

**FINAL
REPORT ON

DETAILED FOUNDATION INVESTIGATION AND DESIGN
TOMIKO RIVER BRIDGE
HIGHWAY 11, NORTH OF NORTH BAY
GWP 711-92-00 (WP 344-00-01)
SITE NUMBER 43-10
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 54, SUDBURY**

Submitted to:

McCormick Rankin Corporation
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McCormick Rankin Corporation
1145 Hunt Club Road
Suite #300
Ottawa, Ontario K1V 0Y3

Attention: Mr. Manny Goetz, P.Eng.
Project Manager

**RE: RESPONSE TO MTO STRUCTURAL SECTION COMMENTS
DETAILED FOUNDATION INVESTIGATION AND DESIGN REPORT
TOMIKO RIVER BRIDGE, SITE 43-10
GWP 711-92-00 (WP 344-00-01)
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 54, SUDBURY**

Dear Sirs:

This letter provides our response to the comments made by MTO Structural Section, Northern Region, on our Foundation Investigation and Design Report for the Tomiko River Bridge. The comments by MTO were outlined in their letter dated December 11, 2001 that was received by us on December 13, 2001. These comments are incorporated, as appropriate, into our Final Detailed Foundation Investigation and Design Report that accompanies this letter. This final report supercedes our previous report dated November 2001.

As requested by MTO Northern Region, our Final Foundations Report dated December 2001 includes all of the borehole information obtained at the Tomiko River Bridge site. The report is prepared instead of the addendum that was originally proposed as outlined in recent letters to you.

This letter, in conjunction with our previous letter dated December 4, 2001, confirms that our Final Foundations Report incorporates all comments made by MTO Structural Section contained in their letters dated August 17 and December 11, 2001.

The following specific comments are provided in connection with the MTO comments contained in their letter dated December 11, 2001.

- **Section 5.2.1.2 Set Criteria.** This section of the final report has been substantially revised to incorporate the MTO comments. The set criteria for driven piles are to be established as per SP903S01. No other mandatory requirements are stated.
- **Disadvantages of rockfill.** The final report contains a sentence stating that a rock fill embankment may hinder some construction activities such as the installation of roadway protection.
- **Section 5.6 – Groundwater Control.** The final report has added a paragraph to state explicitly that groundwater control is the responsibility of the contractor, and that the Contract should make provision for groundwater control. Water levels have been explicitly defined and terms such as significant or significantly to describe levels have not been used. The main intent of this section is to make the designer aware of the potential need for dewatering when the technical specifications are being prepared.

We trust that this letter and our final Foundation Investigation and Design report dated December 2001 will satisfy the requirements of MTO. Should you have any additional questions please contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.

Dennis E. Becker, P. Eng.
Principal

Fin J. Heffernan, P.Eng.
Designated MTO Foundation Contact

/DEB/FJH/ pds

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PART A

**FOUNDATION INVESTIGATION REPORT
TOMIKO RIVER BRIDGE
HIGHWAY 11, NORTH OF NORTH BAY
GWP 711-92-00 (WP 344-00-01)
SITE NUMBER 43-10
MINISTRY OF TRANSPORTATION, ONTARIO
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PART B

**FOUNDATION AND DESIGN
TOMIKO RIVER BRIDGE
HIGHWAY 11,NORTH OF NORTH BAY
GWP 711-92-00 (WP 344-00-01)
SITE NUMBER 43-10
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B Record of Borehole Sheets (1960, Dominion Soils Ltd.)

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a detailed foundation investigation for the proposed widening or replacement of the existing Tomiko River Bridge that carries Highway 11 over the Tomiko River. The proposed work is part of the detailed design for the re-alignment and widening of Highway 11 being carried out for the MTO. The Highway 11 widening and re-alignment project includes reconstruction of the highway to design standards from 9.8 km north of the North Bay City limits, northerly 14.5 km. The general site location is shown in plan on Figure 1.

The terms of reference for the scope of work are outlined in Golder's proposal P01-1284, dated August 30, 2000 and our proposal letter dated November 1, 2001. These letters form part of the Consultant's Agreement (Number 5005-A-000116) for this project.

The field investigation was carried out to establish the subsurface conditions at the Tomiko River Bridge, including the associated approach embankments, by borehole drilling, in-situ testing and laboratory testing on selected samples.

A previous foundation investigation was carried out at the site in 1960 under the supervision of Dominion Soil Investigation Ltd. (DSIL, 1960). The results of this investigation are presented in their report entitled "*Foundation Investigation, Proposed Bridge over Tomiko River, Approximately 21 miles north of North Bay, on Highway 11, District 13, WP 95-60*", dated May 25, 1960. The DSIL report was submitted to Department of Highways, Ontario, Materials and Research Section, Downsview, Ontario and is currently stored under Geocres No. 31L-34. The Record of Borehole sheets for the 1960 investigation are presented in Appendix B.

The details of the proposed Highway 11 re-alignment in the vicinity of the Tomiko River and the preferred location of the bridge structure were provided to Golder by MRC. The boreholes for the current investigation were located in the field relative to existing site features and the extent of the required bridge abutment and pier widenings based on the information provided by MRC.

This report supercedes our Foundation Investigation and Design Report dated November 2001.

2.0 SITE DESCRIPTION

The site is located within the Township of Lyman about 30 km north of North Bay. The existing bridge carrying Highway 11 over the Tomiko River is a three-span, steel beam structure with a concrete deck. The existing bridge stratum is approximately 40 m long and about 11 m wide. From the north and south of the site, the highway slopes downward towards the bridge. In the immediate vicinity of the structure, the ground surface is gently sloping down to the Tomiko River. The river flows in a westerly direction at the bridge location.

The topography in the area adjacent to the Tomiko River is rolling with exposed bedrock to the south of the bridge. On the northwest side of the bridge, the shoreline is relatively well vegetated with mature trees. The existing ground surface is at about Elevation 280 m near the river. The elevation of the existing roadway / bridge deck at the site lies between about Elevations 283 m and 284 m, referenced to Geodetic Datum.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the bridge replacement option was carried out on November 26, 2001 when four boreholes (Boreholes 01-39 to 01-42) were put down at the site. The fieldwork for the bridge widening option was carried out on May 1, 2 and June 11, 2001 when a total of 7 sampled boreholes (Boreholes 01-01 to 01-06 and 01-02A) were advanced at the site. All the boreholes were located at the site with reference to the existing Tomiko River Bridge abutments and piers. Table 1 summarizes the field investigation program carried out.

Both field investigations were carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between about 4.9 m and 12.3 m (to elevations ranging from 276.2 m and 269.2 m), using 108 mm inside diameter (I.D.) continuous flight hollow stem augers and NQ size rock core barrel. Soil samples were obtained at regular intervals of depth, ranging from 0.75 m to 1.5 m, using a 50 mm outside diameter (O.D.) split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedures. Shelby tube samples of the soft cohesive soils were obtained in Borehole 01-2A. Bedrock was cored in Boreholes 01-01, 01-02, 01-04, 01-05, 01-39 and 01-41. Boreholes 01-03 and 01-06 were drilled to refusal at the location of the south and north approaches, respectively. Borehole 01-06 was drilled from the shoulder of the existing Highway 11 roadway to investigate the embankment fill composition and underlying soils at the site. Boreholes 01-40 and 01-42 were advanced to refusal at the location of the south and north abutments of the new proposed replacement bridge structure. A plan and profile showing the borehole locations and interpreted stratigraphy along Highway 11 are shown on Drawing 1.

Field vane shear testing was carried out in the varved silty clay deposit in Borehole 01-01, using an MTO 'N' vane. Specialized laboratory testing such as oedometer (consolidation) and laboratory vane shear, and classification testing including Atterberg Limits, organic content and specific gravity were carried on the Shelby tube samples taken in Borehole 01-02A. Table 2 summarizes the laboratory test program carried out. All of the tests were performed to MTO and / or ASTM Standards, as appropriate.

The groundwater conditions in the open boreholes were observed during the drilling operations. Two piezometers were installed and sealed at the abutment locations to permit groundwater monitoring. The groundwater readings are described on the Record of Borehole sheets that follow the text of this report.

The fieldwork was supervised throughout by a senior member of our technical staff, who located the boreholes in the field, cleared the locations for services, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labeled and

transported to our Mississauga geotechnical laboratory. In the laboratory, the samples underwent detailed visual examination and laboratory tests were carried out on selected samples. The results of these tests are indicated on the Record of Borehole sheets and detailed in Appendix A.

On completion of the fieldwork, all investigated borehole locations were surveyed using the NAD 83 MTM (Zone 12) co-ordinate system and the geodetic datum for elevation. The surveying for the bridge widening option was carried out and the information provided to us by Tulloch Engineers and Surveyors of Thessalon, Ontario. The surveying of the boreholes advanced during the investigation for the bridge replacement option in November 2001 was carried out by Golder Associates Ltd. of Mississauga, Ontario.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Site Geology

From published geologic information, the site is located in the physiographic region known as the Laurentian Highlands that form the southernmost part of the Canadian Shield (Geology of Ontario; OGS Special Volume 4). The Laurentian Highlands are comprised of a southeast trending and slightly elevated region that is underlain by Precambrian bedrock. To the south, the highlands gradually disappear beneath the cover sequences of Paleozoic strata and the southern Borderlands. These Precambrian rocks were eroded to a gently undulating land surface, before the deposition of Paleozoic strata and later erosion, during glaciation, left behind only scarred Precambrian rocks covered in a few places by flat-lying Paleozoic strata. The local physiography is generally characterized by variable overburden materials and an irregular, variable bedrock surface with rock outcrops.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, 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. The results from the laboratory testing from the current investigation are provided in Appendix A. The detailed subsurface soil and groundwater conditions encountered in the 1960 boreholes advanced at this site by DSIL are presented on the Record of Borehole sheets contained in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries therefore represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole and locations.

The inferred soil stratigraphy as encountered at the site is shown on Drawing 1. In general, the subsoils at the site consist of sand and gravel fill overlying a granular deposit which ranges in composition from sand and gravel to gravelly sand, to sand, to sand and silt. At the north side of the Tomiko River Bridge, this granular deposit lies directly over the bedrock. At the south side of the bridge, the granular deposit is underlain by deposits of organic sandy silt and varved silty clay over the bedrock. In Boreholes 01-3, 60-1 and 60-2 on the south side of the Tomiko River, layers of organic silty sand and sand and silt overlying the sand and gravel stratum and immediately

below the sand and gravel fill were encountered. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Embankment Fill

Embankment fill composed of brown sand and gravel, trace to some silt, trace clay with occasional cobbles was encountered in Borehole 01-06 drilled through the existing Highway 11 roadway. Sand and gravel fill was also encountered at the surface in all of the 2001 and 1960 boreholes, except Boreholes 01-02, 01-02A, 01-05 and 60-3 in the river area. The elevation of the top of this layer (i.e. ground surface) ranges from 280.8 m to 283.2 m and the total encountered thickness of the fill varies from 2.9 m to 1.5 m. Traces of organics and rootlets were also noted within this layer, generally to the south of the Tomiko River Bridge.

Standard Penetration Testing (SPT) measured 'N' values ranging from 4 to 43 blows per 0.3 m of penetration, indicating a loose to dense state of packing within the sand and gravel fill. A grain size distribution curve on a selected sample of the sand and gravel embankment fill is shown on Figure A1. The natural water content measured on samples from the sand and gravel fill ranged from about 8 to 10 percent, with an average of about 9 percent.

4.2.2 Cobbles, Sand and Gravel

A 0.6 m to 1.5 m thick layer of cobbles, sand and gravel was encountered at the ground surface in Boreholes 01-05, 01-41 and 01-42. The top of this layer (i.e. ground surface) ranges from Elevation 280.1 m to elevation 280.3 m. It was not feasible to carry out SPT sampling due to the presence of numerous cobbles within the deposit.

4.2.3 Silty Sand Containing Some Organic Matter

Immediately below the sand and gravel fill in Boreholes 01-03, 60-1 and 60-2 (located south of the bridge), a layer of black silty sand containing some organic matter was encountered. The surface of this layer was encountered between Elevations 279.6 m and 279 m. The layer ranges in thickness from 100 mm to about 600 mm. The SPT 'N' values measured in the layer in Boreholes 60-1 and 60-2 were 0 and 2 blows per 0.3 m of penetration, indicating that this silty sand has a very loose relative density.

4.2.4 Sand and Gravel to Gravelly Sand, to Sand, to Sand and Silt

Beneath the embankment fill and organic silty sand (where present), and from ground surface in interlayered sand and gravel, sand, and sand and silt. The sand and gravel to gravelly sand portions of the deposit contain trace silt and occasional cobbles, as shown by the grain size distribution test result plotted on Figures A2 and A13 of Appendix A. Trace to some organics and rootlets were noted within this layer in Boreholes 01-01, 01-02, 01-02A and 01-40 on the south side of the river. The sand layers of this deposit, that were encountered in boreholes drilled on the north side of the river, contain trace to some gravel, trace silt and occasional cobbles, as shown on Figure A3 of Appendix A. The sand and silt to silt and sand layers of this deposit contain trace clay, as shown on Figure A4 of Appendix A. It is noted that a boulder was encountered at the base of the deposit, immediately overlying bedrock, in Borehole 60-4.

The surface of this interlayered deposit was encountered between Elevations 278.6 m and 280.3 m in the boreholes drilled on the north side of the river, and between Elevations 279.5 m and 280.8 m in the boreholes advanced on the south side of the river. On the south side of the river, the deposit ranged from about 2 m to 4 m in thickness as encountered in Boreholes 01-01 to 01-03, 60-1 and 60-2. On the north side of the river, the deposit was between 4 m and 8.5 m thick as encountered in the 2001 and 1960 boreholes.

The measured SPT 'N' values range from 3 to 84 blows per 0.3 m of penetration, but are typically between 5 and 30 blows per 0.3 m of penetration; these 'N' values indicate that this interlayered deposit generally has a loose to compact relative density. Measured natural water contents range from 13 to 45 percent in the sand and gravel to gravelly sand layers, 11 to 24 percent in the sand layers, and 25 to 35 percent in the sand and silt layers.

A faint hydrocarbon / organic odour was noted in the samples of sand and gravel obtained from Borehole 01-03, located approximately 15 m south of the existing bridge. No evidence of any hydrocarbon odour or any sheen or staining was observed in the soil during the drilling and sampling operations in the field. The faint hydrocarbon odour was noted in the samples only after they had been sealed in their jars in the laboratory for a period of approximately two weeks.

4.2.5 Organic Sandy Silts

On the south side of the Tomiko River Bridge, beneath the native granular deposits, a layer of black organic sandy silt with trace clay was encountered in Boreholes 01-02, 01-2A, 01-03 and 01-41. The top of this layer was encountered between Elevations 276.2 m and 277.6 m, and the layer has a thickness ranging from 0.3 m to 0.8 m.

Standard Penetration Testing (SPT) carried out within this deposit in Boreholes 01-02 and 01-41 measured 'N' values of 2 and 3 blows per 0.3 m of penetration, suggesting a soft consistency.

Atterberg Limits testing conducted on two samples obtained from this stratum gave variable results and show liquid limits (w_L) of 50 and 167 percent with a plasticity index (I_p) of 4 and 98 percent, respectively. The results of the Limits testing classify the soil in this stratum as organic and of high plasticity. The test results are plotted on the plasticity chart on Figure A5 in Appendix A.

Organic contents of 9 and 14 percent were measured on two samples from this deposit and the average specific gravity is estimated to be about 2.4 based on the results of two tests. The measurements conducted on a carefully trimmed sample of the organic sandy silt to estimate the natural bulk unit weight resulted in a value of about 13 kN/m³.

A laboratory oedometer (consolidation) test was carried out on a specimen of the organic sandy silt obtained from Borehole 01-02A. A pre-consolidation pressure of approximately 65 kPa for this stratum is estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Details of the test results are shown on Figures A6 and A7 and included on the associated oedometer consolidation summary sheets in Appendix A. The following table summarises the relevant oedometer test results.

<i>Borehole (Sample)</i>	<i>Elevation (Depth) (m)</i>	σ_{vo}' (kPa)	σ_p' (kPa)	<i>OCR</i>	e_o	C_r	C_c	c_v (cm ² /s)
01-02A (1)	275.9 (4.1)	52	65	1.25	2.5	0.11	1.23*	7×10^{-3}

Note: *For stress range of $\sigma_p' \leq \sigma_v' \leq 200$ kPa

where: σ_{vo}' is the effective overburden pressure in kPa
 σ_p' is the preconsolidation pressure in kPa
 OCR is the over-consolidation ratio
 e_o is the initial void ratio
 C_r is the recompression index
 C_c is the compression index
 c_v is the coefficient of consolidation in cm²/s

A grain size distribution curve for a selected sample of this deposit is shown on Figure A8. The natural water content measured on samples of the organic sandy silt range from about 63 to 97 percent, with an average of about 76 percent.

4.2.6 Varved Silty Clay

On the south side of the Tomiko River, beneath the organic sand silt layer in Boreholes 01-02, 01-2A and 01-03, and underlying the sand and gravel in Boreholes 01-01 and 01-40, a deposit of varved silty clay was encountered. The deposit is described as grey varved silty clay with trace sand and some organics. The varves are composed of dark grey clay laminae approximately 5 mm to 8 mm thick and light grey silt laminae approximately 4 mm to 8 mm thick. The elevation of the top of this layer ranged from 276.1 m and 277.1 m and the layer has a thickness ranging from 0.5 m to 0.9 m.

Standard Penetration Testing (SPT) measured 'N' values ranging from 1 blow per 0.3 m of penetration to 10 blows for 0.15 m of penetration, suggesting a very soft to firm consistency. An in-situ vane test carried out in Borehole 01-01 at about Elevation 275.4 m measured an undrained shear strength of about 55 kPa. However, it is likely that this test was affected by the presence of the underlying bedrock that exists at about the same elevation as the test. Three laboratory vane shear tests, carried out on the Shelby tube sample obtained from this deposit in Borehole 01-02A between Elevations 275.1 m and 275.6 m, measured undrained shear strengths ranging from about 16 kPa to 12 kPa with an average of about 13 kPa. The remoulded strength measured in one of the laboratory vane shear tests was about 6 kPa. The results of these tests indicate that the varved silty clay has a very soft to soft consistency and a sensitivity of about 2, implying a clay of medium sensitivity based on the classification system provided in CFEM (1992).

Atterberg limits testing was carried out on three specimens obtained from the varved silty clay deposit. The test results are summarized in the following table.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
01-01	6	276.2 – 275.6	35	20	15
01-02	5	276.2 – 275.6	34	18	15
01-02A	2	275.6 – 275.1	41	18	22
Average	-	-	36	19	18

Based on these results, the varved silty clay has a low to intermediate plasticity. The results of the tests are plotted on the plasticity chart on Figure A9 in Appendix A.

An organic content of about 2 percent was measured on a sample from this deposit and the specific gravity is estimated to be about 2.7 based on the results of one test. The measurements conducted on a carefully trimmed sample of the varved silty clay to estimate the natural bulk unit weight resulted in a value of about 17 kN/m³.

A laboratory oedometer (consolidation) test was carried out on a specimen of the varved silty clay obtained from Borehole 01-02A. A pre-consolidation pressure of approximately 40 kPa for this stratum is estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Details of the test results are shown on Figures A10 and A11 and included on the associated oedometer consolidation summary sheets in Appendix A. The following table summarises the relevant oedometer test results.

<i>Borehole (Sample)</i>	<i>Elevation (Depth) (m)</i>	σ'_{vo} (kPa)	σ'_p (kPa)	<i>OCR</i>	e_o	C_r	C_c	c_v (cm ² /s)
01-02A (2)	275.5 (4.5)	53 ⁺	40	1.0 ⁺	1.4	0.04	0.57*	5 x 10 ⁻³

Note: * For stress range of $\sigma'_p \leq \sigma'_v \leq 200$ kPa
⁺ σ'_{vo} (and OCR) are affected by varying ground water levels at the site

A grain size distribution curve for a selected sample of this deposit is shown on Figure A12. The natural water content measured on samples of the varved silty clay range from about 43 to 49 percent, with an average of about 47 percent.

4.2.7 Sandy Silt

The varved silty clay stratum in Borehole 01-03 and the organic sandy silt layer in Borehole 01-41 are underlain by a deposit of sandy silt containing trace to some gravel. The thickness of this deposit was measured to be 0.3 m and 0.7 m in Boreholes 01-03 and 01-41, respectively. The top of this layer was encountered between about Elevations 276.5 m and 276.0 m.

Standard Penetration Testing (SPT) measured an 'N' value of 61 blows per 0.22 m to 50 blows per 0.08 m of penetration suggesting a very dense state of packing within this deposit. A natural water content of 18 percent was measured on a sample of the sandy silt.

4.2.8 Bedrock

Bedrock was encountered and cored in Boreholes 01-01, 01-02, 01-04, 01-05, 01-39 and 01-41 advanced during the 2001 investigation, and in Boreholes 60-1 to 60-5 advanced during the 1960 investigation. In Boreholes 01-02A, 01-03, 01-06, 01-40 and 01-42 the presence of bedrock was inferred from refusal to further drilling advance. The surface of the bedrock varies

between Elevations 276.3 m and 275.4 m as encountered in the boreholes on the south side of the river, and between Elevations 274.6 m and 271.7 m in the boreholes on the north side.

The bedrock is described as fresh, foliated, very strong, dark grey to pink to white, schistose gneiss. The Rock Quality Designation (RQD) is ranges from about 65 to 90 percent indicating a rock of fair to excellent quality.

4.2.9 Groundwater Conditions

Piezometers were installed within the sand and gravel and sand deposits in Boreholes 01-01 and 01-04 located at the south and north abutments, respectively, to allow groundwater monitoring. The water level measured in the piezometers was at Elevation 278.8 m in Borehole 01-01 (2.0 m below the ground surface) and 278.5 m in Borehole 01-04 (3.0 m below the ground surface) on June 11, 2001. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

Based on the topography and subsurface conditions at the site, it is considered that the groundwater level is controlled by the Tomiko River water level. The water level of the Tomiko River was measured to be at about Elevation 278.8 m on June 22, 2001 which is at or about the elevation measured in the piezometers.

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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation and our recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

5.1 General

The overall project involves the upgrading of about 15 km of Highway 11 in an area located approximately 20 km north of North Bay, Ontario. As part of the upgrading, improvements to the horizontal and vertical roadway alignment require realignment and widening of the existing highway. The cross-section widening and associated centreline shift of approximately 6.0 m will occur to the east for the southern and northern sections of the project, and to the west for the central portion. The centreline shift results in the requirement for a widening or replacement of the existing three-span bridge structure at the Tomiko River crossing in Lyman Township. It is understood that the preferred option at this site involves replacement of the existing structure with a new, 29.5 m long single-span bridge. It is further understood that a detour structure will not be required during construction.

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) to provide recommendations on the foundation aspects related to the final design and construction of the bridge foundations and approach embankments for the Tomiko River structure on Highway 11. The scope of work includes provision of capacities and founding elevations for the structure foundation units; earth pressure design for abutments; stability and settlement analysis of approach embankments; recommendations for stable embankment geometry, fill materials used, implementation of ground improvement techniques that may be required; scope of instrumentation and means to minimize settlements (if required). It also includes addressing specialized construction concerns and addressing potential geotechnical problems including construction dewatering (where applicable), subexcavating soft / organic materials, and placement of fill materials. These requirements are addressed in the following sections.

5.2 Bridge Foundations

The subsoils encountered at the site (i.e. loose to dense sands and gravels and soft to very soft organic silt and varved silty clay) are not considered suitable to support shallow spread footings. It is noted that the existing bridge structure is also pile supported. Deep foundations are recommended for support of the abutments, such as steel H-piles driven to practical refusal in bedrock, or steel tube piles founded on the bedrock.

Based on the General Arrangement Drawing provided by MRC on October 26, 2001, the underside of the pile cap will be at Elevations 278.6 m and 278.9 m at the north and south abutments, respectively. For protection from frost action, the base of the pile caps and other footings should be provided with a minimum soil cover of 2.0 m based on OPSD 3400.010.

It is estimated that the surface of the gneiss bedrock will be encountered at approximately the following elevations:

<i>Foundation Unit</i>	<i>Bedrock Elevation (m)</i>
South Abutment	276 – 275
North Abutment	275 – 271

The feasibility of an integral abutment structure for this site is dependent on pile length. Based on the pile cap elevations and the available subsurface information, it is estimated that piles founded on the surface of the bedrock would be 3 m to 4 m long at the south abutment, and 3.5 m to 7.5 m long at the north abutment. It is understood that MTO requires that piles be a minimum of 5 m in length for integral abutment structural considerations. As a result, some pre-drilling of the bedrock will be required in order to socket the H-piles or tube piles within the bedrock and achieve the required length, particularly at the south abutment.

The construction of sockets in the strong gneiss bedrock will require specialized construction techniques, such as core drilling or percussion drilling with a down-hole hammer. The socketting requirement and the difficulties associated with socket construction may confer an economic advantage on the smaller diameter steel tube piles, as compared with H-piles.

5.2.1 Pile Axial Resistance

For HP 310 x 79 piles driven to refusal or founded socketted in the gneiss bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 1,450 kN may be assumed for design. Alternatively, HP 310 x 110 piles may be used and a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. Where the piles are driven rather than placed in pre-drilled holes, the heavier pile section would be better able to cope with the cobbles in the sand and gravel subsoil during driving.

For 244 mm diameter steel tube piles filled with concrete and founded on or socketted into the strong gneiss bedrock, the axial resistance at ULS will be governed by structural considerations.

The above factored axial resistance values take into account the structural capacity limitation of the pile, and potential difficulties that the pile may have seating into the bedrock surface. The H-pile tips should be suitably reinforced with driving shoes to ensure penetration and adequate seating as per current MTO practice (Standard OPSD 3301.00 and OPSS 903.07.02.05).

Serviceability Limit State (SLS) values are not provided because the gneiss bedrock is considered to be an unyielding material. Under this condition and for the above noted pile sections and estimated pile lengths of less than 10 m, the SLS value (for 25 mm of settlement) does not govern design because the SLS value is higher than the ULS value.

5.2.2 Downdrag Load (Negative Skin Friction)

As will be discussed in Section 5.4.2, the construction and widening of the approach embankments to the Tomiko River structure will cause consolidation settlement of the organic sandy silt and varved silty clay strata encountered at depth on the south side of the Tomiko River. These soil types were not encountered in Boreholes 60-1 and 60-2, advanced by Dominion Soil Ltd. on the south side of the river. However, it is possible that the soft, compressible deposits encountered in the current investigation are continuous beneath the existing south approach embankments and may have been missed by the non-continuous sampling performed during the 1960 investigation. The soft, compressible deposits were also not encountered in any of the 2001 or 1960 boreholes drilled on the north side of the river.

Since the abutment piles will be end-bearing on bedrock, a small amount of settlement of the compressible strata relative to the pile will result in the development of negative skin friction acting on the piles. The consolidation settlement is time-dependent; however, due to the relative thinness of the compressible deposits, it may be possible for the majority of the settlements to

<i>Pile Type</i>	<i>Foundation Unit</i>	<i>Nominal (unfactored) Downdrag Load</i>
HP 310 x 79 or HP 310 x 110	South Abutment	200 kN
244 mm Diameter Tube Pile	South Abutment	125 kN

Although the downdrag load is estimated to be the same for both HP 310 x 79 and HP 310 x 110 pile sections, it is noted that the proportional reduction in capacity for the heavier pile section would be less.

It may be possible to limit the amount of clay settlement caused by the widening of the south approach embankment after pile installation and therefore reduce the downdrag loads on the new piles. This option will be discussed in Section 5.4.2.1. In the absence of such measures, the south abutment piles should be designed to carry downdrag loads.

5.2.3 Set Criteria

As noted in Section 5.2, some of the piles will need to be placed in pre-bored holes in the bedrock. This is necessary to facilitate minimum pile length requirements for integral abutments. The application of set criteria is not applicable for these piles.

At some locations, integral abutment support can be achieved by driving piles to seat on or near the bedrock surface. Set criteria for driven piles are highly dependent on pile driving hammer type and selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer, and traditional use where a substantial database has been developed over the years. The criteria should also be set to avoid overdriving and damage to the piles.

Provision should be made to re-tap piles to confirm the set after adjacent piles have been driven.

All pile driving should be in conformance with SP 903S01.

5.2.4 Pile Driving Note

The pile driving note to be added to the drawings is Note 4 in Clause 2.5.11 of the Structural Manual – “Piles to be driven to or placed on bedrock”.

5.2.5 Horizontal Resistance

The design of piles subjected to lateral loads (such as loading due to thermal expansion and contraction of the bridge superstructure, or ice loads) should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil and rock resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

The horizontal resistance at Ultimate Limit States (ULS) provided by the overburden soils can be estimated using Broms' method as described in the Canadian Foundation Engineering Manual (CFEM, 1992). For vertical piles driven or augered to bedrock through the loose to dense sand and gravel at this site, the following parameters may be assumed for use with Broms' solution:

- Soil unit weight (γ) = 20 kN/m³
- Coefficient of passive earth pressure (K_p) = 3.7

The horizontal resistance at ULS shall be taken as the lesser of the 'short' pile or the 'long' pile case; however, in general, the lateral resistance of HP 310 x 79 piles less than 6 m in length may be considered to be controlled by 'short' pile behaviour. For piles in groups, the horizontal resistance at ULS should be taken as the lesser of:

- The sum of the ultimate lateral resistances of the piles in the group.
- The ultimate lateral load of an equivalent block representing the group.

The ULS values calculated using Broms' method should be checked against the assessed horizontal passive resistance values given in Table C6-9.8.1(a) of the OHBDC.

Where the H-piles or tube piles are required to be socketted into rock in order to achieve the necessary pile lengths, a minimum embedment depth of 1 m below the bedrock surface should be provided. The factored lateral resistance at ULS provided by the rock socket may be taken as:

<i>Rock Embedment Depth</i>	<i>HP 310 x 79 or HP 310 x 110 Piles</i>	<i>244 mm Diameter Tube Piles</i>
1 m	7,500 kN	5,000 kN
2 m	15,000 kN	10,000 kN

The above lateral resistances are based on socket diameters of 500 mm and 350 mm for the H-piles and tube piles, respectively.

At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections of the pile heads being too large to be compatible with the superstructure. In this case, the horizontal resistance of the pile is calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil.

For cohesionless soils, the horizontal soil reaction to a vertical pile can be estimated using the following formulae:

$$k_h = n_h \frac{z}{d}$$

where: k_h = coefficient of horizontal subgrade reaction (kPa/m)
 n_h = coefficient related to soil density (kN/m³)
 z = depth (m)
 d = pile diameter (m)

For cohesive soils, the horizontal soil reaction to a vertical pile can be estimated using the following formulae:

$$k_h = 67 \frac{c_u}{d}$$

where: k_h = coefficient of horizontal subgrade reaction (kPa/m)
 c_u = undrained shear strength of soil (kPa)
 d = pile diameter (m)

The piles will be driven mainly through the predominantly loose to compact sand and gravel subsoils encountered at the site. The abutment piles will be driven through the embankment fill. The south pier and south abutment piles will also be driven through the very soft to soft organic sandy silt and silty clay deposits encountered at depth. The following table summarizes the values of n_h and c_u that may be assumed in the design of the piles.

<i>Soil Unit</i>	<i>n_h (kN/m³)</i>	<i>c_u (kPa)</i>
embankment fill (loose to compact)	4,000	-
sand and gravel (loose to compact)	3,000	-
organic sandy silt (very soft to soft)	-	14
varved silty clay (very soft to soft)	-	8

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

<i>Pile Spacing in Direction of Loading d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.2.6 Construction Considerations

The boreholes drilled in the 2001 and 1960 investigations encountered cobbles within both the sand and gravel embankment fill and the native sand and gravel subsoils existing at the site. A boulder was encountered within the sand and gravel deposit immediately overlying bedrock in Borehole 60-4. In addition, an approximately 1.5 m thick layer of cobbles with sand and gravel was encountered immediately below the ground surface in Borehole 01-05. The presence of cobbles would likely make the driving of piles for the foundation units difficult. Provision should be made for an allowance to pre-auger through the upper 1 m to 2 m of soil at each foundation unit, prior to driving the piles to bedrock where this foundation option is adopted.

The gneiss bedrock at this site is strong, and special construction techniques will be required where socketting of the H-piles or tube piles is required. It is considered that coring or percussive drilling could be adopted at this site to form the bedrock sockets.

5.3 Lateral Earth Pressure

The lateral pressures acting on the 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 retaining walls in accordance with OHBDC.

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B', Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to at least 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501.
- Longitudinal drains and weep holes should be installed in concrete walls to provide positive drainage of the backfill.
- The granular fill may be placed either in a zone with width equal to at least 2.0 m behind the back of the wall (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a 1.5 horizontal to 1 vertical line extending up and back from the bottom of the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).
- 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 kPa should be included in the lateral earth pressures for the structural design of abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be restricted as per 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: 21 kN/m³
 [assuming compacted clean earth fill such
 as Select Subgrade Material (OPSS 1010)]

Coefficients of lateral earth pressure:
 'active' 0.28
 'at rest' 0.44

- 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' <i>Type II</i>
Soil unit weight	22 kN/m ³	21 kN/m ³
Coefficient 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 abutment wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and OPSD 3504.00.

5.4 Approach Embankments

Assuming a finished road surface elevation of about 284 m on the new Tomiko River structure, and based on the existing ground surface elevations as measured at our borings adjacent to the existing bridge, it appears that as much as approximately 4 m of fill is required for the widening of the existing approach embankments.

It is noted that based on the cross-section information provided by MRC, the existing approaches have relatively flat side slopes. On the north side of the bridge, the side slopes vary from approximately 3H:1V to 4H:1V. On the south side of the bridge, the east side of the existing

embankment has a side slope profile of about 3.5H:1V and the west side has a side slope profile of about 6H:1V.

The following sections present the results of stability and settlement analysis associated with the proposed widening of the existing approach embankments to the Tomiko River Bridge as part of the realignment of Highway 11.

5.4.1 Stability

Stability analyses were performed on the critical (i.e. highest) sections of the proposed new approach embankments after widening. As a result of the different subsoil conditions existing on the north and south sides of the river, both the north and south approach embankments were analysed individually using the limit equilibrium method to assess the effects of the proposed new construction.

All slope stability analyses were performed using the commercially available program SLOPE/W (Version 4.0), produced by Geo-Slope International Ltd., employing the Morgenstern - Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions.

The piezometric conditions used in the analyses were based on the groundwater levels noted during drilling and measured in the standpipe installations, as well as the level of the adjacent Tomiko River. In the area at and behind the abutments, the groundwater level was conservatively assumed to be at the ground surface (at about Elevation 280.0 m). The groundwater level was assumed to decrease from Elevation 280.0 m at the abutments to about Elevation 279.0 m in the area of the river.

For the analysis of the approach embankment side and forward slopes on the south side of the river, the average mobilized undrained shear strength (c_u) of the silty clay stratum was assessed using the results of the field vane tests and lab vane tests and inferred from the oedometer (consolidation) test results. The average strength of the organic silt stratum was inferred from the results of the oedometer test. For the oedometer tests, the following correlation proposed by Mesri (1975) was employed to estimated undrained shear strength:

$$c_u = 0.22\sigma_p'$$

where: c_u = average mobilized undrained shear strength (kPa)
 σ_p' = preconsolidation pressure (kPa)

The effective stress parameters for the cohesionless soils existing on both sides of the river were estimated from empirical correlations using the results of in-situ SPT tests. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed and the results tempered by engineering judgement based on our experience in similar soil conditions. The following table summarizes the strengths and unit weights employed for the different soil types in the analyses.

<i>Soil</i>	<i>Unit Weight (kN/m³)</i>	<i>Strength</i>
Sand and Gravel (fill)	20	$c' = 0$ kPa, $\phi' = 32^\circ$ to 35°
Rock fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Cobbles with Sand and Gravel	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Sand and Gravel trace Silt	20	$c' = 0$ kPa, $\phi' = 32^\circ$ to 35°
Silt and Sand	19	$c' = 0$ kPa, $\phi' = 33^\circ$
Organic Sandy Silt *	13.4	$s_u = 14$ kPa
Varved Silty Clay *	16.7	$s_u = 8$ kPa

Note: * strata only encountered on south side of Tomiko River

In the analyses, two different types of fill (i.e. rock fill and earth fill) were considered for the required embankment widenings. The fill alternatives provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to native subsoils), construction cost and time, ease of construction, and availability.

The use of conventional earth fill (sand and gravel) for embankment construction offers the advantage of ease of construction. However, this option will require a larger volume of fill and wider right-of-way because the embankment side slopes tend to be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and / or imported, free-draining granular material.

The rock fill embankment option would employ rock fill in place of earth fill for the embankment outside of the immediate abutment area where piling is to be driven or augered. The main

advantage of this material is the ability to achieve steeper embankment side and forward slopes. This is useful in areas with limited right-of-ways. In addition, surplus rock fill, if available to this project, could have a cost advantage. The use of rock fill, however, may hinder other construction such as the installation of roadway protection as discussed in Section 5.7.

The results from selected stability analyses of the north approach embankment section are shown on Figure 2. Selected results from the stability analyses of the south approach embankment, in section and in profile, are shown on Figures 3 and 4, respectively.

The results of the analyses for the earth fill and rock fill options are summarized in the following table. On each side of the Tomiko River, the highest (i.e. most critical) approach embankment section has been analysed. The side and forward slope profiles and the associated factor of safety for each are presented.

<i>Location</i>	<i>Embankment Height (m)</i>	<i>Earth Fill Option</i>		<i>Rock Fill Option</i>	
		<i>Side and Forward Slope Profile</i>	<i>Factor of Safety</i>	<i>Side and Forward Slope Profile</i>	<i>Factor of Safety</i>
South Approach	4.0	2H:1V	1.5	1.25H:1V	1.4
North Approach	4.0	2H:1V	1.5	1.25H:1V	1.7

The minimum factor of safety at each location summarized above is based on a failure surface that would impact the operation of the roadway.

Therefore, for the rock fill option, side and forward slopes no steeper than 1.25H:1V are recommended for both the north and south approach embankments. For the earth fill option, side and forward slopes no steeper than 2H:1V are recommended for both the north and south approach embankments. To ensure the stability of the abutment foreslopes, erosion and scour protection should be placed at the slope toes and at the banks of the Tomiko River.

Since the approach embankments are less than 6 m in height, the incorporation of a 2 m wide bench (or berm) into the uniform side or forward slope profiles is not required.

5.4.2 Settlement

Analysis has been performed at the south and north approach embankments to assess the settlement of the subsoils below the additional fill as a result of the proposed widenings. The

analyses were based on the information obtained during the current investigation including the results from the boreholes, in-situ tests, and oedometer tests.

The settlement analyses were carried out using the commercially available program UNISSETTLE (Version 3.0) produced by Unisoft Limited. The location of the existing embankments relative to the proposed Highway 11 realignment requires that the new approaches be constructed by widening the adjacent approaches. As such, the geometry and loading of the existing highway was included in the settlement analysis. Based on the stability considerations discussed in the previous section, the embankment widenings were analysed with 2H:1V side slopes.

For the analysis of the south approach, an important element in estimating the settlement due to the consolidation of the soft, compressible layers is the degree to which the organic silt and varved silty clay strata are over-consolidated. The over-consolidation ratio (OCR) for these deposits was established using the results of the oedometer tests.

The following parameters for the soft, compressible layers were used in the analysis:

<i>Soil Unit (Thickness)</i>	<i>Unit Weight (kN/m³)</i>	<i>OCR</i>	<i>Initial Void Ratio</i>	<i>Recompression Index C_r</i>	<i>Compression Index C_c</i>	<i>Coefficient of Consolidation c_v (cm²/s)</i>
Organic Silt (0.8 m)	13.4	1.25	2.5	0.11	1.23	7×10^{-3}
Varved Silty Clay (0.8 m)	16.7	1.0	1.4	0.04	0.57	5×10^{-3}

The immediate compression of the loose to compact sand and gravel, and sand and silt layers overlying the organic silt and varved clay at the south approach was modelled assuming an estimated elastic modulus of deformation of 10 MPa based on SPT 'N' values and a unit weight of 20 kN/m³. For the north approach, an estimated elastic modulus of 15 MPa based on SPT 'N' values was assumed for assessing the immediate settlement of the sand and gravel and sand subsoils due to embankment widening. The fill for the new embankment construction and the sand and gravel fill of the existing highway embankments were assumed to have a unit weight of 20 kN/m³.

Based on the results of the analysis, the following conclusions are made:

- For the south approach at the abutment, the maximum total settlement is estimated to be in the range of 10 mm to 15 mm (from the centreline of the new roadway to the east edge of pavement) and 25 mm to 35 mm (from the new west edge of pavement to west crest

of slope). For the primary consolidation component of the settlements, it is estimated that about 95 percent of the settlement will be completed in about 1 month.

- For the north approach at the abutment, the maximum total settlement is estimated to be approximately 15 mm to 20 mm, which is expected to occur during the construction widening of the approach.

5.4.3 Mitigation of Time Dependent Settlements

The estimated time dependent settlement at the south approach can result in a differential settlement causing a bump at the bridge abutment and the generation of downdrag loads on the south abutment piles as discussed in Section 5.2.1.1. Therefore, consideration should be given to following a construction sequence to limit the maintenance and mitigate the downdrag loads on the new piles.

Since it is estimated that 95 percent of the consolidation settlement of the soft compressible layers will occur within about 1 month of placement of the embankment fill, it is recommended that the south approach embankment be constructed as early as possible to full height (i.e. at the beginning of the construction contract) and allowed to consolidate for at least 6 weeks prior to excavation to pile cap level and installation of the bridge abutment piles. Following this sequence will reduce the post-construction settlements and maintenance concerns at the bridge. In addition, installing the abutment piles after 95 percent of the consolidation is complete will eliminate the need to include the downdrag loads in the design of the abutment pile foundations.

However, it should be noted that regardless of the construction sequence followed, some settlement of the existing south approach embankment will occur as a result of the widening. This settlement will induce downdrag loads on the existing abutment piles while the existing structure remains in place during construction. The magnitude of the downdrag load on the existing H-pile foundations should be taken as 250 kN in the structural evaluation. It should also be recognized that some maintenance at the existing approach will be required once the new embankment fill has been placed, while the existing structure remains in use.

Consideration will have to be given to the type of fill material used in the construction of the embankment and the practicality / ease of installing the abutment piles through the fill. Earth fill should be used to construct the approach embankment widenings in the piling area to facilitate and ease pile installation.

5.5 Subgrade Preparation and Embankment Construction

As discussed in Section 5.1, widening of the existing Highway 11 approach embankments to the Tomiko River Bridge will be required in order to accommodate the new roadway alignment. Prior to the placement of any fill for the widenings, all surficial deposits of topsoil and organic should be stripped from the plan limits of the proposed works.

For the existing sand and gravel fill embankments, benching into the existing side slopes should be carried out as per OPSD 208.010 during the construction and widening.

If earth fill is used for the embankment construction (as recommended in Section 5.4.2.1) the placement of all granular fill material for the embankment and roadbase construction should be carried out in accordance with OPSS 206.07.07, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the materials' Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

If rock fill is employed for the embankment construction, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision, Amendment to OPSS 206 dated September 1999. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading and dozing the rock to form a dense, compact mass.

Assuming the use of earth fill for the approach embankment widening, final side slopes should be no steeper than 2H:1V. If rock fill is employed, final side slopes should be no steeper than 1.25H:1V.

Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion.

5.6 Excavations and Groundwater Control

Local excavations will be required at the abutment locations in order to construct the pile caps. Based on the General Arrangement Drawing provided to us by MRC on October 26, 2001, the

underside of the pile caps will be at about Elevations 278.6 m and 278.9 m at the north and south abutments, respectively. As such, it is anticipated that the excavations at the foundation units will extend to about these elevations.

The results of the June 11, 2001 monitoring of the water levels in the piezometers installed at the site indicate that the groundwater level is, on average, at about Elevation 278.7 m in the vicinity of the bridge. This is consistent with the level of the Tomiko River measured at Elevation 278.8 m on June 22, 2001.

The excavations at the abutments will penetrate either the existing loose to compact sand and gravel embankment fill material and / or the native loose to compact sand and gravel strata. These fills and native soils are classified as Type 3 soils according to the Occupational Health and Safety Act of Ontario. All excavations should be carried out according to the latest edition of the Occupational Health and Safety Act.

Based on the above, it appears that the bottom of the excavation required at the abutments will be at or slightly below the current level of the groundwater table. It should be noted, that the water level of the Tomiko River and the elevation of the groundwater table in the areas adjacent to the bridge will be subject to seasonal variations. It is possible that at certain times of the year, the level of the groundwater table could be above the elevation of the underside of the pile caps.

Groundwater control is the responsibility of the contractor. The groundwater level should be measured by the contractor prior to excavation to establish its location at that time. The degree of groundwater control required should then be assessed and implemented by the contractor. A specialist dewatering firm/contractor should be retained to design and implement appropriate groundwater control. The Contract should make provision for groundwater control, including the requirements of OPSS 518 and OPSS902/SP902.07.03. The following discussion and comments are provided for the benefit of design engineers to make them aware of the potential need for groundwater control when the specifications are prepared.

If the groundwater table is below the founding level of the abutments at the time of construction, no significant groundwater problems are anticipated during the required excavations. Temporary unsupported excavations with side slopes no steeper than 2H:1V should be stable during normal construction duration, although some surficial sloughing may be experienced if the bottom of the excavation reaches the groundwater table. Conventional excavation equipment should be suitable for excavating the soils, although boulders and cobbles should be expected near the surface in some areas which may slow progress. All surface water must be directed away from the excavation and not permitted to enter the excavation.

If the water level in the Tomiko River and the groundwater table in the adjacent areas are above the founding level of the abutments at the time of construction, the bottom of the excavations will be below the groundwater table. In this case, unsupported excavations into the loose to compact, cohesionless soils will not be possible. Conventional groundwater control to lower the water table below the base of the required excavation may not be practical at the site due to the permeable nature of the native soils and the close proximity to the Tomiko River.

For construction under high groundwater conditions, sheet pile cofferdam around the perimeter of the area requiring excavation may be required. The sheet-piles should be interlocking and driven to bedrock. Some difficulties associated with the installation of the sheet piling should be expected due to the presence of cobbles and boulders at some areas.

It is noted that for the conditions and relatively shallow excavation depths assumed above, the potential for a base heave type failure to occur during excavation for the pile cap installation is considered to be low. If the conditions change, or deeper excavations are required, the stability of the excavation and the likelihood of base heave should be reviewed by the design engineer with foundations input and assistance as appropriate.

5.7 Temporary Shoring

Based on the configuration of the existing and proposed highway approach embankments and the foundations for the existing bridge, which will remain in place during the initial stages of the new bridge construction, there may be a requirement for the contractor to install temporary shoring to facilitate construction of abutment walls and pile caps. Protection to any underground service in the vicinity of the excavation would also need to be provided.

A cantilever shoring system may be feasible at this site. However, depending on the final configuration, some form of support would be required near the top of the shoring system. The use of rakers to provide support would be hindered by tight space restrictions. The use of soil anchors is not feasible given that the overburden is not very thick and is generally weak. Anchors to bedrock would be possible at this site.

Consideration could be given to the use of a deadman anchor system (in conjunction with steel sheet piling or a soldier pile and lagging system) to provide adequate temporary support. The use of steel sheeting could be hindered by the presence of rock fill, cobbles and possibly boulders, which would affect driving of the sheeting. The deadman anchorage system could be connected to the temporary shoring by horizontal anchors installed within the embankment.

The shoring system should be designed to resist a rectangular lateral earth pressure distribution as given by the equation:

$$p_h = K(\gamma H + q)$$

where	p_h	=	lateral earth pressure (kPa)
	K	=	lateral earth pressure coefficient (0.3 for horizontal ground; 0.4 for sloping ground surface behind the shoring system)
	γ	=	unit weight of fill (assume 18 kN/m ³)
	H	=	retained height (m)
	q	=	nominal uniform surcharge loading (kPa)

The design of the shoring system should be carried out by a professional engineer experienced in the design of such works.

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JPD/LCC/DEB/FJH/clg/pds

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REFERENCES

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations, 512 p.

Dominion Soils Ltd. 1960. Report on : Foundation Investigation, Proposed Bridge over Tomiko River, Approximately 21 miles north of North Bay on Highway 11, District No. 13, W.P. 95-60.

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Mesri, G. 1975. Discussion on new design procedure for stability of soft clays. ASCE Journal of the Geotechnical Engineering Division, 101 (GT4), pp. 409-412.

Ontario Highway Bridge Design Code. 1991. Third Edition. Ontario. Ministry of Transportation. Quality and Standards Division.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Schmertmann, J.H. 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

U.S. Navy. 1971. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

APPENDIX A
LABORATORY TEST DATA

APPENDIX B

**RECORD OF BOREHOLE SHEETS
(1960, DOMINION SOILS LTD.)**

TABLE 1
Summary of Field Investigation Program
Highway 11, Tomiko River Bridge
G.W.P. 711-92-00
(W.P. 344-00-01)

Township	Borehole Number	Northing (MTM Zone 12)	Easting (MTM Zone 12)	Ground Elevation (m)	Depth (m)	Comments	
Lyman	01-01	5160213.9	294308.4	280.8	9.3	South Abutment	Widening Option
	01-02	5160221.4	294304.4	280.0	7.7	South Pier	
	01-02A	5160222.2	294302.8	280.0	4.9	South Pier	
	01-03	5160197.3	294317.3	281.1	5.0	South Approach	
	01-04	5160246.7	294284.6	281.5	12.3	North Abutment	
	01-05	5160241.5	294294.5	280.1	9.1	North Pier	
	01-06*	5160264.0	294274.8	283.2	8.2	North Approach	Replacement Option
	01-39	5160240.5	294288.4	280.5	9.1	North Abutment	
	01-40	5160217.8	294306.7	280.4	5.0	South Abutment	
	01-41	5160228.6	294320.0	280.3	7.8	South Abutment	
	01-42	5160251.4	294301.7	280.2	8.2	North Abutment	

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* Borehole advanced from Highway 11 roadway grade.

TABLE 2 (Continued)
Summary of Laboratory Tests
Highway 11, Tomiko River Bridge
G.W.P. 711-92-00
(W.P. 344-00-01)

<i>Borehole No.</i>	<i>Sample No.</i>	<i>Simplified Soil Description</i>	<i>Water Content</i>	<i>Grain Size</i>	<i>Atterberg Limits</i>	<i>Oedometer</i>	<i>Organic Content</i>	<i>Specific Gravity</i>	<i>Lab. Vane Testing</i>
01-01	3	Sand and Gravel	x						
	5		x						
	6	Varved Silty Clay	x		x				
01-02	1	Sand and Gravel	x						
	2			x					
	3		x						
	4	Organic Sandy Silt	x		x		x	x	
	5	Varved Silty Clay	x		x				
01-02A	1	Organic Sandy Silt	x	x	x	x	x	x	
	2	Varved Silty Clay	x		x	x	x	x	xxx
01-03	2	Sand and Silt	x	x					
	3	Sand and Gravel	x						
	5	Organic Sandy Silt	x						
		Varved Silty Clay	x	x					
	6	Sandy Silt	x						
01-04	2	Sand and Gravel	x						
	5	Sand	x	x					
	7		x						
	8	Silt and Sand	x	x					

TABLE 2 (Continued)
Summary of Laboratory Tests
Highway 11, Tomiko River Bridge
G.W.P. 711-92-00
(W.P. 344-00-01)

<i>Borehole No.</i>	<i>Sample No.</i>	<i>Simplified Soil Description</i>	<i>Water Content</i>	<i>Grain Size</i>	<i>Atterberg Limits</i>	<i>Oedometer</i>	<i>Organic Content</i>	<i>Specific Gravity</i>	<i>Lab. Vane Testing</i>
	9	Sand	x						
01-05	1	Sand and Gravel	x						
	3		x						
01-06*	1	Sand and Gravel Fill	x	x					
	2		x						
	3		x						
	6	Sand	x						
	7	Silt and Sand	x						
	8	Sand	x	x					
01-39	1	Sand and Gravel to Gravelly Sand	x						
	2		x	x					
	3		x						
	4	Silt and Sand	x						
01-40	1	Sand and Gravel	x						
	2		x						
	3	Varved Silty Clay	x						
01-41	1	Sand and Gravel	x						
	2	Organic Sandy Silt	x						
	3	Sandy Silt	x						

TABLE 2 (Continued)
Summary of Laboratory Tests
Highway 11, Tomiko River Bridge
G.W.P. 711-92-00
(W.P. 344-00-01)

<i>Borehole No.</i>	<i>Sample No.</i>	<i>Simplified Soil Description</i>	<i>Water Content</i>	<i>Grain Size</i>	<i>Atterberg Limits</i>	<i>Oedometer</i>	<i>Organic Content</i>	<i>Specific Gravity</i>	<i>Lab. Vane Testing</i>
01-42	1	Sand and Gravel to Gravelly Sand	x						
	2		x						
	3		x						
	4		x	x					
	5		x						

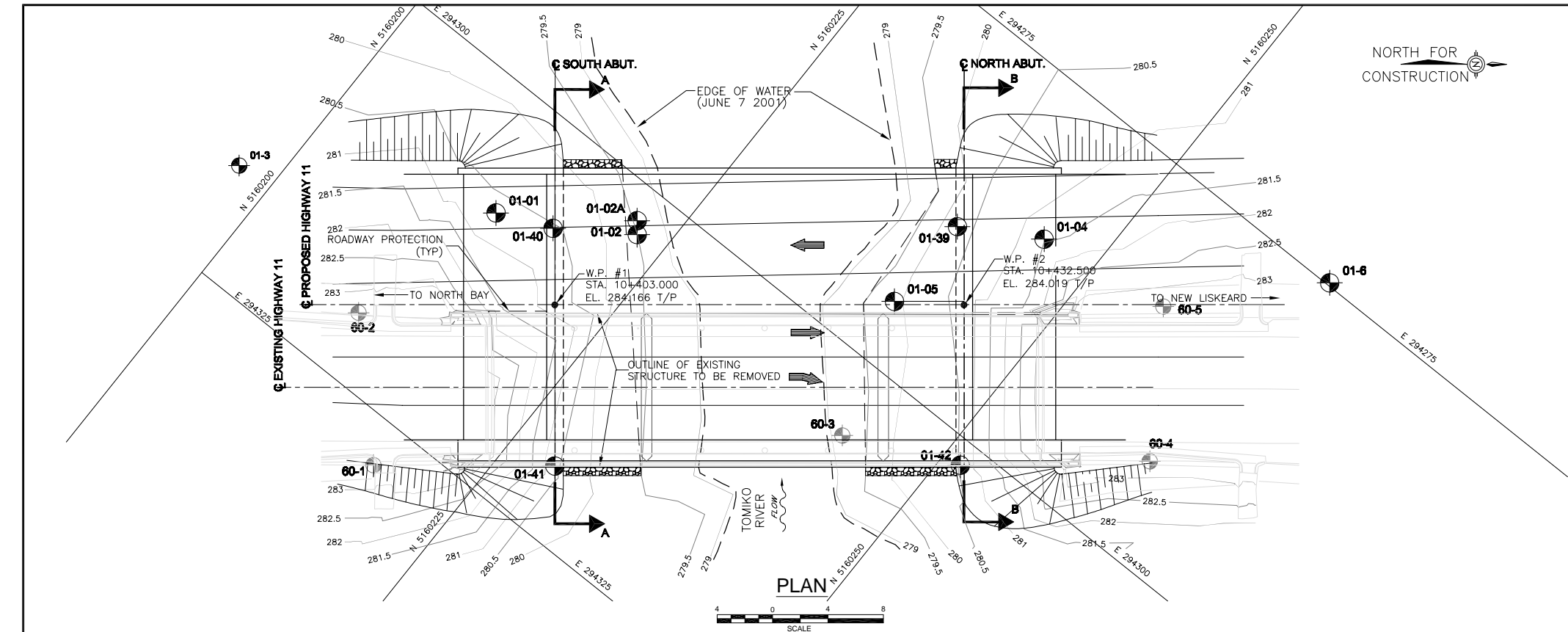
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* Borehole advanced from Highway 11 roadway grade.

SUMMARY OF NUMBER OF TESTS

Natural Water Content		42
Atterberg Limits	5	
Grain Size Distribution	10	
Specific Gravity	3	
Consolidation (Oedometer)		2
Organic Content	3	
Laboratory Vane Shear Tests	3	

Total number of samples obtained in boreholes: 48



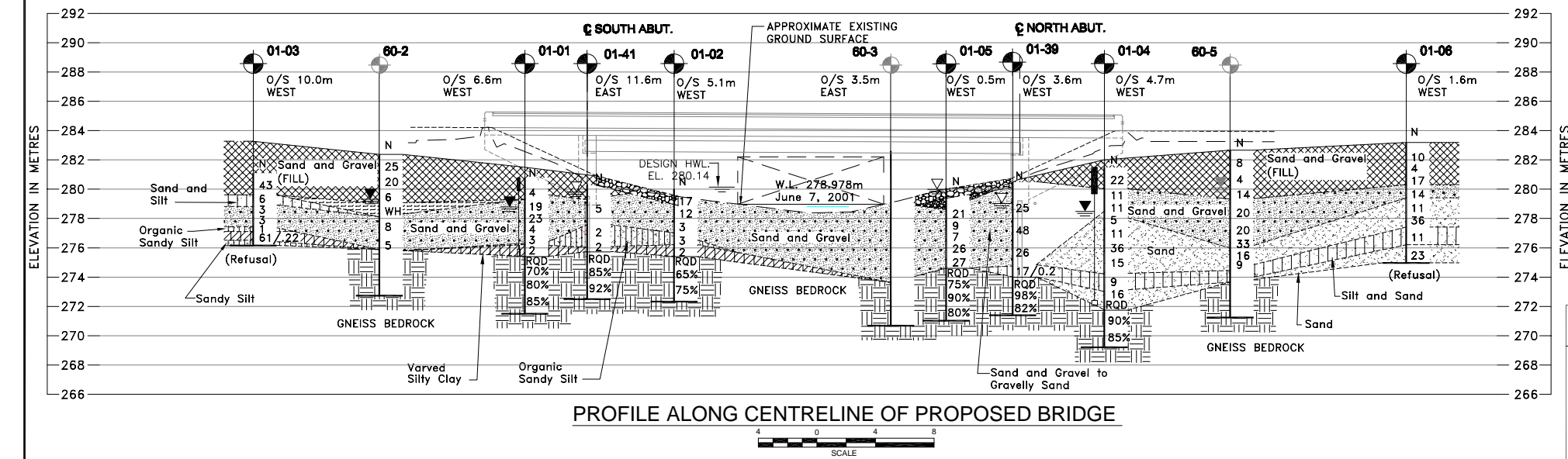
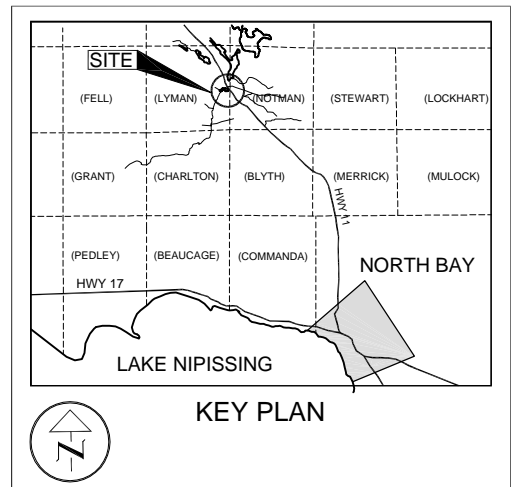
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No.
WP No. 344-00-01

TOMIKO RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



ELEVATION IN METRES

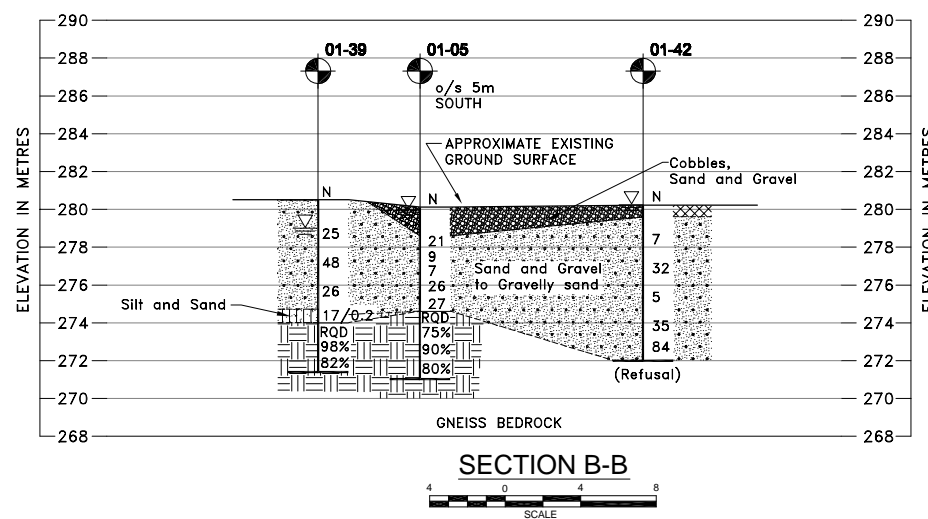
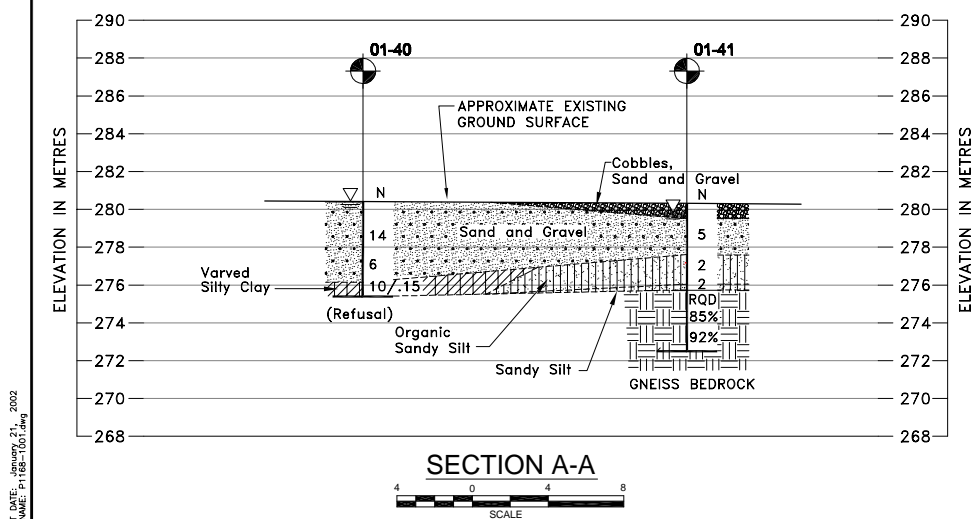
ELEVATION IN METRES

ELEVATION IN METRES

ELEVATION IN METRES

LEGEND							
	Borehole	-	Current Investigation				
	Borehole	-	Previous Report by Dominion Soil Investigation Ltd. Report dated May 1960				
	Seal						
	Piezometer						
N	Standard Penetration Test value						
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)						
100%	Rock Quality Designation (RQD)						
	WL in piezometer, June 11, 2001						
	WL upon completion of drilling						
No.	ELEVATION	LOCATION		No.	ELEVATION	LOCATION	
		NORTHING	EASTING			NORTHING	EASTING
01-01	280.8	5160213.9	294308.4	01-01	280.8	5160213.9	294308.4
01-02	280.0	5160221.4	294304.4	01-02	280.0	5160221.4	294304.4
01-02A	280.0	5160222.2	294302.8	01-02A	280.0	5160222.2	294302.8
01-03	281.1	5160197.3	294317.3	01-03	281.1	5160197.3	294317.3
01-04	281.5	5160246.7	294284.6	01-04	281.5	5160246.7	294284.6
01-05	280.1	5160241.5	294294.5	01-05	280.1	5160241.5	294294.5
01-06	283.2	5160264.0	294274.8	01-06	283.2	5160264.0	294274.8
01-39	280.5	5160240.5	294288.4	01-39	280.5	5160240.5	294288.4
01-40	280.4	5160217.8	294306.7	01-40	280.4	5160217.8	294306.7
01-41	280.3	5160228.6	294320.0	01-41	280.3	5160228.6	294320.0
01-42	280.2	5162051.4	294301.7	01-42	280.2	5162051.4	294301.7

* BOREHOLE LOCATIONS ARE APPROXIMATE ONLY



NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCES
1. This drawing was created from a digital file provided by McCormick Rankin Corporation entitled "TOMIKO RIVER BRIDGE PRELIMINARY GENERAL ARRANGEMENT" dated Oct. 2001.
2. Locations of Previously drilled (1960) Boreholes obtained from a drawing produced by Dominion Soil Investigation Ltd. Report dated May 1960

NO.	DATE	BY	REVISION
Geocres No. 31L - 80			
PROJECT NO. 001-1168-1		DIST. 54	
SUBW'D. AZ	CHKD.	DATE: JANUARY 2002	SITE: 43-10
DRAWN: JFC	CHKD.	APPD.	DWG. 1