**FOUNDATION TECHNICAL MEMORANDUM**

**For**

**COLONEL TALBOT ROAD (HIGHWAY 4) UNDERPASS**
**HIGHWAY 402**
**MTO WEST REGION 59 STRUCTURE REHABILITATIONS**
**SITE 19-544, CONTRACT 9**
**GWP 3098-12-00**
**GEOGRAPHIC TOWNSHIP OF WESTMINSTER**
**MIDDLESEX COUNTY, ONTARIO**


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PML Ref.: 13KF006J-CT
Index No.: 403FTM
GEOCRES No.: 40I14-94
February 26, 2018

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

**FOUNDATION TECHNICAL MEMORANDUM**
**For**
Colonel Talbot Road (Highway 4) Underpass Highway 402
MTO West Region 59 Structure Rehabilitations
Site 19-544, Contract 9, GWP 3098-12-00
Geographic Township of Westminster
Middlesex County, Ontario

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## 1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detailed 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) 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 Colonel Talbot Road (Highway 4) Underpass Highway 402. 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 January 31, 2017, it is understood that the structure will undergo semi-integral conversion to eliminate the abutment expansion joints. It is also recommended to replace the barrier walls at this structure location due to poor condition of the barrier walls.

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.

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## 2. PROJECT SITE BACKGROUND AND GEOLOGY

The Colonel Talbot Road (Highway 4) Underpass Highway 402 is located about 1.6 km (1 mile) south of the Town of Lambeth in the Geographic Township of Westminster, Middlesex County, Ontario. A key plan is shown in Figure 1.

The existing underpass is a two-span pre-stressed girder structure that carries two thru lanes and a speed change lane in each direction of Colonel Talbot Road (Highway 4) over Highway 402. The immediate vicinity is relatively flat. Agricultural lands at the northwest and south sides were observed in the vicinity of the structure and commercial and residential areas at the northeast side were observed.

Physiographically, the site of the underpass is located in the region referred to as the “Mount Elgin Ridges”, which consist of a series of ridges and vales. The ridges are composed mainly of pale brown calcareous clay or silty clay deposits, and it is common to find alluvium of gravel, sand or silt in the vales. The ridges are well drained, while poor drainage prevails in the hollows. The bedrock consists of limestone, dolostone and shale, and belongs to the Dundee Formation from the Middle Devonian Period. The bedrock surface is approximately at the depth of 61.0 to 91.4 m (200.0 to 300.0 ft.), elevation 163.2 to 195.1 m (535.3 to 640.1 ft.) (Preliminary Geological Map No. 238, Ontario Department of Mines, 1964).

## 3. SOURCE OF INFORMATION

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

Reference 1. Foundation Investigation Report for Hwy. 4 Underpass Hwy 402, Twp. Of Westminster, Dist. 2, London, W.P. 41-66-08, Site 19-544, Contract No. 77-61, Soil Mechanics Section, Geotechnical Office, Ministry of Transportation and Communications, dated June 10, 1975, GEOCRES No. 40I14-94.

Reference 2. Pile Load Tests – Hwy. 402, Quality of Timber Piles, W.P. 41-66-08/16, Soil Mechanics Section, Engineering Materials Office, dated August 8, 1977.

Reference 2A. Pile Capacities – Contract 77-61, W.P. 41-66-08/06/07, Hwy. 402, District 2, London, Soil Mechanics Section, Engineering Materials Office, dated August 11, 1977.

Reference 2B. Driving Record of Test Piles Drawing, HWY 4 Interchange Underpass, DWG. 19-544-2-1. Sheet 165-1, Site No. 19-544, Cont. No. 77-61, WP No. 41-66-08, dated Aug. 15, 1977.

Reference 3. General Plan Drawing, Highway 4 Interchange Underpass, DWG 1-A, Sheet 164-A, Site No. 19-544, Dist. 2, Cont. No. 77-61, WP No. 41-66-08, dated August 1976.

Reference 4. Foundation Layout, Reinf & Piers Drawing, Highway 4 Interchange Underpass, DWG 3-A, Sheet 166-A, DWG 3-A, Site No. 19-544, Cont. No. 77-61, WP No. 41-66-08, dated August 1976.

## 4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Colonel Talbot Road (Highway 4) Underpass Highway 402 structure was carried out on September 2, 2015.

The site photographs present the conditions of the underpass structure including visible portions of the abutments, wingwalls and piers and abutment slope assessment based on visible areas, apparent areas of soil erosion and abutment slope cover.

Surficial cracks were observed on the abutment walls. Rock protection was provided for the front slopes of the abutments and the effect of erosion on the slope face was not observed (Photographs 1 and 4). Surficial cracks were observed on the wingwalls and the adjacent slopes of the abutments were vegetated. The effect of erosion was not observed on the slope faces (Photographs 2, 3, 5 and 6). However, a void was observed under the rock protection and geotextile at south abutment that exposed a concrete element (Photographs 6 and 7). Minor surficial cracks were observed on the piers. The ground around the piers was partially covered with vegetation and the effect of erosion on the ground surface was not observed (Photograph 8).

## 5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS

A foundation investigation report (Reference 1) was prepared by the Ministry of Transportation and Communications. The general subsurface conditions presented in this section are based on Reference 1.

The Foundation Investigation Report (Reference 1) includes Records of Borehole Sheet, Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core and the Borehole Locations & Soil Strata Drawing (DRAWING No. 416608-A). The general layout of the structure is shown in Reference 3.

The field investigation was carried out from May 1 to 7, 1975. A total of six sampled boreholes 1 to 6 accompanied by a dynamic cone penetration test (DCPT) adjacent to all boreholes were carried out at the site location. The sampled boreholes were advanced to 9.6 to 44.7 m (31.5 to 146.5 ft.), elevation 210.2 to 245.0 m (689.5 to 803.8 ft.). The DCPTs were advanced to approximate termination depths of 2.4 to 2.8 m (7.9 to 9.1 ft.), elevation 252.0 to 253.3 m (826.8 to 831.1 ft.).

The drilling machine was provided and operated by Master Soil Investigation Limited. A 50 mm (2 in.) O.D. split spoon sampler was driven to collect the disturbed samples in accordance with the specifications for the Standard Penetration Test.

The samples obtained were visually examined and classified in the field after recovery as well as in the laboratory following delivery. Laboratory tests, including natural moisture content determinations, Atterberg limits, grain size distribution and undrained shear strength were conducted on selected samples. The test results were summarized on the Record of Borehole Sheets.

The locations and elevations of all the boreholes were surveyed in the field by personnel from Engineering Surveys Section of London Region.

## 5.1 General

Generally, stiff to hard clayey silt interbedded with layers of compact to very dense silt and sand was encountered at the site location.

### 5.1.1 Clayey Silt

Stiff to hard clayey silt was encountered surficially in all boreholes. The clayey silt extended to termination depths 9.6 to 44.7 m (31.5 to 146.5 ft.), elevation 245.0 to 210.2 m (803.8 to 689.5 ft.) for all boreholes except borehole 4.

In borehole 4, the clayey silt extended from surface, elevation 255.8 m (839.2 ft.), to 16.8 m (55.0 ft.), elevation m 239.0 (784.2 ft.), below which a silt layer was encountered.

Compact to very dense cohesionless layer was encountered within the clayey silt in boreholes 3 and 6.

The upper 4.6 m (15.0 ft.) thick portion of this clayey silt stratum represented the desiccated crust with brown color. The color changed to grey beneath the crust layer. Between 4.6 and 12.2 m (15.0 and 40.0 ft.) the consistency of the clayey silt was stiff to very stiff. The consistency of the clayey silt was very stiff to hard at depths greater than 12.2 m (40.0 ft.).

N values of the clayey silt recorded ranged from 10 to over 100, indicating stiff to hard consistency condition.

Grain size distribution results of selected clayey silt samples included 0 to 3% gravel, 1 to 5% sand, 54 to 74% silt and 25 to 45% clay sized particles. The Atterberg liquid limits ranged from 20 to 38 and the corresponding plastic limits ranged from 13 to 19. The plasticity index values ranged from 6 to 19. Moisture content determinations ranged from 11 to 25%.

It should be noted that the grain size test result of sample 10 from borehole 5 included 0% gravel, 1% sand, 88% silt, and 11% clay sized particles. The result indicated that the sample is comprised of predominately silt sized particles. No Atterberg limit results were available for this sample to identify the soil based on plasticity chart. No moisture content result was available for this sample either.

The results of undrained shear strength of clayey silt obtained from four field vane tests in boreholes 1, 2, 5 and 6 ranged approximately from 69 to 184 kPa (1433 to 3840 psf), indicating stiff to very stiff consistency. The laboratory unconfined tests conducted on clayey silt samples from boreholes 1 and 2 determined undrained shear strength results 83 and 117 kPa (1733 and 2433 psf), respectively, indicating stiff to very stiff consistency. The unit weight of two selected samples from boreholes 1 and 2 were determined 19.8 and 20.7 $kN/m^3$ (126 and 132 $lb/ft^3$), respectively.

### 5.1.2 Silt and Sand/Silt

A 3.5 to 10.4 m (11.5 to 34.0 ft.) thick discontinuous silt and sand/silt was encountered in boreholes 3, 4, and 6 at 6.7 to 16.8 m (22.0 to 55.0 ft.), elevation 239.0 to 248.1 m (784.2 to 814.0 ft.).

The silt and sand/sand and silt layer extended to 17.1 and 17.7 m (56.0 and 58.0 ft.), elevation 237.7 and 238.0 m (780.0 and 781.0 ft.) in boreholes 3 and 6, respectively. N values recorded ranged from 26 to 205 in boreholes 3 and 6, indicating compact to very dense compactness condition.

Grain size distribution results of selected silt and sand/sand and silt samples from boreholes 3 and 6 determined that the samples included 0% gravel, 48 and 47% sand and 52 and 53% silt and clay sized particles, respectively. The moisture content of the samples were approximately 18 to 21%.

It should be noted that the grain size result of sample 9 from borehole 6 included 0% gravel, 1% sand, 97% silt and 2% clay sized particles. The result indicated the sample comprised predominately silt sized particles. The moisture content of the sample was approximately 19%.

The silt layer in borehole 4 extended to the termination depth 20.3 m (66.5 ft.), elevation 235.5 (772.7 ft.). N values recorded were 100, 171 and 221, indicating very dense compactness condition.

Grain size distribution result of one selected sample from borehole 4 determined that the sample included 0% gravel, 1% sand, 89% silt and 10% clay sized particles. The Atterberg liquid limit of one selected sample from borehole 4 was 18 and the corresponding plastic limit was 15. The plasticity index value was 3, indicating non-plastic consistency. One moisture content determination of a silt sample was 18%.

#### 5.1.3 Groundwater

Groundwater was observed in boreholes 1, 3 and 5. The groundwater was encountered in the sand and silt stratum and slowly rose to the depths of 0.8 to 0.9 m (2.6 to 3.0 ft.), elevation 253.7 to 254.3 m (832.3 to 834.2 ft.). Groundwater level was not established in boreholes 2, 4 and 6.

## 6. FOUNDATION

### 6.1 Previous Foundation Recommendations

The previous foundation recommendations presented in the following sections are based on the foundation recommendations presented in Reference 1.

The foundation recommendations in Reference 1 were provided for a two span single structure to carry the Colonel Talbot Road (Highway 4) over proposed Highway 402. Each span was to be 34.7 m (114 ft.) in length and the embankments were to be approximately 6.1 m (20 ft.). At the site location, a deposit of stiff to hard clayey silt with a discontinuous layer of silt and sand/silt was encountered.

### 6.1.1 Structure Foundations

#### *6.1.1.1 Center Pier*

It was recommended that the center pier be supported on spread footings at approximate elevation 253.0 m (830.0 ft.). For the design purpose, a net safe bearing pressure of 287 kPa (3 tsf) was recommended. It was also recommended that to determine the resistance to sliding an adhesion design value of 96 kPa (2000 psf) may be used.

#### *6.1.1.2 Perched Abutments*

Reference 1 recommended that the abutments be constructed in the approach fills and supported on steel tube piles with dimension (manufacture size) of 323.9 mm O.D. and 6.35 mm wall thickness (12 ¾ in. O.D. and ¼ in. wall thickness). It was stated that the piles were to be driven to elevation 252.4 m (828.0 ft.) for the south abutment and to elevation 253.0 m (830.0 ft.) for the north abutment. The report also stated that the piles should not be driven below the recommended elevations as undrained shear strength decreased with depth at the site location. For the design purpose, a safe load of 222 kN (25 tons) per pile was assumed. Battered piles were recommended to provide resistance to any horizontal loading.

As an alternative, the report recommended that the abutments may be supported on spread footings placed on well compacted Granular Base Course (G.B.C.) Class ‘A’ as was shown in the attached Figure 1 - Abutment on Compacted Fill Showing Granular ‘A’ Core with the original report (Reference 1). The report indicated that a net safe design load of 239 kPa (2.5 tsf) may be assumed. Further, the report indicated that a friction coefficient of 0.6 may be assumed to apply between the footing and G.B.C. Class ‘A’ for calculations of sliding resistances.

Based on Figure 1 in Reference 1, it was recommended that 1) the topsoil and / or soft subsoil under area of compacted Granular ‘A’ be removed, 2) the Granular ‘A’ be placed to top of the footing level and compacted according to M.T.C. standards, and 3) excavate compacted Granular ‘A’ material for footing.

Based on References 2, 2A, 3 and 4, “as built” abutments were supported on No. 14 Treated Timber Piles.

According to Reference 2, six timber piles initially were found to be defective in that they split and shattered during the first one or two blows of the pile driving hammer.

The recommended pile design data presented in Reference 2A is summarized below.

| LOCATION | SAFE CAPACITY | REQUIRED LENGTH | DRIVE TO ELEVATION, m (ft.) |
|---|---|---|---|
| North Abutment | 311 kN/pile (35 tons/pile) | 5.5 m (18ft.) | 252.5 (828.5) |
| South Abutment | | 6.7 m (22 ft.) | 251.9 (826.5) |

The field driving records of the test piles are presented in Reference 2B. The timber pile driving tests were carried out near the crossing of Colonel Talbot Road (Highway 4) and Highway 402 near station 202+03.

### 6.1.2 Settlements

Based on Reference 1, the long term settlement of the underlying soil was anticipated to be approximately 50 mm (2 in.) at the abutments and 25 mm (1 in.) at the center pier. Therefore, it was recommended that the ability to tolerate the resulting differential settlement be considered in the underpass structure design. It was also recommended that the approach embankments be constructed in advance of the structure by as long a period as possible to minimize the differential settlements. No documents or construction drawings were available for review to confirm if the approach embankments were constructed in advance of the structure as was recommended in the original report, Reference 1.

#### 6.1.3 Dewatering

Due to the relatively impervious nature of the subsoil, no dewatering problems were anticipated during excavations for footings.

#### 6.1.4 Approach Embankments

No stability problems were expected for the anticipated 6.1 m (20 ft.) high approach fills provided the approaches were constructed with 2 horizontal to 1 vertical slopes. It was recommended that no material that had grain size larger than 76 mm (3 in.) was to be placed at the locations where the piles had to be driven. Reference 3 restricted maximum size compacted boulder free fill to 50 mm (2 in.)

#### 6.1.5 Other Considerations

Due to the relatively impervious subsoil at the site location, it was indicated that no dewatering problems were anticipated during the footing excavations.

A minimum 1.2 m (4 ft.) cover for the pile caps or spread footings for frost protection was recommended.

#### 6.1.6 Drawings

The pile driving records for No. 14 treated timber piles are presented in Reference 2B, which includes the pile location, the stratigraphy of test location, dimensions of tapered timber piles to be driven, types of timber, pile helmet details and the driving records for each pile. Three different sizes of No. 14 Treated Timber Piles (tapered) made from Southern Yellow Pine and Red Pine were driven. The pile driving tests were carried out using a 2766.9 kg (6100 lbs) hammer. The height of hammer fall during the tests varied between 0.30 and 0.83 m (1.0 and 2.75 ft.) The penetration resistance blows per foot for the pile load tests varied from 0 to 65.

Based on the General Arrangement Drawing (Reference 3), a two-span single structure with pre-stressed trapezoidal girders was to carry Colonel Talbot Road (Highway 4) over the future

Highway 402. The elevation of Highway 402 at the centreline of eastbound lanes (EBL) and westbound lanes (WBL) was designed to be at approximate elevation 255.0 m (836.7 ft). The grades of Colonel Talbot Road at the approaches were designed to be raised approximate 5.3 to 5.6 m (17.5 to 18.3 ft.). The average original ground level was at approximate elevation 256.0 m (840.0 ft.).

In Reference 3, concrete slope paving with 2 horizontal to 1 vertical slope was specified in front of the abutments, and compacted boulder free fill (maximum size of 50 mm (2 in.) was to be placed up to underside of abutment footings before driving piles. A 152 mm (6 in.) corrugated steel pipe (C.S.P.) was specified behind the north and south abutments at elevation 258.28 m (847.50 ft.) and 258.93 m (849.50 ft.), respectively. A 762 mm (30 in.) O.D. C.S.P. was shown below the approach fill at the south side of the underpass structure at approximate elevation 253.49 m (831.67 ft.) in Reference 3.

However, during the site reconnaissance on September 2, 2015, crushed rock was observed covering the front slopes of the abutments and the presence of the C.S.P. was not observed or confirmed.

Based on the Foundation Layout, Reinf. and Piers Drawing (Reference 4), No. 14 treated timber piles were to support the north and south abutments and two spread footings were to support the center piers (Pier Nos. 1 to 4). The top of the 1067 mm (3 ft. 6 in.) thick 11.9 x 4.9 m (39.0 x 16.0 ft.) spread footings was specified at elevation 254.05 m (833.50 ft.). Further, it was noted that in the drawing it was stated that the piles were driven to elevation 252.53 m (828.50 ft.) and 251.92 (826.50 ft.) at north and south abutment locations, respectively. In addition, it was noted in the drawing that the timber piles were to be treated with creosote to give a retention of 1.3 kN/m$^3$ (8 pcf).

The following table summarizes the pile data, pile cut-off elevation and top of pile cap elevation based on the Foundation Layout, Reinf. and Piers Drawing (Reference 4). The pile lengths shown in the following table were the lengths below cut-off.

| LOCATION | PILE TYPE | BATTER | QUANTITY | MAX LENGTH m (ft.) | PILE CUT-OFF ELEVATION m (ft.) | TOP OF PILE CAP ELEVATION m (ft.) |
|---|---|---|---|---|---|---|
| North Abutment | No. 14 Treated Timber Piles | 1:4 | 28 | 5.5 (18.0) | 257.71 (845.50) | 258.32 (847.50) |
| | | Vertical | 28 | | | |
| | | 1:8 | 21 | | | |
| South Abutment | | 1:4 | 28 | 6.7 (22.0) | 258.32 (847.50) | 258.93 (849.50) |
| | | Vertical | 28 | | | |
| | | 1:8 | 21 | | | |

## 6.2 Assessment of Foundation Parameters

Based on the previous investigation and subsurface conditions encountered, the following table summarizes the foundation design parameters that were recommended in References 1 and 4 for spread footing and in References 2A, 3 and 4 for piles and the updated geotechnical reaction at SLS and factored geotechnical resistance at ULS are provided.

**FOUNDATION DESIGN PARAMETERS**

| FOUNDATION LOCATION (TYPE) | PROBABLE PILE TIP/ SPREAD FOOTING ELEVATION[1] **m (ft.)** | PREVIOUS EQUIVALENT LIMIT STATE DESIGN VALUES[2] | | PREVIOUS EQUIVALENT LIMIT STATE DESIGN VALUES[3] | |
|---|---|---|---|---|---|
| | | SLS BEARING REACTION/LOAD | SLS BEARING REACTION/LOAD | SLS BEARING REACTION/LOAD | ULS FACTORED GEOTECHNICAL RESISTANCE/LOAD |
| North abutment (No. 14 Treated Timber Piles) | 252.2 to 253.4 (827.5 to 828.0) | 311 (kN) (35 Tons) per pile | 435 (kN) | 311 (kN) | 435 (kN) |
| Piers (Spread Footings) | 253.0 (830.0) | 287 kPa (3.0 tsf) | 430 (kPa) | 287 kPa | 430 (kPa) |
| South Abutment (No. 14 Treated Timber Piles) | 251.6 to 251.8 (825.5 to 826.2) | 311 (kN) (35 Tons) per pile | 435 (kN) | 311 (kN) | 435 (kN) |

**Notes**:
1. Probable pile tip elevations were estimated based on the information provided in Reference 4. The founding elevation for spread footing was based on Reference 4.
2. Working stress design values for spread footing was based on Reference 1 and for No. 14 Timber Piles was based on Reference 2A. The Serviceability Limit State design values are based on the working stress. No field verifications were made.
3. Resistance Factor = 0.5 for shallow foundation and 0.4 for deep foundations (CFEM 4th edition) Assumed Factor of Safety is 3 (CFEM 4th edition).

The Peak Ground Acceleration (PGA) for the site is 0.066 (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 the structure will undergo semi-integral conversion to eliminate the abutment expansion joints. In addition, it is understood that the barrier walls will be replaced at this structure location due to poor condition. Roadway protection will be required during rehabilitation and based on the existing available information no borehole investigation is anticipated.

From a geotechnical point of view, at the present time, foundation work for underpass foundation 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 will 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.

Groundwater control is not anticipated during construction since the excavations are anticipated to be limited to the approach embankments. Perched water, if encountered, can be controlled using sumps and pumps. It should be noted that groundwater levels are subject to seasonal fluctuations and precipitation patterns.

## 8. CLOSURE

This Technical Memorandum was prepared by Mr. Nazibur Rahman, P.Eng., Project Engineer and was reviewed by Mr. Brian R. Gray, MEng, P.Eng., Principal Consultant. Mr. Robert Ng, 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, MEng, P.Eng.
Principal Consultant


Robert Ng, MBA, PhD, P.Eng.
MTO Designated Principal Contact

NR/RN/BRG:nk

# 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 3090.101 | Foundation Frost Depth for Southern Ontario |

## Figure 1 – Key Plan


Imagery ©2016 Google, Map data ©2016 Google 500 m

# APPENDIX A

Appendix A – Previous Foundation Investigation Reports and Drawings

Reference 1. Foundation Investigation Report for Hwy. 4 Underpass Hwy 402, Twp. Of Westminster, Dist. 2, London, W.P. 41-66-08, Site 19-544, Contract No. 77-61, Soil Mechanics Section, Geotechnical Office, Ministry of Transportation and Communications, dated June 10, 1975, GEOCRES No. 40I14-94.

Reference 2. Pile Load Tests – Hwy. 402, Quality of Timber Piles, W.P. 41-66-08/16, Soil Mechanics Section, Engineering Materials Office, dated August 8, 1977.

Reference 2A. Pile Capacities – Contract 77-61, W.P. 41-66-08/06/07, Hwy. 402, District 2, London, Soil Mechanics Section, Engineering Materials Office, dated August 11, 1977.

Reference 2B. Driving Record of Test Piles Drawing, HWY 4 Interchange Underpass, DWG. 19-544-2-1. Sheet 165-1, Site No. 19-544, Cont. No. 77-61, WP No. 41-66-08, dated Aug. 15, 1977.

Reference 3. General Plan Drawing, Highway 4 Interchange Underpass, DWG 1-A, Sheet 164-A, Site No. 19-544, Dist. 2, Cont. No. 77-61, WP No. 41-66-08, dated August 1976.

Reference 4. Foundation Layout, Reinf & Piers Drawing, Highway 4 Interchange Underpass, DWG 3-A, Sheet 166-A, DWG 3-A, Site No. 19-544, Cont. No. 77-61, WP No. 41-66-08, dated August 1976.

DOCUMENT MICROFILMING IDENTIFICATION

Reference 1

GEOCRES No. 40I 14-94

DIST. 2 REGION

W.P. No. 41-66-08

CONT. No. 77-61

W. O. No.

STR. SITE No. 19-544

HWY. No. 402

LOCATION Hwy 4 Interchange Underpass

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

Ministry of Transportation and Communications

Ontario

# Memorandum

Mr. A.P.Watt, (2)
Regional Structural Planning Eng.
Southwestern Region, London

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview.

Attention:

Date: 10th June 1975 JUN 18 1975

Our File Ref.

In Reply to

Subject:

40I14-94
GEOCRES No.

CONTRACT No 77-6

FOUNDATION INVESTIGATION REPORT
for
Hwy. 4 Underpass Hwy 402
Twp. of Westminster, Dist.2,London
W.P. 41-66-08, Site 19-544

Attached we are forwarding to you our detailed Foundation Investigation Report on the subsoil conditions existing at the abovementioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

K. G. Selby

K.G.SELBY
Supervising Engineer

C.C. E.J.Orr
B.R.Davis
B.J.Giroux
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FOUNDATION INVESTIGATION REPORT

for

Hwy. 4 Underpass Hwy. 402
Twp. of Westminster, District 2, London
W.P. 41-66-08 Site 19-544

## 1. INTRODUCTION

A request for a foundation investigation at the above site was received from Mr.A.P.Watt, Regional Structural Planning Engineer, Southwestern Region, London.

A field investigation was subsequently carried out by the Soil Mechanics Section to determine the subsoil conditions existing at the site. This report contains the results of our field and laboratory investigations, together with our recommendations relating to the design of the proposed structure foundations.

## 2. DESCRIPTION OF THE SITE

The proposed underpass is located 1 mile south of the town of Lambeth in an area of mixed farming. The land is gently rolling and in the immediate vicinity of the underpass slopes predominantly to the East.

Physiographically, the site is situated in an area referred to as the Mount Elgin Ridges.

## 3. FIELD AND LABORATORY INVESTIGATION PROCEDURES

Field work consisted of six sampled boreholes advanced employing hollow stem augers, as well as, six dynamic cone penetration tests.

Disturbed samples were obtained using a 2-inch O.D. split spoon sampler driven according to the specifications for the Standard Penetration Test.

All boreholes were surveyed in the field by personnel from London Region, Engineering Surveys Section. The locations and elevations of the boreholes are shown on Drawing No. 416608-A which accompanies this report.

All samples were visually examined and classified at the site as well as in the laboratory. Following this inspection, laboratory tests were carried out on selected representative samples to determine the following physical properties:

Atterberg Limits
Natural Moisture Content
Grain-size Distribution
Undrained Shear Strength

The test results are summarized on the Record of Borehole Sheets contained in the appendix of this report.

## 4. SUBSOIL CONDITIONS

### 4.1 General

Subsoil at this site consists of a deep deposit of clayey silt in excess of 145 ft. in thickness. Contained within this deposit at depths between 25 and 65 ft. is a discontinuous deposit of fine sand and silt.

### 4.2 Clayey Silt

The upper portion of the clayey silt deposit with a thickness of about 15 ft. represents the dessicated crust. It is brown in colour and has a very stiff to hard consistency. Beneath this crust the clayey silt is grey and, between the depth of 15 ft. and 40 ft., has a stiff to very stiff consistency. At depths greater than 40 ft. the consistency is very stiff to hard with Standard Penetration 'N' values ranging from 25 to 90.

### 4.3. Sand and Silt

A layer of compact to very dense sand and silt was found to extend over a portion of the site. Its thickness varies from zero to greater than 30 ft. This sand and silt stratum was encountered in boreholes 2, 3 and 6 at depths ranging between 25 and 55 ft.

It was not, however, encountered in borehole 2 which was terminated at a depth of 75 ft. This deposit has a compact to very dense consistency with Standard Penetration Test 'N' values ranging from 26 to in excess of 200.

### 4.4 Groundwater

Groundwater was encountered in the sand and silt stratum and slowly rose in the boreholes to within 3 ft. of the ground surface or approximate elevation 832.

## 5. RECOMMENDATIONS

### 5.1 General

It is proposed to construct a structure to carry Hwy. 4 over Hwy. 402. This will involve the construction of embankments approximately 20 ft. in height and a bridge of two spans each of which will be 114 in length.

### 5.2 Center Pier

It is recommended that the center pier be supported on spread footings at approximate elev. 830. A net safe bearing pressure of 3 tons per sq. ft. may be used for design purposes. Resistance to sliding may be determined using an adhesion design value of 2000 p.s.f.

## 5.3 Perched Abutments

The abutments may be constructed within the approach fills supported on steel tubes piles (12$^{3}$/4 X"$^{1}$/4") driven into the dessicated crust. These piles should be driven to elevation 828 for the south abutment and 830 for the north abutment. The piles must not be driven below these elevations as undrained shear strength decreases with depth. A safe load of 25 tons per pile should be assumed for design purposes. Any horizontal loading should be resisted by battered piles.

As an alternative, the abutments may be supported on spread footings placed on well compacted G.B.C. Class 'A'. A net safe design load of 2.5 t.s.f. may be assumed. For calculations of sliding resistances, a friction coefficient of 0.6 may be assumed to apply between the footing and G.B.C. Class 'A'. A detailed construction scheme is outlined on Fig. 1 of the appendix.

## 5.4 Settlements

Long term settlements of approximately 2 inches at the abutments and 1 inch under the center pier is anticipated. Any bridge design should therefore have the ability to tolerate the resulting differential settlements.

To minimize these differential settlements between the abutments and the center pier footings, it is recommended that the approach embankments be built in advance of the structure by as long a period as possible.

### 5.5 Dewatering

No dewatering problems are anticipated during excavations for footings due to the relatively impervious nature of the subsoil.

### 5.6 Approach Embankments

No stability problems are anticipated with 20 ft. embankment fills if 2:1 slopes are employed. Care, however, should be taken that no material exceeding 3" grain size is placed in the fills at locations through which piles have to be driven.

### 5.7. Frost Protection

All pile caps or spread footings should be protected against frost action by a minimum 4 ft. of cover.

## 6. MISCELLANEOUS

The field work on the project was carried out May 1 to 7, 1975 under the supervision of Mr. P.J.Stuart, Project Foundation Engineer, who also prepared this report.

The equipment was owned and operated by Master Soil Investigation Limited.

This report was reviewed by Mr. K.G.Selby, Supervising Foundation Engineer.

P.J.STUART
Project Engineer

P. J. STUART

K.G.SELBY
Supervising Engineer.

# APPENDIX

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

# RECORD OF BOREHOLE Nº 1

W.P. 41-66-08 LOCATION Sta. 226+14 52' Rt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 5, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY [signature]

| SOIL PROFILE ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | SAMPLES NUMBER | TYPE | 'N' VALUES | GROUND WATER ELEV. | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000 | LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$ / WATER CONTENT % 10 20 30 | UNIT WEIGHT $\gamma$ | REMARKS % GR.SA.SI.CL. |
|---|---|---|---|---|---|---|---|---|---|---|
| 835.3 | Ground Level | | | | | | | | | |
| 0.0 | Clayey Silt | | | | | ∇ | | | | |
| | trace of sand | | 1 | SS | 16 | 830 | | | | |
| | Stiff to Hard | | 2 | SS | 47 | | 100/7" | | | |
| | | | 3 | SS | 24 | | | | | |
| | | | 4 | TW | PH | | | | 126 | 0 1 54 45 |
| | | | | | | 820 | | | | |
| | | | 5 | TW | PH | | | | | |
| | | | 6 | SS | 10 | | | | | |
| | | | | | | 810 | | | | |
| 803.8 | | | 7 | SS | 48 | | | | | |
| 31.5 | End of Borehole | | | | | | | | | |

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

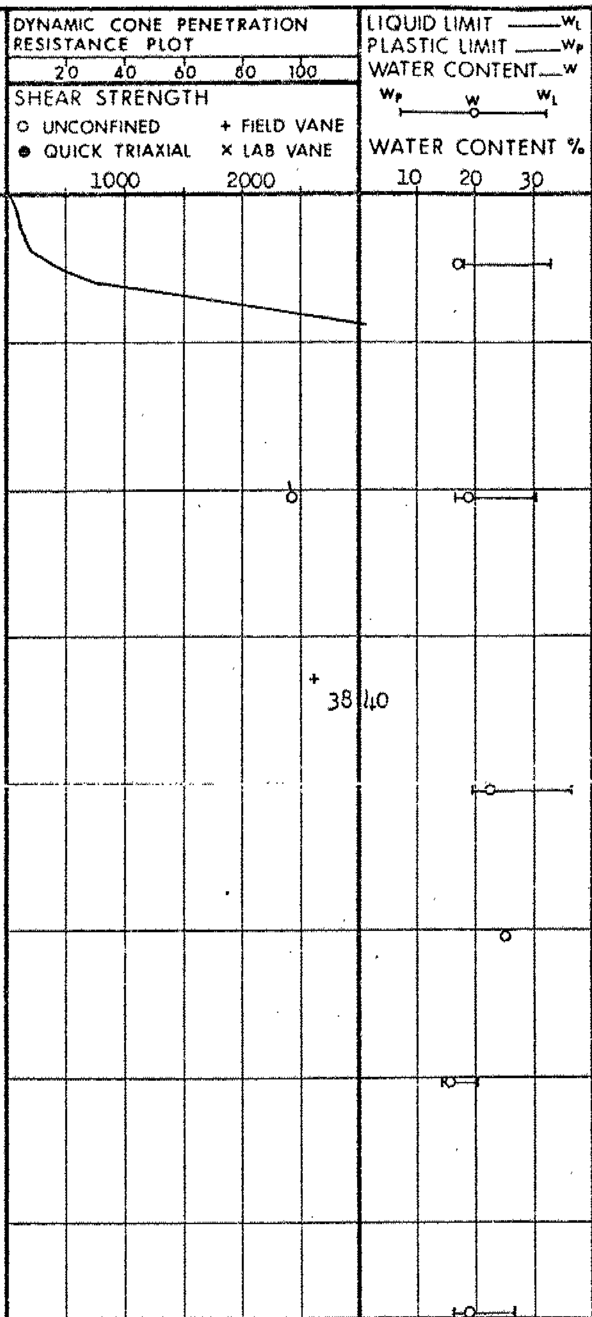
# RECORD OF BOREHOLE No 2

W.P. 41-66-08 LOCATION Sta. 226+08 30' Lt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 7, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

| ELEV. / DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | ELEV. | UNIT WEIGHT γ | REMARKS % GR. SA. SI. CL. |
|---|---|---|---|---|---|---|---|
| 840.1 | Ground Level | | | | | | |
| | Clayey Silt trace of sand Very Stiff to Hard | 1 | SS | 17 | | | |
| | | 2 | SS | 34 | | | |
| | | 3 | SS | 41 | 830 | | |
| | | 4 | SS | 34 | | | |
| | | 5 | SS | 27 | | | |
| | | 6 | TW | PH | 820 | 132 | 1 4 59 36 |
| | | 7 | SS | 24 | | | |
| | | 8 | SS | 23 | 810 | | |
| | | 9 | SS | 24 | 800 | | |
| | | 10 | SS | 26 | 790 | | |
| | | 11 | SS | 33 | 780 | | |
| | | 12 | SS | 32 | | | |
| | | | | | 770 | | |
| 763.6 / 76.5 | End of Borehole. Note - Water level not established | 13 | SS | 35 | | | |




20
15 ⊕ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

# RECORD OF BOREHOLE № 3

W.P. 41-66-08 LOCATION Sta. 227+27 55' Rt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 1st, 2nd, and 5th, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE 3¼" Hollow Stem Auger CHECKED BY

| ELEV. DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | REMARKS % GR.SA.SI.CL. |
|---|---|---|---|---|---|
| 836.0 | Ground Level | | | | |
| | Clayey Silt trace of sand stiff to hard | 1 | SS | 40 | |
| | | 2 | SS | 31 | |
| | | 3 | SS | 45 | 0 5 58 37 |
| | | 4 | SS | 27 | |
| | | 5 | SS | 18 | |
| | | 6 | SS | 14 | |
| 814.0 | | 7 | SS | 19 | |
| 22.0 | Silt and Sand Compact to Very Dense | 8 | SS | 31 | |
| | | 9 | SS | 52 | |
| | | 10 | SS | 64 | 0 48 (52) |
| | | 11 | SS | 26 | |
| 780.0 / 56.0 | Clayey Silt trace of sand very stiff to hard | 12 | SS | 27 | 0 1 74 25 |
| | | 13 | SS | 50 | |

GROUND WATER ELEV.: 830, 820, 810, 800, 790, 780, 770, 760, 750, 740

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT — $W_L$ PLASTIC LIMIT — $W_P$ WATER CONTENT — W

WATER CONTENT %: 10 20 30

UNIT WEIGHT γ

15 ◇ 5 (20 / 10) % STRAIN AT FAILURE

(Continued)

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ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

# RECORD OF BOREHOLE № 3 (Continued)

W.P. 41-66-08 LOCATION Sta. 227+27 55' Rt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 1st, 2nd, and 5th, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE 3¼" Hollow Stem CHECKED BY W.J.

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | LIQUID LIMIT —$W_L$ PLASTIC LIMIT —$W_P$ WATER CONTENT —W; $W_P$ W $W_L$; WATER CONTENT % 10 20 30 | UNIT WEIGHT γ | REMARKS % GR.SA.SI.CL. |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | 'N' VALUES | ELEV. | | | | |
| 732.0 | Continued | | | | | | | | | |
| 104.0 | Clayey Silt trace of sand Very Stiff to Hard | | | | | 730 | | | | |
| | | | 14 | SS | 79 | | | | | |
| | | | | | | 720 | | | | |
| | | | | | | 710 | | | | |
| | | | 15 | SS | 55 | | | | | |
| | | | | | | 700 | | | | |
| 689.5 | | | 16 | SS | 89 | 690 | | | | |
| 146.5 | End of Borehole | | | | | | | | | |

20
15 ✦ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

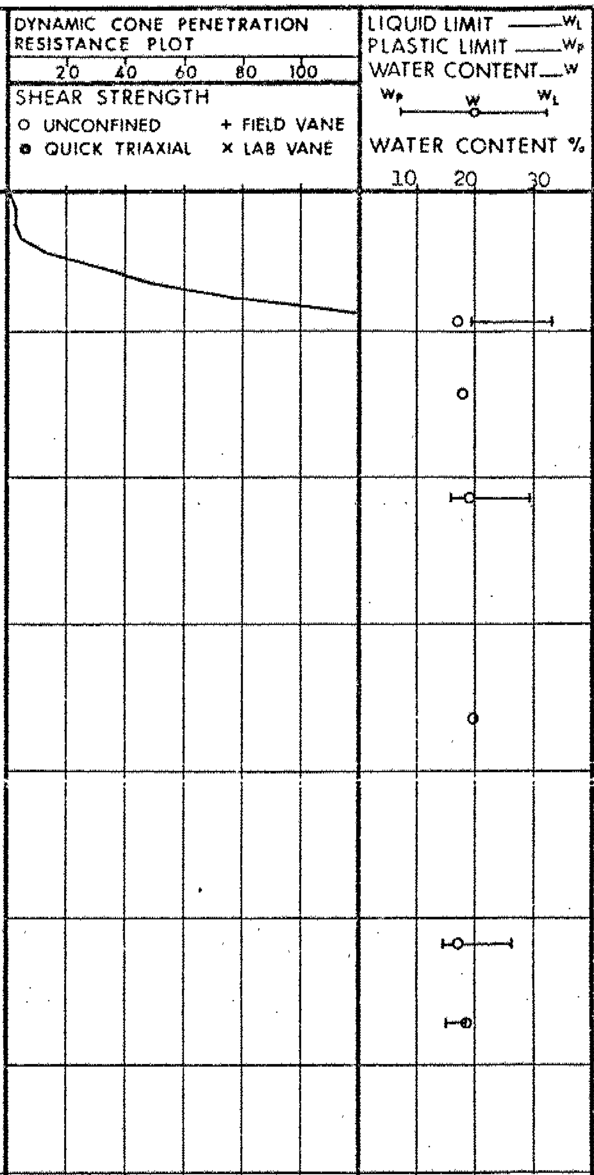
# RECORD OF BOREHOLE № 4

W.P. 41-66-08 LOCATION Sta. 227+25 28' Lt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 7, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

| SOIL PROFILE | | SAMPLES | | | GROUND WATER | REMARKS |
|---|---|---|---|---|---|---|
| ELEV. DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | ELEV. | % GR.SA.SI.CL. |
| 839.2 | Ground Level | | | | | |
| 0.0 | Clayey Silt | 1 | SS | 36 | | |
| | | 2 | SS | 42 | 830 | |
| | Trace of Sand | 3 | SS | 42 | | |
| | | 4 | SS | 23 | | |
| | Very Stiff to Hard | 5 | SS | 17 | 820 | |
| | | 6 | SS | 16 | | 2 4 63 31 |
| | | 7 | SS | 18 | 810 | |
| | | 8 | SS | 23 | 800 | |
| | | 9 | SS | 33 | 790 | |
| | | 10 | SS | 41 | | |
| 784.2 / 55.0 | Silt Very Dense | 11 | SS | 100 | 780 | |
| | | 12 | SS | 171 | | |
| 772.7 | | 13 | SS | 221 | | 0 1 89 10 |
| 66.5 | End of Borehole. Note - Water level not established | | | | | |

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15 ◇ 5 % STRAIN AT FAILURE
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MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

# RECORD OF BOREHOLE № 5

W.P. 41-66-08 LOCATION Sta. 228+48 55' Rt. ORIGINATED BY PJS

DIST. 2 HWY. 402 BORING DATE May 5, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY [illegible]

| SOIL PROFILE | | SAMPLES | | | GROUND WATER | REMARKS |
|---|---|---|---|---|---|---|
| ELEV. DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | ELEV. | % GR.SA.SI.CL. |
| 836.8 | Ground Level | | | | | |
| 0.0 | Clayey silt trace of sand Stiff to Hard | | | | ▽ | |
| | | 1 | SS | 39 | | |
| | | 2 | SS | 63 | 830 | |
| | | 3 | SS | 39 | | |
| | | 4 | SS | 34 | | 3 6 66 25 |
| | | 5 | SS | 29 | | |
| | | | | | 820 | |
| | | 6 | SS | 19 | | |
| | | 7 | SS | 17 | 810 | |
| | | 8 | SS | 20 | | |
| | | 9 | SS | 20 | 800 | |
| 795.3 | | 10 | SS | 77 | | 0 1 88 11 |
| 41.5 | End of Borehole | | | | | |

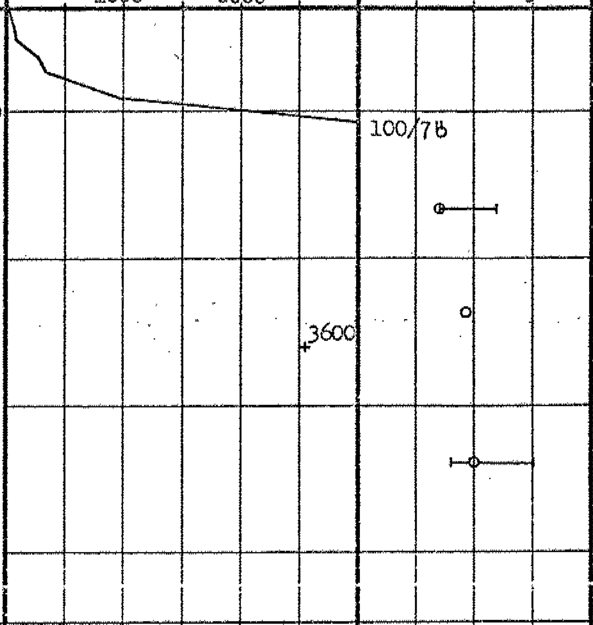
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 1000 2000

LIQUID LIMIT — $W_L$ PLASTIC LIMIT — $W_P$ WATER CONTENT — W

WATER CONTENT %: 10 20 30

UNIT WEIGHT γ




20 15 ⊕ 5 % STRAIN AT FAILURE 10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

# RECORD OF BOREHOLE Nº 6

W.P. 41-66-08 LOCATION Sta. 228+35 34' Lt. ORIGINATED BY PJS

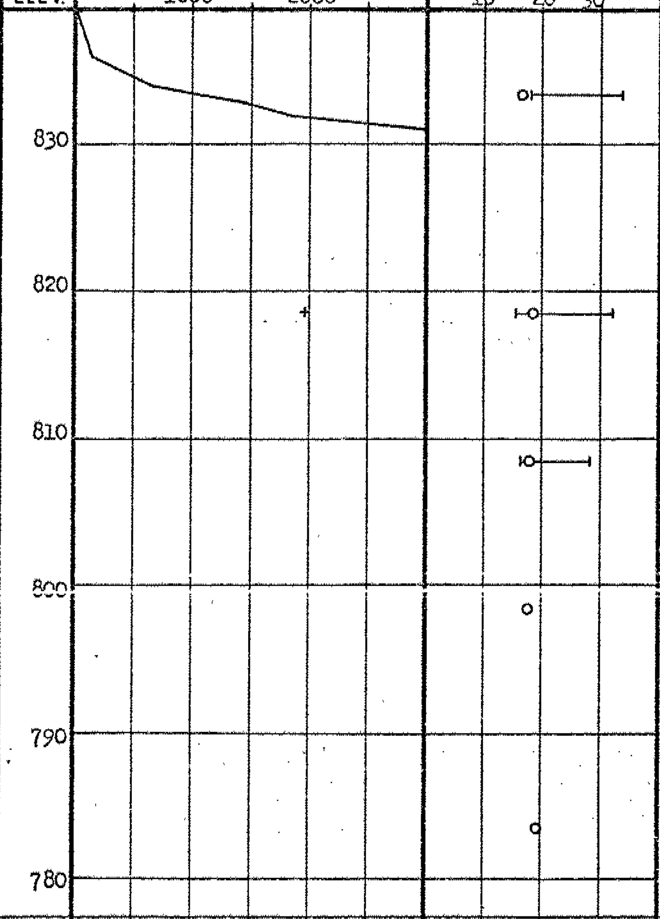
DIST. 2 HWY. 402 BORING DATE May 6, 1975 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE 3¼" Hollow Stem Auger CHECKED BY W.J.

| ELEV. / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | REMARKS % GR. SA. SI. CL. |
|---|---|---|---|---|---|
| 839.0 | Ground Level | | | | |
| 0.0 | Clayey Silt, trace of sand, Very Stiff to Hard | 1 | SS | 57 | |
| | | 2 | SS | 45 | |
| | | 3 | SS | 21 | |
| | | 4 | TW | PH | 1 4 56 39 |
| | | 5 | SS | 17 | |
| | | 6 | SS | 20 | |
| 798.0 / 41.0 | Fine Sand and Silt, Dense to Very Dense | 7 | SS | 132 | 0 47 51 2 |
| | | 8 | SS | 41 | |
| 781.0 / 58.0 | Clayey Silt, trace of sand | 9 | SS | 205 | 0 1 97 2 |
| 777.5 / 61.5 | End of Borehole | 10 | SS | 35 | |

Note: Water level not established.

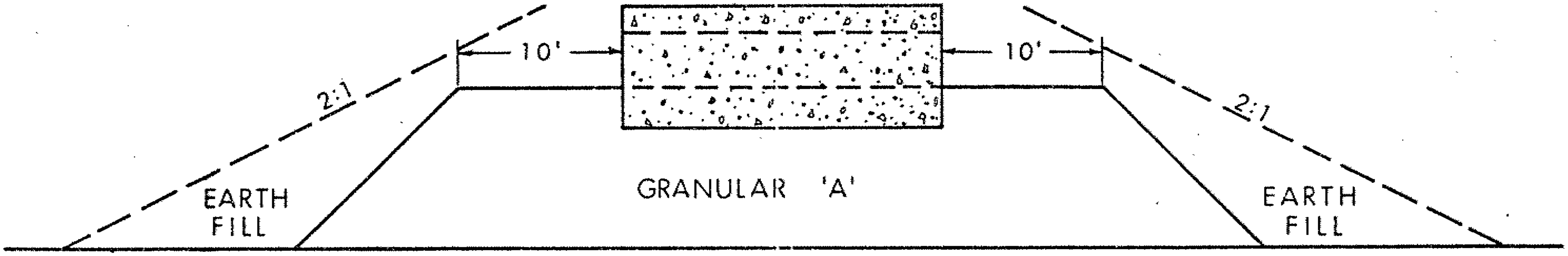
Column headings: SOIL PROFILE; STRAT. PLOT; SAMPLES; GROUND WATER ELEV.; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000; LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$; WATER CONTENT % 10 20 30; UNIT WEIGHT γ




20
15 ◇ 5 % STRAIN AT FAILURE
10

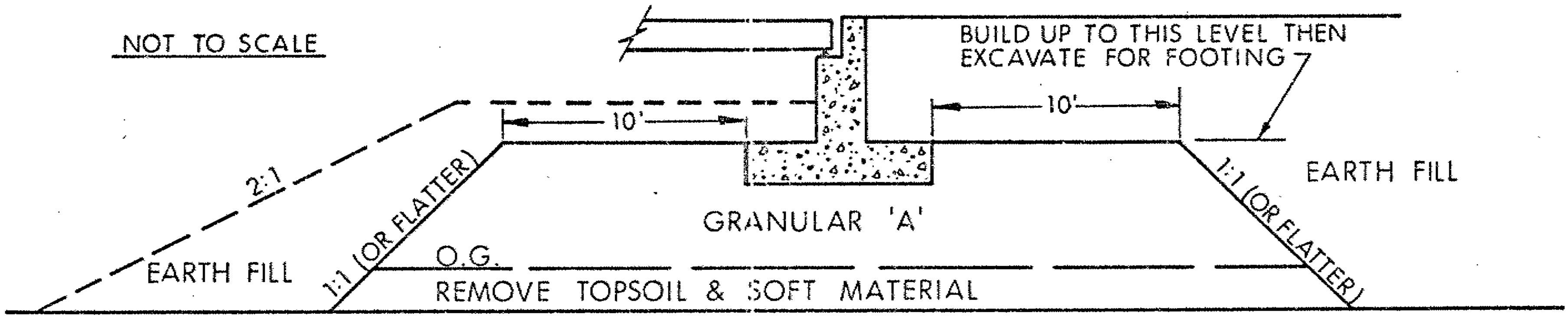
OFFICE REPORT ON SOIL EXPLORATION

# 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'.

2 - PLACE GRANULAR 'A' TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.

3 - EXCAVATE COMPACTED GRANULAR 'A' MATERIAL FOR FOOTING.

FIG. 1

W.P. 41-66-08

# ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

## PENETRATION RESISTANCE

'N'= STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

## DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

| CONSISTENCY | c LB./SQ. FT. |
|---|---|
| VERY SOFT | 0 - 250 |
| SOFT | 250 - 500 |
| FIRM | 500 - 1000 |
| STIFF | 1000 - 2000 |
| VERY STIFF | 2000 - 4000 |
| HARD | > 4000 |

| DENSENESS | 'N' BLOWS / FT. |
|---|---|
| VERY LOOSE | 0 - 4 |
| LOOSE | 4 - 10 |
| COMPACT | 10 - 30 |
| DENSE | 30 - 50 |
| VERY DENSE | > 50 |

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

## TYPE OF SAMPLE

| | |
|---|---|
| S.S. | SPLIT SPOON |
| W.S. | WASHED SAMPLE |
| S.T. | SLOTTED TUBE SAMPLE |
| A.S. | AUGER SAMPLE |
| C.S. | CHUNK SAMPLE |
| T.W. | THINWALL OPEN |
| T.P. | THINWALL PISTON |
| O.S. | OESTERBERG SAMPLE |
| F.S. | FOIL SAMPLE |
| R.C. | ROCK CORE |

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

## SOIL TESTS

| | |
|---|---|
| U | UNCONFINED COMPRESSION |
| UU | UNCONSOLIDATED UNDRAINED TRIAXIAL |
| CIU | CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL |
| CID | " " DRAINED " |
| CAU | " ANISOTROPIC UNDRAINED " |
| CAD | " " DRAINED " |
| L.V. | LABORATORY VANE |
| F.V. | FIELD VANE |
| C | CONSOLIDATION |
| S | SENSITIVITY |

# ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

## SOIL PROPERTIES

| | |
|---|---|
| $\gamma$ | UNIT WEIGHT OF SOIL (BULK DENSITY) |
| $\gamma_s$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\gamma_w$ | UNIT WEIGHT OF WATER |
| $\gamma_d$ | UNIT DRY WEIGHT OF SOIL (DRY DENSITY) |
| $\gamma'$ | UNIT WEIGHT OF SUBMERGED SOIL |
| G | SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$ |
| e | VOID RATIO |
| n | POROSITY |
| w | WATER CONTENT |
| $S_r$ | DEGREE OF SATURATION |
| $w_L$ | LIQUID LIMIT |
| $w_P$ | PLASTIC LIMIT |
| $I_P$ | PLASTICITY INDEX |
| $w_S$ | SHRINKAGE LIMIT |
| $I_L$ | LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$ |
| $I_C$ | CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$ |
| $e_{max}$ | VOID RATIO IN LOOSEST STATE |
| $e_{min}$ | VOID RATIO IN DENSEST STATE |
| $I_D$ | DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| | RELATIVE DENSITY $D_r$ IS ALSO USED |
| h | HYDRAULIC HEAD OR POTENTIAL |
| q | RATE OF DISCHARGE |
| v | VELOCITY OF FLOW |
| i | HYDRAULIC GRADIENT |
| k | COEFFICIENT OF PERMEABILITY |
| j | SEEPAGE FORCE PER UNIT VOLUME |
| $m_v$ | COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$ |
| $c_v$ | COEFFICIENT OF CONSOLIDATION |
| $C_c$ | COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$ |
| $T_v$ | TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH) |
| U | DEGREE OF CONSOLIDATION |
| $\tau_f$ | SHEAR STRENGTH |
| $c'$ | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION |
| | IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$ |
| $c_u$ | APPARENT COHESION |
| $\phi_u$ | APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION |
| | IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$ |
| $\mu$ | COEFFICIENT OF FRICTION |
| $S_t$ | SENSITIVITY |

## GENERAL

| | |
|---|---|
| $\pi$ | = 3·1416 |
| e | BASE OF NATURAL LOGARITHMS 2·7183 |
| $\log_e a$ OR $\ln a$ | NATURAL LOGARITHM OF $a$ |
| $\log_{10} a$ OR $\log a$ | LOGARITHM OF $a$ TO BASE 10 |
| t | TIME |
| g | ACCELERATION DUE TO GRAVITY |
| V | VOLUME |
| W | WEIGHT |
| M | MOMENT |
| F | FACTOR OF SAFETY |

## STRESS AND STRAIN

| | |
|---|---|
| u | PORE PRESSURE |
| $\sigma$ | NORMAL STRESS |
| $\sigma'$ | NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED) |
| $\tau$ | SHEAR STRESS |
| $\varepsilon$ | LINEAR STRAIN |
| $\gamma$ | SHEAR STRAIN |
| $\nu$ | POISSON'S RATIO ($\mu$ IS ALSO USED) |
| E | MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS) |
| G | MODULUS OF SHEAR DEFORMATION |
| K | MODULUS OF COMPRESSIBILITY |
| $\eta$ | COEFFICIENT OF VISCOSITY |

## EARTH PRESSURE

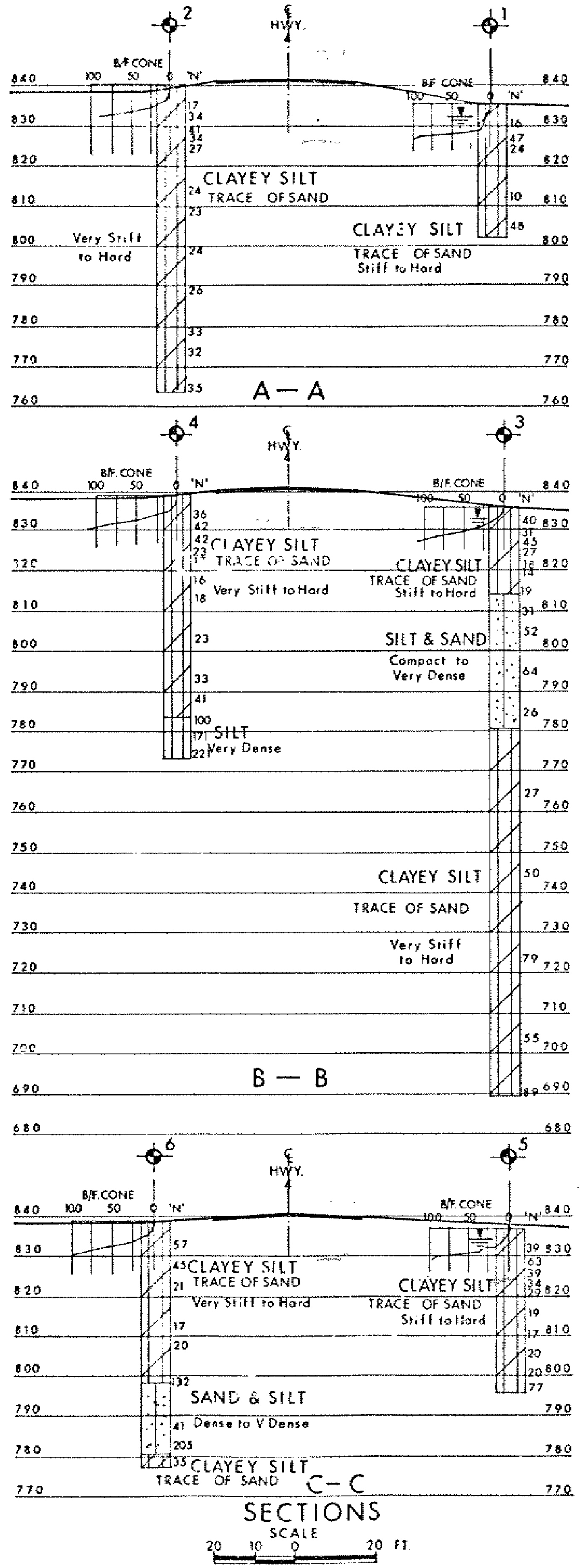
| | |
|---|---|
| d | DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE |
| $\delta$ | ANGLE OF WALL FRICTION |
| K | DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS |
| $K_o$ | COEFFICIENT OF EARTH PRESSURE AT REST |

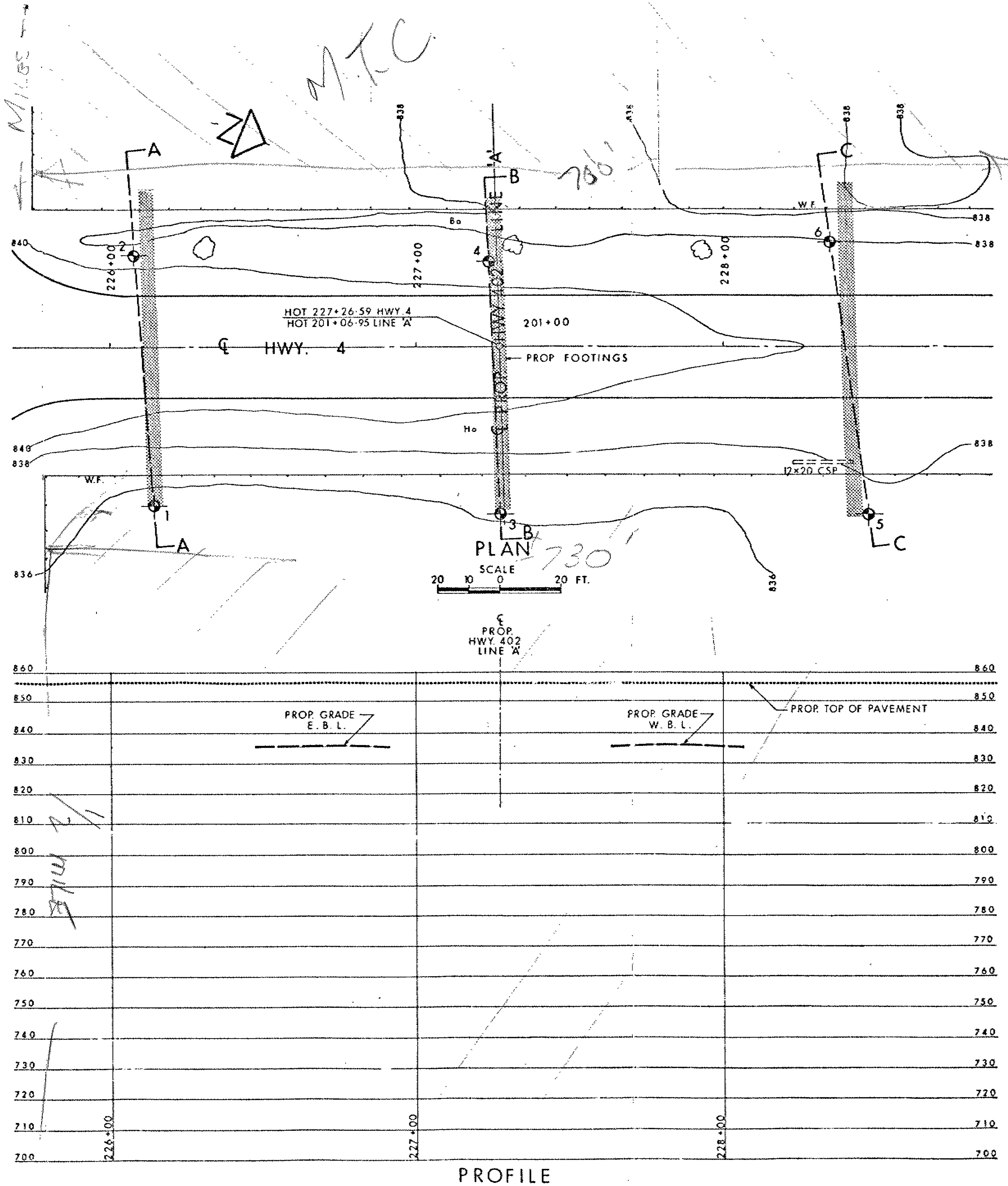
## FOUNDATIONS

| | |
|---|---|
| B | BREADTH OF FOUNDATION |
| L | LENGTH OF FOUNDATION |
| D | DEPTH OF FOUNDATION BENEATH GROUND |
| N | DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY |
| $k_s$ | MODULUS OF SUBGRADE REACTION |

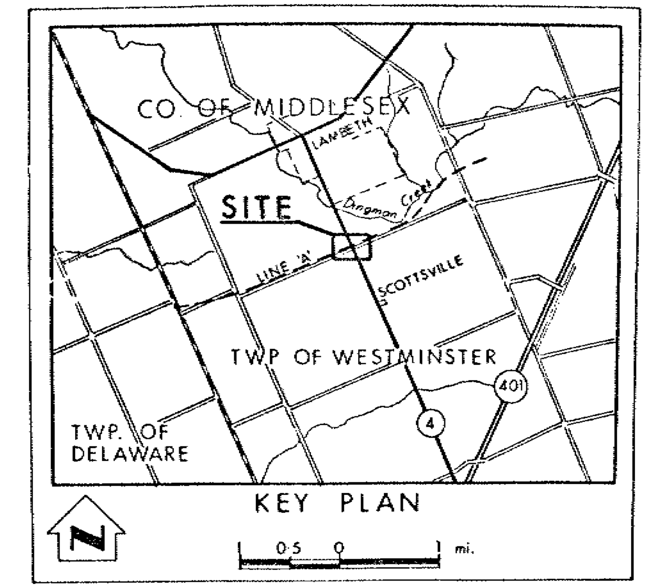
## SLOPES

| | |
|---|---|
| H | VERTICAL HEIGHT OF SLOPE |
| D | DEPTH BELOW TOE OF SLOPE TO HARD STRATUM |
| $\beta$ | ANGLE OF SLOPE TO HORIZONTAL |




REF. NO. E-5373-1, JUNE '74




## LEGEND

- Bore Hole
- Dynamic Cone Penetration Resistance Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. 7 May 1975
  W.L. in B.H. 2, 4, & 6 not established

| NORTH | NO. | ELEVATION | STATIONS | OFFSET | EAST |
|---|---|---|---|---|---|
| 15,586,401 | 1 | 835·3 | 226+14 | 52'RT. | 1,323,413 |
| 15,586,372 | 2 | 840·1 | 226+08 | 30'LT. | 1,323,562 |
| 15,586,513 | 3 | 836·0 | 227+27 | 55'RT. | 1,323,4[illegible] |
| 15,586,483 | 4 | 839·2 | 227+25 | 28'LT. | 1,323,53[illegible] |
| 15,586,627 | 5 | 836·8 | 228+48 | 55'RT. | 1,323,572 |
| 15,586,58[illegible] | 6 | 839·0 | 228+35 | 34'LT. | 1,323,49[illegible] |

NOTE: FOR CONTRACT DOCUMENTS
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the LONDON District Office.

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

| REVISIONS | | |
|---|---|---|
| DATE | BY | DESCRIPTION |

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH—GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

# HWY. NO. 4

HIGHWAY NO PROP. HWY. 402 LINE 'A' DIST NO 2
CO MIDDLESEX
TWP WESTMINSTER LOT 66 CON E&W NBTR

## BORE HOLE LOCATIONS & SOIL STRATA

| | | | |
|---|---|---|---|
| SUBM'D P. S. | CHECKED | W.P. NO 41-66-08 | DRAWING NO 416608-A |
| DRAWN OL J | CHECKED | W.O. NO | |
| DATE 11 JUNE 1975 | | SITE NO 19-544 | BRIDGE DRAWING NO |
| APPROVED | | CONT NO | |

Soil Mechanics Section
Engineering Materials Office
West Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Tel: (416) 248-3282

August 8, 1977

Mr. L. Crane
Western Caissons Ltd.
150 Creditstone Road
Maple, Ontario

SUBJECT: Pile Load Tests - Hwy. 402, Quality of Timber Piles
W.P. 41-66-08/16

Dear Sir

As you are aware the six timber piles initially supplied by your firm for the above mentioned project turned out to be defective in that they split or shattered during the first one or two blows of the pile driving hammer even though the delivered energy was less than 8000 foot/lbs. per blow, in all cases. During an inspection of the broken piles, your Superintendant, Mr. D. Crompton and myself were of the opinion that the piles as supplied were not structurally intact and that excessive drying out prior to the injection of creosote may have caused actual separation between growth rings. Since this type of defect cannot be detected by the superficial inspection, which normally occurs in the field we are concerned that the organization supplying the piles (in this case Domtar) was not able to detect it either and that their quality control methods do not proved for a check on intactness and structural integrity.

We would appreciate it therefore if you could obtain from Domtar a full report as to what caused the defects in the piles and also what quality control procedures would be required to identify these defects.

Yours truly

K.G. Selby, P. Eng
Supervising Engineer

KGS/kr

cc: Files

Reference 2A

Mr. K.C. Bassi
Head, Eastern Section
Structural Office
West Building

Soil Mechanics Section
Engineering Materials Office
West Building

77 08 11

Pile Capacities - Contract 77-61
W.P. 41-66-08/06/07, Hwy. 402
District 2, London

As a result of our recent load tests at London we are revising our recommendations for pile capacities and required lengths as follows on the above mentioned work projects:

(1) W.P. 41-66-08 : Safe Capacity 35T/pile
Length Required
N. Abutment - 18 feet
S. Abutment - 22 feet
Note to Read: Drive to elevation 828.5 (N.A.)
826.5 (S.A.)

(2) W.P. 41-66-07 : Safe Capacity 40T/pile
Length Required
Increase lengths for battered piles:
(N. Abutment - 40 feet
(Pier - 25 feet
(S. Abutment - 40 feet
Note to Read: Drive to elevation 806 (N.A.)
803 (Pier)
805 (S.A.)

(3) W.P. 41-66-06 : Safe Capacity 40T/pile
Length Required
Increase lengths for battered piles:
(N. Abutment - 30 feet
(Pier - 25 feet
(S. Abutment - 30 feet
Note to Read: Drive to elevation 827 (N.A.)
816 (Pier)
828 (S.A.)

If the above recommendations are followed the savings should be:

| | |
|---|---|
| W.P. 41-66-08 | $ 8,000.00 |
| W.P. 41-66-07 | 20,000.00 |
| W.P. 41-66-06 | 14,000.00 |
| Total | $42,000.00 |

... 2

We understand that there is adequate time to issue an appropriate addendum to the contract which is presently being advertised.

K.G. Selby
Supervising Engineer

KGS/kr

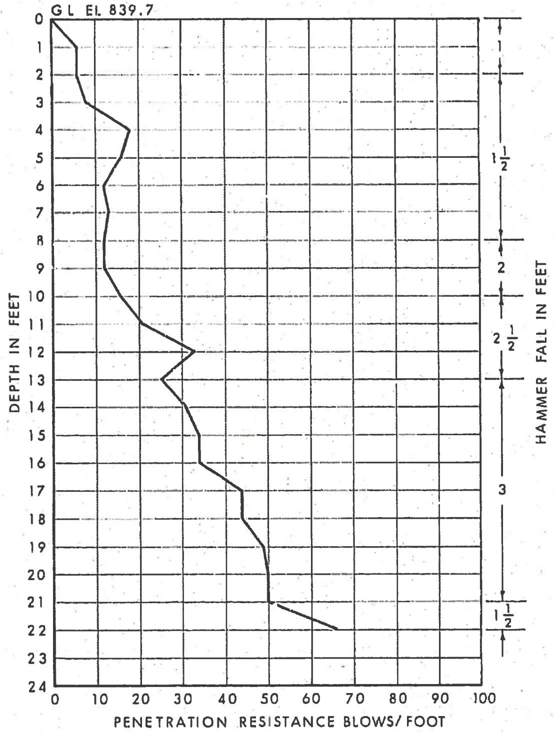
cc: J. Davidson
B. Giroux
J. Keen
Files ✓

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO
CONT No 77-61
WP No 41-66-08

# HWY 4 INTERCHANGE
UNDERPASS
DRIVING RECORD OF TEST PILES

SHEET 165-1

## DRIVING RECORD PILE No 1




PILE No 1

Total continuous driving time 0' - 22', 22 minutes

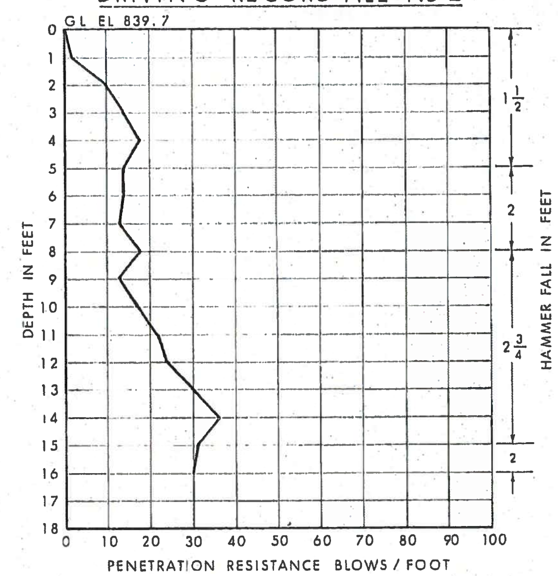
Drop Hammer

Hammer Weight: 6 100 lbs

Height of Fall: As shown

Date Driven: July 14, 1977

## DRIVING RECORD PILE No 2




PILE No 2

Total continuous driving time 0'- 16', 7 minutes

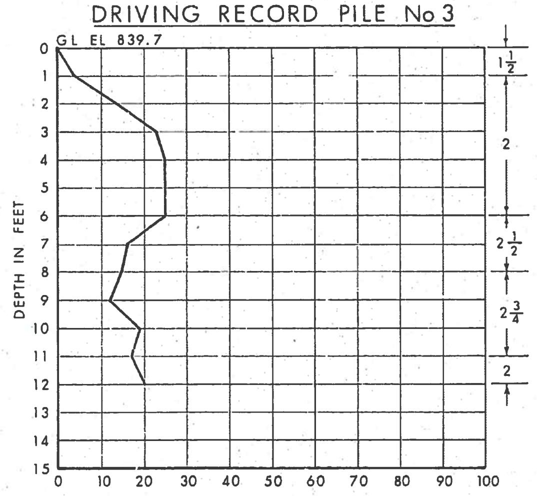
Drop Hammer

Hammer Weight: 6 100 lbs

Height of Fall: As shown

Date Driven: July 14, 1977

## DRIVING RECORD PILE No 3




PILE No 3

Total continuous driving time 0'- 12', 7 min 30 sec

Drop Hammer

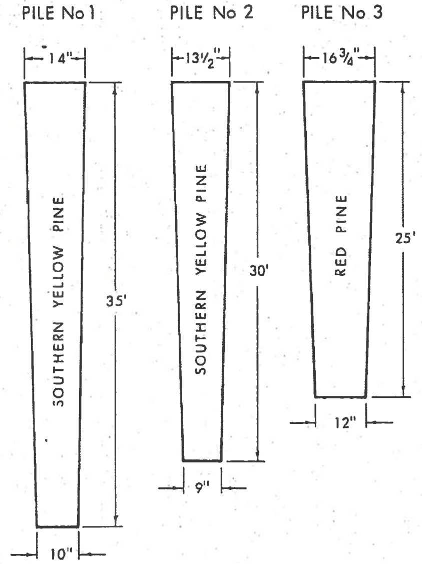
Hammer Weight: 6 100 lbs

Height of Fall: As shown

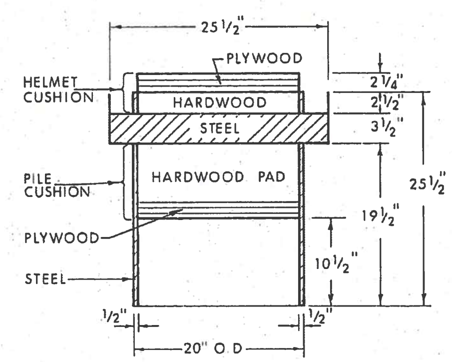
Date Driven: July 14, 1977

## PILE DETAILS AS DRIVEN

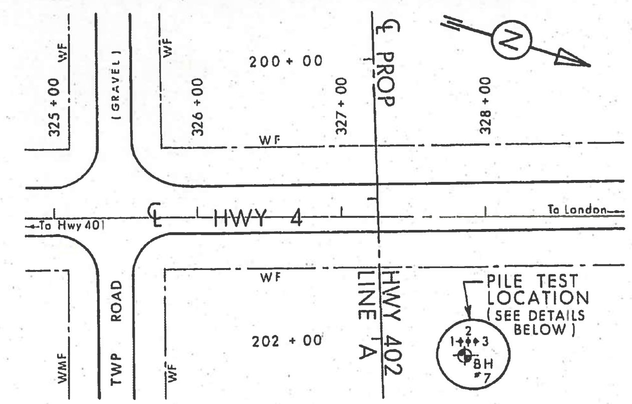
(All Piles No 14 Treated Timber)




## HELMET DETAILS



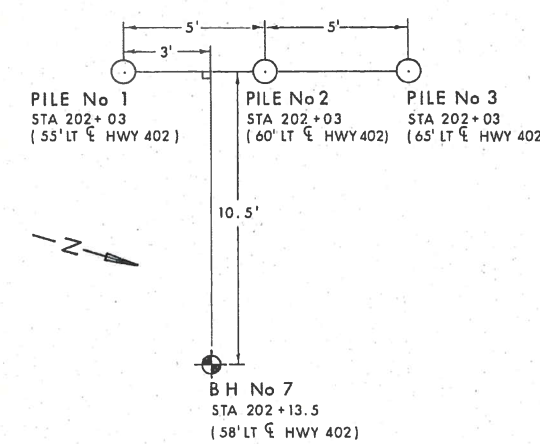

Total Weight of Helmet = 850 lbs

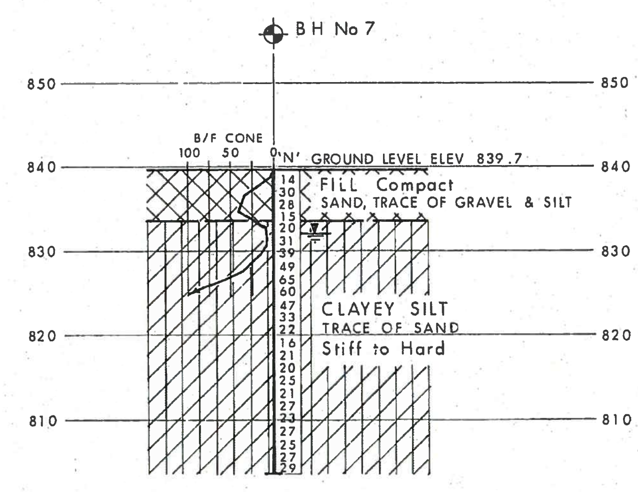



PLAN

SCALE 60 30 0 60 FT

## PILE TEST LOCATION




BORE HOLE No 7 STRATIGRAPHY

SCALE 10 5 0 10 FT

KEY PLAN

SEE DWG 19-544-2

### LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- 'N' Blows/ft (Std Pen Test 350 ft lbs energy)
- CONE Blows/ft (60° Cone, 350 ft lbs energy)
- WL at time of investigation Aug 1977
- Pile

| No | ELEVATION | STATION | OFFSET ℄ HWY 402 |
|---|---|---|---|
| BH 7 | 839.7 | 202 + 13.5 | 58' LT |
| | PILES | | |
| 1 | 839.7 | 202 + 03 | 55' LT |
| 2 | 839.7 | 202 + 03 | 60' LT |
| 3 | 839.7 | 202 + 03 | 65' LT |

-NOTE-

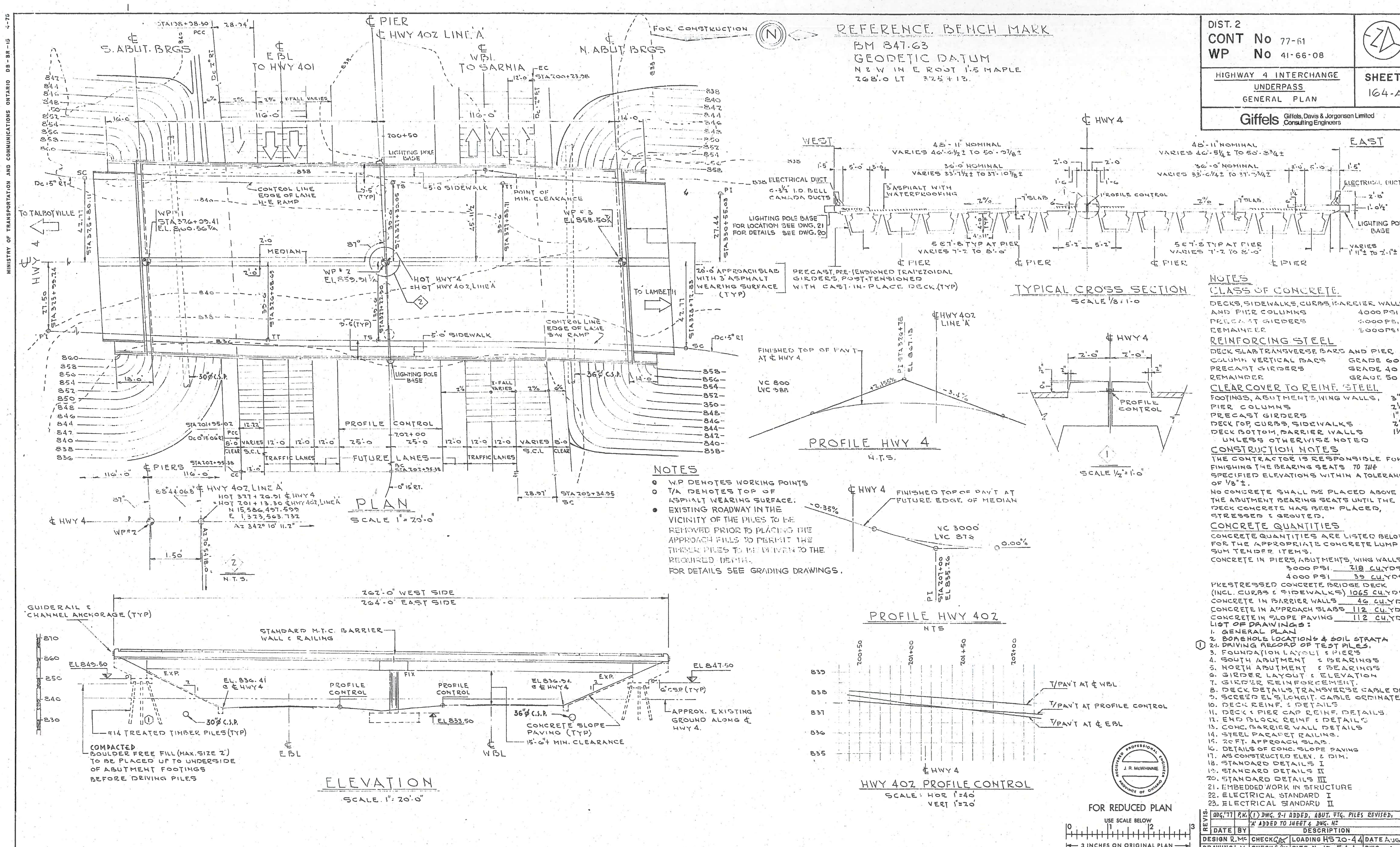
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

| REVISIONS | | |
|---|---|---|
| DATE | BY | DESCRIPTION |

| | | | |
|---|---|---|---|
| HWY No 402 | | | DIST 2 |
| SUBM'D P P | CHECKED | DATE Aug 15, 1977 | SITE 19 - 544 |
| DRAWN R S | CHECKED | APPROVED | DWG 19 - 544 - 2 - 1 |

TWP. 94-544-2-1

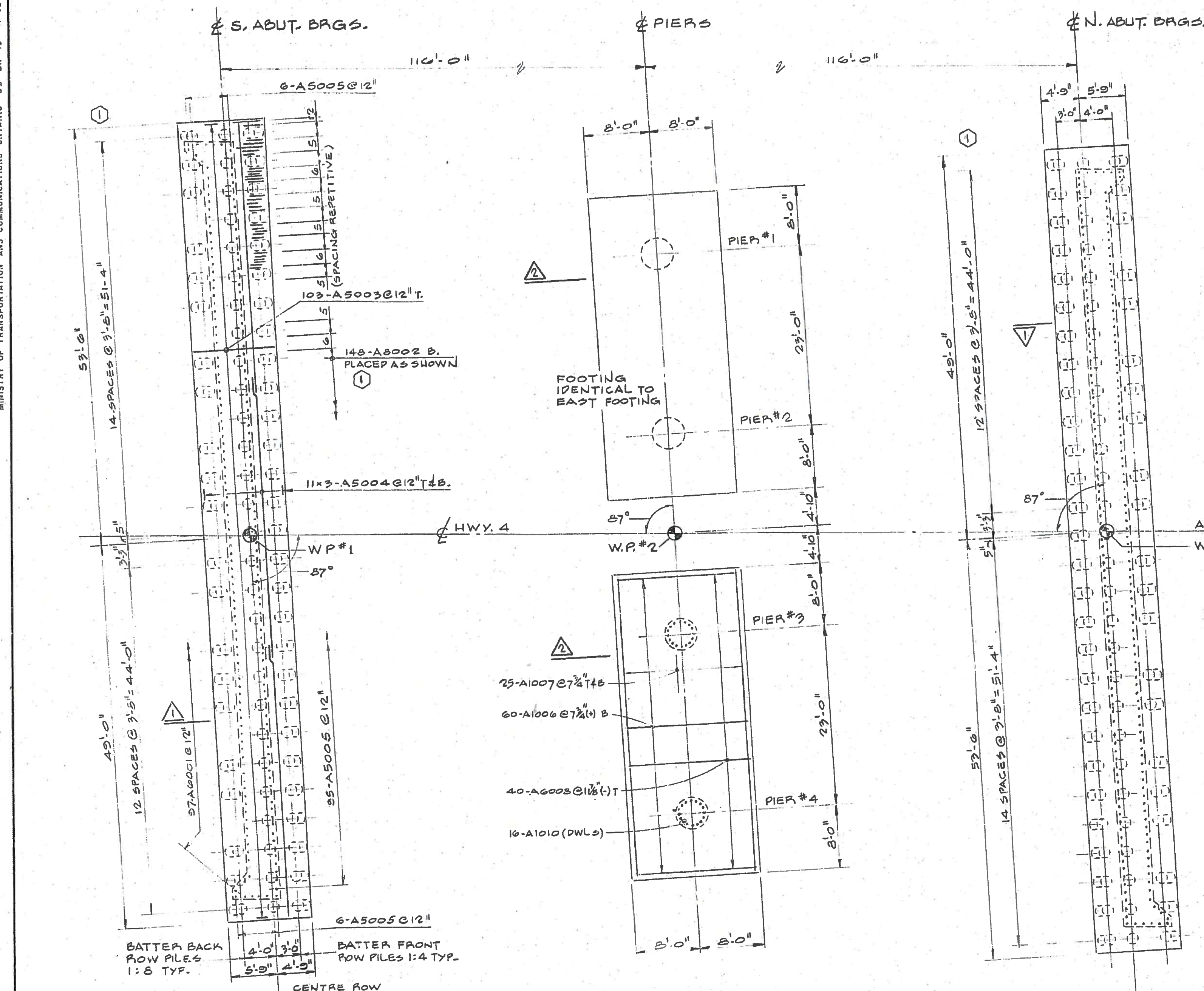
Reference 3




TWP. 94-544-1-A

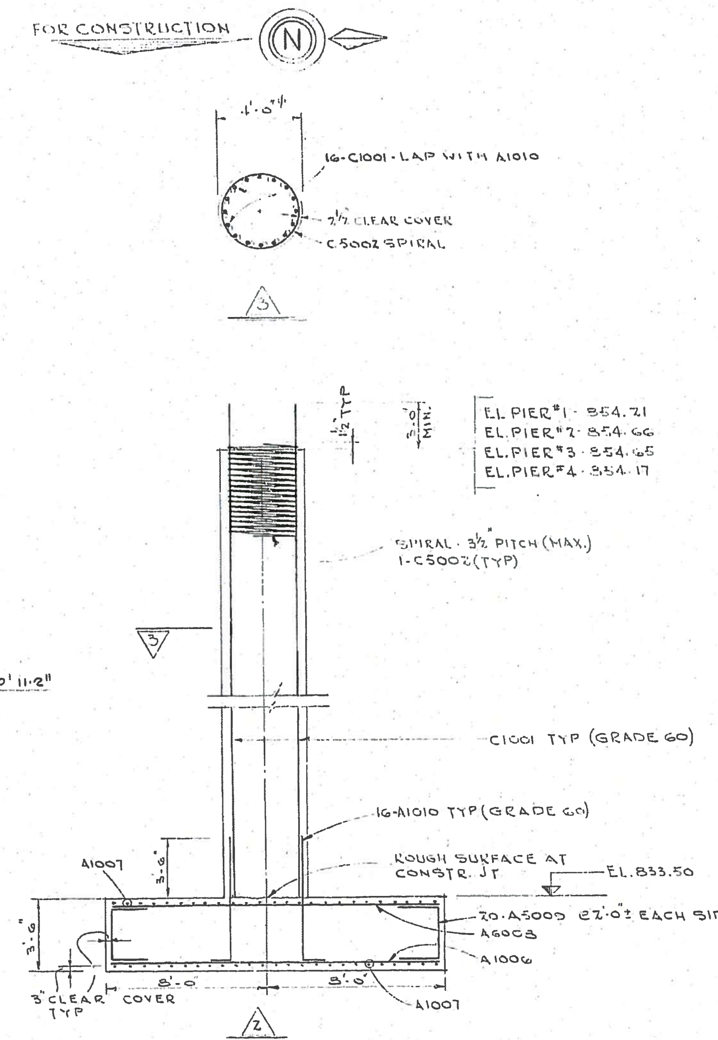
Reference 4

| | |
|---|---|
| CONT No 77-61<br>WP No 41-66-08 | |
| HIGHWAY 4 INTERCHANGE<br>UNDERPASS<br>FOUNDATION LAYOUT, REINF & PIERS | SHEET<br>16-A ⟨1⟩ |
| Giffels — Giffels, Davis & Jorgensen Limited, Consulting Engineers | |



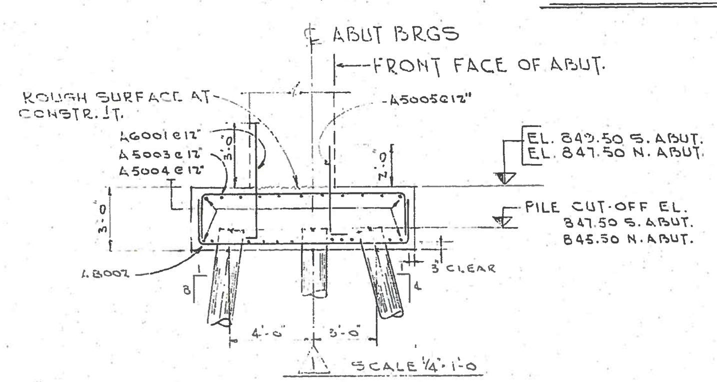

## FOUNDATION LAYOUT & REINF.

SCALE 1/8" = 1'-0"




## PIERS

SCALE 1/4" = 1'-0"




NOTE
T. DENOTES TOP
B. DENOTES BOTTOM

## WORKING POINT DATA

| WP# | STATION | | CO-ORDINATES |
|---|---|---|---|
| 1 | 326+09.41 | HWY 4 | N15,586,385.743<br>E 1,323,599.710 |
| 2 | 327+25.41 | HWY 4 | N15,586,496.171<br>E 1,323,564.192 |
| 3 | 328+41.41 | HWY 4 | N15,586,606.599<br>E 1,323,528.673 |

## PILES

| LOCATION | QTY. | TYPE | BATTER | MAX. LENGTH |
|---|---|---|---|---|
| N. ABUT. | 28 | # 14 TREATED TIMBER PILES | 1:4 | 18'-0" |
| | 28 | | VERT | |
| | 21 | | 1:8 | |
| S. ABUT. | 28 | | 1:4 | 22'-0" |
| | 28 | | VERT | |
| | 21 | | 1:8 | |

⟨1⟩

## NOTES

1. PILE SPACINGS TO BE MEASURED AT UNDERSIDE OF FOOTINGS
2. PILES SHALL BE DRIVEN TO THE FOLLOWING ELEVATIONS :
   ⟨1⟩ NORTH ABUT. EL. 828.5
   SOUTH ABUT. EL. 826.5
3. SOUTH ABUT. FOOTING REINF SAME AS NORTH ABUT. FOOTING.
4. TIMBER PILES SHALL BE TREATED WITH CREOSOTE TO GIVE A RETENTION OF 8PCF

J R McWHINNIE

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

| REVISIONS DATE | BY | DESCRIPTION |
|---|---|---|
| AUG.'77 | P.K. | ⟨1⟩ CAPACITY, LENGTH, No. & SPACING OF TIMBER PILES REV. IN ABUT. FTGS & PLACING OF REINF. STEEL ADJUSTED TO SUIT, 'A' ADDED TO SHEET & DWG. No. |

| | | | |
|---|---|---|---|
| DESIGN R.Mc | CHECK CW | LOADING HS20-44 | DATE AUG/76 |
| DRAWING AM | CHECK RW | SITE No 19-544 | DWG 3-A ⟨1⟩ |

TWP. 94-544-3-A

# APPENDIX B

Site Photographs

**Photograph 1**: Looking at the north abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the abutment wall. Rock protection is provided on the front slope of the abutment. Effect of erosion on the slope face was not observed on the slope face (September 2, 2015).

**Photograph 2:** Looking at the west wingwall and the adjacent slope of north abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (September 2, 2015).

**Photograph 3**: Looking at east wingwall and the adjacent slope of north abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (September 2, 2015).

**Photograph 4**: Looking at the south abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the abutment wall. Rock protection is provided on the front slope of the abutment. Effect of erosion on the slope face was not observed (September 2, 2015).

**Photograph 5**: Looking at the south abutment wall, west wingwall and the adjacent slope of the south abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the abutment wall and wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (September 2, 2015).

**Photograph 6**: Looking at the south abutment, east wingwall and the adjacent slope of the south abutment of the south abutment of the Colonel Talbot Road (Highway 4) Underpass Structure from Highway 402. Surficial cracks were observed on the abutment and wingwalls. Rock protection is provided on the front slope of the abutment. A void was observed underneath the rock protection. The adjacent slope is vegetated and effect of erosion on the slope face was not observed (September 2, 2015).


**Photograph 7**: Looking at the void at the south abutment. A concrete element was exposed (September 2, 2015).

**Photograph 8**: Looking at the center piers at the south shoulder of EBL of Highway 402. Surficial cracks were observed on the piers. The ground around the piers was partially covered with vegetation and the effect of erosion on the ground surface was not observed (September 2, 2015).