



THURBER ENGINEERING LTD.

MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

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From: Alastair Gorman, P.Eng.
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN CAVANVILLE CREEK BRIDGE (SITE 26-120) G.W.P. 4291-11-01

1 INTRODUCTION

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Cavanville Creek Bridge on Highway 115 (Eastbound) in the geographic township of North Monaghan – Municipality of Peterborough, Ontario. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations at the site. It is noted that the proposed structural alternatives are not yet defined.

The recommendations provided in this memorandum are for planning, structure evaluation and preliminary design purposes only. Additional investigation and analysis may be required in any subsequent detail design phase of the project.

The following reference numbers apply to this site:

- Current W.P. 4291-11-01
- Site No. 26-120/1
- GEOCRES No. 31D-299
- Historic W.P. 192-81-04

2 SITE DESCRIPTION

The site is located on Highway 115 approximately 1.0 km east of the Highway 7/Peterborough County Road 28 interchange in North Monaghan Township, approximately 5 km southwest of the City of Peterborough. Based on the historic GA, the existing bridge is a 3-span prestressed concrete girder structure with a total span length of 50 m and a width of 13.5 m. It accommodates 2 lane eastbound traffic on Highway 115. The road grade on the bridge is approximately 4.5 m above the riverbank level.

The natural terrain in the vicinity of the bridge is generally flat. The historic GA indicates that the original grade in the vicinity of the bridge ranged from elevation 189.6 to 190.2 m. Highway 115



was constructed to approximately elevation 193.6. The approach fills were constructed by placing 3.4 to 4.0 m of fill with sideslopes inclined at 2H:1V.

3 SUBSURFACE CONDITIONS

The site is located within the physiographic region known as the Peterborough Drumlin Field, which is a rolling till plain with numerous drumlins, drumlinoid hills and surface flutings of the till sheet. The drumlins locally rise from sand and clay plains, and a veneer of sand overlies the till at the bridge site. The underlying bedrock consists of limestone of the Verulam Formation.

A site investigation was completed by Geocon Inc. between July 7 and 14, 1982. Four boreholes were drilled in conjunction with Standard Penetration Tests (SPTs) to depths of 14.0 to 17.4 m below the original ground surface. Adjacent to the boreholes, dynamic cone penetration tests (DCPTs) were advanced to 4.6 to 7.3 m depth below the original ground surface where practical DCPT refusal (over 100 blows per 0.3 m penetration) was encountered. All boreholes and DCPTs were completed adjacent to the abutment and pier alignments.

Soil conditions encountered in the boreholes generally consist of topsoil over a thin layer of gravelly sand underlain by glacial till overlying limestone bedrock. The topsoil was described as being 610 to 860 mm thick. The gravelly sand layer was 250 to 760 mm thick and described as loose to dense.

The underlying glacial till comprised sandy silt with some clay and gravel, and probable cobbles and boulders. The thickness of the till layer ranged from 10.4 to 12.8 m. The till was loose to compact to depths of about 4.0 to 5.5 m with SPT N-values of 7 to 27 blows/0.3m penetration. Below this depth, the till was generally dense to very dense with N-values of 33 to 149 blows/0.3. Moisture contents of the samples typically ranged from 6 to 16%, locally up to 21% near the ground surface.

A thin layer of sand was encountered between the glacial till and the underlying limestone bedrock. The sand layer was 150 to 890 mm thick.

Limestone bedrock was encountered at depths of 11.8 to 14.0 m (Elev. 177.8 to 176.2) and proven by coring 1.6 to 3.7 m below the bedrock surface. Rock Quality Designation (RQD) values of 52% to 100% were measured in the recovered rock cores, indicating fair to excellent quality. The upper 1.0 to 1.2 m of bedrock recovered from the boreholes on the east bank was described as weathered with sand seams.

Artesian groundwater conditions were reported near the bedrock surface in three of the boreholes. During drilling in these boreholes, the groundwater level in the hollow stem augers rose to 0.6 to 0.9 m above the ground surface (Elev. 190.6 to 191.1 m). A water level at 1.7 m above the ground surface (Elev. 191.9 m) was subsequently measured in a piezometer installed in one borehole.

The available GEOCRE files are attached in Appendix A.



4 SITE OBSERVATIONS

Foundations engineering staff from Thurber visited the site to observe conditions related to the geotechnical performance.

There were no obvious signs of settlement or distress in the foundations.

The approach embankments and forward slopes appeared to be stable, with no obvious signs of instability or bulging. There were no indications of erosion problems in the approaches.

Photographs of the structure and the approaches are attached in Appendix B.

5 EXISTING FOUNDATIONS

Based on the archive general arrangement and foundation drawings for the structure, the abutments and piers are supported on HP 310x110 piles driven to refusal on bedrock. The abutments are carried on 14 piles each (9 piles battered at 1H:3V in front row, and 5 piles battered at 1H:6V in back row), and the piers are carried on 9 piles each (4 outside piles battered at 1H:6V, and 5 remaining piles are vertical). The pile length ranged from 12.0 to 14.5 m.

The Pile Design Data given in the Geocon report for HP 310 X 110 piles is a factored ULS structural resistance of 1,650 kN. The discussion implicitly states that the SLS condition will not govern for piles driven to bedrock.

A 500 mm thick layer of Granular A material wrapped in a geotextile filter blanket was placed below the pile caps to minimize the potential for loss of fines as a result of possible artesian flow along the piles.

6 ASSESSMENT OF EXISTING FOUNDATIONS

6.1 Axial Resistance of H-Piles

Based on the GEOCRE information and the archive foundation drawings, the bridge is supported on HP 310x110 piles driven to bedrock. HP 310x110 piles driven to bedrock are routinely designed using a factored axial resistance of 2,000 kN at Ultimate Limit State (ULS). The SLS condition is considered not to govern for piles driven to bedrock.

It is noted however that the existing design was apparently based on a lower factored ULS capacity of 1,650 kN. The potential exists that refusal to some of the piles may have been met on boulders or very dense till above the bedrock surface, and this lesser criterion may have been accepted during construction. It is therefore recommended that preliminary assessment of the existing structure and its foundations be carried out using the previous geotechnical resistances:

Factored Axial Resistance at ULS = 1,650 kN

Use of higher resistance values may be possible if pile driving records are available to confirm that all piles extended to bedrock.



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If the resistance of 1,650 kN (ULS_r) is not sufficient to meet the assessed load demand, the foundations can be reassessed on the basis of the actual load demand. However, in the absence of pile driving records, potential increases to the allowed resistance may be limited.

6.2 Lateral Resistance

The existing design is based on the lateral forces being carried by the battered piles. It is recommended that no soil lateral resistance be assumed in this case.

7 EXCAVATION AND ROADWAY PROTECTION

If the selected rehabilitation strategy requires excavation in the approach fills behind the abutments, it is recommended that site investigation and field testing be carried out in each approach fill in order to characterize the fill and to select parameters for the design of roadway protection. One borehole within each approach fill and within the probable extent of excavation is considered to be appropriate. The boreholes should extend for the full depth of fill or to twice the depth of excavation, whichever is the greater.

8 CLOSURE

The factual subsurface information used in the preparation of this memorandum was taken from the report by Geocon Inc. titled "Report to Ministry of Transportation and Communications Ontario, Proposed Bridge at Cavanville Creek on Highway 115 (East Bound Lanes), District 7, Port Hope, Ontario (WP192-81-04)" and undated.



The memorandum was prepared by Mr. Murray Anderson, P.Eng., Senior Foundations Engineer and by Mr. Alastair Gorman, P,Eng. Senior Foundations Engineer and was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Alastair Gorman, P.Eng.
Associate, Senior Foundation Engineer

P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

Attachments

Appendix A

GEOCRES Report, Correspondence, and Archive Drawings

T10571

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REPORT TO
MINISTRY OF
TRANSPORTATION AND COMMUNICATIONS
ONTARIO

PROPOSED BRIDGE AT CAVANVILLE CREEK ON HWY 115
(EAST BOUND LANES), DISTRICT 7,
PORT HOPE, ONTARIO.
(WP192-81-04)

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Procedure and Field Equipment

APPENDIX II

Records of Boreholes

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DRAWING

1928104-A

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1.0 INTRODUCTION

Geocon Inc. has been retained by the Ministry of Transportation and Communications, Ontario (M.T.C.) to carry out a foundation investigation at the site of a proposed Bridge to carry the east bound lanes of Highway 115 over Cavanville Creek in the Township of North Monaghan, County of Peterborough, Ontario. The work for this project was authorized under M.T.C. Agreement Number 4242-9082-65 dated. July 8th, 1982 following submission of our proposal of July 5th, 1982.

The purpose of the investigation was to obtain subsurface information for use in the design and construction of foundations for the proposed Bridge structure.

2.0 SITE AND GEOLOGY

The site of the proposed Bridge is approximately 1 kilometre northeast of the junction of Highways 115 and 28, where it is planned that the new Bridge would carry the east bound lanes of Highway 115 over Cavanville Creek. The proposed Bridge lies to the south of the existing Bridge which would then carry only the west bound lanes. The project limit is contained within Highway 115 chainage of 10+850 to 10+950.

The site is mainly covered with grass with some areas of dense tree growth. The relief is generally flat apart from the fill approach embankments supporting the paved surface of the existing Highway 115. The Cavanville Creek at this location flows in a southeasterly direction and is approximately 15 metres wide. The average depth of water at the time of investigation was 0.25 metres.

2.0 SITE AND GEOLOGY (continued)

Available geological information shows that the area is located towards the southern limits of the Peterborough Drumlin Field. The area is underlain mainly by silty and sandy glacial till soils. The bedrock below the till overburden in most of this region is of the Trenton Limestone Formation.

3.0 SUMMARIZED SUBSURFACE CONDITIONS

The stratigraphy encountered at the boreholes of this investigation is shown on the individual Records of Boreholes in Appendix II of this Report, together with details of sampling and drilling and the results of field and laboratory tests. A plan of the site showing borehole locations and stratigraphic sections is presented on the enclosed Drawing 1928104-A.

The field boring programme comprised a total of four boreholes, two on either side of Cavanville Creek, offset approximately 6.5 metres from the centreline of the proposed east bound lanes of Highway 115. The boreholes were located at one end of each of the proposed abutments and piers.

A surficial layer of brown topsoil with organics was encountered in all boreholes and its thickness ranged from 0.61 metre to 0.86 metre. Underlying the surficial topsoil a stratum of gravelly sand was encountered which extends to a depth of approximately 1.37 metres. A stratum of sandy silt (till) with occasional lenses of silt was encountered underlying the gravelly sand layer. The till stratum contains more coarser sizes of gravel, probable cobbles or boulders at depth. This stratum ranges in thickness from 10.44 metres to 12.78 metres. The sand silt (till) is loose to compact within the top 2.5 to 4.5 metres of the stratum whereas it is dense to very dense below. Underlying the sandy silt (till) a layer of sand was encountered in the three easterly boreholes. This sand ranged in thickness from 0.2 metres to 0.9 metres. Underlying the overburden soils, limestone bedrock

3.0 SUMMARIZED SUBSURFACE CONDITIONS (continued)

was encountered in the four boreholes. The bedrock surface ranges in depth from approximately 11.8 to 14.0 metres below ground surface, corresponding to elevations 177.8 to 176.2 metres.

Ground water was encountered in all the boreholes from ground surface to slightly below. From water levels recorded during drilling and sampling, the rate of inflow into casing or augers from the sandy silt (till) stratum was slow.

Artesian water conditions were encountered in the three westerly boreholes, as discussed in detail below.

4.0 DETAILED SUBSURFACE INFORMATION

The individual strata and the ground water conditions encountered at the boreholes are described in the following sections.

4.1 Topsoil

A surficial stratum of topsoil was encountered in all boreholes put down. This stratum extended down for depths ranging from approximately 0.61 metre to 0.86 metre.

The topsoil is brown to dark brown silty clay with abundant organics throughout and was soft to firm.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.2 Gravelly Sand

Underlying the surficial topsoil, a stratum of gravelly sand ranging in thickness from 0.25 metre to 0.76 metre was encountered. It is considered that this material represents an alluvial deposit. Generally, the stratum was loose, except in Borehole 4 where "N" values from the Standard Penetration Test infer a dense relative density. The result of this test may have been influenced by coarser material within the gravelly sand.

4.3 Sandy Silt, Some Clay and Gravel (Till)

Underlying the gravelly sand stratum is a stratum of sandy silt with some clay and gravel (till). The percentage of gravel, sand, silt and clay varied with depth and between boreholes as shown by the grain size distributions given on the Records of Boreholes and on the grain size distribution envelope, Figure 1 in Appendix II. Pockets or layers of silt were also encountered, and typical gradations are shown on Figure 2, in Appendix II.

Except in the silt layers, grain size distribution analyses carried out on representative split spoon samples showed the samples as tested contain 7 to 13 percent clay sizes, 35 to 59 percent of silt sizes, 20 to 43 percent of sand sizes and 8 to 16 percent of gravel sizes. In the silt layers, the percentage of silt ranged from 87 to 93, with traces of clay, sand and gravel sizes. From the test results and tactile examination, the upper part of the till tended to be cohesive in character, while near the base of the stratum it was predominantly non-cohesive.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.3 Sandy Silt, Some Clay and Gravel (Till) (Continued)

Natural moisture content tests and Atterberg Limit tests were made on selected samples of this stratum and the results are shown on the Records of Boreholes and on the Plasticity Chart, Figure 3 in Appendix II. Generally, the Plastic Limit was from 16 percent to 17 percent and the Plasticity Index was about 1, except for Sample 3 in Borehole 3, which had a Plastic Limit of 22 percent and a Plasticity Index of 10. The results show that the passing No. 40 sieve material in the till soil is mainly silt, except for the result from Borehole 3, which would be classified as silty clay of low plasticity. A slight decrease in the natural moisture content is apparent below elevation 182.

The sandy silt (till) stratum varies in thickness from 10.44 metres to 12.78 metres and extends down to an elevation varying from 177.78 metres to 176.41 metres.

The results of the Standard Penetration Tests and the dynamic cone penetration tests (pentests) shown on the Records of Boreholes in Appendix II indicate that the upper part of this stratum down to a depth varying from about 4 metres to 5.5 metres from the ground surface is only loose to compact, whereas it is dense to very dense below this depth. As noted earlier, the upper part of the till tended to be cohesive and, from tactile examination, might be alternatively described as having a firm to stiff consistency. During drilling, occasional coarser sizes were encountered by the augers in Borehole 4 indicating that coarse

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.3 Sandy Silt, Some Clay and Gravel (Till) (Continued)

gravels, cobble sizes, or boulders may be expected within the till particularly in the lower predominantly non-cohesive portion of the stratum. The M.T.C. Foundation Investigation report for the existing Bridge (W.P. 91-72-08) describes cobbles being present in the till stratum below elevation 182.9 m, with cobbles ranging from about 100 mm to 180 mm in size.

4.4 Sand to Sand, Trace Gravel

A thin layer of sand was encountered underlying the sandy silt (till) and overlying the bedrock surface in Boreholes 2, 3 and 4. The identification of the sand layer was based on recovered wash samples, in Boreholes 3 and 4 and observed sand in wash water in Borehole 2. The two wash samples comprise sand sizes, trace fine gravel. The thickness of the sand layer varied from 0.15 metre in Borehole 2, 0.23 metre in Borehole 3 and 0.89 metre in Borehole 4.

4.5 Limestone Bedrock

Below the sand layer in Boreholes 2, 3 and 4 and below the sandy silt (till) in Borehole 1, limestone bedrock was encountered. The depth from ground surface to bedrock surface ranged from about 11.8 to 14.0 metres.

The bedrock was penetrated by core drilling to depths varying from 1.61 metres to 3.83 metres, corresponding to elevations varying from 175.95 metres to 172.83 metres.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.5 Limestone Bedrock (continued)

A visual examination of the bedrock cores, in conjunction with the description of the bedrock in the previous M.T.C. report for this site, indicates that this bedrock is grey, medium grained and medium hard limestone.

The upper portion of the bedrock is weathered and contains close spaced to moderately close spaced discontinuities and occasional sand seams in Boreholes 3 and 4 on the east side of the Creek. The presence of these sand seams is based on observations of return wash water during drilling. The thickness of the weathered zone is judged to vary from about 1 metre in Borehole 4 to 1.2 metres in Borehole 3.

Below the weathered zone the grey limestone bedrock is considered to be generally unweathered or, sound with estimated Rock Quality Designations (R.Q.D.'s) of 69 percent to 100 percent and moderately close spaced discontinuities.

4.6 Ground Water Conditions

Ground water was observed in all boreholes at the time of drilling, from at or near ground surface. One Casagrande type piezometer was installed in Borehole 4. At the time of drilling, artesian water conditions were found to exist in Boreholes 2, 3 and 4. In Boreholes 2 and 3 this condition apparently originated from immediately above the limestone bedrock and water levels of 0.61 m and 0.91 m above ground surface were recorded within the 82.6 mm I.D. hollow stem augers.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.6 Ground Water Conditions (continued)

In Borehole 4, the artesian water conditions were noted when the hollow stem augers penetrated approximately 180 mm into the limestone bedrock and the water level in the hollow stem augers rose to 0.91 m above ground surface. After placing BW casing and core drilling into the bedrock, a piezometer was installed to a depth of 13.1 m (tip elevation 177.1 m). Approximately 1/2 hour after installation, the artesian head in the piezometer tubing was 1.7 metres above ground surface and the rate of water outflow from the 9.5 mm I.D. piezometer tubing at 0.3 m above ground surface was estimated at 0.23 litre per minute. In the previous M.T.C. Foundation Investigation mentioned above, a flow rate of about 7 litres per minute from the 114.3 mm hollow stem auger at ground surface was estimated. Details of the ground water conditions are shown on the attached Records of Boreholes and Drawing No. 1928104-A.

In Boreholes 2 and 4, sand entered the hollow stem augers at depths of about 8 metres and 11 metres, respectively, apparently due to granular seams with water under pressure within the till stratum. Wash boring techniques were used to advance the boreholes past these seams.

5.0 DISCUSSION

It is understood that as presently planned the proposed Bridge on Highway 115 crossing the Cavanville Creek will comprise a three span structure with span lengths of 20 metres between abutments and piers and 24 metres between piers. The Bridge would be designed to carry 2 lanes of eastbound

5.0 DISCUSSION (continued)

traffic and would have a width of some 12.6 metres. The Bridge deck would probably be of continuous pre-stressed concrete design similar to the existing Bridge some 30 metres (centreline to centreline) northwest of the proposed Bridge. As shown on Plan E-6018-1 dated May 1982, the proposed footing locations would be at an angle to the road centreline, and in these positions the south-easterly portion of the east pier would be located in the present Cavanville Creek bed. The proposed grade indicates that the approach embankment fills at the abutments would have a height above existing ground surface of some 3 to 4 metres. The proposed grade rises from east to west from about elevation 193.0 metres to 193.8 metres between the proposed abutments.

Subsurface conditions in the boreholes at the proposed Bridge site comprise surficial layers of topsoil overlying gravelly sand up to 2 metres depth which overlies a stratum of sandy silt glacial till. The till has interbedded layers or lenses of silt and has a variable content of clay and gravel and occasional coarse gravel, cobble, possibly boulder sizes particularly in the lower portion of the stratum. The till above about elevation 185 tended to be predominantly cohesive in character and to have an estimated consistency of firm to stiff based on tactile examination. The characteristics of the till changed gradually to a predominantly non-cohesive material in a dense to very dense state, near the base of the stratum. The stratum has a measured thickness of some 11 to 13 metres. In the three easterly boreholes, a thin layer of sand was encountered between the base of the till and the overlying limestone bedrock. The surface of the bedrock was intersected at

5.0 DISCUSSION (continued)

depths below ground surface ranging from a 11.8 to 14 metres corresponding to elevations 177.8 to 176.4. Groundwater was present at or near ground surface during drilling and sampling. In addition, artesian water conditions were encountered in the thin sand layer and top of limestone bedrock in the three easterly boreholes. The artesian head measured in hollow stem augers and in piezometer tubing varied from 0.91 metres to 1.7 metres above ground surface.

The M.T.C. Foundation Investigation report carried out at the site of the existing Bridge, (WP 91-72-08, Site No. 26-120) describes closely similar subsurface conditions to those of this investigation. It is understood that steel H piles driven to end-bearing on bedrock were used to support the piers and abutments of the existing Bridge, which has a continuous prestressed concrete deck. It is noted that erosion protection in the form of riprap has been placed up to approximately 1 metre above water level on the banks of the Cavanville Creek and riprap placed on the open slopes between the base of piers and abutments beneath the existing Bridge and at the base of the approach embankments close to the Cavanville Creek.

The findings of the present investigation indicate that two alternative systems could be considered, namely piles and spread foundations. These are discussed in preliminary fashion, and from a geotechnical engineering standpoint only, in the following sections. We would propose to make more specific comments and recommendations as the needs of the Bridge Designers arise. Such further input would, for example, need to be made concurrently with consideration of such factors as i) specifics of dead and live loadings, ii) structural details, iii) construction methodology for both the substructure and deck, v) possible influence of

5.0 DISCUSSION (continued)

post-construction effects such as temperature changes and creep in deck concrete, vi) tolerances to deformations such as differential settlements between piers or piers and abutments, vii) comparative economics, and the like.

5.1 Piles

Piles driven to satisfactory end-bearing in bedrock, or drilled concrete caissons socketted into bedrock, would provide a satisfactory foundation for the piers and abutments of the proposed Bridge. Although a number of pile types could be considered in addition to the drilled caissons, it is probable that the steel H pile type as used on the adjoining Bridge, would be found to be the most suitable when geotechnical factors such as the following are taken into account concurrently with other pertinent factors beyond the scope of this report, such as structural details, practical construction considerations, comparative economics, and the like.

- the dense nature of the lower part of the till stratum,
- the presence of water-bearing granular layers or lenses in the till,
- the occurrence of silt layers in the till,
- the sand at the till-to-bedrock contact,
- observed artesian pressures in the region of the overburden-to-bedrock contact.

The steel H pile type only is elaborated on in this report. Should you require it however, we would be pleased to discuss further, pile or pier systems other than H piles which derive their support by end-bearing on or within the bedrock.

5.0 DISCUSSION (continued)

5.1 Piles (continued)

On the basis of the properties of the sound bedrock as estimated from observations during drilling and visual examination of recovered core, it is considered that the criteria for the design of steel piles would be the structural capacity of the piles, with of course due cognizance of the need to penetrate the till to effective seating on or in bedrock. In this respect, it is recommended that H piles be equipped with a suitable rock point, and that other measures be provided to assist in penetrating the till if necessary, and that provision be made for re-driving piles to verify adequate seating on bedrock. For piles driven to adequate bearing on or in the bedrock, settlement under load would comprise mainly elastic compression of the piles.

For preliminary design, it is estimated for example, based on the Ontario Highway Bridge Design Code (1982), that the factored structural capacities of 310HP110 and 310HP79 steel H piles driven to adequate end-bearing in bedrock, would be about 1650 and, 1150 kilonewtons, respectively. In order to finalise selection of the pile working load, however, it is recommended that a load test be carried out on a pile representative of the type selected.

Consideration could also be given to the use of cast-in place expanded base piles of the Franki type, developing their capacity by founding in the dense to very dense till soils below approximately elevation 184.

5.0 DISCUSSION (continued)

5.1 Piles (continued)

With use of a piled foundation, lateral loads should be resisted by a suitable system of battered piles.

5.2 Spread Foundations

Spread foundations could be considered as an alternative to piles or piers end-bearing on bedrock, or expanded base type piles founded in the dense till. Spread foundations would however, also have to be carried in dense till below elevation 185. Because of the depth of excavation involved, and the need to apply special measures, such as advance dewatering during construction, as discussed later, it is doubtful whether the use of spread footings would be as economical as the approach based on the use of end-bearing piles.

For individual spread foundations carried in the till below elevation 185, and of the size that would be required for the abutment and piers in this case, the design bearing value will probably be controlled by considerations of permissible differential settlement rather than bearing capacity.

For purposes of preliminary appraisal of use of spread foundations, the till stratum as a whole below elevation 185 has been assigned an undrained shear strength of 200 kPa based on tactile examination of recovered samples. On this basis, and assuming a footing width of 4 m., a factored bearing capacity at ultimate limit states of about 600 kPa, is obtained. Because of the variable nature of the till in terms of density and consistency, the presence of silt layers and water-bearing granular layers or lenses, and the susceptibility of the till to disturbance during construction, it is

5.0 DISCUSSION (continued)

5.2 Spread Foundations (continued)

judged that the bearing capacity at serviceability limit states Type II should be limited to 300 kPa, subject to review during final design as discussed earlier, depending on footing size, permissible settlements, and the like.

The design of the abutments founded at elevation 185, should be noted that (i) the estimated undrained shear strength of the till above this elevation is about 75 kPa, and (ii) that the stability against sliding, particularly through silt layers in the till, should be checked.

5.3 Approach Embankments

The approach embankments on the east and west side of the Bridge would be 3.5 metres to 4 metres, respectively, above existing ground surface. It is assumed that these embankments would be constructed using free draining non frost susceptible granular materials, compacted to at least 95 percent Standard Proctor maximum dry density and have side slopes of 2 horizontal to 1 vertical as per M.T.C. standards. Provided the topsoil and all loose or soft surficial materials within the plan limits of the embankments are removed, such embankments would have adequate stability. Post-construction settlements would be largely due to compression under the effects of self weight, and should thus be within tolerable limits for the anticipated heights of fill involved. Additional commentary could be provided covering use of fill materials not complying with the above assumption of essentially clean granular material.

5.0 DISCUSSION (continued)

5.4 General Recommendations

A number of other general recommendations from a geotechnical standpoint, are applicable to design and construction, as follows:

- i) Pile caps and foundations should be provided with a minimum of 1.5 metres of earth cover or other suitable measures for protection against frost action.
- ii) Protective measures against scour of the foundations for the piers and abutments should be established through consideration of the hydrology of Cavanville Creek, and protective measures provided for the existing Bridge. Ice loadings should also be accounted for, as applicable.
- iii) Suitable measures should be provided for isolating foundation excavations from the flow in Cavanville Creek, and supporting sides of such excavations. In addition, particularly spread foundations be adopted, suitable measures should be incorporated into the construction methodology to depressurize water-bearing pervious layers or lenses in or under the till, to the extent required. In addition, final trimming of excavations should be carried out with care to minimize disturbance to the foundation till, and a mud mat applied for the same purpose.
- iv) Backfill around pier foundations and for abutment construction should consist of well compacted, select, non-frost susceptible free draining granular materials. In the case of the abutments, suitable drainage measures should be incorporated in the backfill.

5.0 DISCUSSION (continued)

- v) In calculating earth pressures on the abutments due to the granular backfill alone, the angle of internal friction of the backfill may be taken as 30 degrees, in which case coefficients of lateral earth pressure for the at-rest state would be considered applicable i.e. 0.58 for the ultimate limit states and 0.50 for the serviceability limit states. In the passive state, the corresponding coefficients of earth pressure would be 2.4 and 3.0, respectively. Depending on the geometry of the excavation below ground surface to founding elevation 185, in the case of spread footings it may be necessary to also consider the effect of the till above this elevation, on lateral pressure and resistance. Allowance for surcharge and lateral loads should be made in computations.
- vi) It is suggested that available records for the adjacent Bridge be reviewed, particularly with respect to installation of steel H piles through the till to bedrock, and any effects of the artesian water condition close to the bedrock surface.

6.0 CLOSURE

The field work for this investigation was carried out by Mr. J. Zoras under the supervision of the undersigned. This report was written by Mr. K. S. Senathirajah, P. Eng., and the writer, and reviewed by Mr. M. A. J. Matich, P. Eng.

6.0 CLOSURE

As discussed earlier, we would be pleased to continue liaison with the Bridge Designers as design progresses. We wish to record also, our appreciation for this opportunity to have been of further service to the M.T.C. and acknowledge the cooperation extended by involved Ministry personnel.

Yours very truly,

GEOCON INC.



R. C. Sansom, P. Eng.
Assistant District Manager.



RCS:jj
T10571/34681

APPENDIX I

GEOCON

1.0 PROCEDURE AND EQUIPMENT

The field work for this investigation was carried out between 1982-07-07 and 1982-07-14. Atcost Drilling Co. Ltd. supplied all the field equipment together with a 2-man drilling crew. A Bombardier mounted CME power auger drill (Category 5.3 (I)) equipped with hollow stem augers, BW casing for wash boring and BXL rock coring equipment was used to put down a total of 4 boreholes ranging in depth from 14.0 metres to 17.4 metres.

Boreholes were advanced in the overburden with the auger drill and samples obtained at intervals not exceeding 1.5 metres. Standard Penetration Tests were carried out in conjunction with the use of these samplers. A 63.5 kg hammer dropping free fall for 762 mm was used to drive the sampler for the Standard Penetration Tests.

Uncased dynamic cone penetration tests (pentests) were performed adjacent to each borehole to depths ranging from 4.6 to 7.3 metres. Each pentest was driven to refusal (greater than 100 blows per 0.3 metre). A standard 51 mm diameter, 60 degree cone tip was used for each pentest.

Boreholes were advanced into the bedrock in BXL core size for depths ranging from 1.7 metres to 3.8 metres. Samples recovered were examined in the field to determine percent recovery and Rock Quality Designations (R.Q.D.'s).

A 13 mm O.D. flexible plastic tubing attached to a porous Casagrande type piezometer tip was installed in Borehole 4 to a depth of 13.1 metres. The piezometer tip was within the sand layer overlying the bedrock. The piezometer was pushed into the sand which had caved into the hole. The hole above the piezometer was backfilled with the sandy silt till around the tubing. Ground water outflow from the tube was estimated at a rate of 0.23 litres per minute. At the end of the field work the piezometer tubing was pinched, taped and buried below ground surface to prevent further outflow.

1.0 PROCEDURE AND EQUIPMENT (continued)

The locations of the boreholes of this investigation are shown on Drawing 1928104-A which accompanies this report. Locations were obtained by tape measurements referenced to existing Chainage Stations marked by stakes on the centreline of the median between the westbound and proposed eastbound lanes of Highway 115. Borehole elevations were obtained by levelling referenced to a Geodetic Benchmark located on the north side of the west end of the existing Bridge on Highway 115. The elevation of this Benchmark was given as 193.218 metres. The locations of the boreholes and ground surface elevation at the boreholes were determined in the field by Geocon personnel.

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

METRIC

W P 192-81-04 LOCATION Co-ords. 4 899 063.40 N; 393 752.00 E ORIGINATED BY JZ
 DIST 7 HWY 115 BOREHOLE TYPE Hollow Stem Augers, BXL Rock Core & Cone Test COMPILED BY PAD
 DATUM Geodetic DATE 1982 07 14 CHECKED BY RCS

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
189.55	Ground Level												
0.00	Topsoil		1	SS	2								
188.98	Soft Brown												
0.61	Gravelly Sand some silt		2	SS	7								
188.27	Loose Brown												
1.37	Sandy Silt, some clay and gravel (till) Grey Loose to Compact Very Dense		3	SS	19								
			4	SS	12								8 20 59 13
			5	SS	7								
			6	SS	54								
			7	SS	149								
			8 _n	SS	64								
	Layer of Silt		9	SS	63								
			10	SS	45								
177.78	Limestone Bedrock Sound Grey			RC									
11.81			11	BXL	97%								RQD 94%
			12	BXL	98%								87%
174.68													
14.91	End of Borehole												

+³, x⁵: Numbers refer to Sensitivity 20
 15 ϕ 5 (%) STRAIN AT FAILURE
 10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

METRIC

W P 192-81-04 LOCATION Co-ords. 4 899 083.50 N; 393 753.00 E ORIGINATED BY JZ
 DIST 7 HWY 115 BOREHOLE TYPE Hollow Stem Augers, Wash Boring BXL Rock Core & Cone Test COMPILED BY PAD
 DATUM Geodetic DATE 1982 07 12-13 CHECKED BY RCS

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					
189.95	Ground Level													
0.00	Soft Topsoil Dark Brown		1	SS	2									
189.34	Gravelly Sand		2	SS	6									
0.61	Loose Wet Dark Brown		3	SS	14									
188.58	Sandy Silt, some clay and gravel (till)		4	SS	15									
1.37	Grey Compact Dense		5	SS	14									
			6	SS	33									12 35 44 9
			7	SS	20									Soil rose in hole from 8.08 m to 6.19 m Wash Boring from 8.08 m to 9.14 m
			8	SS	35									16 28 45 11
			9	WS										
			10	SS	46									15 43 35 7
			11	SS	34									
177.71	Sand		12	SS	50/7									
12.24	Limestone Bedrock			RC										RQP
12.39	Sand		13	BXL	98%									91%
175.95	Grey		14	RC	94%									94%
14.00	End of Borehole			BXL										

+3, x5: Numbers refer to 20
Sensitivity 15-5 (%) STRAIN AT FAILURE
10

OFFICE REPORT SOIL LABORATORY

RECORD OF BOREHOLE No 3

METRIC

W P 192-81-04 LOCATION Co-ords. 4 899 095.10 N; 393 782.50 E ORIGINATED BY JZ
 DIST 7 HWY 115 BOREHOLE TYPE Hollow Stem Augers, Wash Boring, BXL Rock Core & Cone Test COMPILED BY PAD
 DATUM Geodetic DATE 1982 07 09 & 12 CHECKED BY RCS

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH					
190.20	Ground Level						20 40 60 80 100						
0.00	Topsoil		1A	SS	4								
0.76	Soft Brown		1B	SS	10								
0.76	Gravelly sand		2A	SS	10								
1.01	Sandy Silt, some clay and gravel (Till) Grey <u>Loose to Compact</u> <u>Very Dense</u> <u>Layer of Silt</u>		3	SS	8								
			4	PH									
			5	SS	11								
			6	SS	17								
			7	SS	105								
			8	SS	63								
			9	SS	112								
			10	SS	103								
			11	SS	56								
			12	SS	100	76							
176.41	Sand Brown		13	WS									
14.02	Occasional Sand Seams Weathered		14	RC BXL	98%							RQD 52%	
174.12	Limestone Bedrock Sound Grey		15	RC BXL	100%							100%	
16.08	End of Borehole												

+3, x5: Numbers refer to Sensitivity
 20
 15 ⚡ 5 (%) STRAIN AT FAILURE
 10

OFFSHORE SOIL LORA

RECORD OF BOREHOLE No 4

METRIC

W P 192-81-04 LOCATION Co-ords. 4 899 115.50 N; 393 783.40E. ORIGINATED BY JZ
 DIST 7 HWY 115 BOREHOLE TYPE Hollow Stem Augers, Wash Boring, BXL Rock Core & Cone Test COMPILED BY PAD
 DATUM Geodetic DATE 1982 07 07-09 CHECKED BY RCS

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60					
190.20	Ground Level														
0.00	Topsoil		1	SS	6										
189.34	Firm Dark Brown		2	SS	44										
0.86	Gravelly Sand														
188.83	Dense Brown		3	SS	10										
1.37	Sandy Silt, some clay and gravel (till)		4	SS	8										
			5	PH											
	Grey Loose to Compact Dense to Very Dense		6	SS	27										
			7	SS	47										
	Probable cobbles or boulders		8	SS	50/25										
			9	SS	70										
			10	SS	74										
			11	SS	119										
177.55	Sand, trace gravel														
12.65	Brown		12	WS											
176.66	Occasional Sand Seams		13	RC	100										
13.54	Weathered		14	SS	100/130										
	Limestone Sound Bedrock Grey		15	RC BXL	96%										RQD 69%
			16	RC BXL	100%										
172.93															
17.27	End of Borehole														

+³, x⁵: Numbers refer to Sensitivity 20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT SOIL LABORATORY

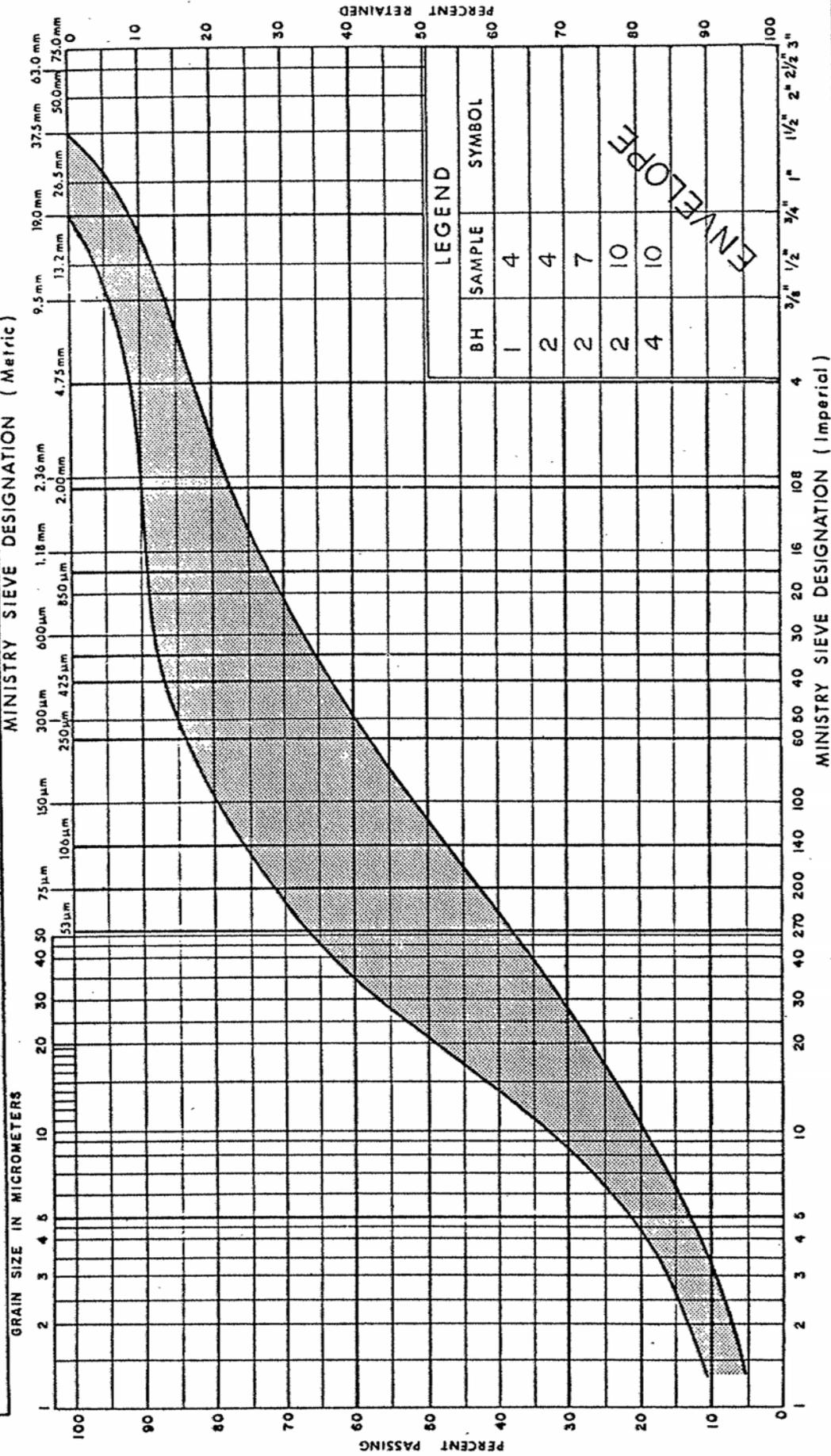
APPENDIX II

FIGURES-LABORATORY TESTING

GEOCON

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
Fine		Medium	Coarse	Fine	Course	

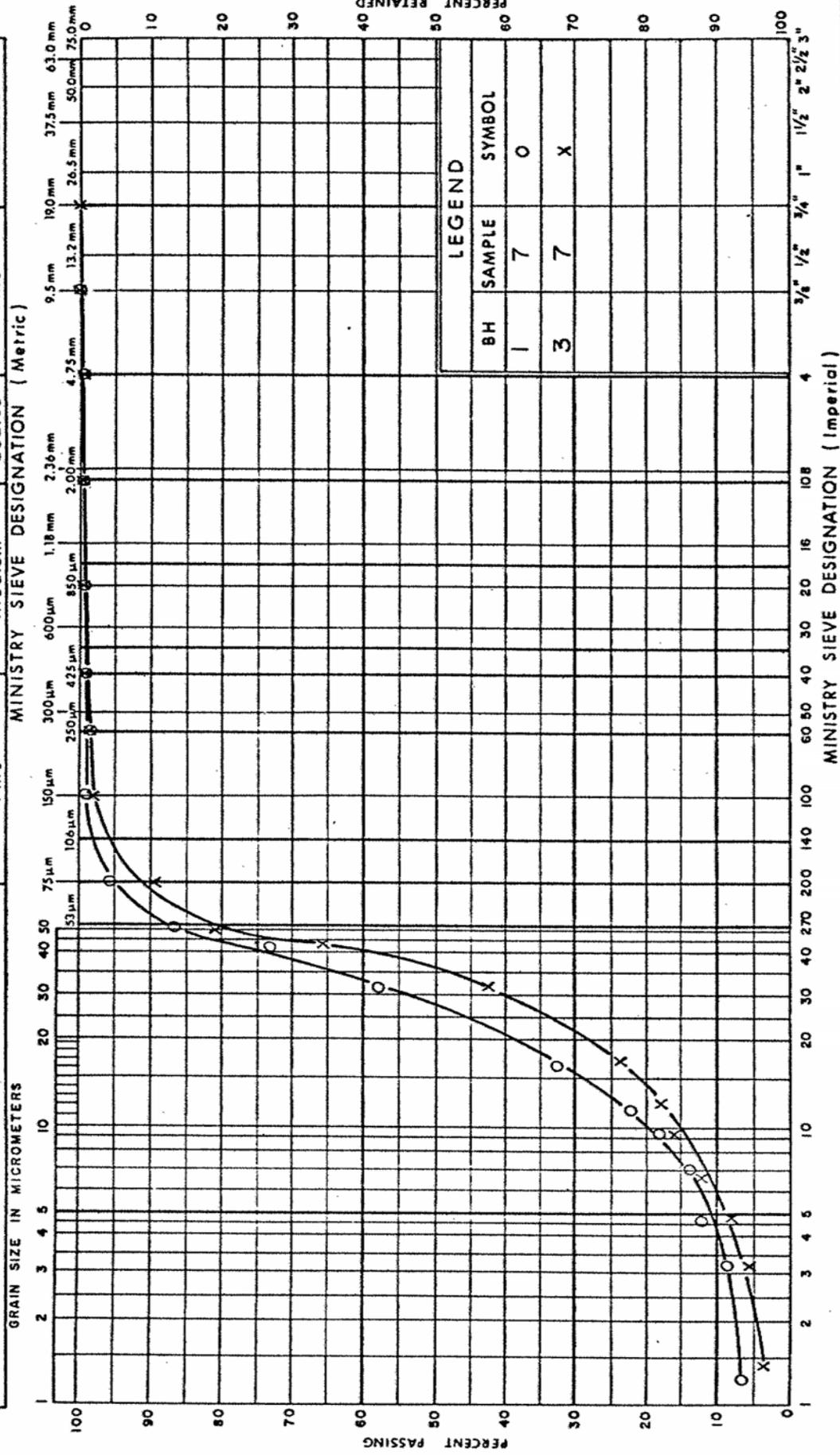


GRAIN SIZE DISTRIBUTION SANDY SILT, SOME CLAY AND GRAVEL (TILL)

FIG No 1
 WP 192 - 81 - 04

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
GRAIN SIZE IN MICROMETERS		Fine	Medium	Coarse	Fine	Course



Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILT LAYERS IN SANDY SILT (TILL)

FIG No 2

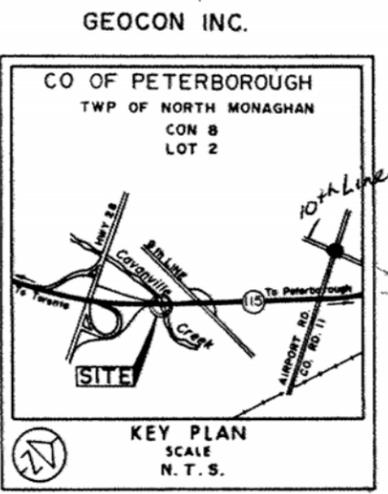
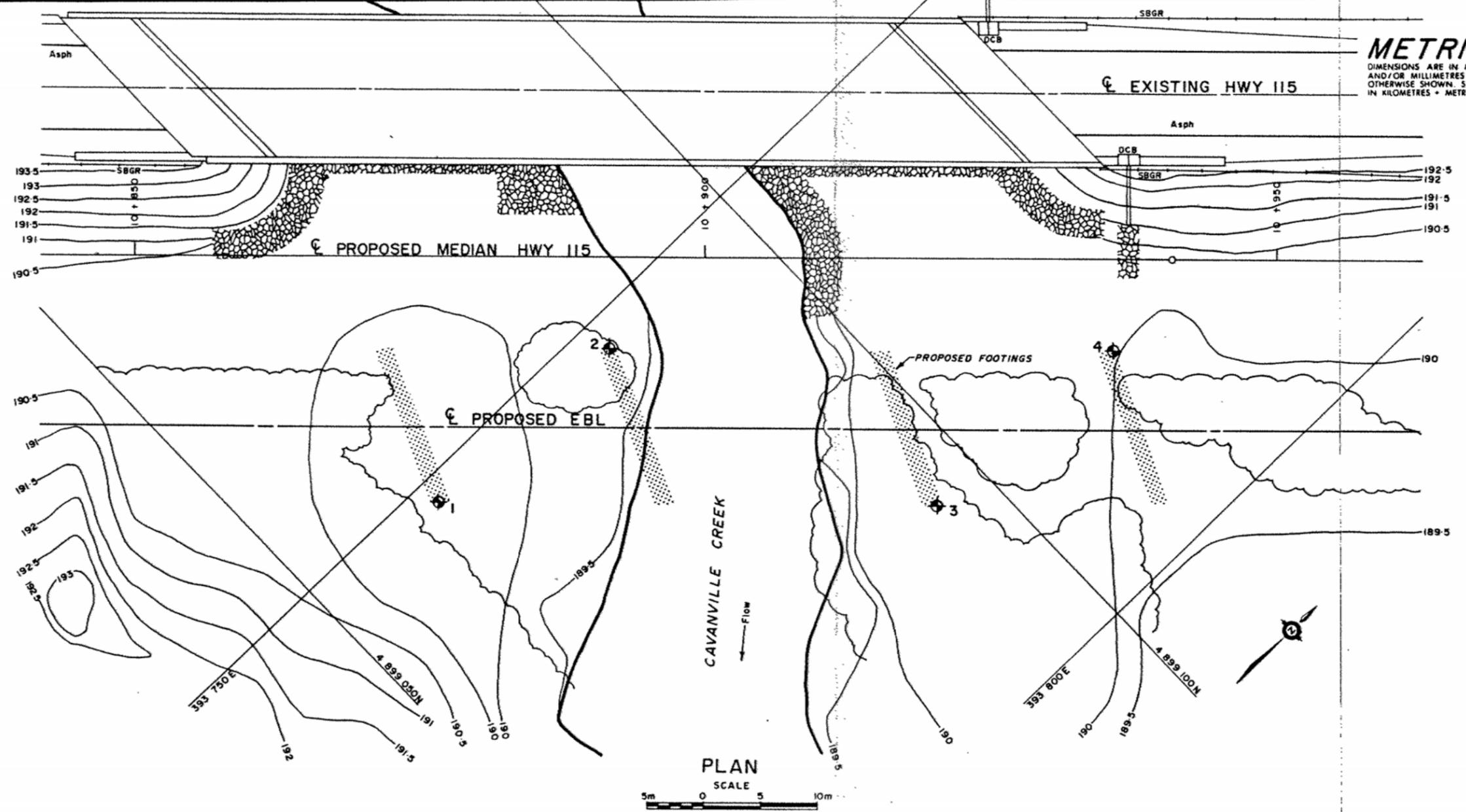
WP 192 - 81 - 04



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO PH-D-207 81-04

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

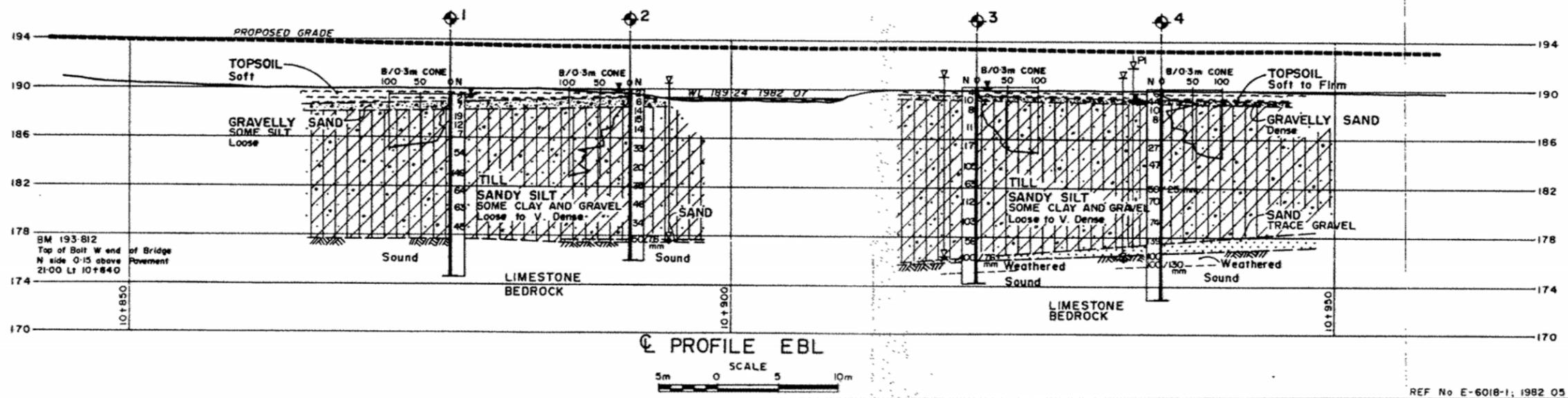
CONT No
 WP No 192-81-04
**PROPOSED
 CAVANVILLE CREEK BRIDGE
 AT HWY 115 (EASTBOUND ONLY)**
 BORE HOLE LOCATIONS & SOIL STRATA



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ▽ W.L. at time of investigation 1982 07
- ▽ Head ARTESIAN WATER Encountered
- ▽ PIEZOMETER

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	189.59	4 899 063.40	393 752.00
2	189.95	4 899 063.50	393 753.00
3	190.20	4 899 095.10	393 782.50
4	190.20	4 899 115.50	393 783.40



NOTE
 The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION

Geocres No 31D-299

HWY No 115 E B L	DATE 1982 08 09	DIST 7
SUBM'D RCS/CHECKED	DATE 1982 08 09	SITE 26-458-120
DRAWN PAD/CHECKED	DATE 1982 08 09	DWG 1928104-A

REF No E-6018-1; 1982 05

memorandum



To: Mr. M. Holowka
Design Engineer
Structural Office
Kingston

Date: 83 06 30

From: Pavement & Foundation Design Section
Room 315, Central Building
Downsview

Re: Cavanville Creek Bridge
(Eastbound Only) at Highway 115,
W.P. 192-81-004, Site 26-458-120
District 7

We have reviewed the D4, Special Provisions and half size prints for the above-noted structure that were attached to your letter of 83 06 17.

We have no comments to make at this time.

M. MacLean

M. MacLean, P.Eng.
Foundations Engineer

MM:gm

cc: G.C.E. Burkhardt
Files

memorandum



To: Mr. M. Holowka
Design Engineer (Eastern)
Structural Office
3501 Dufferin St., 4th Floor

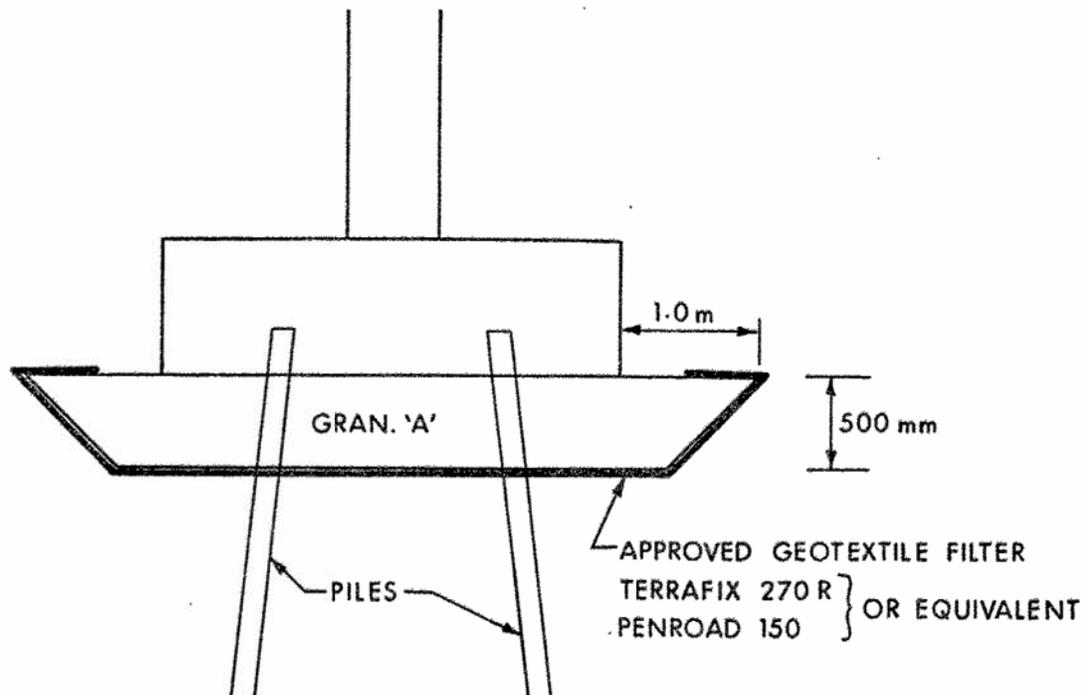
Date: 83 01 05

From: Pavement & Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Cavanville Creek Bridge E.B.
W.P. 192-81-04, Site 26-120
Hwy. 115, District 7

We have reviewed the preliminary general arrangement drawing for the above-mentioned structural site and provide the following comments:

- 1) Due to anticipated difficulty driving through the glacial till deposit, all steel 'H' section piles should be equipped with reinforced flange tip plates.
- 2) Details for the filter blanket to minimize loss of fines as a result of possible artesian flows should be shown as follows:



Construction sequence:

- 1) Excavate for blanket and place geotextile along base and sides.
- 2) Place well compacted Granular 'A' filter blanket to bottom of pile cap and overlap geotextile.
- 3) Drive piles through granular blanket and geotextile.



Tom Kazmierowski, P. Eng.
Foundations Engineer

TK:syc

cc: G.C.E. Burkhardt

memorandum



To: Mr. G.C.E. Burkhardt
Head, Structural Section
Central (5000 Yonge St.) Region

Date: 82 09 17

From: Pavement & Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Foundation Investigation
Proposed Cavanville Creek Bridge
at Hwy. 115 (E.B. Lane Only)
W.P. 192-81-04, Site 26-458
District #7, (Port Hope), Central Region

Geocon Inc., has been retained by the Ministry to carry out a foundation investigation at the above-mentioned site and provide factual subsurface data together with recommendations for the design and construction of foundation and associated earthworks. Attached please find their final report and drawings describing the subsurface conditions and foundation recommendations. We have reviewed the report for the technical content and format and our comments are as follows:

In our opinion drilled concrete caissons socketted into bedrock would not provide satisfactory economical foundation for the piers and abutments in view of the presence of artesian water conditions at and above the bedrock surface. The most viable alternative is to support the foundation elements on end bearing steel 'H' piles driven to bedrock. The M.T.C. Foundation Investigation Report for the existing E.B. Lane Bridge (W.P. 91-72-08) describes cobbles being present in the glacial till stratum below Elev. 182.9. Therefore, it will be necessary to provide reinforced tips to these piles. The suggested design parameters are as follows:

<u>Pile Type</u>	<u>Factored Capacity at Ultimate Limit States</u>	<u>Capacity at Serviceability Limit States Type II</u>
310 HP 110	1650 kN	1150 kN
310 HP 79	1150 kN	820 kN

It is understood that perched abutments will be adopted and the backfill behind the abutments should consist of free draining granular material as per current M.T.C. requirements. Earth pressures should be computed as per subsections 6.6.1.2.2 of the O.H.B.D. code.

It is suggested that care should be exercised particularly with respect to installation of Steel 'H' piles through the glacial till to bedrock which is subjected to artesian water conditions. Any artesian flow up the underside of the pile cap can be accommodated by an appropriately designed filter blanket which will prevent the loss of fines. In addition a permanent drainage system be installed to accommodate the future artesian flow.

The surficial stratum of dark brown soft silty clay with organics approximately 0.6 meters to 0.9 meters should completely subexcavated within the plan limits of the proposed approach embankments for a minimum distance of 30 meters behind the abutments.

In our opinion, spread foundations alternative is not an economical solution in view of the deep excavations required to reach the desired footing formation elevation below river water level.

We believe the aforementioned comments together with information contained in the enclosed foundation report will be adequate for your requirements. Should you require further clarification or additional information, please feel free to contact us.


M. Devata, P. Eng.
Senior Foundation Engineer

MD:syc

Encls.

cc: G.C.E. Burkhardt (3)
R.D. Gunter
F. Norman
J. Smrcka (2)
K. Bassi
B.J. Giroux
R. Hore

R. Fitzgibbon (Cover Only)
T.J. Kovich (Cover Only)

IC JAY 115 AND CAVANVILLE RE
W.P. 192-81-05, SITE 26-458-120
DISTRICT 7, PORT HOPE



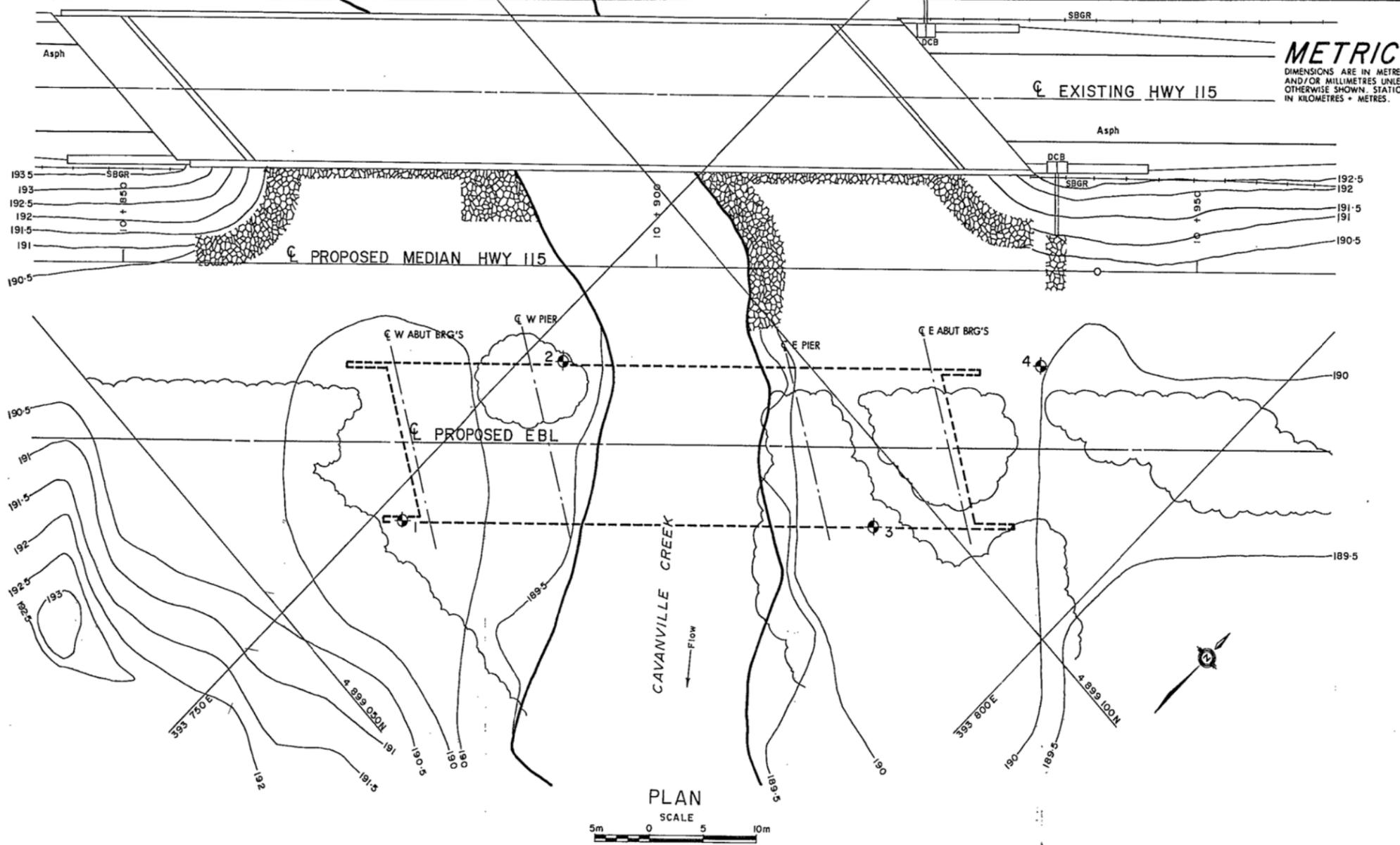
VIEW OF LOCATION (LOOKING SOUTH-EAST)

RECEIVED
JUN 10 1982
Foundation Design Section



VIEW OF LOCATION (LOOKING SOUTH-EAST)

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS BRID PH-D-207 82 04



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

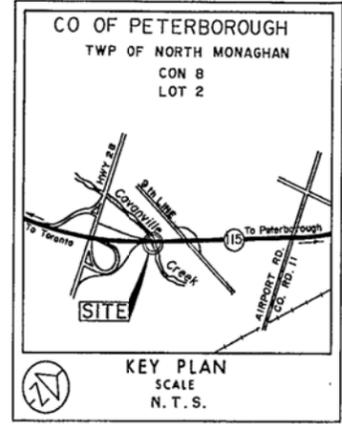
CONT No 85-40
WP No 192-81-04



PROPOSED
CAVANVILLE CREEK BRIDGE
AT HWY 115 (EASTBOUND ONLY)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
172

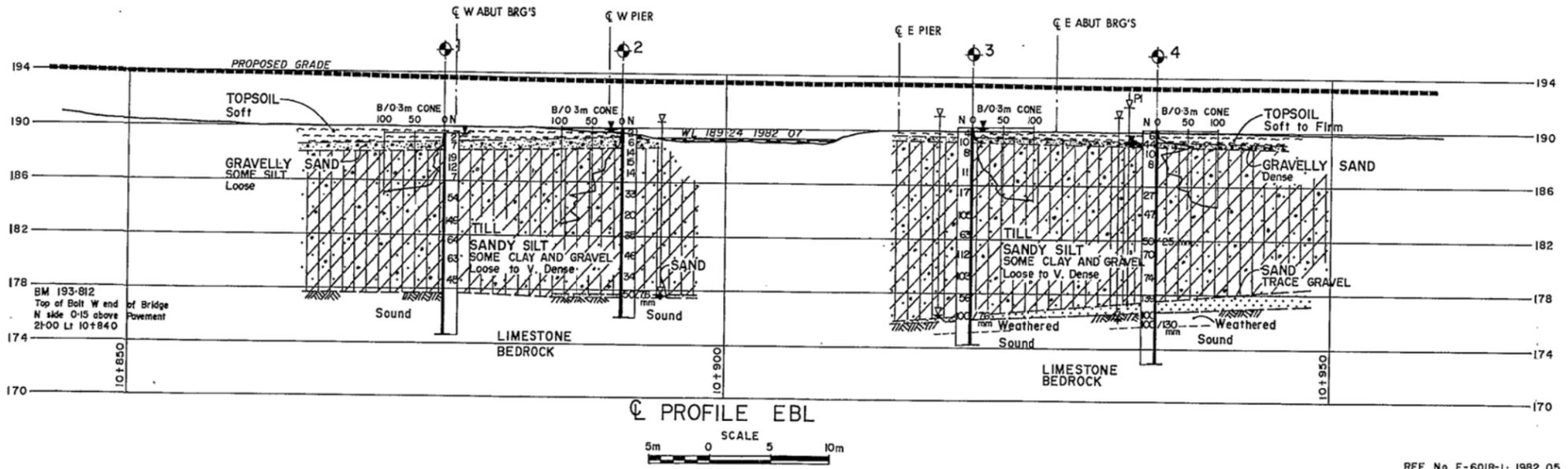
GEOCON INC.



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ▽ WL at time of investigation 1982 07
- ▽ Head ARTESIAN WATER Encountered
- PIEZOMETER

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	189.59	4 899 063.40	393 752.00
2	189.95	4 899 083.50	393 753.00
3	190.20	4 899 095.10	393 782.50
4	190.20	4 899 115.50	393 783.40



NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

Geocres No 31D-299

HWY No 115 EBL	DIST 7
SUBAPP RCS CHECKED	DATE 1982 08 09
DRAWN PAD CHECKED	APPROVER RCS
	SITE 26-458-120
	DWG 2

REF No E-6018-1; 1982 05

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO PROJECT NO. 85-13 (FORMERLY 085-13)

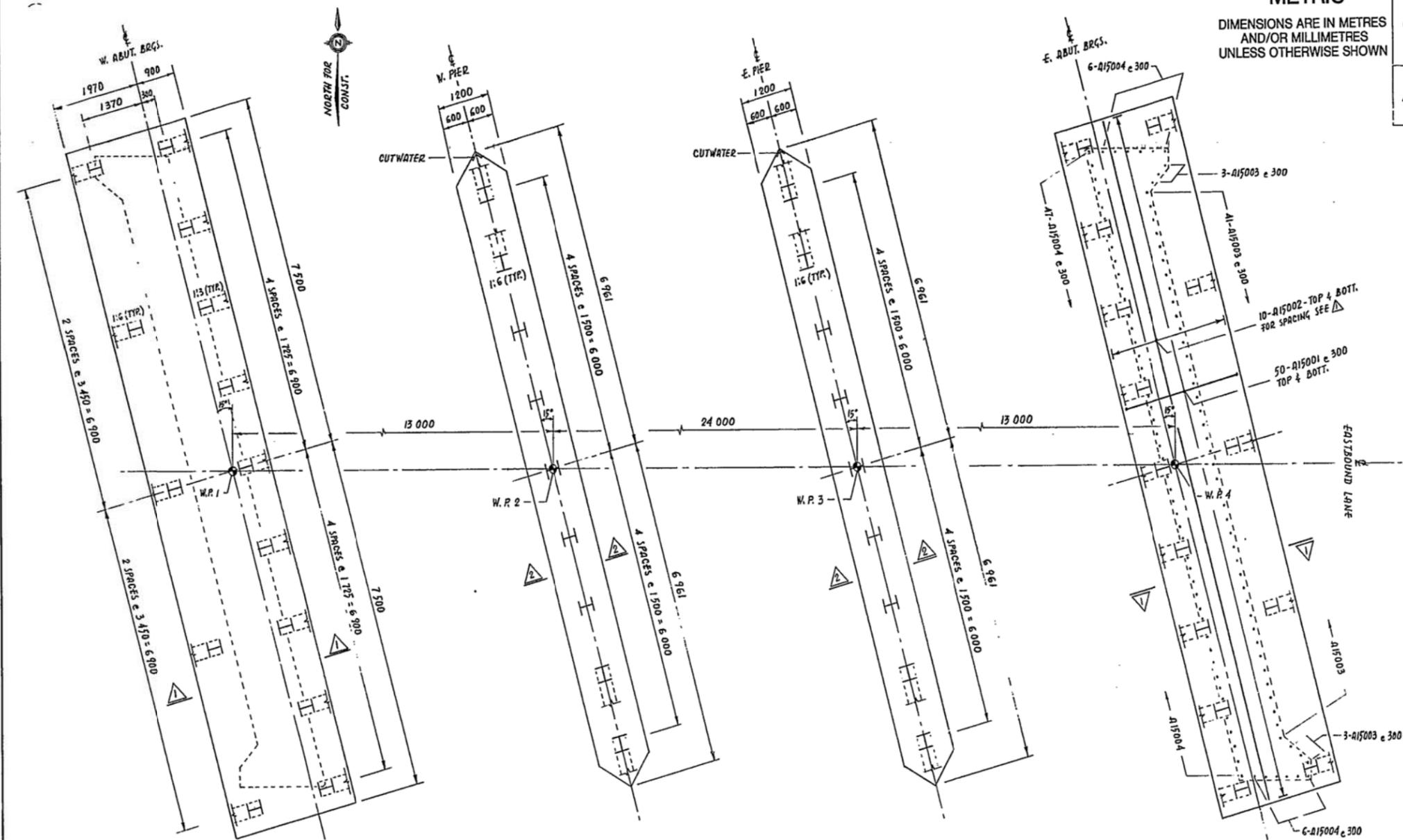
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 85-40
WP No 192-81-04

CAVANVILLE CREEK BRIDGE
AT HWY. 115 (EASTBOUND ONLY)
FOOTINGS

SHEET
173



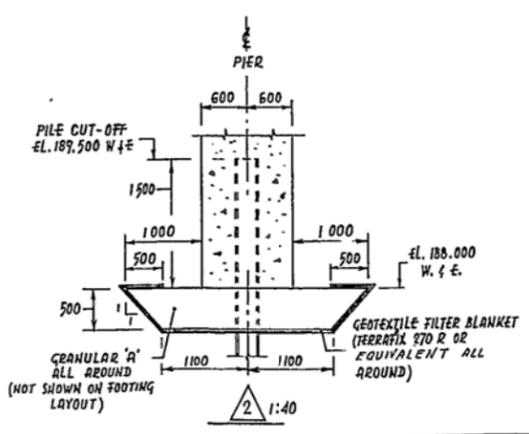
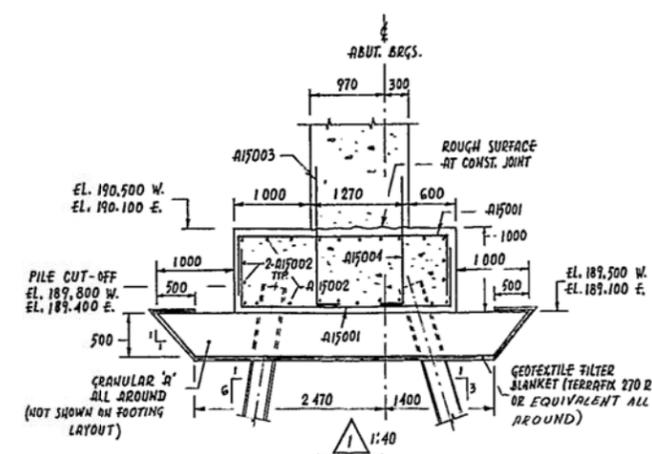
NOTE:
E. ABUTMENT FOOTING DIMENSIONS, PILE LAYOUT & REINFORCEMENT SIMILAR TO THOSE OF W. ABUTMENT, BUT ROTATED 180°

FOOTING LAYOUT
1:50

- CONSTRUCTION SEQUENCE**
- EXCAVATE FOR BLANKET AND PLACE GEOTEXTILE ALONG BASE AND SIDES.
 - PLACE WELL COMPACTED GRANULAR "A" FILTER BLANKET TO BOTTOM OF PILE CAP AND OVERLAP GEOTEXTILE.
 - DRIVE PILES THROUGH GRANULAR BLANKET AND GEOTEXTILE.
 - CONSTRUCT THE REMAINING PORTION OF THE STRUCTURE.

- PILE DESIGN DATA**
- DESIGN LOAD AT S.L.S. TYPE II = 1150 KN
 - FACTORED CAPACITY AT U.L.S. = 1600 KN

- PILE CONSTRUCTION DATA**
- SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF ABUTMENT FOOTINGS AND AT BOTTOM OF PIERS.
 - DRIVING SHOES SHALL BE INSTALLED ON ALL PILE TIPS IN ACCORDANCE WITH STD. DD-3301.
 - PILES SHALL BE DRIVEN TO BEDROCK.



PILE DATA

LOCATION	BATTER	TYPE	NO REQ'D	LENGTH
WEST ABUT.	1:3	HP310x110	9	13.000
	1:6	HP310x110	5	12.500
EAST ABUT.	1:3	HP310x110	9	14.500
	1:6	HP310x110	5	14.000
WEST PIER	VERTICAL	HP310x110	5	12.000
	1:6	HP310x110	4	12.500
EAST PIER	VERTICAL	HP310x110	5	13.500
	1:6	HP310x110	4	14.000



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN	A.L.	CHECK	M.G.	LOADING	048DC-A-79	DATE	13 MAR.
DRAWING	P.K.	CHECK	A.S.	SITE No	26-156-120	DWG	3

Appendix B
Site Photographs



South Side of Cavanville Creek Bridge Eastbound Lanes



North Side of Cavanville Creek Bridge Eastbound Lanes