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FOREWORD

This report presents the results of a geotechnical foundation investigation that was completed by Terraprobe for the proposed replacement of the Liffy Drain Bridge structure located along Highway 8 in Dublin, Ontario as shown on the attached Figure 1.

In accordance with MTO standards for this type of project, this report has been prepared in two distinct sections,

Part A - Foundation Investigation Report, and
Part B - Foundation Design Report.

Part A addresses only the factual data aspects of the project whereas Part B addresses the design aspects of the project.

PART A - FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION

Terraprobe Limited was retained by Stantec Consulting Limited to conduct a geotechnical foundation investigation and design report for the replacement of the Liffy Drain Bridge in Dublin, Ontario. The work was completed in accordance with our proposal, dated June 30, 2000, and the MTO Request for Proposal (RFP).

The purpose of the investigation was to assess the existing subsurface conditions in the area of the Liffy Drain Bridge in order to provide geotechnical engineering recommendations for design of the proposed replacement structure. Comments are also provided on the anticipated construction conditions.

The existing structure is a single span, steel beam bridge over Silver Creek. It is understood that the original construction occurred in 1923 and the bridge was repaired in 1959. The original construction consisted of shallow foundations on “gravel and hard clay” approximately 0.8 m below the stream bed (MTO Drawing No. 1155, July 1923), with steel sheet piling driven along the footings in 1959 for scour protection. The span of the bridge is approximately 12 m and the structure is about 12.3 m in width. The original construction drawings also indicate the following elevations;

	Elevation, m.
Bridge Deck Top	335.09
Creek Water Level	333.03
Creek Bed	332.52

2. SITE DESCRIPTION

Liffy Drain Bridge is located along Highway 8 in Dublin, Ontario, as indicated on the Site Location Plan, Figure 1.

The bridge is located in an urban setting with developed surrounding land. A gas station with a restaurant is located to the south west; a general store is located to the north west; a farm house with agricultural land is located to the north; and residential properties are located to the east.

The local topography is gently rolling to flat. The vegetation generally consists of manicured grass on the developed lands. Along the stream (Silver Creek) which passes beneath the bridge there is tall grass, shrubs and about a half dozen trees to the north side of the bridge. There is also a drainage ditch to the north west of the bridge which has tall bulrushes along it. Other drainage features in the vicinity of the bridge include a culvert which outlets to the southeast of the bridge. The stream itself flows south-west and at the time of the site visit had a low flow rate.

Based on the mapped geology and well records for this area, the soil conditions are recent alluvial deposits over glaciolacustrine deposits which in turn overlie silty glacial till. Bedrock in this area is Middle Devonian limestone, dolostone or shale, located at depths of about 40 to 50 m.

3. INVESTIGATION PROCEDURES

The field investigation for the project was conducted on September 8 and 11, 2000 when, two (2) exploratory boreholes were drilled at the abutment foundation locations to depths of 13.9 and 15.6 metres, and two (2) exploratory boreholes were drilled along the approach embankments to depths of about 3.5 metres. The borehole locations are shown on the attached Figure 2.

The borings were drilled using a truck mounted CME 75 power auger. The borings were advanced using 100 mm diameter solid stem augers. During auger drilling, Standard Penetration Tests (SPT) and associated split spoon soil sampling were completed at regular intervals of depth (0.76 m to 3 m and 1.5 m thereafter) (ASTM D 1586).

The field work was supervised throughout by a member of our technical staff, who directed the drilling and sampling operations, and transported the samples to our laboratory. The samples were stored in plastic containers and transported to the laboratory for detailed examination and testing. All

of the borehole samples were examined (tactile) in detail by the project engineer, and classified according to visual and index properties.

Ground water observations were made in the boreholes as drilling proceeded. Additionally, standpipe type piezometers were sealed into Boreholes 1 and 3, in order to permit long-term observation of ground water levels. The standpipes consist of 12 mm I.D. CPVC tubing, with a 1.5 m screen section near the base, and fitted with a sand filter, bentonite seal and grout, as shown on the accompanying Borehole Logs. The remaining Boreholes 2 and 4 were grouted to just below road surface, then hole plug and asphalt cold patch were added.

The locations of the borings were determined relative to on site features. The borehole elevations were determined relative to a Geodetic benchmark (Dublin 78U355, Elevation 335.205 m) from the “Geodetic Survey of Canada, 1990.”

Geotechnical laboratory testing consisted of water content determination on each sample, and a total of eight (8) grain size distribution analyses and six (6) Atterberg limit determinations on select samples. Laboratory testing also included soluble sulphate testing on a total of two (2) samples. The results of this testing are summarized on the attached Borehole logs, with complete grain size distribution curves included in the attached Appendix B.

These investigations have been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from these investigations.

4. SUBSURFACE CONDITIONS

The soil conditions encountered during the investigation are detailed on the attached Borehole Logs in Appendix 'A', and are summarized in more detail in the following sections. It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary at other locations. It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling.

In general, the encountered soil conditions consisted of fill material overlying undisturbed glacial till which increased in strength with depth. The glacial till generally has a stiff to very stiff consistency to a depth of about 10 m below existing grades, with a hard consistency to depths of 15 m or more. Exception to this general condition was a coarse to medium grained sand layer encountered at Elevation 326.6 within Borehole 1.

4.1 Pavement Structure

The four boreholes encountered a pavement structure at the ground surface consisting of an asphalt layer overlying a concrete layer overlying a granular layer. The asphalt layer extended to a depth of about 150 to 300 mm, the concrete layer extended to a depth of about 300 to 460 mm (thickness of 80 to 200 mm), and the granular layer extended to a depth of about 500 to 600 mm (thickness of about 50 to 200 mm).

4.2 Earth Fill

The pavement structure was underlain by earth fill material which consisted of intermixed clayey silt, sandy silt, a thin layer of organic silt and sand and gravel materials. Further stratigraphic details of the fill materials are provided on the appended Borehole Logs.

At the abutment boreholes (BH1 and 3) the earth fill material extended to depths of about 3.8 and 4.6 metres, or Elevation 331.3 and 330.5 metres. In Boreholes 2 and 4 well back from the abutments, the earth fill material extended to depths of about 2.7 and 2.1 metres, or Elevation 332.3 to 333.2 metres

SPT 'N' values within the cohesive earth fill materials varied from 3 to 13 blows per 0.3 m, indicating a soft to stiff consistency. SPT 'N' values within the cohesionless fill materials were 11 and 85 blows per 0.3 m, indicating a compact relative density and possibly some gravel, cobbles, or boulders. Natural water contents of the fill were measured from 8 to 29 percent by weight.

4.3 Glacial Till

Underlying the surficial pavement structure and earth fill materials, native glacial till was encountered within each of the four boreholes to the bottom of the sampled holes. The glacial till mainly consists of low plasticity clayey silt to silty clay with some sand and gravel, and occasional sandy silt zones. A sand and gravel layer was encountered within the glacial till, at depths of 8.5 to 10 m (Elev. 326.6 to 325.1 m).

The glacial till had a firm to very stiff consistency to a depth of about 8.5 and 9.7 m (Elevation 326.6 and 325.4 m) in Boreholes 1 and 3, respectively. Within Boreholes 2 and 4, the firm to very stiff glacial till extended to the maximum depth of the boreholes, 3.5 m (Elevation 331.5 and 331.8 m in Boreholes 2 and 4, respectively).

SPT 'N' values within the upper clayey silt to silty clay till materials (to 8.5 to 9.7 m depth) varied from 6 to 25 blows per 0.3 m, with an average of 15 blows per 0.3 m, indicating a firm to very stiff consistency. Natural water contents of this material were measured from 8 to 22 percent by weight.

In Boreholes 1 and 3, the glacial till had a hard consistency below depths of about 10 and 9.7 m (Elev. 325.1 and 325.4 m) respectively. This hard glacial till extended to at least the bottoms of Boreholes 1 and 3, at depths of about 15.6 and 13.9 m respectively (Elev. 319.5 and 321.2 m).

SPT 'N' values within the lower clayey silt to silty clay till materials (below 10 to 9.7 m depth) were greater than 100 blows per 0.3 m, indicating a hard consistency. Natural water contents of this material were measured at 6 and 7 percent by weight. This non-yielding layer was encountered within Boreholes 1 and 3 at a depth of 10.0 and 9.7 m, respectively (Elevation 325.1 and 324.4 m).

Embedded gravel was evident in the till samples, and cobbles and boulders are probably present but would not be representatively sampled with the equipment used for this investigation. Further details on the textural variation of these materials can be obtained by review of the grain size distribution data presented in the attached Appendix B and the Atterberg Limits results plotted on an “A-line” graph in the attached Appendix C. The following table summarizes the grain size distribution and Atterberg Limits data for the tested glacial till.

Borehole No.	Sample No.	Depth (m)	Grain Size Data			Atterberg Limit Data		UCS
			Gr	Sa	Si - Cl	WI	Ip	
1	6	6.1	10	33	55	15	5	CL-ML
1	10	12.2	10	24	66	17	6	CL-ML
2	4	3.0	11	19	70	20	8	CL
3	5	4.6	3	16	81	22	11	CL
3	7	7.6	2	9	90			CL
3	9	10.7	24	26	50	non-plastic		ML-SP
4	4	3.0	9	23	68	20	8	CL-ML

4.4 Sand Layer

Between the upper firm to very stiff glacial till and the lower hard glacial till of Borehole 1, a coarse to medium grained sand layer was encountered from about 8.5 to 10.0 m depth below ground surface (Elevation 326.6 to 325.1 m). Gravel was evident in the sample, and cobbles and boulders are probably present. Grain size distribution data for this sample is summarized below and presented in the attached Appendix B.

Borehole No.	Sample No.	Depth (m)	Grain Size Data			UCS
			Gr	Sa	Si - Cl	
1	8	9.1	20	69	11	SP-SM

The SPT ‘N’ value in the sand layer was 92 blows/ 15cm, indicating a very dense state. The measured natural water content of this material was 19 percent by weight.

4.5 Ground Water

During and upon completion of drilling, ground water conditions within the boreholes were noted. The shallow depth boreholes in the approach embankments (BH's 2 and 4), remained dry during drilling.

Boreholes 1 and 3 beside the bridge abutments and close to the watercourse, had ground water seepage during the drilling. The ground water seepage generally consisted of perched flow from the earth fill material found down to depths of about 3.8 and 4.6 m (Elev. 331.3 and 330.5 m). The creek water level has previously been indicated to be at about Elevation 333.03 m or about 2 m below ground surface at the boreholes. On completion of drilling the water level in the Boreholes 1 and 3 was at a depth of about 10.4 and 4.6 m below ground level, respectively (Elev. 324.7 and 330.5 m).

Standpipe piezometers were installed within Boreholes 1 and 3 to permit monitoring of long term ground water conditions. Details of the piezometer installations are provided on the appended Borehole Logs. Water levels within the piezometers were measured on October 5, some 3 to 4 weeks after installation as follows:

Borehole No.	Depth to water	Water Level Elevation
1	4.16 m	330.90 m
3	2.65 m	332.45 m

It is noted that the water level in the piezometers was about 0.5 to 2 m lower than the creek water level. It should be noted that ground water levels may vary seasonally.

4.3 Sulphate Analysis

Soluble sulphate testing was conducted on two (2) soil samples, as summarized in the following table and on the attached Borehole logs.

Borehole No.	Sample No.	Depth (m)	Sulphate (mg/kg)
1	9	10.7	154.1
3	8	9.1	159.4

PART B - FOUNDATION DESIGN REPORT

5. DISCUSSION AND RECOMMENDATIONS

The following discussions and recommendations are based on the factual data obtained from the investigation, and are presented for guidance of the design professionals only. The comments pertain to a specific project and location. If significant parts of the project change, they should be reviewed by Terraprobe to determine the effect of the changes on the recommendations.

Comments about construction are presented only to bring attention to aspects which might impact the design. Contractors bidding on or conducting work associated with this project should review the factual data presented in the preceding sections of the report, to assess their effect on proposed construction methods and scheduling.

5.1 Structure Foundations Design Recommendations

The proposed Liffy Drain Bridge replacement structure will consist of a single span supported on two contained abutments. The approaches on the east and west ends of the bridge are level with the relatively flat-lying surrounding land and road alignment. The span of the bridge is approximately 12 m and the structure is about 12.3 m in width.

The estimated depth of frost penetration at the site is 1.2 m. Foundation elements should be provided with this minimum depth of soil cover or equivalent exterior-grade insulation.

The subsurface soils at the site are favourable for support of the bridge using one of the following foundation systems.

- 1) End bearing Steel H Piles driven to practical refusal into the non-yielding glacial till below a depth of about 10 m (below Elev. 325.1 and 325.4 m) or
- 2) Conventional Shallow Spread Footings design on the underlying stiff native glacial till below Elevation 331.3 and 330.5 m, or

- 3) End bearing drilled caissons founded into the non-yielding glacial till below depths of about 10 m (below Elev. 325.1 and 325.4 m).

Selection of the most appropriate foundation system should be based on cost and site accessibility considerations. The most appropriate and economical foundation system is considered to be driven piles. Appropriate foundation design recommendations are presented in the following sections.

5.1.2 End Bearing Steel 'H' Piles

The bridge replacement structure can be founded on end bearing steel 'H' piles driven to practical refusal into the underlying hard glacial till below depths of about 10 m (below Elevation 325.1 m for the west abutment and Elevation 325.4 m for the east abutment). When founded in the "non-yielding" hard glacial till, the following typical steel HP sections, could be designed for the values in the following table:

	HP 310 x 110	HP 310 x 79
factored axial capacity at ULS	2,000 kN	1,450 kN
axial capacity at SLS	1,450 kN	1,050 kN

The above SLS values are based on an estimated settlement of about 10 mm.

It is required that pile driving be monitored and the "setting" of the piles be controlled by reference to the Hiley Dynamic Pile Driving Formula, in accordance with MTO Standards SS103-11, assuming:

	HP 310 x 110	HP 310 x 79
ultimate capacity of piles	3,800 kN	2,700 kN

During the driving of the piles there should be a quite dramatic increase in driving resistance expected when the tip of the pile encounters the underlying hard glacial till below depths of about 10 m or more. For the purposes of estimating pile driving lengths, extrapolation can be made between the available

borehole data. However, actual installed pile lengths and set must be determined in the field based on continuous monitoring of the piling operations.

Piles should be driven to a final set of 25 blows per 25 mm. In this regard, care should be taken in selecting the piled driving hammer so that driving stresses in the piles are limited to less than 90% of the yield strength.

It is noted that driving piles through the firm clay may cause heave or displacement of previously driven piles. Piles must be surveyed for position, elevation and plumbness, upon completion of driving and again following the driving of the adjacent piles. If significant heave occurs, re-driving of the piles may be necessary. There have also been instances in our experience where driving in the firm clay has created sufficiently high stresses in the clay mass as to impede the driving process. This type of effect is characterized by a general increase in driving resistance, and increased rebounding upon hammer strike. When this happens, the energy of the hammer is effectively absorbed by the adhesion of the stresses clay to the pile surface.

Since the glacial till deposits likely contain cobbles and boulders, it is recommended that the pile tips be reinforced with steel plates per MTO Standard DD-3301.

Based on the borehole information, the subgrade surrounding the piles should be considered to be stiff cohesive materials. Therefore, based on Table C6-9.8.1 (a) of the Ontario Highway Bridge Design Code for this material type, the factored lateral resistance is as follows for various pile types:

	HP 310 x 110	HP 310 x 79
horizontal passive resistance - ULS	160 kN	160 kN
horizontal passive resistance- SLS	80 kN	65 kN

Based on the encountered stratigraphy, downdrag or negative skin friction loads are not anticipated to be significant for this structure.

5.1.1 Spread Footing Foundations

Alternatively, design of conventional shallow spread footings may be completed in accordance with the following recommended values, provided they are formed at least 0.2 m into native glacial till and below the anticipated depth of frost penetration (minimum 1.2 m depth);

Factored bearing capacity at ultimate limit states (Q_u)	550 kPa
Bearing capacity at serviceability limit states (Q_s)	200 kPa.

Based on the available borehole information, the highest permissible footing elevations to permit the footing to be formed 0.2 m into the stiff native clayey silt till will be at or below elevation 331 m at the west abutment and, at or below elevation 330 m at the east abutment.

However, these anticipated founding elevations should be confirmed by appropriate footing inspection during construction. The base of all footings should be dry and free of all organic, deleterious or disturbed materials prior to placing concrete.

As noted above, exterior foundations should be provided with a minimum soil cover of 1.2 m or equivalent insulation, for frost protection purposes. If construction proceeds during freezing weather conditions, adequate temporary frost protection must be provided to the footing bases and concrete.

Anticipated settlements at the SLS condition should not exceed 25 mm and should occur within about 3 to 6 months after load application. Differential settlements between adjacent elements should not exceed 50% of this value.

The coefficient of friction, f , for assessing sliding resistance between cast-in-place concrete and the soil subgrade is recommended as 0.6 (unfactored).

5.1.3 End Bearing Drilled Caissons

Alternatively, large diameter caissons (greater than 0.9 m) may be designed to rest on the “non-yielding” glacial till below depths of about 10 m (Elev. 325). Based on the available borehole information, the highest permissible founding elevation will be about 324 m at the west abutment and 325 m at the east abutment.

For end bearing caissons founded within the “non-yielding” glacial till, the following design values are recommended;

Factored bearing capacity at ultimate limit states (Q_u)	3,400 kPa
Bearing capacity at serviceability limit states (Q_s)	1,000 kPa.

Settlements for the above SLS should not exceed 25 mm, with differential settlements limited to less than about 50% of this value. In our opinion, it is also the bearing resistance at SLS.

Caisson excavations are expected to be experience ground water seepage from the earth fill materials near the ground surface (to depths of 3.8 to 4.6 m), and from the dense sand layer at a depth of about 8.5 to 10 m. Wall instability problems can be expected. For this reason, and to permit downhole inspection and cleaning of the caisson base, the contractor should be instructed to have suitably sized temporary casing on site.

However, please note that all manned entry to the caissons to be in accordance with the latest edition of the Ontario Health and Safety Guidelines for Construction Projects. Any casing installed to maintain excavation stability should be removed during concreting in such a fashion that the level of concrete within the casing is no less than 0.5 m above the tip of the casing or, if water is present, at such a level that the pressure exerted by the concrete at the tip of the casing is at least 10% greater than the water pressure outside the casing.

5.2 Abutments

If integral abutments are considered, the anticipated horizontal movements should be compared with the allowable horizontal movements from the surrounding soils. If the surrounding soils do not allow for the anticipated movements, consideration should be given to the use of CSPs filled with loose sand to allow for the horizontal movement. The horizontal modulus of subgrade reaction is provided for each of the anticipated soil types as follows,

Strata	Horizontal Modulus of Subgrade Reaction
Earth Fill, soft to firm; to 3.8 to 4.6 m depths	1 MN/cu.m.
Silty clay to clayey silt, firm to very stiff; to 8.5 to 9.7 m depths	8 MN/cu.m.
Silty clay to clayey silt, hard; more than 10 to 9.7 m depths	20 MN/cu.m.

5.2.1 Abutment Backfill

Backfill to the abutments should be completed with free-draining granular fill such as OPSS Granular 'A' or 'B' compacted to achieve at least 95% of its Standard Proctor Maximum Dry Density (SPMDD). The granular backfill should be placed in a wedge-shaped zone extending from 1.2 m behind the base of the abutment and rising upward at an inclination of 1 horizontal to 1.5 vertical to the horizontal. The granular backfill should extend at least 0.6 m out from the structure, with a frost taper. The backfill should be drained by providing perforated or weep holes slightly above the creek level. The outlets should not be subject to freezing or flooding.

Heavy compaction equipment should not be used immediately behind the abutment or retaining walls within the lateral distance equal to the height of the backfill being compacted. The use of heavy compaction equipment close to the abutment or retaining walls may cause deflection or damage.

Provided the granular backfill is used, the following earth parameters are recommended for design in estimating lateral earth pressures on the abutments or retaining walls:

	Granular 'A'	Granular 'B'
Effective Angle of Internal Friction (Unfactored)	35E	32E
Soil unit weight	22.8 kN/m ³	21.2 kN/m ³
Active Earth Pressure Co-efficient, K_a	0.27	0.31
At Rest Earth Pressure Co-efficient, K_o	0.43	0.47

The active values should be used where the top of the wall is not fixed and can moved slightly outward. The at rest values should be used where the top of the wall is fixed and cannot move.

All fill materials placed beneath the roadway areas should be compacted to a minimum of 95 percent of Standard Proctor Maximum Dry Density (SPMDD). The fill should be placed in lift thicknesses not exceeding 200 mm.

The abutment backfill should be benched into the cut slopes in accordance with OPSD 208.01.

5.3 Erosion and Scour Protection

Scour protection should be provided for the proposed replacement structure. Based on the borehole information, the creek bed is expected to consist of firm to stiff clayey silt to silty clay or similar fill. This soil is described as having a medium scourability.

At the time of our site visits, the creek had a low flow velocities. However, the design flow velocities should be evaluated to determine if scour protection (rip rap) should be considered. As a minimum, at least 1.5 m of earth cover should be provided for scour protection from water flow and ice action, or an erosion resistant lining (such as rip rap stone) should be provided. The erosion resistant lining should extend at least 1.5 m upstream and 1.5 m downstream of the structure. The rip rap lining should extend at least 0.5 m above the design high water level.

If deep foundations are utilized, it is recommended that the underside elevation of the pile cap be set at least 1.5 m below the ultimate scour bed elevation.

5.4 Temporary Excavations

Temporary open excavations at the site are expected to extend to a maximum depth of about 4.5 m for spread footings and to a maximum depth of about 2 m for pile caps or grade beams for deep foundations. Excavations to a maximum of 5 m deep are expected to encounter a combination of earth fill materials (soft to firm) and, firm to stiff glacial tills. Excavations up to 2 m deep are expected to remain stable if excavated at side slopes not steeper than 1 horizontal to 1 vertical. Excavations below 2 m deep and to a maximum of 5 m deep are expected to remain stable at 1.5 horizontal to 1 vertical above the water table and at 2 horizontal to 1 vertical below the water table.

Alternatively, if the excavation width cannot be accommodated, a shoring system can be utilized. Excavations into the underlying soils can be shored with an appropriate system designed to accommodate an equivalent rectangular earth pressure (p) in accordance with Section 5.2.

Some groundwater flow should be expected to occur from the cohesionless materials that are expected to be encountered within the fill. However, flow rates can be readily accommodated with a properly designed sump and pump system. If the excavation extends below the creek level, measures will have to be taken to temporarily divert the flow.

Where workmen must enter excavations carried deeper than 1.2 m, the excavations should be inspected and certified by a geotechnical engineer, or suitably sloped and/or braced in accordance with the Occupational Health and Safety Act. The Occupational Health and Safety Act recognizes four (4) broad classifications of soils, as follows:

TYPE 1 SOIL

- a. is hard, solid, only able to be penetrated by a small sharp object with difficulty;
- b. can only be excavated by mechanical equipment;
- c. shows no sign of visible cracks after excavation;
- d. exhibits a dry, shiny appearance after excavation; and
- e. possesses a low moisture content and a high degree of internal strength.

TYPE 2 SOIL

- a. cracks or crumbles;
- b. can be penetrated by small sharp objects easily;
- c. can be excavated by hand tools with moderate difficulty;
- d. exhibits signs of surface cracking;
- e. exhibits a damp appearance after excavation; and
- f. possesses a low to medium moisture content and a medium degree of internal strength.

TYPE 3 SOIL

- a. is loose, soft, sandy, or previously excavated;
- b. can be excavated with hand tools easily;
- c. will run easily into a well defined conical pile if dry;
- d. will flow or shift unless supported if wet; and
- e. possesses a low degree of internal strength.

TYPE 4 SOIL

- a. is wet or muddy;
- b. will run easily or flow unless completely supported immediately after excavation;
- c. exerts substantial fluid pressure upon its supporting system; and
- d. possesses almost no internal strength.

The existing earth fill materials (soft to firm) at the site may be assumed to be Type 3 soils above the water table and Type 4 soils below it. The upper firm to very stiff glacial till soils should be assumed to be Type 2. The deep underlying hard glacial till and sand layer should be assumed to be Type 1 soils.

5.5 Sulphate Attack

Soluble sulphate testing was conducted on two (2) soil samples, as summarized on the attached Borehole logs. Based on the tested soil samples, there should not be any significant sulphate attack on a concrete structure. However, this should be confirmed by the Structural engineer.

5.6 Slope Stability

In regards to the slope stability of the area, the existing side slopes of the creek are currently in a relatively gentle and stable configuration and they are well vegetated. Revegetation or erosion protection should be provided for the side slopes if the existing vegetation is removed for construction purposes.

6.0 CLOSURE

It is recommended that the foundation preparation works (construction) be completed with regular and frequent inspection by an experienced geotechnical inspector, to check if the construction details are consistent with the recommended design parameters enclosed within. Routine quality control testing of the foundation concrete and fill placement is also recommended.

We trust that this report is sufficient for your present requirements. If you have any questions or require clarification on any matter, please do not hesitate to contact us.

Respectfully submitted,

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