillway area.<sup>1</sup>

The quaternary geology mapping indicates that surficial materials at Station 17+450 will consist of stream deposits containing gravel, sand, silt and clay. There is also a rock outcrop to the southwest of Station 17+450.<sup>2</sup> The underlying bedrock surface at the ATMS pole location is found approximately at elevation 276.7 metres.<sup>3</sup> The rock formation is mapped and described as cream and brown, fine to medium crystalline dolomite of the Guelph Formation.<sup>4</sup>

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<sup>1</sup> Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

<sup>2</sup> Karrow, P.F., 1987: Quaternary Geology of the Cambridge Area, Southern Ontario. Ontario Geological Survey, Map 2508, scale 1:50,000.

<sup>3</sup> Ontario Department of Mines, 1960: Bedrock Topography, Galt Area, Southern Ontario. Map 2030, scale 1:50,000

<sup>4</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

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### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on July 31, 2013, during which time one borehole (borehole 7) was drilled at the location shown on the Borehole Location Plan, Drawing 1. The table below summarizes the location, ground surface elevation and depth of borehole 7:

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
7	4 808 565	235 994	277.57	4.75

The investigation was carried out using track mounted drilling equipment supplied and operated by a specialist drilling contractor. A sample of the overburden was obtained using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D1586. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres after first having penetrated 150 millimetres. The bedrock was encountered within a metre of the ground surface and was cored using NQ-sized rock coring equipment. Groundwater conditions were observed throughout the drilling operations. A standpipe piezometer was installed at the soil-bedrock interface to measure groundwater levels. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended). The standpipe was decommissioned on October 17, 2013.

The field work was monitored on a full-time basis by an experienced Golder staff member who also located the borehole in the field, monitored the drilling, sampling and in situ testing operations, logged and surveyed the borehole. The soil sample and bedrock cores were identified in the field, placed in labelled containers and transported to our London laboratory for further examination.



## **4.0 SUBSURFACE CONDITIONS**

### **4.1 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the borehole, together with the results of the in situ testing, are provided on the attached Record of Borehole sheet following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheet are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil and rock types rather than exact planes of geological change. Further, the subsurface conditions will vary beyond the borehole location.

Borehole 7 encountered thin layers of topsoil and silty sand overlying dolostone bedrock. Detailed description of the subsurface conditions encountered in borehole 7 is provided on the Record of Borehole sheet and are summarized in the following sections.

#### **4.1.1 Topsoil**

A 370 millimetre thick topsoil layer was encountered at the ground surface at borehole 7. Material designated as topsoil in this report was classified solely based on visual inspection and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

#### **4.1.2 Silty Sand**

The topsoil in borehole 7 was underlain by very dense silty sand at elevation 277.2 metres. The silty sand layer was about 470 millimetres thick and contained limestone fragments. An SPT N value of 100 for 75 millimetres of penetration was noted in the deposit.

#### **4.1.3 Bedrock**

The bedrock surface in borehole 7 was encountered at elevation 276.7 metres at a depth of about 0.8 metres. The bedrock was cored using NQ sized equipment to a depth of 3.9 metres below the bedrock surface. The bedrock was found to be fresh, thickly bedded microcrystalline dolostone of the Guelph Formation. Numerous fine mud-filled fractures were noted between depths of 1.8 to 2.4 metres or between elevation 275.1 and 275.7 metres. Occasional vugs were observed below a depth of 3.7 metres or elevation 273.9 metres. The strength of the bedrock was assessed as R4 or strong based on the Canadian Foundation Engineering Manual (2006) rock strength classification system. Although the intact rock is classified as strong to very strong, the behaviour of the rock mass will also be influenced by fissures, joints and other discontinuities. The bedrock was found to be of fair to excellent quality but generally good quality based on Rock Quality Designation (R.Q.D.) values ranging from 68 to 100 per cent with an average of 83 per cent. The Total Core Recovery (T.C.R.), Solid Core Recovery (S.C.R.) and R.Q.D. for the core obtained at borehole 7 are summarized in the table below:





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Elevation (m)		T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)
From	To			
276.7	275.9	84	68	68
275.9	274.3	101	85	82
274.3	272.8	100	100	100
Average		95	84	83

### 4.2 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling. A standpipe piezometer was installed in borehole 7 and sealed near the soil-bedrock interface. The groundwater was encountered at about elevation 277.4 metres during drilling on July 31, 2013. The most recent piezometer reading was obtained on October 17, 2013 at which time the groundwater level was measured at about elevation 277.3 metres. The details are summarized below:

Ground Surface Elevation (m)	July 31, 2013 (after installation)		August 12, 2013		October 17, 2013	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
277.57	0.56	277.01	0.41	277.16	0.28	277.29

Based on the measured and encountered groundwater levels, the inferred groundwater level is approximately at elevation 277.3 metres. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.



## **5.0 MISCELLANEOUS**

This investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of Mr. David J. Mitchell.

This report was prepared by Mr. Mrinmoy Kanungo, P.Eng. under the supervision of the Project Engineer, Ms. Dirka U. Prout, P.Eng. and the direction of the Team Leader, Dr. Storer J. Boone, P. Eng. This report was reviewed by Mr. Azmi M. Hammoud, P.Eng., an Associate with Golder Associates Ltd. An independent quality control review was carried out by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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**FOUNDATION INVESTIGATION AND DESIGN REPORT  
ATMS POLE - SITE 33-828-W-S**

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**PART B**

**FOUNDATION DESIGN REPORT**

**ATMS POLE - SITE 33-828-W-S**

**RECONSTRUCTION AND WIDENING OF HIGHWAY 401**

**FROM 0.5 KM WEST OF REGIONAL ROAD 8/KING STREET EASTERLY  
TO 0.5 KM EAST OF REGIONAL ROAD 24/HESPELER ROAD – 5.5 KM**

**GWP 4-00-00**

**MINISTRY OF TRANSPORTATION - WEST REGION**



## **6.0 ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides our recommendations on the foundation aspects of the design of the ATMS pole. The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling. It has been assumed that the VMS sign at Station 17+450 Lt will be pole mounted in the ground.

### **6.2 Foundation Design for ATMS Poles**

Foundations for ATMS poles should be designed in accordance with the requirements in MTO's Sign Support Manual (MTO, April 2011). Typically, caissons or drilled shafts of relatively short lengths are used to support highway structures such as sign posts, high mast lightings and noise walls. The design of such caissons is governed by wind/lateral loading considerations given the relatively light vertical load on these structures. The Sign Support Manual includes a standard caisson foundation design (Section 8 and Standard Drawings SS118-6, SS118-11, SS118-36 to SS118-38) in which the caissons are extended 5 metres below the design frost depth, except where bedrock is encountered within this depth. The presence of certain subsurface conditions, such as the presence of bedrock within the foundation zone, trigger a site-specific design with input from a foundation engineer. If sound rock is encountered at a depth of "Y" < 5 metres from the bottom of the frost layer, the caisson length can be reduced to  $Y + (5-Y)/2$  metres, subject to MTO's approval. The frost depth for this area is 1.4 metres. Based on the conditions encountered in borehole 7, the top of sound rock can be taken as elevation 275.0 metres (a depth of about 1.2 metres below the frost level) accounting for the presence of mud-filled fractures above this elevation. Therefore, according to the Sign Support Manual, the caisson length below the frost level should be a minimum of 3.1 metres, i.e., a total minimum length of 4.5 metres below grade.

Further guidelines for caisson foundations in rock are provided in Section 6.0 of the "Guidelines for the Design of High Mast Pole Foundations" (MTO, 2004). These guidelines, however, are for much taller poles than will be required for the ATMS sign and hence may be considered as a conservative basis for design. Three foundation types in rock are discussed in the High Mast Pole Guidelines:

- i) Caissons embedded in rock: A minimum embedment below the frost level of 2.5 metres into sound rock is suggested to develop adequate lateral load resistance. In this case, this yields a total minimum caisson length of about 5.1 metres below grade. Considering that an ATMS will be shorter than a high mast pole, a minimum caisson length of 4.5 metres below grade, as suggested by the sign support manual, is considered adequate.
- ii) Caissons anchored to rock: Where bedrock is at a relatively shallow depth and full embedment is uneconomical, an alternative is provided consisting of caissons socketed and anchored into bedrock. The socket depth is to be equivalent to the depth of the weathered rock which is 1.7 metres in this case. The resulting caisson will have a total length of 4.3 metres. Considering that a large caisson may be required to



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accommodate the rock anchor reinforcements, the use of anchors provide a minor reduction in rock coring depth and additional construction effort compared to a caisson embedded in sound rock. Therefore, this option was not considered further from a foundation engineering perspective.

- iii) Caissons socketed in rock: The tip of the caisson can be socketed into the bedrock where there is an insufficient depth of overburden to provide the required lateral resistance and only a minimum socket depth of one half the pile diameter into sound rock is needed to achieve the design lateral resistance. This condition is not applicable due to the very shallow overburden depth of 0.8 metres.

While the above guidelines provide the minimum lengths necessary for a caisson embedded in rock, the actual foundation type and dimensions should be determined by the structural engineer in consultation with MTO. The following sections provide general recommendations and design parameters for a standard 920 millimetre diameter caisson embedded in rock without the use of anchors. In case caisson construction is deemed to be unfeasible or cost-prohibitive due to the required embedment length or the strength of the bedrock, a spread footing option is also provided as an alternative. In addition to the two MTO documents noted above, the Canadian Foundation Engineering Manual (2006, 4<sup>th</sup> edition) and the Federal Highway Administration document "Drilled Shafts: Construction Procedures and LRFD Design Methods" (FHWA-NHI-10-016, May 2010) were referenced.

### 6.2.1 Caissons

#### 6.2.1.1 Resistance to Vertical Loads

The vertical/axial load capacity of an embedded caisson in rock and the relative contribution of shaft resistance and end bearing will depend on a number of factors including length-to-diameter ratio, rock strength and modulus, and condition of the drillhole sidewalls and base. General recommendations are provided here assuming contribution from end bearing resistance only for caissons constructed using conventional methods and good construction practices.

While no compressive strength testing was performed on the core sample from borehole 7,  $q_u$  data is available from the geotechnical investigation (Geocres No. 40P8-204) performed for the Speedville Road Underpass located about 80 metres west of this site. The data indicates strong to very strong bedrock with an average  $q_u$  value of 54 MPa. Since the MTO caisson designs are based on a compressive strength of concrete ( $\sigma_c$ ) of 30 MPa, a value of 30 Megapascals (MPa) should be used for  $q_u$ .

The unfactored end bearing component of the axial resistance of a caisson founded on sound rock can be calculated as follows:

$$Q_b = 2.5q_u\pi B^2 / 4$$

Based on the above and a resistance factor of 0.4, a factored axial resistance at ultimate limit states (ULS) of 19 megapascals can be used for design. No reduction factor was applied to this estimate since the RQD of the bedrock at or near the founding elevation was observed to be 100 per cent in borehole 7. The axial resistance at serviceability limit states (SLS) is greater than at ULS and ULS values will govern design.



### 6.2.1.2 Resistance to Lateral Loads

According to the MTO High Mast Pole Guidelines, caissons in rock need only be proportioned based on the ultimate lateral resistance. The SLS criterion is ignored because the rotation of caissons embedded in sound rock is assumed to be insignificant. The ultimate lateral resistance of the rock will be mobilized when the resisting rock first reaches its horizontal bearing resistance defined by  $f_{\text{horiz}}$  B. The horizontal bearing resistance ( $f_{\text{horiz}}$ ) should be taken equal to the compressive strength of the caisson concrete. The bedrock above elevation 275.0 metres should be ignored to account for the effect of weathering.

A resistance factor of 0.5 should be applied to obtain the factored lateral resistance at ultimate limit states (ULS).

### 6.2.2 Spread Footings

Spread footings constructed directly on bedrock will require the removal of up to 1 metre of existing overburden material to expose the bedrock surface, encountered at about elevation 276.7 metres in borehole 7. However, it is recommended to found the spread footing at or below elevation 275.0 metres to avoid the overlying zone of mud-filled fractures. Since the footing will be founded on bedrock, frost susceptibility is not a concern.

Detailed comments pertaining to construction of the footings on bedrock are presented in Section 6.3.2.

#### 6.2.2.1 Resistance to Vertical Loads

A factored geotechnical resistance at ULS of 1,200 kilopascals can be used for design. A geotechnical reaction at SLS is not provided since the bedrock is considered to be effectively unyielding (i.e., settlement values typically used to define the serviceability limit should not occur).

#### 6.2.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding between the concrete spread footing and the bedrock should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). Assuming that the surface of the bedrock is adequately prepared during excavation, the following angle of friction between the concrete and the founding bedrock and corresponding unfactored coefficient of friction,  $\tan \delta$ , may be used:

Footings on dolostone - angle of friction  $38^\circ$   
 $\tan \delta \quad 0.78$

For footings on bedrock, the sliding/lateral resistance between the concrete footing and the bedrock may be supplemented by dowelling/anchoring into the bedrock, if necessary. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels into the rock may be considered in the same way as dowels embedded into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a 1 metre minimum embedded length within the bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at these sites, a Non-Standard Special Provision (NSSP) should be included in the Contract Documents to specify the installation, materials and testing of the dowels.



## **6.3 Construction Considerations**

### **6.3.1 Excavations**

Open cut excavation for spread footings founded on bedrock will require removal of the surficial topsoil and silty sand overburden. The silty sand is saturated but since the depth is limited, dewatering may be accomplished using properly filtered sumps located outside of the footing limits. A Permit to Take Water is not considered necessary. If work is carried out during dry periods, the volume of saturated materials may be reduced. The estimated hydraulic conductivities of the materials which may be dewatered are:

- Silty sand -  $4 \times 10^{-6}$  m/s; and
- Dolostone bedrock -  $3 \times 10^{-6}$  m/s.

Surficial water seepage into the excavation should be expected and will be heavier during periods of sustained precipitation. Surface water should be directed away from the excavations at all times. The appropriate NSSP should be included in the Contract Documents to alert the contractor about the need for adequate control of surface and subsurface flows.

All excavations for the proposed footings should be carried out in accordance with the latest Occupational Health and Safety Act for Construction Projects (OHSA). All saturated granular materials would be classified by Type 3 soils. All properly dewatered granular materials would be classified as Type 2 soils.

### **6.3.2 Caissons**

For caissons embedded into the bedrock, a temporary liner may be required to support the excavation to the rock surface due to the presence of saturated granular native overburden soils. Spoil removal will require a use of augers and coring for overburden and bedrock, respectively. Alternatively, a 'Down-the-Hole' or rotary percussive hammer with carbide tips could be used. Coring for caisson construction will, however, be problematic because of the tendency for the fractured rock to jam coring equipment. The cleaned excavation base and the sidewalls of the socket should be inspected by the geotechnical QVE prior to placing reinforcement and pouring concrete. The concrete should be poured as soon as practicable after excavation to the required depth of embedment and subsequent examination to avoid any degradation/softening of the rock surface due to prolonged exposure.

### **6.3.3 Spread Footings**

For the excavation of a spread footing, blasting is not recommended since it may generate more fractures and widen existing fractures in the bedrock. It is expected that the bedrock at the surface will be weakened by water present in the overlying granular soil. If close control of the excavation dimensions is required, then impact hammers or hydraulic splitters may be used. Use of hydraulic splitters and expansion agents, however, is less preferable since it is difficult to control the direction of the cracks in fractured rock with these methods.

Footings must be formed in competent bedrock that is relatively free of defects. The bearing surface should be cleared of all loose and broken rock. The cleaned excavation base should be inspected by a geotechnical QVE.



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It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical QVE on site and the footing constructed immediately after bedrock inspection. Additional excavation may be required if inspection reveals that the bedrock at the design foundation depth is unsuitable.

Bearing surfaces may be improved by use of non-shrink grout if open joints or cracks are present. Use of non-shrink grout depends on the discontinuity width and is only suitable if the fault, crack or seam of crushed/faulted bedrock is not more than one third of the footing width. The discontinuity should be cleaned to a depth equal to twice its width and backfilled with non-shrink grout or structural concrete. Grouting through drilled holes may also be used to remediate discontinuities or other natural openings in the bedrock. A Non-Standard Special Provision (NSSP) should be added to the Contract Documents to alert the Contractor to the requirements for constructing shallow foundations on rock.

The excavation around and above the spread footing may be backfilled using an approved granular material such as OPSS Granular 'A' or 'B' (Type I or II) placed in 0.3 metres thick loose lifts and uniformly compacted to not less than 95 percent of the standard Proctor maximum dry density of the material. The use of native excavated materials, where encountered, as backfill is not recommended.

The final grade surrounding the ATMS pole should be sloped to promote surface water drainage and pavement structure drainage away from the pavement and sign support, to the adjacent ditch.





## **7.0 MISCELLANEOUS**

This report was prepared by Mr. Mrinmoy Kanungo, P.Eng. under the supervision of the Project Engineer, Ms. Dirka U. Prout, P.Eng. and the direction of the Team Leader, Dr. Storer J. Boone, P. Eng. This report was reviewed by Mr. Azmi M. Hammoud, P.Eng., an Associate with Golder Associates Ltd. An independent quality control review was carried out by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

#### Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### (b) Cohesive Soils

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note: 1** Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

## LIST OF SYMBOLS

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

### I. General

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

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

#### (d) Shear Strength

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

- Notes:**
- 1  $\tau = c' + \sigma' \tan \phi'$
  - 2 shear strength = (compressive strength)/2
  - \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)

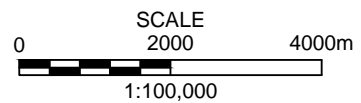
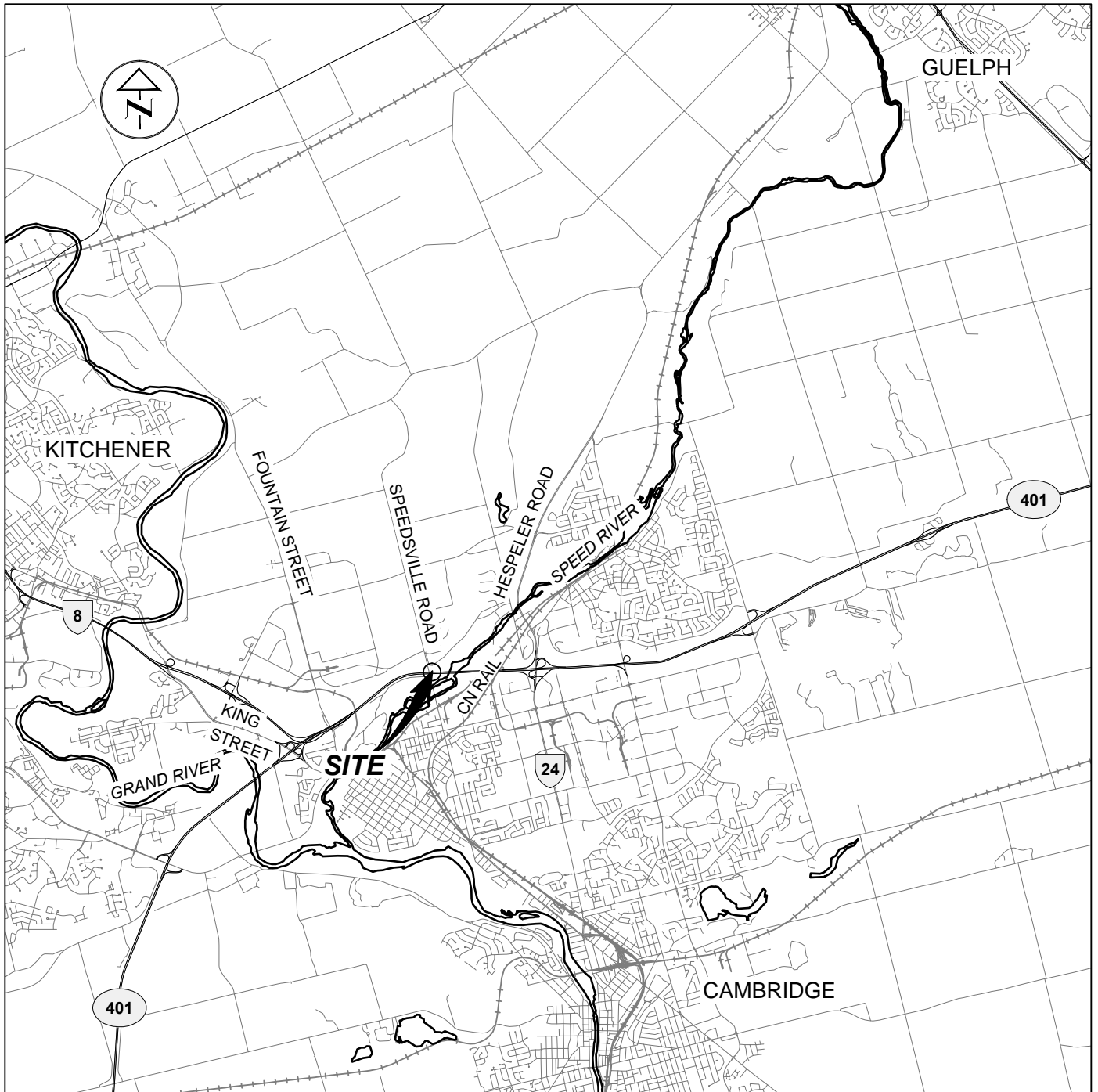
**RECORD OF BOREHOLE No 7**

1 OF 1

**METRIC**

PROJECT 10-1132-0056  
W.P. 4-00-00 LOCATION N 4808564.5 , E 235993.6 ORIGINATED BY MA  
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / NQ ROCKCORE COMPILED BY LMK  
DATUM GEODETIC DATE July 31, 2013 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE									
277.57	GROUND SURFACE						20	40	60	80	100									
0.00	TOPSOIL, silty, with roots																			
277.20	Brown																			
0.37	SILTY SAND, with limestone fragments, trace organic material																			
276.73	Very dense		1	SS	100/75mm															
0.84	Grey																			
	DOLOSTONE, fresh, thickly bedded, buff grey to blue grey, microcrystalline, strong (R4), common stylolites, occasional vugs below elev. 274.2m. Zone of dark grey dolostone with numerous fine mud filled fractures between about elev. 275.1m and elev. 275.7m		2	NQ RC			84	68	68											
			3	NQ RC																
			4	NQ RC																
272.82	END OF BOREHOLE																			
4.75	Groundwater encountered at about elev. 277.4m during drilling on July 31, 2013.  Water level measured at elev. 277.01m after installation on July 31, 2013.  Water level measured at elev. 277.16m on August 12, 2013.  Water level measured at elev. 277.29m on October 17, 2013 prior to decommissioning.																			



## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

## NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT

ATMS POLE  
HIGHWAY 401 IMPROVEMENTS  
GWP 4-00-00

TITLE

## KEY PLAN



PROJECT No. 10-1132-0056		FILE No. 1011320056-2000-F14001	
CADD	LMKWDF	Dec. 17/13	SCALE AS SHOWN REV. 0
CHECK			

**FIGURE 1**

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

CONT No.  
WP No. 4-00-00

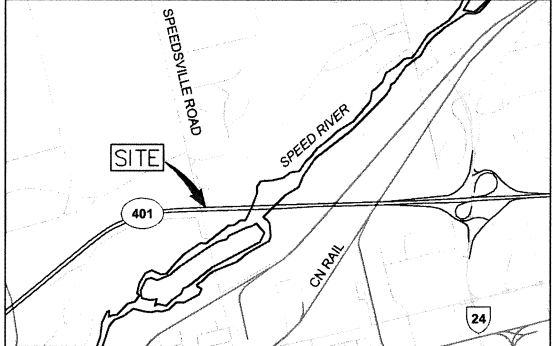


ATMS POLE  
HIGHWAY 401 IMPROVEMENTS  
BOREHOLE LOCATIONS

SHEET



**Golder Associates Ltd.**  
LONDON, ONTARIO, CANADA



KEY PLAN

SCALE IN KILOMETRES  
0 0.5 1.0

LEGEND

● Borehole - Current Investigation

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
7	277.57	4 808 564.5	235 993.6

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Delcan.



NO.	DATE	BY	REVISION
Geocres No. 40P8-219			
HWY.	401	PROJECT No. 10-1132-0056	DIST.
SUBM'D.	AW	CHKD. DUP	DATE: Dec. 17/13
DRAWN:	LMK\WDF	CHKD. SUB	APPD. FJH
SITE: 33-828-W-S		DWG. 1	



# **APPENDIX A**

## **Rock Core Photographs**





276.70 m

275.86 m

274.34 m



275.86 m

274.34 m

272.82 m

Photograph 1: BH7 Elevation 276.50 metres to 272.82 metres.



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