REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
FISH CREEK (CENTRE) BRIDGE REPLACEMENT
HIGHWAY 23, SITE 25-225, GWP 313-94-00
AGREEMENT NUMBER 3005-A-000078

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PART A –FOUNDATION INVESTIGATION REPORT

FISH CREEK (CENTRE) BRIDGE REPLACEMENT

HIGHWAY 23, SITE 25-225, 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 centre bridge (Site 25-225). MTO has assigned Geocres No. 40P6-19 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. Golder Report No. 931-3077 entitled "Geotechnical Investigation, Proposed Highway 23 and Fish Creek Crossings, Duct Bank Installation, Blanshard Municipal Telephone System, Woodham to Kirkton, Ontario", dated June 7, 1993 has been referenced during the preparation of this report. The relevant borehole records from that report are provided in Appendix A.

2.0 SITE DESCRIPTION

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

The existing bridge deck is at elevation 302.6 metres, the water level in the creek is currently at about elevation 299.9 metres and the creek bed is at about elevation 299.5 metres.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between November 9 and 24, 2000. At that time seven 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 2.0 to 9.8 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 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. 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 B. Site photographs taken during the drilling operations are provided in Appendix C.

The as-drilled borehole locations and elevations were surveyed by Golder staff using the co-ordinate system and benchmarks on the drawings provided by TSH. The elevations at the borehole locations were referenced to a tablet set on the east face of the south bridge abutment at Station 18+107.8, 6.4 metres right. The benchmark is numbered 306 (MTO Benchmark 848076) and is understood to have an elevation of 302.289 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 12 to 27 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 following the text of this report and in Appendix B. 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 a variable thicknesses of topsoil and fill materials to about elevations 298.9 to 301.1 metres underlain by clayey silt till and sandy silt till materials. Sand and gravel layers were encountered above and below the till layers at two borehole locations.

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 0.1 to 0.2 metres thick were encountered at ground surface in boreholes 2, 3, 6 and 7.

Layers of sand and gravel fill underlain by clayey silt fill were encountered under 50 millimetres of asphalt in borehole 1, at ground surface in boreholes 4 and 5 and beneath the topsoil in boreholes 2, 3, 6 and 7. Cobbles and boulders were noted in the fill layers in boreholes 3, 6 and 7. Auger refusal was met within the fill in borehole 3 on a probable boulder and the borehole had to be advanced at a second adjacent location in order to penetrate the fill. The fill layers were 1.4 to 3.7 metres thick and had N values between 6 and 24 blows per 0.3 metres of penetration and water contents between about 15 and 25 per cent.

4.2.2 Sand and Gravel

Beneath the fill in borehole 2, a layer of sand and gravel 0.3 metres thick was encountered at elevation 299.8 metres. Borehole 3 was terminated in a sand and gravel deposit beneath the sandy silt till at elevation 292.64 metres after exploring it for 1.2 metres. The sand and gravel layer in borehole 3 had a standard penetration test N value of 111 blows per 0.3 metres penetration and a water content of about 10 per cent, based on the single sample collected.

4.2.3 Clayey Silt Till

Beneath the fill in borehole 4, a 0.9 metre thick layer of clayey silt till was encountered at elevation 299.7 metres. The clayey silt till had a standard penetration test N value of 36 blows per 0.3 metres penetration and a water content of about 20 per cent, based on the single sample collected.

4.2.4 Sandy Silt to Silty Sand Till

All of the boreholes encountered brown and grey sandy silt to silty sand till below elevation 298.9 to 301.1 metres and all of the boreholes, except borehole 3, were terminated in the till stratum at elevations between 297.7 and 300.4 metres after penetrating it for some 0.6 to 6.0 metres. Borehole 3 encountered 5.3 metres of sandy silt till and was terminated in a sand and gravel deposit. Boreholes 2 and 4 to 7 were terminated after refusal to further auger penetration within the till. The till deposit contained cobbles and boulders. In boreholes 2, 4 and 7, auger refusal on probable boulders was met at relatively shallow depths into the till and the boreholes had to be put down at a second and third adjacent location in order to penetrate the till to a sufficient depth. Standard penetration test N values of 14 blows per 0.3 metres penetration to 100 blows per 50 millimetres penetration were measured within the till, with an average N value in the order of 45 blows per 0.3 metres penetration. Based on these N values, the density of the till is considered to be compact to very dense. Figure B-1 in Appendix B shows gradation curves for three samples recovered from the till deposit.

The water contents of selected sandy silt till samples were between about 6 and 12 per cent, with an average of about 9 per cent. The average plastic and liquid limits for the sandy silt till, based on 2 of 3 samples tested, are 12 and 15 per cent, respectively, with an average plasticity index of 3 per cent. One of the samples tested (sample 5 from borehole 4) was non-plastic.

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 2 and piezometers were sealed in boreholes 2, 3 and 6 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 5 and 6, at elevations between 293.8 and 299.8 metres. The encountered water levels and those measured in the piezometers on November 29, December 13 and December 20, 2000 are summarized in the following table. 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*			
					November 29, 2000		*December 13/20, 2000*	
			Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
1		302.72	3.81	298.91				
2	Standpipe Piezometer	302.52	2.74	299.78	1.98 2.16	300.54 300.36	2.77 2.13	299.75 300.39
3	Piezometer	302.58	8.79	293.79	3.09	299.49	2.97	299.61
4		302.47	2.74	299.73				
5		302.42	Dry					
6	Piezometer	301.93	Dry		6.92	295.01	3.66	298.27
7		302.33	3.81	298.52				

The measured water level in the creek was at elevation 299.9 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 (Centre) 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 methods and scheduling.

It is understood that the existing bridge will be replaced with a new, somewhat 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 downstream 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 topsoil and road fills over hard clayey silt till and/or compact to very dense sandy silt till below elevation 298.7 to 301.1 metres. The fill layers and till deposit contain cobbles and boulders. Boulders were encountered in the fill in boreholes 3, 6 and 7, and boreholes 2 to 7 were terminated on boulders in the till during drilling operations. The water table was measured between about elevations 298.3 and 300.4 metres and the creek water level was at elevation 299.9 metres in the immediate vicinity of the proposed structure.

Based on the subsurface information above, consideration may be given to supporting the replacement structure on spread footings bearing on the sandy silt till materials at or below about elevation 299 metres. The use of deep foundations has not been considered due to the presence of competent founding soils at relatively shallow depth.

5.2.1 Geotechnical Resistance

Based on the subsoil conditions encountered, the highest founding level for spread footings founded 0.5 metres into the sandy silt till would be about elevation 299 metres for the south abutment and elevation 298.5 metres for the north abutment. The design values are given in the table below.

Founding Option	***Factored Geotechnical Resistance***	***Geotechnical Resistance***
	ULS	***SLS***
Spread Footings on Sandy Silt Till	500 kPa	350 kPa

These founding levels for the footings on the till are up to about 4 metres below the existing road surface and up to 2 metres below the groundwater level. The above geotechnical resistances assume that appropriate construction procedures are adopted during footing construction to ensure that the till is not softened/disturbed prior to concrete placement.

5.2.2 Resistance to Lateral Forces

Resistance to lateral forces/sliding resistance between the concrete spread footings and subsoil should be calculated in accordance with Section 6-8.4.3 of the Ontario Highway Bridge Design Code (OHBDC). Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angle of friction between the concrete and the founding soils and corresponding coefficient of friction, tan δ may be used:

Footings on sandy silt till	angle of friction	29°
	tan δ	0.55

5.2.3 Frost Protection

All footings should be provided with a minimum of 1.2 metres of earth cover for frost protection purposes.

5.2.4 Construction Considerations

The excavations for the abutment footings will be up to about 2 metres below the groundwater level. Seepage through the clayey silt fill and the sandy silt till deposit is expected to be minimal due to the generally fine grained nature of the materials. There will be some seepage, however, and there is potential for softening of the founding soils due to upward seepage. Significant quantities of water inflow into footing excavations should be expected where the sand and gravel deposits, such as encountered in borehole 2, are intercepted and where sandier zones within the till are encountered. Pumping from well filtered sumps placed at the base of the excavation should generally provide sufficient groundwater control during foundation excavations except where the sand and gravel deposits are encountered. Sumps should be maintained outside of the footing area. It should be noted that the water levels could be higher during wet periods of the year. Where the sand and gravel deposits are intercepted in the footing excavations, groundwater control by pumping from deep wells screened through the water bearing deposits will be required. In addition, the excavations will be adjacent to the creek and therefore diversion/control of the creek water will be required.

The founding soils are sensitive to disturbance and softening due to water seepage and/or ponding. Placement of a mud coat will be required at the base of excavation for the footing area. Exposure without protection of the mud coat will result in softening of the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and that the mud coat be placed immediately after footing inspection.

5.3 Detour Bridge Foundations

The subsoils encountered at the location of the proposed detour bridge are considered suitable to support shallow spread footings. The detour bridge may be erected as shown on the preliminary General Arrangement drawings on a Granular A pad constructed on the surface of the underlying till materials after removal of the existing fill materials. The Granular A pad should be at least 0.6 metres thick. Alternatively, if some total and differential settlements can be tolerated for this temporary structure, consideration could be given to placing the Granular A pad within the fill materials where present without subexcavating to the surface of the till. Based on the proposed grades, the base of the Granular A pad would be about elevations 299.5 metres to 299.7 metres which would generally be within the underlying till deposit so this option may not be required. All topsoil and organic deposits should be removed from within the limits of the Granular A pad where present outside the area of existing embankment fill. 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.

In the case of the abutment footings placed on a Granular A pad placed on the till deposits, a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kilopascals and a geotechnical resistance at Serviceability Limit States (SLS) of 350 kilopascals may be used for design. Total and differential settlement will be less than 25 millimetres. The surface of the till may be taken at elevation 299.6 metres at the north abutment and at elevation 300 metres at the south abutment.

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 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 0.075 millimetre 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 (assuming clean earth fill)	20 kN/m^3
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:

	Granular A	***Granular B Type II***
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 sub-drains 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 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 under the direction of the geotechnical engineer 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 footing construction will extend through fill materials and into the till deposits. At the proposed replacement bridge location, the excavations for the spread footings at the abutments will be up to about 4 metres in depth below Highway 23. Cobbles and boulders were noted in the fill and overburden materials and should be expected during excavation. The base of the excavations will be up to about 2 metres below the groundwater level as measured in the piezometers. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within a fully braced excavation. The excavation for spread footings or pile cap construction may be made with supported vertical sides to minimize the size of the excavation and disruption to traffic.

Water seepage into the excavations should be expected through the sand and gravel deposits where intersected and will be heavier during periods of sustained precipitation. The considerations with respect to groundwater control and protection of the founding soils, as given in Section 5.2.4, must be recognized. 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 construction, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. 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 = the height of the excavation at any point in metres
- K_a = 0.3 for level ground behind excavation
- γ = soil unit weight = 20 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 = the height of the excavation
- K = 0.3 for level ground behind excavation
- γ = soil unit weight = 20 kN/m^3

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 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 299.7 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 and granular deposits at this site would be classified as Type 3 soils and the till deposits as Type 1.

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PART B – FOUNDATION DESIGN REPORT

FISH CREEK (CENTRE) BRIDGE REPLACEMENT

HIGHWAY 23, SITE 25-225, GWP 313-94-00

AGREEMENT NUMBER 3005-A-000078

A P P E N D I X   A

RECORD OF PREVIOUS BOREHOLES

A P P E N D I X   B

LABORATORY TEST DATA (FIGURE B-1)

A P P E N D I X   C

SITE PHOTOGRAPHS