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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

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WP	531-91-01	REGION	Central
HWY	9	STR SITE	24-40

Highway 9 - Humber River Bridge

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FOUNDATION INVESTIGATION REPORT

FOR

HIGHWAY 9 - HUMBER RIVER BRIDGE

WP 531-91-01, Site 24-40, CENTRAL REGION

1.0 INTRODUCTION

This report summarizes the results of a foundation investigation conducted for the proposed widening of the existing Highway 9 - Humber River Bridge at Ballycroy. The investigation was carried out at the request of the Central Region Structural Section. This report is applicable to a bridge widening of approximately 5m on either side of the existing structure and the immediate approaches (up to 25m) beyond the abutments.

2.0 SITE DESCRIPTION AND GEOLOGY

The site is situated at the existing Humber River Bridge located on Highway 9 approximately 0.8 km west of Highway 50 in the County of Simcoe, Regional Municipality of Peel as shown on the Key Plan in the Appendix. The existing Humber River Bridge is a single span rigid frame structure that carries Highway 9 over the Humber river. The highway is a two lane divided highway with adjoining shoulders.

The Humber River, a tributary of Lake Ontario, is a meandering river that has a width at the site of approximately 2.5 metres. The river flows in a southerly direction and the water depth is known to vary throughout the year. During the investigation the depth of the water was approximately 0.2m at the structure and approximately 1.2m immediately upstream of the structure.

The topography of the area immediately surrounding the site is generally flat. At the site, approach embankment fill slopes to the existing structure have been superimposed on the native ground surface. These slopes, constructed with a 2H:1V geometry, are in a stable condition. Immediately south of the site, the land is covered with tall coniferous trees.

Land use in the area is primarily commercial. A gas service station and restaurant is located immediately west of the site.

Physiographically, the site is located within the geological domain known as the Oak Ridges Moraine. In general, the surficial geology of the area is comprised of ice-contact stratified drift of glacial origin deposited during the retreat of the Wisconsinan glaciation approximately 12,000 years ago. At the site location, the soils consist of recent alluvium comprised of silts, sands and gravels. The soils have excellent drainage characteristics and surface drainage is obviously directed towards the Humber River.

The overburden at the site is underlain by grey and black shales of the Georgian Bay bedrock formation of the Palaeozoic era. At the site, the bedrock exists at approximate depths of 30 metres.

3.0 INVESTIGATION PROCEDURE

3.1 GENERAL

The subsurface conditions at the site were determined by a field investigation and laboratory testing program. Details of the field investigation and laboratory testing program are discussed below.

3.2 FIELD INVESTIGATION

The field investigation, which consisted of a total 4 boreholes, was carried out between December 12, 1995 and January 04, 1996. Boreholes were advanced at each quadrant of the site. All boreholes were terminated within the overburden. Survey information related to the location and the ground surface elevation of the boreholes was provided by Central Region Surveys and Plans.

The boreholes were advanced using a track mounted CME 55 diesel drilling unit to depths ranging from 24.8m to 27.6m. Both hollow stem augers and casing/washboring techniques were used to advance the boreholes. Generally, the surficial 3 metres of the boreholes were advanced with hollow stem augers followed by casing, washboring and mud drilling techniques. A head of water was used throughout the drilling operation to neutralize the unbalanced hydrostatic head condition and hence prevent soil sloughing and soil disturbance at the bottom of the borehole.

Subsoil samples of the native soils were generally retrieved at 0.76m intervals within the surficial 6m or so and at 1.5m intervals beyond this depth. Subsoil samples were taken at 0.76m to 1.5m intervals within the fill material encountered on the north side. Subsoil samples were generally retrieved in accordance with the Standard Penetration Test (ASTM D1586). All subsoil samples were identified and then sealed in plastic containers in the field to preserve their natural moisture contents. The samples were then transported to the laboratory where additional visual classifications and pertinent laboratory tests were conducted.

Groundwater levels were obtained by measuring the levels in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled at the completion of the fieldwork.

3.3 LABORATORY ANALYSES

All subsoil samples were carefully examined in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. In view of the cohesionless nature of the native subsoils at the site, laboratory testing consisted of particle size analysis and natural moisture contents. Laboratory test results are shown on the individual borehole logs and figures in the Appendix. The laboratory test results have been summarized in the subsequent section entitled "Subsurface Conditions".

4.0 SUBSURFACE CONDITIONS

4.1 GENERAL

The ground surface elevation at the boreholes advanced at the site range from approximately 282.7m to 285.6m reflecting the placement of fill material at the approaches to the existing bridge. The fill material is comprised of an irregular mixture of Silt, Sand and Gravel, and is of a thickness ranging from 2.9m to 4.4m.

Underlying the fill material and present surficially across the remainder of the site is a cohesionless deposit of Silt, some sand to Sand, some silt which varies randomly in silt and sand composition. Traces to some gravel are also present within various zones of this deposit. This deposit has a thickness ranging from 22.3m to 23.9m extending to an elevation of 260.4m to 263.3m. The denseness of this material varies from very loose to very dense.

The surficial native deposit of Silt, some sand to Sand, some silt is underlain by a cohesionless Heterogenous Mixture of Silt, Sand and Gravel (Glacial Till). This deposit has a very dense state of denseness.

Groundwater Elevations at the site were approximately $282.5 \pm$ at the time of the investigation.

A plan of the site illustrating the locations and elevations of the boreholes is shown on Dwg. No. 5319101-A. Stratigraphical sections illustrating the subsurface conditions at the site are also provided in the Appendix. The boundaries between the various soil types, in situ and laboratory test results and groundwater levels established at the time of the investigation are shown on the stratigraphical sections and also on the individual record of borehole sheets.

4.2 IRREGULAR MIXTURE OF SILT, SAND AND GRAVEL (FILL MATERIAL)

An irregular mixture of Silt, Sand and Gravel (Fill Material) exists adjacent to the wingwalls of the structure and comprises the approach embankments at the site. Although this material is only identified in boreholes #3 and #4, it does exist adjacent to the wingwalls in all four quadrants of the site. This fill material explored up to 4.2 metres also contains traces of organics.

Figure 1 in the Appendix illustrates grain size distribution curves produced by mechanical sieve analysis. The grain size distribution envelope illustrates a broad gradation of silt, sand and gravel.

The 'N' values obtained from the Standard Penetration Test range from 2 blows/0.3m to 14 blows/0.3m which indicates a very loose to compact denseness.

4.3 SILT, SOME SAND TO SAND, SOME SILT

The native subsoil at the site that underlies the fill material or exists surficially across the site consists of Silt, some sand to Sand, some silt. The thickness of this deposit varies from approximately 17.9m on the north side to 22.3m on the south side of the site. A trace to some gravel is present in localized zones throughout this soil stratum.

A grain size distribution envelope for this soil stratum is illustrated in Figure 2 of the Appendix. The gradations reveal a wide range in silt and sand composition. With silt percentages ranging from 3.9% to 94.4% and sand percentages ranging from 5.6% to 57.2%.

The 'N' values obtained from the Standard Penetration Test also reveal a wide range of values from 1 blow/0.3m to 127 blows/0.3m. Although the drilling and sampling procedure was carried out to minimize soil disturbance, some sample disturbance could have been encountered. In general, the upper portion of the deposit is in a loose to compact state of denseness and the lower 4 or 5 metres is in a dense to very dense state of denseness. Very loose to loose zones occur within the upper portion. At boreholes 3 and 4, where the deposit is overlain by fill material, 'N' values are generally higher and in a compact state of denseness.

4.4 HETEROGENEOUS MIXTURE OF SILT, SAND AND GRAVEL (GLACIAL TILL)

The native surficial deposit of Silt, some sand to Sand, some silt is underlain by a Heterogeneous Mixture of silt, sand and gravel. The thickness of this glacial till deposit was explored for approximately 4.5 metres.

Figure 3 in the Appendix illustrates a grain size distribution envelope for this deposit. It depicts a

material primarily of coarse grained sands and gravels (50-60 %). The fine grained portion of this deposit is comprised primarily of silts. Although cobbles and boulders were not encountered during the sampling process, they were inferred during the drilling process.

Natural moisture contents of the deposit range from 8.0% to 9.5%

The denseness of the deposit is based on 'N' values ranging from 228 blows/0.28m to 100 blows/0.08m is categorized as very dense.

5.0 GROUNDWATER CONDITIONS

The groundwater level was determined by tape measuring the water levels from the ground surface in the open boreholes upon completion of the field investigation. The results are summarized in Table 1 below.

Table 1 - Groundwater Elevations*

Borehole	Ground Surface Elevation (m)	Water Elevation (m)	Depth below Ground Surface (m)
1	282.984	282.534	0.45
2	282.670	282.360	0.31
3	**	**	**
4	285.635	282.535	3.1

* As of 96/01/02

** Not measured

The groundwater levels reported above are subject to seasonal fluctuations. It appears, however, that the groundwater elevation is approximately equal to that of the river's water surface elevation at the site.

6.0 DISCUSSION AND RECOMMENDATIONS

It is proposed to widen the existing Humber River Bridge at Ballycroy as part of the Highway 9 reconstruction program. It is planned to initially widen Highway 9 to two 3.75m lanes with 3.0m shoulders and ultimately to four 3.75m lanes with 3.0m shoulders. The Humber River Bridge will be widened to accommodate the ultimate Highway design.

The proposed widening is symmetrical about the centre-line of Highway 9 and will be approximately 5m on either side of the existing structure. The widened structure will be a single span concrete rigid frame structure similar to the existing structure. The span length is approximately 12.19m on a 0° skew.

Approach embankment fills will also be required as part of the widening. Embankment fill heights in the order of 4m or so will be required to elevate the widened portion of the highway from the existing ground surface to a profile grade of approximately 286.3m at the structure location.

A plan illustrating the existing and proposed widened structure is shown on Drawing 5319101-A in the Appendix. Two stratigraphical sections illustrating the subsurface conditions at the proposed structure widening locations are also included on Drawing 5319101-A. The Highway 9 profile grade is also superimposed on the stratigraphical section.

Recommendations pertaining to the following foundation and geotechnical considerations are included in the purview of this report:

1. Structure Foundations
2. Backfill to Structure
3. Approach Embankments, and
4. Construction Considerations.

6.1 STRUCTURE FOUNDATIONS

6.1.1 GENERAL

The abutments of the existing structure are supported on spread footings founded at an Elevation of 280.1m or approximately 2.6m below the natural ground surface in the area. It is expected that the spread footings are founded on the native cohesionless Sand, some silt to Silt, some sand deposit.

The native surficial subsurface conditions at the proposed structure foundation widening locations are comprised of the cohesionless Sand, some silt to Silt, some sand deposit. The native cohesionless soils are generally variable in composition and in a loose to compact state of denseness. The groundwater table at the site is approximately at an elevation of 282.5m and hence the native soils are submerged below the groundwater table.

In view of the conditions present, supporting the abutment widening on shallow foundations is not recommended. From a design point of view, excessive settlements could occur for spread footings founded on the native soils. In addition, a dewatering scheme that would be necessary to construct the foundations could result in differential settlement of the existing structure foundation.

It is therefore recommended that the structure foundations be founded on deep foundation units constructed of steel H-piles driven in holes pre-augered to a predetermined elevation as described below.

6.1.2 DEEP FOUNDATION STEEL H-PILES

Axial Capacity

The structure foundations at all four quadrants of the widening can be founded on steel H-piles driven to the very dense Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till) deposit at a tip elevation of approximately 260.0m. For purposes of the O.H.B.D.C., the steel H-piles can be designed employing the axial capacities summarized in Table 2 below.

Table 2 - Driven Steel H Piles : Vertical Axial Capacity

Structure Widening Quadrants	Pile Type	Factored Axial Capacity at U.L.S. (kN)	Axial Capacity at S.L.S. (kN)	Pile Tip Elevation (m)
NW,NE,SW,SE	HP 310X110	1600	1600	260.5±

Axial capacities tabulated in Table 2 represent the geotechnical capacities. The structural engineer shall calculate the structural capacity of the pile to ensure that this structural capacity is not exceeded. In cases, where the structural capacity of the pile is less than the geotechnical capacity, the structural capacity of the pile shall govern the design.

The Axial Capacity at S.L.S. represents the capacity at which the applied load will yield a settlement not exceeding 25mm. However, it is anticipated that settlement of pile foundations will be less than 25mm. It is expected that the load required to yield this deformation is equivalent to the load that will cause ultimate geotechnical failure of the founding soil.

Lateral Capacity

Resistance to lateral loads can be achieved by inclining the piles at a batter. The lateral resistance of battered piles shall be computed in accordance with Section 6-9.8 of the O.H.B.D.C. and shall be taken as the horizontal component of the factored axial capacity in the inclined pile.

Alternatively, the lateral capacity of vertical piles can also be computed by modelling techniques. Soil parameters to facilitate the design of the horizontal capacities of vertical piles using the spring constant method are given in Table 3.

The method of lateral capacity computation and the magnitude of lateral pile capacity calculated shall be reviewed by our office prior to design finalization. It is requested that the Structural Engineer contact our office following lateral capacity calculation.

Pile Installation

Special consideration is needed to avoid excessive vibration during pile installation and the potential for densification of the cohesionless soil supporting the existing foundations. It is therefore recommended that in order to preserve the integrity of the existing structure that the piles be driven in pre-augered holes of minimum 0.6m diameter and drilled to a depth equivalent to two times the existing footing width or 4m below the existing foundation (Elevation 276.1m). Pile driving can then commence below this significant depth of the existing foundation. The pre-augered holes can then be backfilled with a lean mix concrete placed using tremie methods.

To ensure the stability of the drilled shaft within the cohesionless soils submerged below the groundwater table at the site and to prevent soil cave-in and sloughing, slurry method techniques is

recommended to advance the pre-augered hole to the significant depth stated above. Vibrating or driving liners shall not be permitted.

A Non Standard Special Provision (N.S.S.P) shall be included in the contract documents that alerts the Contractor of the potential of soil cave-in and sloughing of the cohesionless soils submerged below the groundwater table and that specifies the material and construction procedure of the slurry method technique. This N.S.S.P. can be obtained from our office. The Contractor shall submit a slurry method procedure for pile installation for our review prior to construction.

The Contractor shall control the intensity of ground vibrations during pile installation. It is recommended that the intensity of ground vibrations generated by pile driving shall be restricted to a maximum peak particle velocity of 100 mm/s in three mutually perpendicular directions. The Contractor shall monitor the vibrations at the closest abutment structure location. Detailed records of monitoring shall be maintained at all times during pile installation.

To facilitate the installation of the steel H-piles, it is recommended that the piles be equipped with reinforced tips as illustrated on MTO standard drawing DD-3301 or alternatively driving shoes. The steel H-piles shall be spliced in accordance with OPSS 903.07.01.03 and as shown on the above mentioned drawing DD-3301.

It is recommended that the Hiley Dynamic Formula be employed to control the pile installation process. All piles shall be controlled using an ultimate capacity equivalent to 3200 kN.

Pile spacing shall conform with Section 6-11 of the O.H.B.D.C. All pile caps shall be protected against frost penetration by providing a minimum 1.2m earth cover or equivalent frost penetration.

Table 3 - Horizontal Resistance Design Parameters

Structure Widening Quadrant	Soil	Elevation (m)	Angle of Internal Friction (ϕ)	*Bulk Unit Weight γ (kN/m ³)	Coefficient of Horizontal Subgrade Reaction (kN/m ³)
SE	Silt, some Sand to Sand, some Silt	283 - 262	30°	20	1300
	Glacial Till	≤262	35°	20	10700
SW	Silt, some Sand to Sand, some Silt	282.7 - 276	30°	20	1300
		276 - 260	28°	20	1000
	Glacial Till	≤260		20	10700
NE	Silt, some Sand to Sand, some Silt	281.3 - 264	30°	20	1300
		264 - 261	32°	20	4400
	Glacial Till	≤261	35°	20	10700
NW	Silt, some Sand to Sand, some Silt	281.4 - 267	30°	20	1300
		≤267	35°	20	10700

* Buoyant unit weights to be used for soils submerged below the groundwater table.

6.2 BACKFILL TO STRUCTURE

6.2.1 MATERIAL

It is recommended that Granular 'A' or Granular 'B' (conforming to Special Provision SP109F03) material be placed behind the abutment wall as illustrated in Figure 6-7.4.1 of the O.H.B.D.C. The use of a granular material combined with weep holes in the abutment walls or pipe subdrains to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up.

Design parameters of the soil are given in Table 4 below. Computations of lateral earth pressure shall be in accordance with Section 6-7 of the O.H.B.D.C.

Table 4 - Structure Backfill Properties

Backfill Property	Granular 'A'	Granular 'B'
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m ³)	22.8	21.2
*Coefficient of Active Earth Pressure (K_a)	0.27	0.33
*Coefficient of Earth Pressure at Rest (K_o)	0.43	0.5

* These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping backfill. The coefficient of earth pressure at rest shall be applied for rigid and unyielding walls. The active condition applies for flexible walls where sufficient movement is permitted to mobilize the active pressure. Figure C6-7.1 and Table C6-7.1 in the O.H.B.D.C. commentary depict movements required to achieve the active state.

The compaction surcharge shall be calculated in accordance with Section 6-7.4.3 of the O.H.B.D.C.

The calculated lateral pressure is a function of the mass and type of compaction equipment and the material being compacted. A minimum compaction surcharge pressure of 16 kPa should be used for a vibratory compactor with a mass of approximately 400 kg.

6.2.2 BACKFILLING AND COMPACTION

The backfill shall be placed in 300 mm lifts in accordance with OPSS 902 series and compacted to achieve 100 percent of the target maximum dry density as outlined in OPSS 501 series. The backfill shall be placed simultaneously and evenly on both side such that the maximum differential settlement in the fill placement at the abutments does not exceed 600mm.

Heavy vibratory equipment is not permitted in the backfill construction adjacent to the structure to minimize deflection or possible damage of the wall. Vibratory equipment exceeding 6000 kg operating weight should be kept outside of a 1.5 vertical to 1 horizontal line extending upward from the base of the abutment footing. Hand compaction equipment shall be used within these limits.

6.3 APPROACH EMBANKMENTS

6.3.1 GENERAL

There are two major factors that must be considered in the design of approach embankments:

- (1) Stability
- and (2) Settlement.

The stability and settlement of the approach embankments with fill heights up to approximately 4m to 5m are discussed below.

6.3.2 STABILITY

In view of the cohesionless nature of the native subsoils at the site, there are no deep seated or global (external) slope instabilities expected for slopes constructed at 2H:1V both in the longitudinal and transverse directions.

For slopes less than 8m in height, the 2H:1V geometry slopes will also be adequate to avoid surficial slope instabilities provided that an effective erosion control protection scheme such as conventional seeding and mulching or sod be applied with a surface runoff drainage system at the toe of the slope.

To ensure the internal stability within the embankment fill, it is recommended that the new fills be "benched" into the existing approach embankments in accordance with OPSD 208.01.

6.3.3 SETTLEMENT

Settlements induced as a result of the applied embankment loading will be the result of the elastic compression of the native subsoil and as a result of settlements within the fill material itself. It is anticipated that approximately 25 to 50 mm of settlement attributable to the elastic compression of the native subsoil will be realized. These settlements will be elastic in nature and hence will occur almost immediately.

Settlements within the fill itself can amount to approximately 25 to 50 mm due to its self weight. These settlements will be immediate in nature if a granular fill material is used. For fill materials that are cohesive, settlements will be more time dependent, but should be realized within a 3 month period following the fill placement.

6.3.4 EMBANKMENT CONSTRUCTION

Embankment fills shall be placed and compacted as specified in OPSS 206.07.07 and OPSS 501 series. As mentioned previously, new fills constructed adjacent to existing fills shall be benched in accordance with OPSD 208.01.

A NSSP shall be included in the Contract Documents which makes the Contractor aware of the dewatering requirement within the cohesionless silts submerged below the groundwater table. The Contractor shall be advised to assess the groundwater conditions in advance of construction to determine any impact on the construction procedure. A detailed dewatering proposal shall be submitted to the Contract Administrator for review a minimum 15 days prior to commencing dewatering operations. Disposal of water shall conform to OPSS 518.

6.4.2 TEMPORARY SHORING

General

In order to facilitate the construction of the widening while maintaining traffic on Highway 9, temporary shoring walls may be required. One viable shoring scheme that can be used at the site is a soldier pile-timber lagging wall as described below. The design and construction of the protection scheme shall comply to OPSS 539. Discussion of the design and construction of the shoring wall is given below.

Shoring Design

The design of the shoring system shall include the appropriate earth pressures computed using an acceptable method specific to the site conditions. Loadings induced by any surcharge traffic shall be incorporated in the design. Lateral earth pressures can be computed using the soil design parameters tabulated in Table 5. The active earth pressure coefficient can be used in the design of the shoring wall.

The shoring system must be designed to satisfy earth pressure equilibrium using an appropriate restraining system. At the site, this can be achieved by either a cantilever wall or raker supported

wall. An appropriate triangular, rectangular or trapezoidal stress envelope shall be chosen to accurately represent the retained soils and the restraining system.

The shoring system method that proves to be the most practical and economical shall be selected.

The two options, namely the cantilever and the raker supported wall are briefly discussed below.

Further discussions with our office can be subsequently coordinated during the design stage.

Table 5 - Shoring Design Parameters

Structure Widening	Soil	Elevation (m)	Angle of Internal Friction (ϕ)	*Bulk Unit Weight (γ) (kN/m ³)
SE	Irregular Mixture of Silt, Sand and Gravel (Fill Material)	286 - 281.5	30	20
	Sand, some Silt to Silt, some Sand	281.5 - 262	30	20
SW	Irregular Mixture of Silt, Sand and Gravel (Fill Material)	286 - 281.5	30	20
	Sand, some Silt to Silt, some Sand	281.5 - 276	30	20
		276 - 262	28	20
NE	Irregular Mixture of Silt, Sand and Gravel (Fill Material)	286 - 281.5	30	20
	Sand, some Silt to Silt, some Sand	281.5 - 264	30	20
NW	Irregular Mixture of Silt, Sand and Gravel (Fill Material)	286 - 281.5	30	20
	Sand, some Silt to Silt, some Sand	281.5 - 267	30	20

* Buoyant unit weights (γ') are to be used below the groundwater table.

In the design of a cantilever wall, the depth of embedment shall be sufficient to ensure that the wall does not overturn. The active pressures exerted on the wall must be resisted by the passive pressures below the dredge line. Triangular earth pressure distributions can be used in the calculation of the active and passive earth pressures.

Should the cantilever wall embedment depth be considered excessive, rakers propped against the wall can be used. To avoid dewatering at the site, it is recommended that the rakers be founded on granular 'A' pads placed and compacted on the native soils. The granular 'A' pad shall have a minimum thickness of 1m and shall be constructed as illustrated in Figure 4 in the Appendix. All organic and deleterious native soils and any fill material shall be removed prior to the construction of the granular 'A' pad. Table 6 below provides recommendations for bearing capacities for the granular 'A' pad supported raker foundation.

Table 6 - Raker Foundations - Granular 'A' Pad

Structure Widening	Founding Elevation (m)	Bearing Capacity at S.L.S.(kPa)	Factored Capacity at U.L.S.(kPa)
SE	282.5	50	300
SW	282.5	150	500
NE	281.5	100	300
NW	281.0	150	400

Shoring Construction

To avoid excessive vibration caused by driving and the detrimental impact that this may have on the existing structure foundations, it is recommended that the soldier piles be installed in pre-augered holes advanced using slurry method techniques. Vibrating or driving liners to facilitate the pre-

angering is not considered acceptable at the site.

Using the slurry method technique, concrete tremie methods can be used to construct the soldier pile toes and to fill the annular space between the soldier pile and the pre-augered hole with lean mix concrete.

An NSSP shall be included in the contract documents that advises the Contractor that the cohesionless sands and silts submerged below the groundwater table may slough into the hole as a result of unbalanced hydrostatic conditions. The NSSP shall also include the requirement of pre-augering the soldier pile hole using slurry method techniques.

Rakers shall be installed while an earth berm remains in front of the pile. Slots should be cut into this berm to install the rakers before the supporting berm is removed.

Temporary Excavation Cut Slopes

Temporary excavation cut slopes within the irregular mixture of silt, sand and gravel shall not be steeper than 1.5H:1V.

Utility Coordination

A high pressure gas main (Consumers Gas) is located at the south end of the site within the limits of the widening. This gas main must be relocated to facilitate the construction of the structure foundations at this location.

6.5 OTHER CONSIDERATIONS

6.5.1 SCOUR PROTECTION

A scour protection scheme to protect the new and existing structure foundation may be required depending on the hydrological analysis at the site. A scour protection scheme shall consider the native subsoil conditions at the site and shall be coordinated with our office.

7.0 MISCELLANEOUS

The field work for this investigation was carried out under the supervision of P. Pasqualini, Engineer in Training and T. Sangiuliano, Foundation Engineer, utilizing equipment owned and operated by Malones Soil Samples.

The report was prepared by T. Sangiuliano and P. Pasqualini and reviewed by D. Dundas, Senior Foundation Engineer.



A handwritten signature in black ink, appearing to read "T. Sangiuliano".

T. Sangiuliano, P. Eng.
Foundation Engineer

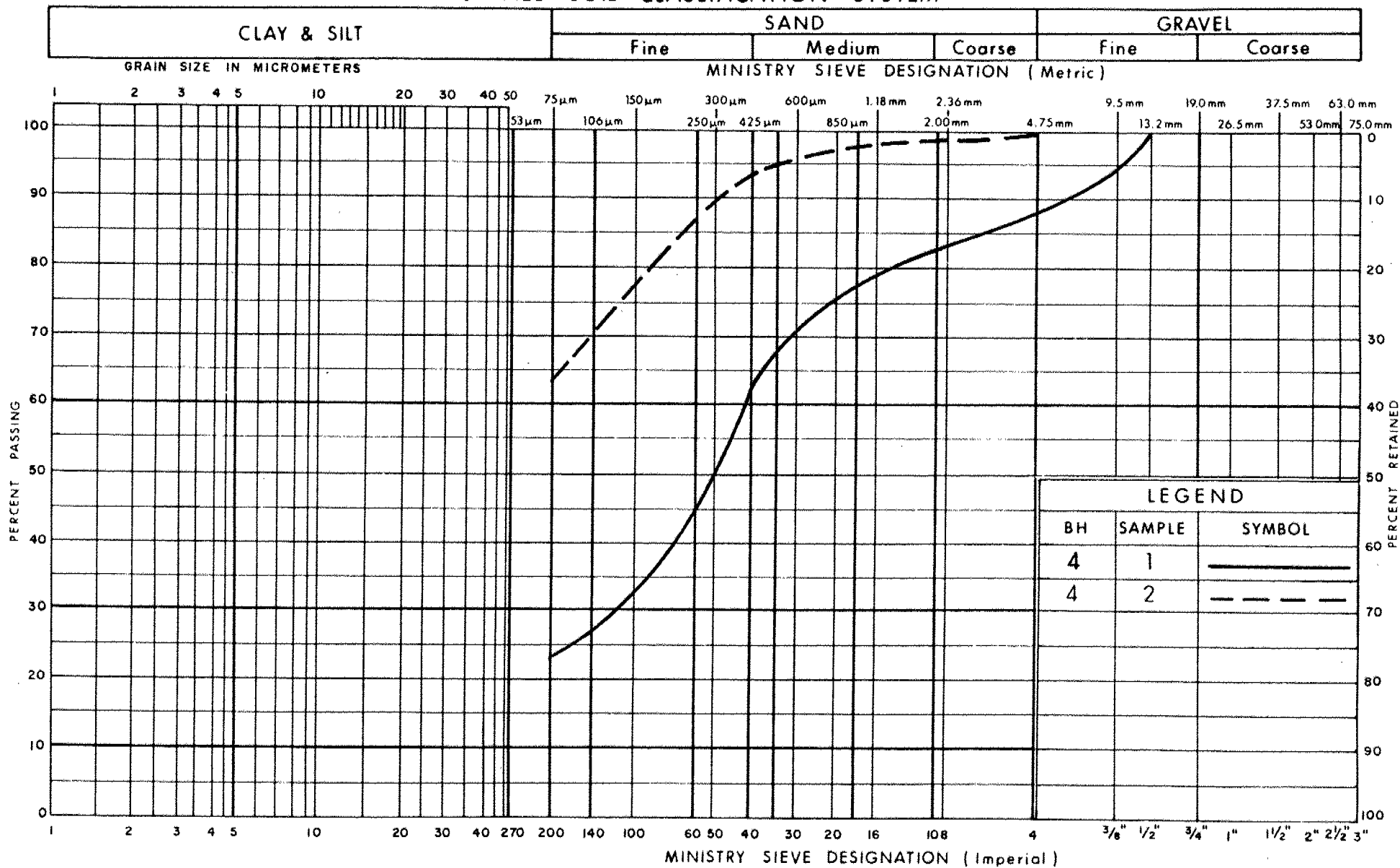


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D. Dundas, P. Eng.
Senior Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM


 Ministry of
Transportation

Ontario

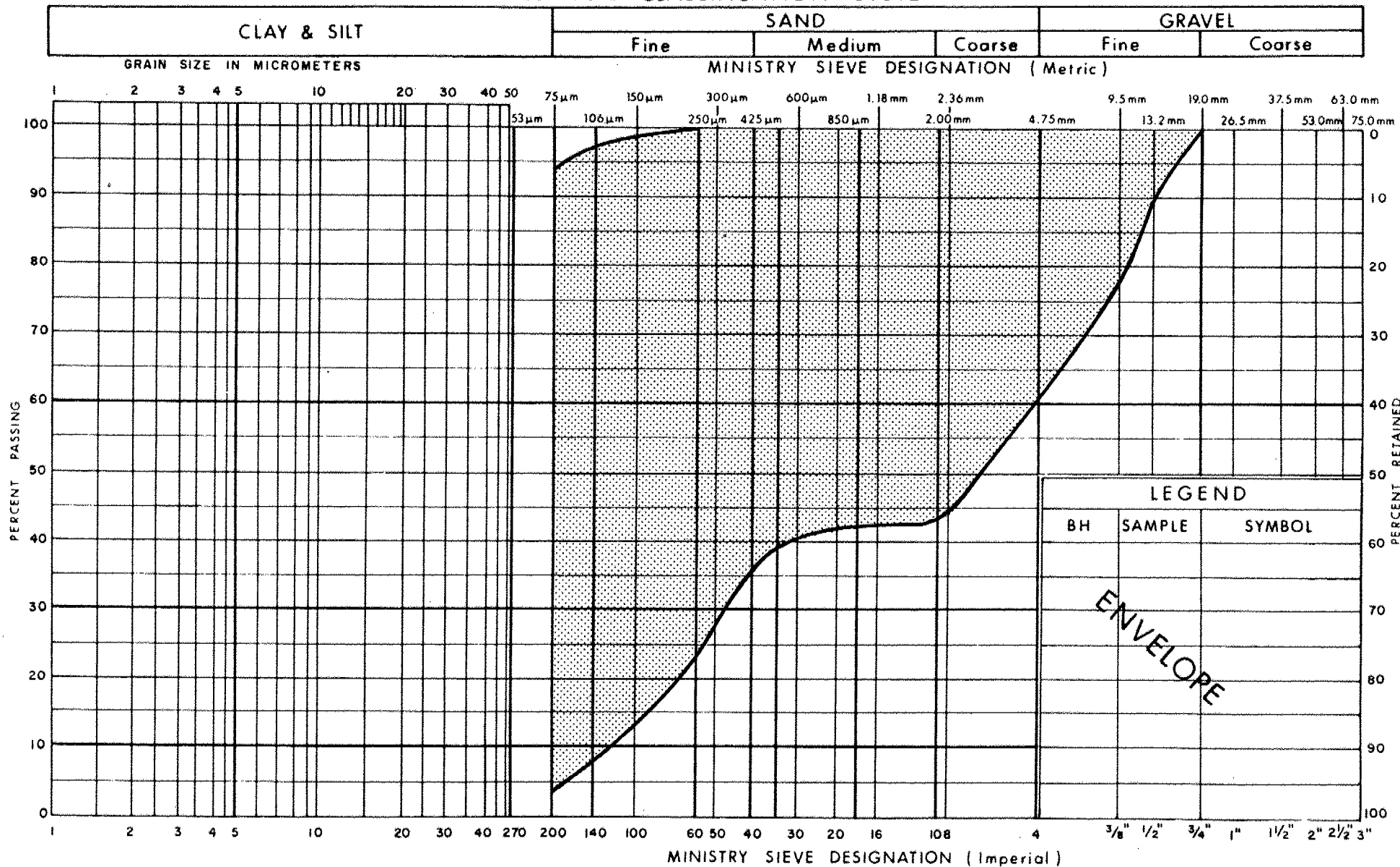
GRAIN SIZE DISTRIBUTION

IRREGULAR MIXTURE OF SILT, SAND & GRAVEL (FILL MATERIAL)

FIG No 1

W P 531-91-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Ministry of
Transportation

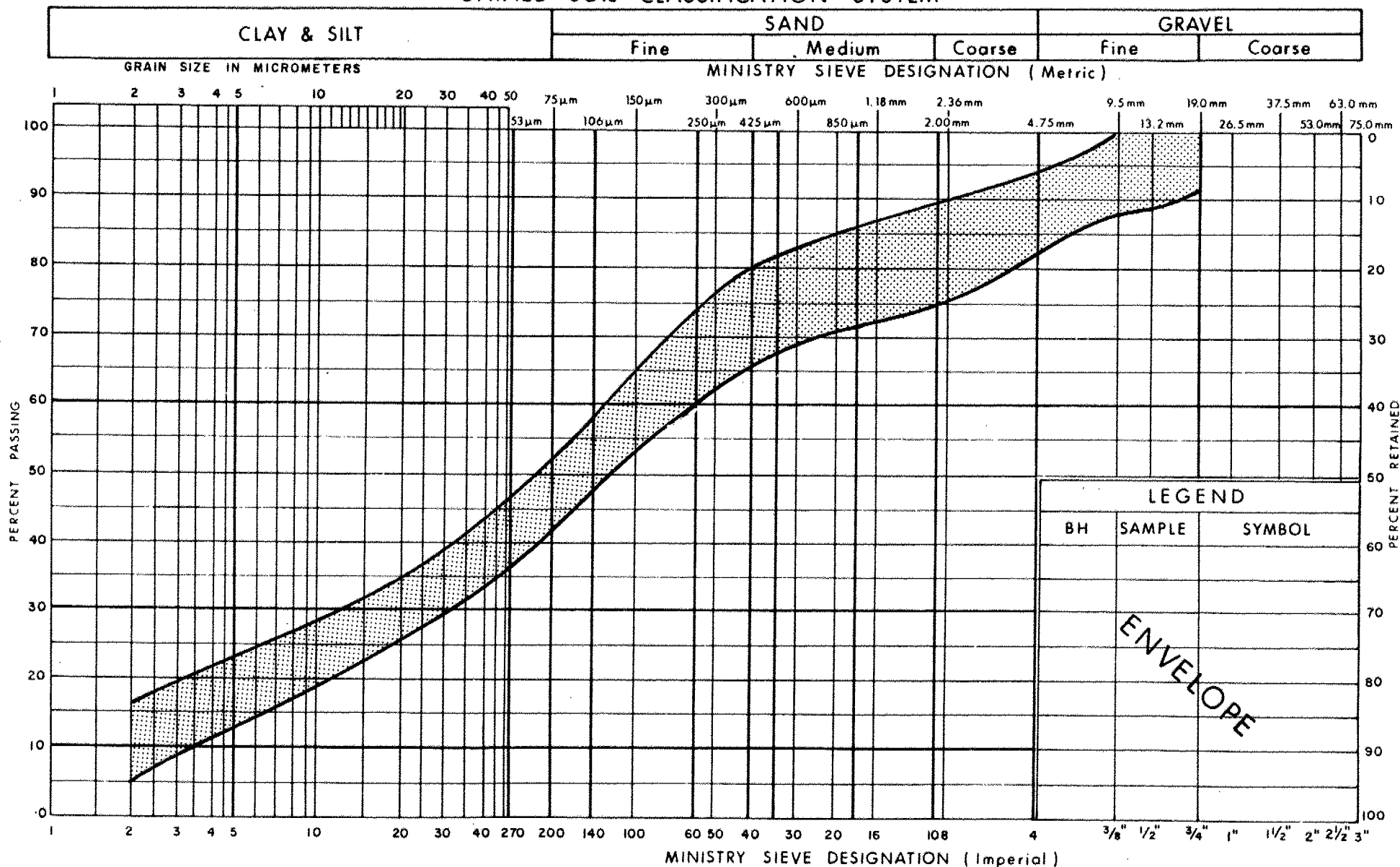
GRAIN SIZE DISTRIBUTION

SILT, SOME SAND to SAND, SOME SILT

FIG No 2

WP 531-91-01

UNIFIED SOIL CLASSIFICATION SYSTEM

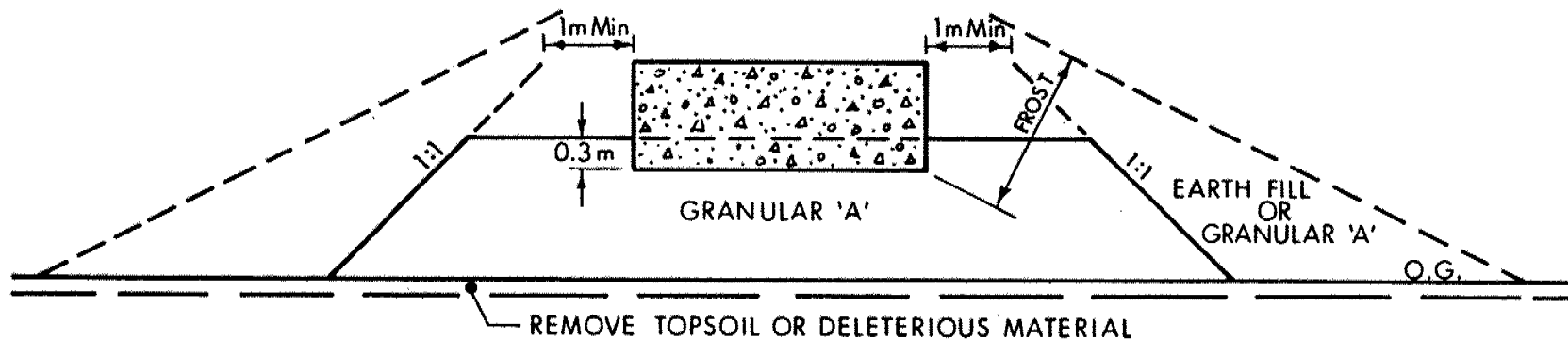


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
HET MIXTURE OF
SILT, SAND & GRAVEL (Glacial Till)

FIG No 3

W P 531 - 91 - 01



TYPICAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL & /OR DELETERIOUS MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A'.
- 2 - PLACE GRANULAR 'A' TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT MTO STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Figure 4 - RAKER FOUNDATIONS ON GRANULAR 'A' PAD

WP 531-91-01

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 531-91-01 LOCATION Coords: N 4 869 671.9 E 274 912.5 ORIGINATED BY P.P.
 DIST Central HWY 9 BOREHOLE TYPE HS Auger, NW Casing/Washboring COMPILED BY T.S.
 DATUM Geodetic DATE 95/12/15 CHECKED BY D.D.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20	40							60	80	100
283.0	Ground Surface																	
0.0	Brown Grey		1	SS	7		282											
			2	SS	9													
			3	SS	5													
			4	SS	3		280							1 67 (32)				
			5	SS	5													
			6	SS	13													
			7	SS	20		278							0 25 (75)				
			8	SS	18													
			9	SS	31		276											
			10	SS	50		274							0 61 (39)				
	Silt, some Sand to Sand, some Silt		11	SS	16		272							0 24 (76)				
			12	SS	6		270											
			13	SS	18		268											
	Loose to Compact		14	SS	10		266											
	Dense to Very Dense		15	SS	52		264											
			16	SS	14		262											
			17	SS	45		260											
			18	SS	90		258											
260.7			19	SS	130	/15cm	260							0 16 (84)				
22.3	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till) Grey, Very Dense		20	SS	140	/8cm	258							15 40 38 7				
256.8			21	SS	100	/15cm								6 42 36 16				
26.2	End of Borehole * 96/01/02																	

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 531-91-01 LOCATION Coords: N 4 869 668.5 E 274 898.9 ORIGINATED BY P.P.
DIST Central HWY 9 BOREHOLE TYPE HS Auger, NW Casing/Washboring COMPILED BY T.S.
DATUM Geodetic DATE 95/12/20 CHECKED BY D.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
282.7	Ground Surface													
0.0														
	Brown to Black Trace Organics		1	SS	1		282							
	Very Loose		2	SS	4									
	Grey, Loose to Compact		3	SS	13		280							2 78 (20)
	Some gravel		4	SS	27									
			5	SS	27		278							39 57 (4)
			6	SS	29									
			7	SS	17									
			8	SS	22		276							0 88 (12)
	Very Loose to Loose		9	SS	3									
			10	SS	4		274							0 68 (32)
			11	SS	1									
	Silt, some sand to Sand, some silt		12	SS	6		272							0 39 (61)
			13	SS	7									
			14	SS	7		270							1 64 (35)
			15	SS	3									
			16	SS	18		268							
	Compact to Very Dense		17	SS	26		266							
			18	SS	40		264							
	Some gravel		19	SS	69		262							17 71 (12)
260.4			20	SS	228	/28cm	260							
22.3	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)		21	SS	140	/15cm	258							18 40 34 8
	Grey, Very Dense		22	SS	130	/10cm								
256.5														
26.2	End of Borehole * 96/01/02													

RECORD OF BOREHOLE No 3

1 OF 1 METRIC

W.P. 531-91-01 LOCATION Coords: N 4 869 691.7 E 274 905.2 ORIGINATED BY P.P.
 DIST Central HWY 9 BOREHOLE TYPE HS Auger, NW Casing/Washboring COMPILED BY T.S.
 DATUM Geodetic DATE 96/01/02 CHECKED BY D.D.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								UNCONFINED								FIELD VANE		QUICK TRIAXIAL
284.3	Ground Surface						20	40	60	80	100	10	20	30				
0.0	Irregular Mixture of Brown Silt, Sand and Gravel (Fill Material)		1	SS	4													
281.4	Very Loose		2	SS	2													
2.9			3	SS	17													
			4	SS	12													
			5	SS	12													
	Trace/some gravel		6	SS	39													
			7	SS	21													
			8	SS	26													
			9	SS	20													
			10	SS	12													
			11	SS	12													
			12	SS	4													
	Silt, some sand to Sand, some silt		13	SS	17													
			14	SS	19													
	Compact		15	SS	70													
	Dense to Very Dense		16	SS	46													
			17	SS	64													
			18	SS	43													
			19	SS	96													
260.4			20	SS	62													
23.9	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)	21	SS	100	/8cm													
	Grey, Very Dense	22	SS	185	/10cm													
256.7		23	SS	137	/13cm													
27.6	End of Borehole * Not Established																	

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 531-91-01 LOCATION Coords: N 4 869 682.4 E 274 887.5 ORIGINATED BY P.P.
DIST Central HWY 9 BOREHOLE TYPE HS Auger, NW Casing/Washboring COMPILED BY T.S.
DATUM Geodetic DATE 95/12/12 CHECKED BY D.D.

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			SHEAR STRENGTH kPa									WATER CONTENT (%)
								20	40	60	80						
285.6	Ground Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Loose to Compact		1	SS	14											12 65 (23)	
	Brown																
	Black, trace Organics		2	SS	5											0 37 (63)	
281.2			3	SS	12												
4.4			4	SS	11												
			5	SS	25												
			6	SS	25											0 52 (48)	
			7	SS	38												
			8	SS	32												
	Compact to Dense		9	SS	85											0 46 (54)	
	Very Dense		10	SS	88												
	Silt, some sand to Sand, some silt		11	SS	3											0 41 (59)	
	Loose to Very Dense		12	SS	4												
			13	SS	9												
	Clayey Silt		14	SS	127												
	Very Dense		15	SS	110											0 41 (59)	
263.3			16	SS	47												
22.3	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)		17	SS	128												
260.8	Grey, Very Dense		18	SS	115												
24.8	End of Borehole																
	* 96/01/02																

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
WS	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

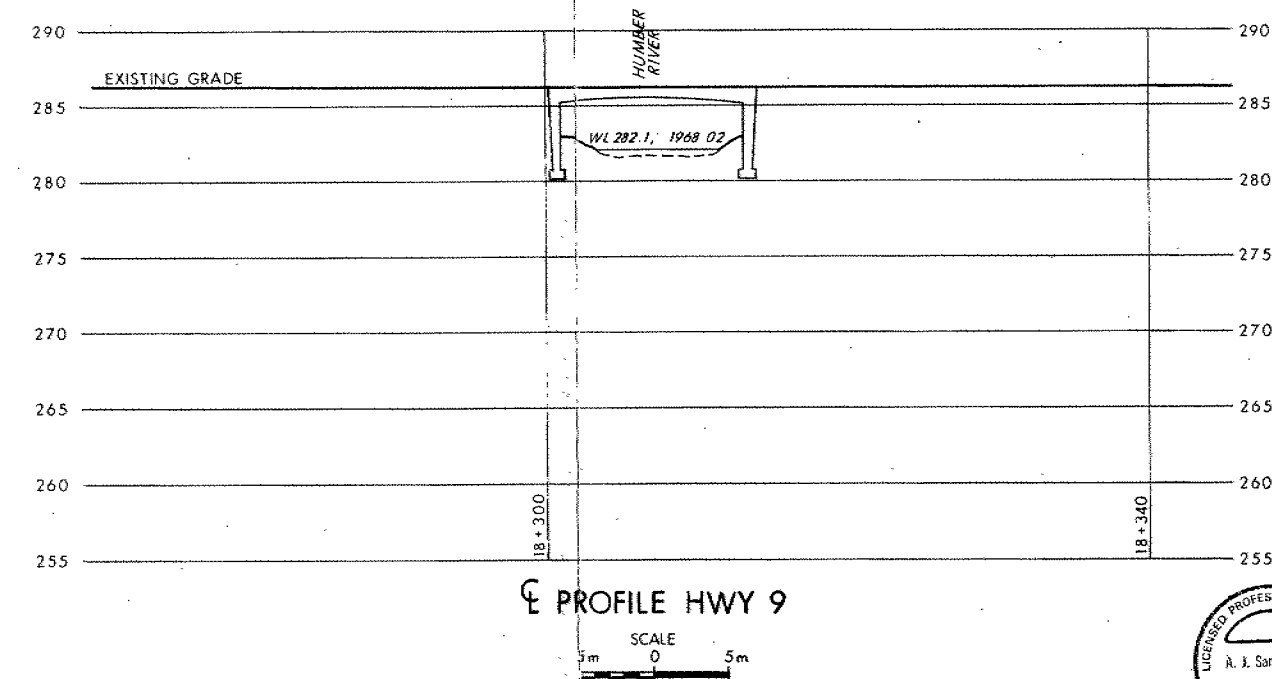
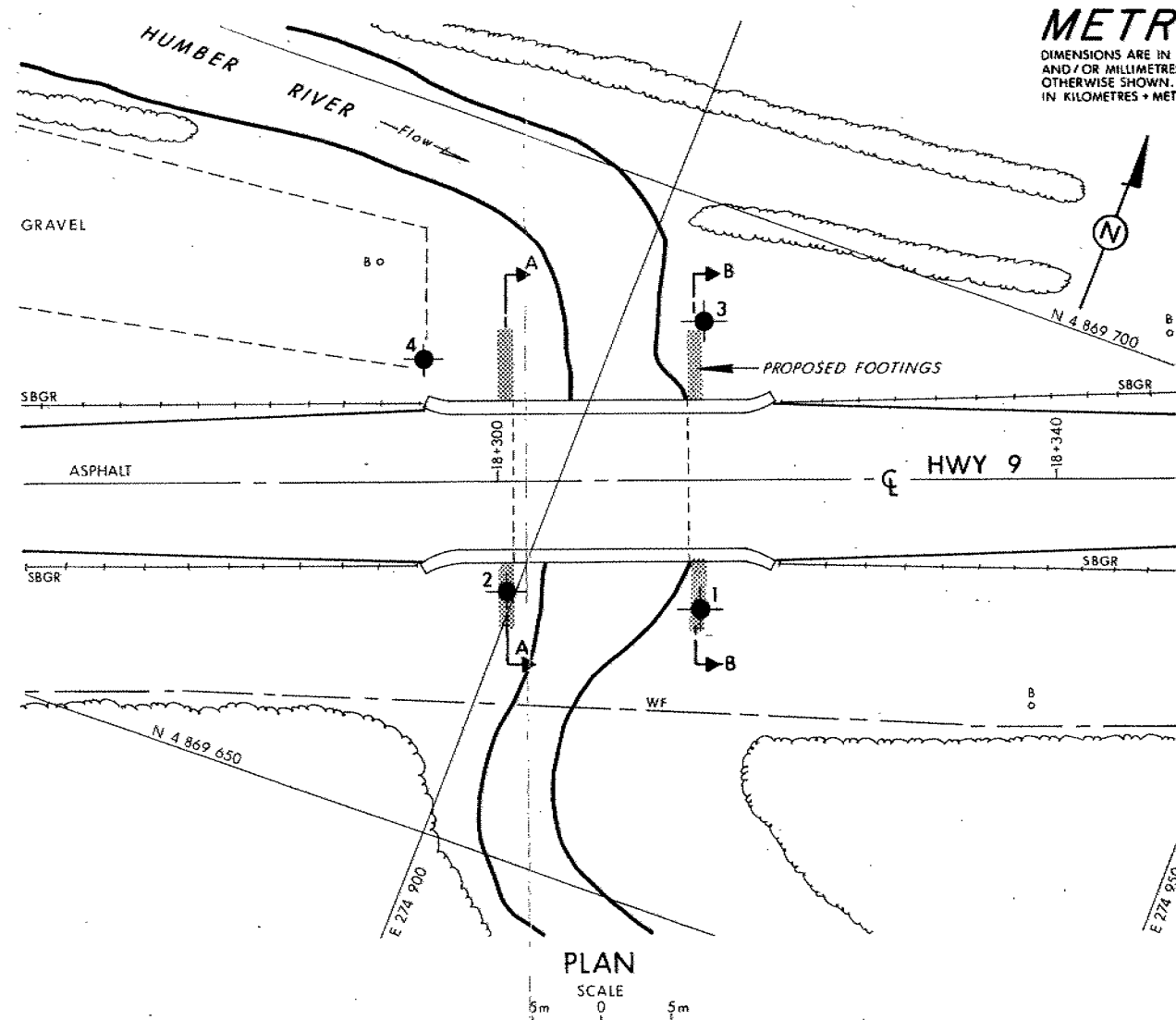
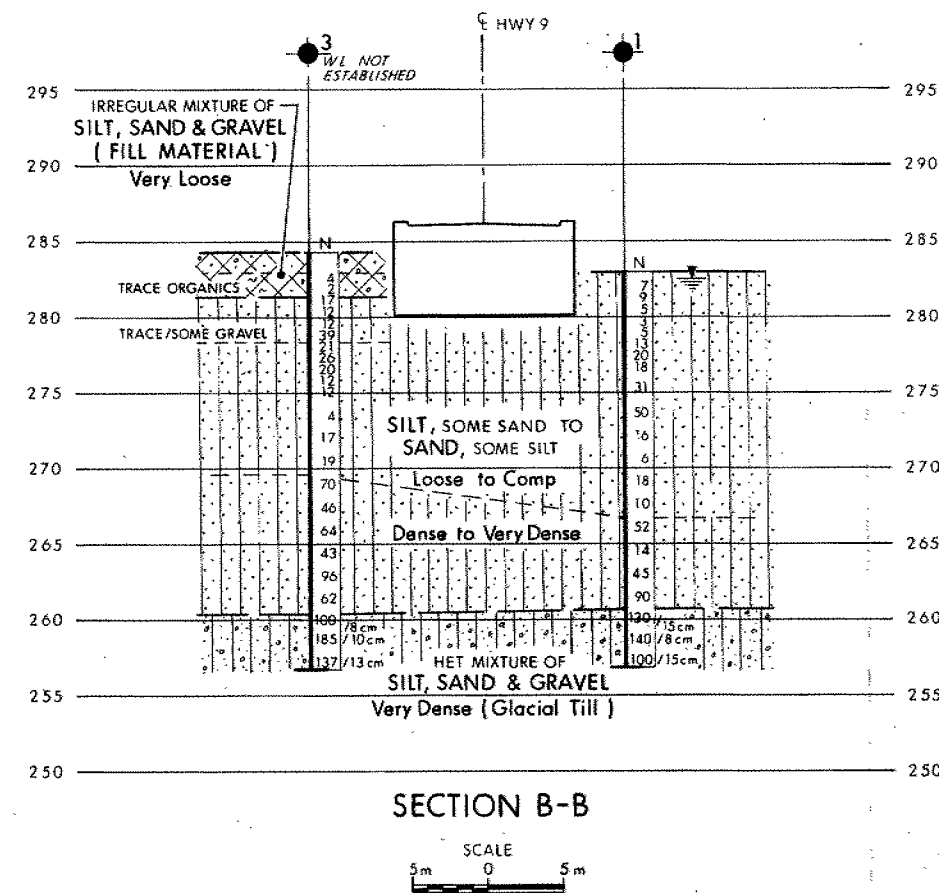
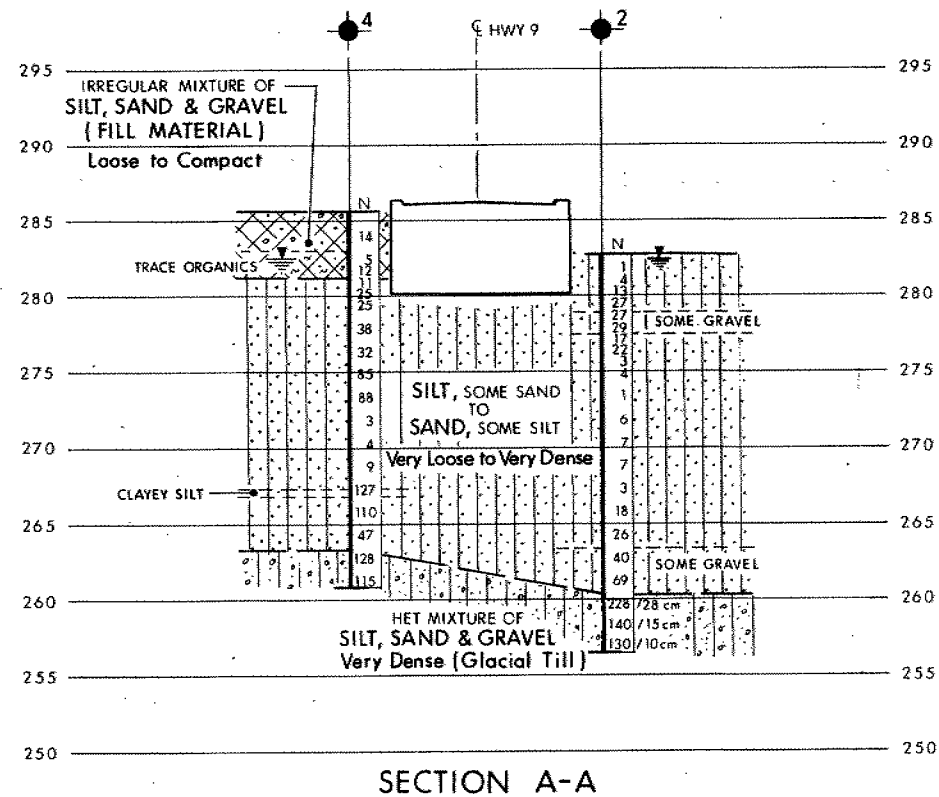
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa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



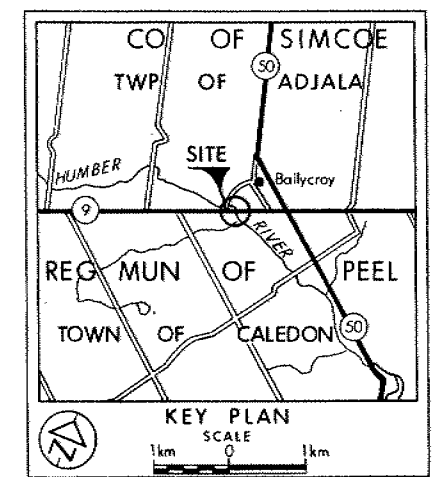
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 531-91-01

HUMBER RIVER

BORE HOLE LOCATIONS & SOIL STRATA

SHEET



- 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 1995 12

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	283.0	4 869 671.9	274 912.5
2	282.7	4 869 668.5	274 898.9
3	284.3	4 869 691.7	274 905.2
4	285.6	4 869 682.4	274 887.5

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 GC 2.01 of OPS Gen.Cord.

REV	DATE	BY	DESCRIPTION
1			

Geocres No 30M13-134

HWY No	DATE	SITE	DIST
9	1996 05 22	24-40	CENTRAL

DRAWN R.S. CHECKED R.S. APPROVED R.S.

DWG 5319101-A



Bennett, Betty (MTO)

From: Bennett, Betty (MTO)
Sent: Monday, July 24, 2000 9:48 AM
To: 'David_Leblanc@urscorp.com'
Cc: Wong, Dennis (MTO)
Subject: RE: Contract 2000-0047 - Humber River Bridge

David

Under Limit States Design, the ultimate capacity is twice the factored ULS resistance. The MTO Drawings are not quite up to date. At this site, for the Hiley Formula, 2x the factored ULS or 3200 kN should be used.

Betty

-----Original Message-----

From: David_Leblanc@urscorp.com [mailto:David_Leblanc@urscorp.com]
Sent: Monday, July 24, 2000 9:12 AM
To: Betty.Bennett@mto.gov.on.ca
Cc: WongD@mto.gov.on.ca; Duane_Girard@urscorp.com
Subject: Contract 2000-0047 - Humber River Bridge

Betty,

We are reviewing the contractor's proposed pile-driving methodology for the Humber River Bridges.

The ULS capacity of the piles given in the Foundation report is 1600 kN. On page 12 of the foundation report, it is stated that an ultimate capacity of 3200 kN should be used in the Hiley Dynamic Formula. Is this correct, or should a value of 3 times the ULS capacity be used as given on MTO Standard Dwg 22103-11?

David LeBlanc
(905) 882-4401

Bennett, Betty (MTO)

From: Wong, Dennis (MTO)
Sent: Tuesday, July 18, 2000 2:51 PM
To: 'David_Leblanc@urscorp.com'
Cc: Bennett, Betty (MTO)
Subject: RE: Hwy 9 at Humber River

Dave,

We seem to have covered both situations. If this is fine with Betty, that'll be OK.....Dennis

-----Original Message-----

From: David_Leblanc@urscorp.com [mailto:David_Leblanc@urscorp.com]
Sent: Tuesday, July 18, 2000 02:19 PM
To: Bennett, Betty (MTO)
Cc: Dave.Dundas@mto.gov.on.ca; Dennis.Wong@mto.gov.on.ca;
Tony.Sangiuliano@mto.gov.on.ca; Duane_Girard@urscorp.com
Subject: RE: Hwy 9 at Humber River

*ok'd verbally
with Dennis 06/07/19
DB*

Dennis/Betty,

We have included the requirements limiting the pile driving energy to control the intensity of ground vibrations in an NSSP under item 130, therefore no additional changes are required to the contract. However, there appears to have been some confusion as to the need for pre-augering holes prior to driving piles and using slurry methods, as these requirements are still included in the contract documents. We will therefore instruct the contractor that he does not need to pre-auger or use a slurry method of pile-driving.

Dave

"Bennett, Betty (MTO)" To: "Wong, Dennis (MTO)"
<Dennis.Wong@mto.gov.on.ca>
<Betty.Bennett@mto.gov.on.ca> cc: "Dundas, Dave (MTO)"
<Dave.Dundas@mto.gov.on.ca>, "Sangiuliano, Tony (MTO)" <Tony.Sangiuliano@mto.gov.on.ca>, "david_leblanc@urscorp.com"
07/18/00 01:47 PM <david_leblanc@urscorp.com>
Subject: RE: Hwy 9 at Humber River

Dennis

In a fax to Allan Cox of Cole Sherman dated April 7, 1999, the issue of pre-augering the piles was revisited by the Foundations Group. It was recommended that the piles be driven without pre-augering under the monitoring program outlined in the Foundation Investigation and Design Report. The Contractor was to be advised to limit the driving energy and the number of blows of the hammer for the first 4 m of driving.

This recommendation is not reflected in the special provision under Item 130 of Contract 2000-0047. The Contractor should proceed as per the recommendations in the fax as described in the above paragraph.

Regarding the vibration monitoring, should the peak particle velocity at

any
time exceed the 100mm/s, the Contractor should cease pile driving and
propose a method of proceeding.

Betty

-----Original Message-----

From: Wong, Dennis (MTO)
Sent: Tuesday, July 18, 2000 11:12 AM
To: Bennett, Betty (MTO)
Cc: Dundas, Dave (MTO); Sangiuliano, Tony (MTO);
'david_leblanc@urscorp.com'
Subject: FW: Hwy 9 at Humber River

Betty,

I would appreciate your review of the Contractor's proposal in the earliest
opportunity so that we can respond to the Contractor.

If there is any chance allowing the Contractor to do so, we probably have
to
set up monitoring stations (elevation check points) at the 4 corners of the
existing structure and readings be obtained and reported on a daily basis.

For your easy reference, the original Foundation Investigation and Report
was done by Tony dd June 96 and I have a copy in my office. The contract
number is 2000-0047; item #130 with NSP on page #157.

Thank you for your help.....Dennis

-----Original Message-----

From: David_Leblanc@urscorp.com [mailto:David_Leblanc@urscorp.com]
Sent: Tuesday, July 18, 2000 09:13 AM
To: Duane_Girard@urscorp.com; Roy_Pritchard@urscorp.com
Cc: WongD@mto.gov.on.ca
Subject: Hwy 9 at Humber River

Roy/Duane

Graham Bros has requested that we review the need for driving the piles
using the slurry method of construction as specified in the contract
documents (to minimize vibrations on the existing footings). Given the
weak nature of the soils at the site, I have no objection to Graham Bros'
proposal not to use the slurry method, providing they take measures to
minimize vibrations on the existing foundations to remain, and any damage
to the existing structure is repaired by the contractor at no cost to the
Ministry.

I have left a message with Betty Bennett of MTO to confirm if this is
acceptable with MTO Foundations. I have asked her to respond asap, as the
contractor wishes to drive piles tomorrow,

Dave

Construction

Prior to construction of the dewatering system, the Contractor shall assess the groundwater conditions to determine whether there exists any impact on the proposed construction. A detailed dewatering scheme shall be submitted to the Contract Administrator for review a minimum of fifteen (15) days prior to commencing dewatering operations. The groundwater level shall be lowered to a minimum depth of 0.5 m above the existing foundation level.

Disposal of water shall conform to OPSS 518.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials necessary to do the work.

H-PILES - HP 310x110 - Item No. 130**Special Provision****Construction**

The piles for the widened footings and the temporary shoring system shall be installed in 600 mm diameter pre-augered holes. Slurry method techniques shall be utilized to advance the pre-augered holes as the use of driving liners or vibrating is not permitted. The pre-augered holes at the widening locations shall be drilled to a depth of 4 m below the founding level of the existing structure footings (Elevation 276.1 m).

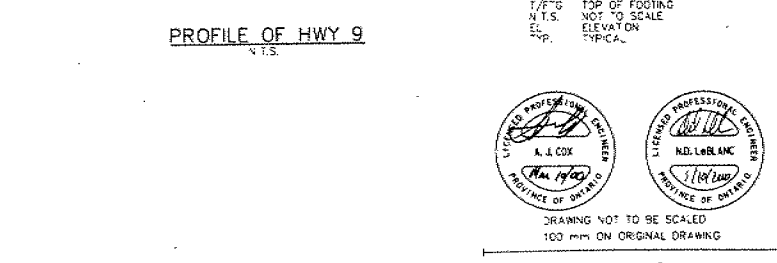
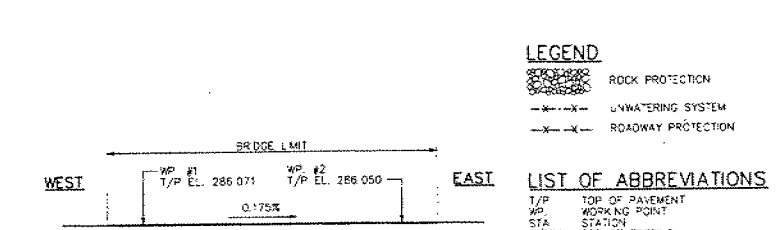
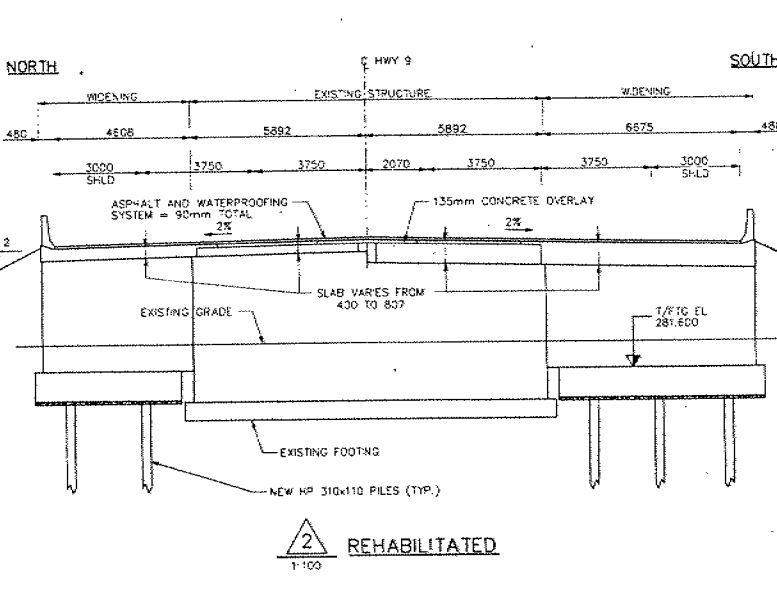
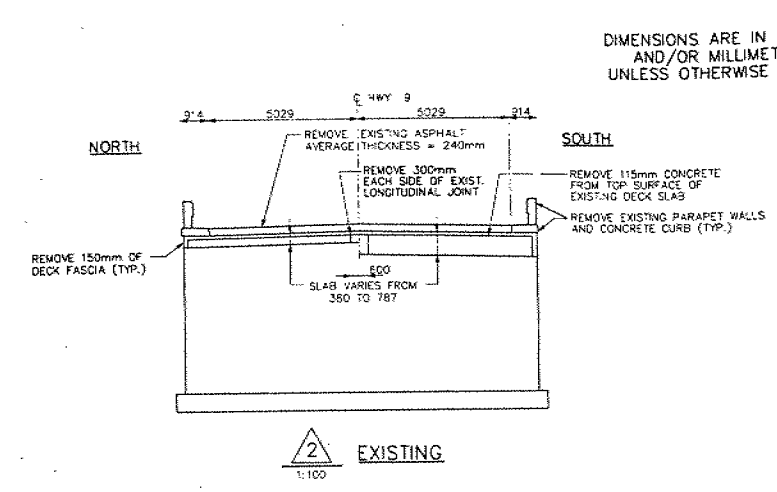
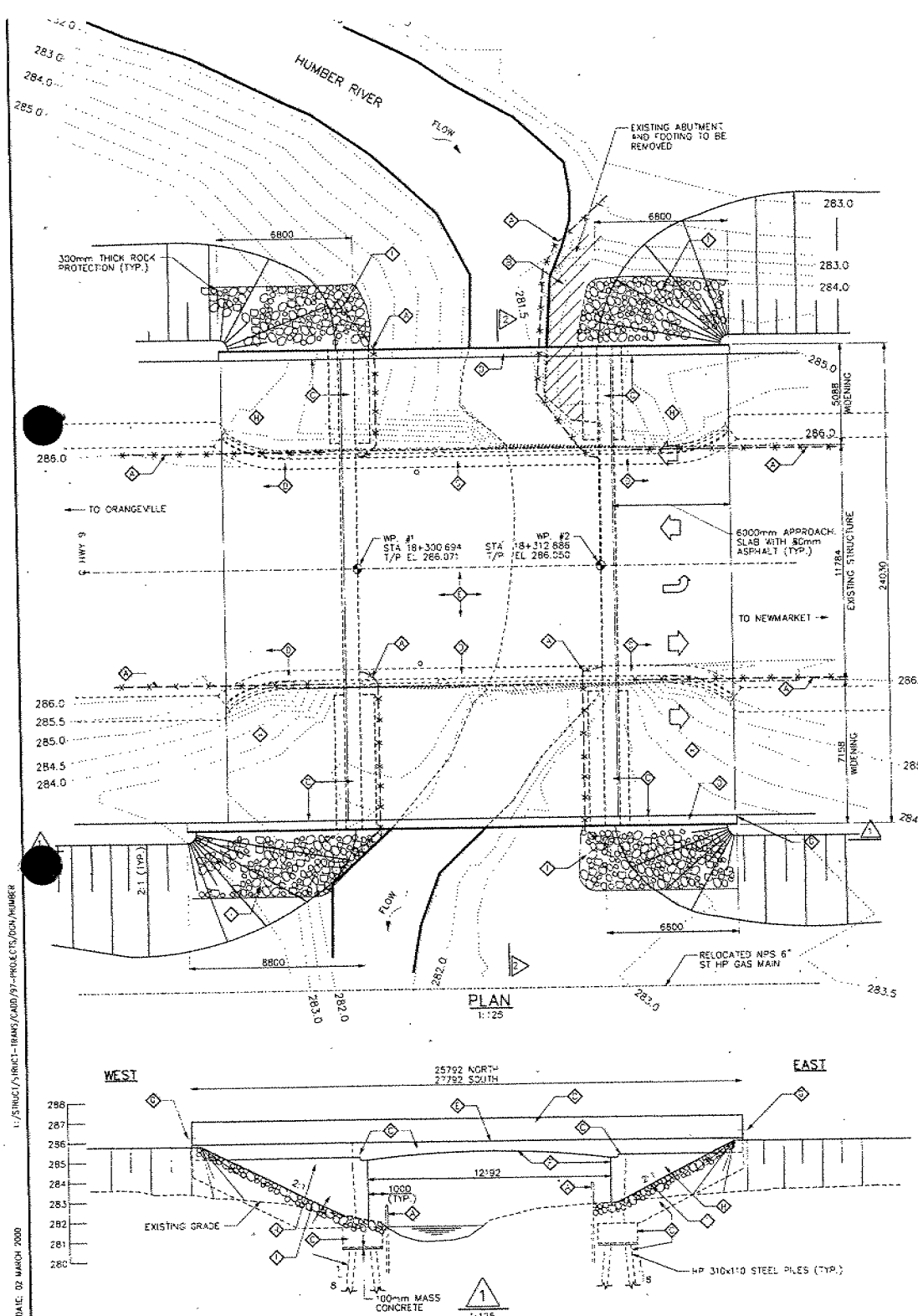
The Contractor shall submit a slurry method procedure for pile installation to the Contract Administrator for review prior to construction.

The Contractor shall control the intensity of ground vibrations during installation of piles closest to the existing structure abutments. The ground vibrations generated by pile driving shall be restricted to a maximum peak velocity of 100 mm/s in three (3) mutually perpendicular directions. Detailed records of monitoring shall be maintained at all times during pile installation.

CONCRETE IN FOOTINGS - Item No. 133**Special Provision****Scope of Work**

Work under this tender item shall consist of the placement of concrete in the widened footing locations, including the mass concrete plug at the bottom of each footing excavation, as shown on the Contract drawings.

11/STRUCT/STRUCT-TRANS/CADD/97-PROJECTS/CON/HUMBER
DATE: 02 MARCH 2000



DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No	2000-0047
WP No	530-91-00
HUMBER RIVER BRIDGE	SHEET 253
GENERAL ARRANGEMENT	
CS COLE SHERMAN	

- GENERAL NOTES**
- CLASS OF CONCRETE:
 - MASS CONCRETE: 20 MPa
 - FOOTINGS: 20 MPa
 - REMAINDER: 50 MPa HPC
 - CLEAR COVER TO REINFORCING STEEL:
 - FOOTINGS: 100 ± 25
 - DECK: TOP: 70 ± 20
 - REMAINDER: 50 ± 10
 - DECK: BOTTOM: 70 ± 20
 - REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX "C" DENOTE EPOXY COATED BARS.
 - HOOKS AND BENDS FOR REINFORCING STEEL SHALL BE DETAILED ACCORDING TO OHBCC-81. UNLESS SHOWN OTHERWISE, THE FOLLOWING SHALL APPLY:
 - a) STANDARD HOOKS WITH MINIMUM BEND DIAMETERS SHALL BE USED FOR STIRRUPS AND TIES ACCORDING TO CLAUSE 8-14.1.
 - b) OTHER BARS SHALL HAVE STANDARD HOOKS WITH BEND DIAMETERS ACCORDING TO CLAUSE 8-14.1.

- CONSTRUCTION NOTES**
- SANDFILLS IN CONCRETE, WHERE DESIGNATED, SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL WHICHEVER IS LESS.
 - THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
 - ALL CROSS FALLS ON DECK AND APPROACH SLABS TO MATCH EXISTING UNLESS OTHERWISE NOTED.
 - BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
 - PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING A4 ULTIMATE CAPACITY OF 3200 kN PER PILE.
 - UNWATERING SYSTEM TO BE INSTALLED FOR CONSTRUCTION OF NEW FOOTINGS AS PER CONTRACT DOCUMENTS.

- WORK DESCRIPTION**
- INSTALL ROADWAY PROTECTION AND SHEET PILING.
 - REMOVE EXISTING ABANDONED ABUTMENT AND FOOTING NORTH-EAST OF EXISTING STRUCTURE.
 - EXCAVATE FOR PILE AND FOOTING INSTALLATION AS REQUIRED. INSTALL PILES, POUR MASS CONCRETE AND CONSTRUCT NEW WIDENED FOOTING, ABUTMENT WALL, WINGWALL, AND SUPERSTRUCTURE.
 - REMOVE DECK DRAINS, TOP OF WINGWALLS, EXISTING CURB AND EXISTING CONCRETE HANDRAIL, AND CONSTRUCT NEW BARRIER WALLS (2 LOCATIONS).
 - REMOVE ASPHALT AND DECK WATERPROOFING. REMOVE 115mm OF CONCRETE FROM TOP SURFACE OF EXISTING DECK SLAB. PLACE 135mm CONCRETE OVERLAY. CONSTRUCT NEW APPROACH SLABS. WATERPROOF DECK AND PAVE.
 - CARRY OUT MISCELLANEOUS REPAIRS TO SOFFIT AND ABUTMENT WALL AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
 - INSTALL NEW STEEL BEAM GUDDERAIL AND CHANNEL (4 LOCATIONS).
 - BACKFILL BEHIND STRUCTURE AND REMOVE ROAD PROTECTION AND SHEET PILING.
 - INSTALL 300mm THICK ROCK PROTECTION (4 LOCATIONS).

- LIST OF DRAWINGS**
- GENERAL ARRANGEMENT
 - BOREHOLE LOCATIONS AND SOIL STRATA - SEE FDTN. REPORT
 - CONSTRUCTION STAGING
 - REMOVAL/REPAIR DETAILS I
 - REMOVAL/REPAIR DETAILS II
 - FOOTING LAYOUT AND REINFORCEMENT
 - EAST ABUTMENT LAYOUT AND REINFORCEMENT
 - WEST ABUTMENT LAYOUT AND REINFORCEMENT
 - WINGWALLS
 - DECK REINFORCEMENT AND SCREED ELEVATIONS
 - BARRIER WALLS W/O RAILING PERFORMANCE LEVEL 3
 - 6000 mm APPROACH SLAB
 - PILE DRIVING - STEAM & DIESEL HAMMERS
 - QUANTITIES - STRUCTURE
 - QUANTITIES - STRUCTURE

APPLICABLE STANDARD DRAWINGS

OPSD 3501.00	GRANULAR BACKFILL REQUIREMENTS - ABUTMENTS
OPSD 3506.02	BRIDGE DECK WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD.
OPSD 3506.03	BRIDGE DECK WATERPROOFING DETAILS AT ACTIVE WIDE CRACKS (GREATER THAN 2mm) AND CONSTRUCTION JOINTS.
OPSD 3922.00	SUPPORTS FOR BOTTOM REINFORCING STEEL FOR SLAB DEPTH > 500mm.
OPSD 4010.00	GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 4531.00	LOCATION OF SITE NUMBERS AND DATE FIGURES
OPSD 4670.00	TYPICAL JOINT DETAILS

REVISIONS

DATE	BY	DESCRIPTION	DATE
DESIGN	A.C.	CHK	31
DRAWN	B.O.	CHK	31
	A.C.	SITE	24-04-02
		STRUCT	
		SCHEME	

NO

NORTH

LEGEND

- ROCK PROTECTION
- UNWATERING SYSTEM
- ROADWAY PROTECTION

LIST OF ABBREVIATIONS

- T/P: TOP OF PAVEMENT
- WP: WORKING POINT
 STA: STATION | T/F: TOP OF FOOTING N.T.S.: NOT TO SCALE | EL: ELEVATION | TYP: TYPICAL | |


DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING



FAXGRAM

DATE April 7, 1999

PAGE 1 of 7

TO: Allan Cox
Cole Sherman
Fax: (905) 882-4399

FROM: Betty Bennett
MTO, Pavements & Foundations
Ph: (416) 235-4333
Fax: (416) 235-5240

SUBJECT: Highway 9 - Humber River Bridge Widening (Site No. 24-40)

Allan,

Further to your request for an NSSP covering the pre-augering of piles at the proposed widenings of the Humber River structure, we have revisited the piling recommendations provided in the Foundation Design report dated June 1996. Because of the nature of the subsurface material and the high groundwater table, it was felt that the pre-augering of piles might create greater disturbance to the existing structure than driving them from ground surface. It is recommended that the piles be driven without the pre-augering requirement and under the monitoring conditions outlined in the report. The Contractor should be advised to limit the driving energy and the number of blows of the hammer for the first 4 m of driving.

The temporary shoring requirement should refer to the NSSP for Roadway Protection Schemes (OPSS 941). The Contractor is responsible for the design of the shoring scheme, hence the provision for pre-augering and type of shoring system is not required.

I have appended a copy of the NSSP for Roadway Protection. It may not be the most recent version, so I will try to track down the most recent copy.

Regards,

B. Bennett

