



**FOUNDATION TECHNICAL MEMORANDUM
For
HIGHWAY 401 INTERCHANGE UNDERPASS
HIGHWAY 403 (WBL)
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
SITE 23-311, CONTRACT 8
GWP 3094-12-00
GEOGRAPHICAL TOWNSHIP OF OXFORD
OXFORD COUNTY, ONTARIO**

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- Reference 1. Foundation Investigation Report for W.P. 166-60-01, Site 23-89-311, Hwy. 401 Interchange, (Highway 403 W.B.L.), Highway 403, District 2, London, Dated November 16, 1982, GEOCRES No. 40P2-40.
- Reference 2. General Arrangement Drawing, Hwy. 401 Interchange, Hwy. 403 W.B.L., DWG 1, Sheet 59, Site 23-89-311, District No. 2, WP 166-60-01, Cont No. 87 20, , dated Feb, 1983.
- Reference 3. Foundation Layout Drawing, Highway 401 Interchange, Highway 403 W.B.L., DWG 3, Sheet 61, Site No. 23-89-311, District No. 2, WP 166-60-01, Cont No. 87-20, dated Feb, 1983.

Appendix B – Site Photographs

FOUNDATION TECHNICAL MEMORANDUM

For

Highway 401 Interchange Underpass/Highway 403 (WBL)
MTO West Region 59 Structure Rehabilitations
Site 23-311, Contract 8, GWP 3094-12-00
Geographical Township of Oxford
Oxford County, Ontario

1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detail foundation investigation and design for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. Ten (10) Group Work Projects (GWP's) are contemplated to be completed between 2014 and 2020.

This technical memorandum summarizes the factual results of geotechnical data based on the review and compilation of existing subsurface information from relevant reports in the MTO GEOCRES Library for the Highway 401 Underpass of Highway 403 West Bound Lane (WBL). The Foundation Engineering recommendations from the initial foundation reports are summarized with reference to the "Canadian Highway Bridge Design Code" (CHBDC) and follow in general the "Guidelines for Professional Engineers providing Geotechnical Engineering Services".

From the Minutes of Meeting Report, dated June 3, 2016, it is understood that the rehabilitation of the underpass structure will include conversion to semi-integral abutments.

The purpose of the technical memorandum is to summarize the subsurface and groundwater conditions and foundation recommendations based on available reports at the structure location for the design project team's reference.

The elevations in this report are expressed in meters, unless otherwise noted.



2. PROJECT SITE BACKGROUND AND GEOLOGY

The Highway 401 Interchange Underpass of Highway 403 is located about 33.0 km west of Brantford west limits in Geographical Township of Oxford, Oxford County, Ontario. A key plan is shown in Figure 1.

The existing underpass is a three span post tensioned concrete voided slab structure that carries two through lanes of Highway 403 WBL over Highway 401. The immediate vicinity is relatively flat. Agricultural and commercial lands at the north and south sides were observed in the vicinity of the structure. Generally, the terrain is gently sloping from east to west in the area.

Physiographically, the site of the underpass is located in the region referred to as Oxford Till Plain. The region consists of a drumlinized till plain formed when glacier advance overrode an existing moraine from north-westerly direction. The till is a calcareous boulder load in which limestone is the dominant material. The bedrock in the area is primarily Onondaga Formation limestone of Middle Devonian. The bedrock surface at the site location is between elevation 266.7 and 269.4 (Preliminary Map No. P.169, Bedrock Topography Series, Woodstock Sheet, Scale 1:50,000, Ontario Department of Mines, 1962).

3. SOURCE OF INFORMATION

The following reports and drawings, appended in Appendix A, were available for review and information for the underpass structure, subsoil information and original foundation recommendations.

1. Foundation Investigation Report for W.P. 166-60-01, Site 23-89-311, Hwy. 401 Interchange, (Highway 403 W.B.L.), Highway 403, District 2, London, Pavement & Foundation Design Section, Engineering Materials Office, Dated November 16, 1982, GEOCREC No. 40P2-40. (Reference 1)
2. General Arrangement Drawing, Hwy. 401 Interchange, Hwy. 403 W.B.L., DWG 1, Sheet 59, Site 23-89-311, District No. 2, WP 166-60-01, Cont No. 87 20, Dillon Consulting Engineers & Planners, dated Feb, 1983. (Reference 2)



3. Foundation Layout Drawing, Highway 401 Interchange, Highway 403 W.B.L., DWG 3, Sheet 61, Site No. 23-89-311, District No. 2, WP 166-60-01, Cont No. 87-20, Dillon Consulting Engineers & Planners, dated Feb, 1983. (Reference 3)

4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Highway 401 Interchange Underpass of Highway 403 (WBL) was carried out on August 28, 2015 by PML representative.

The site photographs present the current conditions of Highway 401 Interchange Underpass/Highway 403 (WBL) including visible portions of the abutments and pier, and abutment slope assessment based on visible areas, apparent areas of soil erosion and abutment slope cover.

The site inspection revealed that the vicinity of the underpass structure abutment locations was covered by vegetation. Both abutment front slopes were covered by concrete panels (Photographs 1 and 2). Numerous cracks were observed on the abutment walls. Sealant between the concrete panels degraded at locations where grasses were growing between the panels (Photographs 1 and 2). No weep holes were observed in the abutment walls. The piers at the time of site reconnaissance were observed with surficial cracks with no spalling of concrete or exposure of rebar (Photograph 3). However, numerous random cracks were observed on the west pier (Photograph 4). Both north and south slopes adjacent to the abutments were observed to be vegetated and no evidence of slope erosion was noted (Photographs 5 to 8). Surficial cracks were observed on the wingwalls of the east and west abutments (Photographs 5 to 8).

5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS

The general subsurface conditions presented in this section are based on the foundation investigation report referred in Reference 1.



The field investigation was carried out during the period from July 21 to 27, 1982. The investigation was carried out using a continuous flight auger drill rig equipped with 82 mm I.D. hollow stem augers. The investigation included five sampled boreholes accompanied with dynamic cone penetration tests (DCPTs). The sampled boreholes were drilled to 14.0 to 23.5 m, elevation 280.3 to 286.7. DCPTs were driven to practical refusal at 2.1 to 5.4 m, elevation 296.4 to 303.3.

The Foundation Investigation Report (Reference 1) includes the Borehole Locations & Soil Strata Drawing (Drawing No. 1666001-A, dated November 12, 1982) and Record of Borehole sheets (1 to 5). The general layout of the structure is shown in Reference 2.

5.1 General

In general, cohesive silty clay till was underlain by non-cohesive silty sand to sandy silt till at the site location.

5.2 Silty Clay Till

A 10.1 to 21.3 m thick firm to hard silty clay till was encountered surficially in all boreholes, which extended to 10.1 to 21.3 m, elevation 284.7 to 290.1. The silty clay till encountered was of low plasticity containing some sand and traces of gravel. The sand content generally increased with depth especially in boreholes 1 to 3. Moreover, in boreholes 1 and 4, dense silty sand layers were encountered within this deposit between elevation 300.0 and 298.0. N values recorded were between 11 and 47, indicating stiff to hard, with a local N value of 5 in borehole 3, indicating firm consistency.

Generally, undisturbed shear strengths in excess of 107 kPa were recorded where the cohesive till material did not fail during field vane shear testing. A field vane shear strength value of 34.5 kPa obtained in borehole 3 at approximate elevation 298.0 was considered to be non-representative of the general cohesive till deposit. An average shear strength value of 150.0 kPa for the silty clay till deposit was selected based on shear strength values from unconfined compression tests of similar material in the immediate vicinity of the site.



The grain size distribution result of a selected borehole 3 silty clay till sample included 30% clay, 55% silt, 11% sand and 4% gravel sized particles. Two Atterberg liquid limits obtained were 14 and 26 and corresponding plastic limits were 11 and 12, respectively, in boreholes 1 and 3. Plasticity index values obtained were 3 and 14. Moisture content determinations obtained were between 8.0 and 14.5%.

Grain size distribution results of two silty sand samples encountered within the silty clay till in borehole 1 indicated that the sample contained 10 and 14% clay, 43 and 37% silt, 42 and 40% sand, and 5 and 9% gravel sized particles, respectively.

5.3 Sandy Silt to Silty Sand Till

A 2.2 to 8.7 m thick very dense cohesionless sandy silt to silty sand till layer was encountered below the surficial silty clay till in all boreholes at 10.1 to 21.3 m, elevation 284.7 to 290.1. This till layer extended to the termination depths of 14.0 to 23.5 m, elevation 280.3 to 286.7. Auger refusal was met on probable boulder in borehole 5 at the termination depth 23.5 m, elevation 282.5. N values recorded for the sandy silt to silty sand till layer ranged from 100 and greater. Boulders were encountered while augering within this deposit.

Grain size distribution results indicated that the selected sandy silt to silty sand till samples contained 4 to 10% clay, 36 to 56% silt, 30 to 43% sand, and 4 to 27% gravel sized particles. Moisture content determinations obtained ranged from 6.0 to 10.5%.

5.4 Groundwater

The groundwater elevation was estimated at 298.0 based on field observations, data from previous foundation investigations in the immediate vicinity and well records.



6. FOUNDATION

6.1 Previous Foundation Recommendations

A grade separation was proposed so that the Highway 403 WBL would underpass Highway 401. The report indicated that the approach fills up to 10± m high would be required to establish the proposed profile grade of Highway 403. The Reference 1 report proposed a four-span structure.

In the report (Reference 1), the four alternatives proposed for the crossing of the proposed Highway 403 WBL over Highway 401, are summarized below.

Alternative 1 - Spread Footings on Silty Clay

The report proposed that the entire structure be supported on spread footings founded at or below the following elevations:

STATION (HIGHWAY 403)	PROPOSED STRUCTURE	ELEVATION
Station 11+257	West Abutment	297.6
Station 11+278	West Pier	298.6
Station 11+320	Middle Pier	297.5
Station 11+354	East Pier	300.1
Station 11+371	East Abutment	302.0

The report recommended a lateral resistance of 60 kPa and to cover the foundation with a 15.0 cm thick mass concrete pad within 18 hours of exposure.

A net safe bearing design pressure of 300.0 kPa was recommended. For purposes of the Ontario Highway Bridge Design Code (OHBDC), a factored bearing capacity of 450 kPa at ULS and a bearing capacity at SLS, Type II, of 300 kPa were recommended. It should be noted that the SLS Type II considered for a “substructure of a bridge or a retaining wall shall be proportioned so



that the estimated total and differential movements are not greater than the movements that can be accommodated" (OHBDC, 1979).

Alternative 2- Perched Footings on Compacted Fill

The report (Reference 1) recommended that alternatively the abutments could be supported on perched abutments on granular fill. Design details were provided in Figure 2 enclosed in the report. The report recommended that in the vicinity of the abutment locations all loose or soft material should be removed.

A friction coefficient of 0.6 was recommended in the report for computing sliding resistance between the base of the concrete footing and the compacted fill.

A net safe bearing design pressure of 340 kPa was recommended. For purposes of the OHBDC, a factored bearing capacity of 600 kPa at ULS and a bearing capacity at SLS, Type II, of 340 kPa were recommended.

Alternative 3 – Steel H-Piles in Overburden

Alternatively, it was recommended, in the report, that the entire structure may be supported on Steel H-Piles equipped with reinforced tips and driven in accordance with Ministry of Transportation and Communications Standards SS-103-10 or SS-103-11. The report recommended Ultimate Capacity values of 3000 and 2450 kN for 310 HP 110 and 310 HP 79, respectively, for calculation purposes. Furthermore, the following elevations, for estimation purposes, where it was assumed that the recommended pile capacities would be achieved.



STATION (HIGHWAY 403) CHAINAGE)	PROPOSED STRUCTURE LOCATION	ELEVATION
Station 11+257	West Abutment	287.0
Station 11+278	West Pier	285.0
Station 11+320	Middle Pier	285.0
Station 11+354	East Pier	283.0
Station 11+371	East Abutment	281.0

In the report, it was recommended to use particle sizes not exceeding 75 mm in the fill beneath the pile locations to facilitate pile driving, if the option to perch the abutment footings, supported on Steel H-piles, within the embankment fill was selected.

The following design values were recommended.

PILE TYPE	FACTORED CAPACITY AT ULS (KN)	CAPACITY AT SLS TYPE II (KN)
310 HP 110	1350	1000
310 HP 79	970	720

Alternative 4 – Combination of Previous Alternatives

The report suggested that the foundation alternatives presented could be combined and employed at different footing locations. As an example, the report presented that the centre pier could be supported on Steel H-piles while the abutment footings could be perched on compacted granular fill.

General Recommendations (for all alternatives)

- The report indicated that no stability problem was anticipated for the proposed embankment heights with 2:1 or flatter slopes.



- It was estimated that the total settlements under the embankments would be less than 75 mm of which 50% would occur during construction. It was estimated that the total settlement for the recommended footing foundation alternatives would be less than 25 mm. A differential settlement of less than 25 mm was anticipated. The report indicated that the anticipated settlements be considered during the design stage of the structure.
- The report recommended that the earth pressure acting on abutments and retaining walls be computed as per Subsection 6.6.1.2.2 of OHBDC assuming a yielding foundation with k_a of 0.33 for granular backfill.
- It was recommended to provide greater than 1.2 m cover for frost protection.
- In the report it was mentioned that dewatering was not anticipated to be a major problem because of the impermeable nature of the foundation soil.

Based on the General Arrangement Drawing (Reference 2), a three-span structure was to be built and approximately 10.0 and 4.0 m approach fills would be required at the west and east abutment locations, respectively, to establish the proposed profile grade of Highway 403.

Based on the General Arrangement and Foundation Layout Drawings (References 2 and 3), the west pier (Pier No. 1) was to be founded on 8.5 by 8.5 m, 1.8 m thick spread footing. The top of the spread footing at Pier No. 1 was to be placed at elevation 299.800. A 150 mm thick mass concrete was to be placed within 12 hours of completing excavation. The founding elevation for the footing was elevation 297.850.

The abutments and east pier (Pier No. 2) were to be founded on 310 HP 110 H-piles. The following table summarizes the pile data based on Foundation Layout Drawing (Reference 3).



LOCATION	NUMBER OF HP 310 X 110 PILES	BATTER	LENGTH* (mm)	PILE CUT-OFF ELEVATION
West Abutment	18	1:3 to 1:12	20,500	305.300
Pier No. 2	28	1:3 to 1:5 and Vertical	16,250	300.300
East Abutment	21	1:3 to 1:12	26,750	305.300

*Theoretical length below cut-off.

Furthermore, based on General Arrangement and Foundation Layout Drawings, temporary steel sheet piling was provided for road protection at Pier No. 2 only, which was to be removed after the excavation had been backfilled to grade

6.2 Assessment of Foundation Alternatives

Based on the original investigation and subsurface conditions encountered, the following table summarizes the foundation design parameters that were recommended in the Reference 1 report and updated in References 2 and 3. The updated geotechnical reaction at SLS and factored geotechnical resistance at ULS are provided.



FOUNDATION DESIGN PARAMETERS

FOUNDATION TYPE (LOCATION)	PROBABLE PILE TIP/ SPREAD FOOTING ELEVATION ¹	PREVIOUS LIMIT STATE DESIGN VALUES, OHBDC ²		LIMIT STATE DESIGN VALUES UPDATED TO CURRENT INDUSTRY PRACTICE ³	
		SLS TYPE II BEARING CAPACITY	ULS FACTORED CAPACITY	SLS BEARING REACTION/LOAD	ULS FACTORED GEOTECHNICAL RESISTANCE/LOAD
HP 310x110 Piles (West and east abutments and Pier No.2)	281 to 287	1000 (kN)	1350 (kN)	1000 (kN)	1350 (kN)
Spread Footing (Pier No. 1)	297.850	300 (kPa)	450 (kPa)	300 (kPa)	450 (kPa)

Notes:

1. Founding elevations from References 1 (Pile founding elevations) and 3 (Spread footing elevation).
2. Limit State Design values based on OHBDC from Reference 1.
3. Limit State Design values based on CHBDC (2014 Edition).

The Peak Ground Acceleration (PGA) for the site is 0.073 (National Building Code of Canada, 2015). The soil classification for seismic design should be in accordance with Clause 4.4.3.2 of the CHBDC (2014).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC 2014 Edition.

The foundation frost penetration depth at the site is 1.2 m according to OPSD 3090.101.

7. DISCUSSION

It is understood that for the Highway 401 Interchange Underpass, the rehabilitation will include conversion to semi-integral abutments. From a geotechnical point of view, at the present time, foundation work for the underpass structure is not expected provided that the total load on the structure does not increase or decrease by more than 10%.



A temporary support system may be required for the rehabilitation of the underpass structure and the construction for temporary support system should conform to OPSS 404 and 539. The contractor is responsible for the selection, detailed design and performance of the roadway protection scheme. The contractor should monitor the movement of the roadway protection system.

It is recommended that a preliminary investigation be carried out to supplement geotechnical data for the design of temporary roadway protection.

The slopes adjacent to both abutments were visually stable without signs of erosion. However, the embankments which are greater than 8.0 m height were constructed with a 2H:1V slope but not benched as per current practice (OPSD 202.010).

It is suggested that during the conversion of the existing abutments to semi-integral abutments, installation of wall drains to the abutments be considered to facilitate better drainage from behind the abutment walls. Wall drains are to be installed according to OPSD 3190.100.



8. CLOSURE

This Technical Memorandum was prepared by Mr. N. Rahman, P.Eng, Project Engineer and was reviewed by Mr. B. R. Gray, M.Eng, P.Eng., Principal Consultant. Mr. R. Ng, MBA, PhD, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

We trust this memo is sufficient for your immediate needs. Please, do not hesitate to contact us if you have any inquiries and/or comments.

Yours very truly,

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng
Project Engineer



Brian R. Gray, M.Eng, P.Eng.
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Robert Ng, MBA, PhD, P.Eng.
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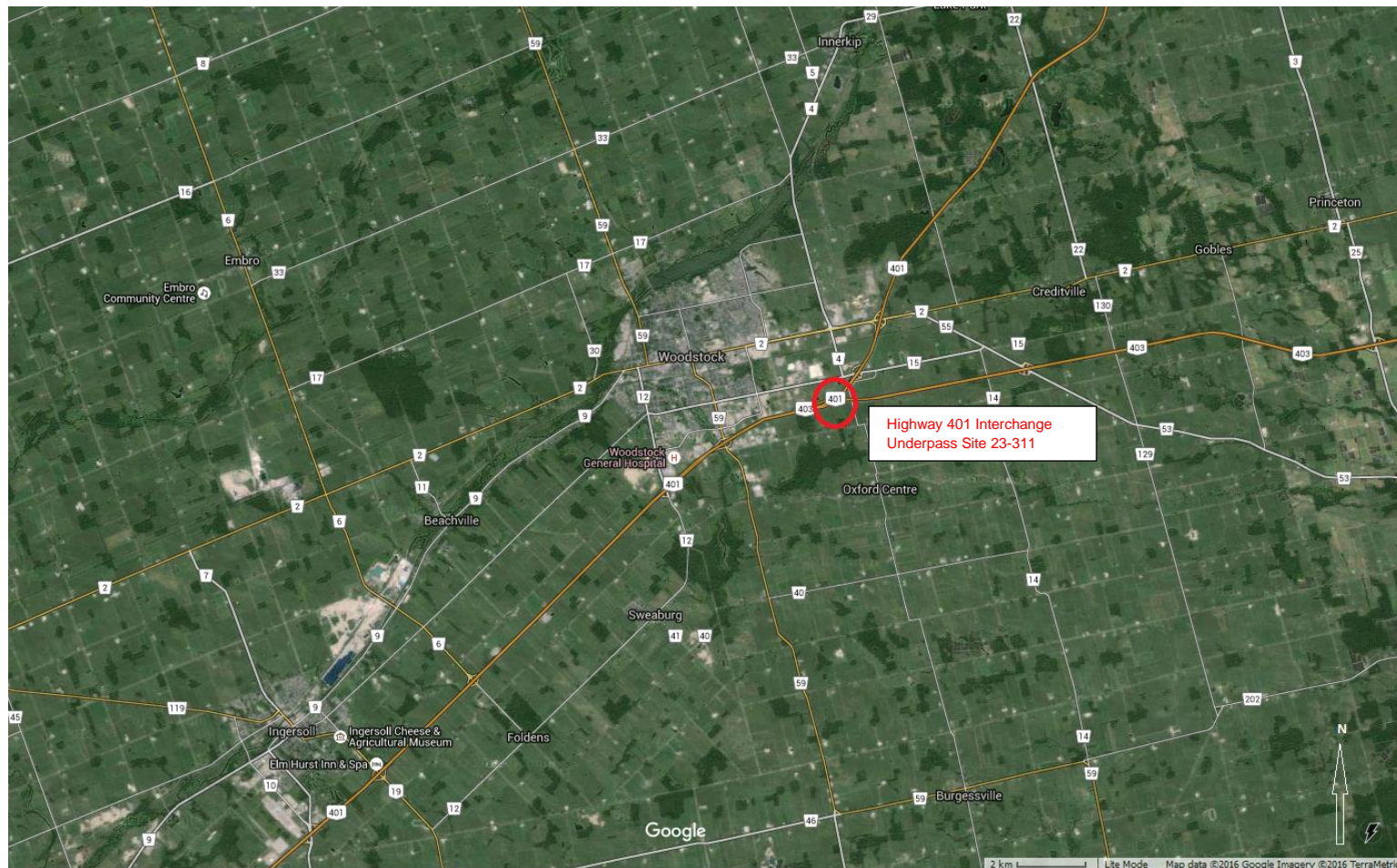


TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 404	Construction Specification for Support Systems
OPSS 539	Construction Specification for Temporary Protection Systems
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 3190.100	Walls, Retaining and Abutment Wall Drain
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan





APPENDIX A

Previous Foundation Investigation Report (GEOCRES 40P2-40)

- Reference 1. Foundation Investigation Report for W.P. 166-60-01, Site 23-89-311, Hwy. 401 Interchange, (Highway 403 W.B.L.), Highway 403, District 2, London, Dated November 16, 1982, GEOCRES No. 40P2-40.
- Reference 2. General Arrangement Drawing, Hwy. 401 Interchange, Hwy. 403 W.B.L., DWG 1, Sheet 59, Site 23-89-311, District No. 2, WP 166-60-01, Cont No. 87 20, , dated Feb, 1983.
- Reference 3. Foundation Layout Drawing, Highway 401 Interchange, Highway 403 W.B.L., DWG 3, Sheet 61, Site No. 23-89-311, District No. 2, WP 166-60-01, Cont No. 87-20, dated Feb, 1983.

**ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION**

WP 166-60-01 DIST 2
HWY 403 STR SITE 23-89-311
Hwy. 401 Interchange, (Hwy. 403 W.B.L.)

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FOUNDATION INVESTIGATION REPORT

For

W.P. 166-60-01, Site 23-89-311

Hwy. 401 Interchange, (Hwy. 403 W.B.L.)

Hwy. 403, District 2, London

INTRODUCTION:

This report summarizes the results of the foundation investigation required for the proposed structure and approach embankments at this site.

The fieldwork was conducted during the period from 82 07 21 - 27 utilizing a continuous-flight auger machine equipped with 82 mm I.D. hollow-stem augers.

This work consisted of 5 sampled boreholes/dynamic cone penetration tests.

SITE DESCRIPTION

The site is located east of Co. Rd. 15 along Hwy. 401 at Woodstock (Lot 13, Con. II, Twp. Norwich, Geog. Twp. (East) Oxford, County of Oxford).

Physiographically, the site is located in the Oxford Till Plain.

SUBSURFACE CONDITIONS

General

The Record of Borehole Sheets (Appendix) illustrate the conditions at the borehole locations. The location and elevations of the boreholes, and a stratigraphical profile based on the borehole data are shown on Drawing No. 1666001-A.

The overburden at this site is composed of till. A predominantly cohesive silty clay till overlies a predominantly non-cohesive sandy silt to silty sand till. The boundary between these two general soil types dips gradually towards the south (from elev. 290.1 m at BH #1 to elev. 284.7 m at BH #5).

Silty Clay (CL), Some Sand, Trace Gravel

This firm to hard till material extends from the surface for depths ranging from 10.1 m at BH #1 to 21.3 m at BH #5. The deposit is generally silty clay of low plasticity containing some sand and traces of gravel. The sand content generally increases with depth especially at BH #1, #2 and #3. Occasional layers of dense silty sand were encountered within this deposit at BH #1 and BH #4, generally between elev. 300 and elev. 298.

Physical properties of the cohesive portion of this material, as determined from field and laboratory tests, are summarized below:

	<u>Range</u>	<u>Average</u>	<u>Median</u>
Natural Water Content (w) %	8.0-14.5%	11.3%	N/A
Liquid Limit (W _L) %	14.0-26.0%	20.0%	N/A
Plastic Limit (W _p) %	11.0-15.0%	13.0%	N/A

Generally the material did not fail during field vane shear testing, indicating undisturbed shear strengths in excess of 107 kPa. A field vane shear strength value of 34.5 kPa, obtained near elev. 298 m at BH #3, was considered to be non-representative of the general deposit. Shear strength values from unconfined compression tests of similar material in the immediate vicinity of this site led to the selection of 150 kPa as the average shear strength for the silty clay till deposit.

Figure 1 illustrates a typical grain size distribution for the cohesive portion of this deposit.

Sandy Silt to Silty Sand, Some Gravel, Trace Clay

This very dense till deposit underlies the silty clay till across the entire site. As indicated in Drawing No. 1666001-A, occasional layers of cohesive silty clay were encountered between elev. 286 m and elev. 282 m at BH #1 and BH #3. Occasional boulders were encountered while augering within this deposit. The natural water content of the non-cohesive portion of this deposit ranged from 6.0 - 10.5%.

Figure 2 illustrates a typical grain size distribution for the non-cohesive portion of this deposit.

Groundwater

The groundwater elevation was estimated at 298 m. This estimate was based on field observations, data from previous foundation investigations in the immediate vicinity, and well records.

DISCUSSION AND RECOMMENDATIONS

A grade separation is proposed so that the west-bound lanes of Hwy. 403 will overpass Hwy. 401. In order to establish the proposed profile grade of Hwy. 403, approach fills up to 10± m high will be required. A four-span structure is proposed.

General Recommendations (Applicable To All Alternatives)

- No stability problems are anticipated for the proposed embankment heights with slopes of 2:1 or flatter.
- Total settlements under the embankments will be less than 75 mm, 50% of which should occur during construction. Total settlement for the recommended footing foundation alternatives will be less than 25 mm. The resulting differential settlements will be less than 25 mm. The structure should be designed to accommodate the anticipated settlements.
- Earth pressure acting on abutments and retaining walls should be computed as per Subsection 6.6.1.2.2 of the O.H.B.D.C. assuming a yielding foundation with $k_a = 0.33$ for granular backfill
- For frost protection, cover should be greater than 1.2 m
- Dewatering is not anticipated to be a major problem because of the impermeable nature of the foundation soil

Foundation Alternatives

The foundation alternative which leads to the least expensive design should be adopted.

Alternative 1 - Spread Footings on Silty Clay

The entire structure may be supported on spread footings founded at or below the following elevations:

<u>Station (Hwy. 403 Chainage)</u>	<u>Elevation</u>
Station 11 + 257	297.6
Station 11 + 278	298.6
Station 11 + 320	297.5
Station 11 + 354	300.1
Station 11 + 371	302.0

For resistance to lateral forces, the adhesion between the base of the footings and the foundation soil = 60 kPa.

Cover the foundation soil with a 15 cm pad of mass concrete within 18 hours of exposure.

The following design values are recommended:

- net safe bearing pressure = 300 kPa

and for purposes of the O.H.B.D.C.:

- Factored Bearing Capacity at U.L.S. = 450 kPa
- Bearing Capacity at S.L.S. Type II = 300 kPa

Alternative 2 - Perched Footings on Compacted Fill

The abutments may be supported on perched abutments on compacted granular fill. Refer to the enclosed Figure 2 for design details.

All loose or soft material beneath the approach embankment in the vicinity of the abutment locations should be removed.

For computing sliding resistance between the base of the concrete footing and the compacted fill, the friction coefficient = 0.6.

The following design values are recommended:

- net safe bearing capacity = 340 kPa

and for purposes of the O.H.B.D.C.:

- Factored Bearing Capacity at U.L.S. = 600 kPa
- Bearing Capacity at S.L.S. Type II = 340 kPa

Alternative 3 - Steel H-Piles in Overburden

The entire structure may be supported on Steel H-Piles equipped with reinforced tips and driven in accordance with M.T.C. Standards SS103-10 or SS103-11. For calculation purposes the following values are recommended:

<u>Pile Type</u>	<u>Ultimate Capacity</u>
310 HP 110	3000 kN
310 HP 79	2150 kN

For estimation purposes, it may be assumed that the recommended pile capacities will be achieved at the following elevations:

<u>Station (Hwy. 403 Chainage)</u>	<u>Elevation</u>
Station 11 + 257	287
Station 11 + 278	285
Station 11 + 320	285
Station 11 + 354	283
Station 11 + 371	281

If desired, the abutment footings (supported on Steel H-Piles) may be perched within the embankment fill. In this case, to facilitate pile driving, particle sizes in the fill immediately beneath the pile locations should not exceed 75 mm.

The following design values are recommended:

<u>Pile Type</u>	<u>Safe Capacity</u>
310 HP 110	1000 kN
310 HP 79	720 kN

and for the purposes of the O.H.B.D.C.:

<u>Pile Type</u>	<u>Factored Capacity at U.L.S.</u>	<u>Capacity at S.L.S. Type</u>
310 HP 110	1350 kN	1000 kN
310 HP 79	970 kN	720 kN

Alternative 4 - Combination of Previous Alternatives

If desired, the previous alternatives may be combined so that different alternatives are employed at different footing locations.

For example, the centre pier footing may be supported on Steel H-piles while the abutment footings are perched on compacted granular fill.

MISCELLANEOUS

The fieldwork for this project was carried out under the supervision of Mr. B. Yiu (student field technician). The report was written by Mr. D. H. Dundas, Project Foundations Engineer, and reviewed by Mr. K. G. Selby, Senior Foundations Engineer. The equipment used was owned and operated by Atcost Soil Drilling Inc.



D. H. Dundas

D. H. Dundas, P. Eng.
Project Foundations Engineer

K. G. Selby

K. G. Selby, P. Eng.
Senior Foundations Engineer

A P P E N D I X

RECORD OF BOREHOLE No 1

METRIC

W P 166-60-01 LOCATION Sta. 11 + 257 & W.B.L. ORIGINATED BY BY
DIST 2 HWY 403/401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DD
DATUM Geodetic DATE 82 07 22 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
300.2	Ground Surface															GR SA SI CL
0.0	occ. layers of Silty Sand, Compact		1	SS	16	**										5 42 43 10
			2	SS	17											9 40 37 14
	Silty Clay (CL)		3	SS	29											
	Some Sand		4	SS	24											
	Trace Gravel		5	SS	11											
	Stiff to Very Stiff (Till)		6	SS	10											
	with Sand		7	SS	20											
			8	SS	18											5 43 38 14
290.1	Sandy Silt to Silty Sand		9	SS	100/	24 cm										19 37 37 7
	Some Gravel		10	SS	100/	22 cm										15 37 42 6
	Trace Clay		11	SS	100/	22 cm										
	Very Dense (Till)		12	SS	100/	18 cm										
	occ. layers of Silty Clay (CL) Hard		13	SS	100/	24 cm										
281.9			14	SS	100/	19 cm										
18.3	End of Borehole															*Cu > 107kPa
	** Stabilized ground water level estimated from well records															

RECORD OF BOREHOLE No 2

METRIC

W P 166-60-01 LOCATION Sta. 11 + 278 & W.B.L. ORIGINATED BY HY
DIST 2 HWY 403/401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DP
DATUM Geodetic DATE 82 07 23 CHECKED BY DP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
300.7	Ground Surface																
0.0	Silty Clay (CL)																
	Some Sand		1	SS	13												
	Trace Gravel		2	SS	18												
	Stiff to Hard		3	SS	33												
	(Till)		4	SS	34												
			5	SS	29												
			6	SS	10												
			7	SS	20												
	with Sand		8	SS	15												
			9	SS	16												
289.1																	
11.6	Sandy Silt to Silty Sand, Some Gravel		10	SS	1007	18 cm											
	Trace Clay																
	Very Dense (Till)		11	SS	1007	17 cm											
286.7																	
14.0	End of Borehole																
	** Stabilized ground water level estimated from well records																*C _u > 107kPa

+3, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL BAWLOCATION

RECORD OF BOREHOLE No 3

METRIC

W P 166-60-01 LOCATION Sta. 11 + 320 ± W.B.L. ORIGINATED BY BY
DIST 2 HWY 403/401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DD
DATUM Geodetic DATE 82 07 21 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
302.1	Ground Surface															GR SA SI CL
0.0	Silty Clay (CL) Some Sand Trace Gravel Firm to Hard (Till)		1	SS	16	** ↓	302									4 11 55 30
			2	SS	18		300									
			3	SS	15		298									
			4	SS	5		296									
			5	SS	22		294									
			6	SS	24		292									
			7	SS	17		290									
			8	SS	20		288									
			9	SS	21		286									
			10	SS	12		284									
289.0	with Sand						282									
13.1	Sandy Silt to Silty Sand Some Gravel Trace Clay Very Dense (Till)		11	SS	100/	15 cm	288									27 33 36 4
			12	SS	100		286									
							284									
							282									
	occ. layers of Silty Clay (CL) Hard		13	SS	100											*Cu > 107kPa
280.3			14	SS	100											
21.8	End of Borehole															
	** Stabilized ground water level estimated from well records															

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



Ministry of
Transportation and
Communications

RECORD OF BOREHOLE No 4

METRIC

W P 166-60-01

LOCATION Sta. 11 + 354 P.W.B.L.

ORIGINATED BY BY

DIST 2 HWY 403/401

BOREHOLE TYPE Hollow Stem Auger

COMPILED BY DD

DATUM Geodetic

DATE 82 07 27

CHECKED BY DD

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	W' VALUES			SHEAR STRENGTH kPa								
							20	40	60	80	100	20	40	60	80	100
301.9	Ground Surface															
0.0			1	SS	31											
			2	SS	29											
	occ. layers of Silty Sand, Compact		3	SS	19											
			4	SS	15											
	Silty Clay (CL)															
	Some Sand		5	SS	21											
	Trace Gravel															
	Stiff to Hard (Till)		6	SS	26											
			7	SS	29											
			8	SS	36											
			9	SS	47											
			10	SS	32											
288.0			11	SS	57											
13.9	Sandy Silt to Silty Sand															
	Some Gravel		12	SS	100											4 30 56 10
	Trace Clay															
	Very Dense (Till)		13	SS	100											9 43 42 6
284.7	End of Borehole															*C _u > 107kPa
17.2	** Stabilized ground water level estimated from well records															

+3, x⁵: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 5

METRIC

W P 166-60-01 LOCATION Sta. 11 + 375, 5.0 m Rt. of W.B.L. ORIGINATED BY BY
DIST 2 HWY 403/401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DD
DATUM Geodetic DATE 82 07 23-26 CHECKED BY DD

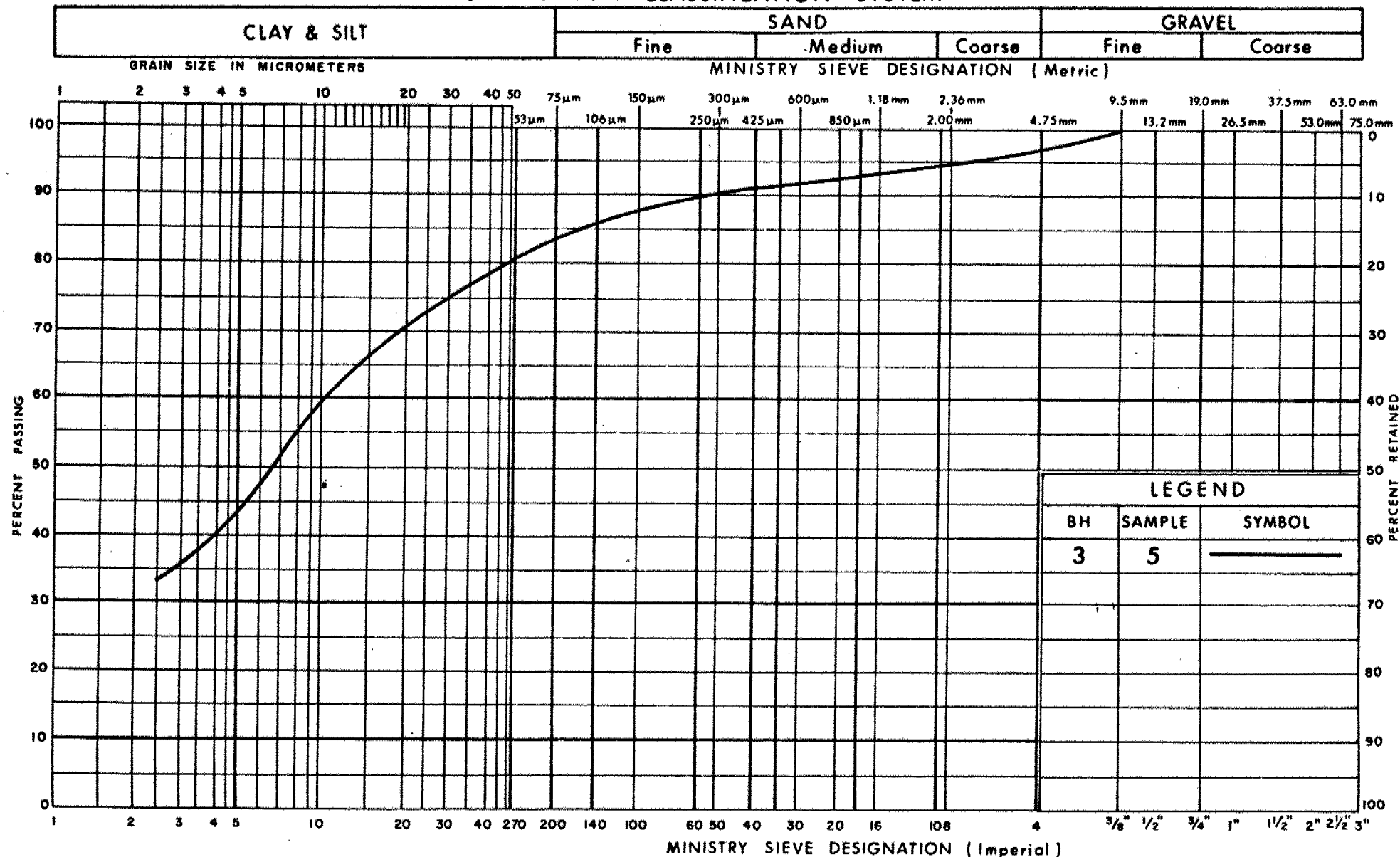
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
306.0	Ground Surface												GR SA SI CL
0.0	Silty Clay (CL) Some Sand Trace Gravel Stiff to Hard (Till)		1	SS	29								
			2	SS	37								
			3	SS	29								
			4	SS	15								
			5	SS	19								
			6	SS	17								
			7	SS	17								
			8	SS	27								
			9	SS	19								
			10	SS	25								
			11	SS	27								
			12	SS	29								
			13	SS	31								
284.7			14	SS	100	20 cm							
21.3	Sandy Silt to Silty Sand Some Gravel, Trace Clay Very Dense (Till)		15	SS	100	28 cm							
282.5	Probable Boulder Refusal to Auger End of Borehole ** Stabilized ground water level estimated from well records												*Cu > 107kPa

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXAMINATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM



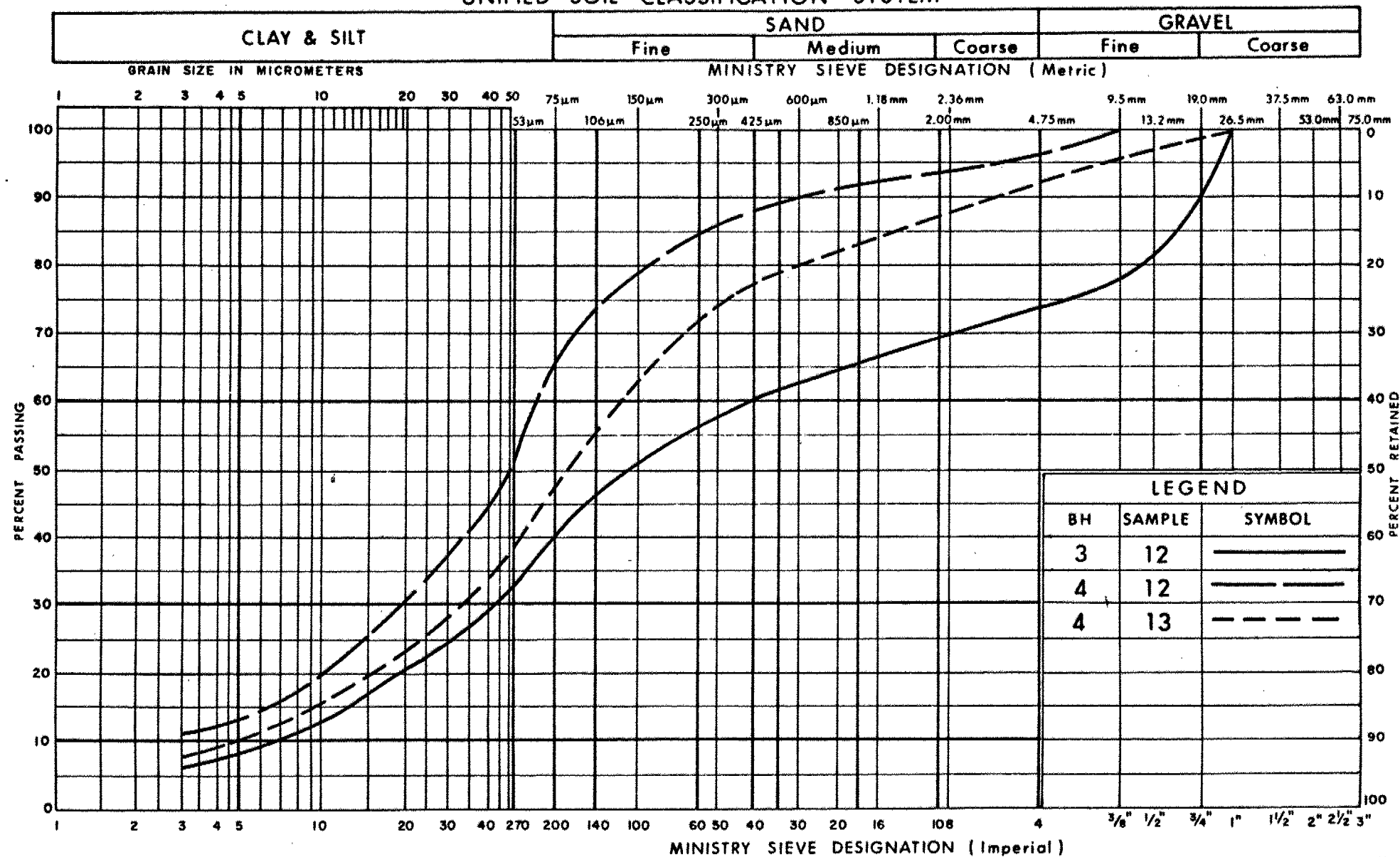
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GRAIN SIZE DISTRIBUTION
SILTY CLAY (Till)
SOME SAND TRACE GRAVEL

FIG No 1

W P 166-60-01

UNIFIED SOIL CLASSIFICATION SYSTEM



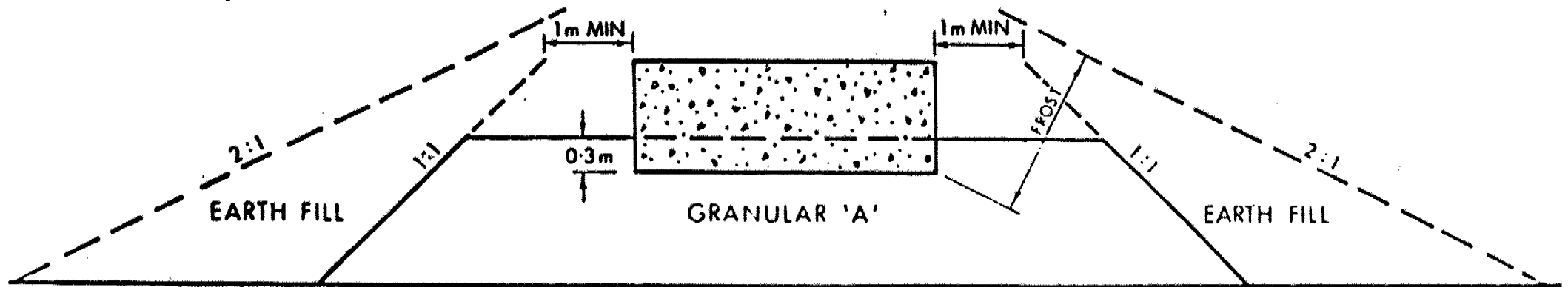
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND
SOME GRAVEL TRACE CLAY

FIG No 2

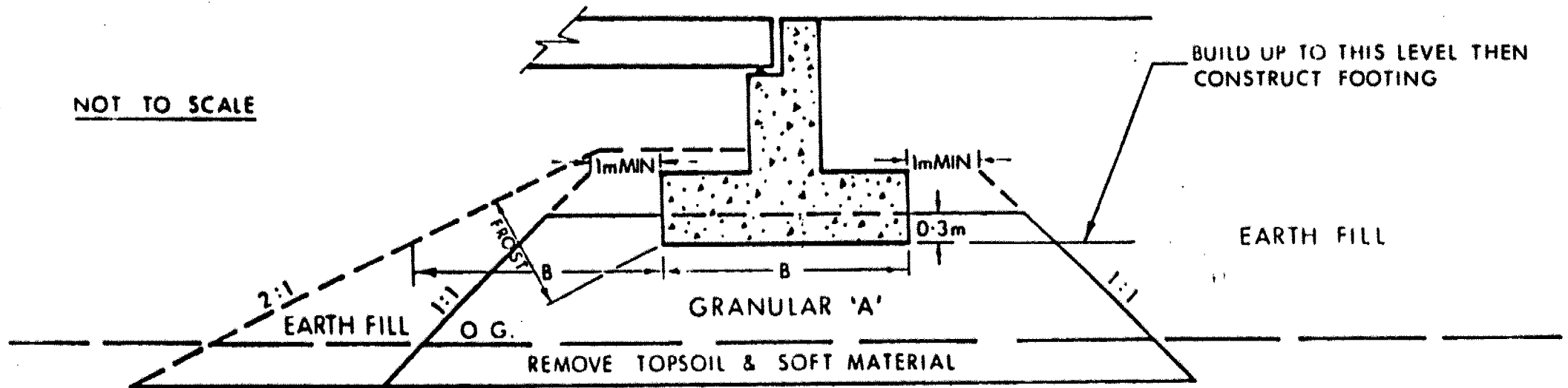
W P 166-60-01

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



X SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_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'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kn/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m^2	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.

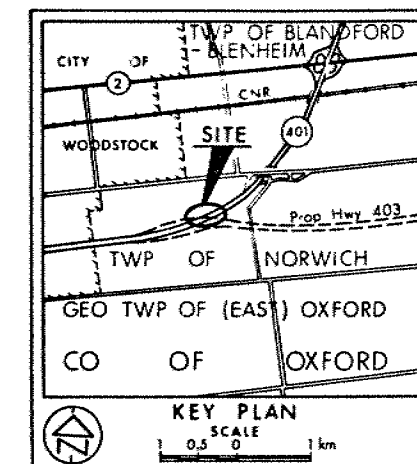
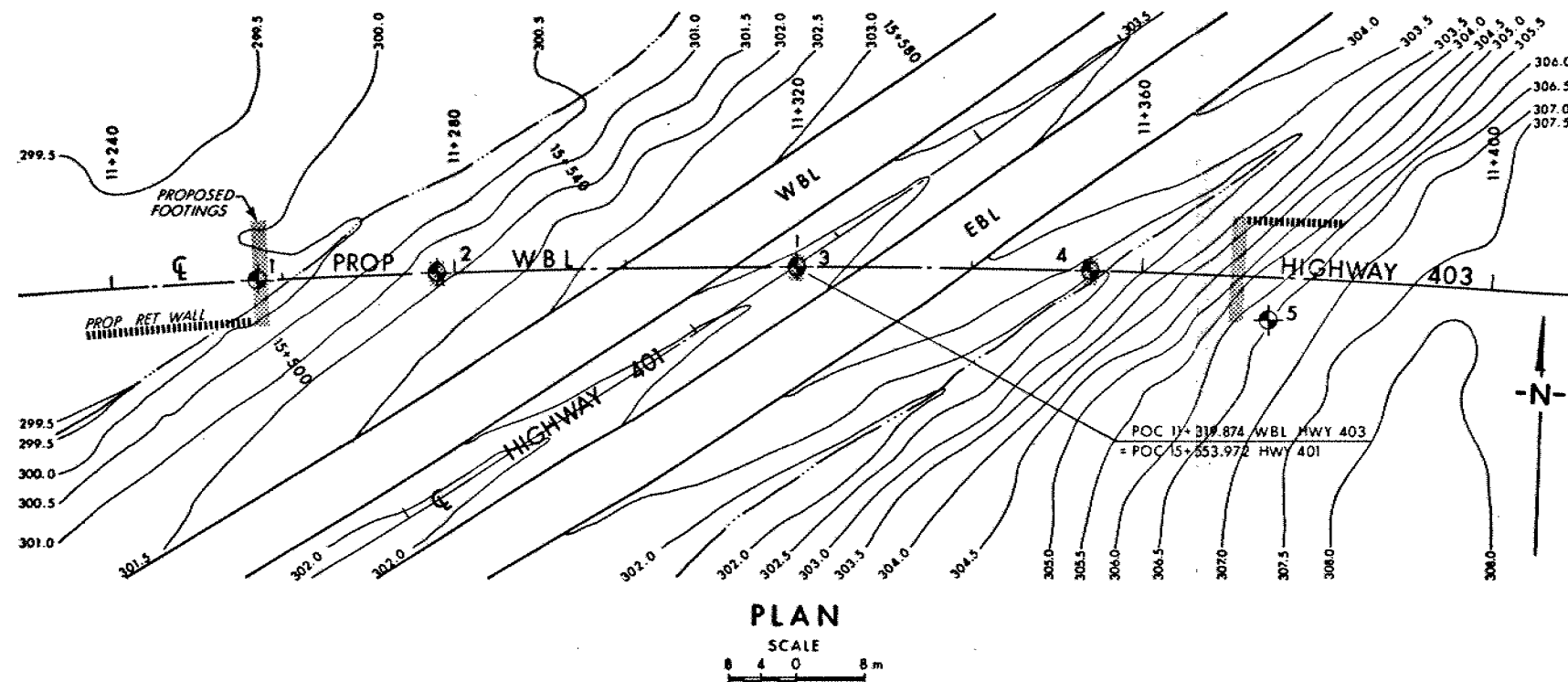
CONT No
WP No 166-60-01



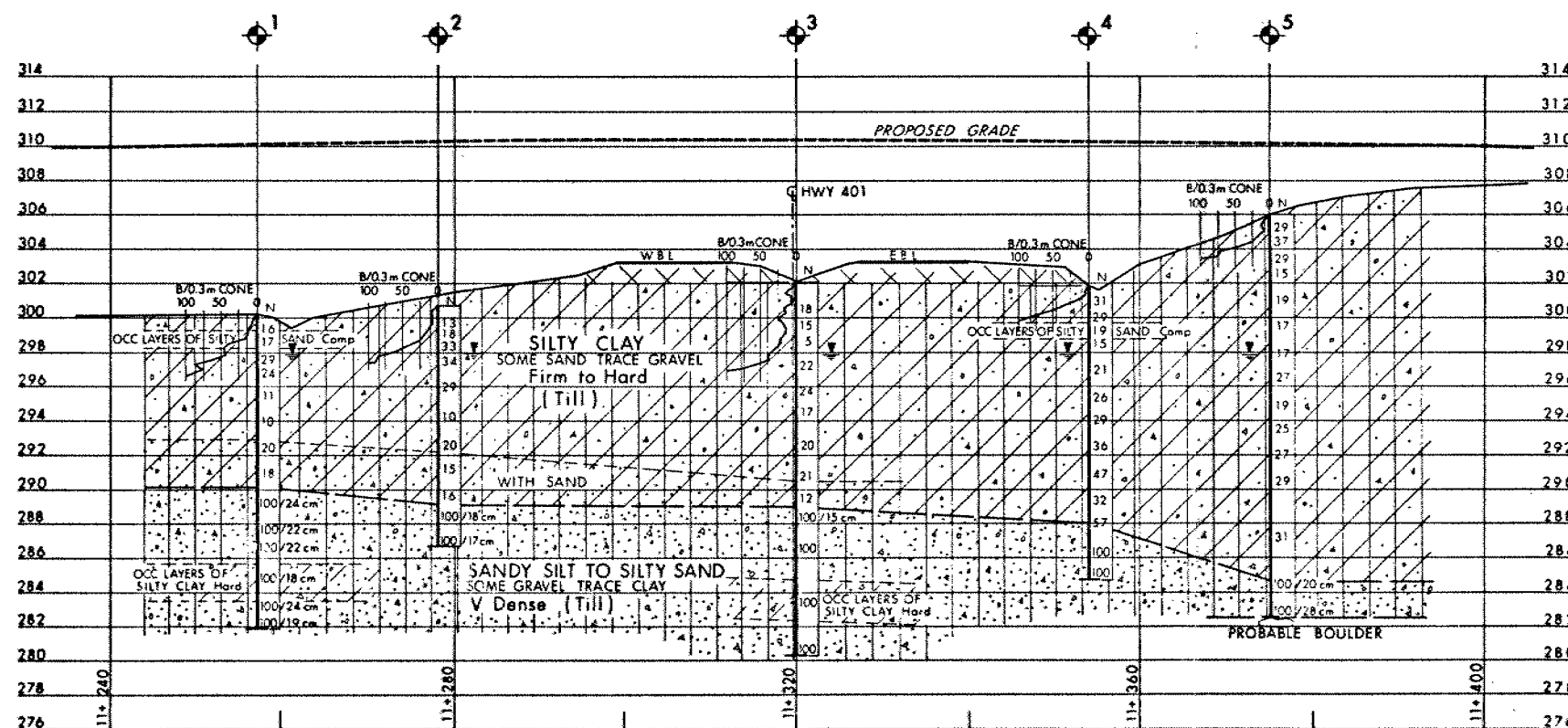
HWY 401 INTERCH O'PASS

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



- LEGEND**
- ◆ Bore Hole
 - ⊕ Dynamic Cone Penetration Test (Cone)
 - ⊕ Bore Hole & Cone
 - N Blows/0.3m (Std Pen Test, 475 J/blow)
 - CONE Blows/0.3m (60° Cone, 475 J/blow)
 - W.L. at time of investigation 82.07
 - NOTE: Stabilized Ground Water Level Estimated from Well Records



No	ELEVATION	STATION	OFFSET
1	300.2	11+257	CL
2	300.7	11+278	CL
3	302.1	11+320	CL
4	301.9	11+354	CL
5	306.0	11+375	5 m RT

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

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

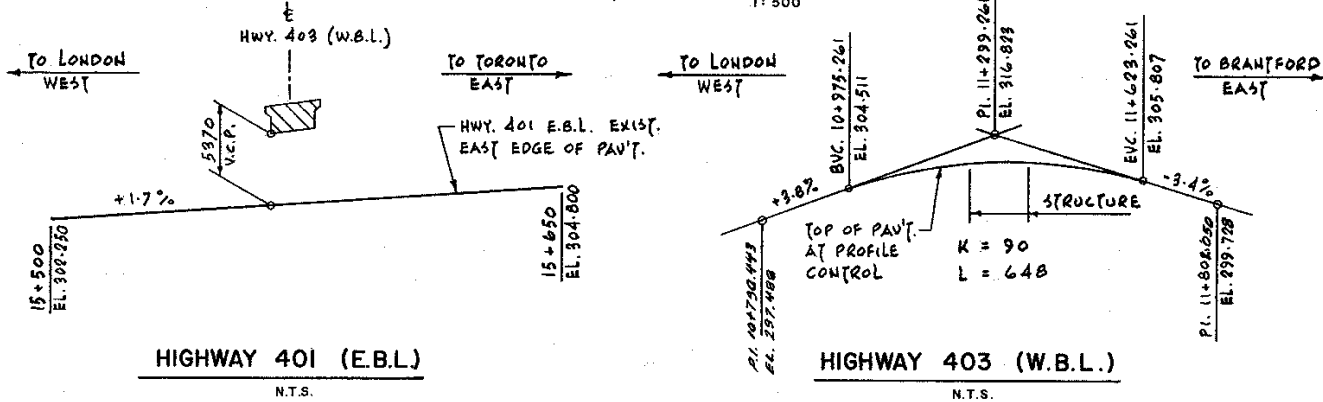
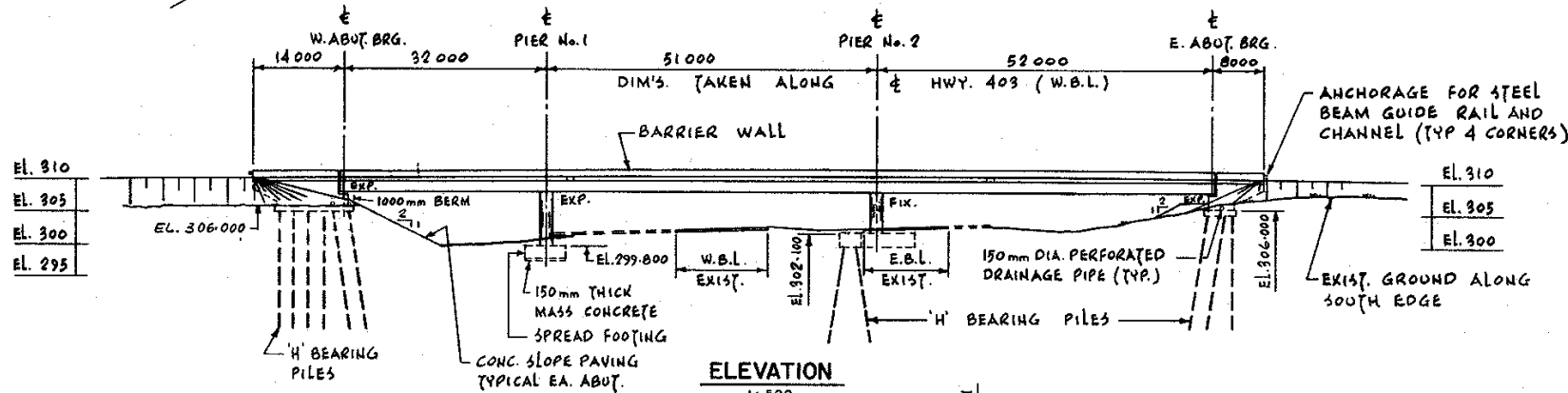
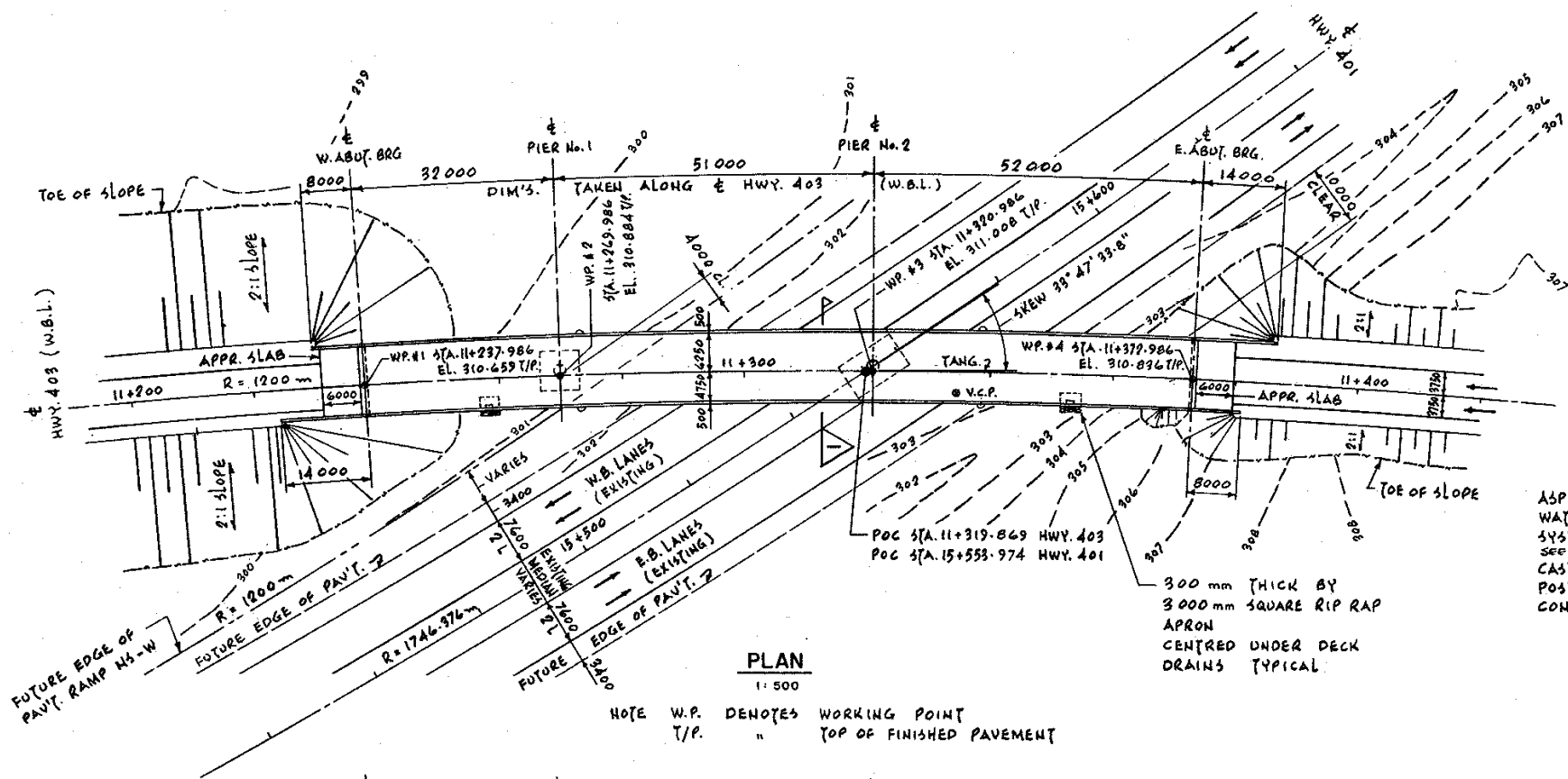
REV.	DATE	BY	DESCRIPTION

Geocres No 40P2-40

HWY No 403 WBL	DIST 2
SUBM'D D.D. CHECKED	DATE 82.11.12
DRAWN SO CHECKED	DATE 82.11.12

166-60-01-A

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO DESIGN 104 91-86



METRIC

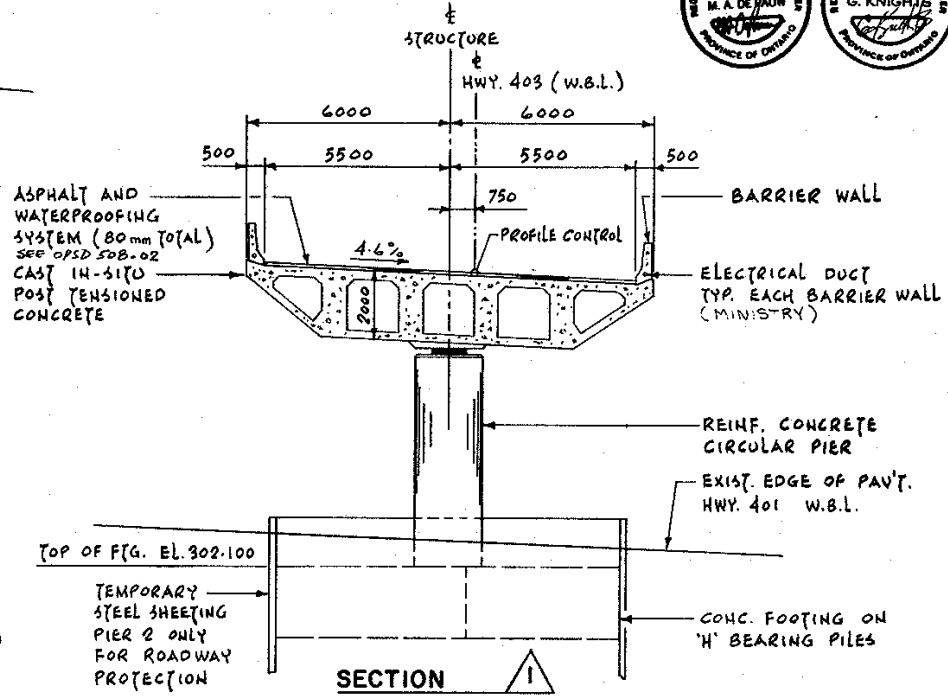
DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN. ELEVATIONS, COORDINATES, CURVE AND ALIGNMENT DATA ARE IN METRES. STATIONS ARE IN KILOMETRES + METRES.

DISTRICT No 2
CONT No 87-20
WP No 166-60-01

HWY. 401 INTERCHANGE
HWY. 403 W.B.L.
GENERAL ARRANGEMENT

SHEET
59

DILLON
Consulting Engineers & Planners



LIST OF DRAWINGS

- 1 GENERAL ARRANGEMENT
- 2 BOREHOLE LOCATION & SOIL STRATA
- 3 FOUNDATION LAYOUT
- 4 FOUNDATION REINFORCEMENT
- 5 WEST ABUTMENT LAYOUT
- 6 WEST ABUTMENT REINFORCEMENT
- 7 EAST ABUTMENT LAYOUT
- 8 EAST ABUTMENT REINFORCEMENT
- 9 PIERS AND BEARINGS
- 10 DECK LAYOUT & SCREED ELEVATIONS
- 11 LONGITUDINAL CABLE DETAILS I
- 12 LONGITUDINAL CABLE DETAILS II
- 13 TRANSVERSE CABLES I
- 14 TRANSVERSE CABLES II
- 15 DECK REINFORCEMENT I
- 16 DECK REINFORCEMENT II
- 17 DECK REINFORCEMENT III
- 18 EXPANSION JOINTS
- 19 BARRIER WALL
- 20 APPROACH SLABS
- 21 DETAILS OF CONC. SLOPE PAVING
- 22 STANDARD DETAILS
- 23 EMBEDDED ELECTRICAL I
- 24 EMBEDDED ELECTRICAL II - N/A
- 25 PILE DRIVING
- 26 BRIDGE DATE AND SITE NO.
- 27 AS CONSTRUCTED ELEVATIONS AND DIMENSIONS
- 28 QUANTITIES - STRUCTURE
- 29 QUANTITIES - STRUCTURE

GENERAL NOTES

CLASS OF CONCRETE

PIERS, DECK	35 MPa
BARRIER WALLS, ABUTMENTS	
WINGWALLS, APPR. SLABS	30 MPa
REMAINDER UNLESS OTHERWISE NOTED	20 MPa

CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25
PIERS	80 ± 20
ABUTMENTS AND WINGWALLS	FF 80 ± 20
DECK TOP SLAB	TOP BARS 70 ± 20
	BOT. BARS 40 ± 10
BOT. SLAB & WEBS	40 ± 10

REMAINDER 70 ± 20 OR AS SHOWN ON STANDARD DRAWINGS

GRADE OF REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400

REINFORCING BARS MARKED WITH THE SUFFIX "C" SHALL BE EPOXY COATED.

CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 3mm.

NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND GROUTED.

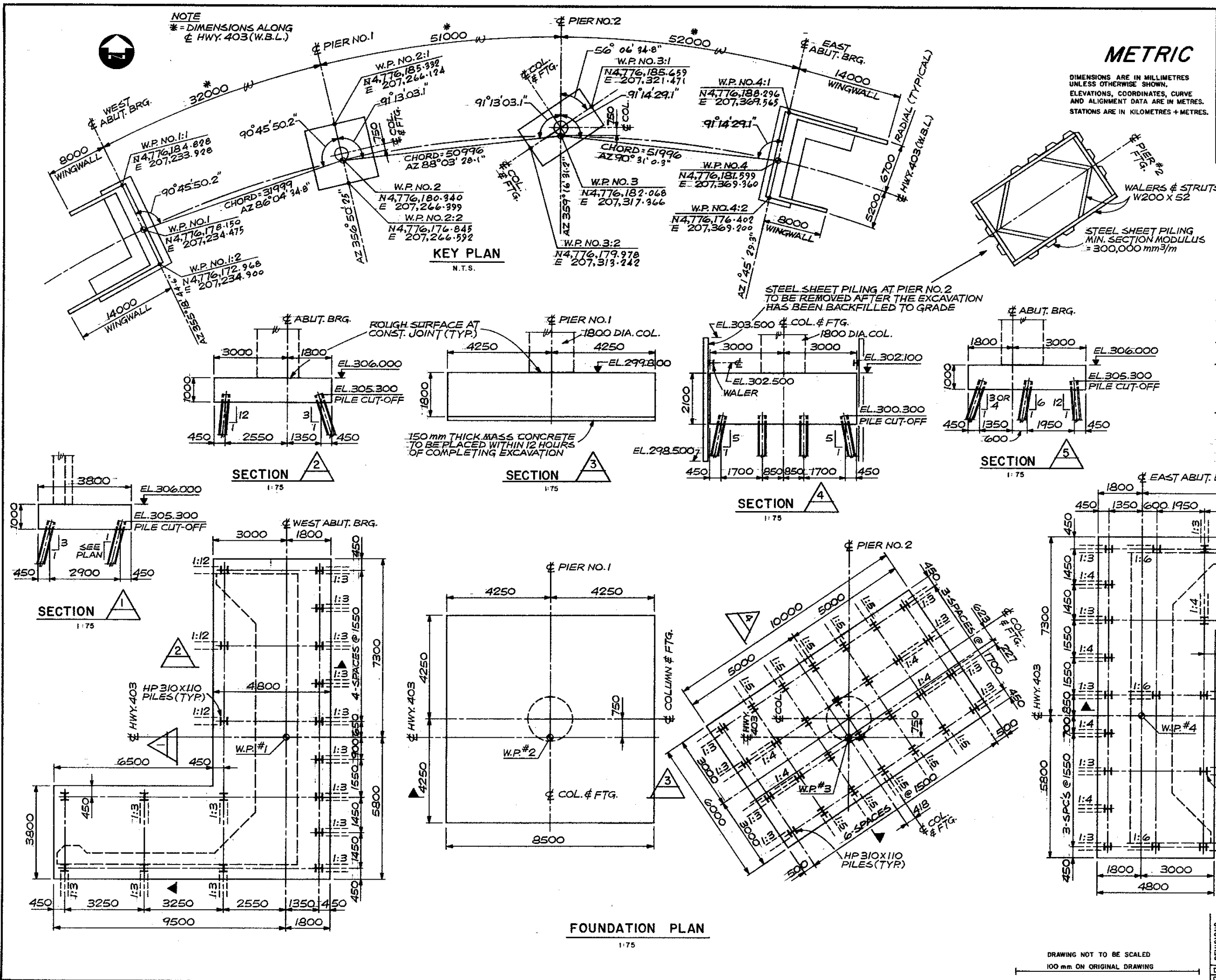
BM 301-282
Nail & Washer in Root of 0.3m
Maple Tree 44.4m LI of E. 401
Sta 15+582.7

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN M.D.P.	CHECK G.K.	LOADING O.H.D.C.A.79	DATE FEB 1983
DRAWING L.T.L.	CHECK G.K.	SITE 23-89-311	DWG 1

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO DB-BR 15M 91-99



DISTRICT No 2
CONT No 87-20
WP No 166-60-01

HWY. 401 INTERCHANGE
HWY. 403 W.B.L.

FOUNDATION LAYOUT

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SHEET
61

PILE DATA TABLE		
LOCATION	NO.	LENGTH
WEST ABUT.	18	20500
PIER NO. 2	28	16250
EAST ABUT.	21	26750

- NOTES**
- ALL PILES ARE HP 310 X 110
 - PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTING
 - DESIGN LOAD AT S.L.S. II = 1000 KN
FACTORED CAPACITY AT U.L.S. = 1350 KN
 - PILE LENGTH SHOWN ON DRAWING IS THEORETICAL LENGTH BELOW CUT-OFF.
 - LEGEND
H = VERTICAL PILE
H --- = DIRECTION OF PILE BATTER
1:3, 1:4, 1:5, 1:6 & 1:12 = RATE OF BATTER
 - PROVIDE DRIVING SHOES FOR ALL PILES AS PER DD-3301
 - PILE CONSTRUCTION DATA
PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING AN ULTIMATE CAPACITY OF 3000 KN PER PILE (SEE DWG. 25)



REVISIONS		DATE	BY	DESCRIPTION

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DESIGN M.D.P. CHECK G.K. LOADING OHBDC-A79 DATE FEB. 93
DRAWING G.L.B. CHECK G.K. SITE 23-89-311 DWG 3



APPENDIX B

Site Photographs



Photograph 1: Looking at the east abutment of the Highway 401 Underpass Structure. Random cracks were observed on the abutment wall. Sealant between the concrete panels appeared degraded at locations where grasses are growing. The toe of the front slope was vegetated with localized erosion of soil at ground surface (August 28, 2015).



Photograph 2: Looking at the west abutment of the Highway 401 Underpass Structure. Random cracks were observed on the abutment wall. Sealant between the concrete panels appeared degraded at locations where grasses are growing. The toe of the front slope was vegetated at ground surface (August 28, 2015).



Photograph 3: Looking west at the piers of the Highway 401 Underpass Structure. Numerous cracks were observed on the west pier. (August 28, 2015).



Photograph 4: Looking west at the west pier. Numerous random cracks were observed on the pier. The ground around the pier was well vegetated (August 28, 2015).



Photograph 5: Looking at the south wingwall and the adjacent slope of the east abutment of the Highway 401 Underpass Structure. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 6: Looking at the north wingwall and the adjacent slope of the east abutment of the structure. Surficial vertical cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 7: Looking at the south wingwall and the adjacent slope of the west abutment of the Highway 401 Underpass Structure. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 8: Looking at the north wingwall and the adjacent slope of the west abutment of the Highway 401 Underpass Structure. Minor surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).