

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
FISH CREEK (NORTH) BRIDGE REPLACEMENT
HIGHWAY 23, SITE 25-224, GWP 313-94-00
AGREEMENT NUMBER 3005-A-000078

Submitted to:

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LIST OF SYMBOLS

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PART A –FOUNDATION INVESTIGATION REPORT
FISH CREEK (NORTH) BRIDGE REPLACEMENT
HIGHWAY 23, SITE 25-224, GWP 313-94-00
AGREEMENT NUMBER 3005-A-000078

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by TSH Engineers Architects Planners (TSH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the sites of three proposed Highway 23 bridge replacements over Fish Creek near Kirkton, Ontario. This report addresses the north bridge (Site 25-224). MTO has assigned Geocres No. 40P6-18 to this site.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed replacement bridge, approaches and detour bridge by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. The terms of reference for the scope of work are outlined in our Total Project Management proposal P01-1047, dated February 2000. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, Agreement No. 3005-A-000078, dated March 2000.

TSH provided Golder with preliminary drawings for the three Highway 23 crossings of Fish Creek. The centreline and stations of the proposed alignments were surveyed by others prior to commencing the foundation field investigation program. The General Arrangement plans showing the proposed abutment layout of the replacement structures and detours had been provided to us in digital format on December 12, 2000.

2.0 SITE DESCRIPTION

The project area covered by this report extends along Highway 23 at the crossing with Fish Creek approximately 1.4 kilometres north of Kirkton, Ontario (see Figure 1). The highway runs approximately northeast-southwest and the creek flows in a southerly direction towards the North Branch of the Thames River.

The existing bridge deck is at elevation 305.1 metres, the water level in the creek is currently at about elevation 302.1 metres and the creek bed is at about elevation 301.1 metres.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between November 10 and 29, 2000. At that time nine boreholes were put down at the site of the proposed replacement bridge and detour. The boreholes were drilled and sampled to depths ranging from about 3.5 to 16.5 metres. The borehole locations are shown in plan on Drawing 1.

The investigation was carried out using truck and all terrain vehicle mounted CME 75 and CME 750 drill rigs supplied and operated by Lantech Drilling Services Inc. The boreholes were advanced using 208 millimetre outside diameter continuous flight hollow stem augers. In the boreholes, samples of the overburden were obtained at regular intervals of depth using 50 millimetre outside diameter split spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. In-situ vane shear tests and cone penetration tests were also carried out in select boreholes. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Piezometers were installed in four boreholes to permit monitoring of the groundwater levels at the site. All of the boreholes were backfilled using MTO recommended procedures. Water levels in the installations were obtained on November 29, December 13 and December 20, 2000 to determine stabilized levels.

The field work was supervised on a full-time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported back to our laboratory in London, Ontario for further examination. Index and classification tests consisting of grain size analyses, Atterberg limits tests and water content determinations were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendix A. Site photographs taken during the drilling operations are provided in Appendix B.

The as drilled borehole locations and elevations were surveyed by Golder staff using the coordinate system and benchmarks on the drawings provided by TSH. The elevations at the borehole locations were referenced to a cut cross on the south end of the bridge at Station 18+587.1, 6.0 metres right. The benchmark is numbered 310 and is understood to have an elevation of 305.032 metres, referred to geodetic datum.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site lies within the physiographic region of Southwestern Ontario known as the Stratford Till Plain, specifically an area known as the Lucan Moraine. The soil conditions in the area generally consist of stoney and bouldery clayey silt till. The region is underlain by limestone of the Dundee formation. The rock surface is typically found at depths of about 14 to 21 metres below the ground surface.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and Appendix A following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site generally consist of variable thicknesses of fill, topsoil and organic materials to about elevations 300.3 to 302.6 metres underlain by 4.3 to 7.3 metres of silt and/or clayey silt and 6.7 to 9.6 metres of sand and gravel. These layers were underlain by sandy silt till materials starting between elevation 295.4 and 298.0 metres.

Locations and elevations of the borings, together with the interpreted stratigraphical profile and sections, are shown on the attached Drawings 1 and 2. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2.1 Topsoil and Fill

Topsoil and/or fill layers were encountered in all of the boreholes. Topsoil layers 0.2 to 0.4 metres thick were encountered at ground surface in boreholes 4, 7 and 8 and beneath layers of fill in boreholes 2, 3, 5, 6 and 9. Standard penetration tests in the topsoil indicated N values of 2 to 5 blows per 0.3 metres penetration and water contents between about 20 and 52 per cent.

Layers of sand and gravel fill underlain by generally clayey silt fill were encountered at ground surface or beneath surficial topsoil layers in boreholes 1 to 6 and 9. The granular fill in boreholes 1, 2, 3, 5 and 6 is associated with the granular shoulders for Highway 23. Cobbles and/or boulders were noted in the fill layers in boreholes 1, 3, 5, 6 and 9. The fill layers were 2.0 to 4.0 metres thick and had N values between 3 and 51 blows per 0.3 metres of penetration and water contents between 11 and 27 per cent.

4.2.2 Organic Deposits

The top portion of the silt deposits between elevation 300.8 and 302.1 metres in boreholes 2, 3, 4, 7 and 8 contained varying amounts of organic materials. In borehole 2, a 0.8 metre thick layer of organic silt was encountered. The organic silt had an N value of 3 blows per 0.3 metres penetration and a water content of about 92 per cent. In borehole 7, a 0.2 metre thick layer of peat was encountered. The peat had an N value of 2 blows per 0.3 metre penetration and a water content of about 57 per cent.

4.2.3 Layered Silt and Clayey Silt

Beneath the topsoil and organic deposits in boreholes 2, 3, 5 to 8 and 9 and the fill in borehole 4, layers of silt and clayey silt 1.1 to 3.9 metres thick were encountered. The deposit contains frequent interlayers of silt and clayey silt. Boreholes 6 and 7 were terminated in the deposit at elevations of 301.9 and 296.5 metres after exploring it for 0.8 and 3.5 metres, respectively. Cone penetration testing was carried out in borehole 7. The deposit had standard penetration test N values of 2 to 10 blows per 0.3 metres penetration and cone penetration values of 1 to 39 blows per 0.3 metres of penetration at the bottom of borehole 7 indicating a very loose to loose density in the silt and very soft to firm density in the clayey silt. Two in-situ vane shear tests carried out in borehole 8 indicated undrained shear strengths of 67 and 72 kilopascals, indicating a stiff consistency of the clayey silt layers.

Figure A-1 in Appendix A shows the gradation curves for two samples of the deposit from boreholes 7 and 8. Measured water contents of the samples from this deposit were between about 21 and 38 per cent, with an average of about 26 per cent. The average plastic and liquid limits for two samples of the clayey silt interlayers were 21 and 28 per cent, respectively, with an average plasticity index of 7 per cent. The Atterberg limits test results are summarized on a Plasticity Chart, Figure A-4.

4.2.4 Sand and Gravel

In Boreholes 2 to 5, 8 and 9, a 0.3 to 2.7 metres thick layer of wet sand and gravel was encountered between the silt and till deposits at about elevation 295.4 to 299.2 metres. The sand and gravel layers had standard penetration test N values of 14 to 56 blows per 0.3 metres penetration indicating a compact to very dense density. Water contents were between about 10 and 14 per cent

4.2.5 Sandy Silt Till

Boreholes 2 to 5, 8 and 9 encountered brown and grey sandy silt till and/or silty sand till, trace to some clay, below elevation 295.4 to 298.0 metres and were terminated in the till stratum at elevations between 288.4 and 296.5 metres after penetrating it for some 1.6 to 9.5 metres. Boreholes 3 and 9 were terminated after refusal to further auger penetration within the till at elevations of 288.4 and 296.1 metres, respectively. The till deposit had varying amounts of cobbles and boulders and had standard penetration test N values of 26 blows per 0.3 metres penetration to 100 blows per 75 millimetres penetration indicating a generally dense to very dense density. Figures A-2 and A-3 in Appendix A show gradation curves for samples of the sandy silt till and silty sand till deposits, respectively.

The water contents of selected till samples were between about 6 and 11 per cent. The average plastic and liquid limits for the sandy silt till, based on 2 samples tested, are 12 and 17 per cent, respectively, with an average plasticity index of 5 per cent. A single Atterberg limit determination for a sample of the silty sand till from borehole 8 indicated that the material was non-plastic. The Atterberg limits of the till deposit are summarized on a Plasticity Chart, Figure A-4.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operations. These levels are shown on the attached Record of Borehole sheets. A standpipe was installed in borehole 3 and piezometers were sealed in boreholes 3, 4 and 9 to permit the monitoring of the groundwater levels at the site. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets.

Water was noted during drilling in all of the open boreholes, except Boreholes 1 and 6, at elevations between 291.2 and 301.2 metres. The encountered water levels and those measured in the standpipe and piezometers on November 29, 2000 and December 13 and 20, 2000 are summarized in the following table. On the latter dates the water levels were at elevations 301.2 to 302.1. It should be noted that the groundwater level is subject to seasonal fluctuations.

Borehole Number	Installation	Ground Surface Elevation (m)	Groundwater Level Encountered During Drilling		Water Levels in Piezometers			
			Depth (m)	Elevation (m)	November 29, 2000		December 13/20, 2000	
					Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
1		304.67	Dry					
2		304.73	6.10	298.63				
3	Standpipe Piezometer	304.90	7.01	297.89	2.90 3.35	302.00 301.55	3.20 3.72	301.70 301.18
4	Piezometer	303.69	5.49	298.20	3.81	299.88	1.55	302.14
5		305.25	7.32	297.93				
6		305.36	Dry					
7		302.56	1.37	301.19				
8		302.45	11.27	291.18				
9	Piezometer	304.53	5.33	299.20	2.65	301.88	2.99	301.54

The measured water level in the creek was at elevation 302.10 metres on December 13, 2000.

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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the foundation aspects of design of the proposed Fish Creek (North) Bridge 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 method and scheduling.

It is understood that the existing bridge will be replaced with a new, slightly longer and wider structure and that the new approaches will tie in with the existing pavements. Highway 23 will remain at the current grade, and the new bridge structure centreline will be at the same location as the existing bridge. The detour bridge will consist of a two lane Bailey Bridge constructed on the upstream side of the replacement bridge.

5.2 Replacement Bridge Foundations

The subsoils encountered in the boreholes put down during the present investigation typically consist of surficial road fills, topsoil and organics over loose silt and soft silty clay. These deposits were in turn underlain by compact to very dense sand and gravel deposits over dense to very dense sandy silt till and/or silty sand till below elevation 295.7 to 298.0 metres. The fill layers and till deposit contain varying amounts of cobbles and boulders. Boreholes 3 and 9 were terminated on boulders in the till during the drilling operation. The water table was measured between about elevations 301.2 and 302.1 metres and the creek water level was at elevation 302.1 metres in the immediate vicinity of the proposed structure.

Based on the subsurface information above, the shallow subsoils (fills, loose silt and soft clay) are not considered suitable for shallow spread footings for support of the bridge. Consideration may be given to supporting the replacement structure on steel piles driven to practical refusal within the dense to very dense till deposit. It is understood that the base of the pile cap will be at elevation 301 metres. For the minimum structural pile length of 5 metres, the pile tips would

have to be at or below elevation 296 metres which will penetrate the sand and gravel deposit and extend up to 2 metres into the underlying till.

5.2.1 Geotechnical Axial Resistance – Driven Steel H-Piles

It is understood that consideration is being given to designing the replacement structure with integral abutments. Driven steel H-piles are considered suitable for the abutment support. For design, the factored axial resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to practical refusal in the till may be taken at 1,400 kilonewton (kN). The axial resistance at Serviceability Limit States (SLS) for 25 millimetres of settlement may be taken at 900 kN.

The till surface is variable at the site and ranges between about elevations 295.7 and 298.0 metres. Based on the preliminary General Arrangement drawings, the base of the pile cap will be at about elevation 301 metres. In order to achieve the minimum structural pile length of 5 metres required for integral abutments, the piles will have to be driven to about elevation 296 metres, up to 2 metres into the till founding stratum. Given the variability of the till deposit, it is anticipated that some of the piles will have to be driven deeper before achieving the required set, such as in the vicinity of borehole 3 where piles may extend to about elevation 290 metres. As noted below, however, some of the piles may "hang up" within the sand and gravel/till deposits and the design and contract should be flexible to permit relocation and/or re-driving of piles, if required.

Some of the SPT 'N' values measured within the till through which the piles are to be driven are greater than 100 blows per 0.3 metres of penetration. Driving of the piles could be difficult given the denseness of the subsoils and the presence of cobbles and boulders in the till. Heavy driving should be anticipated and the piles may "hang up" on boulders within the till. Stiffening of the pile tip with cast steel driving shoes will be required for protection during driving. Flange plate stiffeners are not considered suitable at this site.

The H-piles should be driven to at least elevation 296 metres and to a final set of no less than 15 blows per 25 millimetres of penetration, or practical refusal, using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01.

The appropriate notes which should be shown on the General Arrangement drawing are:

"Piles to be driven to at least the pile tip elevations as shown."

"Piles to be fitted with cast steel driving shoes and driven in accordance with OPSS 903.07.02.05."

5.2.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. In this case, the horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

k_s	=	coefficient of horizontal subgrade reaction (MPa/m) = $n_h (z/d)$
d	=	pile width or diameter (m)
n_h	=	constant of horizontal subgrade reaction (MPa/m)
z	=	depth below adjacent road grade (m)

Above elevation 298 metres	Very loose to loose silt	n_h varies with depth $n_h = 1$ to 3 MPa/m
Below elevation 298 metres	Dense sand and gravel, dense to very dense sandy silt till	n_h varies with depth $n_h = 10$ to 15 MPa/m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of d Loading, d</i>	<i>Subgrade Reaction</i>
--	--------------------------

<i>= Pile Diameter</i>	<i>Reduction Factor, R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.2.3 Frost Protection

The pile caps should be provided with a minimum of 1.2 metres soil cover for frost protection.

5.3 Detour Bridge Foundations

Based on the design elevations for the detour bridge and the thicknesses of loose silt and soft clayey silt materials, excavations of these materials is not considered feasible. The detour bridge may be erected as shown on the preliminary General Arrangement drawings on a minimum 0.6 metre thick Granular A pad constructed within the existing fill embankment on the near surface soils.

The topsoil should be removed from below the Granular A pad where present at ground surface. As expected at the north abutment a founding level for the granular pad of elevation 302.2 metres should be assumed for design. For the existing and proposed grades, the base of the Granular A pad will be at about elevation 301.9 metres and will be within the existing fill materials at the south abutment. The exposed subgrade should be proofrolled prior to placing the granular fill.

It should be noted that there will be settlement of the abutments supported on the Granular A pad due to consolidation of the founding materials. This settlement will be differential due to the variability of the founding soils. The settlement of the bridge abutments will be governed mainly by the loading induced by placement of the embankment fill material. There will not be fill placement required at the south abutment. About 3 metres of fill placement is required at the north abutment. Under these conditions, total and differential settlement of the abutments is not expected to exceed 50 millimetres. It is understood that the settlement of this temporary structure can be tolerated and will be accommodated by jacking the abutments as needed.

The granular pad should be compacted in lifts of loose thickness not greater than 200 millimetres to 98 per cent of the material's Standard Proctor maximum dry density. A factored geotechnical resistance at ULS of 150 kilopascals and a geotechnical resistance at SLS of 100 kilopascals may be used for design. It should be noted that the geotechnical resistance at SLS given above is for

50 millimetres of settlement and is based on the conditions expected at the north abutment. It is anticipated that the settlement of the south abutment under the above loading will be less than 25 millimetres.

5.4 Lateral Earth Pressures

The lateral pressures acting on the replacement bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments and the retaining walls in accordance with Ontario Highways Bridge Design Code (OHBDC):

- Select, free draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 millimetres to 95 per cent of the material's standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- The granular fill may be placed either in a zone with width equal to at least 1.2 metres behind the back of the stem (Case I) or within the wedge shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at rest pressures should be assumed for geotechnical design.
- A compaction surcharge equal to 16 kilopascals (kPa) should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.
- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight 20 kN/m³
(assuming clean earth fill)

Coefficients of lateral earth pressure:
 'active' 0.43
 'at rest' 0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<i>Granular A</i>	<i>Granular B Type II</i>
Soil Unit Weight	22 kN/m ³	21 kN/m ³
Coefficients of Lateral Earth Pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD)-3501.00.

5.5 Embankments

It is understood that the proposed replacement bridge will require minimal changes in alignment and grade and the existing embankments will generally remain unchanged. Temporary approach embankments will be required for the detour bridge. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site. A Factor of Safety against deep seated failure of greater than 1.3 is obtained for the embankments which will generally be less than 2.5 metres in height.

The topsoil and organic materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled prior to fill placement.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or select subgrade material (in accordance with OPSS 1010) depending on material availability. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 millimetres and be compacted to at least 95 per cent of the material's standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the standard Proctor maximum dry density.

5.6 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will extend through fill materials, organics and silt deposits. At the proposed replacement bridge location, the excavations for the pile caps at the abutments will be up to about 4 metres in depth below Highway 23. The base of the excavations will be about 1 metre below the groundwater level as measured in the standpipe and up to about 0.5 metres below the water level measured in the piezometer sealed in the sand and gravel deposit at borehole 3.

The groundwater level is expected to fluctuate and the depth of excavation below the groundwater will depend on the time of year of construction. Temporary open cut slopes within the fill above the groundwater level should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Temporary open cut slopes within the silt deposits below the groundwater level should be maintained no steeper than 1.5H:1V.

Based on the above noted piezometric level measured in the sand and gravel deposit and the proposed base of pile cap excavation, there will be about 3 metres of the silt deposit below the base of the excavation to resist about 1 metre of piezometric pressure.

This thickness of silt is considered adequate to prevent heave of the excavation base provided that there is no disturbance of the silt deposit during excavation and construction of the pile caps. There will, however, be upward flow of water along the piles where they penetrate the sand and gravel deposits. Installation of the piles themselves will cause disturbance of the silt deposit.

Therefore, depressurization of the sand and gravel layer will be required prior to installation of piles. Depressurization can be achieved by pumping from a series of deep wells at each abutment location. The design and installation of the dewatering system should be carried out by a qualified dewatering contractor. The water level within the sand and gravel must be lowered to below the base of the pile cap excavation prior to driving piles.

In addition, the excavations will be adjacent to the creek and diversion/control of the creek water will be required. Consideration could be given to the use of closed steel sheet piling driven to below the base of the pile cap excavation. It is likely not possible to drive the sheet piling through the sand and gravel and to achieve a cutoff within the underlying till deposits. It will be necessary, therefore, to depressurize the sand and gravel using deep wells as described above.

Pumping from well filtered sumps located at the base of the excavations may also be required to provide adequate groundwater control during foundation excavations. The silt deposit is sensitive to disturbance and softening due to water seepage and/or ponding and placement of a mud coat will be required at the base of the pile cap excavation. Sumps should be maintained outside the actual pile cap/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.

Where space is restricted and will not permit open cuts for footing/pile cap construction, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. Depressurization of the sand and gravel for pile cap construction must be carried out as discussed above for excavations made within soldier pile and lagging walls. The temporary support system could consist of soldier piles and lagging where the piles would

be socketed into pre-augered holes extended into the till below the excavation base. Cobbles and boulders should be expected during augering for the soldier pile installation. Support to the soldier pile and lagging wall system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection excavations.

The design of braced soldier pile and lagging walls should be based on a 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.

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

$$p = K_a \gamma H$$

where

$$H = \text{the height of the excavation at any point in metres}$$

$$K_a = 0.3 \text{ for level ground behind excavation}$$

$$\gamma = \text{soil unit weight} = 20 \text{ kN/m}^3$$

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

$$p = K \gamma H$$

where

$$H = \text{the height of the excavation}$$

$$K = 0.3 \text{ for level ground behind excavation}$$

$$\gamma = \text{soil unit weight} = 20 \text{ kN/m}^3$$

Passive tip 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 coefficient of passive lateral earth pressure, K_p , for the socket within the very dense till may be taken as 8.7. The soil unit weight should be taken as 22 kN/m^3 and the unit weight of water should be taken as 9.8 kN/m^3 . A groundwater level at elevation 302.1 metres can be assumed at the bridge footing locations.

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, organics and silt at this site would be classified as Type 3 soils and the clayey silt deposits as Type 2.

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PART B – FOUNDATION DESIGN REPORT

**FISH CREEK (NORTH) BRIDGE REPLACEMENT
HIGHWAY 23, SITE 25-224, GWP 313-94-00
AGREEMENT NUMBER 3005-A-000078**

APPENDIX A

LABORATORY TEST DATA (FIGURES A-1 TO A-4)

A P P E N D I X B

S I T E P H O T O G R A P H S