Golder Associates

TECHNICAL MEMORANDUM

DATE June 16, 2015 **PROJECT No.** 13-1184-0171

TO Dennis Baxter, P.Eng.
AECOM **GEOCRES No.** 30M14-030

CC David Leblanc, P.Eng.

FROM André Bom, P.Eng.
Kevin Bentley, P.Eng.

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DESKTOP STUDY FOUNDATION ASSESSMENT
SITE NO. 37-1007/1&2 NEILSON ROAD OVER MILNER AVENUE BRIDGE REHABILITATION
PART OF HIGHWAY 401/404 BRIDGE REHABILITATIONS
TORONTO, ONTARIO, GWP#2029-13-00

This technical memorandum summarizes the results of a desktop study of available subsurface information and provides preliminary foundation recommendations for the existing structure at Site No. 37-1007/1&2, which carries Neilson Road over Milner Avenue. The foundations recommendations provided in this technical memorandum are intended to support the structural design of the proposed rehabilitation being considered at this site.

The proposed rehabilitation will include the construction of semi-integral abutments and the replacement of the approach slabs, parapet/barrier walls and railings, sidewalks, asphalt and repair deck and girders. The draft General Arrangement (GA), Drawing R1-1 (dated March 2015), was provided to us by AECOM and is included in Drawing 1 following the text of this technical memorandum.

1.0 REVIEW OF AVAILABLE INFORMATION

The following report and design drawings have been obtained and reviewed to carry out the required foundation engineering assessment for the bridge structure:

- MTO GEOCRES No. 30M14-030: Report titled "*Foundation Investigation Report for Proposed Neilson Road Overpass at Milner Avenue Extension (Neilson Road and Hwy. 401 Interchange), Scarborough Twp., District No. 6, W.P. 43-69-04*", prepared by the MTO Foundations Section, dated February 16, 1973.
- GA Drawing 1 and Footing Layout and Reinforcement Drawing 3 prepared by the Albery, Pullerits, Dickson & Associates (APD), titled "*Milner Avenue Overpass*", Site No. 37-1007, undated but stamped "Received Nov. 17, 1978".

Based on the four boreholes advanced in 1972, the original ground surface at the site was between approximately Elevations 161.4 m and 161.8 m.

Golder visited the site on May 7 and 14, 2015, to perform a cursory review of the structure from a foundations perspective. There were no visual signs of obvious foundation-related issues in terms of foundation and embankment stability and settlement at the time of our field visit. It was noted that the expansion joint gaps between the abutment walls and retaining walls, which consist of separate foundation units according to the design drawings, varied slightly at each quadrant of the bridge. The gap at the expansion joints near the bottom of the structure was measured to be about 15-25 mm wide at the southwest, northwest and southeast quadrants, and 25-30 mm at the northeast quadrant. The gap width appeared to be consistent along the length of the joint at each quadrant except at the southwest quadrant where the gap width appeared to increase slightly up to the bridge deck level. Polystyrene was observed to be present within the gaps and was not tight between the retaining wall and abutment wall concrete (i.e. an air gap of several millimetres was typically observed) at the time of the site visit.

2.0 SUBSURFACE CONDITIONS

Four boreholes (Boreholes No. 1 to 4) were advanced near the four corners of the bridge as part of the previous investigation in 1972, as shown on Drawing 1, attached. A copy of the Record of Boreholes No. 1 to 4 is also attached, and a stratigraphic profile along Neilson Road and cross-sections at the abutments are presented on Drawing No. 72-11130A from the Foundation Investigation Report.

The GEOCRES sourced borehole locations shown on Drawing 1 were obtained from converting the co-ordinates shown on the Record of Borehole. Thus, borehole locations shown on Drawing 1 are considered approximate.

Based on Borehole No. 1 to 4 and the stratigraphic profile and cross-sections, the subsurface conditions at the bridge site are similar at the south and north abutment and consist of:

- A 6.1 m to 7.9 m thick deposit of glacial till consisting of a heterogeneous mixture of clayey silt with sand and gravel. Standard Penetration Test (SPT) 'N'-values in the glacial till range from 16 blows per 0.3 m of penetration to 175 blows per 0.25 m of penetration, suggesting a very stiff to hard consistency. Assuming a base of footing at Elevation 158.5 m (520 ft) as per the Foundation Investigation Report and Drawings 1 and 3, the SPT 'N'-values below the footing level are greater than 30 blows per 0.3 m of penetration.
- The glacial till is underlain by a deposit of silty sand to the bottom of the boreholes which was not fully penetrated after terminating the boreholes between 12.7 m and 29.0 m below original ground surface (Elevations 148.8 m and 132.5 m, respectively). SPT 'N'-values in the silty sand deposit are typically greater than 50 blows per 0.3 m of penetration, indicating a very dense relative density, except at the following locations:
  - Boreholes 1 and 2 (of northeast and southeast corners of the south and north abutment, respectively): the uppermost 1 m of the deposit, from about Elevation 155.5 to 154.5 (510 ft to 507 ft) is compact with SPT 'N'-values of 43 blows and 27 blows per 0.3 m of penetration;
  - Boreholes 1 and 2: an approximately 2.5 m thick zone of loose relative density silty sand was encountered from about Elevations 153.0 m to 150.3 m (502 ft to 493 ft) with SPT 'N'-value of 6 blows per 0.3 m of penetration; and
  - Borehole 1: and approximately 1.2 m thick zone of dense relative density silty sand was encountered from about Elevations 150.0 m to 148.8 m (492 ft to 488 ft) with an SPT 'N'-value of 36 blows per 0.3 m of penetration.
- The unstabilized water level in the open boreholes upon completion of drilling (December 1972) ranges from about 0.9 m to 5.8 m below ground surface (Elevations 160.5 m to 156.0 m). The groundwater level(s) at the bridge site should be expected to have fluctuated seasonally in response to changes in precipitation and snow melt and local construction activities, and should be expected to be higher during the spring season or during periods of heavy precipitation. Considering the groundwater level

measurements were taken in 1972, it is likely that the static groundwater level in the area has changed over the past 50 years.

3.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

We understand that preliminary geotechnical resistance values for the existing foundations at this structure are required for the proposed rehabilitation design to proceed. We further understand that the rehabilitation works proposed do not envisage the need for new foundations or widening of existing footings to accommodate the anticipated additional design loads, if the foundation subsoils provide sufficient geotechnical resistances presently estimated to be 370 kPa at Serviceability Limits State (SLS) for 25 mm of settlement.

3.1 ASSESSMENT OF EXISTING FOUNDATIONS

Based on the original GA Drawing 1 and Footing Layout Drawing 3 by APD, the existing bridge is a single-span structure with a total length of approximately 15.2 m. The north and south abutments are shown to consist of reinforced concrete supported on conventional strip footings. The original GA depicts the top of the footing to be at Elevation 159.4 m and based on a 0.9 m thick footing, and as shown on Drawing 3, the founding level is at Elevation 158.5 m, consistent with the recommended founding level in the GEOCRES Foundation Investigation Report. The founding elevation is about 3 m below the original ground surface shown on the Record of Borehole Sheets and the bridge abutment footings are about 5.8 m (19 ft) wide based on the original Foundation Layout Drawings. The foundation design bearing capacities are not indicated on the original drawings, however, the GEOCRES Foundation Investigation Report specifies an allowable bearing pressure of 3 tsf (300 kPa) may be used for design.

Retaining walls extend from each of the four corners of the bridge, perpendicular to the abutment stem walls, to a distance of 10.7 m and 14.3 m from the north and south abutment stem walls, respectively. The retaining walls are founded on 5.0 m wide strip/spread footings at the same founding elevation as the bridge footings.

For 5 m to 6 m wide spread footings founded on the very stiff to hard glacial till, underlain by very dense silty sand, interlayered with an approximately 2.5 m thick layer of loose silty sand between about Elevations 153.0 m and 150.5 m at the East corner of both abutments, the factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical reaction at SLS for 25 mm of settlement provided below may be used for preliminary design.

Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS[1]	
	East Half of Abutment Footings	West Half of Abutment Footings
600 kPa	300 kPa	500 kPa

[1] For 25 mm of settlement

The geotechnical resistance values provided above and in Section 3.2 are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the CHBDC.

3.2 REQUIRED FOUNDATION RESISTANCE FOR REHABILITATION OPTION

Based on the foundation evaluations carried out by AECOM (as per Section 6 of the 2006 Canadian Highway Bridge Design Code), the required foundation resistances for the existing and proposed future bridge loads (and % difference) are provided below, together with the preliminary design geotechnical resistances identified in Section 3.1.

Foundation Element	Ultimate Limit State			Serviceability Limit State[1]			
	Existing Load/ Required Resistance (kPa)	Proposed Load/Require d Resistance (kPa) (% difference)	Preliminary Design Geotechnical Axial Resistance (Factored ULS) (kPa)	Existing Load/ Required Resistance (kPa)	Proposed Load / Required Resistance (kPa) (% difference)	Preliminary Design Geotechnical Reaction (SLS) (kPa)	
						East Half	West Half
North Abutment	369	373 (+1%)	600	343	346 (+1%)	300[2]	500[2]
South Abutment	319	322 (+1%)	600	281	284 (+1%)	300[2]	500[2]

Note: [1] For 25 mm of settlement
[2] See further discussion below

As presented above, the preliminary design geotechnical resistances at ULS are considered adequate for the support of the existing and proposed new loads at the abutments for the proposed rehabilitation.

The recommended geotechnical reaction at SLS of 300 kPa is consistent with the working stress provided in MTO's 1973 Foundation Investigation Report. The SLS value is governed by the stratum of loose sand located at a depth equivalent to approximately one footing width below the bottom of the footing. Since construction of the structure in the 1970's, it is likely that the bridge loading on the existing 5.8 m wide footings influenced the loose zone of silty sand, however the increase in the relative density of this layer, and its load-carrying capacity is not quantifiable without further subsurface information.

Based on a review of the GEOCRES report and original design drawings, and based on our site review of the existing bridge from a geotechnical perspective, it is considered that an increase of 1% in loading would not impact the existing foundations significantly, especially the western half of the structure. However, as the new design loadings will increase relative to the existing design loads, but which are greater than the originally recommended design bearing capacity for the foundations at the north abutment, the eastern half of the north abutment may not perform adequately or may require more frequent maintenance in the future depending on the actual amount of differential settlement and tolerable limits of movement of the new bridge structure and adjacent retaining walls. Based on a 1% increase in proposed serviceability loading, it is logical to assume that the founding soils have already experienced and are currently experiencing the majority of the expected total load. Therefore, it is estimated that less than 10 mm of differential settlement between and within foundation elements would occur under the new loading compared to the existing loading. Temporary additional elastic settlements/movements may occur if the foundations are significantly unloaded and reloaded depending on construction staging.

If the expected settlements are not considered to be tolerable (i.e., potentially not satisfying the SLS requirements) new foundation elements may be required or the existing foundations may need to be modified or supplemented to resist the additional loads. If this is the case, it is recommended that a supplementary Foundation Investigation be carried out consistent with the Terms of Reference outlined in "*Attachment 6.8 - Minimum Requirements for Foundation Engineering Applications*" contained in the original MTO Request for Proposal.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 TEMPORARY EXCAVATION/DEWATERING

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects (OHSA). Provided that the fill materials comprising the backfill adjacent and behind the abutments and immediate approaches is of granular composition and were properly compacted during original construction, they are assumed to be classified as Type 3 soils, however this should be confirmed during construction. As such, temporary open cut slopes within the fill materials should be made no steeper than 1H:1V. Localized slope flattening may be required within cohesionless fill soils near the ground surface. Perched water within the fill soils and above the native till soils should be expected. Excavations for construction of new foundations extending into the glacial till and silty sand above the groundwater level should also be excavated no steeper than 1H:1V.

Excavations that penetrate below the groundwater level or in areas where perched water levels are present will require dewatering methods such as pumping from properly filtered sumps if placement of concrete in the dry is required. Surface water runoff should be directed away from the excavations at all times.

4.2 TEMPORARY SHORING

Temporary shoring may be required if there is insufficient space for open cut excavation to be made within or adjacent to the existing structure or near utilities. Any temporary excavation support system should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any adjacent utilities can tolerate this magnitude of deformation.

5.0 CLOSURE

This technical memorandum was prepared by Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact and Principal of Golder, conducted an independent review of the technical memorandum. We trust the above information meets with your current requirements; should you have any questions, please do not hesitate to contact us.

Yours very truly,

GOLDER ASSOCIATES LTD.

André Bom, P.Eng.
Geotechnical Engineer, Associate

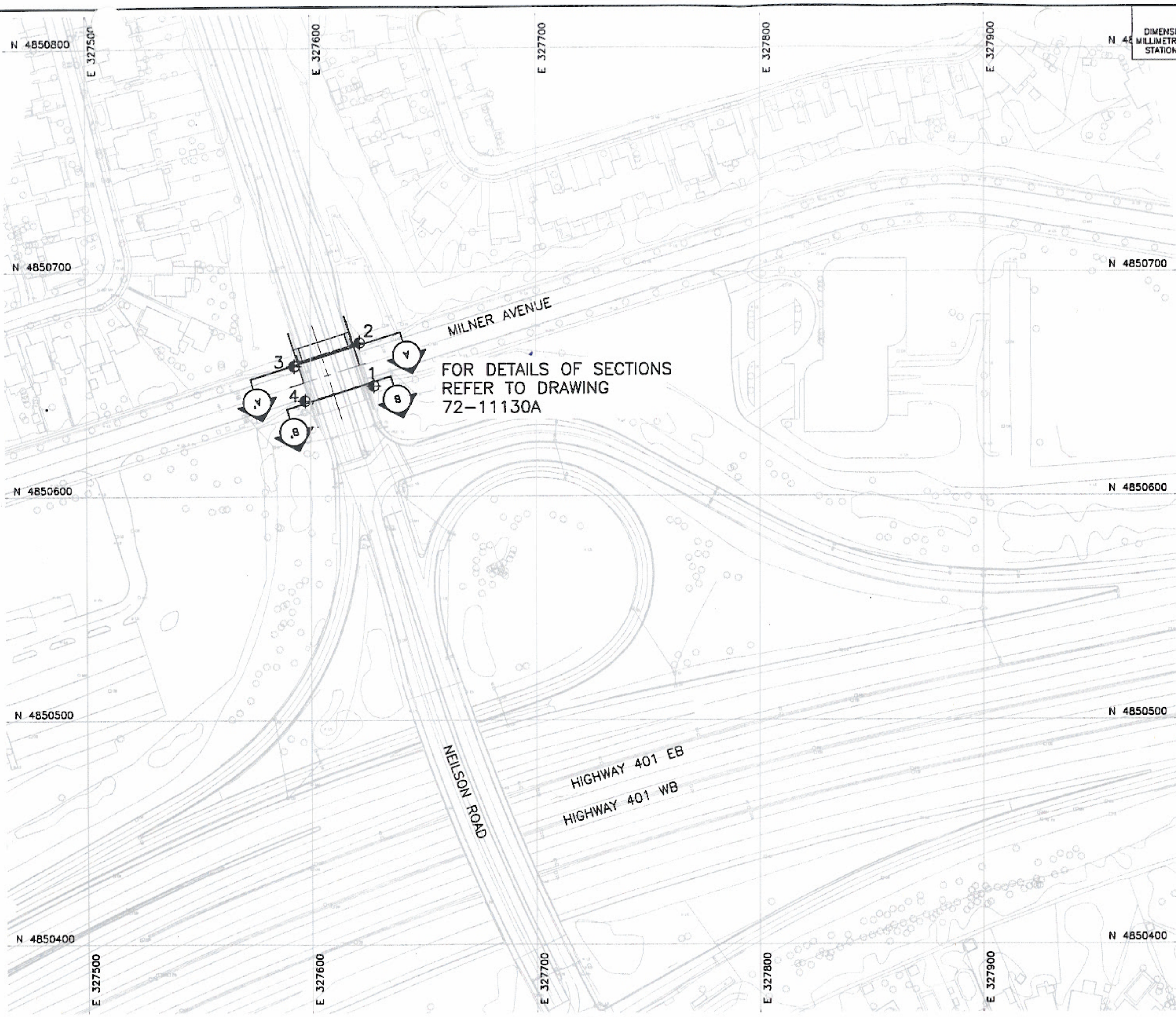
Jorge M.A. Costa, P.Eng.
Designated MTO Contact, Principal

AB/KJB/JMAC/kp

Attachments:
1. Drawing 1 – Borehole Locations
2. Drawing 72-11130A, titled "Milner Ave. & Neilson Rd. Ext'n Bore Hole Locations & Soil Strata", dated February 6, 1973.
3. Record of Boreholes No. 1 to 4
4. Drawings 1 and 3, titled "Milner Avenue Overpass, General Arrangement" and "Milner Avenue Overpass, Footing Layout and Reinforcement", undated but stamped "Received Nov. 17, 1978.

\\golder.gds\gal\whitby\active_2013\1184 pavement and materials\13-1184-0171 aecom 2013-e-0030 hwy 401-404 bridge rehab\1111-foundations\05 - reporting\site 37-1007-1 and -2 neilson at milner\draft\13-1184-0171 tm 15june16 site 37-1001.docx

MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

SCALE

20 0 20 40 m

METRIC

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

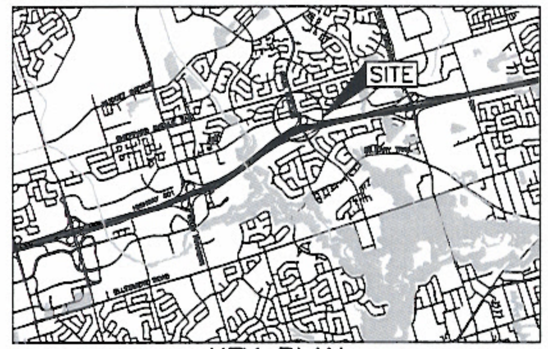
CONT No.

GWP No. 2029-13-00

NEILSON ROAD OVER MILNER AVENUE
BRIDGE REHABILITATION
(SITE #37-1007/1&2)
BOREHOLE LOCATIONS

SHEET

Golder Associates



KEY PLAN

SCALE

1 0 1 2 km

LEGEND

Borehole – Geocres No. 30M14-030

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	161.8	4850649.2	327627.9
2	161.7	4850668.4	327621.6
3	161.5	4850658.0	327592.7
4	161.5	4850642.5	327597.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by AECOM, drawing file no.s X-60317606-401-NEILSON-BASE-2007.dwg and 60317606-(37-1007) GA.dwg, received March 31, 2015. Datum of file is MTM NAD83 Zone 10

NO.	DATE	BY	REVISION

Geocres No.

HWY. 401	PROJECT NO. 13-1184-0171		DIST.
SUBM'D. AB	CHKD. KJB	DATE: 4/21/2015	SITE37-1007
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 1

DRAFT

MINIS[illegible] OF TRANSPORTATION AND COMMUNICA[illegible]S-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 1

JOB 72-11130

W.P. 43-69-04

DATUM Geodetic

LOCATION Co-ords. 65,895 N. 107.736 E.

BORING DATE December 5, 1972

BOREHOLE TYPE Auger & Sample with CME machine

ORIGINATED BY VK.

COMPILED BY VK.

CHECKED BY [illegible]

(m)	ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS (GR.SA.SI.CL.)
161.8 / 0.0	530.7	Ground Level				530	
	0.0	Heterogeneous mixture of clayey silt, sand Brown	1	SS	23		
			2	SS	24		9 37 43 11
			3	SS	[illegible]	520	
155.5 / 6.3		Grey and Gravel-Glacial Till V. Stiff to Hard	4	SS	44		(156.0)
			5	SS	33		▼ 511.7
	510.2		6	SS	40	510	
	20.5	Silty Sand with occasional gravel (loose to V.Dense)	7	SS	43		0 46 54
			8	SS	80		
			9	SS	6	500	
			10	SS	36	490	
			11	SS	180	480	3 85 12
			12	SS	117	470	
						460	
						450	
135.8 / 25.9	445.7 / 85.0	End of Borehole					
						440	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE Nº 2

FOUNDATIONS OFFICE

JOB 72-11130
W.P. 43-69-04
DATUM Geodetic

LOCATION Co-ords. 65,948 N 107,718 E
BORING DATE December 7, 1972
BOREHOLE TYPE Auger & Sample with CME machine

ORIGINATED BY VK.
COMPILED BY VK.
CHECKED BY [signature]

(m)	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
161.6 / 0.0	530.3 / 0.0	Ground Level				530	
		Heterogeneous mixture of Clayey Silt, sand & Gravel- Gladal Till	1	SS	16		
			2	SS	36		39 44 14
			3	SS	87	520	
		Brown	4	SS	78		517.8 (157.8)
		Grey	5	SS	35		
155.5 / 6.1	510.3 / 20.0	V. Stiff to Hard	6	SS	75	510	
		Silty sand with occassional Gravel	7	SS	27		
			8	SS	53		31 63
		(loose to V. Dense)	9	SS	6	500	
			10	SS	6		
			11	SS	61	490	88 (8)
						480	
145.9 / 15.7	478.8 / 51.5	End of Borehole	12	SS	154/11"		
						470	

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W
WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.

15 20 5 10 % STRAIN AT FAILURE

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 3

JOB 72-11130 | LOCATION Co-ords 65,910 N 107,603 E | ORIGINATED BY VK

W.P. 43-69-04 | BORING DATE December 8, 1972 | COMPILED BY VK

DATUM Geodetic | BOREHOLE TYPE Auger & Sample with CME machine | CHECKED BY

(m)	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	WATER CONTENT %	REMARKS GR.SA.SI.CL
161.4 / 0.0	529.6 / 0.0	Ground Level						(160.5) ▼526.6
		Heterogeneous mixture of clayey silt, sand and gravel- Brown	1	SS	19			
			2	SS	22			42 40 1
			3	SS	56	520		
		Grey Glacial Till V. Stiff to Hard	4	SS	43			
			5	SS	58			
			6	SS	162	510		
			7	SS	175/10"			
153.5 / 7.9	503.6 / 26.0	Silty Sand with occassional grav Very Dense	8	SS	162/11"	500		
			9	SS	110/10"			71 27 2
			10	SS	160/9"	490		
			11	SS	190/10"	480		
			12	SS	100/6"			92 (5)
			13	SS	161	470		
						460		
						450		
						440		
132.5 / 29.0	434.6 / 95.0	End of Borehole						
						430		

Column headings: SOIL PROFILE; STRAT. PLOT; SAMPLES; DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30; BULK DENSITY γ P.C.F.

15 20 5 10 % STRAIN AT FAILURE

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE Nº 4

FOUNDATIONS OFFICE

JOB 72-11130
W.P. 43-69-04
DATUM Geodetic

LOCATION Co-ords 65,859 N 107,619 E
BORING DATE December 13, 1972
BOREHOLE TYPE Auger Sample with CME machine

ORIGINATED BY VK
COMPILED BY VK
CHECKED BY [signature]

(m)	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FOOT	ELEV. SCALE	WATER CONTENT %	REMARKS GR.SA.SI.CL.
161.5 / 0.0	529.7 / 0.0	Ground Level							526.7
		Heterogeneous mixture of		1	SS	17			
				2	SS	26			3 50 34 1
		Brown					520		
		Grey		3	SS	41			
		Clayey Silt,		4	SS	37			
		sand & gravel Glacial Till		5	SS	51			
		V. Stiff to Hard		6	SS	31	510		
154.7 / 6.7	507.7 / 22.0			7	SS	135/10"			
				8	SS	145/11"			5 30 65 0
		Silty sand with occasional gravel		9	SS	100/6"	500		
		Very Dense					490		
				10	SS	79			
148.8 / 12.7	488.2 / 41.5	End of Borehole							
							480		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT w_L
PLASTIC LIMIT w_P
WATER CONTENT w
WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

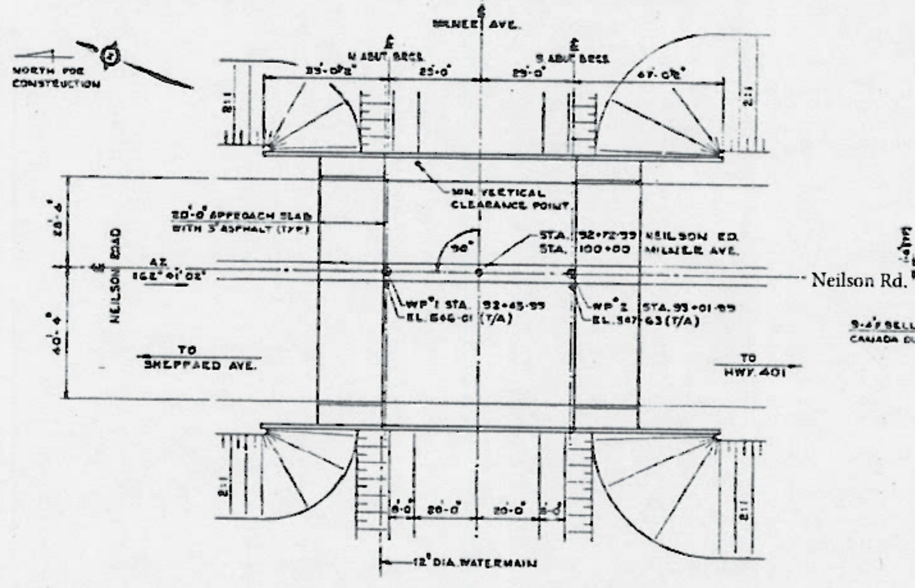
OFFICE REPORT SOIL EXPLORATION

DISTRICT No 6
CONT No
WP No 43-69-04

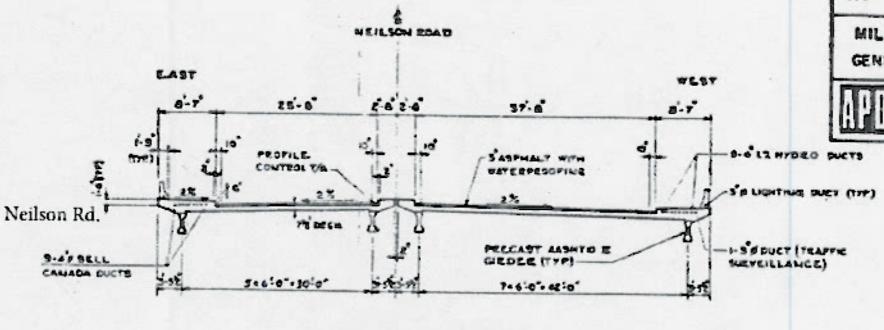
MILNER AVE OVERPASS
GENERAL ARRANGEMENT

SHEET

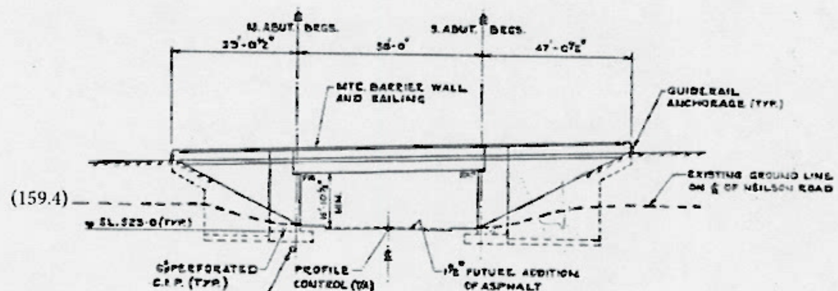
APD ALBERY, PULLERITS, DICKSON & ASSOCIATES
CONSULTING ENGINEERS



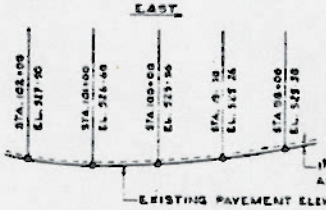
PLAN
SCALE 1"=20'



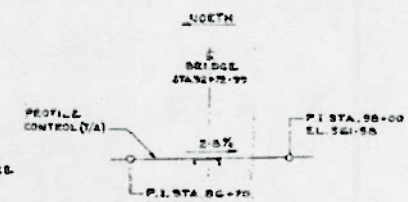
TYPICAL CROSS SECTION
SCALE 1"=10'



ELEVATION
SCALE 1"=20'



PROFILE OF MILNER AVE
N.T.S.



PROFILE OF NEILSON RD.
N.T.S.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. FOOTING LAYOUT & REINFORCEMENT
4. ABUTMENTS
5. RETAINING WALLS & REINFORCEMENT
6. DECK REINFORCEMENT (SCREED ELEVATIONS)
7. PRESTRESSED GIRDERS & BEARINGS
8. BARRIER WALL DETAILS
9. STEEL BARRIER WALL RAIL
10. APPROACH SLAB
11. STANDARD DETAILS I
12. STANDARD DETAILS II
13. AS CONSTRUCTED ELEVATIONS & DETAILS

CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEM.

6. ABUTMENTS, WINGWALLS, RETAINING WALLS.	688 cu yd
9. DECKS AND DIAPHRAGMS.	186 cu yd
11. BARRIER WALLS.	22 cu yd
12. APPROACH SLABS.	93 cu yd

NOTES

1. CLASS OF CONCRETE

DECK, SIDEWALKS & BARRIER WALLS	- 4000 PSI
PRESTRESSED GIRDERS	- 5000 PSI
REMAINDER	- 3000 PSI
AND/OR AS NOTED	

2. CLEAR COVER ON REINFORCING STEEL

FOOTINGS, ABUTMENTS & RETAINING WALLS - 3"
SIDEWALKS & APPROACH SLAB - 2"
TOP OF DECK - 2" ●
BOT OF DECK - 1½"
DIAPHRAGMS & BARRIER WALLS - AS NOTED

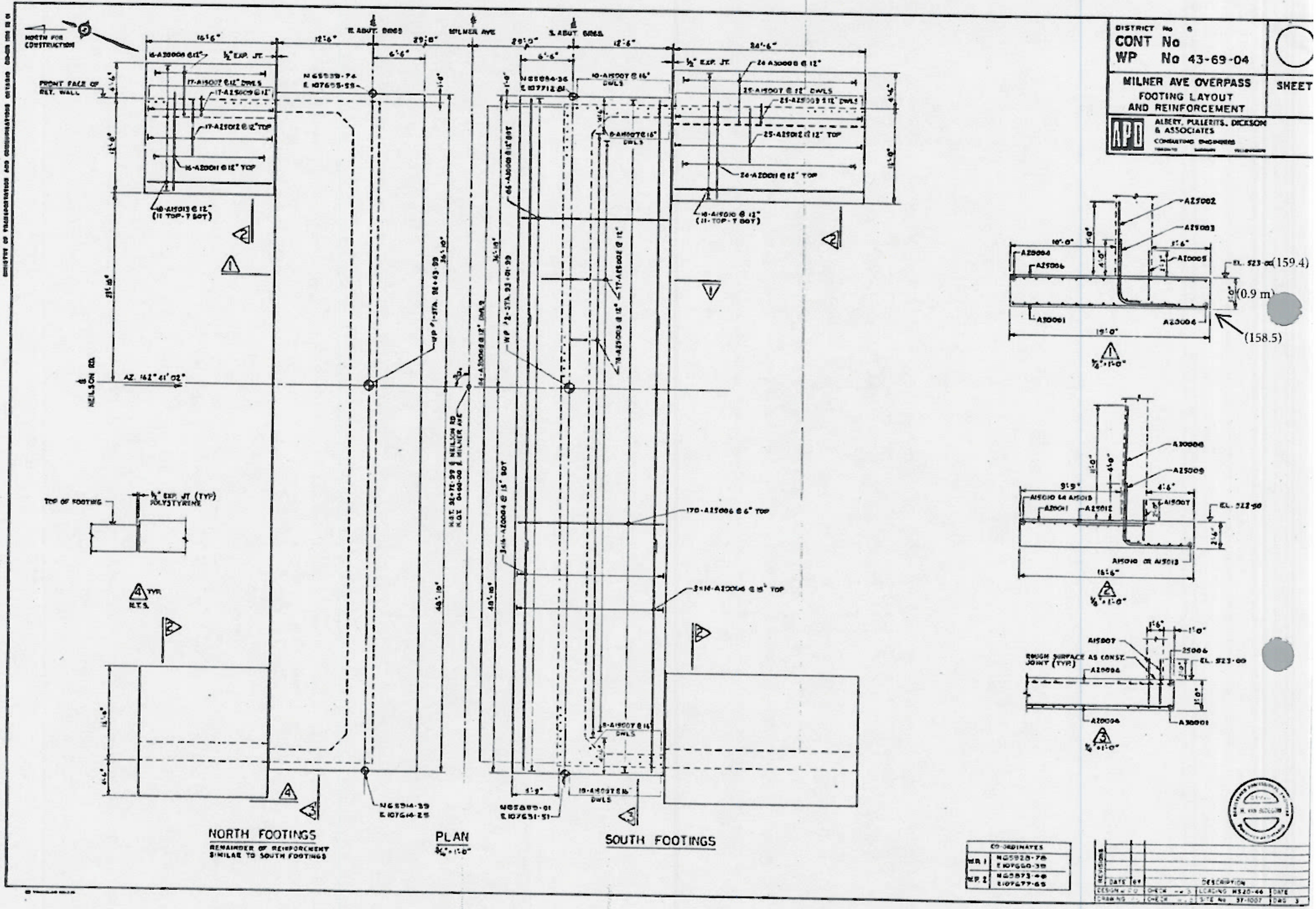
3. CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

REINFORCING STEEL SHALL BE GRADE 400 EXCEPT AS NOTED. REINFORCING BARS WITH THE DESIGNATION 'C' AT THE END OF BAR MARKS SHALL BE COATED BARS.

● TO ACHIEVE THE MINIMUM CLEAR COVER OF 2" SPECIFIED, THE TOP LAYER OF DECK STEEL SHALL BE PLACED PRIOR TO CONCRETING WITH A CLEAR COVER OF 2½" ± ½" TOLERANCE.

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CHECK	LOADING MS20-44	DATE
DRAWING	CHECK	SITE No 37-1007	DWG 1



DISTRICT No 6
CONT No
WP No 43-69-04
MILNER AVE OVERPASS
FOOTING LAYOUT
AND REINFORCEMENT
ALBERY, PULLERITS, DICKSON & ASSOCIATES
CONSULTING ENGINEERS
SHEET
NORTH FOR CONSTRUCTION
FRONT FACE OF RET. WALL
½" EXP. JT.
NEILSON RD.
MILNER AVE
TOP OF FOOTING
½" EXP. JT (TYP) POLYSTYRENE
N.T.S.
170-A25006 @ 6" TOP
(159.4)
(0.9 m)
(158.5)
EL. 523·00
ROUGH SURFACE AS CONST. JOINT (TYP.)
NORTH FOOTINGS
REMAINDER OF REINFORCEMENT SIMILAR TO SOUTH FOOTINGS
PLAN
SOUTH FOOTINGS
CO-ORDINATES
REVISIONS
DATE
DESCRIPTION