

**FOUNDATION INVESTIGATION AND DESIGN REPORT
RETAINING WALL
TRANS CANADA TRAIL UNDERPASS
HIGHWAY 403 AND OAK PARK ROAD
INTERCHANGE IMPROVEMENTS
GWP 3950-01-00, AGREEMENT NO. 3005-E-0067
MINISTRY OF TRANSPORTATION - WEST REGION**

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PART A – FOUNDATION INVESTIGATION REPORT

**RETAINING WALL
TRANS CANADA TRAIL UNDERPASS
HIGHWAY 403 AND OAK PARK ROAD INTERCHANGE IMPROVEMENTS
GWP 3950-01-00, AGREEMENT NO. 3005-E-0067
MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION**

October 2008

07-1130-204-1-R03

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 3950-01-00. The detail design for the interchange improvements at the Highway 403 and Oak Park Road Interchange includes:

- Reconstruction of the interchange to a Parclo A-4;
- Widening of Oak Park Road;
- Conventional illumination;
- Signalization at the W-N/S and E-N/S Ramp terminals;
- Retaining walls at both Oak Park Road Underpass and the Trans Canada Trail Underpass; and,
- Rehabilitation of the Oak Park Road Underpass.

This report was prepared for the design of retaining wall at the Trans Canada Trail (Site 1-158) which will be constructed in conjunction with the proposed works. A widened speed change lane is required for the proposed N-W ramp. Part of the existing slope paving adjacent to the north side of the Highway 403 Trans Canada Trail Underpass structure will be removed to accommodate the widened speed change lane. The lower portion of the slope fills will be retained by a small retaining wall approximately 0.825 metres in height. Three alternative retaining wall designs have been proposed. At this stage only Alternatives 1 and 2 are under consideration.

The purpose of the foundation investigation is to determine the subsurface conditions at the locations of the proposed works by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal, Golder Associates' proposal P61-3107 dated October 17, 2007 and our modified Innovation Plan as described in a letter dated June 20, 2008. The work was carried out in accordance with our revised Quality Control Plan for Foundation Engineering dated January 30, 2008 and our Innovation Plan modification letter June 20, 2008.

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

2.0 SITE DESCRIPTION

Detail design of reconstruction of the Highway 403 and Oak Park Road Interchange to a Parclo A-4 configuration is the focus of GWP 3950-01-00. This improvement will require widening of Oak Park Road, construction of retaining walls at the Oak Park Road Underpass (Site 1-139) and Trans Canada Trail Underpass (Site 1-158) and illumination and traffic control improvements. At the Trans Canada Trail Underpass, a portion of the front slope paving on the north side will have to be removed in order to accommodate the speed change lane for the new N-W ramp. The retaining wall will be approximately 50 metres long and about 0.8 metres high.

The site is situated in Brantford, Ontario approximately 450 metres east of the Grand River and 550 metres west of Paris Road. The site location is shown in the Key Plan, Figure 1. Site photographs are presented in Appendix C.

Highway 403 is a divided highway with two lanes and a speed change lane in each direction. The Trans Canada Trail in the vicinity of Highway 403 follows the former Lake Erie and Northern Railway (LENR) alignment. The existing underpass structure, formerly known as the LENR Subway, was constructed in 1975/1976. The bridge has two 36.6 metre long spans and a cast in place post tensioned concrete bridge deck 5.6 metres wide. The centre pier and both abutments are founded on spread footings. According to the Ontario Ministry of Transportation and Communications General Layout for W.P. 158-60-00 dated May 1975, the design underside of footing elevation for the centre piers was 236.52 metres. The design underside of footing elevations for the north and south abutments were 241.40 metres and 240.64 metres, respectively. Examination of the March 1972 Ontario Department of Transportation and Communications Drawing No. 71-11110 A entitled "Borehole Locations and Soil Strata, Lake Erie and Northern Railway, C.A.H. 403 Line 'K'" indicated that Highway 403 was constructed in a cut approximately 7.1 metres deep. The approximate elevation of Highway 403 is 238.87 metres. The cut slopes reportedly consisted of loose to very dense but generally very dense sandy gravel to gravelly sand. Information from borehole 2 of the geotechnical investigation report prepared for W.P. 158-60-00 (Geocres 40P1-55) has been incorporated into this report.

The Trans Canada Trail is situated on the plateau above the steep sided east valley wall of the Grand River. Former gravel pits are located on both sides of Highway 403, east of the trail. These lands are now being redeveloped as the North West Industrial Subdivision and the Oak Park North Industrial Subdivision. West of the trail is mainly vacant land. The ground surface in the gently sloping plateau area is between 243 and 245 metres. A storm sewer is located approximately 11.43 metres south of the face of the north abutment.

2.1 Site Geology

The Trans Canada Trail Underpass is situated on a former spillway in the physiographic region of southern Ontario known as the Horseshoe Moraines¹. The Horseshoe Moraines stretch from the Niagara Escarpment to the Regional Municipality of Haldimand-Norfolk. South of Paris, the moraines flatten out and disappear under the adjacent Norfolk sands. Associated with the moraines are old spillways with broad sand and gravel terraces with swampy floors.

The surficial soils are primarily composed of glaciofluvial outwash and deltaic deposits of gravel and gravely sand. In several places the gravely deposits are overlain by several metres of sand. These advanced and recessional outwash deposits are derived from meltwaters of the Wentworth Ice. The largest gravel masses are found in Brantford and Paris.²

The underlying bedrock surface lies between elevations 198 and 213 metres.³ The bedrock is reported to be tan dolomite and grey mudstone of the Salina Formation.⁴

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

² Ontario Department of Mines and Northern Affairs: Pleistocene Geology of the Brantford Area, Southern Ontario, 1972. Preliminary Map 2240. Scale 1:63, 360.

³ Karrow, P.F. and Sprague, D.J.: Bedrock Topography Series, Brantford Area, Southern Ontario, 1975. Preliminary Map 1049. Ontario Division of Mines. Scale 1:50, 000.

⁴ Sandford, B.V. Bedrock Geology Toronto-Windsor Area. 1969. Geological Survey of Canada Map 1263 A. Scale 1:250, 000.

3.0 INVESTIGATION PROCEDURES

A single test pit, test pit 307, was excavated on the northwest side of the Trans Canada Trail Underpass structure on July 3, 2008. The test pit was excavated using a JD 590 D excavator operated by a local excavating contractor. Chunk samples of the excavated materials from the test pit were taken at suitable intervals.

Groundwater conditions in the test pit were observed throughout the excavating operations and these observations are provided on the corresponding Record Test Pit sheet. The area disturbed by the test pit was regraded.

The field work was supervised on a full-time basis by experienced members of our engineering staff who arranged for utility locates, directed the excavation and sampling operations, logged the test pit, cared for the samples obtained and surveyed the test pit elevation. Traffic control was provided by the area maintenance contractor. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' London laboratory for further examination and testing. Index and classification tests consisting of water content determinations and grain size distribution analyses were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Test Pit sheets in Appendix A.

Information from the test pit was supplemented with information from borehole 2 which was drilled for the original geotechnical investigation for the structure at this site.

The table below summarizes the locations, ground surface elevations and depths of the borehole and the test pit:

<u>BOREHOLE/ TEST PIT</u>	<u>LOCATION (m)</u>		<u>GROUND SURFACE ELEVATION</u>	<u>DEPTH</u>
	<u>Northing</u>	<u>Easting</u>	<u>(m)</u>	<u>(m)</u>
307	4 781 156.2	236 052.8	237.71	1.80
2	4 781 158.2	236 076.7	243.99	11.52

The Geocres 40P1-55 borehole and test pit locations are shown in plan on Drawing 1.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the borehole and test pit together with the results of the in situ and laboratory testing carried out on selected samples, are given on the attached Record of Test Pit sheet following the text of this report and the Record of Borehole sheet in Appendix A. The stratigraphic boundaries shown on the Record of Borehole and Record of Test Pit sheets are inferred from non-continuous sampling and observations of drilling resistance and represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole and test pit locations.

The locations of the borehole and the test pit and a stratigraphic profile along the retaining wall are shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole and Test Pit sheets and is summarized in the following sections.

4.1.1 Topsoil

A 300 millimetre thick layer of topsoil was encountered at the surface of test pit 307.

4.1.2 Sand and Gravel

Sand and gravel with cobbles and boulders was encountered below the topsoil in test pit 307 from elevation 237.4 metres and from the surface of borehole 2 from the previous investigation. Material described as sandy gravel to gravely sand on the Record of Borehole 2, has been inferred to be sand and gravel. The description of sand and gravel will be used in the remainder of this report to refer to materials formerly described as sandy gravel to gravely sand.

The sand and gravel is loose to very dense but generally very dense with N values ranging from 6 to over 100 blows per 0.3 metres. Water contents of 2 to 22 per cent were measured in the samples of the sand and gravel.

A grain size distribution curve for a sample of sand and gravel recovered from test pit 307 is shown on Figure A-1.

4.2 Groundwater Conditions

Test pit 307 was dry during and upon completion of excavation. Groundwater was encountered in borehole 2 at about elevation 234.2 metres on January 27, 1972. Groundwater levels are subject to climatic and seasonal variations.

5.0 MISCELLANEOUS

The test pit was excavated using equipment supplied and operated by Bellhouse Excavating. The field operations were supervised by Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates.

This report was prepared by Ms. Dirka U. Prout, P. Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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PART B – FOUNDATION DESIGN REPORT

**RETAINING WALL
TRANS CANADA TRAIL UNDERPASS
HIGHWAY 403 AND OAK PARK ROAD INTERCHANGE IMPROVEMENTS
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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the retaining wall to be constructed along the front slope of the Trans Canada Trail north abutment. The approximate limits of the wall are Stations 10+640 to 10+688 based on the N-W ramp chainage. 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.

Retaining wall Alternatives 1 and 2 are currently under consideration. Alternative 1 involves construction of a retaining (toe) wall approximately 6.3 metres in front of the north abutment face. The front slope will be maintained at a 2 horizontal to 1 vertical slope. The fill will be placed the full height of the wall. For Alternative 2, the inclination of the front slope will be increased to 1.5 horizontal to 1 vertical and the retaining wall will be at the same location as in Alternative 1. The height of fill behind the wall will be 0.3 metres. With both Alternatives 1 and 2 the shoulder width of 2.5 metres will be maintained and the existing storm sewer, situated about 11.4 metres in front of the north abutment face, will be removed or filled.

The current retaining wall design is tentatively following Alternative 2. It has been proposed to construct a retaining wall approximately 0.825 metres high. The footing of the retaining (toe) wall will be 0.63 metres wide with a footing depth of 0.5 metres or near elevation 238.5 metres. A pavement subdrain is to be installed in front of the wall. The height of backfill on the walls will be 0.305 metres with a 1.5 horizontal to 1.0 vertical backslope. The selected retaining wall alternative is No. 2 which is a modified Type "A" wall constructed in general accordance with OPSD 911.231 but modified to have a vertical back.

6.2 Comparison of Retaining Wall Alternatives

Retaining wall Alternatives 1 and 2 both result in wall heights in the order of 1 metre or less. However, since the height of backfill behind the wall is higher for Alternative 1 than Alternative 2, temporary shoring will be required for Alternative 1. Shoring alternatives are discussed in detail in Section 6.6. The most suitable shoring options are soldier piles and lagging or tiebacks. Other options such as soil nails and sheet piling are not considered suitable. Temporary shoring is discussed in detail in Section 6.6. Due to the relatively low wall height, soldier piles and lagging may be the more economical shoring option.

The steepened front slope with Alternative 2 increases the risk of movement of the north abutment footing compared to Alternative 1. Since the slope will be steepened and the fill height behind the wall will be about 0.3 metres, minimal to no shoring will be required for Alternative 2. The cost of constructing the retaining wall foundation is expected to be medium to low for Alternative 1 and low for Alternative 2 since retaining wall construction for Alternative 2 will likely be more straightforward than Alternative 1. Alternative 2 is the preferred technical alternative. A summary of the comparison of the two retaining wall alternatives from a foundation engineering point of view is presented in Table I.

6.3 Retaining Wall Foundations

The subsurface conditions encountered in the retaining wall area typically consist of surficial topsoil overlying loose to very dense but typically very dense sand and gravel with cobbles and boulders. The groundwater level is approximately at elevation 234 metres. The very dense, coarse granular materials are suitable for shallow foundations for RSS, gravity and cantilever walls. The following proposed wall designs are considered to be geotechnically feasible based on the encountered subsurface conditions:

- Cast-in-place concrete cantilever or gravity retaining wall with temporary tied protection system (designed by the contractor);
- Soldier pile with timber lagging and a cast-in-place concrete facing, tied back with grouted earth anchors;
- RSS wall with temporary tied protection (designed by the contractor).

6.3.1 Shallow Foundations

6.3.2 Geotechnical Resistance

Strip or spread footings for a gravity or cantilever wall founded in the very dense sand and gravel near elevation 238.5 metres can be designed using a factored geotechnical resistance at Ultimate Limit States of 500 kilopascals and 350 kilopascals at Serviceability Limit States (SLS). The SLS value allows for 25 millimetres of settlement. The geotechnical resistances provided are based on a 0.63 metre wide footing.

Alternatively, an RSS wall footing designed with the geotechnical resistances given above may be founded on a 0.3 metre thick compacted Granular A leveling pad constructed on the surface of the very dense sand and gravel. It is anticipated that the reinforced width of the RSS wall would be approximately 75 per cent of the wall height.

6.3.3 Resistance to Lateral Forces

The lateral pressures acting on the retaining wall will depend on the backfill soils, the type and method of placement of the backfill materials behind the wall, the subsequent lateral movement of the structure and the provision of reinforcement grids if an RSS wall is constructed. The use of Granular A or coarse free draining granular material as backfill is recommended due to the 1.5 horizontal to 1 vertical slope above the wall.

The resistance to lateral forces/sliding resistance between the compacted granular fill (assumed to be Granular A) and the subgrade soils should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). Also, the retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angles of friction and corresponding unfactored coefficient of interface friction, $\tan \delta$, may be used for the interaction between the concrete and the founding soil:

Footings on sand and gravel	angle of friction	34°
	$\tan \delta$	0.67
Footings on Granular A (RSS wall)	angle of friction	33°
	$\tan \delta$	0.65

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If an RSS wall is to be constructed, the internal stability of the mechanically-reinforced soil wall should be checked by the RSS supplier/designer. The Factor of Safety related to the global stability under static loading for properly designed and constructed RSS wall at this site is greater than 1.3. The design and construction of the RSS wall should be carried out in accordance with the manufacturer's design recommendations and MTO Special Provisions SP599S22 and SP599S23.

6.3.4 Construction Considerations

The chief design challenges with installation of the retaining walls are the very dense coarse grained soils as identified in Test Pit 307 which contain cobbles and boulders, the necessity to work in an area with low headroom, and the requirement to excavate in close proximity to the existing abutment footings. Depending on the wall option selected, a carefully designed and constructed shoring system will be required to maintain the integrity of the foundations for the north abutment of the Trans Canada Trail structure. Shoring is discussed further in Section 6.6.

Frost Protection

The abutment footing of the existing underpass should be provided with a minimum of 1.2 metres of earth cover or equivalent thermal insulation for frost protection purposes. However, the footings for the new retaining wall (toe wall) will have a cover of only 0.5 metres. The results of the geotechnical investigation have confirmed that the groundwater table is deep and the soils in the area of the retaining wall are free draining. Noting the high permeability of the native sand and gravel, and a groundwater level approximately 4.5 metres below the anticipated footing depth, the potential for frost heave is very low provided there is adequate surface drainage.

6.4 Front Slope Modifications

The inclinations of the existing front slope at the abutment will be increased from 2 horizontal to 1 vertical to 1.5 horizontal to 1 vertical. Stability analyses carried out based on the results of the borehole indicate that an adequate factor of safety exists for the steepened slopes both for overall slope stability and for the existing north abutment foundation.

6.5 Lateral Earth Pressures

The lateral pressures acting on the retaining walls will depend on the wall type, the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the retaining wall in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A but with less than 5 per cent passing the 75 micron sieve should be used as backfill behind the wall. The granular fill should be placed in accordance with Ontario Provincial Standard Drawing (OPSD) 3121.150 and compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP105 S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains should be in accordance with OPSD 3190.100.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with CHBDC, Figure 6.9.3. Compaction equipment should be used in accordance with SP105 S10.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Clause C6.9.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Clause C6.9.1).

- For Cases a and b, the pressures are based on compacted Granular A fill and the following parameters (unfactored) may be assumed:

GRANULAR A

Soil unit weight:	22 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.53
At rest, K_o	0.72
Passive, K_p	0.88

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design parameters assume a backfill inclination of 1.5 horizontal to 1 vertical and a horizontal surface in front of the wall. If the final design slopes differ, these parameters should be adjusted as in CHDBC C6.9.1 (e).

6.6 Excavations and Temporary Cut Slopes

Excavations for the retaining wall footings will extend through the existing surficial topsoil materials into the underlying very dense sand and gravel. Excavation in the very dense granular materials will likely be difficult and the presence of cobbles and boulders should be anticipated. In no case should the 1 horizontal to 1 vertical temporary excavation slope encroach onto the bearing surface soils of the abutment footing. A statement to this effect should be included in the Contract Documents.

Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils. The native granular soils would be classified as Type 1 soil.

Temporary Shoring

Localized widening of Highway 403 and construction of the new N-W ramp will result in partial removal of the slope paving at the north abutment of the Trans Canada Trail Overpass structure. Depending on the wall type selected, temporary shoring, which can be designed by a contractor, will be required to construct the new retaining walls. Various shoring alternatives such as soil nail walls, sheet piling, soldier piles with rib and lagging and tieback shoring were considered. Support to the system could be in the form of rakers and anchors. The raker/anchor support must

be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2.

The very dense sand and gravel deposits contain cobbles and occasional boulders were observed in the nearby gravel pits. Although it was possible during the field investigation to advance the boreholes using hollow stem augers and conduct standard penetration testing, excavation and tieback installation activities are expected to be difficult. For this reason, the selection of a specialist shoring contractor experienced in working in such ground conditions is important. Sheet piling cannot be installed in the very dense sand and gravel materials. Therefore it will be necessary to use a soldier pile and lagging system for temporary shoring with the piles preferably installed in predrilled holes. The sands and gravel materials are dry and will be subject to running if left unsupported and, as such, the lagging should be installed immediately as the excavation proceeds. The low headroom will require the contractor to use equipment specially adapted for such purposes.

Installation of a soil nail wall would be difficult and would require use of hollow core nails and a cement drilling grout due to the limited standup time of the generally very dense native deposits. There is potential for ravelling as the sand and gravel materials have a low water content and no cohesion. It will be necessary to very carefully expose a minimal length of face at any one time. In addition, a structural concrete facing would be required. These special measures will increase the construction costs.

The design of braced soldier pile and lagging walls should be based on rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

The unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

	p	=	$K_a (\gamma H + q)$
where	H	=	the height of the excavation at any point in metres
	K_a	=	active coefficient of earth pressure
	γ	=	soil unit weight
	q	=	surcharge for traffic and other loading

For the granular fill, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows:

$$p = 0.65 (K_a \gamma H + q)$$

where H = the total height of the excavation
 K_a = active coefficient of earth pressure
 γ = soil unit weight
 q = surcharge for traffic and other loading

The support systems may be designed using the following parameters:

<u>SOIL TYPE</u>	<u>COEFFICIENT OF EARTH PRESSURE</u>			<u>INTERNAL ANGLE OF FRICTION (degrees)</u>	<u>UNIT WEIGHT (kN/m^3)</u>
	<u>Active, K_a</u>	<u>At Rest, K_o</u>	<u>Passive, K_p</u>		
Granular Fill	0.53	0.72	0.88	35.0	21
Sand and Gravel	0.50	0.70	0.91	35.5	21

The earth pressure coefficients noted above are based on a 1.5 horizontal to 1.0 vertical slope behind the walls and a horizontal surface in front of the walls. If differently sloped surfaces are present, the coefficients should be adjusted accordingly.

7.0 MISCELLANEOUS

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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TABLE I

FOUNDATION CONSIDERATIONS COMPARISON FOR RETAINING WALL ALTERNATIVES

Highway 403/Trans Canada Trail Underpass
Highway 403 and Oak Park Road Interchange Improvements
GWP 3950-01-00

RETAINING WALL ALTERNATIVE	ADVANTAGES	DISADVANTAGES	FOUNDATION COST AND RISK RATING
1. Retaining Wall with 2 horizontal to 1 vertical front slope	<ul style="list-style-type: none"> • Gravity toe wall adequate • Sufficient room for installation of tieback shoring (designed by contractor) 	<ul style="list-style-type: none"> • Requires shoring • Difficulties due to the presence of cobbles in the very dense sand and gravel should be anticipated • Specialized equipment needed • Specialized subcontractor required • Alternative with second highest magnitude of relative structure settlement • Horizontal and vertical pressures exerted on wall by structure must be considered 	<ul style="list-style-type: none"> • Medium/Low Cost • Medium/Low Risk
2. Modified Type "A" Wall with 1.5 horizontal to 1 vertical front slope	<ul style="list-style-type: none"> • Gravity toe wall adequate • Lower wall required compared to Alternative 1; minimizes wall height • Minimal to no shoring required • Preferred technical alternative 	<ul style="list-style-type: none"> • Difficulties due to the presence of cobbles in the very dense sand and gravel should be anticipated 	<ul style="list-style-type: none"> • Low Cost • Medium Risk

NOTE: 1. Table to be read in conjunction with accompanying report.

Prepared By: DUP
Checked By: PRB

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 cm² 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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
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)
γ	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

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content	l
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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

RECORD OF TEST PIT No 307

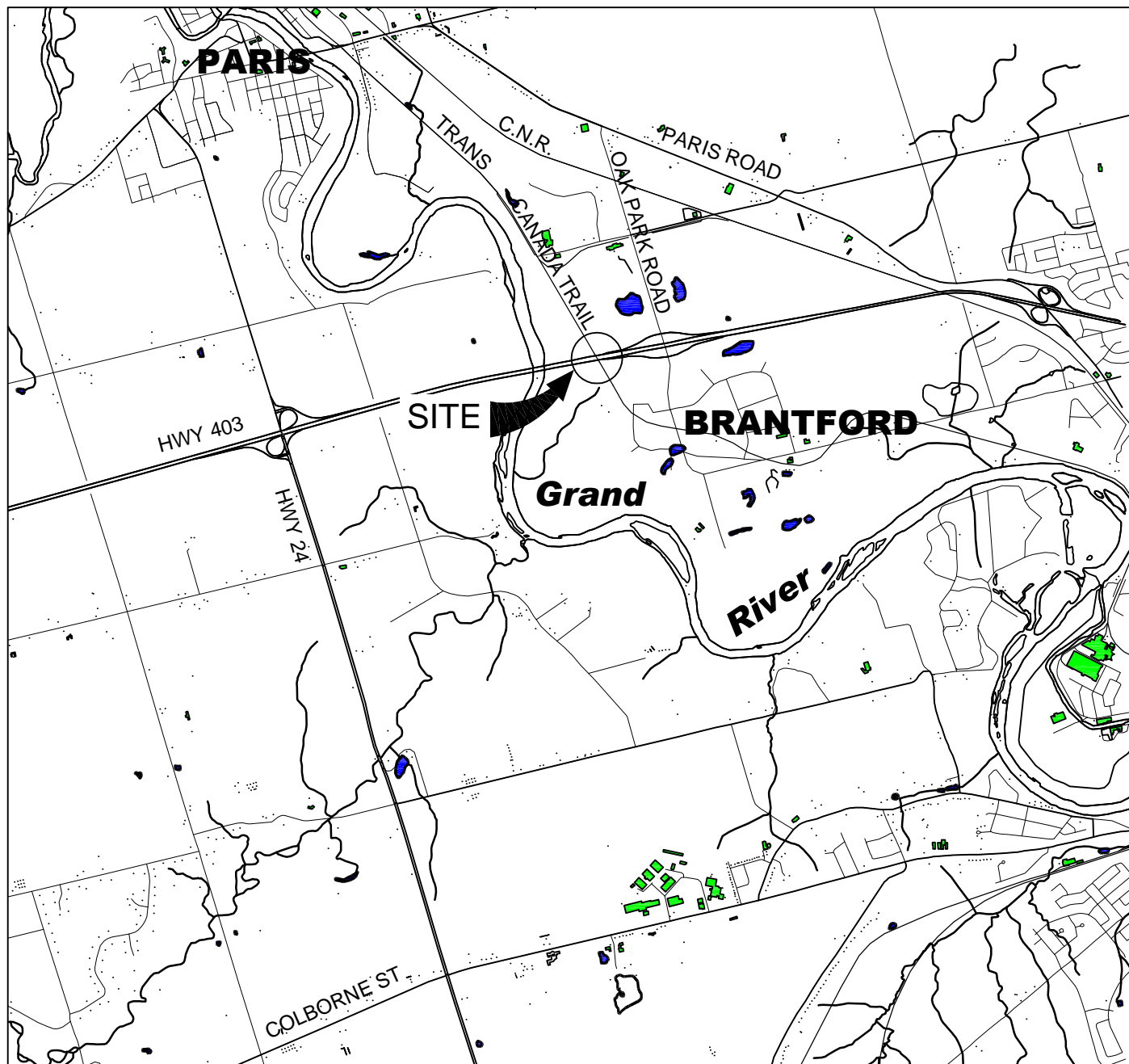
1 OF 1

METRIC

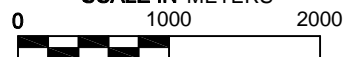
PROJECT 07-1130-204-1
G.W.P. 3950-01-00 LOCATION N 4781156.2 ; E 236052.8 ORIGINATED BY D.J.M.
DIST HWY 403 TEST PIT TYPE Backhoe Dug COMPILED BY BRS
DATUM GEODETIC DATE July 3, 2008 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
237.71	GROUND SURFACE																
0.00	TOPSOIL, silty Black																
0.30	SAND AND GRAVEL, with cobbles and boulders Brown		1	CS			237										56 41 (3)
235.91			2	CS			236										
1.80	END OF TESTPIT Test pit dry during excavation on July 3, 2008.																

LDN_MTO_01_0711302041.GPJ LDN_MTO.GDT 10/14/08



SCALE IN METERS



1:50000

PROJECT HIGHWAY 403 / TRANS CANADA TRAIL UNDERPASS
HIGHWAY 403 & OAK PARK ROAD INTERCHANGE
IMPROVEMENTS
GWP 3950-01-00

TITLE

KEY PLAN



**Golder
Associates**
LONDON, ONTARIO

PROJECT No. 07-1130-204-1

FILE No. 0711302041-F03001

CADD WDF Sept. 4/08

CHECK

SCALE AS SHOWN REV. 0

FIGURE 1



SCALE
10 0 10 m



SCALE
1.5 0 1.5 m

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

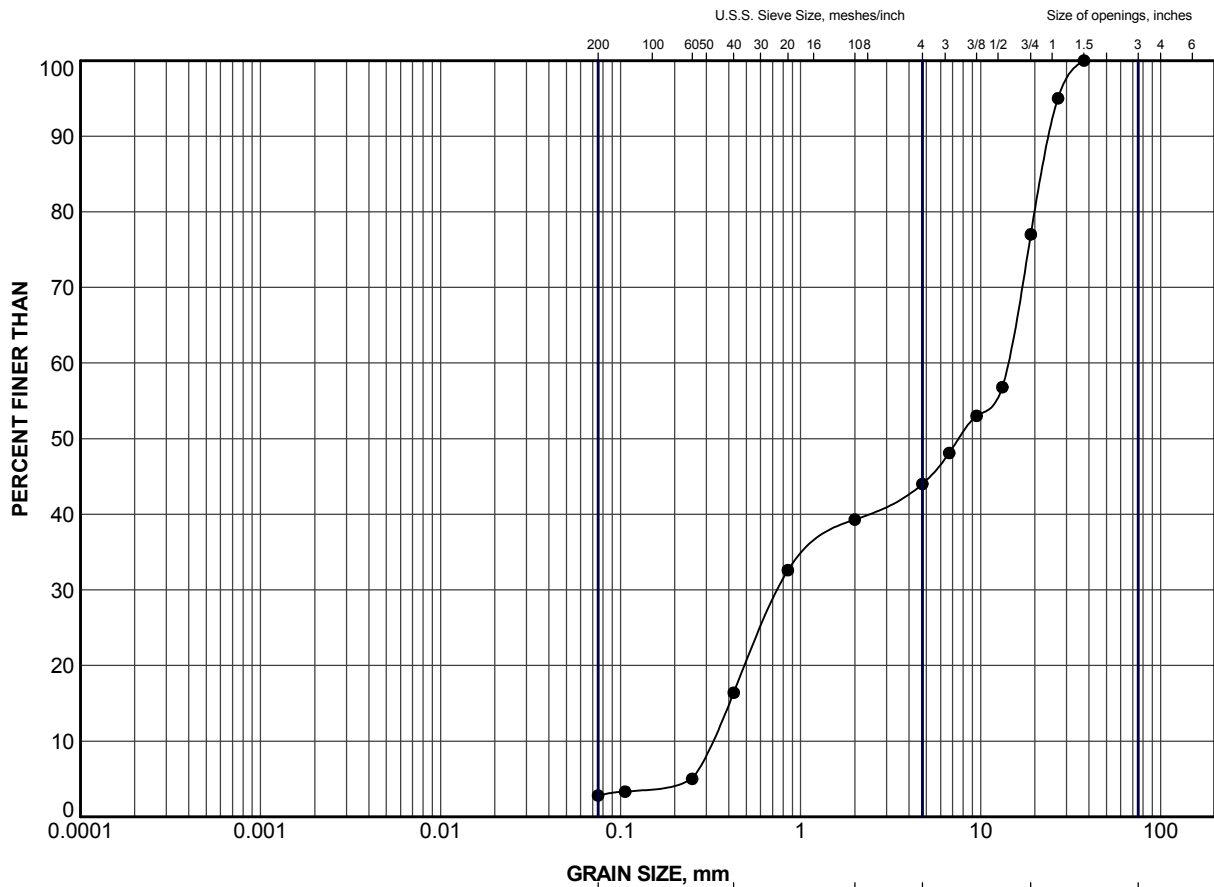
SCALE IN KILOMETRES

0 1 2



NO.	DATE	BY	REVISION	
Geocres No. 40P1-98				
HWY. 403		PROJECT NO. 07-1130-204-1		DIST.
SUBM'D.	DUP	CHKD.	DATE: OCT. 8/08	SITE: 1-158
DRAWN: LMK		CHKD.	APPD.	DWG. 1

APPENDIX A
LABORATORY TEST DATA



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	307	1	236.9

PROJECT	HIGHWAY 403 / TRANS CANADA TRAIL UNDERPASS HIGHWAY 403 & OAK PARK ROAD INTERCHANGE IMPROVEMENTS GWP 3950-01-00				
TITLE	GRAIN SIZE DISTRIBUTION SAND and GRAVEL				
 Golder Associates LONDON, ONTARIO	PROJECT No.	07-1130-204-1	FILE No.	0711302041-R030A1	
	DRAWN	LMK	Aug 24/08	SCALE	N/A
	CHECK			REV.	
			FIGURE A-1		

APPENDIX B

PREVIOUS BOREHOLE DATA

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 71-11110 LOCATION Sta. 444. + 59 86' Rt. ORIGINATED BY PK

W.P. 158-60-00 BORING DATE Jan. 26, 1972 & 27 Jan/72 COMPILED BY PK

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY PK

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		REMARKS
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE		WATER CONTENT % 10 20 30	BACK DENSITY γ	
800.5	Ground Level								
0.0		1	SS	6					0 52 34 14
		2	SS	38					28 47 17 8
	Sandy gravel to	3	SS	100/3 1/2"					
	gravelly sand, some	4	SS	100/9"					
	silt, traces of clay.	5	SS	100/5"					56 40 (4)
		6	SS	100/5"					48 47 (5)
		7	SS	100/1 1/2"					
		8	SS	100/8"					
		9	SS	100/6 1/2"					28 68 (4)
	Loose to Very Dense	10	SS	100/1 1/2"					
762.7	rown	11	SS	100/3"					3 86 1 11
37.3	End of Borehole								

APPENDIX C
SITE PHOTOGRAPHS

SITE PHOTOGRAPHS
GWP 3950-01-00



Photo 1 – North Abutment – Highway 403 / Trans Canada Trail Underpass.
Looking east from eastbound lane.



Photo 2 – North abutment as seen from westbound lanes.