



**THURBER** ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD  
HIGHWAY 16 OVERHEAD AT CNR RAILWAY, SITE NO. 16X-0131  
GWP 4024-20-00 / ASSIGNMENT NO.: 4019-E-0010.2**

**SITE NO. 16X-0131/B0**

**Geocres No.: 31B12-003**

Report to:

**MTO c/o AECOM Canada Ltd.**

Latitude: 44.752392°  
Longitude: -75.476957°

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**PART 1. FACTUAL INFORMATION**

**1 INTRODUCTION**

Thurber Engineering Ltd. (Thurber) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation Ontario (MTO) under Assignment No. 4019-E-0010, Work Item No. 2, to carry out foundation investigations to support the Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 16 to Maitland Road. The overall scope of work comprises replacement or rehabilitation of 14 existing structures, including 10 bridges and four structural culverts.

This report addresses the Highway 16 structure (Site No.16X-131/B0) that crosses over the Canada National Railway (CNR) Line in the Township of Edwardsburgh within Leeds and Grenville County, Ontario.

This section of the report presents the factual findings obtained from a foundation investigation completed at the site and was informed by existing subsurface information pertinent to the site, obtained from the MTO's Foundation Library (Geocres No. 31B00-034 and 31B00-047).

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, provide a borehole location plan, records of boreholes, a stratigraphic profile, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and replacement of the structure was developed in the course of the current investigation.

It is a condition of this report that Thurber's performance of its professional services will be subject to the attached Statement of Limitations and Conditions.

**2 BACKGROUND AND SITE DESCRIPTION**

**2.1 General**

The Highway 16 overhead structure crosses over the CNR railway approximately 1.2 km northwest of County Rd 2 in the community of Johnstown and approximately 285 m south of the Highway 401 Underpass at Highway 16. For project orientation purposes, Highway 16 will be described as oriented north-south and the CNR railway as oriented east-west.



The Highway 16 structure carries two through lanes in each direction over the CNR railway. Traffic volume on this section of Highway 16 is understood to have been 2,400 AADT in 2016. Concrete parapet walls are present along the east and west edges of the structure deck, and an approximately 3-m wide non-traversable paved median is present between the north and south bound lanes. Galvanized W-beam guiderails supported on metal posts are present at all four quadrants and extend to as much as approximately 300 m back from the abutments. Narrow shoulders are present between the travelled lanes and the concrete parapet walls. No sidewalks are present along the structure. Concrete wingwalls are present at the four quadrants. The embankments are approximately 10 m in height with side slopes inclined as steep as approximately 2H:1V and are vegetated with deciduous trees, shrubs, and grasses. No signs of global instability of the embankments were noted during the field investigation.

The CNR railway at the location of the site includes two lines of tracks and has a total right-of-way width of about 11.1 m between the main span piers. The Highway 401 W-N/S off-ramp and the N-E and S-E on-ramps are located immediately north of the site. Traffic volume on Highway 401 is understood to have been 19,900 AADT in 2016.

The site is in a semi-rural setting, and the area directly adjacent to the bridge is undeveloped land with a mix of cleared private properties and swampy and densely vegetated areas with deciduous trees and shrubs. Overhead utility lines are present parallel the south side of the railway alignment, beneath the bridge structure. Light poles are present near the entry/exit of the Highway 401 ramps. The terrain is relatively flat, aside from the existing embankments, ramps, and associated ditches, which are relatively rugged. Storm water drainage in the area is to roadside ditches.

Photographs showing general conditions in the project area at the time of the field investigation in December 2022 are presented in Appendix D.

## **2.2 Existing Structure Information**

As per the original General Plan Drawing (No. TWP#27-131-1-A) dated 1959, the existing structure was built in 1963 and is a five-span cast-in-place reinforced concrete slab bridge. The drawing indicates that the bridge deck is approximately 54.0 m long and 20.7 m wide, with an approximate 14-degree skew to the CNR tracks. The structure contains a total of four piers which comprise five columns each. There are cast-in-place concrete retaining wing walls located at all four quadrants of the bridge to retain the embankment slopes. It is understood that the structure was rehabilitated in 1989, 1997, 2014 and 2016 under Contract No. 89-37 (WP No. 58-81-01), Contract No. 40-97-35 (WP No. 81-91-01), Contract No. 2014-4010 and Contract No. 2016-4036 (WP No. 4094-13-01), respectively. The rehabilitation included modification to the existing expansion joints, installation of drainage improvements, replacement of the abutment bearings, waterproofing and deck paving works, conversion to semi-integral abutments, installation of a 60 mm concrete overlay, replacement of the existing steel handrails with new concrete barrier walls with railing, replacement of the approach slabs and installation of a thermally sprayed anode system on the soffit.



Drawing No. TWP#27-131-2-A indicates that the abutments and piers are supported on BP 12x53 H-Piles driven to bedrock and installed on batter and vertically. The design load per pile is indicated to be 45 tons.

### 2.3 Existing Subsurface Information

The following historical foundation investigation report was available for this site within MTO's online Geocres library:

- Geocres Report No. 31B00-034 (e. m. peto associates ltd., March 1959) presents the results of a foundation investigation carried out for the design and construction of the existing bridge structure. The field investigation included six boreholes drilled near the four quadrants of the proposed bridge structure or along the Highway 16 centreline. In general, the boreholes indicated the presence of topsoil over silty clay underlain by clayey silt which is, in turn, underlain by bedrock. Bedrock was proven by coring in two boreholes in the investigation. The boreholes were advanced to depths ranging from 10.4 m to 13.1 m below the existing ground surface (base elev. 71.3 m to 69.2 m).
- Geocres Report No. 31B00-047 (e. m. peto associates ltd., August 1959) presents the results of a supplementary foundation investigation carried out for the design and construction of the existing bridge structure. The field investigation included three boreholes drilled near the existing centreline of Highway 16 to better assess the strength-depth characteristics of the sub-soil. In general, the boreholes indicated the presence of topsoil over silty clay underlain by clayey silt. No bedrock was cored during the investigation. The boreholes were advanced to depths ranging from 10.4 m to 11.0 m below the existing ground surface.

The historical stratigraphy drawings and borehole logs have been included in Appendix A and Appendix B.

### 2.4 Site Geology

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984) and the Ontario Geological Survey maps (MRD228), the site lies on the border of the physiographic regions known as the Glengarry Till Plain and the Edwardsburg Sand Plain. The Glengarry Till Plain is characterized by typically undulating to rolling surface containing well-formed drumlins, intervening clay flats, and stony glacial tills with a high proportion of limestone pieces. The Edwardsburg Sand Plain is characterized by slightly undulating sand plain of glaciofluvial origin. The rock strata in both areas are generally composed of limestone, dolostone, and calcareous sandstone.

According to Crins et al. (2009)<sup>i</sup>, the project area is described as Ecoregion 6E (Lake Simcoe-Rideau) within the Ontario Shield Ecozone. According to Wester et al. (2018)<sup>ii</sup>, the ecoregion is subdivided into Ecodistrict 6E-11 (Smiths Falls Ecodistrict). The area is characterized by discontinuous layer of shallow calcareous morainal material overlying Paleozoic bedrock.



The Ontario Geological Survey maps (MRD126) suggest the site is underlain by dolostone and sandstone. Map P.2722<sup>iii</sup> indicates that the bedrock in the project area is of Oxford Formation that consists of sub lithographic to fine crystalline dolostone.

### 3 SITE INVESTIGATIONS AND FIELD TESTING

A site investigation and field-testing program was carried out between December 1 and 5, 2022, and consisted of two on-road boreholes identified as 131-22-01 and 131-22-02 put down near the Highway 16 Overhead abutments. The boreholes were advanced using a truck mounted CME 55 drill rig equipped with Hollow Stem Augers, NW casing, and NQ coring equipment. Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locates/clearances in the vicinity of the borehole locations. In addition, MTO was contacted to obtain Electrical and Fibre Optic locates for the project limits.

The borehole coordinates, elevations, and termination depths are provided in Table 3-1. The as-drilled elevations of all boreholes were surveyed by Thurber with a Trimble Catalyst DA1 antenna with centimeter accuracy. The elevations were surveyed relative to the benchmark information provided by AECOM, and the horizontal locations were measured by Thurber relative to existing site features. The borehole coordinates and elevation are shown on the Borehole Location and Soil Strata Drawings in Appendix A and on the individual Record of Borehole sheets included in Appendix B. The borehole coordinates are referenced to MTM Zone 9.

**Table 3-1: Borehole Summary**

<b>Borehole No.</b>	<b>Drilled Location</b>	<b>Northing (Latitude)</b>	<b>Easting (Longitude)</b>	<b>Ground Surface Elevation (m)</b>	<b>Termination Depth (m)</b>
131-22-01	Near the South Abutment	4 957 388.9 (44.751917)	385 837.4 (-75.476522)	91.8	23.4
131-22-02	Near the North Abutment	4 957 445.7 (44.752432)	385 801.1 (-75.476972)	92.4	26.4

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Tests (SPT) in general accordance with ASTM D 1586. In-situ shear vane testing was carried out within the cohesive strata, where possible, using an MTO 'N' sized vane in general accordance with ASTM D 2573. The boreholes were advanced to depths ranging from 23.4 to 26.4 m (base elev. 68.4 m to 66.0 m). Coring was required to advance the boreholes through portions of the fill and into bedrock. A standpipe piezometer was installed in Borehole 131-22-02 to allow for measurements of the groundwater level after drilling. The details for the standpipe piezometer are illustrated on the respective Record of Borehole sheets provided in Appendix B.



The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The drilling supervisor logged the boreholes and processed the recovered soil and rock samples for transport to the Thurber's Ottawa laboratory for further examination and testing.

Following completion of the field investigation, Borehole 131-22-01 was decommissioned in general accordance with O.Reg. 903, as amended. The standpipe piezometer at Borehole 131-22-02 was decommissioned in accordance with MOE requirements on April 26, 2023. The pavement surface at both borehole locations was reinstated with cold patch asphalt.

#### **4 LABORATORY TESTING**

Geotechnical laboratory testing carried out as part of the current investigation included natural moisture content determination and visual identification of all retained soil samples. Testing for grain size distribution and Atterberg Limits were also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. One Unconfined Compressive Strength (UCS) Test was conducted on a recovered core sample from Borehole 131-22-02.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory test results are presented on the figures included in Appendix C.

#### **5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS**

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and on the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following sections. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions will vary between and beyond borehole locations. Soil classification is in general accordance with ASTM D 2487 with descriptions of cohesive soils and secondary components as outlined in the MTO Guideline for Foundation Engineering Services Manual (April 2022).

In general, the encountered stratigraphy consisted of fill over a native deposit of clay to clayey silt underlain by dolostone bedrock.

##### **5.1 Pavement Structure**

Both boreholes were advanced from the road surface and encountered a 100 mm to 255 mm layer of asphalt. A 200 mm thick layer of concrete was encountered beneath the asphalt in Borehole 131-22-02.



A fill layer composed of gravelly sand with some fines was encountered below the asphalt/concrete in both boreholes. The layer was 0.2 m to 1.2 m thick (base elev. 90.9 m to 91.3 m).

A single Standard Penetration Test (SPT) N-value of 30 blows per 0.3 m of penetration was recorded in the layer, indicating a dense relative density. The recorded moisture content on one sample of the fill was 3%.

The results of a gradation analysis completed on one sample of the fill are illustrated on Figure C1 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	24
Sand	57
Silt	19
Clay	

## 5.2 Embankment Fill

### 5.2.1 Silty Sand Embankment Fill

Silty sand embankment fill was encountered beneath the gravelly sand fill in both boreholes. Some gravel was noted within the layer. The silty sand embankment fill layer was 4.1 m to 6.1 m thick (base elev. 87.2 m to 84.8 m). SPT N-values ranged from 6 to 64 blows per 0.3 m of penetration, but were typically over 15, indicating a loose to very dense, typically compact to dense material.

The recorded moisture contents of samples of the silty sand embankment fill ranged from 5% to 12%. The results of gradation analyses completed on two samples of the fill are illustrated on Figure C2 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	11 to 12
Sand	46 to 53
Silt	36 to 42
Clay	

### 5.2.2 Gravelly Silty Sand Fill

Gravelly silty sand fill was encountered beneath the silty sand fill in Borehole 131-22-02. Cobbles and boulders were inferred within the layer. The fill layer was 3.1 m thick (base elev. 81.7 m). SPT



N-values ranged from 110 blows per 0.3 m of penetration to greater than 100 blows for 75 mm of penetration, indicating a very dense relative density.

The recorded moisture contents ranged from 3% to 5%. The results of a gradation analysis completed on one sample of the fill are illustrated on Figure C3 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	26
Sand	49
Silt	25
Clay	

### 5.2.3 Clay Fill

Clay fill was encountered beneath the silty sand fill in Borehole 131-22-01. The fill layer was 4.5 m thick (base elev. 82.7 m). SPT N-values ranged from 7 to 14 blows for 0.3 m of penetration. In-situ undrained shear strength testing was attempted but the vane was unable to be turned as the material has an undrained shear strength greater than 102 kPa. The clay fill can be described as very stiff in consistency.

The recorded moisture contents ranged from 27 to 33%. The results of a gradation analysis completed on one sample of the fill are summarized in the table below and on the Record of Borehole sheet in Appendix B, and are illustrated on Figure C4 of Appendix C.

Soil Particle	Percentage (%)
Gravel	0
Sand	9
Silt	40
Clay	51

Results of Atterberg Limit testing carried out on the sample are illustrated in Figure C5 of Appendix C. The results of this test are summarized below and on the Record of Borehole sheet in Appendix B. The laboratory results indicate that the clay exhibits intermediate plastic behaviour (CI).

Parameter	Value
Liquid Limit	45
Plastic Limit	23
Plasticity Index	22





### 5.3 Silty Sand (SM)

A native deposit of silty sand with organics, inferred to be a historical topsoil layer, was encountered below the gravelly silty sand fill in Borehole 131-22-02. The deposit was approximately 100 mm thick (base elev. 81.6 m).

The recorded moisture content of one sample of the deposit was 25%. The results of Atterberg Limit testing conducted on the fines portion sample from Borehole 131-22-02 indicate a non-plastic material. The results of a gradation analysis completed on the sample of the deposit are illustrated on Figure C6 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	59
Silt	28
Clay	13

### 5.4 Clay (CH) to Silty Clay (CI)

A deposit of clay to silty clay was encountered below the embankment fill and the silty sand with organics. The thickness of the layer ranged from 11.0 to 11.9 m (base elev. 71.7 m to 69.7 m). Where SPTs were conducted within the layer, the N-values ranged from 2 to 33 blows per 0.3 m of penetration but were typically between about 4 and 14 blows per 0.3 m of penetration. Several attempts were made to carry out in-situ undrained shear strength testing but the vane was unable to be turned as the material has an undrained shear strength greater than 102 kPa. The clay to silty clay can be described as very stiff in consistency.

Recorded moisture contents ranged from 14% to 43%. Atterberg Limit testing was completed on five samples of the deposit. Results are illustrated in Figure C7 of Appendix C. The results of these tests are summarized below and on the Record of Borehole sheets in Appendix B. The laboratory results indicate that the clay to silty clay generally exhibits intermediate to high plastic behaviour (CI to CH). One sample towards the bottom of the deposit in Borehole 131-22-02 (elev. 70.8 m) exhibited low plasticity (CL-ML).

Parameter	Value
Liquid Limit	22 - 56
Plastic Limit	17 - 26
Plasticity Index	5 - 31

The results of gradation analyses completed on five samples of the layer are illustrated on Figure C8 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.



Soil Particle	Percentage (%)
Gravel	0
Sand	0 – 4
Silt	39 – 78
Clay	19 – 61

## 5.5 Bedrock

Bedrock was proven by coring in both boreholes. The depth to bedrock from the existing road grade ranged from 20.1 m to 22.7 m (elev. 71.7 m to 69.7 m). The bedrock encountered consisted of fresh, fine-grained, grey dolostone. Quartz inclusions were noted in Borehole 131-22-02. In general, the discontinuities were rough, undulating bedding joints. Bedrock logs are provided in Appendix B, and photographs of the bedrock cores are provided in Appendix C. The rock core quality and strength are summarized in Table 5-1.

**Table 5-1: Bedrock Details**

Parameter	Range
Total Core Recovery (TCR), %	100
Solid Core Recovery (SCR), %	67 – 98
Rock Quality Designation (RQD), %	57 – 100
Fracture Index (fractures per 0.3 m) <sup>(1)</sup>	0 – 7
Unconfined Compressive Strength (UCS) <sup>(2)</sup> , MPa	237

Notes: (1) Indicated as "FI" on Borehole Logs

(2) Sample tested from Borehole 131-22-02

Based on the RQD, the bedrock quality is classified as fair to excellent (CFEM, 2006). The result of an unconfined compressive strength test was 237 MPa, indicating that the bedrock is very strong (CFEM, 2006).

## 5.6 Groundwater

A 19 mm diameter standpipe piezometer was installed in Borehole 131-22-02 to allow for measurements of the stabilized groundwater level. The measured groundwater levels are summarized in Table 5-2.

**Table 5-2: Groundwater Level Observations**

Borehole	Bottom of Screen Elev. (m)	Screened Unit	Groundwater Level		Date of Measurement
			Depth (mbgs)	Elevation (m)	
131-22-02	71.2	Silty Clay	11.2	81.2	2022-12-05
			11.4	81.0	2022-12-18
			10.9	81.5	2023-04-26



It should be noted that the values shown above are considered short-term readings and may not reflect groundwater levels at the time of construction. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.



## 6 MISCELLANEOUS

The borehole locations reflect existing site features and access constraints. The as-drilled locations and ground surface elevations were measured by Thurber. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, supplied and operated the drill rig used to drill, test, sample, install a standpipe piezometer, and decommission the boreholes. Traffic control and water were provided by T.G. Carroll Cartage Ltd. of Carp, Ontario. Traffic control was performed in accordance with Ontario Book 7 for short duration closures. The field work was supervised on a full-time basis by I. Khan, E.I.T., under the direction of K. Walker, P.Eng.

Geotechnical laboratory testing was carried out by Thurber's geotechnical laboratory in Ottawa, Ontario. Unconfined Compressive Strength testing was carried out by Stantec in Ottawa, Ontario.

Interpretation of the data and preparation of this report were carried out by A. de Oliveira, E.I.T., I. Khan, E.I.T., and M. Kennedy, P.Eng. The report was reviewed by Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.  
Report Prepared By:

Anderson de Oliveira, EIT  
Engineering Intern

Ibrahim Khan, EIT  
Engineering Intern



Matt Kennedy, M.Sc. (Eng)., P.Eng.  
Senior Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.  
Designated Principal Contact  
Senior Geotechnical Engineer



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**PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 INTRODUCTION**

This report presents the interpretation of the factual data obtained from a preliminary foundation investigation and a desktop review of the available subsurface information conducted by Thurber for the replacement of the existing Highway 16 Overhead of the Canadian National Railway (CNR) in Leeds and Greenville County, Ontario.

This preliminary foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The following sections provide preliminary geotechnical recommendations for the design of the foundation elements as part of the structural assessment and rehabilitation. The discussions and recommendations presented in this report are based on the information provided by the AECOM and the MTO, and on the factual data obtained during the course of this investigation.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

**7.1 Existing Structure**

The Highway 16 overhead structure carries four lanes over the CNR railway and is a five-span cast-in-place reinforced concrete slab bridge with semi-integral abutments. It was constructed in 1963. The bridge deck is approximately 54.0 m long and 20.7 m wide, with an approximate skew angle of 14 degrees to the CNR. For project orientation purposes, Highway 16 will be described as oriented north-south and the CNR railway as oriented east-west.

As per the original General Plan Drawing (No. TWP#27-131-1-A) dated 1959 (see Appendix F), the existing structure contains a total of four piers which are comprised of five columns each.



There are cast-in-place concrete retaining wing walls located at all four quadrants of the bridge to retain the embankment slopes. The bridge was designed with abutments, wing walls and all the piers to be supported on BP 12x53 H-Piles piles driven to bear on bedrock. The abutment piles were designed with a 6:1 batter.

Both abutment pile caps are perched within the existing embankments, designed with an underside at approximate Elevation of 88.1 m and 88.6 m in the south and north, respectively. The drawings indicate 2H:1V side slopes on the all the sides of approach embankments. The central and exterior pier pile caps were designed with undersides at approximate Elevations 82.1 m (Piers B and C) and 85.5 m (Piers A and D).

## **7.2 Proposed Structure**

As per the AutoCAD alignments provided by AECOM on October 20, 2022, the preliminary design of the Highway 16 Overhead at CNR is to accommodate the proposed updates to Highway 16 / Highway 401 interchange, which includes a shift of the Highway 16 centerline of the roadway approximately 20 m to the west. The deck width will vary from about 34.4 m at the north abutment to 31.0 m at the south abutment.

The new approach embankments are anticipated to be on the order of 10 m high. An approximate grade raise at the existing embankment is 0.6 m, as per an email received from AECOM on March 23, 2023.

## **7.3 Applicable Codes and Design Considerations**

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and is in accordance with the Canadian Highway Bridge Design Code, version CSA S6:19, (CHBDC).

In accordance with CHBDC, the analysis and design of the structure takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation of Ontario (MTO).

It is understood that the structure is classified as being part of the “Major Route” importance category.

This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor ( $\Psi$ ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing the factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

The degree of site and prediction model understanding for this site has been assessed to be typical understanding (Section 6.5.3 of CHBDC).



## 8 SEISMIC CONSIDERATIONS

### 8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC)<sup>iv</sup>. Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data include peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ( $S_a(T)$ ) for the reference ground condition (Site Class C) for a range of periods ( $T$ ) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix E.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class, the peak ground acceleration (PGA) and  $S_a(0.2)$ . The PGA for this location for a *reference* Site Class C with a 2% probability of exceedance in 50 years is 0.25 g (1 in 2475 year). This value is to be scaled by the  $F(PGA)$  based on the site-specific Site Class as per Section 4.4.3.3 (Table 4.8) of the CHBDC (see Section 8.2).

### 8.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy below the reference elevation of the foundation element. As outlined in Section 4.4.3.2 of the CHBDC, if the shear wave velocity of the site soil and bedrock is not known, as is the case at this site, the seismic site class may be determined by the harmonic mean of the energy-corrected SPT-N values ( $N_{60}$ ) and/or the undrained shear strength ( $s_u$ ) encountered below the foundation element(s). Based on the range of  $N_{60}$  values recorded in the embankment fill and the  $s_u$  values recorded in the underlying clay to silty clay, a Site Class at these foundation elements may be taken as Site Class D.

The site-specific evaluation and classification should be completed during detailed design with measurement of the shear wave velocity in the upper 30 m of the stratigraphy below the reference elevation and should consider the influence of embankment fill thickness and composition at perched foundation elements.

### 8.3 Seismic Performance Category

In consideration of the Site Class D spectral values for the site and the designated *Major Route* importance category, the bridge structure would fall into Seismic Performance Category 2, if the bridge has a fundamental period greater than or equal to 0.5 seconds, or Seismic Performance Category 3, if the bridge has a fundamental period less than 0.5 seconds, as per Section 4.4.4 (Table 4.10) of the CHBDC.

### 8.4 Liquefaction Potential

The susceptibility of the embankment fill at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and



Idriss (2014)<sup>v</sup>. Based on the SPT data and measured groundwater table elevation, the analysis results indicated that the cohesionless soils are generally not susceptible to liquefaction.

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Bray et al. (2004)<sup>vi</sup> criteria using index properties. The results of the analysis indicate the cohesive material is not susceptible to cyclic mobility.

## 9 STRUCTURE FOUNDATION ALTERNATIVES

### 9.1 Foundation Alternatives

Based on the subsurface conditions at the site, deep foundations have been considered for the replacement of the Highway 16 Overhead. Shallow foundation options are considered to not be feasible at the site due to the deep clay to silty clay deposit and, as such, are not discussed further in this report.

The deep foundation alternatives are presented below and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks, and consequences. The evaluation is summarized in the table provided in Appendix G.

- Driven Steel H-piles

Steel H-piles driven to bedrock through the new approach embankments to support “perched” abutments should be considered and would allow construction of integral abutments. H-piles could also be considered to support the central pier.

- Caissons

Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however, the high lateral stiffness of caissons is not compatible or suitable for integral abutments. The groundwater level within the clay deposit may pose additional construction challenges resulting from potential unbalanced hydraulic pressure heads and caisson base heave during construction. This would require the use of temporary liners or synthetic slurry to counterbalance groundwater pressure.

Caissons are considered feasible but are not the preferred option from a foundations perspective to support the new bridge.

### 9.2 Recommended Foundation

Based on an evaluation of the foundation alternatives presented above and in Appendix G, the recommended foundation approach from a geotechnical perspective is to support the new bridge abutments and pier on driven steel H-piles.



### 9.3 Construction Methodology

It is assumed that staging areas for the bridge construction will be set up west of the existing bridge alignment and will be accessible from the existing Highway 16 to minimize conflict with the Highway 16 traffic. Due to the existing subgrade soils present, embankment construction in advance of construction of the foundation elements (including preload and surcharge) will likely be required for the new bridge approach embankments and abutment locations. It is anticipated that the majority of the construction of the new approach embankments and structure can take place while maintaining traffic flow on existing Highway 16. However, consideration of the staging requirements and impacts from the construction of the new structure and removal of the existing structure should be considered during subsequent design stages.

## 10 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

Key elevations (approximate) based on the results of the field investigation and the proposed structure are as follows:

- Existing Highway 16 grade 91.8 m to 92.4 m
- Proposed Highway 16 grade 92.4 m to 93.0 m
- Underside of abutment pile caps, assumed 89.1 m to 89.6 m
- Underside of pier pile cap, assumed 82.0 m
- Base of existing embankment fill 81.7 m to 82.7 m
- Base of Clay to Silty Clay/Top of Bedrock 69.7 m to 71.7 m

As noted above, it is understood that the grade of Highway 16 is to be increased by approximately 0.6 m which sets the new pavement elevation for Highway 16 at approximately 92.7 m.

### 10.1 Driven Steel H-Pile Foundations

#### 10.1.1 Axial Resistance

The new abutments and pier may be founded on steel H-piles end-bearing on the bedrock. The estimated pile tip elevations based on piles reaching refusal at the bedrock surface are summarized in Table 10-1.

**Table 10-1: Estimated Pile Tip Elevations**

Foundation Element	Approximate Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (top of bedrock) (m)	Estimated Pile Length (m)
South Abutment	89.1	71.7	17.4
Pier	82.0	70.7 <sup>1</sup>	11.3
North Abutment	89.6	69.7	19.9

Note: (1) Bedrock surface elevation has been extrapolated from existing subsurface information





For preliminary purposes, the factored geotechnical axial resistance at ULS of Grade 350W HP 310x110 steel piles driven to refusal on or in the dolostone bedrock can be taken as 2,500 kN. The SLS condition will not govern for piles founded on bedrock.

The factored geotechnical axial resistances provided include the following factors:

Consequence factor ( $\Psi$ ) of 1.0 (as per CHBDC Table 6.1)

Geotechnical resistance factors (CHBDC Table 6.2):

- $\phi_{gu} = 0.4$ , ULS (static analysis; typical degree of understanding)

*The structural resistance of the pile must be checked by the structural engineer which may govern the design.*

The geotechnical resistance provided above is applicable for pile spacing greater than 3 pile widths. Driven piles must be installed in accordance with OPSS.PROV 903. The potential for conflict with the existing steel H-piles supporting the existing structure must be checked.

As the piles are to be driven to bedrock the pile tips of the new piles at the site should be protected from damage during driving with pile tip protection from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent. The appropriate pile driving note is "Piles to be driven to bedrock."

If driven piles are employed, the effects of pile driving on nearby structures and underground utilities will need to be assessed during detailed design. The need for vibration monitoring and settlement monitoring of nearby utilities and structures during pile driving should be evaluated on a site-specific basis to limit potential impacts on existing facilities such as the existing rail infrastructure. Clarification of the particular settlement tolerances allowed within the adjacent CNR right-of-way must be determined prior to development construction methodologies and vibration monitoring plans. A preconstruction condition survey of the existing structures and utilities in the vicinity may be required prior to commencement of pile installation.

#### 10.1.2 Downdrag

A thick cohesive deposit of clay to silty clay is present beneath the embankment fill at both abutments of the existing bridge. Settlement of the clay is expected to occur beneath the widened portions of the 10 m high embankments to be constructed west of the existing embankment. Based on the preliminary data available this settlement will result in a preliminary unfactored downdrag load of up to 900 kN per pile at the abutments. Downdrag forces are not expected at the pier.

This downdrag load should be multiplied by a load factor as per CHBDC Table 3.3 and Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.



The downdrag loading on new foundations could be reduced with construction of a preload prior to the installation of deep foundations, however, the impact of the preload on existing foundations would need to be assessed. Ultimately downdrag loading will be dependant on the grade raise and new embankment requirements and will need to be further assessed at the detailed design stage.

### 10.1.3 Uplift Resistance

The embankment fill at the abutments (if the pile caps are perched within the embankments) and the native clay at all foundation elements will provide uplift resistance to the piles. Shaft friction of the embankment fill and native clay along the piles were calculated, assuming the piles met effective refusal to driving at the elevations provided in Table 10-1, above.

The factored geotechnical tensile resistance for a single HP 310x110 pile at either abutment may be taken as 300 kN under static conditions and 1,000 kN under seismic conditions. For piles at the pier, the factored geotechnical tensile resistance for a single HP 310x110 pile may be taken as 210 kN under static conditions and 700 kN under seismic conditions. These values include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2) of  $\phi_{gu} = 0.3$  (static analysis; typical degree of understanding)
- Geotechnical resistance factors (CHBDC Table 6.3) of  $\phi_{gu} = 1.0$  (seismic analysis; typical degree of understanding, performance-based design)

### 10.1.4 Lateral Resistance of Piles

P-Y data can be provided upon request following the receipt of a foundation layout plan. As a preliminary guideline, lateral resistance of the piles can be estimated using a value for the coefficient horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ).

For cohesionless soils:

$$\begin{aligned} k_s &= n_h \cdot z / D & (\text{kN/m}^3) \\ p_{ult} &= 3 \cdot \gamma' \cdot z \cdot K_p & (\text{kPa}) \end{aligned}$$

For cohesive soils:

$$\begin{aligned} k_s &= 67 \cdot c_u / D & (\text{kN/m}^3) \\ p_{ult} &= 0 & (\text{kPa}) \text{ at the top of the pile, increasing linearly to} \\ &= 9 \cdot c_u & (\text{kPa}) \text{ at } z \geq 3 \cdot D \end{aligned}$$

where:



$z$	=	depth of embedment along pile (m)
$D$	=	pile width or diameter (m)
$n_h$	=	coefficient related to soil density ( $\text{kN/m}^3$ )
$c_u$	=	undrained shear strength (kPa)
$\gamma'$	=	effective unit weight ( $\text{kN/m}^3$ )
$K_p$	=	coefficient of passive lateral earth pressure

The above equations and recommended parameters in Table 10-2 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance as defined above. A geotechnical resistance factor of 0.5 ( $\phi_{gu}$ ) and 0.8 ( $\phi_{gs}$ ), as per Table 6.2 of the CHBDC, is to be applied to the calculated ultimate ULS and SLS values, respectively.

The spring constant,  $K_s$ , for analysis may be obtained by the expression,  $K_s = k_s \cdot L \cdot D$  ( $\text{kN/m}$ ), where  $L$  is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \cdot L \cdot D$ . This represents the ultimate load at which geotechnical failure of the pile occurs and will not support any additional load at greater displacement.

**Table 10-2: Soil Parameters for Lateral Pile Design**

Soil Type	$\gamma'$ ( $\text{kN/m}^3$ )	$n_h$ ( $\text{kN/m}^3$ )	$c_u$ (kPa)	$K_p$
Existing/New <sup>1</sup> Fill	20 (above WT <sup>2</sup> )	3,000	N/A	3.0
Native Cohesive Soils	7.5 (below WT)	N/A	100	2.6

Note: (1) Assuming new embankment fill consists of well-compacted engineered fill  
(2) Water Table

The modulus of horizontal subgrade reaction may have to be reduced based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Figures C6.23, C6.24 and C6.25 in the Commentary to the CHBDC 2019.

## 10.2 Caisson Foundations

Support of the abutments and/or pier may be provided by caisson foundations. The caissons would have to be constructed by installing a steel casing into the top of the bedrock using drilling methods that would allow reliable penetration through potential obstructions that may be encountered in any existing fill layers. A socket would then be drilled below the steel casing and into the bedrock, cleaned, and the casing and socket would be filled with concrete in a single pour after installation of reinforcing steel. Coring equipment must be able to seat the casing into the bedrock and penetrate the bedrock without fracturing the sidewalls. The tension/uplift resistances provided are based on full contact of the caisson concrete with the socket sidewalls. Depth of socket should be measured downward from the lower bedrock elevation for a sloping bedrock scenario. Caissons should be installed in accordance with OPSS.PROV 903. The strength and



hardness of bedrock at this site must be considered when selecting equipment to excavate the rock socket.

The caisson construction method should include use of temporary or permanent casings (liners) sealed into the bedrock. The caisson socket would extend into the bedrock below. Using a temporary casing that is extracted during the concrete pour to reduce material costs is feasible, if the caissons are installed in combination with drilling fluid to maintain the stability of the side walls. The use of temporary casing would require careful control of the concrete level. Alternatively, caisson casings may be left in place as permanent liners to reduce the potential for disturbance of the soil-concrete interface that may occur during removal of temporary liners. Permanent liners would assist in maintaining the integrity of the concrete caisson by reducing the risk of infiltration of soil or water prior to concrete curing. Ultimately, the contractor will be responsible for selecting the construction means and methods based on cost and risk considerations.

The Contractor shall use appropriate means to clean and inspect the caisson base. The Contractor shall apply means necessary (such as air lift pump or hydraulic pump, etc.) to clean the base of the caissons. The base cleaning method, inspection method, and any additional measures required to satisfy the acceptance criteria must be selected by the Contractor to ensure direct contact between the concrete and un-weathered bedrock over the entire area of the base.

#### 10.2.1 Axial Resistance

The axial geotechnical capacity at factored ULS for a caisson with a permanent steel casing filled with concrete and socketed a minimum of 2 caisson diameters below the steel casing and into sound bedrock is provided in the table below. The caisson capacities include resistance factors ( $\phi_{gu}$ ) of 0.4, 0.3 and 1.0 for compressive, tensile-static and tensile-seismic resistance, respectively, at ULS as per Table 6.2 of the CHBDC (static analysis – typical understanding). The SLS condition will not govern for a caisson socketed into sound bedrock.

**Table 10-3: Axial Geotechnical Resistance for Caissons**

Caisson Diameter (mm)	Factored ULS Compression (kN)	Factored SLS Compression (kN)	Factored ULS Tension Static (kN)	Factored ULS Tension Seismic (kN)
915	11,000	will not govern	8,000	27,500
1220	15,000	will not govern	11,000	37,500
1525	25,000	will not govern	18,500	62,500
1830	38,000	will not govern	28,500	95,000

The structural resistance of the caissons must be checked by the structural designer. The depth of socket into sound bedrock should be lengthened, if required, based on the required lateral capacity requirements (recommendations provided in Section 10.1.4), moment capacity and seismic analysis to satisfy the structural assessment.



### 10.2.2 Downdrag

A cohesive deposit of clay, silty clay and clayey silt was identified beneath the fill at both abutments of the existing bridge. Settlement of the clay is expected to occur if a 10 m embankment is constructed west of the existing embankment. Based on the preliminary data available and the assumed underside of pile caps provided in Section 10, this settlement will result in a preliminary unfactored downdrag load of up to 2,000 kN, 2,700 kN, 3,500 kN, and 4,100 kN per caisson for each 915 mm, 1,220 mm, 1,525 mm, and 1,830 mm diameter caissons, respectively, if the caissons are constructed prior to the full embankments.

This downdrag load should be multiplied by a load factor as per CHBDC Table 3.3 and Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

The downdrag loading on new foundations could be reduced with construction of an embankment preload prior to the installation of deep foundations, however, the impact of the preload on existing foundations would need to be assessed. Ultimately downdrag loading will be dependant on the grade raise, construction sequencing, caisson cap elevations, and new embankment requirements and will need to be further assessed at the detailed design stage.

Downdrag forces acting upon pier caissons are expected to be negligible provided the grade of the CNR is reinstated to match existing.

### 10.2.3 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 10.1.4 above for driven piles may be used to estimate the lateral geotechnical resistance of the caissons.

## 10.3 Frost Depth

The depth of frost penetration at this site is estimated to be 1.6 m (as per OPSD 3090.101). Footings and pile caps for the abutments and retaining walls should be founded at or below this depth or provided with equivalent insulation.

## 10.4 Backfill and Lateral Earth Pressures

Backfill to the abutments should consist of free-draining granular material conforming to OPSS Granular A or Granular B Type II specifications. The granular material should be placed to the extents shown in OPSD 3101.150. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Lateral earth pressure parameters provided in Table 10-4 and Table 10-5 in the sections below consider that the wall is vertical and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. Where back slopes are horizontal, the corresponding coefficients provided in Table 10-4 and Table 10-5 should be used. If other backfill and wall geometries are to be considered, Thurber will need to calculate the appropriate earth pressure coefficients.

#### 10.4.1 Static Lateral Earth Pressure

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but under drained conditions the lateral earth pressure is generally given by the following expression:

$$\sigma_h = K * (\gamma \cdot h + q)$$

where:

$\sigma_h$	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see Table 10-4) ( $K_a$ for yielding walls, $K_o$ for non-yielding walls)
$\gamma$	=	unit weight of retained soil (see Table 10-4), use submerged unit weight below groundwater level
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 10-4.

**Table 10-4 Static Earth Pressure Coefficients**

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	OPSS SSM & Existing Granular Fill
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	22.8	21.2	20.0
Angle of Internal Friction, $\phi$	35°	32°	30°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.47	0.50
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.31	0.33
Passive, $K_P$ (Movement towards Soil Mass) in front of wall	3.7	3.3	3.0

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The movement required can be assessed from Table C6.12 of the Commentary to the



CHBDC. Active earth pressures should be used for unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

#### 10.4.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14.7.2 of the CHBDC, retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14.7.2 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$ , for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$ , for non-yielding walls

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-5 may be used for a horizontal back-slope. The provided earth pressure coefficients are calculated using a site-adjusted PGA of 0.27 g, based on a Seismic Site Class D, a reference (Site Class C) PGA with a 2% probability of exceedance in 50 years of 0.25 g (Geological Survey of Canada – Fifth Generation) and a  $F(PGA)$  of 1.25 as per Table 4.8 of the CHBDC.

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_h = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d)$$

where:

$\sigma_h$	=	lateral earth pressure at depth d (kPa)
d	=	depth below the top of the wall (m)
K	=	static earth pressure coefficient ( $K_A$ for yielding walls, $K_o$ for non-yielding walls)
$\gamma$	=	unit weight of retained soil, use submerged unit weight below groundwater level
$K_{AE}$	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)



**Table 10-5 Combined Static and Seismic Earth Pressure Coefficients**

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	OPSS SSM & Existing Granular Fill
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	22.8	21.2	20.0
Angle of Internal Friction, $\phi$	35°	32°	30°
<b>Non-Yielding Wall</b>			
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.45	0.50	0.54
<b>Yielding Wall</b>			
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.34	0.39	0.42

## 10.5 Wing Wall Foundations

If wing walls are required for the structure, they may design to be founded on driven steel H-piles for continuity with the pile-supported structure or designed to be uncoupled from the structure and supported on Retained Soils System (RSS) walls.

### 10.5.1 Driven Steel H-Pile Foundations

Conventional concrete wing walls may be supported on piles with an assumed pile cap elevation ranging from Elevation 89.1 m to 89.6 m. Please see Section 10.1 for driven steel H-Pile foundation recommendations.

### 10.5.2 RSS Walls

Retained soil system (RSS) walls are feasible at this site. The design of proprietary RSS walls is the responsibility of the supplier. Typically, such systems do not require full frost protection as they are able to tolerate some movement due to frost heave. The RSS system should be designed in accordance with the MTO RSS Design Guidelines. Once the location and height of the wall is established, the following recommendations should be confirmed:

Performance	H
Appearance	H
Acceptance	A

RSS walls should have a minimum embedment of 0.8 m. The underside elevation of the RSS walls adjacent to the abutments are anticipated to be at approximate Elevations 82.7 m to 81.6 m. A minimum 1 m thick engineered fill pad constructed on the underlying undisturbed native clay to silty clay soils should be provided below the RSS wall as well as under the reinforced retained soil. The engineered fill pads should consist of OPSS Granular A placed and compacted in accordance with OPSS.PROV 501. Engineered fill pads should be constructed with 1H:1V sides slopes with the crest of slope a minimum of 1 m from the edge of footing and reinforced retained soil on all sides. The subgrade soils may become disturbed when saturated and should be protected by prompt placement of a geotextile separator (Class II non-woven geotextile with a





maximum FOS of 150  $\mu$ m: OPSS.PROV 1860) and the engineered fill pad placed immediately after excavation and inspection.

RSS walls with a minimum embedment of 0.8 m and bearing on an engineered fill pad and constructed as outlined above may be designed based on a factored geotechnical resistance at ULS 300 kPa. The ULS bearing resistance value is for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.2 and Clause 6.10.5.

The factored geotechnical resistance includes the following factors:

- Consequence factor ( $\Psi$ ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factor ( $\phi_{gu}$ ) of 0.5 (static analysis; typical degree of understanding as per CHBDC Table 6.2)

The factored geotechnical resistance at SLS will be significantly influenced by the construction sequencing and the consolidation settlement of the compressible clay to silty clay under the embankment loading. Further, RSS walls on the order of 10 m high may not be feasible if founded on the native clay to silty clay without a gravel pad of sufficient thickness. RSS walls are considered feasible provided:

- Preloading/pre-construction of the embankments is carried out prior to the RSS walls
- A granular pad of sufficient thickness is provided below the RSS walls
- Consideration is given to founding the RSS walls at a higher elevation, within the embankments

Further discussion on anticipated settlement magnitudes beneath the embankment fill and potential mitigation measures are described below in Section 10.7.

The lateral pressure comments provided in Section 10.4 may be used in RSS design. Please refer to Section 10.6 for comments on global stability.

## 10.6 Embankment Stability

Based on the available original structure drawings and observations during the 2022 field investigation, the grade of the current Highway 16 lanes ranges from about 91.8 to 92.4 m with embankments of up to about 10 m in height and sloped at roughly 2H:1V.

The embankment to be constructed for the proposed new alignment will shift the centerline of the roadway approximately 20 m to the west and is anticipated to be roughly 10 m in height. For the purpose of preparing preliminary geotechnical design recommendations, a number of assumptions have been made that are consistent with MTO's standard highway design practices:

- Topsoil and other deleterious material will be removed from within the footprint (including from the existing sideslope) prior to constructing the new embankment

- Where new fill is placed against an existing embankment slope or on a sloping ground surface steeper than 3H:1V, the existing slope will be benched (OPSD 208.010).
- The embankment will be constructed using granular fill meeting at a minimum the requirements of Select Subgrade Material (SSM) as per OPSS.PROV 1010.
- Conventional (non-reinforced) granular fill embankments will be constructed with side slopes not steeper than 2H:1V.
- Retained soil at RSS walls are to consist of OPSS Granular B Type II, up to 6 m in width and supported on a 1 m thick Granular A bedding layer.
- Mid-height 2 m wide benches should be incorporated along the length of embankments with heights at or exceeding 8 m in granular fill.
- Permanent drainage and erosion protection will be provided for all granular embankment slopes.

Table 6.2 of the CHBDC for embankment fills with a *typical* degree of site understanding and a *typical* consequence level (a Consequence Factor,  $\Psi$ , of 1.0) generates minimum Factors of Safety of 1.5 and 1.3 for static permanent and static temporary conditions respectively.

For seismic analysis, Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum resistance factor of 0.95 ( $\Phi_{gu, static(temporary)} = 0.75 + 0.2$ ) for force-based design and 1.0 for performance-based design. Based on these values and  $\Psi$  of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. However, as is stated in Section 6.14.9.1 of the CHBDC, some embankment displacement can occur where the pseudo-static Factor of Safety is less than 1.3; in this case, the bridge foundations must be designed to withstand the permanent deformations and/or slope stabilizing measures shall be incorporated into the design. Where the pseudo-static Factor of Safety is greater than or equal to 1.3, the slope is considered to be seismically stable with deformations of less than 50 mm.

In addition, Sections 6.14.2.1 and 6.14.2.3 of the CHBDC present performance criteria requirements for Major Route geotechnical systems (embankments) inside and outside the bridge interface zone, respectively. Based on Clause 6.14.2.2, the bridge interface zone at this site extends to 20 m behind the abutments (based on fill heights of up to about 10 m). The performance criteria for the Major Route embankments are as follows:

- Within the bridge interface zone (bridge approaches): 100% of the travelled lanes shall be available for use following a ground motion event with a return period of at least 475 years.
- Outside the bridge interface zone (beyond bridge approaches): sites that fall within Seismic Performance Category 2 or 3 (See Section 8.3) shall have at least 50% of travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years.



The stability analyses considered site-adjusted (Site Class D) design PGA values of 0.27 g and 0.10 g for ground motions with return periods of 2,475 and 475 years, respectively, as per Section 4.4.3.2 of the CHBDC.

Slope stability assessments have been carried out for the highest/critical embankment slope, just behind the abutment. Embankment slope stability was evaluated using GeoStudio 2021 Slope/W software for limit equilibrium analysis. Input parameters for the analyses are based on the findings of the 2022 boreholes and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy based on the existing ground surface contours and nearest boreholes;
- Embankment widening of approx. 20 m to the west, with maximum fill height of 10 m;
- A traffic surcharge of 17 kPa applied as a temporary load.
- The slip surface was forced to daylight at the edge of the outside traffic lane to eliminate non-consequential surficial failures.

The results of the stability analyses are provided on the figures presented in Appendix H. Each figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis.

The stability analyses generated the following factor of safety values for the critical embankment slope:

**Table 10-6 Slope Stability Analysis Results**

Condition	Case	Factor of Safety	
		SSM Slope	RSS Wall
Permanent (traffic loading)	Long-Term Static (Drained)	1.6 (Fig H1.1)	1.5 (Fig H2.1)
Temporary (traffic loading)	Short-Term Static (Undrained)	1.6 (Fig H1.2)	1.6 (Fig H2.2)
Temporary (seismic)	Pseudo-Static Seismic, 2,475yr (Undrained)	1.1 (Fig H1.3)	1.4 (Fig H2.3)
	Pseudo-Static Seismic, 475-yr (Undrained)	1.3 (Fig H1.4)	-

All of the static results presented in Appendix H achieve the target Factors of Safety described above. The pseudo-static result presented in Table 10-6 above exceeds the target Factor of Safety for seismic design.

All of the static results presented in Table 10-6 achieve the target Factors of Safety described above. The pseudo-static result considering the 2,475-year earthquake presented in Table 10-6 meets the target Factor of Safety of 1.1 for seismic design. However, it is noted that some

embankment displacement can occur where the pseudo-static Factor of Safety is less than 1.3 as is the case for the SSM slope during the 1 in 2,475 year seismic event (Figure H1.3). Additional analyses were carried out for that case to determine if performance criteria would be met for the Major Route geotechnical systems inside and outside the bridge interface zone. Pseudo-static analyses considering the 475-year earthquake event were completed and yielded a factor of safety of 1.3 for the west abutment at that location (Figure H1.4) indicating that the performance requirements would be met for that scenario.

## 10.7 Embankment Settlement

The settlement resulting from a 10 m high embankment constructed with conventional granular fill on a new alignment approximately 20 m to the west of the existing alignment was assessed using the multi-layer settlement analysis in Rocscience's Settle3 software. Subsurface stratigraphy was based on the boreholes drilled in the area. Loading was applied based on the assumed geometry of the new alignment using a unit weight for new embankment SSM fill of 20 kN/m<sup>3</sup>. The water table was defined based on piezometer readings. It is noted that results extracted from Geocres No. 31B00-034, engineering judgment and experience were used to select the material properties based on the stress range anticipated due to loading. Further consolidation testing will be required during detailed foundation investigations to confirm the assumed values and verify representative time-dependent consolidation parameters. The following discussion and recommendations must be carefully reviewed once site specific compression parameters are available. Soil parameters used in the analysis of the proposed new embankment are presented in Table 10-7 below.

**Table 10-7: Properties of Soil Used in Settlement Calculations**

Property	Embankment Fill	Upper Clay to Silty Clay to Clayey Silt	Lower Clay to Silty Clay to Clayey Silt
Unit Weight [kN/m <sup>3</sup> ]	19	17.5	17.5
E [kPa]	15,000	-	-
e <sub>o</sub>	-	1.1	1.1
P <sub>c</sub> ' [kPa]	-	190	280
C <sub>c</sub>	-	0.68	0.2
C <sub>r</sub>	-	0.05	0.05
C <sub>v</sub> (m <sup>2</sup> /s)	-	2.5e-06	2.5e-06
C <sub>vr</sub> (m <sup>2</sup> /s)	-	1.4e-05	1.4e-05

Based on the above parameters and loading from the assumed new embankment geometry, the expected total settlement is estimated to be up to 250 mm at the crest of the proposed embankment, just behind the abutment. The majority of the settlement is expected to be



completed within one year of fill placement based on time-dependent consolidation parameters through a literature review.

Based on the estimated settlement and time required for that settlement to occur, a preload period of at least 1 year is recommended prior to bridge construction. The embankment preload should be overbuilt to accommodate settlement during the preload period, the settlement should be monitored with monitoring points to confirm when the preload period is complete. Options such as wick drains to expedite the preload time period could be assessed during detailed design.

Provided the new embankment is constructed with a 1-year preload period prior to bridge construction, the approach embankments are expected to meet the MTO guidelines for settlement of approach embankments behind bridge abutments for post construction settlement over a period of 20 years after paving is outlined below:

- 25 mm within 20 m of the structure.
- 50 mm from 20 to 50 m from the structure.
- 75 mm from 50 to 75 m from the structure; and
- 100 mm greater than 75 m from the structure

When the preferred new alignment geometry is determined during detailed design, the impact of the new embankment settlement on the existing bridge foundations and lanes of Highway 16 and CNR railway line will need to be assessed. For preliminary purposes, it is estimated that the settlement generated by construction of the new embankment alignment will be up to 50 mm and 40 mm at the western crest of the existing embankment and below the existing embankment, respectively. This settlement is expected to occur during the preload period and construction staging will need to be planned accordingly. It is recommended to review the existing pile capacity of the current structure against downdrag as a result of the above-mentioned expected settlements at the existing embankment.

## **11 CONSTRUCTION CONSIDERATIONS**

### **11.1 Temporary Excavations**

Excavations at the abutments will extend through the existing embankment and highway fill. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, existing fill may be classified as Type 3 soil. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation.

At locations where there are space restrictions or where a slope must be retained, the excavations will need to be carried out within a protection system. For preliminary purposes, sheet pile will likely be suitable for this site. Further discussion on temporary protection systems (TPS) should be provided at a subsequent design stage, as required.



## 11.2 Surface and Groundwater Control

At the site, the natural groundwater level was measured (in a piezometer installed in Borehole 131-22-02 at the north abutment with screen in the clay) up to about Elevation 81.5 m, which is near the base of the existing embankments. Surface runoff will tend to seep into and accumulate in excavations. The Contractor must control groundwater, perched groundwater, and surface water flow at the site to permit foundation construction, subgrade preparation, and placement and compaction of granular bedding must be carried out in a dry and stable excavation.

Based on the anticipated depths of excavation required for pile caps, for temporary excavations it is considered likely that groundwater may be controlled with sump pumps in the bottom of the excavations.

## 12 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed replacement of the Highway 16 Overhead (Site No. 16X-0131/B0) as part of the overall Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 416 to Maitland Road. Additional foundation investigation will be required following the selection of the Technically Preferred Alternative (TPA). Additional field investigation should be carried out to provide additional foundation design input to the following:

- Shear wave velocity measurements below the foundation elements to confirm Seismic Site Classification. Seismic CPTu holes with periodic dissipation testing are recommended due to the sensitive nature of the silty clay to clayey silt layer and the requirement for a site-specific seismic assessment.
- Testing of soil and/or groundwater at the site to determine degree of corrosiveness of the sub-surface environment and potential implications to steel and concrete elements in contact with the soil and groundwater at the site.
- Additional field and laboratory testing to better define compressibility characteristics of the cohesive soils beneath the abutments to allow estimation of settlement and downdrag loads on abutment piles. Samples should be collected with thin-walled tubes. Consolidation testing should be completed.
- The RSS factored geotechnical resistance at SLS will be significantly influenced by the construction sequencing and the consolidation settlement of the compressible silty clay to clayey silt under the embankment loading. Further analysis should be carried out once the cohesive soils are better defined through additional consolidation testing.

The required supplementary foundation field investigation scope should be reviewed following the selection of the TPA.



### 13 CLOSURE

Engineering analysis and preparation of this report was carried out by Katya Walker, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.  
Report Prepared By:

  
for Katya Walker, M.Eng., P.Eng.  
Geotechnical Engineer



Paul Carnaffan, M.Eng., P.Eng.  
Partner | Branch Manager  
Senior Geotechnical Engineer



Matt Kennedy, M.Sc. (Eng)., P.Eng.  
Senior Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.  
Designated Principal Contact  
Senior Geotechnical Engineer



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- <sup>iii</sup> <http://www.geologyontario.mndm.gov.on.ca/index.html>
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- <sup>vi</sup> Bray, J. D. et al. (2004b). Liquefaction susceptibility of fine-grained soils. Proc., 11th Int. Conf. on Soil Dynamics and Earthquake Engineering and 3rd Int. Conf. on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Singapore, 655–662



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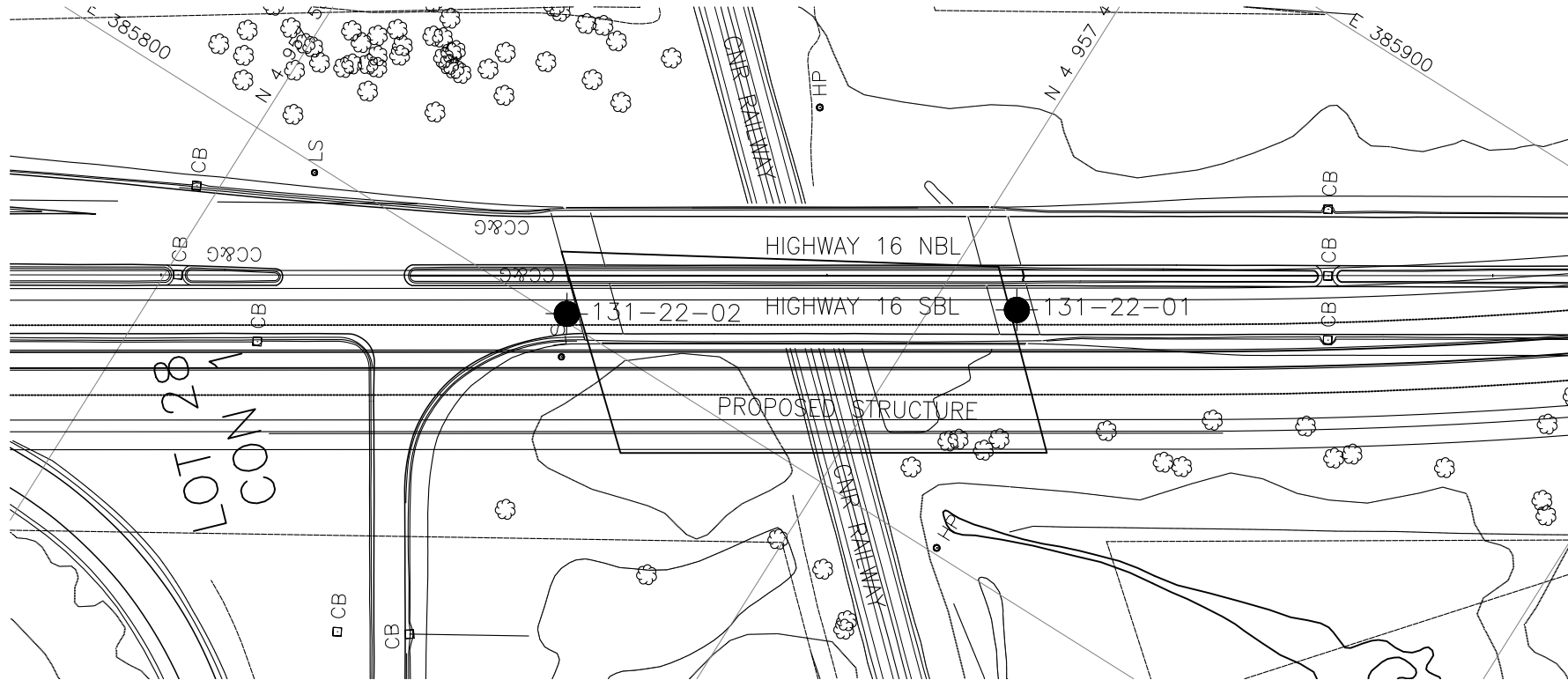
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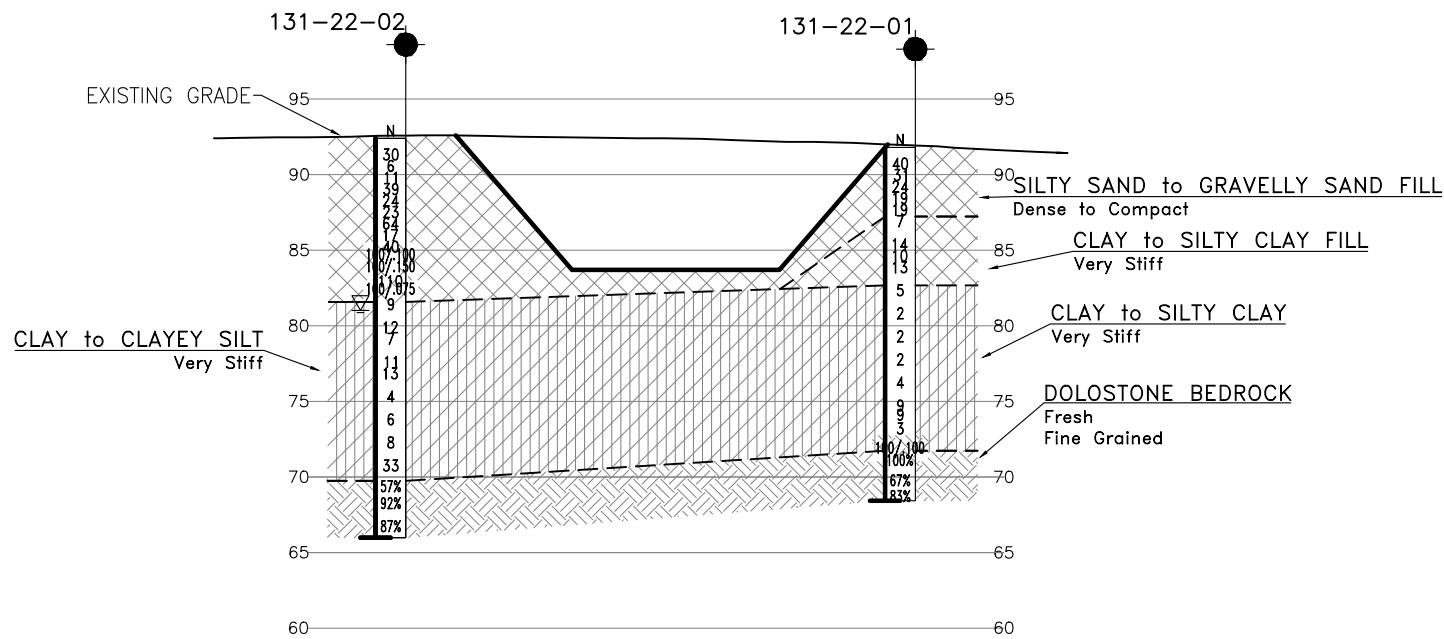


## **Appendix A.**

### **Borehole Location Plan and Stratigraphic Drawing**



PLAN  
SCALE 1:1000



PROFILE ALONG  $\phi$  HIGHWAY 16  
H 1:1000  
V 1:500

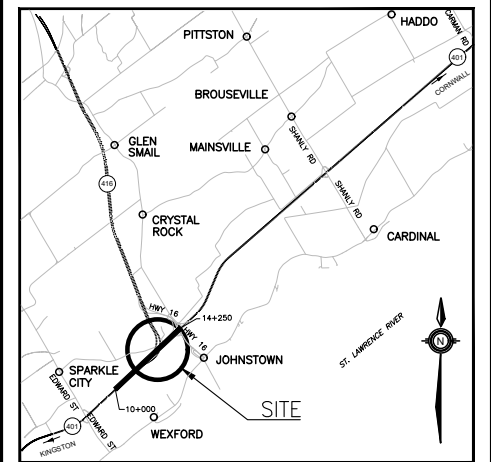
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 4024-20-00

HIGHWAY 16  
CNR OVERPASS  
REHABILITATION  
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
131-22-01	91.8	4 957 388.9	385 837.4
131-22-02	92.4	4 957 445.7	385 801.1

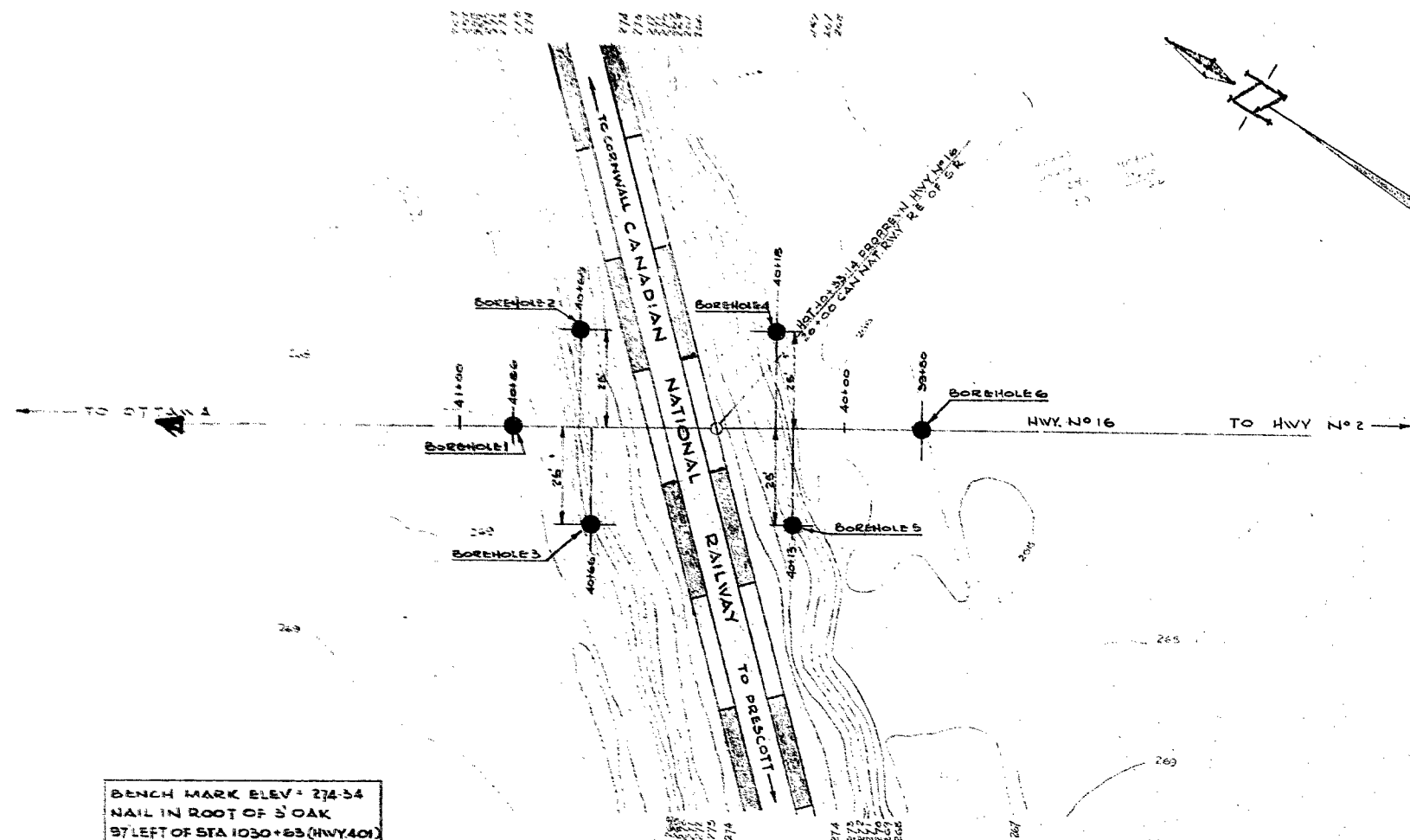
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 9.

GEOCRES No. 31B12-003

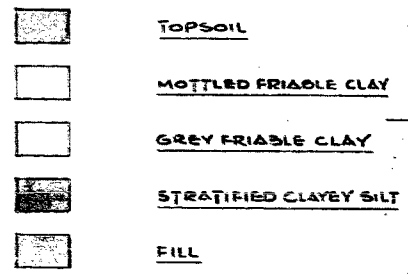


REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AO	CHK MK	CODE
DRAWN	JW	CHK AO	SITE
LOAD		STRUCT	
DATE	MARCH 2024	DWG	1

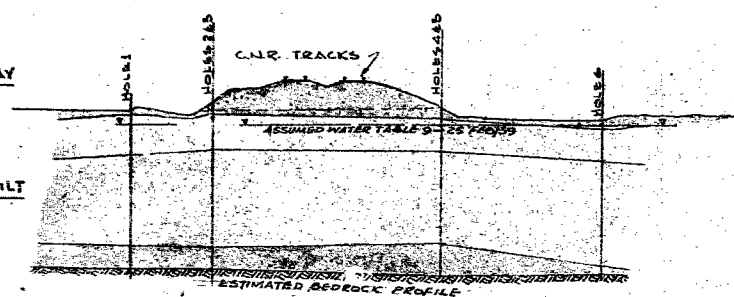


PLAN of SITE  
SCALE 20:1

COLOUR KEY



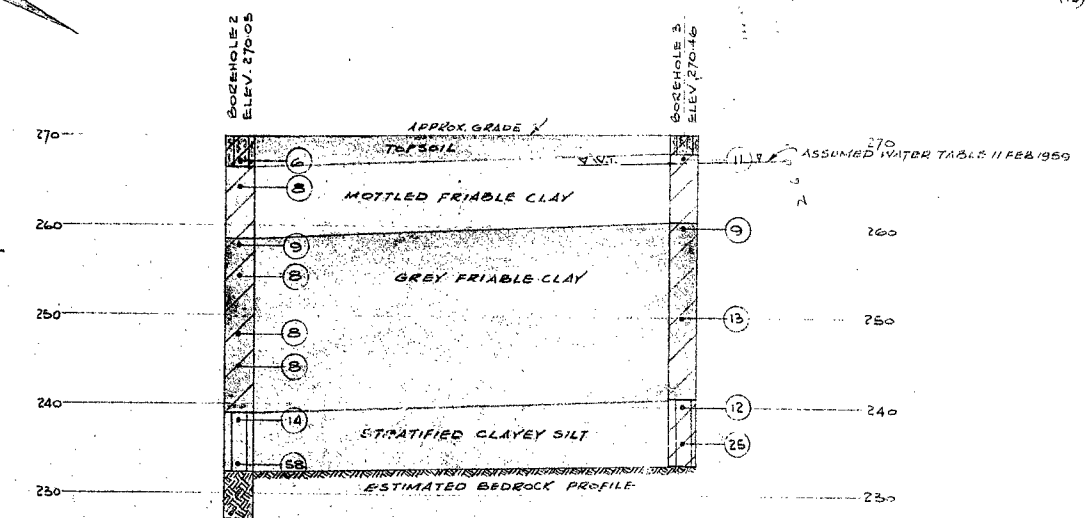
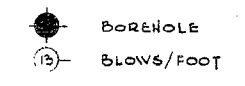
NOTE: HOLES 2&3 AND 4&5 HAVE BEEN INTERPOLATED TO PRODUCE THESE RESULTS



SECTION ALONG LINE HWY 16 SHOWING STRATIFICATION

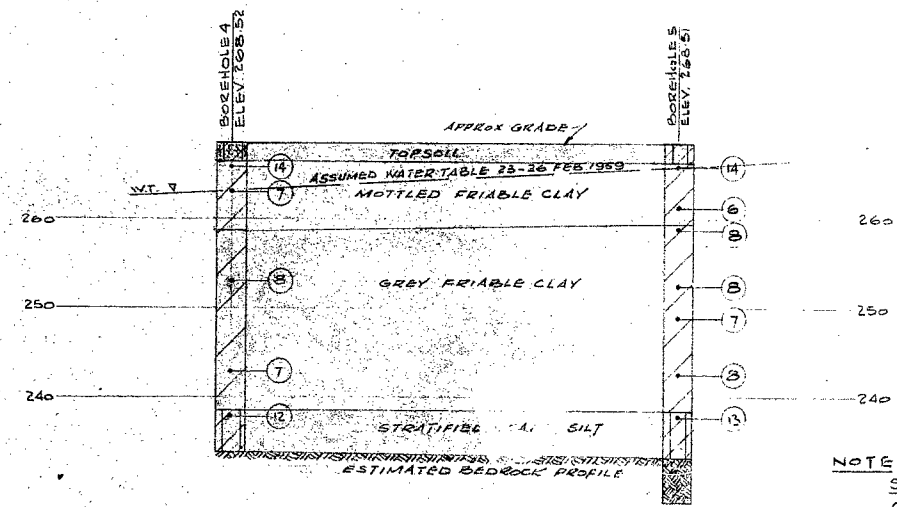
PROFILE SCALES  
VERT 20:1  
HOR 20:1

LEGEND



SECTION THROUGH HOLES 2&3

PROFILE SCALE  
VERT 10:1  
HOR 10:1



SECTION THROUGH HOLES 4&5

PROFILE SCALE  
VERT 10:1  
HOR 10:1

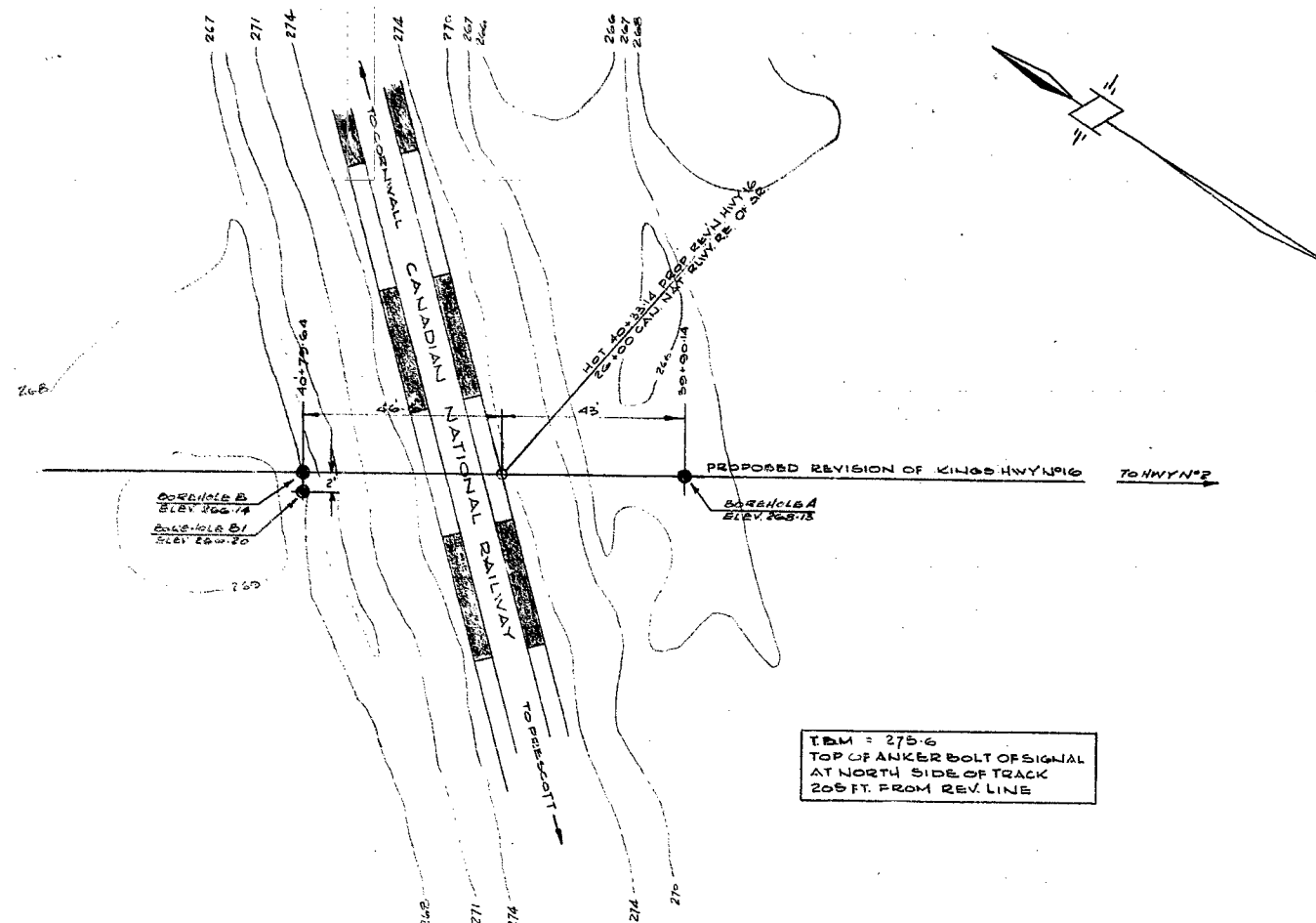
NOTE: SEE BOREHOLE LOGS FOR COMPLETE SOIL DATA



e.m. peto & associates ltd.

SOIL SITE INVESTIGATION  
AT  
C.N.R. OVERHEAD HWY 16  
EDWARDSBURG TWP JOHNSTOWN  
FOR  
DEPT. OF HIGHWAYS OF ONTARIO  
OUR JOB NO. 5920 DATE 5 MAR 1959  
CLIENTS PLAN NO. E3554-1 PER. C.J.W.

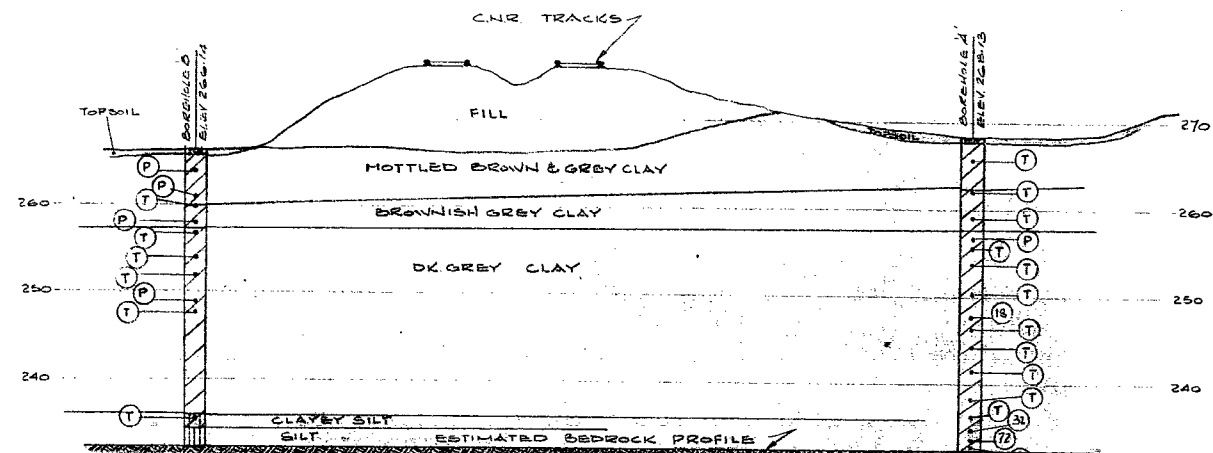
TO OTTAWA



- LEGEND**
- BOREHOLE
  - (B) BLOWG/FOOT
  - (T) TAPPED OR PUSHED

TBM = 275.6  
TOP OF ANKER BOLT OF SIGNAL  
AT NORTH SIDE OF TRACK  
205 FT. FROM REV. LINE

**SITE PLAN**  
SCALE 20:1



**SECTION ALONG C/L OF PROPOSED HWY.**  
SECTION SCALES VERT. 10:1  
HOR. 10:1

**NOTE:**  
SEE BOREHOLE LOGS FOR COMPLETE  
SOIL DETAILS  
STRATIFICATION OF SOIL BETWEEN  
BOREHOLES HAS BEEN ESTIMATED  
AND MAY DIFFER FROM THAT SHOWN



**e.m. peto & associates Ltd.**  
SOIL SITE INVESTIGATION  
AT  
C.N.R. OVERHEAD HWY 16  
EDWARDSBURG TWP. JOHNSTOWN  
FOR  
DEPT OF HIGHWAYS OF ONTARIO  
OUR FILE NO. 59129 DATE 14 JULY/59  
CLIENTS PLAN No. E3554-1 PLOT C J.V.



## **Appendix B.**

### **Record of Borehole Sheets**



## SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

### TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

### TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

### RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

### N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

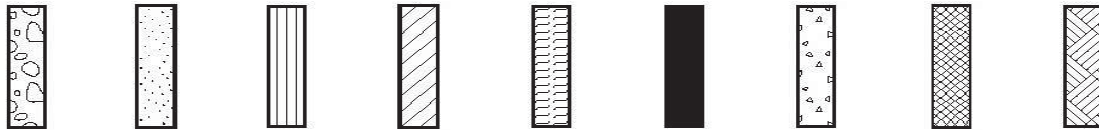
### DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



### STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel      Sand      Silt      Clay      Organics      Asphalt      Concrete      Fill      Bedrock

### TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

### TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

### SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

### TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50



### MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note -  $W_L$  = Liquid Limit



## EXPLANATION OF ROCK LOGGING TERMS

### ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

### TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

### DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 to 2 m
Medium bedded	0.2 to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 to 60 mm
Laminated	6 to 20 mm
Thinly laminated	Less than 6 mm

### STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

# RECORD OF BOREHOLE No 131-22-01

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.751917°, Long: -75.476522° Highway 16 & Railway, Edwardsburgh, MTM z9: N 4 957 388.9 E 385 837.4 ORIGINATED BY IK  
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO  
 DATUM Geodetic DATE 2022.12.05 - 2022.12.06 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
91.8	Ground Surface						20	40	60	80	100	20	40	60						
0.0 91.5	ASPHALT (255 mm)																			
91.3 0.5	GRAVELLY SAND, some fines Brown FILL (BASE)																			
	SILTY SAND, some gravel Dense to compact Brown FILL		1	SS	40								○							
			2	SS	31								○							
			3	SS	24								○			11 53 36 (SI+CL)				
			4	SS	19								○							
			5	SS	19								○							
87.2 4.6	CLAY to SILTY CLAY Very stiff Brown to grey FILL		6	SS	7								○							
			7	SS	14								○			0 9 40 51				
			8	SS	10								○							
			9	SS	13								○							
82.7 9.1	CLAY (CH) to SILTY CLAY (CI) Very stiff Brown to grey		10	SS	5								○							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE


DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 3-19-24

RECORD OF BOREHOLE No 131-22-01

2 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.751917°, Long: -75.476522° Highway 16 & Railway, Edwardsburgh, MTM z9: N 4 957 388.9 E 385 837.4 ORIGINATED BY IK  
HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO  
DATUM Geodetic DATE 2022.12.05 - 2022.12.06 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE						
Continued From Previous Page								20 40 60 80 100	20 40 60					
	CLAY (CH) to SILTY CLAY (CI) Very stiff Brown to grey							>102 kPa +						
			11	SS	2									
									>102 kPa +					
									>102 kPa +					
			12	SS	2									0 0 40 60
									>102 kPa +					
									>102 kPa +					
			13	SS	2									
									>102 kPa +					
									>102 kPa +					
			14	SS	4									
									>102 kPa +					
			15	SS	9									
			16	SS	9							0 4 48 48		
			17	SS	3									
								>102 kPa +						
								>102 kPa +						
			18	SS	100/									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20  
15 10 5 10 (%) STRAIN AT FAILURE


DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 3-19-24

RECORD OF BOREHOLE No 131-22-01

3 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.751917°, Long: -75.476522°  
Highway 16 & Railway, Edwardsburgh, MTM z9: N 4 957 388.9 E 385 837.4 ORIGINATED BY IK  
HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO  
DATUM Geodetic DATE 2022.12.05 - 2022.12.06 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											
								20 40 60 80 100											
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
Continued From Previous Page							20 40 60 80 100						WATER CONTENT (%)						
71.7 20.1	<b>DOLOSTONE BEDROCK</b> Fresh Fine grained Grey Medium bedded Very strong		1	RUN	-		71										1 1 1 0 1 1 7 4 1 2 2	RUN #1 TCR=100% SCR=98% RQD=100%  RUN #2 TCR=100% SCR=79% RQD=67%  RUN #3 TCR=100% SCR=83% RQD=83%	
68.4 23.4	<b>End of Borehole</b>  Unstabilized water level at a depth of 3.7 m (elev. 88.1 m) in the open borehole. Water was introduced in the borehole during drilling.																		









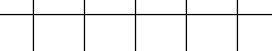
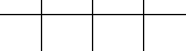





DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 3-19-24

# RECORD OF BOREHOLE No 131-22-02

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.752432°, Long: -75.476972° Highway 16 & Railway, Edwardsburgh, MTM z9: N 4 957 445.7 E 385 801.1 ORIGINATED BY IK  
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / NW Casing / NQ Coring COMPILED BY AO  
 DATUM Geodetic DATE 2022.12.01 - 2022.12.05 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
92.4	Ground Surface		1	NQ	-										
0.0	ASPHALT (100 mm)														
92.1	CONCRETE (200 mm)														
0.3	GRAVELLY SAND, some fines Dense Brown FILL (BASE/SUBBASE)		1	SS	30										
90.9															
1.5	SILTY SAND, some gravel Loose to very dense Brown FILL		2	SS	6										
			3	SS	11										
			4	SS	39										
			5	SS	24										
			6	SS	23										
			7	SS	64										
			8	SS	17										
			9	SS	40										
84.8															
7.6	GRAVELLY SILTY SAND Contains cobbles and boulders Very dense Brown FILL		10	SS	100/ 100mm										
			11	SS	100/ 150mm										
			12	SS	110										

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

**METRIC**[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 3-19-24

## METRIC

Lat: 44.752432°, Long: -75.476972°  
Highway 16 & Railway, Edwardsburgh, MTM z9: N 4 957 445.7 E 385 801.1

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

























DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 3-19-24



# BOREHOLE LOG

Checked By ..... **E.M.P.**

W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
BLACK SANDY SILT LOAM TOP SOIL WITH ROOTS AND ORGANIC MATTER			268.14 1' 0"		1 	FROM CASING		SATURATED
MOTTLED, FRIABLE CLAY	BROWN & GREY	STIFF 3' 8" 264.47	267.14 5' 0"		2 	S.S. H	11	M.C.=40.2% MUCH WETTER THAN PLASTIC LIMIT FEBRUARY 11, 1959 WATER FILLED HOLE OVERNIGHT
AS ABOVE	AS ABOVE	STIFF			3 	S.S.	9	M.C.=36.0% W.T.P.L.
			10' 0"					
FRIABLE CLAY	DK. GREY		258.14		4 	S.L. TAPPED		M.C.=35.5% ; S <sub>r</sub> =100% C=984 TSF ; e=.964
			15' 0"					
AS ABOVE, OCCASIONAL PEBBLES	AS ABOVE	FIRM TO STIFF			5 	S.S.	8	M.C.=39.4% M.W.T.P.L.
			20' 0"					
AS ABOVE, GRITS	AS ABOVE				6 	S.L. TAPPED		M.C.=31.7% ; C=590 TSF e=.877
			25' 0"					
AS ABOVE	AS ABOVE	FIRM TO STIFF			7 	S.S.	8	M.C.=32.0% M.W.T.P.L.
			30' 0"					
CLAYEY SILT WITH THIN SEAMS OF VERY FINE SAND (SLIGHTLY PLASTIC)	DK. GREY		238.14		8 	S.L. TAPPED		WATER SEEPAGE AT 30' 0" M.C.=22.1% W.T.P.L.
								
AS ABOVE	AS ABOVE	HARD	35' 9" 232.39		9 	S.S.	100	M.C.=15.1% DRIER THAN PLASTIC LIMIT
				REFUSAL				NOTE: HOLE SAMPLED DRY

# BOREHOLE LOG

Datum Geodetic Compiled By K.P.

Checked By E. M. P.

### ABBREVIATIONS

 UNDISTURBED

 FAIR

☒ DISTURBED

**LOST**

S.S. 2" STANDARD SPLIT TUBE SAMPLE

S. L. SPLIT BARREL WITH LINERS

5. T. THIN-WALLED SHELBY TUBE SAMPLE

W.S. WASH SAMPLE

R. C. ROCK CORE

### V. T. IN SITU VANE SHEAR TEST

Q/u UNCONFINED COMPRESSIVE STRENGTH

W.L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL





SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
ORGANIC TOPSOIL WITH ROOTS & STONES	BLACK		270-05 2'0"	1	X FROM CASING			SATURATED
SANDY SILT WITH PEBBLES, ROOTS, ORG. MATTER	DK.BROWN	FIRM	268-05 3'4" 266-92 5'0"	2	X S.S.	6		SATURATED
MOTTLED, FRIABLE CLAY OCCASIONAL PEBBLES	BROWN & GREY	FIRM TO STIFF		3	X S.S.	8		M.C.=44.4% MUCH WETTER THAN PLASTIC LIMIT
			10'0"					
FRIABLE CLAY	DK.GREY	STIFF	11'5" 258-53	4 5	S.L. X S.S.	TAPPED 9		SAMPLE LOST M.C.=34.5% M.W.T.P.L. e=1.042
			15'0"					
AS ABOVE, SOME ORGANIC MATTER	AS ABOVE	FIRM TO STIFF		6	X S.S.	8		M.C.=36.6% M.W.T.P.L.
			20'0"					
AS ABOVE, FEW SMALL STONES	AS ABOVE			7	X S.L.	TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF		8	X S.S.	8		W.T.P.L.,
			25'0"					
AS ABOVE	AS A	FIRM TO STIFF		9	X S.S.	8		M.C.=21.7% W.T.P.L.
AS ABOVE, GRITS STRATIFIED CLAYEY SILT	AS ABOVE DK.GREY	STIFF	31'0" 239-05	10 11	X S.L. X S.S.	TAPPED 14		M.C.=24.6% M.W.T.P.L.
			35'0"					
AS ABOVE	AS ABOVE	HARD		12	X S.S.	58		M.C.=17.5% W.T.P.L.
			37'9" 232-30					
			40'0"		R.C.			RECOVERY 98%
LIMESTONE	GREY		42'11" 227-13					NOTE: HOLE DRIVEN WITH THE USE OF WASH WATER
			HOLE TERMINATED					

**e. m. peto & associates ltd.**  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
**BOREHOLE LOG**

Job Name Edwardsburg Twp.C.N.R. Overhead  
Hwy. 16 Johnstown. Job No. 5920  
Client Dep't. of Highways of Ontario Casing BX (2 1/2" Dia.)  
Datum Geodetic. Compiled By K.P.R.

Borehole No. 3  
Boring Date Feb. 12th. - 13th. 1959.  
Checked By E.M.P.

**SAMPLE CONDITION**










 UNDISTURBED  
 FAIR  
 DISTURBED  
 LOST

**SAMPLE TYPE**

S.S. 2" STANDARD SPLIT TUBE SAMPLE  
S.L. SPLIT BARREL WITH LINERS  
S.T. THIN-WALLED SHELBY TUBE SAMPLE  
W.S. WASH SAMPLE  
R.C. ROCK CORE

**ABBREVIATIONS**





V. T. IN SITU VANE SHEAR TEST  
Q/u UNCONFINED COMPRESSIVE STRENGTH  
W.L. WATER LEVEL IN CASING  
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
ORGANIC TOPSOIL WITH ROOTS	BLACK		270'46		1	FROM CASING		VERY WET
MOTTLED, FRIABLE CLAY (FEW ROOTS)	BROWN & GREY	STIFF	3'0" 268'46		2	S.S.	W.T. 11	M.C.=38.2% MUCH WETTER THAN PLASTIC LIMIT e=.931
AS ABOVE	AS ABOVE		5'0"		3	S.L. TAPPED		FEBRUARY 13-14, 1959 WATER FILLED HOLE OVERNIGHT
FRIABLE CLAY	DK. GREY	STIFF	10'0" 260'46		4	S.S.	9	M.C.=33.5% M.W.T.P.L.
AS ABOVE	AS ABOVE		15'0"		5	S.L. TAPPED		M.C.=40.5% M.W.T.P.L. e=1.043
AS ABOVE	AS ABOVE	STIFF	20'0"		6	S.S.	13	M.C.=27.1% W.T.P.L.
AS ABOVE, FEW SMALL STONES	AS ABOVE		25'0"		7	S.L. TAPPED		WATER SEEPAGE AT 25' M.C.=37.7% M.W.T.P.L. e=.915
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	29'6" 240'96		8	S.S.	12	M.C.=24.4% M.W.T.P.L.
AS ABOVE	AS ABOVE	VERY STIFF	35'0"		9	S.S.	25	M.C.=19.7% W.T.P.L.
			37'0" 233'46					NOTE: HOLE SAMPLED DRY

# BOREHOLE LOG

Checked By ..... **E.M.P.**

### ABBREVIATIONS

	UNDISTURBED
	FAIR
	DISTURBED
	LOST

S. S. 2" STANDARD SPLIT TUBE SAMPLE  
S. L. SPLIT BARREL WITH LINERS  
S. T. THIN-WALLED SHELBY TUBE SAMPLE  
W. S. WASH SAMPLE  
R. C. ROCK CORE

V. T. IN SITU VANE SHEAR TEST  
Q/u UNCONFINED COMPRESSIVE STRENGTH  
W. L. WATER LEVEL IN CASING  
W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
ORGANIC TOPSOIL & SOME COARSE SAND	BLACK		268'52"	1	FROM CASING			SATURATED
		STIFF	2' 0" 266'52"	2	S.S.	14		W.T. WITH CASING AT 5'
MOTTLED, FRIABLE CLAY	GREY & BROWN		5' 0" 263'02"	2/A	AUGER			
AS ABOVE	AS ABOVE	FIRM	5' 6" 263'02"	3	S.S.	7		FEBRUARY 26, 1959. W.T. WATER LEVEL WITH CASING AT 20' M.C. = 39.4% MUCH WETTER THAN PLASTIC LIMIT
			9' 8" 258'85"	4	S.L.	TAPPED		
FRIABLE CLAY	DK. GREY							
			15' 0"					
AS ABOVE	AS ABOVE	FIRM TO STIFF		5	S.S.	8		M.C. = 33.4% M.W.T. P.L.
			20' 0"					
AS ABOVE	AS ABOVE			6	S.L.	TAPPED		
			25' 0"					
AS ABOVE	AS ABOVE	FIRM		7	S.S.	7		M.C. = 29.1% W.T. P.L.
			30' 0"					
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	238'52"	8	S.S.	12		M.C. = 19.2% W.T. P.L.
				9	S.L.	TAPPED		
			34' 8" 233'85"					
					REFUSAL			NOTE: HOLE SAMPLED DRY





# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Edwardsburg Twp. C.N.R. Overhead  
 Job Name Hwy. 16. Johnstown. Job No. 5920 Borehole No. 5  
 Client Dep't. of Highways of Ontario. Casing 4" Pipe Boring Date Feb. 19th. - 24th. 1959.  
 Datum Geodetic. Compiled By K. P. Checked By E. M. P.

### SAMPLE CONDITION



-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

### SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

### ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL





SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
ORGANIC TOPSOIL, FEW STONES	BLACK		268'51"		1	FROM CASING		WATER SEEPAGE IN TOPSOIL
MOTTLED FRIABLE CLAY	BROWN & GREY	2 W.T. 2'-5" STIFF	2' 0" 266'51"		2	S.S.	14	M.C. = 43.1% WATER TABLE 23-24 FEB.
AS ABOVE	AS ABOVE		5' 0"		3	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM			4	S.S.	6	
FRIABLE CLAY	DK. GREY	FIRM TO STIFF	9' 0" 259'51"		5	S.S.	8	M.C. = 30.9% MUCH WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE		15' 0"		6	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF			7	S.S.	8	
AS ABOVE, SEAMS OF SILT	AS ABOVE	FIRM	20' 0"		8	S.S.	7	M.C. = 26.2% W.T. P.L.
AS ABOVE	AS ABOVE		25' 0"		9	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF			10	S.S.	8	
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	30' 0" 238'51"		11	S.S.	13	M.C. = 21.3% W.T. P.L.
LIMESTONE	GREY		35' 2" 233'34"			R.C.		RECOVERY 98%
			40' 2" 228'34"					HOLE TERMINATED

**e. m. peto associates ltd.**  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
**BOREHOLE LOG**

Edwardsburg Twp. C.N.R. Overhead  
Job Name Hwy. 16 Johnstown Job No. 5020  
Client Dep't. of Highways of Ontario Casing BX (2 1/2" Dia.)  
Datum Geodetic Compiled By K.P.

Borehole No. 10  
Boring Date Feb. 17th. - 18th. 1959.  
Checked By E.M.P.

**SAMPLE CONDITION**

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

**SAMPLE TYPE**

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

**ABBREVIATIONS**

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
SANDY SILT TOPSOIL, ROOTS, STONES, ORG. MATTER	BLACK	W.T. 1'-0"	266'32"		1	FROM CASING		19 FEB
MOTTLED, FRIABLE CLAY	BROWN & GREY	FIRM	264'65"		2	S.S.	7	M.C. = 46.3% MUCH WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE	FIRM	5'0"		3	S.S.	7	M.C. = 44.8% M.W.T.P.L.
			9'0"					
			257'32"					
FRIABLE CLAY	DK. GREY				4	S.L. TAPPED		
			15'0"					
AS ABOVE	AS ABOVE	FIRM TO STIFF			5	S.S.	8	M.C. = 33.5% M.W.T.P.L.
			20'0"					
AS ABOVE	AS ABOVE				6	S.L. TAPPED		M.C. = 30.6% M.W.T.P.L.
			25'0"					
AS ABOVE	AS ABOVE	STIFF			7	S.S.	12	M.C. = 28.5% W.T.P.L.
			30'0"					
AS ABOVE	AS ABOVE				8	S.L. TAPPED		
			32'8"					
STRATIFIED CLAYEY SILT	DK. GREY	HARD	233'65" 34'3" 232'07"		9	S.S.	60	M.C. = 19.2% W.T.P.L.
						REFUSAL		



# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Hwy.# 16 & C.N.R. Crossing  
 Job Name Edwardsburg Twp. Job No. 59129 Borehole No. "A"  
 Client Dep't. of Highways of Ontario. Casing 4" Pipe Boring Date July 10th. 1959.  
 Datum Geodetic Compiled By B.L. Checked By

### SAMPLE CONDITION

UNDISTURBED  
 FAIR  
 DISTURBED  
 LOST

### SAMPLE TYPE

A.S. AUGER SAMPLE  
 C.S. CASING SAMPLE  
 S.S. 2" STANDARD SPLIT TUBE SAMPLE  
 S.L. SPLIT BARREL WITH LINERS  
 S.T. THIN-WALLED SHELBY TUBE SAMPLE  
 W.S. WASH SAMPLE  
 R.C. ROCK CORE

### ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST  
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.  
 W.L. WATER LEVEL IN CASING  
 W.T. GROUND WATER TABLE IN SOIL  
 W.T.P.L. WETTER THAN PLASTIC LIMIT  
 D.T.P.L. DRIER THAN PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
Ground surface.			0' 0"						
Topsoil	Dk. Brown to Black		0' 6"						
Clay	Mottled brown & grey		5' 0"			3"S.L. Tapped			
			6' 2"			V.T. V.T.			
Clay	Brownish-grey		10' 0"			2"S.L. Tapped	31.1%		
Clay	Dk. Grey		10' 4"			V.T. 3"S.L. Tapped	38.4%		
At 12'6" seam (2" thick) of grey fine silty sand.						V.T. 2"S.L. Pushed	35.9%		
At 13'6"-2" thick seam of grey silt.			15' 0"			V.T. Tapped	43.1%		
Clay	Dk. Grey					2"S.L. Tapped	29.2%		
At 19'10" seam of gravel & large stones. (Boulder?)			20' 0"			3"S.T. 18			
						2"S.L. Tapped	24.4%		
			25' 0"			3"S.L. Tapped	42.7%		
Clay	Dark grey					2"S.L. Tapped	24.1%		
			30' 0"			3"S.L. Tapped	20.2%		
						2"S.L. Tapped			From 33'0" Dutch Cone penetration test.
			35' 0"				32		
Refusal (Bedrock)			36' 0"				72		
							112		





Hole terminated at 36'0"

### WATER LEVEL READINGS.

July 10th. 1959.	Time	Depth of Casing	Depth of Hole	Depth of Water	
July 10th. 1959.	9.30 am.	2'3"	5'0"	4'10"	
	11.20 am.	5'0"	10'9"	3'11"	
	12.10 pm.	10'0"	13'6"	None	After Bailing Out.
	12.30 pm.	10'0"	15'7"	15'2"	
	1.15 pm.	10'0"	15'7"	12'4"	
	4.50 pm.	20'0"	26'0"	8'7"	Note: Water came up very fast from a depth of 20'0" where the seam of gravel and stones was encountered/
	4.55 pm.	20'0"	26'0"	7'10"	
	5.00 pm.	20'0"	26'0"	7'10"	
	6.45 pm.	29'0"	32'10"	24'4"	After bailing out.
July 11th. 1959.	6.45 am.	29'0"	36'0"	2'2"	
	8.05 am.	None	36'0"	4'1"	
	10.35 am.	None	36'0"	2'5"	

e. m. peto associates ltd.  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
BOREHOLE LOG

Hwy.# 16 & C.N.R.Crossing  
Job Name Edwardsburg Twp. Job No. 59129 Borehole No. "B"  
Client Dep't.of Highways of Ontario. Casing 4" Pipe Boring Date July 7th.& 8th. 1959.  
Datum Geodetic. Compiled By B.L. Checked By

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	A.S.	AUGER SAMPLE	V.T.	IN SITU VANE SHEAR TEST
	FAIR	C.S.	CASING SAMPLE	C.	SOIL SHEAR STRENGTH LBS/SQ.FT.
	DISTURBED	S.S.	2" STANDARD SPLIT TUBESAMPLE	W.L.	WATER LEVEL IN CASING
	LOST	S.L.	SPLIT BARREL WITH LINERS	W.T.	GROUND WATER TABLE IN SOIL
		S.T.	THIN-WALLED SHELBY TUBE SAMPLE	W.T.P.L.	WETTER THAN PLASTIC LIMIT
		W.S.	WASH SAMPLE	D.T.P.L.	DRIER THAN PLASTIC LIMIT
		R.C.	ROCK CORE		

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
Ground surface			0'0"						
Topsoil	Dr. Brown to Black	266.14	0'0"						
Clay	Mottled brown & grey		5'0"			3"S.L. Pushed 18.4%			
			6' 3"			V.T. Pushed 38.9%			
Clay	Brownish-grey		10'0"			2"S.L. Tapped			
			15'0"			V.T. Tapped			
			20'0"			3"S.L. Pushed 38.1%			
			25'0"			V.T. Tapped			
			30'3"			2"S.L. Tapped 34.8%			
			31'10"			V.T. Tapped 33.1%			
Clay	Dk. Grey		34'1"			V.T. Pushed 32.3%			
						V.T. Tapped			
									Note: No sampling was performed from 19'11" to 30'0" depth as the sub-soil was disturbed by vane which broke down at 19'11"
Clayey silt	Grey		30'3"			2"S.L. Tapped 18.9%			
Silt	Grey		31'10"			3"S.L. 19.3%			Seepage observed at 32'0" depth.
Refusal (Bedrock)			34'1"						

Hole terminated at 34'1"				
WATER LEVEL READINGS.				
DATE	TIME	DEPTH OF CASING	DEPTH OF HOLE	DEPTH OF WATER
July 7th. 1959.	1.10 pm.	5'0"	8'3"	6'2"
	1.13 pm.	5'0"	8'3"	5'4"
	1.40 pm.	5'0"	8'3"	0'10"
	1.45 pm.	5'0"	8'3"	0' 7"
	1.50 pm.	5'0"	8'3"	0' 5"
	3.35 pm.	5'0"	10'10"	0' 4"
	3.45 pm.	10'0"	11' 0"	None
	3.55 pm.	10'0"	13'11"	None
	4.05 pm.	10'0"	15'4"	None
	4.50 pm.	10'0"	15'4½"	14'11½"
	5.10 pm.	10'0"	17' 0"	None
	7.55 am.	15' 0"	17'0"	16' 9"
	11.05 am.	15'0"	25' 0"	13' 5"
	11.10 am.	15'0"	25'0"	12'0½"
	11.15 am.	15'0"	25'0"	11'1½"
	11.19 am.	15'0"	25'0"	10' 6"
				3.40pm. 30'0" 32'9" 31'0"
				5.00pm. None 32'9" 27'4"
	11.31 am.	15'0"	25'0"	8' 8"
	1.15 pm.	30'0"	27'0"	26'11"
	3.20 pm.			



# BOREHOLE LOG

Checked By .....

V.T.	IN SITU VANE SHEAR TEST
C.	SOIL SHEAR STRENGTH LBS/SQ.FT.
W.L.	WATER LEVEL IN CASING
W.T.	GROUND WATER TABLE IN SOIL
W.T.P.L.	WETTER THAN PLASTIC LIMIT
D.T.P.L.	DRIER THAN PLASTIC LIMIT

[illegible]



## **Appendix C.**

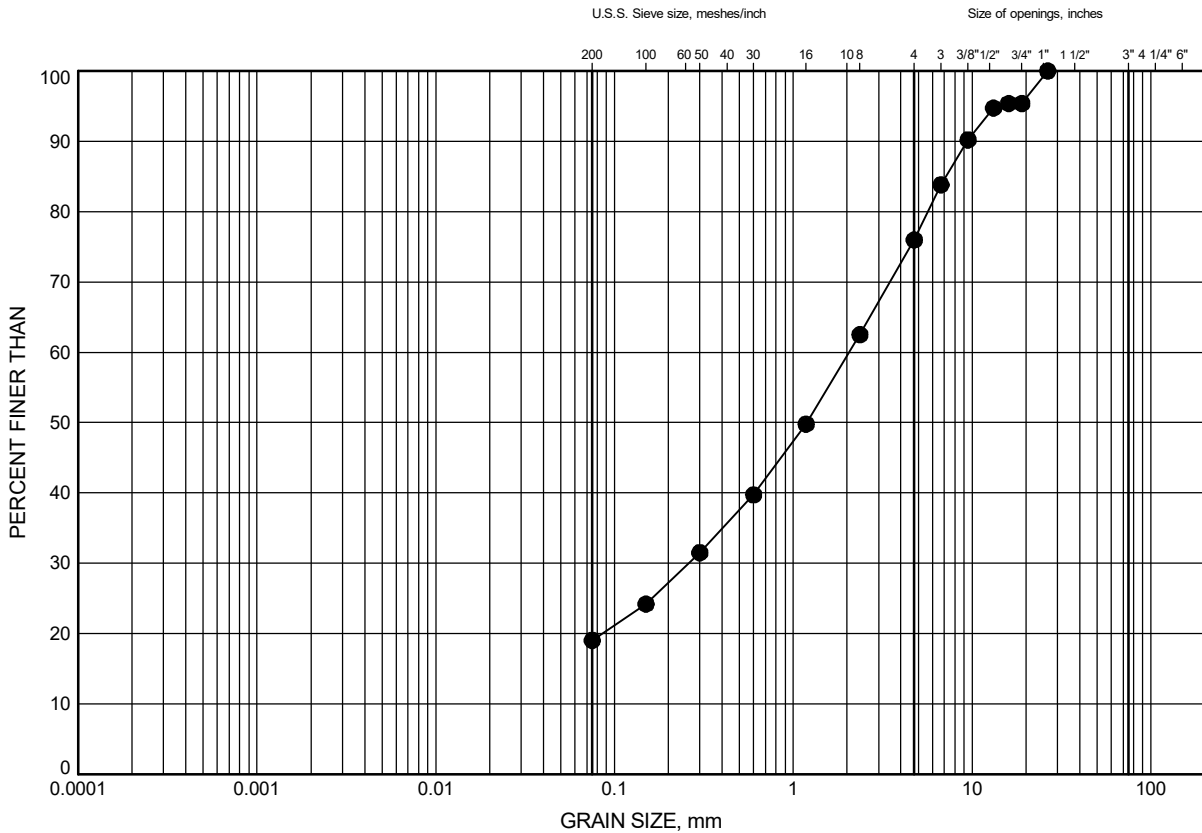
### **Laboratory Testing**



**Appendix C.1**  
**Particle Size Analysis Figures**  
**Atterberg Limit Test Results**

# GRAIN SIZE DISTRIBUTION

FILL: Gravelly Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-02	1.1	91.3

Date March 2023

GWP# 4024-20-00

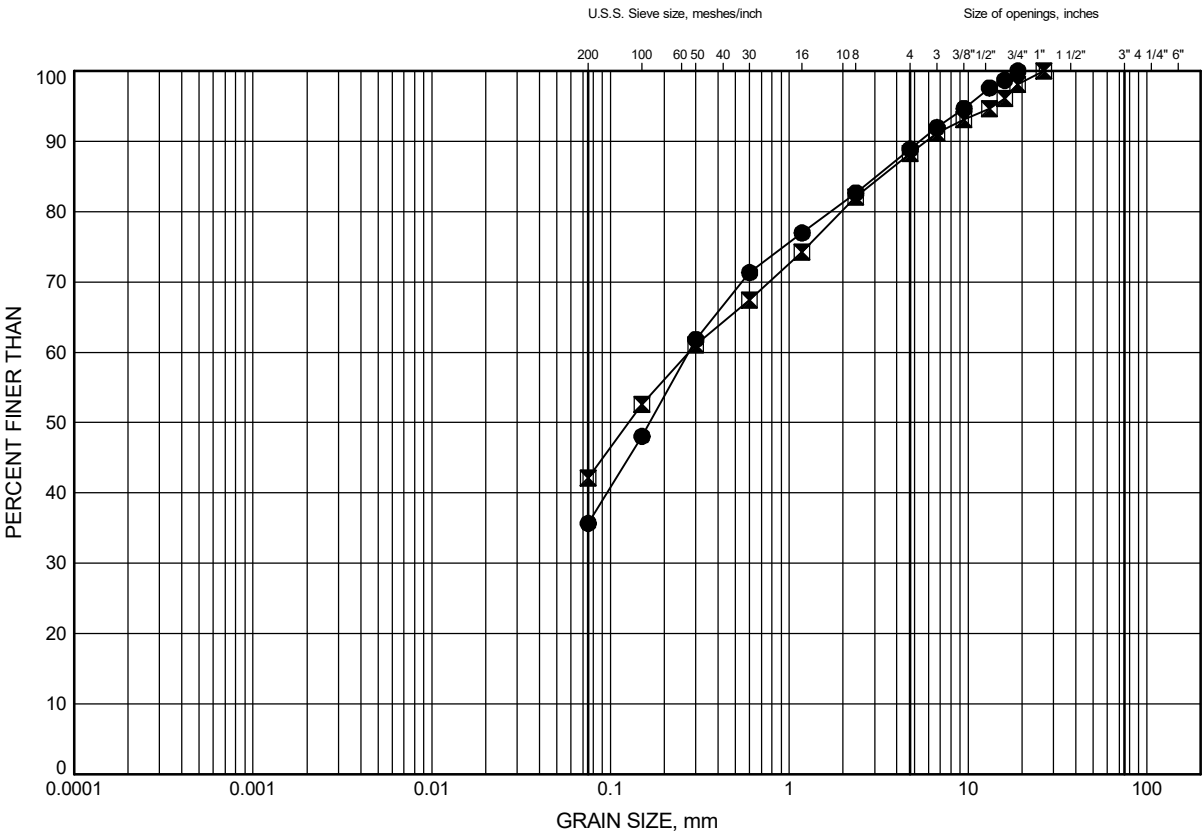


Prep'd RH

Chkd. CM

GRAIN SIZE DISTRIBUTION

FILL: Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

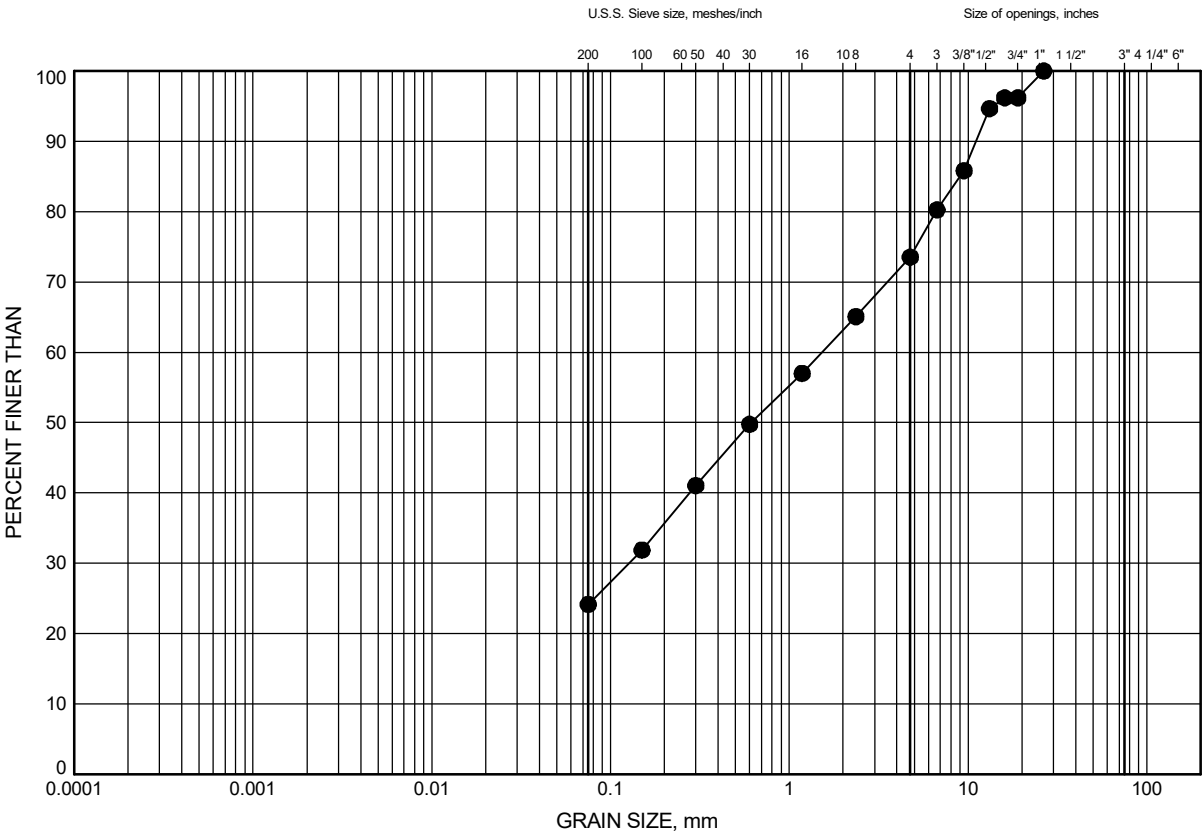
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-01	2.6	89.2
⊠	131-22-02	5.6	86.8

GRAIN SIZE DISTRIBUTION - THURBER 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 8-22-23



GRAIN SIZE DISTRIBUTION

FILL: Gravelly Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

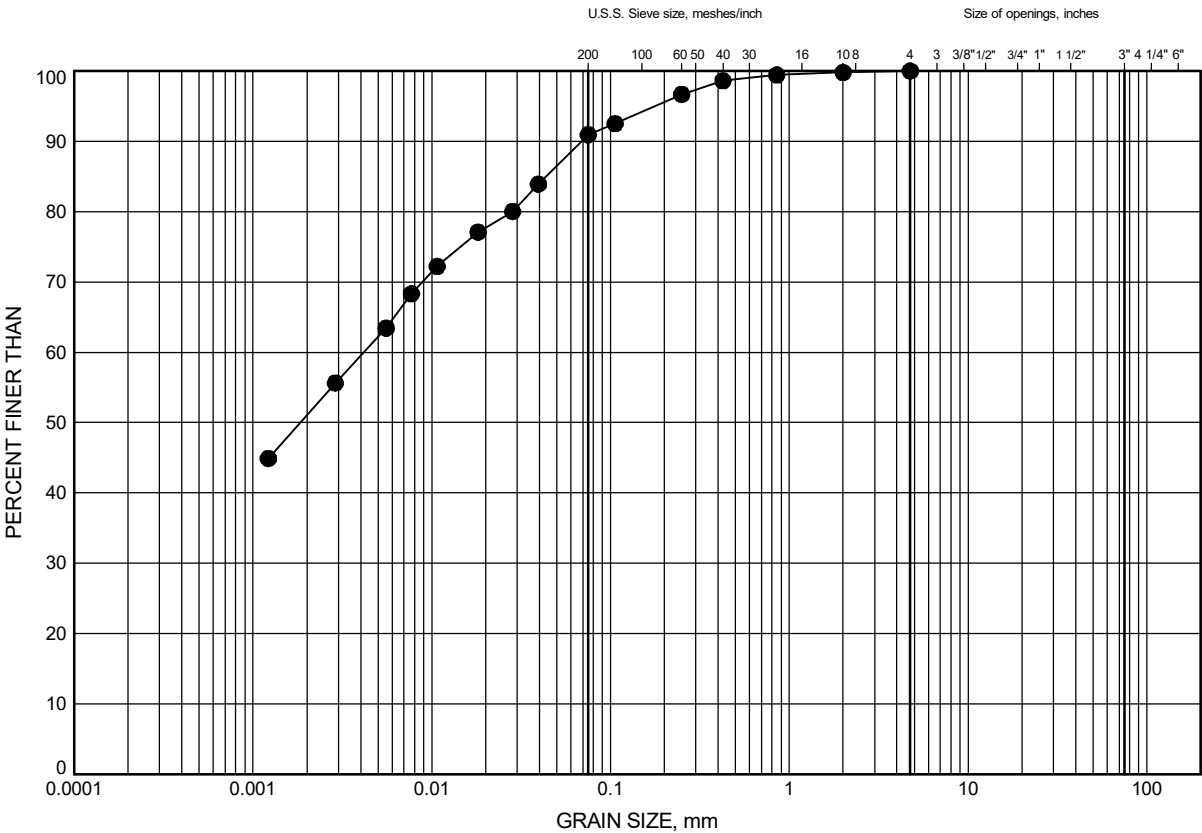
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-02	9.4	83.0

GRAIN SIZE DISTRIBUTION - THURBER 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 3-13-23



GRAIN SIZE DISTRIBUTION

FILL: Clay



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-01	6.4	85.4

Date August 2023  
GWP# 4024-20-00

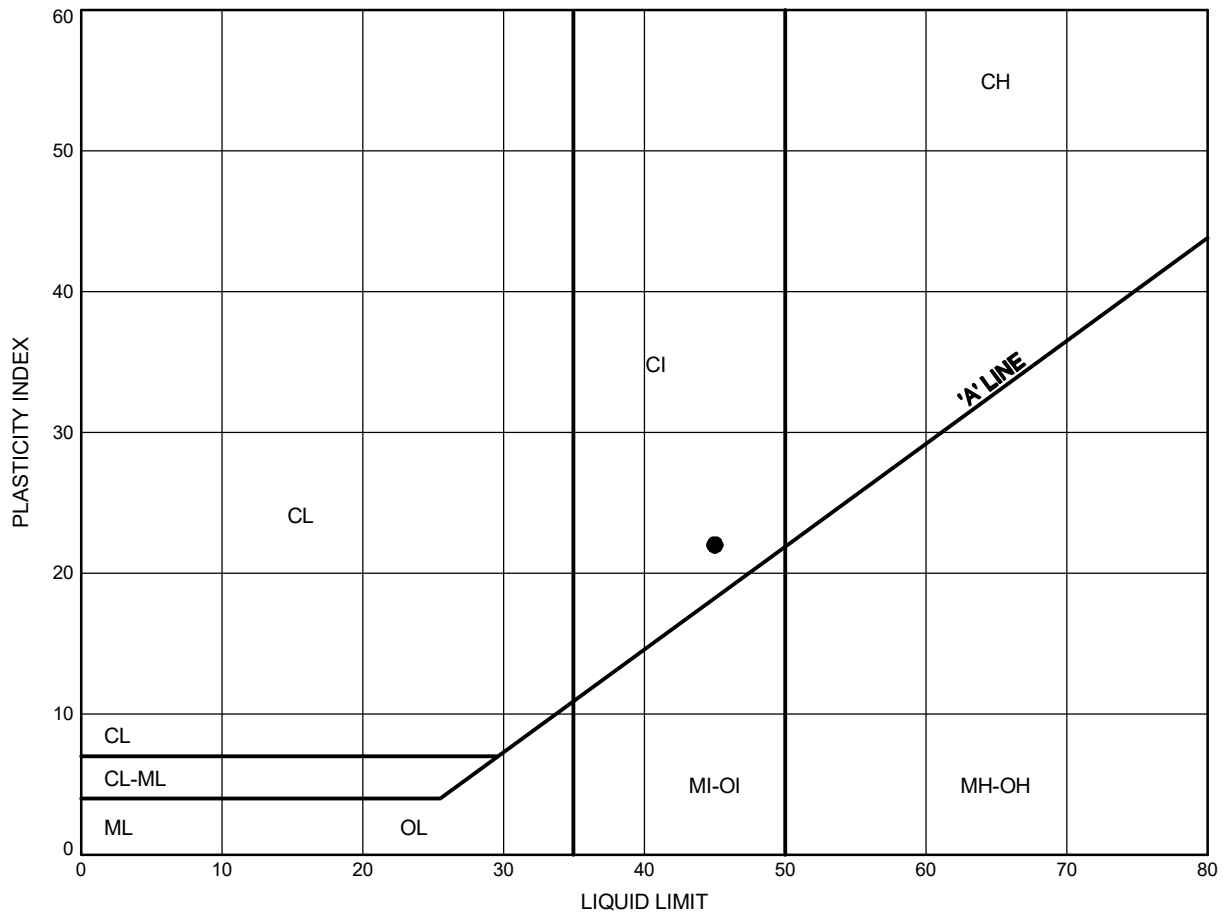


Prep'd RH  
Chkd. KW

Highway 16 Overhead at CNR Railway (Site No. 16X-0131)  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE C5

FILL: Clay



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-01	6.4	85.4

Date August 2023  
GWP# 4024-20-00

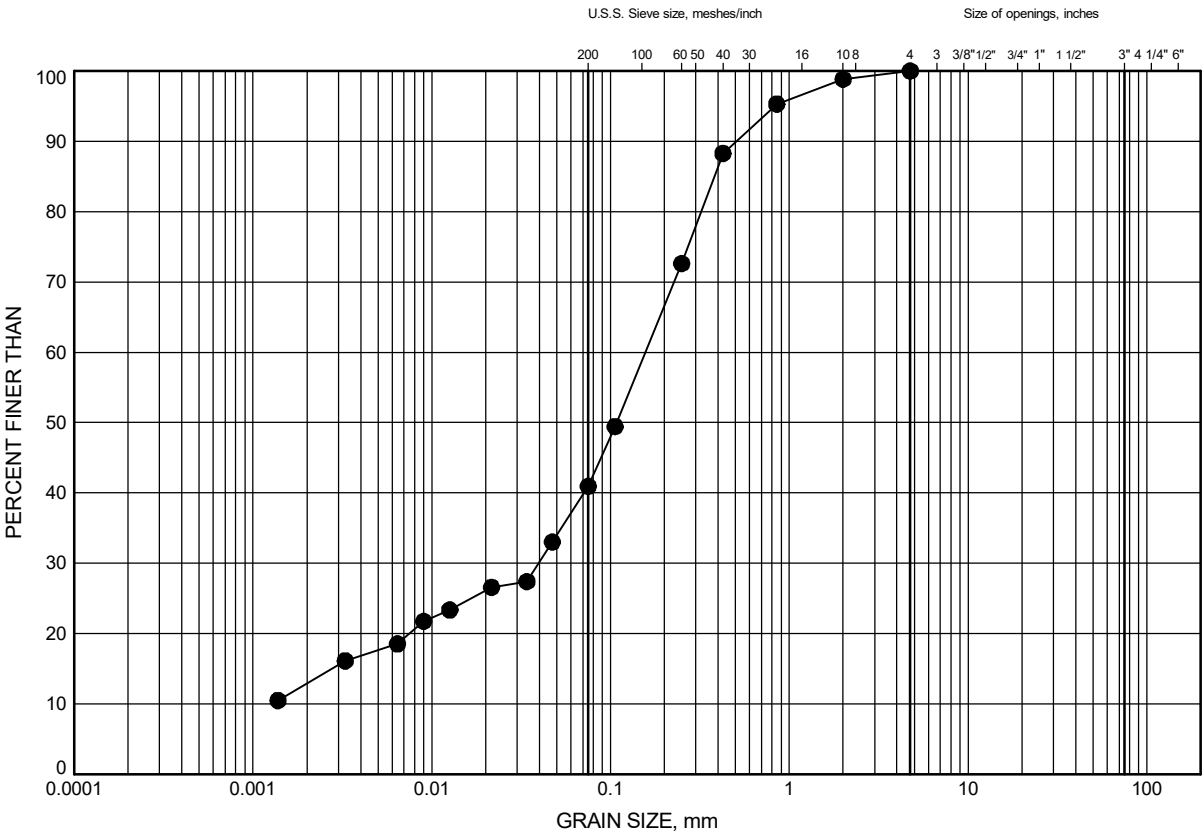


Prep'd RH  
Chkd. KW



GRAIN SIZE DISTRIBUTION

Silty Sand (SM) with organics



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-02	10.7	81.7

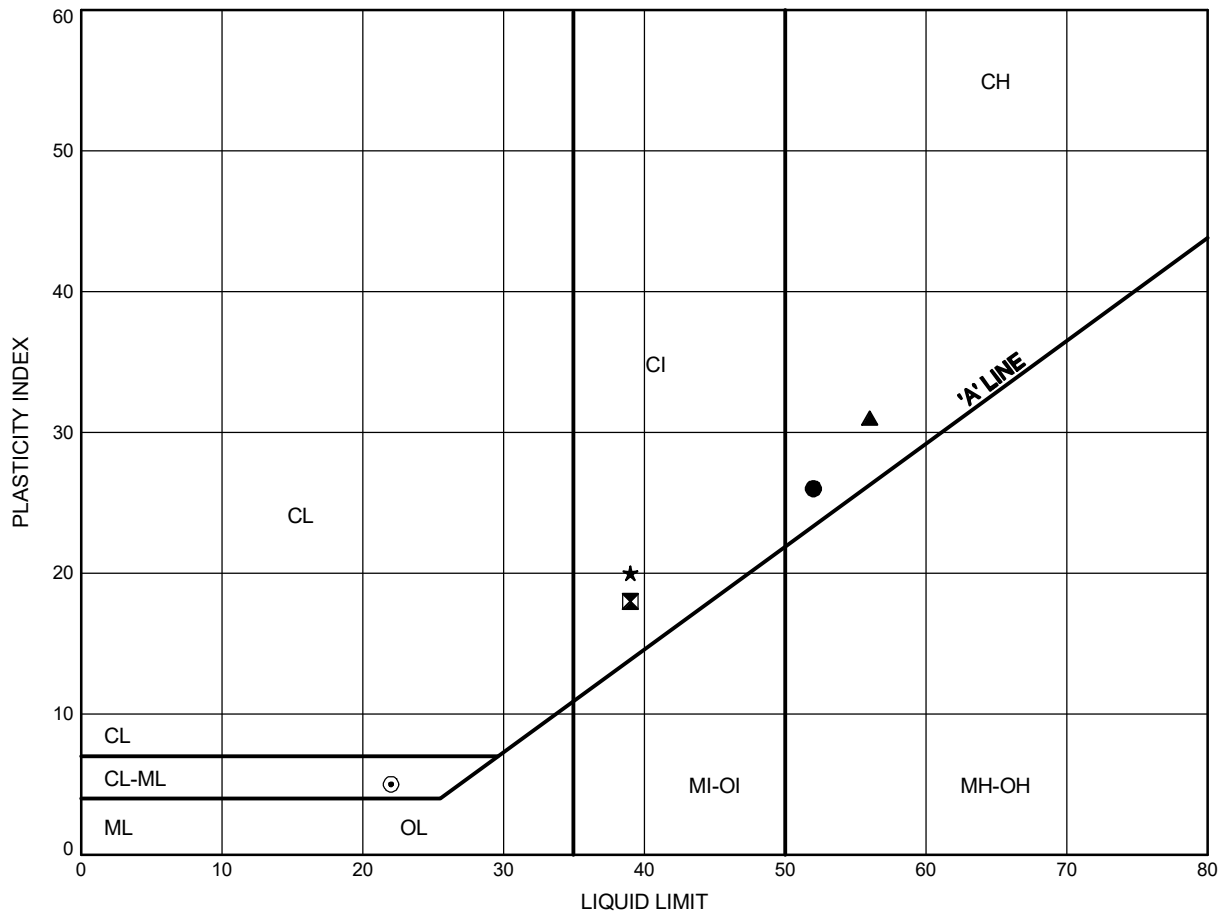
GRAIN SIZE DISTRIBUTION - THURBER 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 8-21-23



Highway 16 Overhead at CNR Railway (Site No. 16X-0131)  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE C7

Clay (CH) to Silty Clay (CI)



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	131-22-01	12.5	79.3
⊠	131-22-01	17.7	74.1
▲	131-22-02	13.3	79.1
★	131-22-02	17.1	75.3
⊙	131-22-02	21.6	70.8

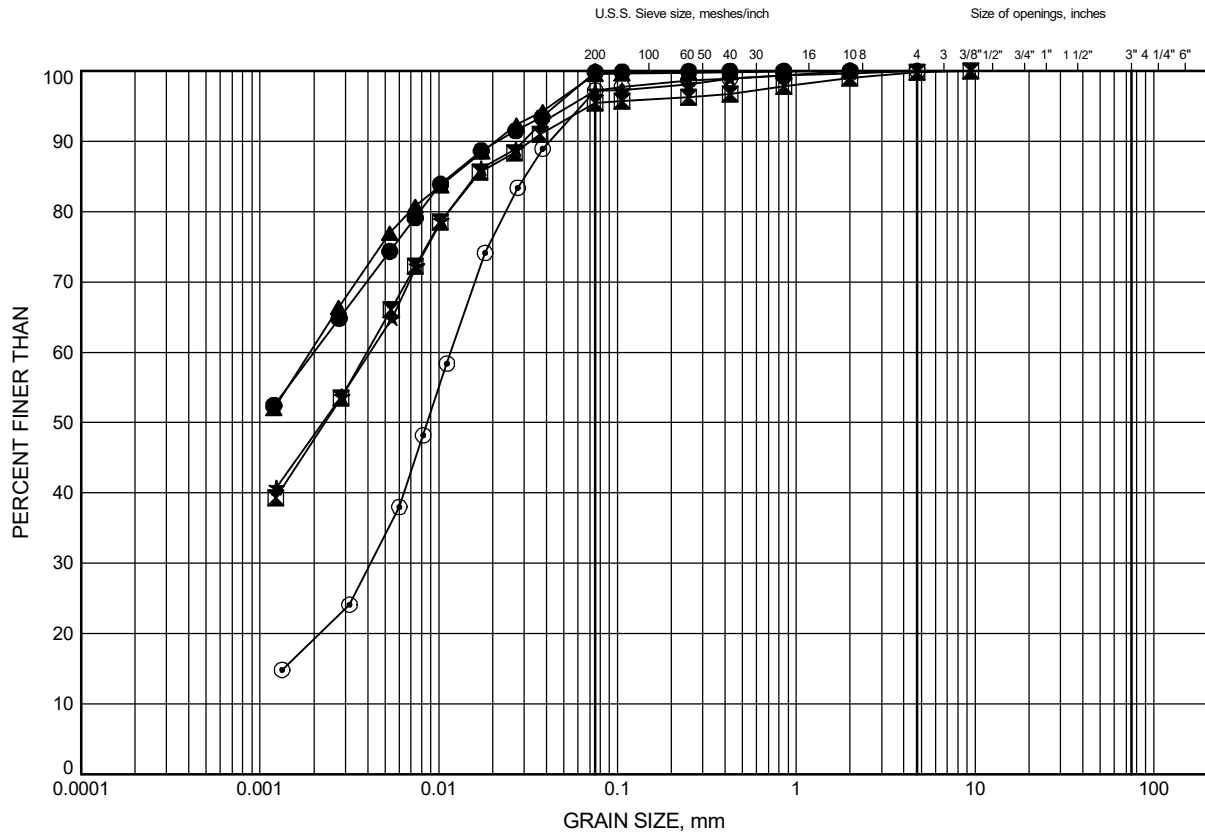
Date August 2023  
 GWP# 4024-20-00



Prep'd RH  
 Chkd. KW

# GRAIN SIZE DISTRIBUTION

Clay (CH) to Silty Clay (CI)





## **Appendix C.2**

### **UCS Test Results**



Stantec Consulting Ltd.  
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

May 2, 2023  
File: 122410864

Client: Thurber Engineering, File #29381

**Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core**

The following table summarizes unconfined compressive strength results for five intact rock cores.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH129-22-1 Run-1	88'7"-89'3"	212.5	Well-formed cones at both ends
BH129-22-2 Run-1	85'9"-86'3"	224.3	Well-formed cones at both ends.
BH130-22-1 Run-3	86'6"-87'4"	196.3	Vertical cracking throughout, no cones formed
BH131-22-2 Run-2	77'1"-77'10"	237.2	Vertical cracking throughout, no cones formed
BH166-22-2 Run-3	33'8"-34'2"	192.6	Well-formed cones at both ends

Sincerely,

Stantec Consulting Ltd.

Brian Prevost  
Laboratory Supervisor  
Tel: 613-738-6075  
Fax: 613-722-2799  
[brian.prevost@stantec.com](mailto:brian.prevost@stantec.com)



## **Appendix C.3**

### **Bedrock Core Photographs**

# Borehole 131-22-01

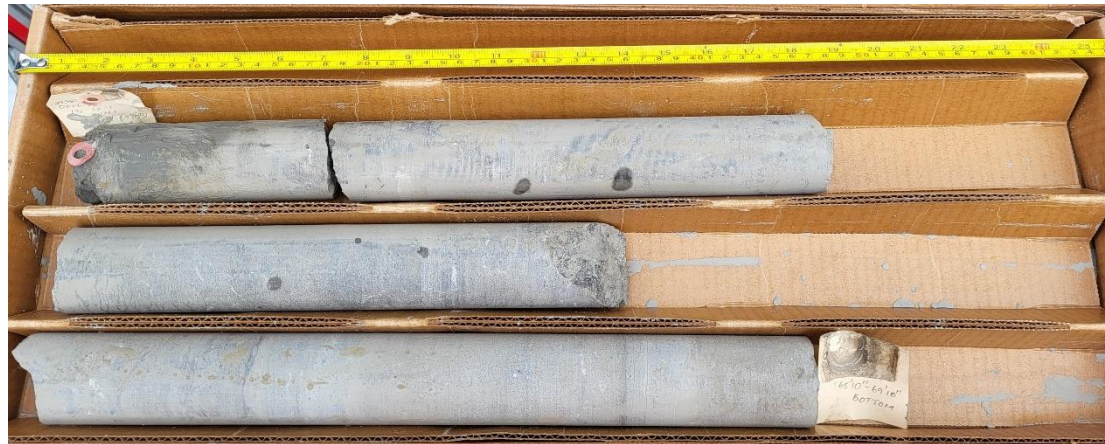
## RUN 1

Depth 20.1 m to 21.3 m

Elevation 71.7 m to 70.5 m

Dry Sample

Run 1 Start  
elev. 71.7 m



Run 1 End  
elev. 70.5 m

# Borehole 131-22-01

## RUN 1

Depth 20.1 m to 21.3 m

Elevation 71.7 m to 70.5 m

Wet Sample

Run 1 Start  
elev. 71.7 m



Run 1 End  
elev. 70.5 m



# Borehole 131-22-01

RUNS 2 and 3

Depth 21.3 m to 23.4 m

Elevation 70.5 m to 68.4 m

Dry Sample

Run 2 Start  
elev. 70.5 m



Run 2 End  
elev. 69.0 m



Run 3 Start  
elev. 69.0 m

Run 3 End  
elev. 68.4 m

# Borehole 131-22-01

RUNS 2 and 3

Depth 21.3 m to 23.4 m

Elevation 70.5 m to 68.4 m

Wet Sample

Run 2 Start  
elev. 70.5 m



Run 2 End  
elev. 69.0 m



Run 3 Start  
elev. 69.0 m

Run 3 End  
elev. 68.4 m

## Borehole 131-22-02

RUNS 1 and 2

Depth 22.7 m to 24.9 m

Elevation 69.7 m to 67.5 m

Dry Sample

Run 1 Start  
elev. 69.7 m



Run 1 End  
elev. 69.0 m

Run 2 Start  
elev. 69.0 m



Run 2 End  
elev. 67.5 m



# Borehole 131-22-02

RUNS 1 and 2

Depth 22.7 m to 24.9 m

Elevation 69.7 m to 67.5 m

Wet Sample

Run 1 Start  
elev. 69.7 m



Run 1 End  
elev. 69.0 m

Run 2 Start  
elev. 69.0 m



Run 2 End  
elev. 67.5 m

# Borehole 131-22-02

## RUN 3

Depth 24.9 m to 26.4 m

Elevation 67.5 m to 66.0 m

Dry Sample

Run 3 Start  
elev. 67.5 m



Run 3 End  
elev. 66.0 m



**THURBER** ENGINEERING LTD.

Geotechnical Investigation  
Highway 16 Overpass at CNR Railway  
Edwardsburgh Township

BH 131-22-02  
Project No.: 29381

# Borehole 131-22-02

## RUN 3

Depth 24.9 m to 26.4 m

Elevation 67.5 m to 66.0 m

Wet Sample





## **Appendix D.**

### **Site Photographs**





Photograph 1: Looking project south of from Highway 16  
[taken on December 19, 2022]



Photograph 2: Looking project north of from Highway 16 Overhead  
[taken on December 19, 2022]





Photograph 3: Looking southeast at the southbound embankment  
[taken on December 19, 2022]



Photograph 4: Looking southwest at the southbound embankment  
[taken on December 19, 2022]



Photograph 5: Looking northwest at the northbound embankment  
[taken on December 19, 2022]



Photograph 6: Looking northeast at the southbound embankment  
[taken on December 19, 2022]



## **Appendix E.**

### **GSC Seismic Hazard Calculation**



# 2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.752N 75.477W

User File Reference: Highway 16 CNR Overhead

2023-02-02 15:14 UT

Requested by: Thurber Engineering Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.385	0.217	0.130	0.037
Sa (0.1)	0.455	0.266	0.165	0.052
Sa (0.2)	0.383	0.228	0.145	0.048
Sa (0.3)	0.293	0.176	0.113	0.039
Sa (0.5)	0.209	0.126	0.081	0.028
Sa (1.0)	0.106	0.064	0.042	0.014
Sa (2.0)	0.051	0.030	0.019	0.006
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.246	0.145	0.090	0.028
PGV (m/s)	0.173	0.100	0.062	0.019

**Notes:** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

## References

**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**  
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information



Natural Resources  
Canada

Ressources naturelles  
Canada

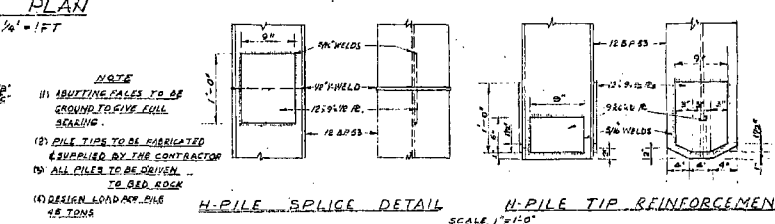
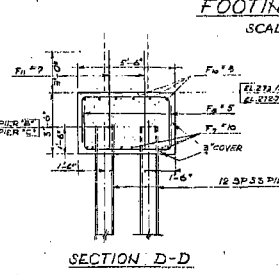
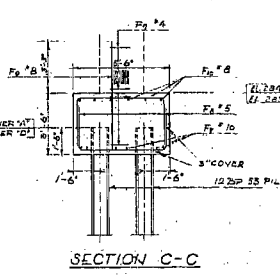
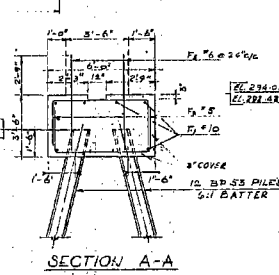
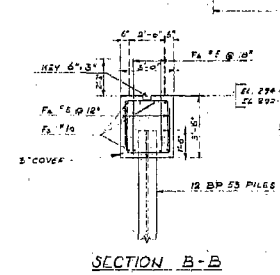
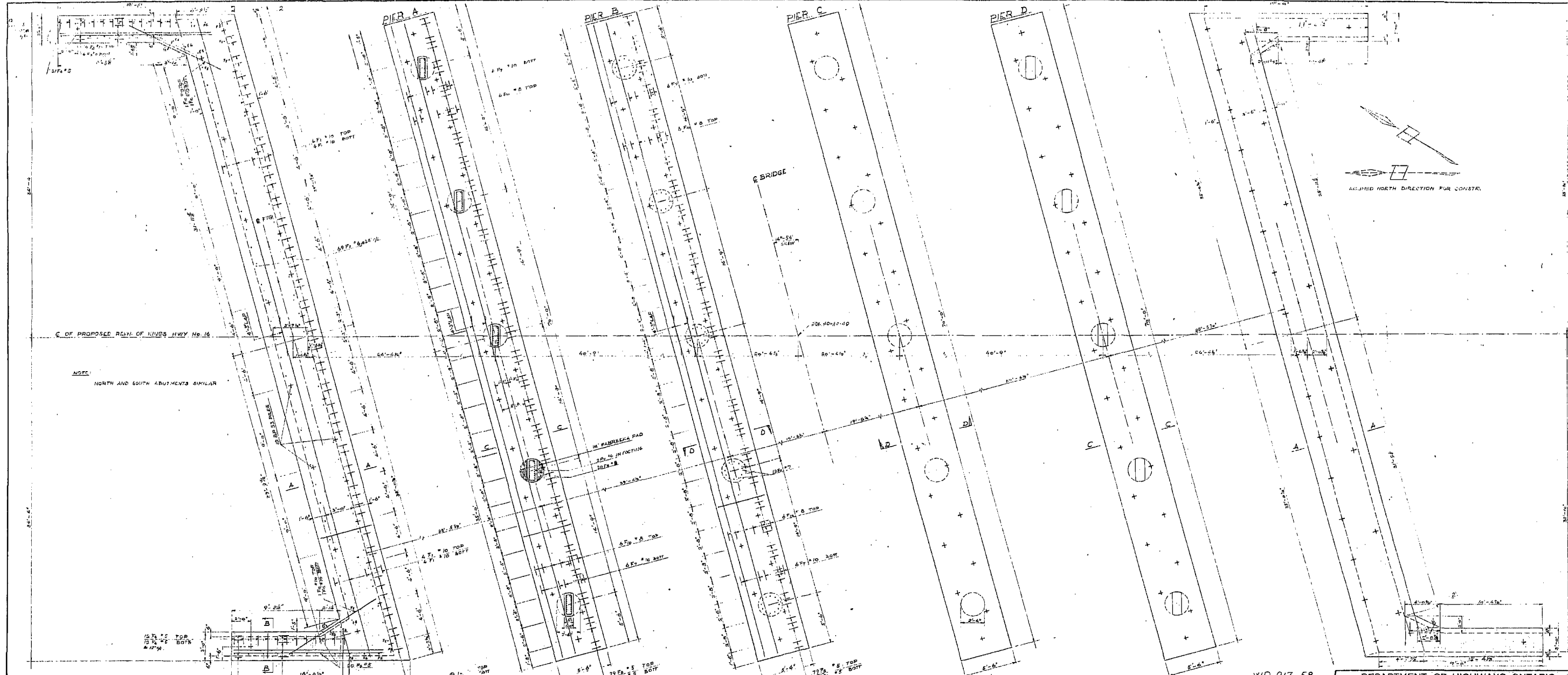
Canada



## **Appendix F.**

### **General Plan Drawing (1959)**





PRINT RECORD	TOP	DATE
1	10/1/58	10/1/58
2	10/1/58	10/1/58
3	10/1/58	10/1/58
4	10/1/58	10/1/58
5	10/1/58	10/1/58
6	10/1/58	10/1/58
7	10/1/58	10/1/58
8	10/1/58	10/1/58
9	10/1/58	10/1/58
10	10/1/58	10/1/58

DEPARTMENT OF HIGHWAYS-ONTARIO  
BRIDGE OFFICE-TORONTO

EDWARDSBURG TWP. CNR.  
OVERHEAD BRIDGE

THE KING'S HIGHWAY NO. 16 DIST. NO. 9  
CO. GRENVILLE  
TWP. EDWARDSBURG LOT 27 & 28 CON. 1/3

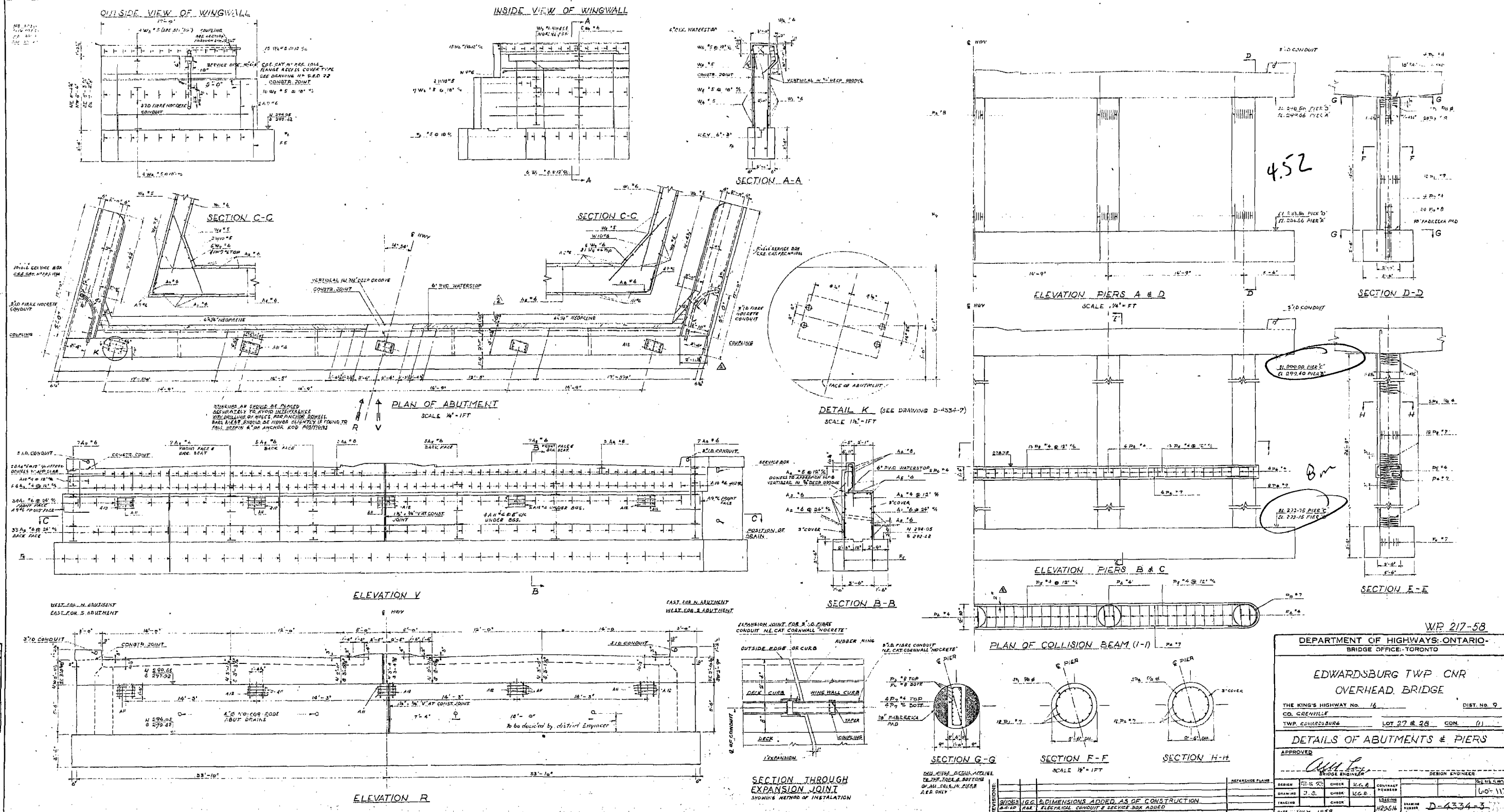
DETAILS OF FOOTING FOR ABUTMENTS & PIERS

APPROVED: *[Signature]*  
BRIDGE ENGINEER

DESIGN ENGINEER

DESIGN	7.1.58	CHECK	7.1.58	CONTRACT NUMBER	420810	REVISION	60-113
TRACING	7.1.58	CHECK	7.1.58	LOADING	420810	DESIGNED BY	D-4334-2
DATE	JULY 1958	BY		DESCRIPTION			

TWP# 27-131-2-A



452

Bm

W.P. 217-58

DEPARTMENT OF HIGHWAYS, ONTARIO  
BRIDGE OFFICE, TORONTO

EDWARDSBURG TWP. CNR  
OVERHEAD BRIDGE

THE KING'S HIGHWAY NO. 16 DIST. NO. 9  
CD. GREENVILLE  
TWP. EDWARDSBURG LOT 27 & 28 CON. (1)

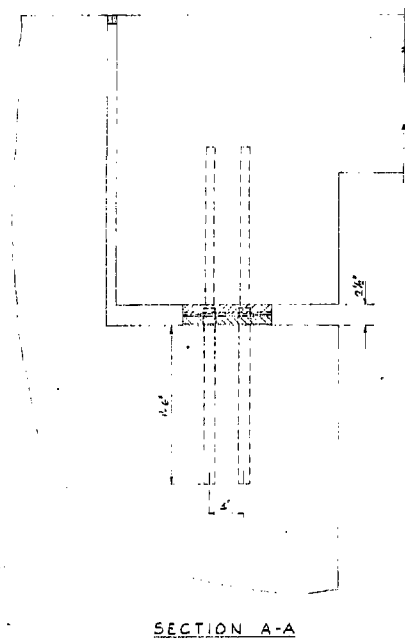
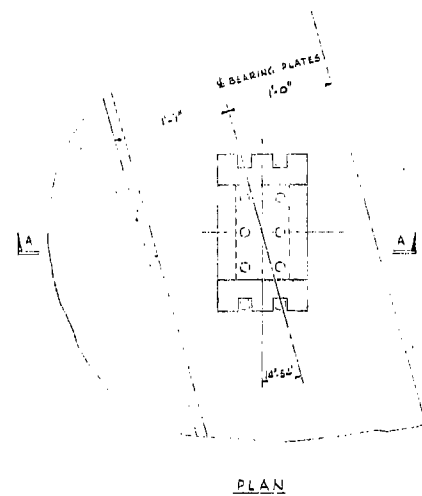
DETAILS OF ABUTMENTS & PIERS

APPROVED				DESIGN ENGINEER				CHECKS				REVISIONS			
<i>Am L</i>				BRIDGE ENGINEER				DESIGN				DATE			
								CHECKED				BY			
								DATE				BY			

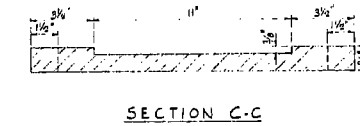
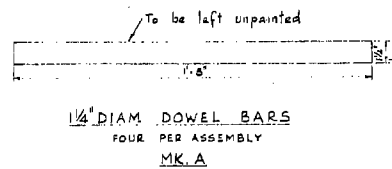
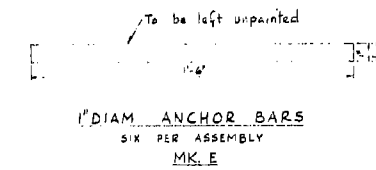
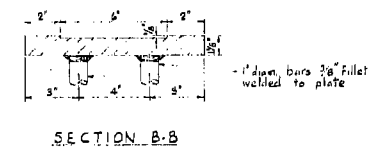
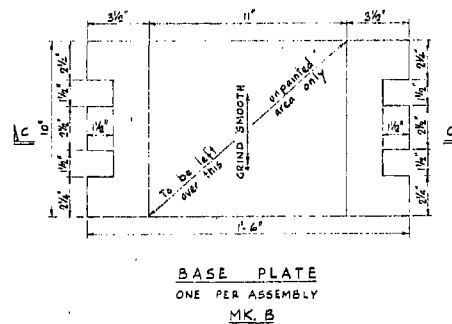
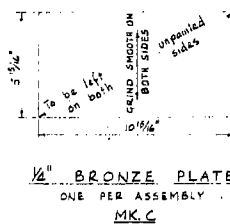
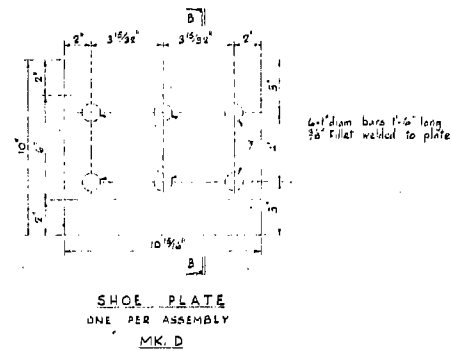
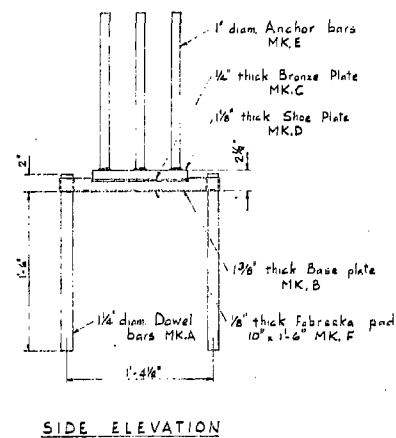
TWP 27-131-3-A







BEARING PLATE ASSEMBLY  
10 OFF THIS REOD.  
Scale 1/4" to 1'-0"



SCHEDULE OF PARTS			
PART	MK.	TOTAL N° REQD.	
Dowel bars	A	40	
Base plate	B	10	
Bronze plate	C	10	
Shoe plate	D	10	
Anchor bars	E	60	
Fabreska pad	F	10	

- NOTES
- Bearing plate assemblies to be fabricated and supplied by the D.H.O.
  - Steel plates shall be "Medium Structural Grade" in accordance with C.S.A. Specifications C40.4.
  - Bronze plates to be hard grade B100 or B160 alloy D.
  - Sliding surfaces to be coated with silicone grease containing rust inhibitor by the Contractor.
  - Holes for dowel bars must be coredrilled. Dowel bars are to be grouted after fixing base plates in correct positions.
  - All surfaces, other than those indicated, to be given one shop coat of Flintkote C24 and two shop coats of Flintkote C12 in accordance with the manufacturers recommendations.

Scale 3" to 1'-0"

NO.	DATE	BY
1	10/1/58	W.S.
2	10/1/58	W.S.
3	10/1/58	W.S.
4	10/1/58	W.S.
5	10/1/58	W.S.
6	10/1/58	W.S.
7	10/1/58	W.S.
8	10/1/58	W.S.
9	10/1/58	W.S.
10	10/1/58	W.S.

REFERENCE PLANS			
DESIGN	DESIGNER	DESIGNER	DESIGNER
DRAWING	DRAWING	DRAWING	DRAWING
TRACING	TRACING	TRACING	TRACING
DATE	DATE	DATE	DATE

Reyn A 49387 WP 217-58

DEPARTMENT OF HIGHWAYS-ONTARIO  
BRIDGE OFFICE-TORONTO

EDWARDSBURG TWP. C.N.R.  
OVERHEAD BRIDGE

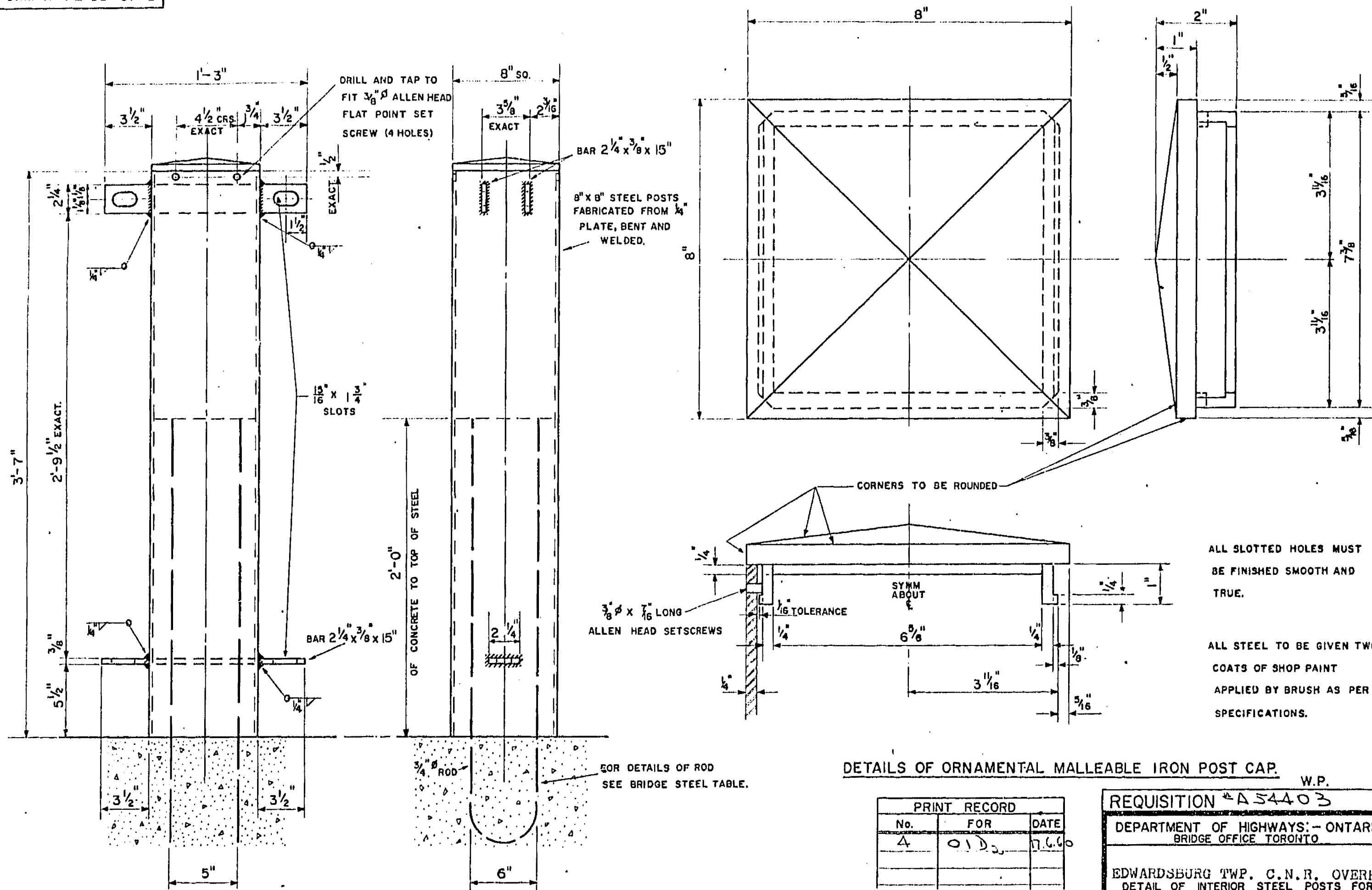
THE KING'S HIGHWAY No. 16 DIST. No. 9  
ED. GREENVILLE  
TWP. EDWARDSBURG LOT 27 & 28 CON. 1

DETAILS OF EXPANSION BEARINGS

APPROVED  
[Signature]  
DESIGN ENGINEER

60-113

TWP #27-131-5-7



ALL SLOTTED HOLES MUST BE FINISHED SMOOTH AND TRUE.

ALL STEEL TO BE GIVEN TWO COATS OF SHOP PAINT APPLIED BY BRUSH AS PER SPECIFICATIONS.

# DETAILS OF ORNAMENTAL MALLEABLE IRON POST CAP.

W.P.

PRINT RECORD		
No.	FOR	DATE
4	01 D <sub>2</sub>	7.6.60

REQUISITION #A54403

DEPARTMENT OF HIGHWAYS:- ONTARIO  
BRIDGE OFFICE TORONTO

EDWARDSBURG TWP. C.N.R. OVERHEAD  
DETAIL OF INTERIOR STEEL POSTS FOR  
HANDRAIL PANELS

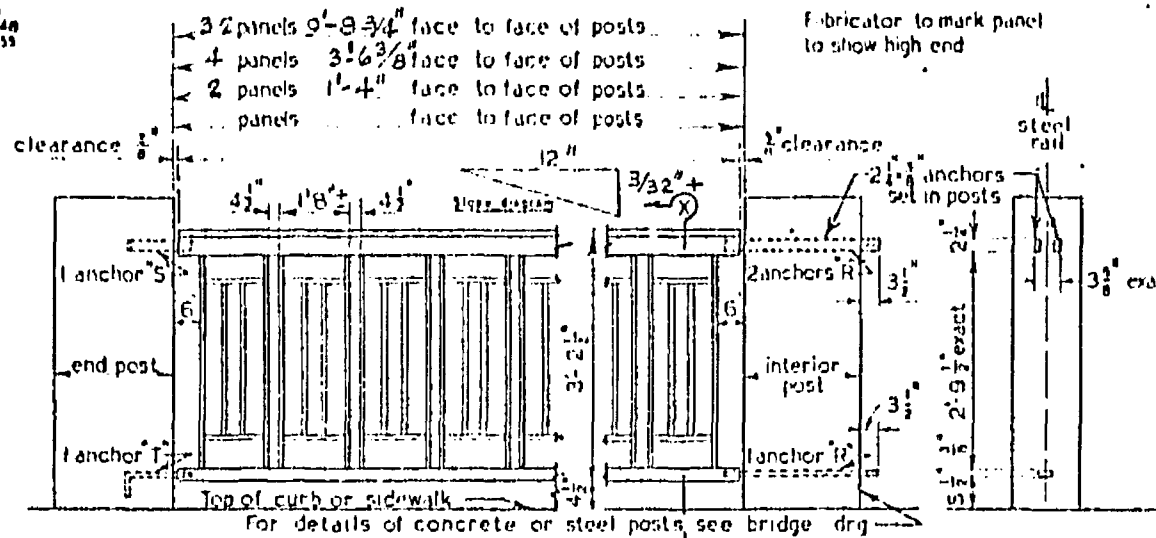
HWY No. 16 DIST. 9

TORONTO ONTARIO BRIDGE ENGINEER

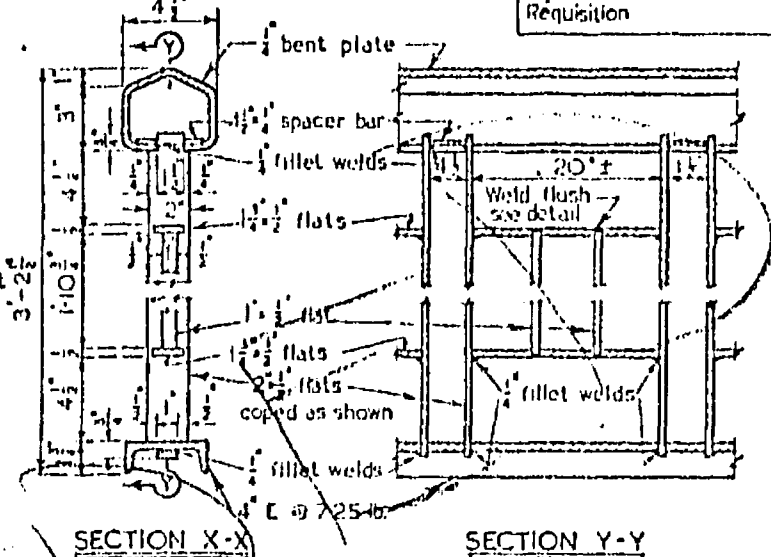
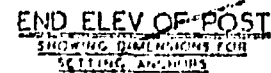
CONTRACT NUMBER 60-113 DRAWING NUMBER D4334-6A

REVISIONS	DATE	BY	DESCRIPTION
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TWP #27-131-6A

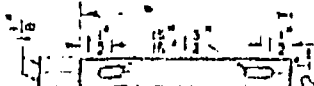
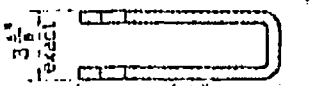
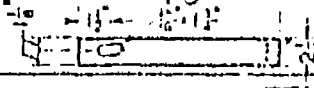


ELEVATION



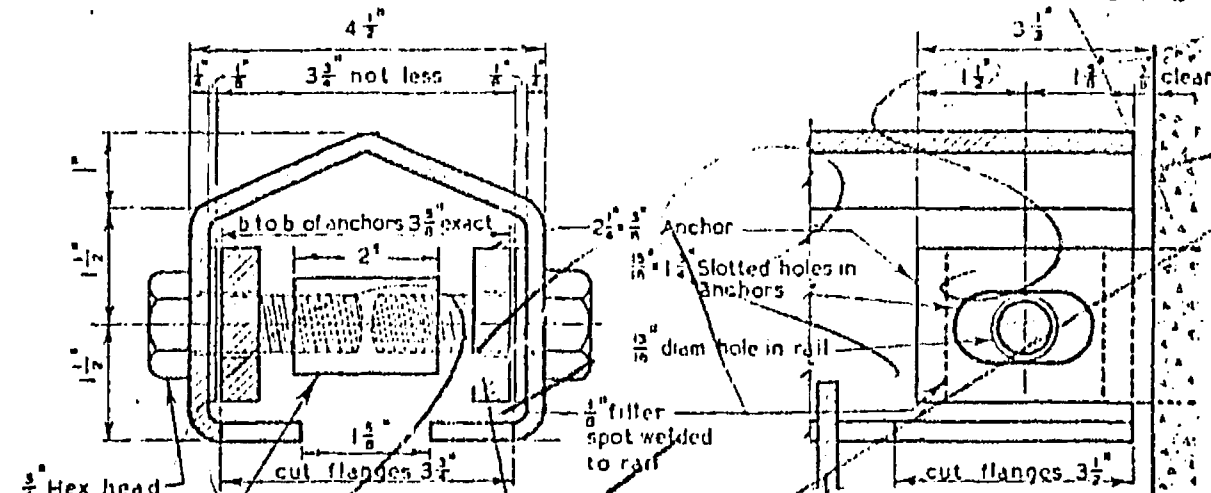
SECTION X-X

SECTION Y-Y

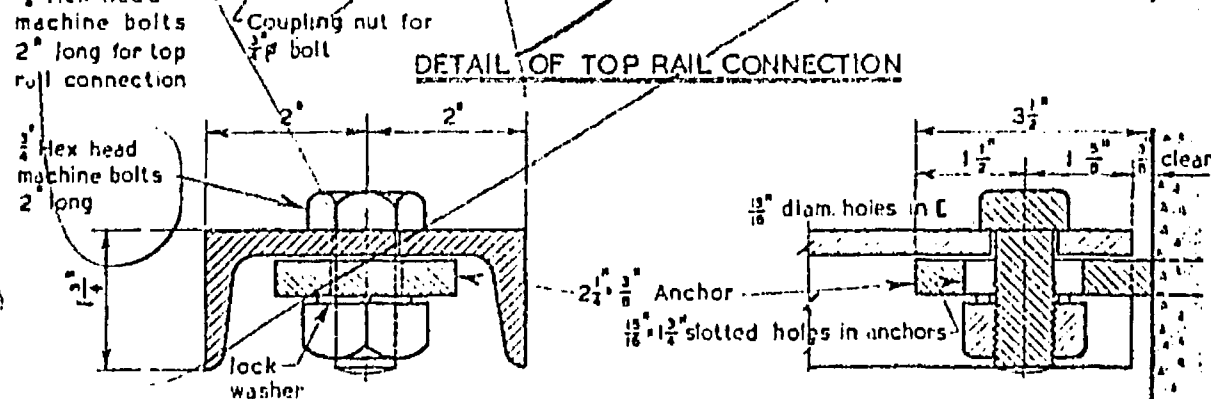
BILL OF MATERIALS			
No.	Details		
36	Steel posts	Interior fas per	3 D, 17-6
	Steel posts	End fas per	
4	Anchors "R"		2 1/4" x 3/8" x 5"
4	Anchors "S"		2 1/4" x 3/8" x 5"
4	Anchors "T"		2 1/4" x 3/8" x 5"
76	3/4" Hexagon head machine bolts 2" long each with one square nut and one lock washer		
152	3/4" Hexagon head machine bolts 2" long		
76	Coupling nuts for 3/4" bolts		

NOTE: Slotted holes in anchors must be finished smooth and true

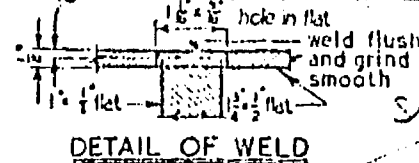
NOTE: Slotted holes in anchors must be finished smooth and true



DETAIL OF TOP RAIL CONNECTION



### DETAIL OF BOTTOM RAIL CONNECTION

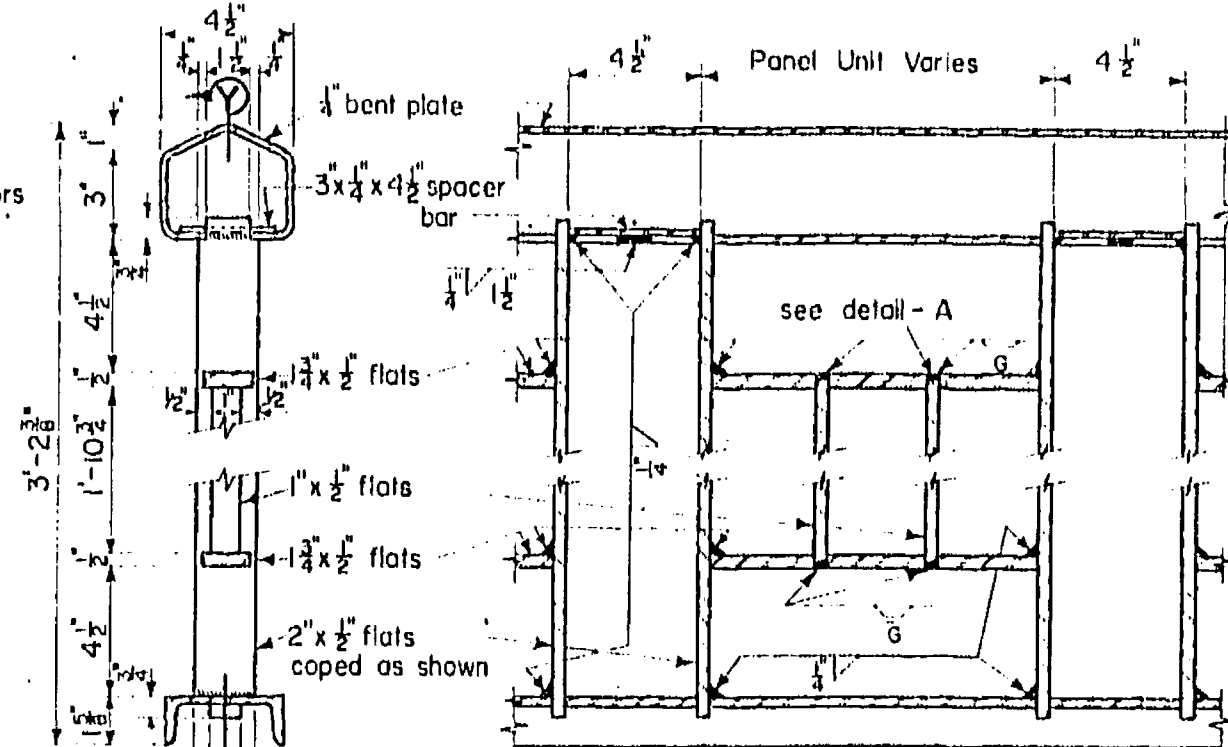


### DETAIL OF WELD

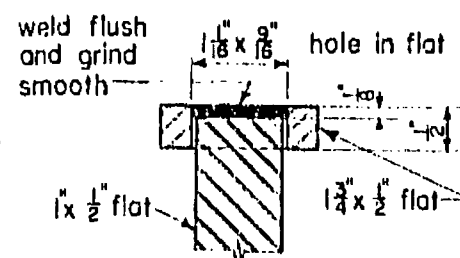
NOTE: - All steel including bolt heads, nuts, and anchors, to be given one shop coat of paint applied by brush as per specifications.

*per [unclear]*  
DETAILS OF  
STEEL  
HANDRAIL PANELS  
EDWARDSBURG TWP.  
C.N.R. OVERHEAD BRIDGE

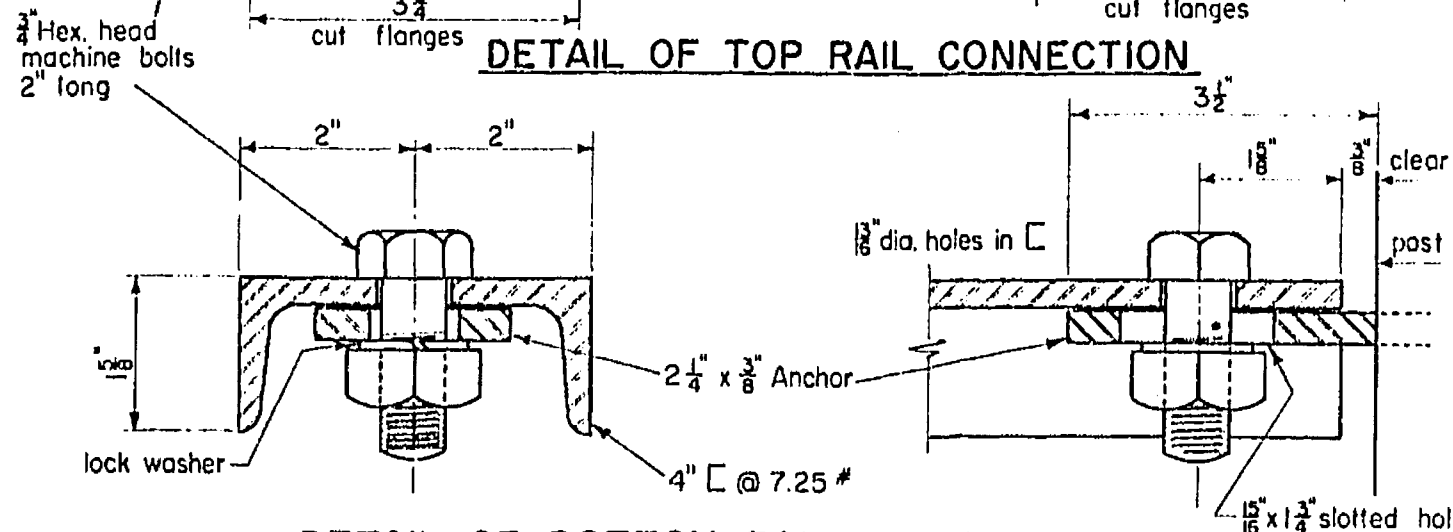
HWY NO. <u>13</u> DEPARTMENT OF HIGHWAYS TORONTO ONTARIO	DIV. <u>2</u> BRIDGE ENGINEER
Contract Number...	Drawing Number, <u>D4334-6</u>



SECTION Y-Y



### DETAIL OF TOP RAIL CONNECTION



### DETAIL OF BOTTOM RAIL CONNECTION:

NOTE: Slotted holes in anchors must be finished smooth and true.

PRINT RECORD		
No.	FOR	DATE
8	OIF DS	17.6.

NOTE: ALL STEEL INCLUDING BOLT HEADS, NUTS, AND ANCHORS, TO BE GIVEN TWO SHOP COATS OF PAINT APPLIED BY BRUSH AS PER SPECIFICATIONS. ALL SMALL PARTS TO BE PROPERLY BOXED OR BAGGED. W. P.

REQUISITION # A 54403

DEPARTMENT OF HIGHWAYS: - ONTARIO  
BRIDGE OFFICE TORONTO

EDWARDSBURG TWP. C.N.R. OVERHEAD  
DETAILS OF STEEL HANDRAIL PANELS:

HWY NO. 16		DIST. 9	
TORONTO ONTARIO		BRIDGE ENGINEER	
CONTRACT NUMBER 60-113		DRAWING NUMBER D4534-6	

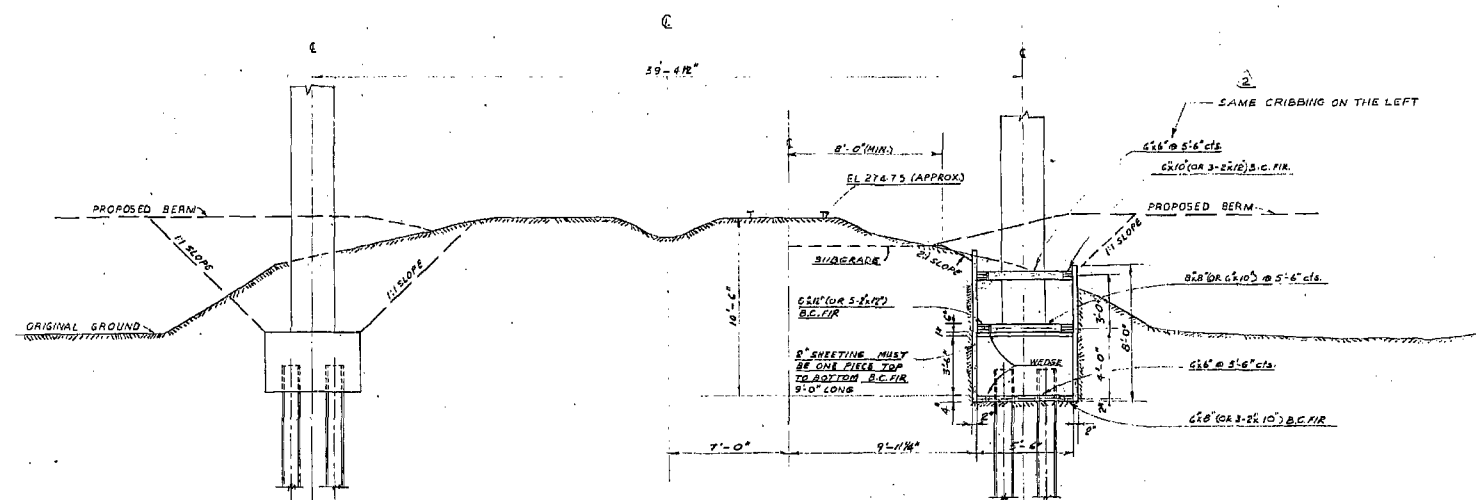
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15.6.60	JW	TYPING REVISED	
DATE	BY	DESCRIPTION	

DRWG. A. W.	CH	P.B.M.
DATE		- 59.

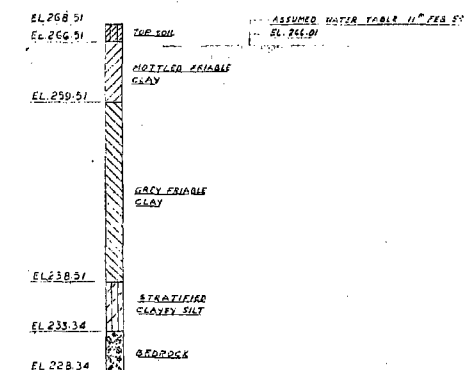


TWP #27-131

[illegible]



SECTION THROUGH TRACK  
N.T.S.



BOREHOLE NO. 5  
N.T.S.  
FOR POSITION OF BOREHOLES  
SEE DRAWING D-5334-1

NO.	FOR	DATE
1	W. J. L.	10-10-58
2	W. J. L.	11-11-58
3	W. J. L.	11-11-58
4	W. J. L.	11-11-58
5	W. J. L.	11-11-58
6	W. J. L.	11-11-58
7	W. J. L.	11-11-58
8	W. J. L.	11-11-58
9	W. J. L.	11-11-58
10	W. J. L.	11-11-58

W.P. 217-58  
DEPARTMENT OF HIGHWAYS-ONTARIO  
BRIDGE OFFICE-TORONTO

EDWARDSBURG TWP. C.N.R.  
OVERHEAD BRIDGE

THE KING'S HIGHWAY NO. 16 DIST. NO. 9  
CO. GRENVILLE  
TWP. EDWARDSBURG LOT 27228 CON. 111

TRACK STRUCTURE PROTECTION

APPROVED *[Signature]* DESIGN ENGINEER

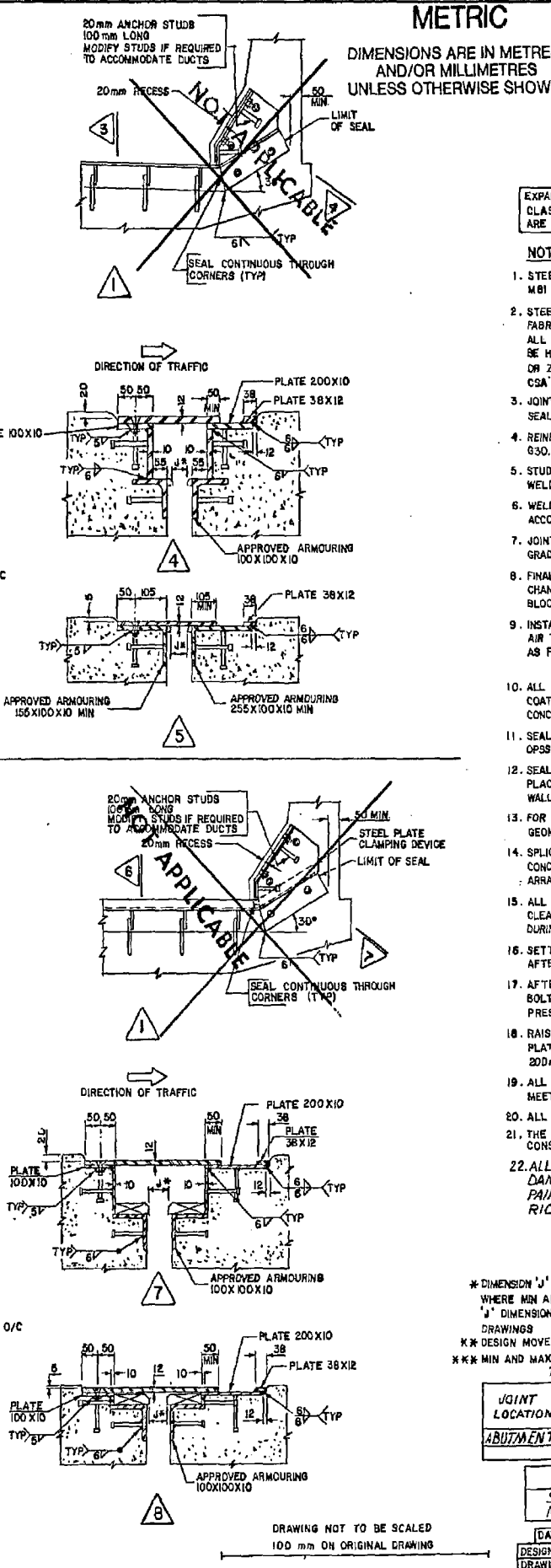
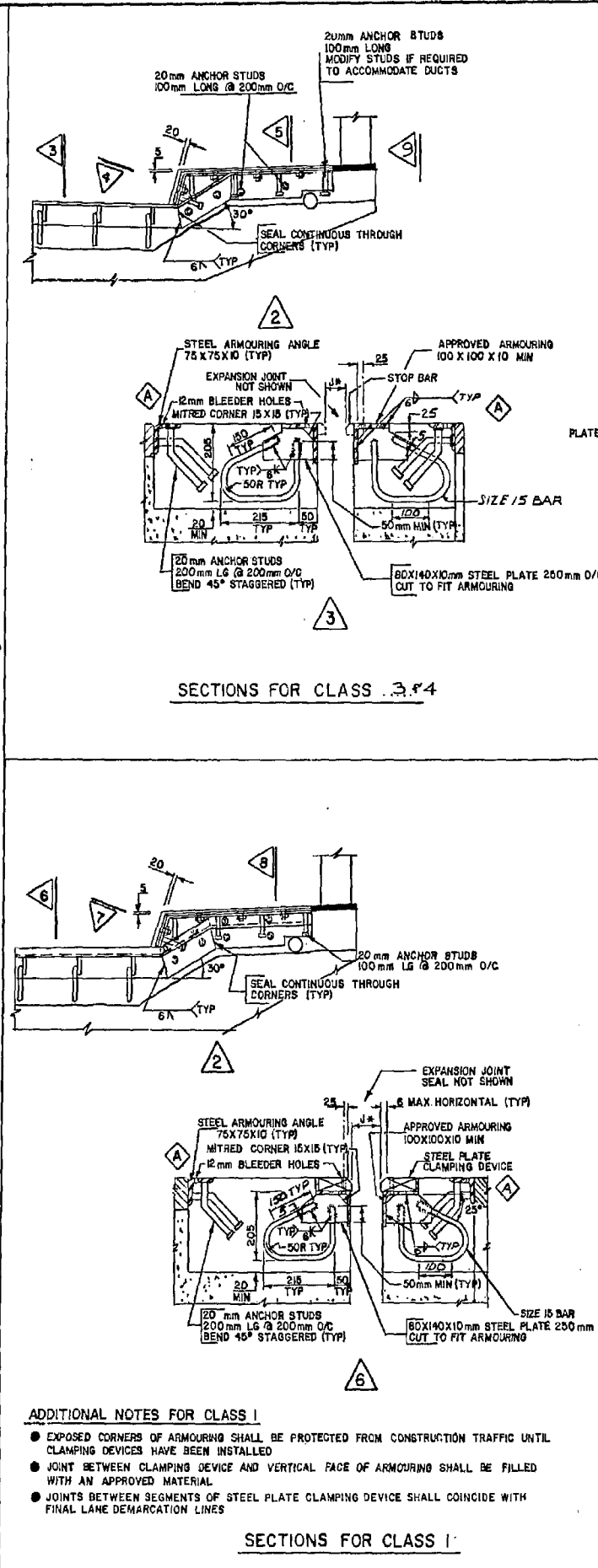
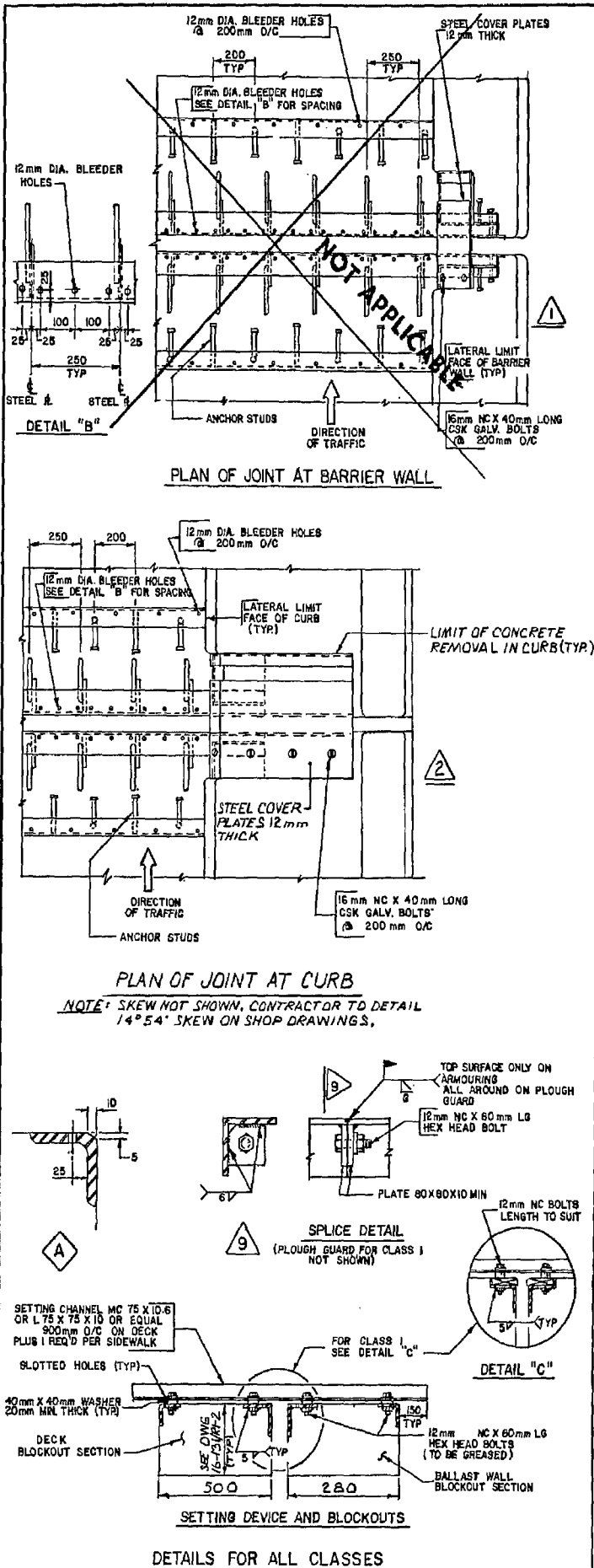
REVISIONS	DATE	BY	DESCRIPTION
1	11-11-58	W.J.L.	POWER LINES INDICATED AT 3 M.A. DISTANCE
2	11-11-58	W.J.L.	NOTE ADDED, AS OF CONSTRUCTION

REVISIONS	DATE	BY	DESCRIPTION
1	11-11-58	W.J.L.	POWER LINES INDICATED AT 3 M.A. DISTANCE
2	11-11-58	W.J.L.	NOTE ADDED, AS OF CONSTRUCTION

DATE NOVEMBER 1958

W.P. 217-131-9-A





CONT No 89-37 WP No 58-81-01		SHEET 43	
C.N.R. OVERHEAD		JOINT ANCHORAGE AND ARMOURING	

**NOTES:**

- STEEL SHALL BE IN ACCORDANCE WITH CSA STANDARD CAN 3.640.21-M81 GRADE 300 W
- STEEL COVER PLATES SHALL BE HOT DIP GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH CSA STANDARD G164-M. ALL OTHER STEEL SURFACES OF JOINT ASSEMBLY AND ARMOURING SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH CSA STANDARD G164-M OR ZINC METALLIZED AFTER FABRICATION IN ACCORDANCE WITH CSA STANDARD G108
- JOINT ASSEMBLY SHALL BE COMPLETELY SHOP ASSEMBLED (EXCEPT FOR SEAL) AND PRESET TO DIMENSION 'U' FOR 15°C PRIOR TO SHIPMENT
- REINFORCING STEEL SHALL BE IN ACCORDANCE WITH CSA STANDARD G30.18-M GRADE 400
- STUDS SHALL BE IN ACCORDANCE WITH DS 185 USING FUSION WELD PROCESS
- WELDING SHALL BE OF A LOW HYDROGEN CLASSIFICATION AND IN ACCORDANCE WITH CSA STANDARD W58-M1984 (E 480XX ELECTRODES)
- JOINT ASSEMBLY SHALL BE INSTALLED TO MATCH ELEVATION AND GRADE OF CONCRETE END DAMS, USING CONCRETE BLOCKOUT METHOD
- FINAL WIDTH ADJUSTMENT (DIM. 'U') SHALL BE MADE, AND SETTING CHANNELS TIGHTENED, JUST PRIOR TO PLACING 30 MPa CONCRETE IN BLOCKOUTS
- INSTALLATION TEMPERATURE SHALL BE TAKEN AS THE MEAN SHADE AIR TEMPERATURE, PRIOR TO JOINT INSTALLATION AT THE STRUCTURE AS FOLLOWS: CONCRETE STRUCTURE-48 HOURS, STEEL STRUCTURE-24 HOURS
- ALL CONSTRUCTION JOINT SURFACES SHALL BE ABRASIVE BLAST CLEANED & COATED WITH NEAT CEMENT PASTE IMMEDIATELY BEFORE PLACING OF CONCRETE IN BLOCKOUTS
- SEAL SHALL BE PREFORMED NEOPRENE SEAL IN ACCORDANCE WITH OPSS 1210 AND SHALL BE INSTALLED IN ONE CONTINUOUS PIECE
- SEAL SHALL BE INSTALLED AFTER JOINT ASSEMBLY HAS BEEN CAST IN PLACE AND STYROFOAM OR FILLER BETWEEN BRIDGE AND BALLAST WALL REMOVED AND EXPANSION GAP PROPERLY CLEARED
- FOR SKEWED STRUCTURE DETAILS SHALL BE ADJUSTED TO SUIT GEOMETRY OF STRUCTURE
- SPLICING IN ARMOURING SHALL BE WELDED BEFORE PLACING OF CONCRETE IN BLOCKOUTS. FOR LOCATION OF SPLICES SEE GENERAL ARRANGEMENT DRAWING
- ALL STEEL SURFACES IN CONTACT WITH NEOPRENE SEAL SHALL BE CLEANED PRIOR TO THE INSTALLATION OF THE SEAL AND PROTECTED DURING PLACING OF CONCRETE
- SETTING CHANNELS SHALL BE RELEASED NOT LATER THAN 4 HOURS AFTER PLACING OF CONCRETE IN BLOCKOUTS
- AFTER REMOVAL OF SETTING CHANNELS AND SPACER WASHERS, BOLT HOLES, BLEEDER HOLES AND VOIDS SHALL BE FILLED UNDER PRESSURE WITH SIKADUR 35 (HI MOD LV EPOXY)
- RAISED MEDIANS ON STRUCTURES SHALL BE PROVIDED WITH SLIDING PLATES AS DETAILED FOR SIDEWALKS. STUDS SHALL BE AT 200mm O/C
- ALL BOLTS SHALL BE LIBERALLY COATED WITH ANTISEIZE COMPOUND MEETING U.S. MILITARY SPECIFICATION MIL-A-907D
- ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN
- THE 'U' DIMENSIONS SHOWN ON SHOP DRAWINGS ARE TO BE USED FOR CONSTRUCTION
- ALL FIELD WELD AREAS OF STEEL SURFACE AND ANY DAMAGED AREAS OF THE TREATED SURFACE SHALL BE PAINTED WITH TWO COATS OF AN APPROVED ZINC RICH PAINT.

**\* DIMENSION 'U' MEASURED PERPENDICULAR TO CENTRELINE EXPANSION JOINT, WHERE MIN AND MAX 'U' FOR JOINT SUPPLIED DIFFER FROM THOSE SHOWN IN TABLE, 'U' DIMENSIONS SHALL BE REVISED BY CONTRACTOR AND SHOWN ON SHOP DRAWINGS**

**\*\* DESIGN MOVEMENT IS THE CALCULATED MOVEMENT OF THE STRUCTURE**

**\*\*\* MIN AND MAX JOINT GAP RATING SHOWN IS TAKEN FROM DS 1412 CLASS 3.4**

**TABLE OF 'U' DIMENSIONS (mm)**

JOINT LOCATION	JOINT *** GAP RATING		DESIGN ** MOVEMENT	INSTALLATION TEMPERATURE (°C)					
	MIN	MAX		5°	10°	15°	20°	25°	30°
ABUTMENT	35	75	2.5	60	58	56	54	52	50

**MODIFIED**

**STANDARD DRAWING NOVEMBER 1987**

**SS 113-11**

**DATE BY**

**DESIGN/DOPT**

**CHECK Q I**

**LOADING**

**OHBC-83**

**DATE 07-88**

**DWG 4**



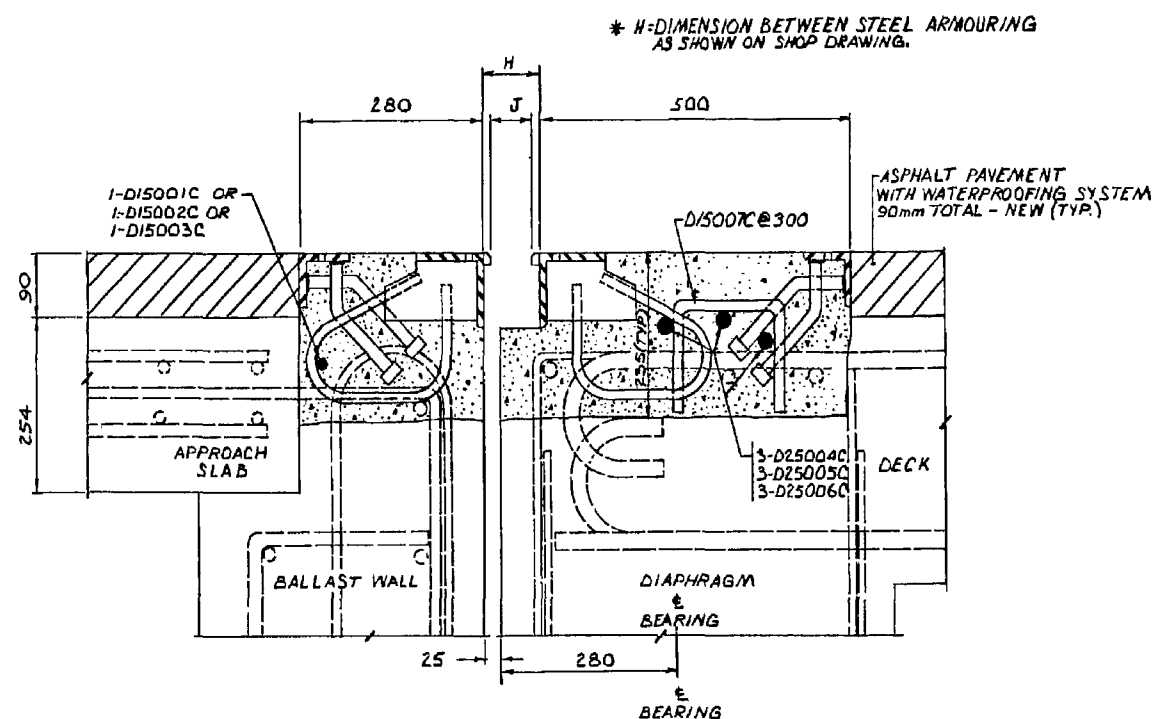
**METRIC**

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No 89-37  
WP No 58-81-01

C.N.R. OVERHEAD  
EXPANSION JOINT DETAILS

SHEET  
42



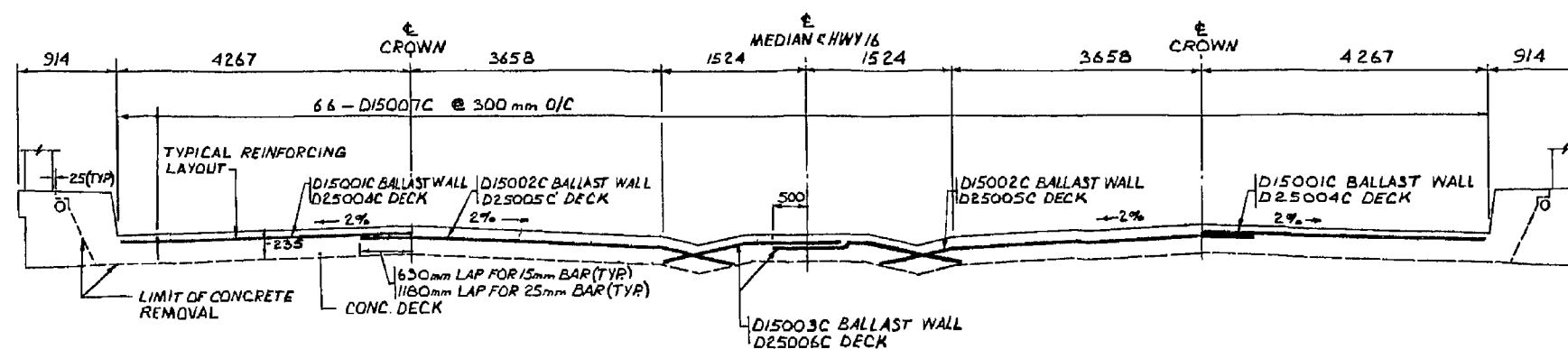
NEW EXPANSION JOINT AT ABUTMENT (TYP.)  
1:5

REINFORCING STEEL SCHEDULE				
BAR MARK	NO REQ'D	SIZE	LENGTH	DETAIL
D15001C	4	15	4415	STRAIGHT
D15002C	4	15	5140	4415 90 75
D15003C	4	15	1985	1285 160 75
D25004C	12	25	4415	STRAIGHT
D25005C	12	25	5690	4965 90 75
D25006C	12	25	1985	1285 160 75
D15007C	132	15	500	180 160 75

**NOTE**

PLACE REINFORCING STEEL TO SUIT CONSTRUCTION  
STAGES SHOWN ON DWG. NO 16-131/R1-2

REINFORCING STEEL SCHEDULE  
PREPARED BY: W. SIMPSON  
CHECKED BY: T. NADAVALLIL  
REVIEWED BY: H. KLEYWEBT




CROSS-SECTION  
N.T.S

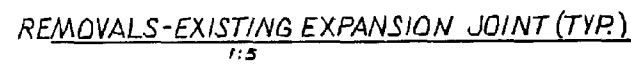
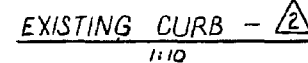
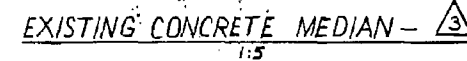


DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN H.K.	CHK A.I.	CODE	LOAD
DRAWN W.S.	CHK H.K.	SITE	16-131 STRUCT
			SCHEMATIC DWG 3



SHEET  
41



REVISIONS							
	DATE	BY	DESCRIPTION				
	DESIGN	H.K.	CHK	Q.1.	CODE	LOAD	DATE JULY 1988
	DRAWN	W.S.	CHK	H.K.	SITE 16-731	STRUCT	SCHEME R1 DWG 2

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

DISTRICT 9 - OTTAWA  
CONT No 89-37  
WP No 58-81-01



C.N.R. OVERHEAD  
GENERAL ARRANGEMENT  
AND DETAILS

SHEET  
40

### GENERAL NOTES

1. CLASS OF CONCRETE 30 MPa
2. CLEAR COVER TO REINFORCING STEEL SHALL BE 70 mm UNLESS OTHERWISE NOTED.
3. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BARS MARKED WITH SUFFIX 'C' SHALL BE COATED BARS.
4. THE CONTRACTOR SHALL DETERMINE AND CHECK ALL RELEVANT DIMENSIONS AND SKEWS OF EXISTING STRUCTURE ON SITE PRIOR TO FABRICATION OF EXPANSION JOINT ASSEMBLY.

### WORK DESCRIPTION

- A. REMOVE EXISTING 80mm± ASPHALT PAVEMENT AND WATERPROOFING FROM DECK. REMOVE CONCRETE FROM DECK AS PER SPECIFICATION, PATCH WITH CONCRETE, WATERPROOF DECK AS PER OPSD 508-02 AND REPAVE.
- B. MODIFY EXISTING EXPANSION JOINTS AS DETAILED.
- C. INSTALL 25mm I.D. P.V.C. DRAINAGE TUBES IN DECK AS PER M.T.C. STANDARD DD-3951.
- D. PLUG OPENINGS IN MEDIAN WITH CONCRETE.

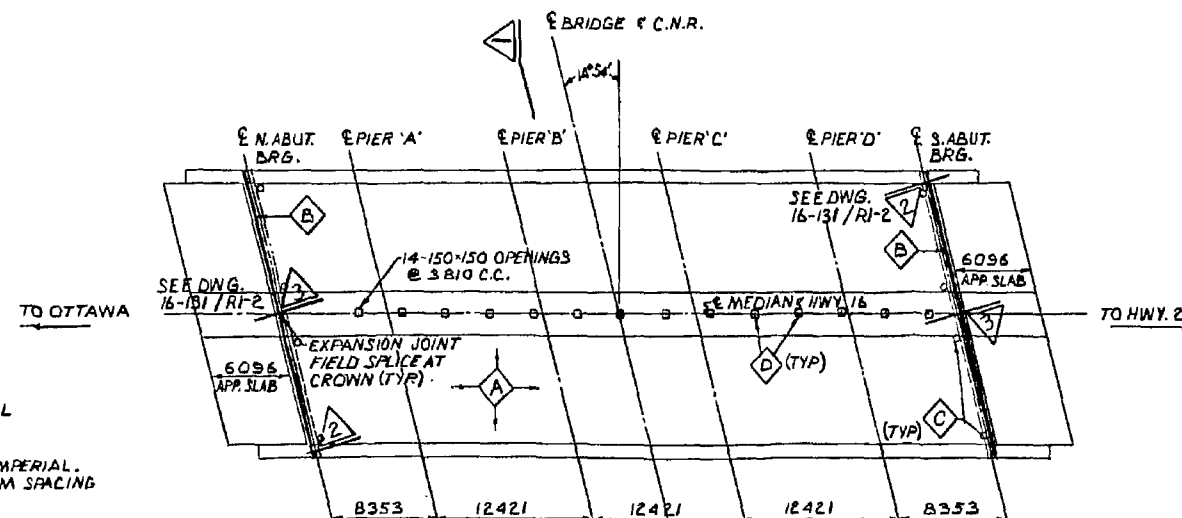
### LIST OF DRAWINGS

- 16-131/RI-1 - GENERAL ARRANGEMENT AND DETAILS
- 16-131/RI-2 - REMOVALS, STAGING, CURB AND MEDIAN DETAILS
- 16-131/RI-3 - EXPANSION JOINT DETAILS
- 16-131/RI-4 - JOINT ANCHORAGE AND ARMOURING
- 16-131/RI-5 - QUANTITIES STRUCTURE

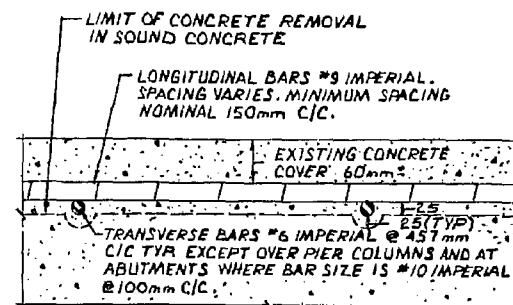
### REFERENCE DRAWINGS

D4334-1, 2, 4.

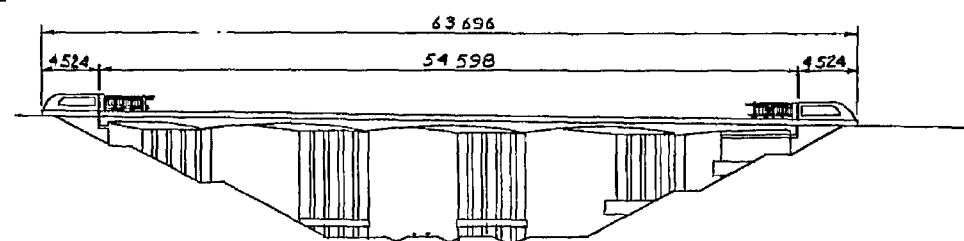
APPLICABLE STANDARD DRAWINGS			
STANDARD No	REVISION	DESCRIPTION	
OP.S.D. 508-02 DD 3951	1	BRIDGE DECK WATERPROOFING DRAINAGE OF ASPHALT WEARING SURFACE OF EXISTING BRIDGES	
REVISIONS			
DATE	BY	DESCRIPTION	DATE
DESIGN H.K. CHK G.J.	CODE	LOAD	DATE JULY 88
DRAWN W.S. CHK H.K. SITE 16-131	STRUCT	SCHEMER 1 DWG 1	



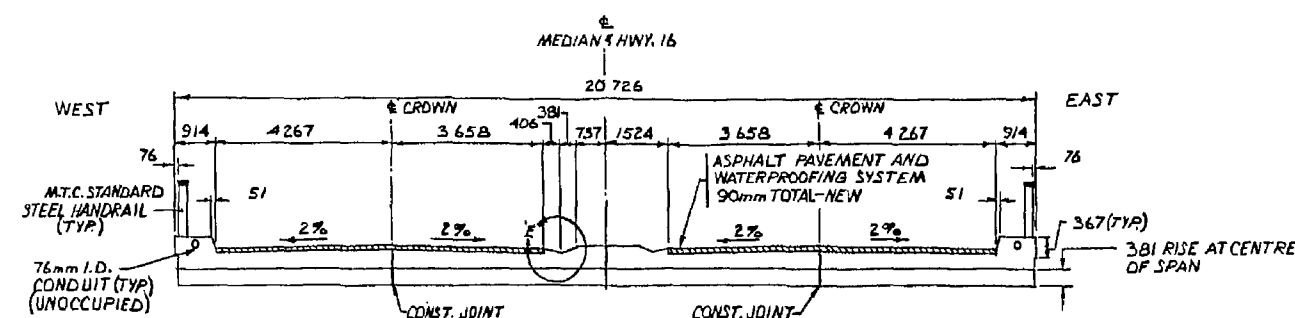
PLAN  
N.T.S.



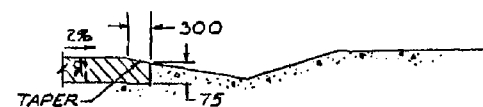
TYPICAL DECK TOP MAT REINFORCING  
AND CONCRETE REMOVAL DETAIL  
N.T.S.



ELEVATION  
N.T.S.



NOTE:  
FOR STAGING DETAIL  
SEE DWG. 16-131/RI-2

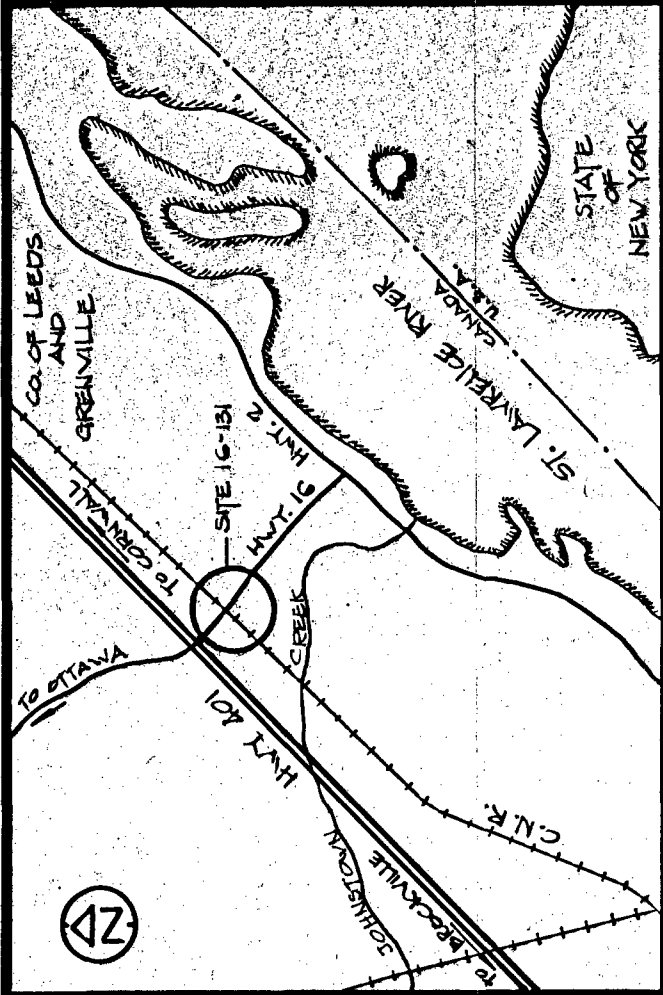


E-PAVING DETAIL AT MEDIAN (TYP.)  
N.T.S.



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING





KEY PLAN

LEGEND

- CORE LOCATION
- SAWN SAMPLE LOCATION
- DECK DRAIN
- ASPHALT THICKNESS (mm)
- SEALED CRACKS
- UNSEALED CRACKS
- CRACKS WITH EFFLORESCENCE
- REINFORCING CORROSION
- DAMP AREAS
- SEALED AREAS
- DELAMINATED AREAS
- SPALLED AREAS
- HONEYCOMBED AREAS
- PATCHED AREAS
- LIGHT
- MEDIUM
- SEVERE
- VS. VERY SEVERE

CORROSION POTENTIAL

- 0.0V TO -0.20V
- 0.20V TO -0.35V
- 0.35V TO -0.45V
- 0.45V TO -0.55V
- OVER -0.55V
- CORROSION POTENTIALS ( $\times 10^3$  VOLTS)
- VS SIGN OMITTED

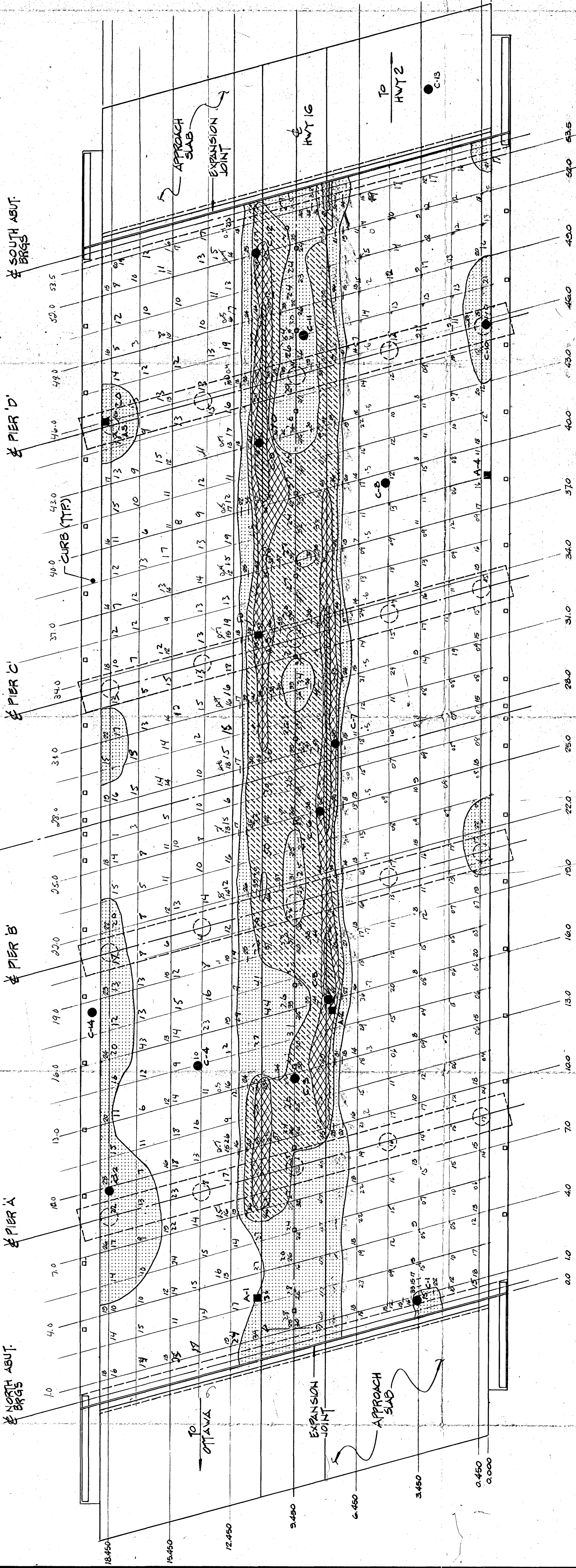
Giffels  
Giffels Associates Limited  
Consulting Engineers and Architects

MINISTRY OF TRANSPORTATION  
AND COMMUNICATIONS  
EASTERN REGION  
DISTRICT 9

CNR OVERHEAD  
HIGHWAY 16

CORROSION POTENTIALS	
SCALE 1:100	FIGURE 2
DRAWN A.P.	CHECKED M.V.
DATE SEPT. 1987	SITE 16-131

STRUCTURE



TEST = 20m (bottom in black)  
USE - DCV AT 20

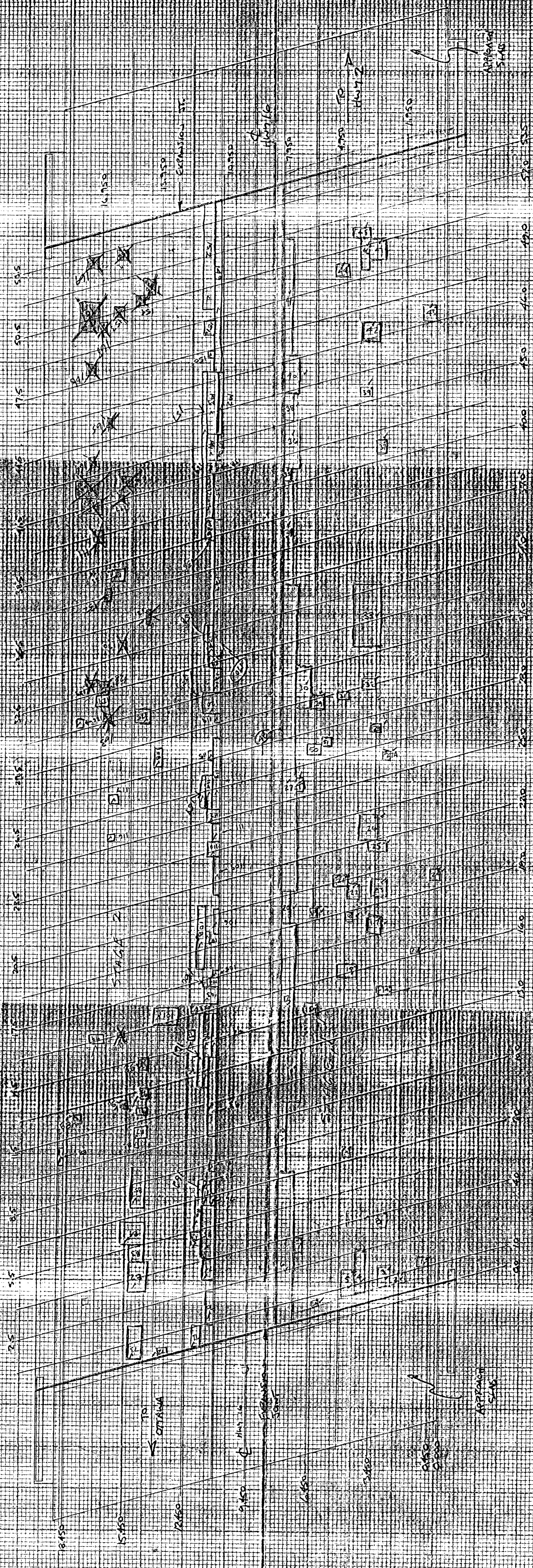
25.8 m<sup>2</sup> ± -35V EAST FIDE  
20.7 m<sup>2</sup> ± -35V WEST FIDE  
46.5 m<sup>2</sup> TOTAL (1987)  
+ 1.12 = 56.3 m<sup>2</sup> (1987)  
SAT 56 m<sup>2</sup>  
ACTUAL 99 m<sup>2</sup>

Actual Volume  
from Red  
black

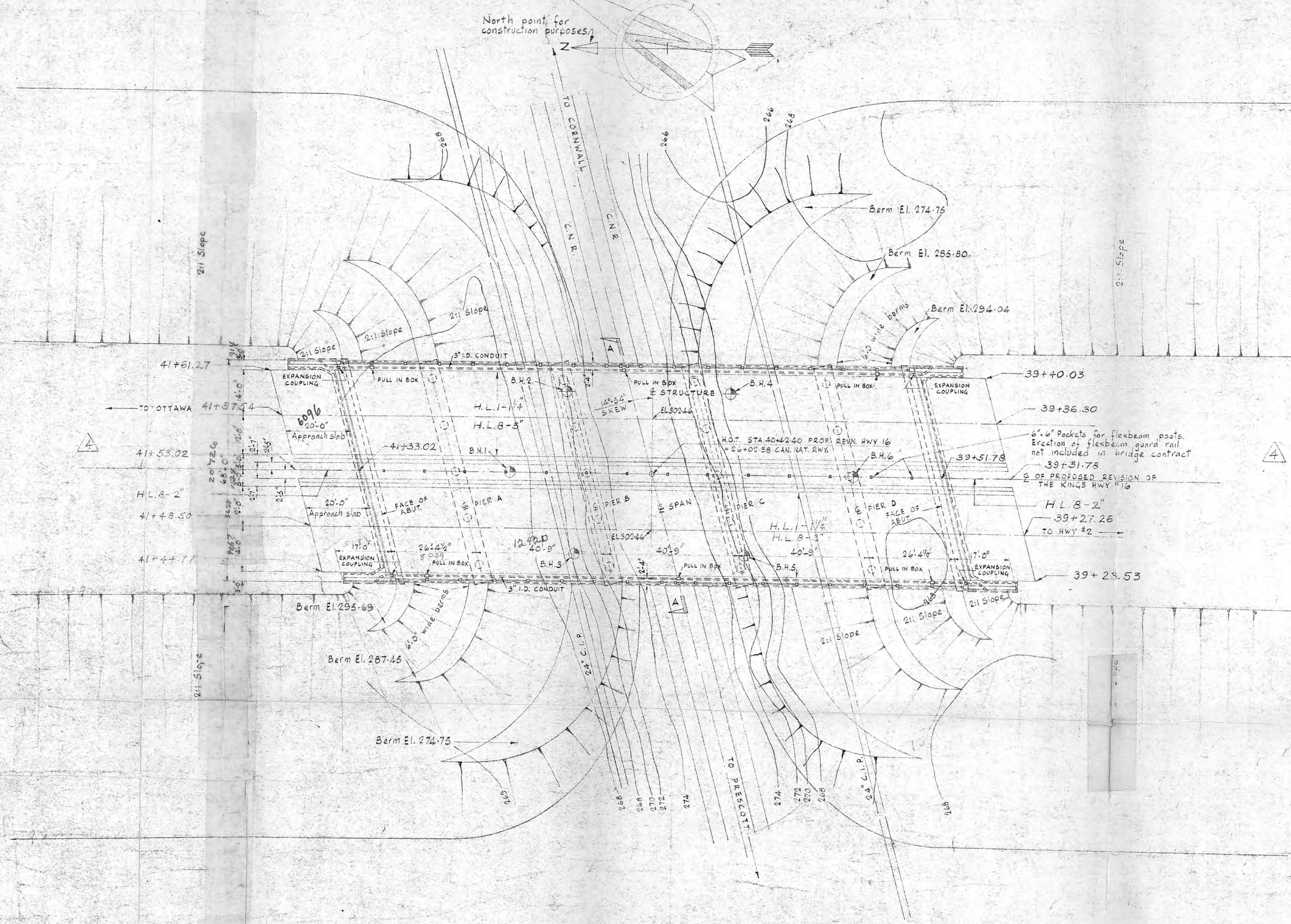


N  
V

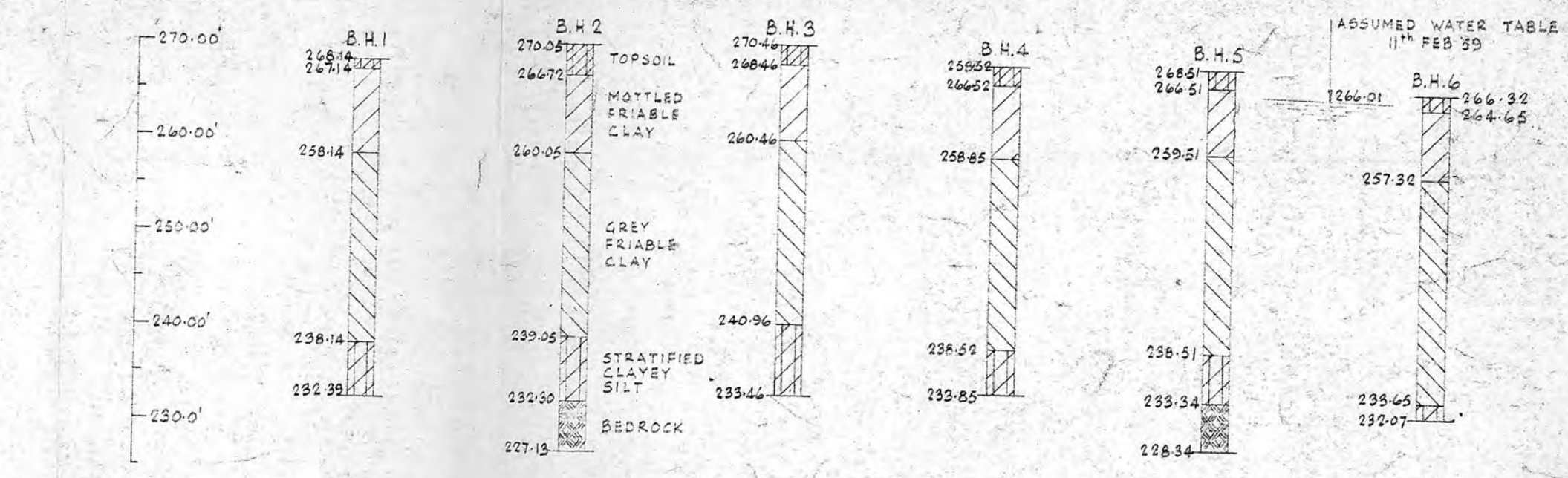
Notes: - Areas in Red are Sought for by US  
These were Spots near boundary  
included mapping these areas were  
paid with flat for points see 60.2



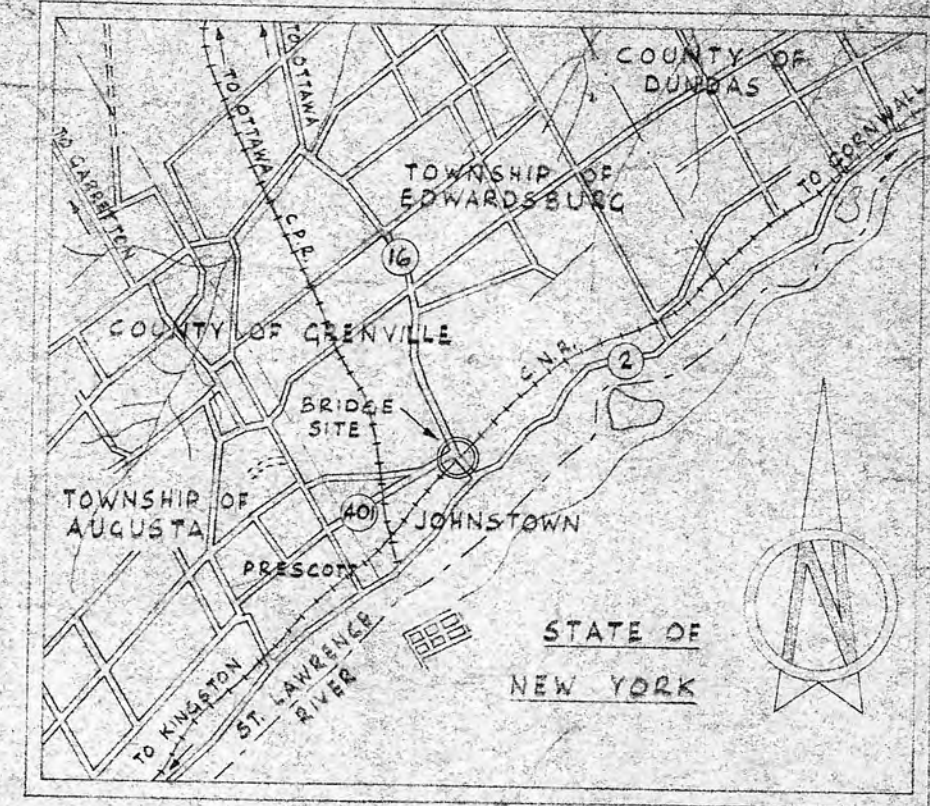




PLAN  
SCALE 1" TO 20'-0"



BOREHOLE DETAILS



KEY PLAN  
SCALE 1" TO 1 MILE

**NOTE TO DISTRICT ENGINEER**  
Concrete work on this structure must not be commenced until monuments to six control points have been erected and checked by the District Engineer.

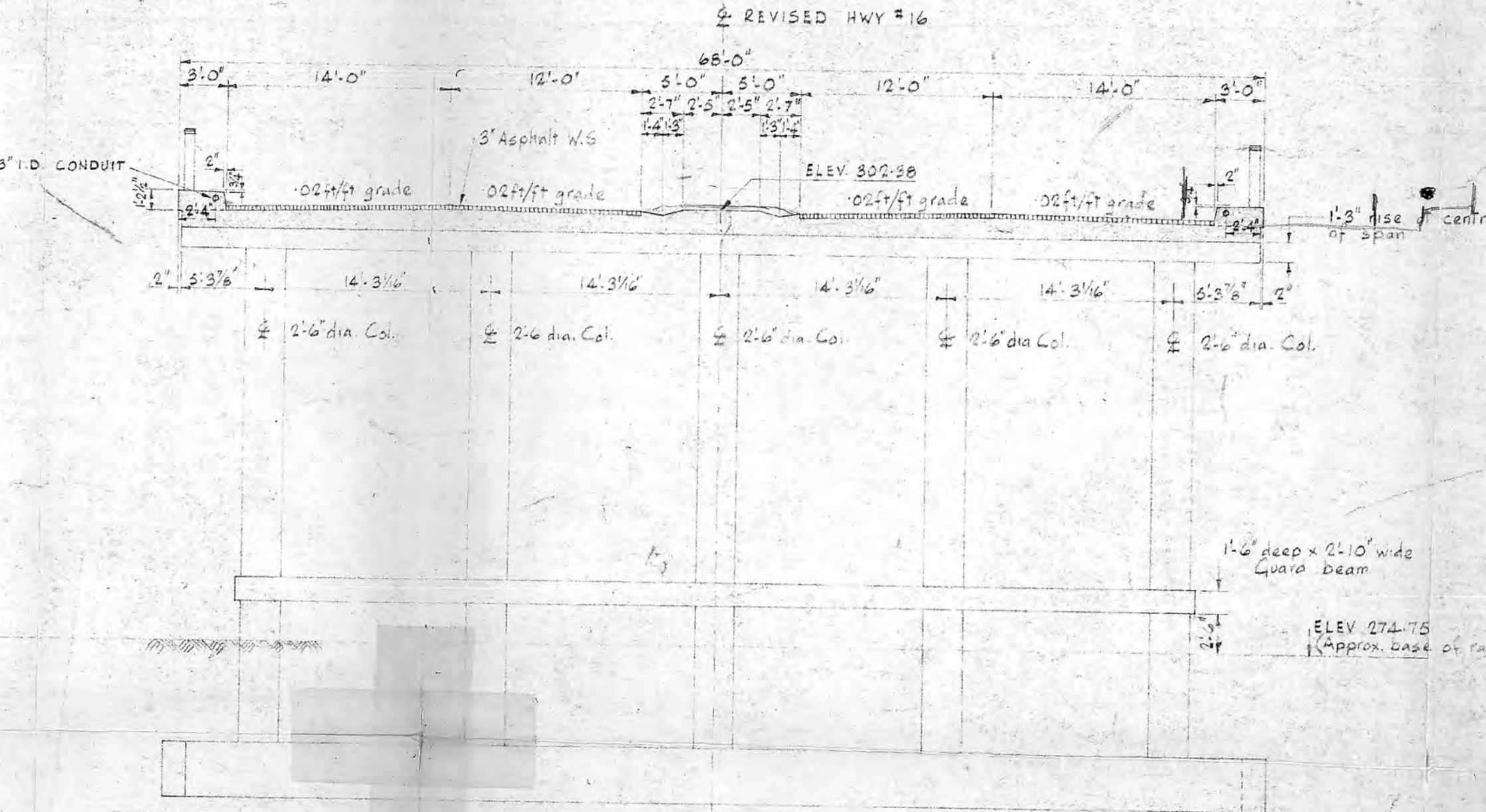
**NOTE TO CONTRACTOR**  
Structure to be built in accordance with Form No. 9 and the Special Provisions extra copies of which may be obtained from the District Engineer.  
All Construction Joints must be approved by the Bridge Engineer.

**CONCRETE MIX**  
Footings: min. strength at 28 days = 2500 p.s.i.  
Remainder: " " " " = 3000 p.s.i.  
Max. size of aggregate is 3/4" throughout structure.  
An approved admixture supplied by the Department will be added to all concrete as specified by the Engineer.

**BORING DATA**  
The complete soil investigation report BA 877 was examined at the District Office, File 200, Toronto. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.

**REINFORCING STEEL**  
Clear cover in Footings, Abutments and Columns = 3" Deck 2" End posts 1 1/2"

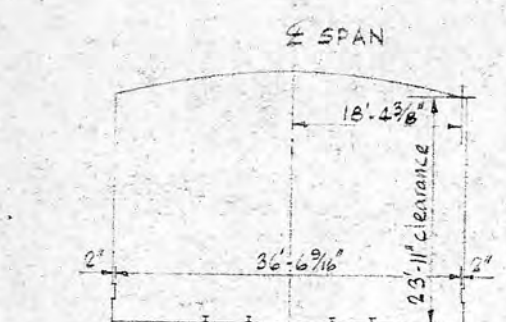
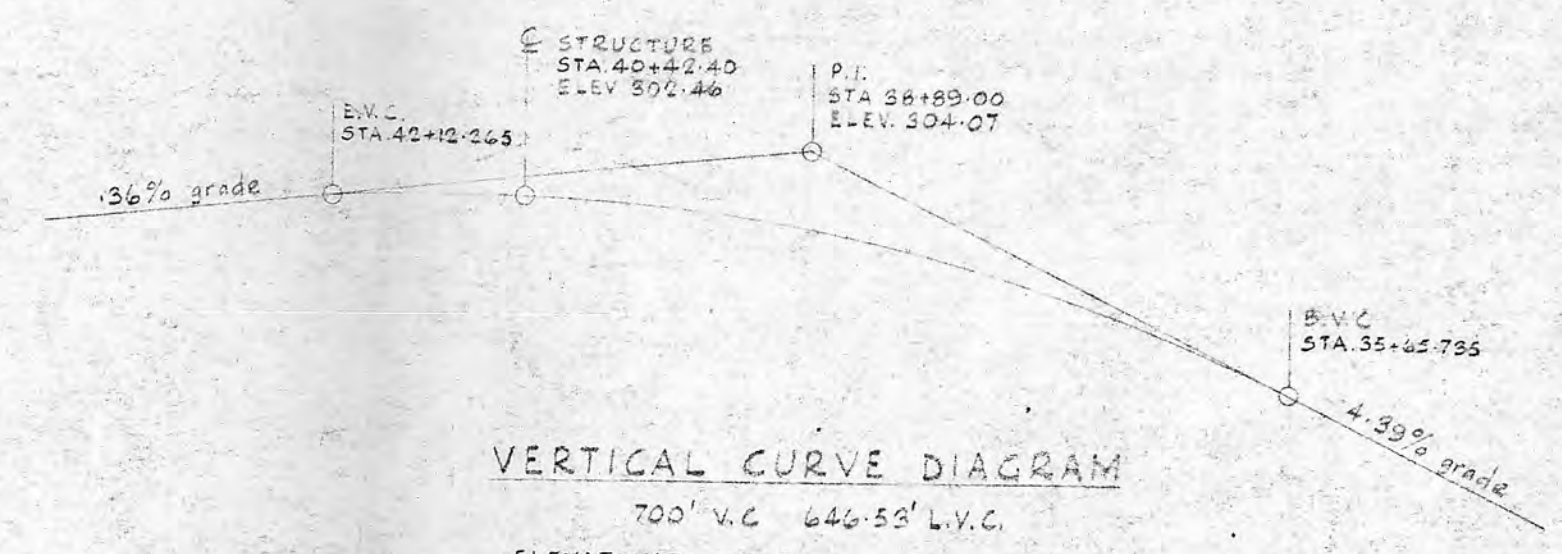
**CONSTRUCTION NOTES**  
All exposed edges to be chamfered 1"x1"



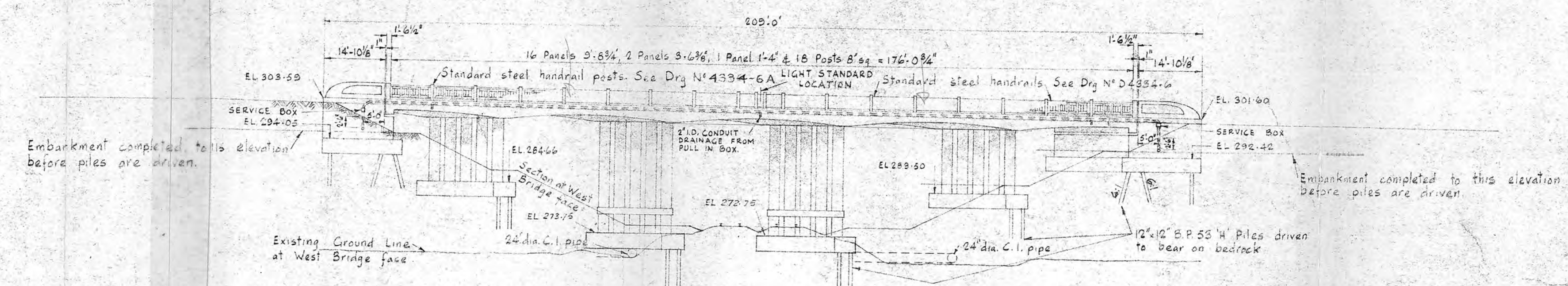
SECTION A-A  
SCALE 1/2" TO 1'-0"

**SKREW DATA**  
θ = 14° 54'  
SIN θ = 0.2571328  
COS θ = 0.9663761  
TAN θ = 0.2660794

**VERTICAL CURVE DATA**  
700' V.C. 646.58' L.V.C.  
ELEVATIONS ON TOP OF PAVEMENT



CLEARANCE DIAGRAM  
SECTION AT 90° TO TRACKS  
SCALE 1" TO 20'-0"



ELEVATION  
SCALE 1" TO 20'-0"

- LIST OF DRAWINGS**
- D 434-1 General Plan
  - " 2 Details of Footings for Abutment & Piers
  - " 3 Details of Abutments & Piers
  - " 4 Details of Deck & End Posts
  - " 5 Details of Expansion Bearings
  - " 6 Details for Panels
  - " 7 Reinforcing bar list
  - " 8 " " "
  - " 9 Track Structure Protection
  - " 10 Earth Fill Details
  - " 6A Details for Interior Posts
  - " 6B Details for Filler Panel

NO.	DATE	DESCRIPTION
1	10/1/59	REVISED
2	10/1/59	REVISED
3	10/1/59	REVISED
4	10/1/59	REVISED
5	10/1/59	REVISED
6	10/1/59	REVISED
7	10/1/59	REVISED
8	10/1/59	REVISED
9	10/1/59	REVISED
10	10/1/59	REVISED

Site 16-131  
W.P. 217-58

**DEPARTMENT OF HIGHWAYS-ONTARIO**  
BRIDGE OFFICE-TORONTO

**EDWARDSBURG TWP. C.N.R. OVERHEAD BRIDGE**

THE KING'S HIGHWAY No. 16 DIST. No. 9  
CO. GRENVILLE  
TWP. EDWARDSBURG LOT 274/26 CON. 1

**GENERAL PLAN**

APPROVED  
BRIDGE ENGINEER  
DESIGN ENGINEER

REVISION	DATE	BY	DESCRIPTION	REFERENCE PLANS
1	9/1/63	J.G.G.	ELEVATIONS ADDED AS OF CONSTRUCTION	E 875-1
2	2/1/64	P.B.M.	HANDRAIL PANELS & POSTS DWG. REV.	G 2674
3	8-2-60	D.R.H.	Number of handrail panels increased & electrical conduit added	C 1150
4	7-1-60	D.R.H.	Embankments changed from 3:1 to 2:1 - Rake of piles changed	E 875-1

DATE MAY 1959





## **Appendix G.**

### **Comparison of Foundation Alternatives**

## COMPARISON OF FOUNDATION ALTERNATIVES






	<i><b>Spread Footings</b></i>	<i><b>Driven Steel H-Piles</b></i>	<i><b>Concrete Caissons</b></i>
Description	<ul style="list-style-type: none"> <li>Foundation element founded on traditional shallow spread footings on native clay surface.</li> </ul>	<ul style="list-style-type: none"> <li>The abutments would be supported by a single row of steel H-piles driven to refusal on bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>A reinforced concrete column installed within an augered hole in the ground that derives axial resistance from end bearing</li> </ul>
Advantages	<ul style="list-style-type: none"> <li>Lower construction costs</li> </ul>	<ul style="list-style-type: none"> <li>Steel H-piles are well suited for use in integral abutment design.</li> <li>Requires less excavation than spread footings.</li> <li>Requires less concrete than caissons.</li> <li>Alleviate constraints related to limited construction space near CNR.</li> </ul>	<ul style="list-style-type: none"> <li>Moderate to high axial geotechnical resistance.</li> <li>Can handle obstructions.</li> <li>Suitable for semi-integral abutment design approach.</li> <li>Requires less excavation than spread footings.</li> <li>Alleviate constraints related to limited construction space near CNR.</li> </ul>
Disadvantages	<ul style="list-style-type: none"> <li>Significant excavations required, particularly at abutments beneath portions of existing embankments, adjacent to existing structure.</li> <li>Risk of settlement</li> <li>Low Bearing Capacity</li> </ul>	<ul style="list-style-type: none"> <li>Has potential to encounter obstructions in the fill.</li> <li>Lower geotechnical resistance than caissons.</li> </ul>	<ul style="list-style-type: none"> <li>Not compatible with integral abutment design approach.</li> <li>Likely requires concrete to be placed using tremie techniques.</li> <li>Temporary steel casing required to keep hole open during drilling.</li> <li>The base of the caisson would need to be inspected to ensure end bearing capacity.</li> </ul>
Risks / Consequences	<ul style="list-style-type: none"> <li>Settlement Concerns</li> </ul>	<ul style="list-style-type: none"> <li>Difficult advancing through possible obstructions in fill</li> </ul>	<ul style="list-style-type: none"> <li>Increased concrete volume may be required if additional soil is pulled in from sidewall while advancing through obstructions.</li> <li>Position and alignment could be affected by obstructions.</li> </ul>
Relative Cost	<ul style="list-style-type: none"> <li>Low</li> </ul>	<ul style="list-style-type: none"> <li>Moderate</li> </ul>	<ul style="list-style-type: none"> <li>High</li> </ul>
Conclusion	<ul style="list-style-type: none"> <li>Not Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Recommended for all foundations</li> </ul>	<ul style="list-style-type: none"> <li>Feasible, but not recommended</li> </ul>

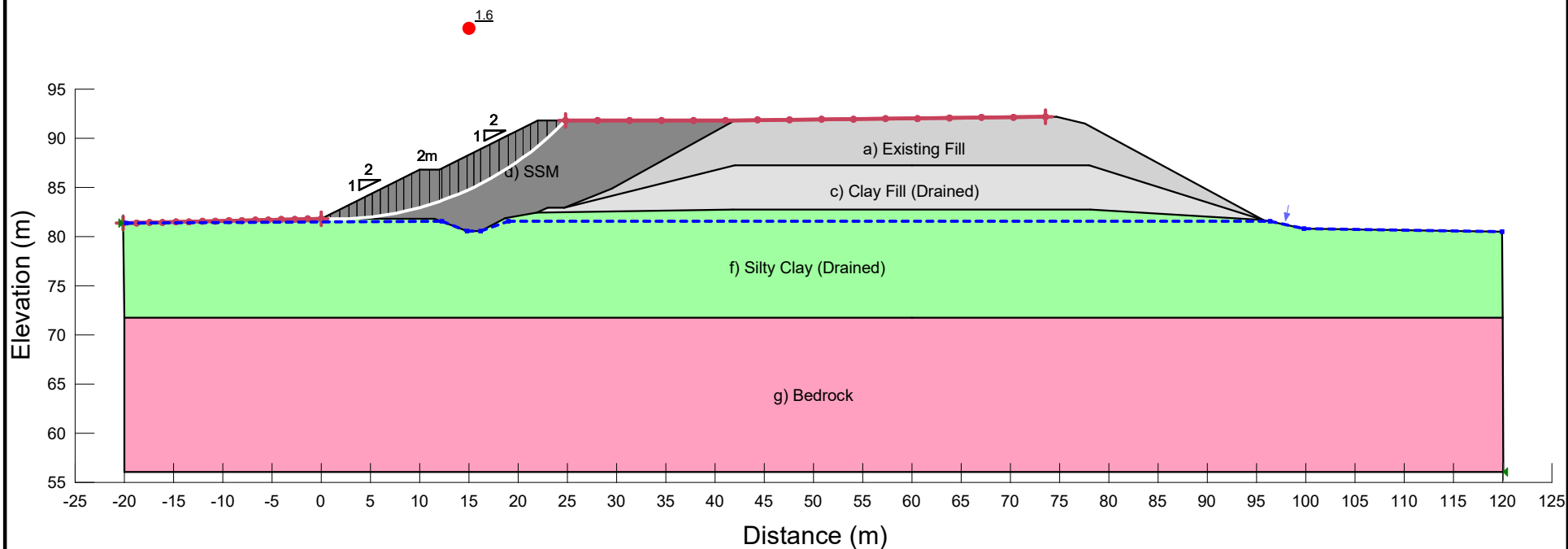




## **Appendix H.**

### **Slope Stability Analysis Figures**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20	0	30
	c) Clay Fill (Drained)	Mohr-Coulomb	17.5	10	29
	d) SSM	Mohr-Coulomb	20	0	30
	f) Silty Clay (Drained)	Mohr-Coulomb	17.5	7	29
	g) Bedrock	Bedrock (Impenetrable)			




	Project			Additional Details	
	Hwy 16 Overpass at CNR			Name: 1. South Embankment - SSM Slope	
	Analysis			Comments:	
	1.1) Permanent - Long Term			Method: Morgenstern-Price, Half-Sine	
Seismic Coefficient		Last Run		Minimum Slip Surface Depth: 1.52 m Entry: (24.84879, 91.8) m, Exit: (-0.0012599999, 81.799971) m Center: (1.398936, 114.19666) m, Radius: 32.426935 m	
H: g, V: g		2023/08/28, 01:51:48 PM			
		1:625			

Figure H1.1

**Figure H1.1**



## Project

Hwy 16 Overpass at CNR

## Analysis

### 1.2) Temporary (traffic)

Seismic Coefficient

H: g, V: g

Last Run

2023/08/28, 01:51:49 PM

Scale

1:625

### Additional Details

Name: 1. South Embankment - SSM Slope

Comments:






Method: Morgenstern-Price, Half-Sine

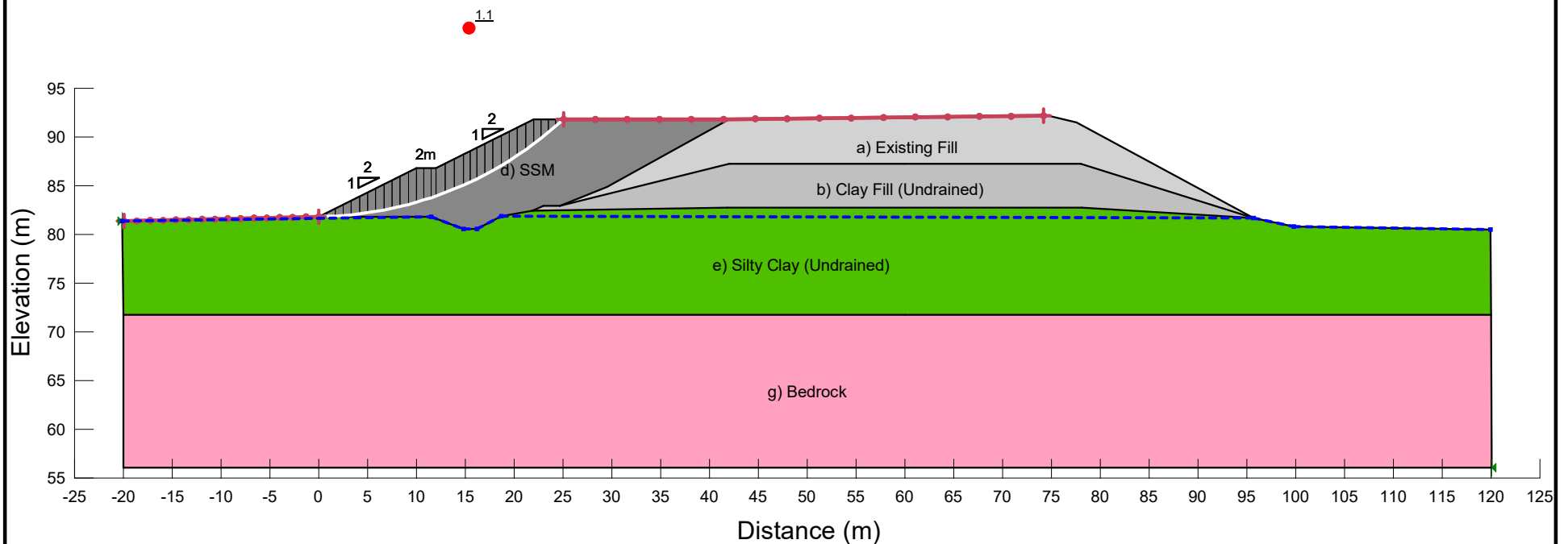
Minimum Slip Surface Depth: 1.52 m

Entry: (28.50154, 91.8) m, Exit: (-0.0015699967, 81.799964) m

Center: (-0.67503869, 129.34079) m, Radius: 47.545594 m

### Figure H1.2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	30
	b) Clay Fill (Undrained)	Undrained (Phi=0)	17.5	120		
	d) SSM	Mohr-Coulomb	20		0	30
	e) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	g) Bedrock	Bedrock (Impenetrable)				









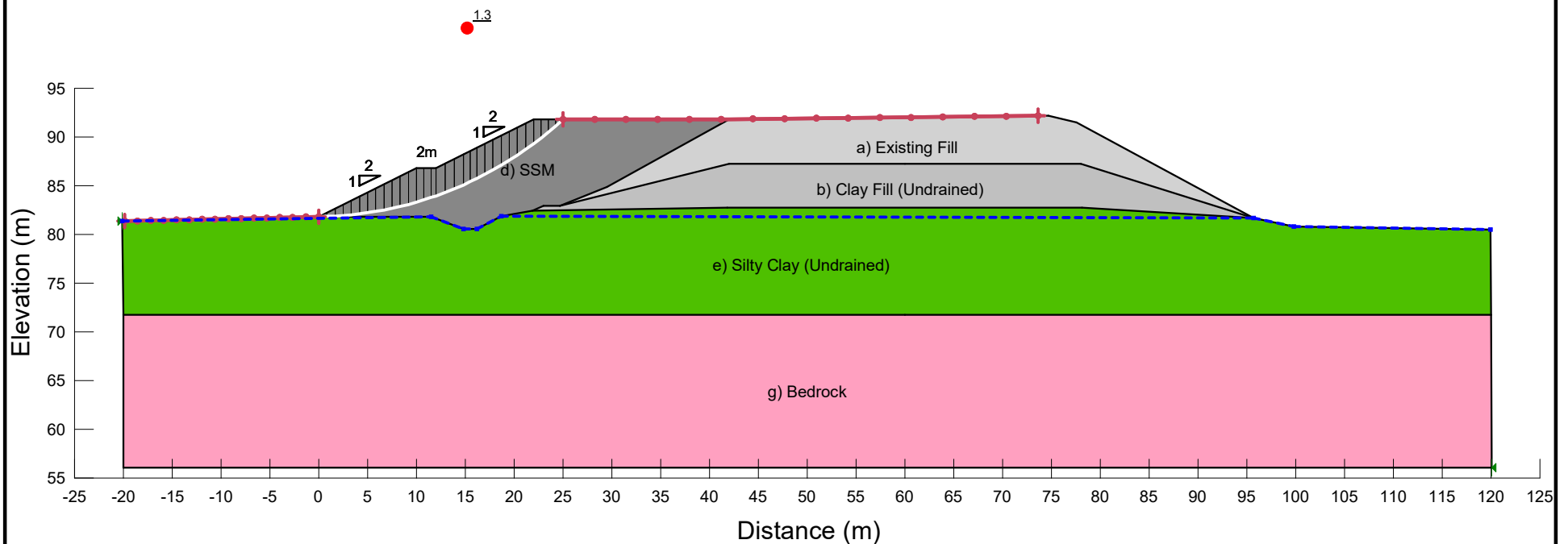
 <b>THURBER</b>	Project			Additional Details		
	Hwy 16 Overpass at CNR			Name: 1. South Embankment - SSM Slope		
	Analysis			Comments:		
	1.3) Pseudo-Static (2,475 yr EQ)			Method: Morgenstern-Price, Half-Sine		
	Seismic Coefficient			Minimum Slip Surface Depth: 1.52 m		
H: 0.14g, V: g		Last Run		Entry: (25.04238, 91.8) m, Exit: (-0.002139994, 81.799951) m		
		2023/08/28, 01:51:50 PM		Center: (-0.43291095, 119.24006) m, Radius: 37.442587 m		
		Scale				
		1:625				

Figure H1.3

**Figure H1.3**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	30
	b) Clay Fill (Undrained)	Undrained (Phi=0)	17.5	120		
	d) SSM	Mohr-Coulomb	20		0	30
	e) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	g) Bedrock	Bedrock (Impenetrable)				










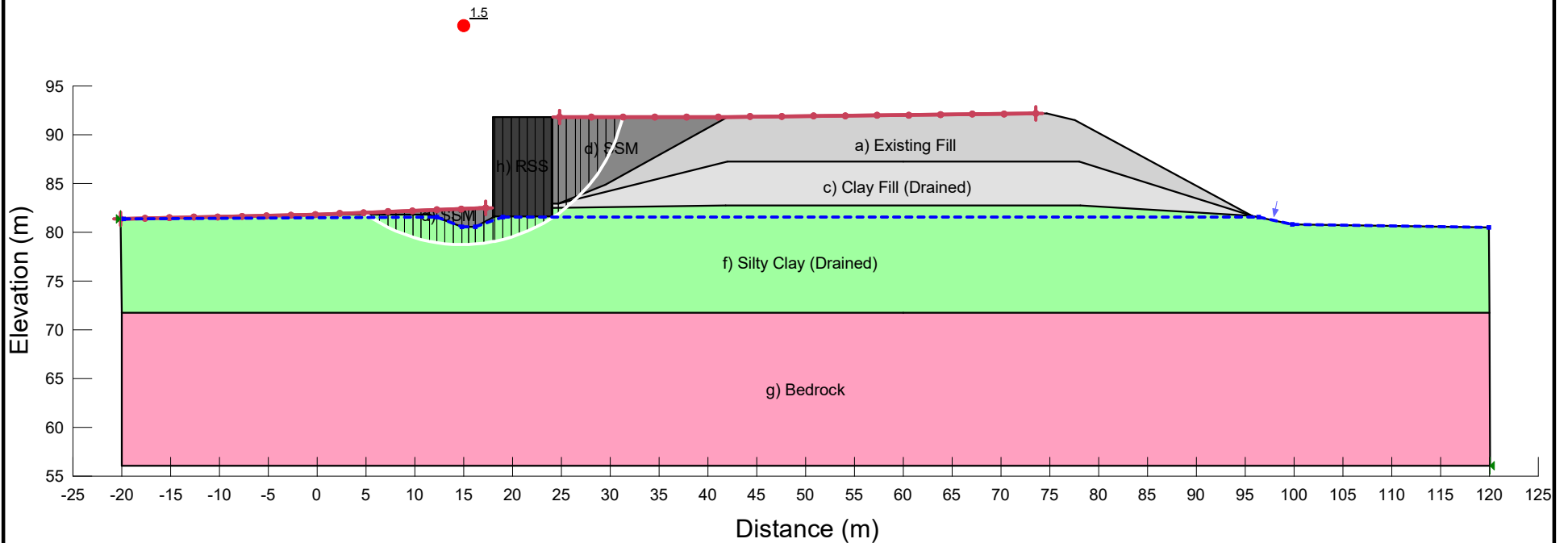
 <b>THURBER</b>	Project			Additional Details Name: 1. South Embankment - SSM Slope Comments: Method: Morgenstern-Price, Half-Sine Minimum Slip Surface Depth: 1.52 m Entry: (24.99071, 91.8) m, Exit: (-0.0016199933, 81.799963) m Center: (-0.46921835, 119.19933) m, Radius: 37.402286 m
	Hwy 16 Overpass at CNR			
	Analysis			
	1.4) Pseudo-Static (475 yr EQ)			
Seismic Coefficient	Last Run		Scale	
H: 0.06g, V: g	2023/08/28, 01:51:51 PM		1:625	

Figure H1.4

**Figure H1.4**







Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20	0	30
	c) Clay Fill (Drained)	Mohr-Coulomb	17.5	10	29
	d) SSM	Mohr-Coulomb	20	0	30
	f) Silty Clay (Drained)	Mohr-Coulomb	17.5	7	29
	g) Bedrock	Bedrock (Impenetrable)			
	h) RSS	Mohr-Coulomb	22.8	250	35

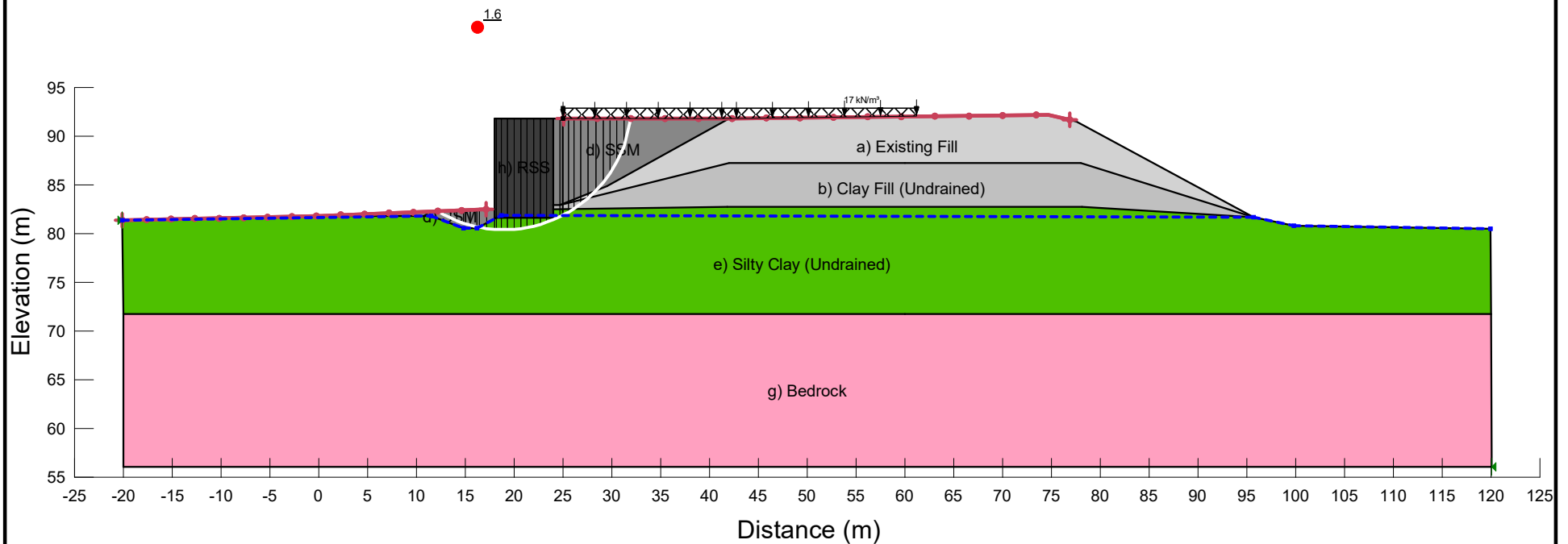


Project		
Hwy 16 Overpass at CNR		
Analysis		
2.1) Permanent - Long Term		
Seismic Coefficient	Last Run	Scale
H: g, V: g	2023/08/28, 01:51:43 PM	1:625

Additional Details  
 Name: 2. South Embankment - RSS  
 Comments:  
 Method: Morgenstern-Price, Half-Sine  
 Minimum Slip Surface Depth: 1.52 m  
 Entry: (31.346694, 91.8) m, Exit: (4.7884081, 81.986216) m  
 Center: (14.809212, 95.710898) m, Radius: 16.993629 m

**Figure H2.1**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	30
	b) Clay Fill (Undrained)	Undrained (Phi=0)	17.5	120		
	d) SSM	Mohr-Coulomb	20		0	30
	e) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	g) Bedrock	Bedrock (Impenetrable)				
	h) RSS	Mohr-Coulomb	22.8		250	35










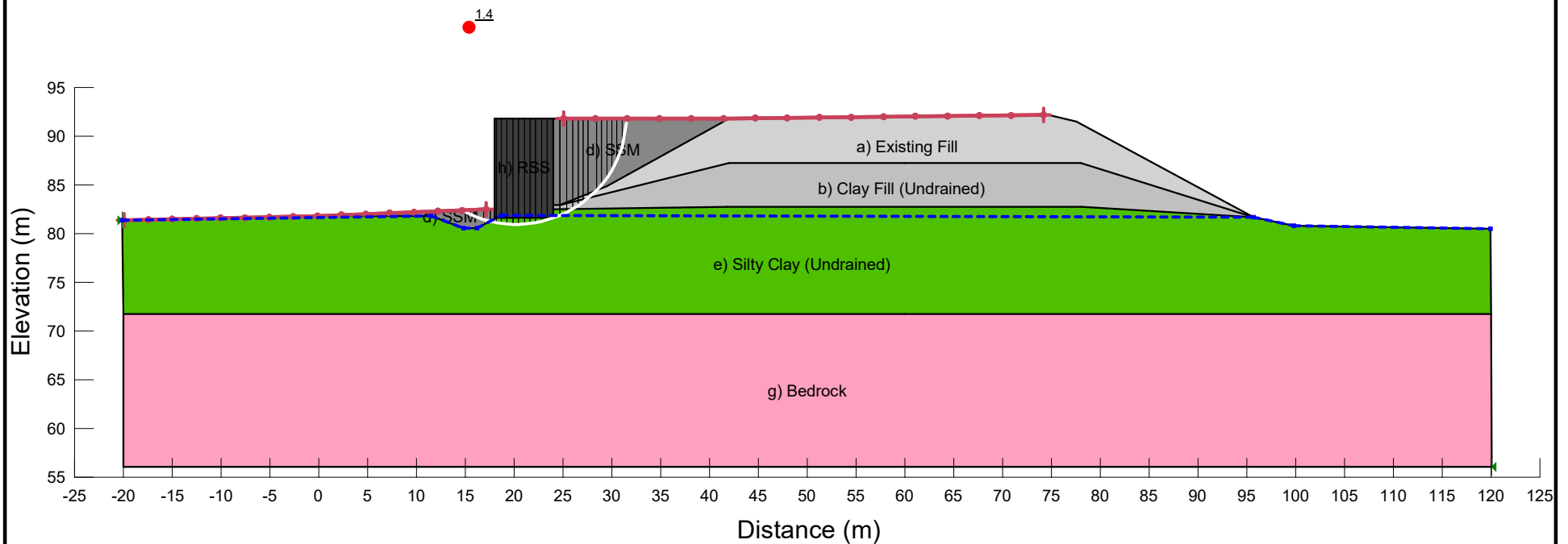
 <b>THURBER</b>	Project			Additional Details		
	Hwy 16 Overpass at CNR			Name: 2. South Embankment - RSS		
	Analysis			Comments:		
	2.2) Temporary (traffic)			Method: Morgenstern-Price, Half-Sine		
	Seismic Coefficient		Last Run	Minimum Slip Surface Depth: 1.52 m		
H: g, V: g		2023/08/28, 01:51:44 PM	Entry: (31.9607, 91.8) m, Exit: (12.166486, 82.273141) m			
Scale			Center: (18.944512, 93.517171) m, Radius: 13.12897 m			
1:625						

Figure H2.2

**Figure H2.2**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	30
	b) Clay Fill (Undrained)	Undrained (Phi=0)	17.5	120		
	d) SSM	Mohr-Coulomb	20		0	30
	e) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	g) Bedrock	Bedrock (Impenetrable)				
	h) RSS	Mohr-Coulomb	22.8		250	35




 <b>THURBER</b>	Project			Additional Details		
	Hwy 16 Overpass at CNR			Name: 2. South Embankment - RSS		
	Analysis			Comments:		
	2.3) Pseudo-Static (2,475 yr EQ)			Method: Morgenstern-Price, Half-Sine		
	Seismic Coefficient			Minimum Slip Surface Depth: 1.52 m		
	H: 0.14g, V: g			Entry: (31.595627, 91.8) m, Exit: (14.686589, 82.371145) m		
	Last Run			Center: (20.231785, 92.302946) m, Radius: 11.374967 m		
	2023/08/28, 01:51:45 PM					
	Scale					
	1:625					

Figure H2.3

**Figure H2.3**





## **Appendix I.**

### **List of Referenced Specifications**



1. The following Special Provisions and OPSS Documents are referenced in this report:

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 903	Construction Specification for Deep Foundations

2. Contract Provision – Vibration and Settlement Limits for Local Infrastructure

The Contractor is advised that a CNR railway line runs below the Highway 16 Overhead. CNR must be contacted prior to any construction activities to determine vibration limits and maximum allowable particle velocity and acceptable settlement limits during construction activities.