



Preliminary Foundation Investigation and Design Report

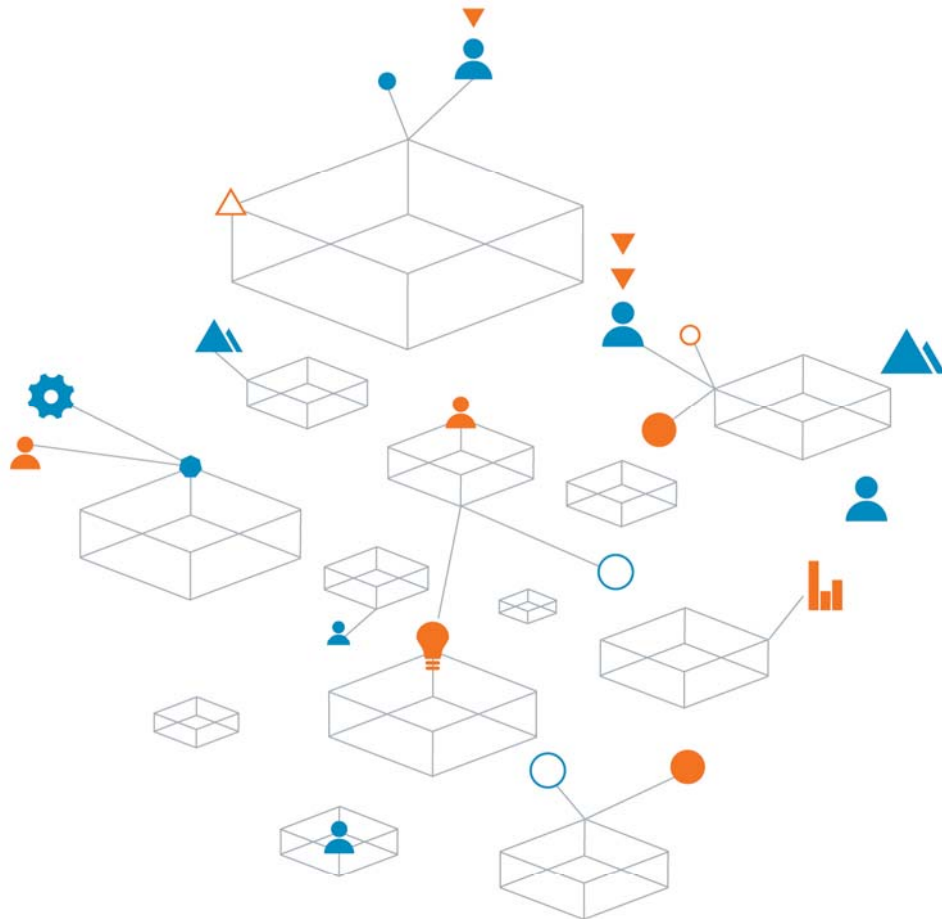
Highway 400/Tiffin Street NB & SB Overpass Structures,

G.W.P. 2074-11-00, Site No's. 30-176/1 & 2,

Design-Build Ready Package, GEOCREC No. 31D-587

GEOTETOB22161AA

11 February, 2015



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of all our
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11 February, 2015

Attention: **Bruce Dickey**, P.Eng., AVS

Dear Sir

RE: Preliminary Foundation Investigation and Design Reports, Highway 400/Tiffin Street NB & SB Overpass Structures, G.W.P. 2074-11-00, Site No's. 30-176/1&2, Design-Build Ready Package

Coffey is pleased to present the preliminary foundation Investigation and Design Reports (for a Design-Build Ready Package) relating to the above noted site.

Please call us on 416 213 5357 should you require further clarification on any aspects of the reports.

For and on behalf of Coffey.

A handwritten signature in blue ink, appearing to read "Sanket Shah", written over a horizontal line.

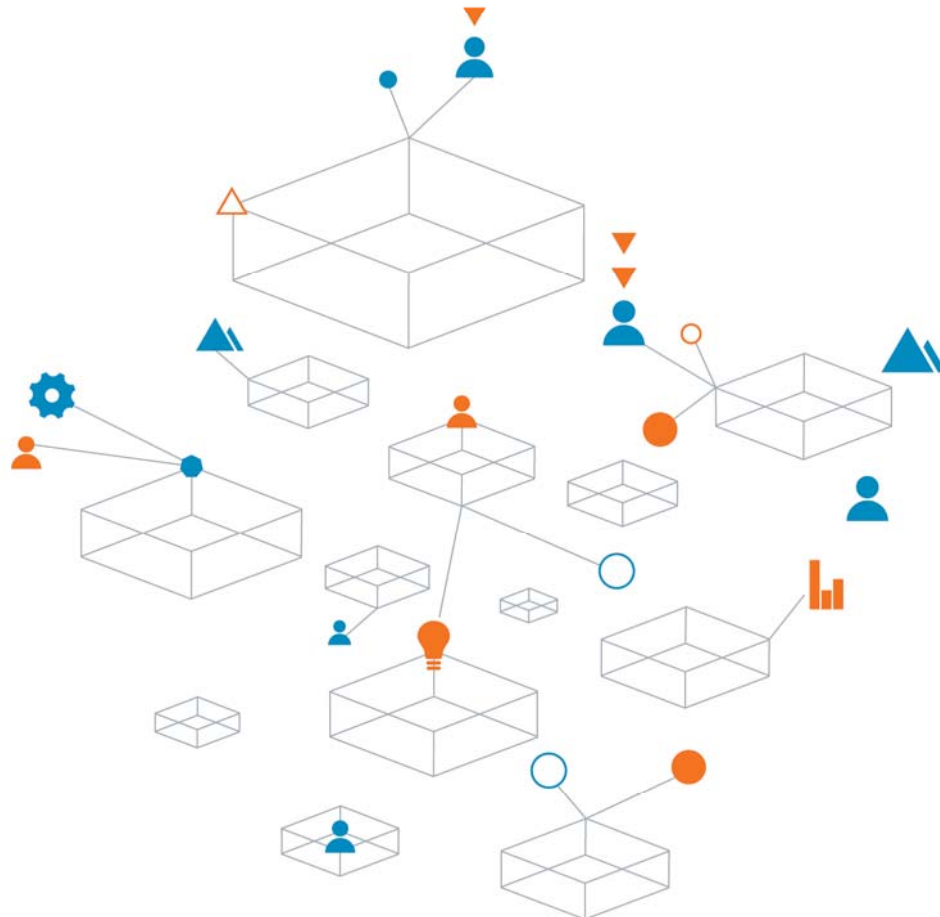
Sanket Shah, P.Eng.

Project Manager, Geotechnical Engineer



Preliminary Foundation Investigation Report

Highway 400/Tiffin Street NB & SB Overpass Structures,
G.W.P. 2074-11-00, Site No's. 30-176/1 & 2,
Design-Build Ready Package, GEOCRES No. 31D-587
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**PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 400/TIFFIN STREET NB & SB OVERPASS STRUCTURES
G.W.P. 2074-11-00, SITE NO'S. 30-176/1&2,
DESIGN-BUILD READY PACKAGE**

1 Introduction

Coffey was retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation Ontario (MTO) to provide preliminary foundation investigation and engineering services for the proposed design-build (DB) ready package for MTO G.W.P. 2074-11-00, *Highway 400/Tiffin Street Overpass Structure Replacements and Highway 400/Barrie-Collingwood Railway (BCR) Overhead Structure Rehabilitation and Addition*. The project extends from just north of the existing Essa Road – Highway 400 Interchange to just south of the Dunlop Street – Highway 400 Interchange. This investigation report is prepared for the proposed north bound (NB) and south bound (SB) Tiffin Street overpass structures in the City of Barrie.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to assess the engineering characteristics of the subsurface soils by means of field and laboratory tests. The findings of the investigation are presented in this report. It provides factual information on subsurface soil and groundwater conditions, in-situ testing, and laboratory test results. Owing to suspected TCE (trichloroethylene) contamination and the design-build nature of the project, the subsurface investigation scope was limited to a reduced number of boreholes and a requirement not to investigate the subsurface conditions below certain pre-specified depths/elevations.

2 Site Description and Physiography

2.1 Site and Structure Description

The overall project is located in the City of Barrie (Townships of Innisfil and Vespra). The existing rigid frame concrete structure has a span of 16 m and is 32 m long. It was built in the 1950's. Tiffin Street is a two-lane roadway with a turning median lane. The existing structure is visually in fair to good condition. Detailed bridge inspection reports are available elsewhere.

The areas on the east and west sides of Highway 400 and adjoining Tiffin Street have been developed and include both residential and mixed commercial and industrial land uses.

Photographs of the site are presented in **Appendix C**.

2.2 Physiography

The project site is located in the Simcoe Lowlands Physiographic Region of Southern Ontario. The soil deposits are either deltaic or lacustrine in origin. They consist of fine grained non-cohesive silts and fine sands intermixed with thin (< 1 m thickness) stringers of clayey silt deposited during quieter periods of sedimentation.

Due to the depositional environment and lack of adequate drainage that encouraged in situ decay of growing vegetation, peat and muck lenses and layers are present in depressed areas in the upper horizons of deltaic and lacustrine silt and sand deposits.

3 Method of Investigation

3.1 Field Investigation

The borehole locations and depths were discussed with MH to maximize borehole coverage to develop an effective design-build ready package. Due to suspected trichloroethylene (TCE) contamination at the bridge site, borehole depths/elevations were determined by MH environmental specialists to minimize possible environmental issues.

Five (5) boreholes were advanced adjacent to the proposed Tiffin Street overpass structures (4 for bridge foundations and 1 for a retaining wall). The borehole locations were laid out by Coffey personnel on the basis of chainage painted by MH along Highway 400. Underground services were cleared using Ontario One Call and private locators. The field work was conducted from October 2nd to 23rd, 2014 under Coffey supervision. Boreholes F1, F2, F4 and RW 11 were advanced from existing Highway 400 grade during nightly lane closures as directed by MTO COMPASS.

The first borehole (BH F3) was drilled from the existing Tiffin Street grade with an MH environmental specialist being present to provide guidance. Borehole F3 established the lowest elevation to which the remaining boreholes in the vicinity could be drilled and sampled. Table 3.1 provides a summary of the field work.

Table 3.1 – Summary of Boreholes

Structure	BH No.	Borehole Locations (Station and Offset from the centreline)	Ground Elevation (m)	Borehole Depth (m)	Borehole Bottom Elevation (m)	Piezometer/ Monitoring Well
Tiffin Street SB New	F1	29+725, 2 m Lt	240.6	15.9	224.8	
	F2	10+019, 12 m Lt	239.5	15.9	223.7	Piezometer
Tiffin Street NB New	F3	29+710, 30 m Rt	234.1	9.8	224.4	Monitoring well
	F4	10+010, 10 m Rt	239.9	15.9	224.1	
Temporary Retaining Walls	RW11	29+696, 3 m Lt	241.0	14.3	226.7	

One monitoring well and one piezometer were installed in Boreholes F3 and F2 for groundwater sampling and long term groundwater monitoring. The remaining boreholes were backfilled and sealed in accordance with MOE Reg. 903.

The boreholes were drilled with truck mounted CME-75 machines (owned and operated by Davis Drilling of Milton, Ontario) equipped with solid stem and hollow stem augers). Soil samples were obtained in the Standard Penetration test (ASTM D-1586), with N values noted in blows/0.3m. All samples were placed in moisture proof bags after field classification. They were subsequently re-examined under controlled laboratory conditions prior to assigning laboratory tests.

The borehole locations were tied in to NAD83 coordinates and the geodetic elevations at the borehole locations were determined by MH surveyors.

3.2 Laboratory Testing

The following tests were performed on selected soil samples:

- Natural moisture content; and
- Grain size analyses (sieve and hydrometer).

Laboratory test results are presented in **Appendix B**. The results of laboratory tests are also presented on the individual Record of Borehole Sheets in **Appendix A**.

4 Subsurface Conditions

The major native soil deposits at the project site below and around the Highway 400 embankment fill are silty sand, sandy silt and silt.

Detailed descriptions of the materials encountered in the boreholes are presented on the Record of Borehole Sheets presented in **Appendix A**, which includes Explanation of Terms Used in the Report.

Borehole location plan and the generalized subsurface condition are presented on **Drawing 1**.

4.1 Topsoil

Adjacent to Tiffin Street in the southeast corner, the topsoil thickness was 100 mm in a grassed area adjacent to the south side sidewalk.

4.2 Pavement Structure

On Highway 400 above Tiffin Street, the asphaltic concrete pavement was 200-400 mm thick (average 275 mm) above 0.6 m of sand and gravel base course and subbase course. The granular base and subbase was dense (average N = 34 blows/0.3 m).

The pavement structure of Tiffin Street was not investigated as the rehabilitation and widening of Tiffin Street, to be undertaken by the City of Barrie, is not part of this assignment.

4.3 Embankment Fill

The highway embankment fill consists of silty sand, trace gravel. Gradation tests on three fill samples (see **Figure B-1**) gave the following particle size distribution ranges:

Gravel:	4-8%
Sand:	71-84%
Silt and Clay:	12-21% (3-6% clay sized particles)

The natural moisture content was 5-23% with an average of 9%.

Standard Penetration Test N values ranged from 3 to 36 blows/0.3 m (average 17 blows/0.3 m), indicating a compact to dense condition. It appears some portions of the fill may not have been appropriately compacted.

Cobbles, boulders and rock fill were not encountered in boreholes drilled through the fill, but their likely presence elsewhere within the Highway 400 embankment fill should not be discounted.

4.4 Sandy Silt, Silty Sand and Silt

The native soils beneath and adjacent to the Highway 400 embankment consists of stratified sandy silt, silty sand and silt containing trace gravel and clay. The deposit extended to the limiting depth of boreholes at elev. 226.7-223.6 m. Its extent below the final depth of boreholes and the presence of other stratigraphic units below these terminal elevations is not known.

Gradation tests on six samples (see **Figure B-2**) gave the following ranges in grain sizes:

Gravel:	0-1%
Sand:	1-77%
Silt and Clay:	23-99% (3-8% clay sized particles)

The natural moisture content was 8-30% with an average of 19%.

N values of 7 to 47 blows/0.3 m indicate a loose to locally dense condition. Overall, this deposit is considered to be compact.

4.5 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole, and in a monitoring well and piezometer. The groundwater level observations are summarized in Table 4.5.1 and are also presented on the Record of Borehole Sheets (**Appendix A**).

Table 4.5.1. Groundwater Observations

Borehole No.	Ground Elevation (m)	Date	Depth to Water Level (m)	Groundwater Elevation (m)
F1	240.6	Upon completion	10.7	229.9 (wet cave in)
F2	239.5	October 31, 2014 (about 4 weeks after installation)	9.1	230.4
F3	234.1	October 31, 2014 (about a week after installation)	4.1	230.0
F4	239.9	Upon completion	9.8	230.1 (cave-in @ 228.3)
RW11	241.0	Upon completion	10.7	230.3

Groundwater levels measured on completion are considered not stabilized and therefore do not represent the groundwater table at the site.

Based on the observations recorded in Table 4.5.1, the groundwater table at the Tiffin Street overpass site is, for design purposes, situated at about elev. 230.5 m.

It should be noted that groundwater levels are subject to variation due to the influence of rainfall, seasons and other factors.

For and on behalf of Coffey.



Gwangha Roh, P.Eng., Ph.D.
Associate Geotechnical Engineer



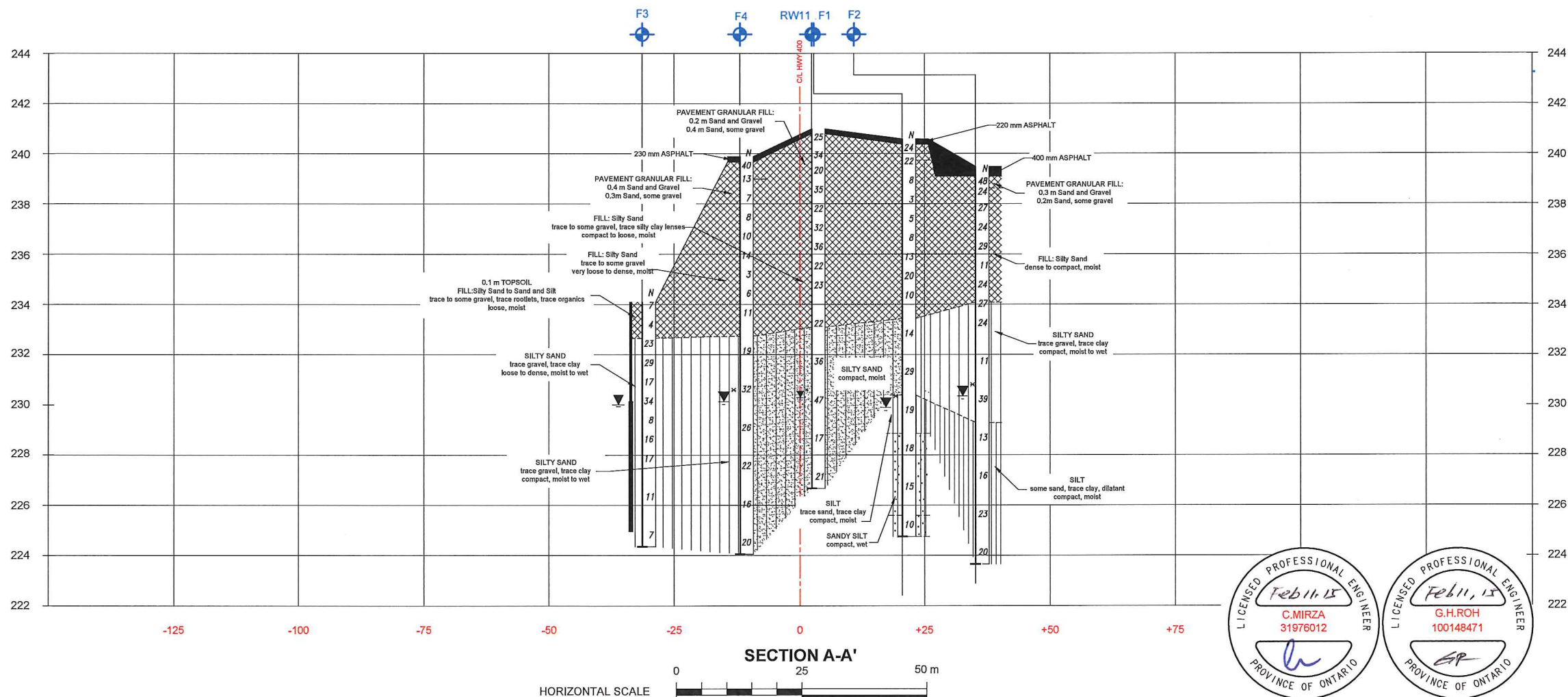
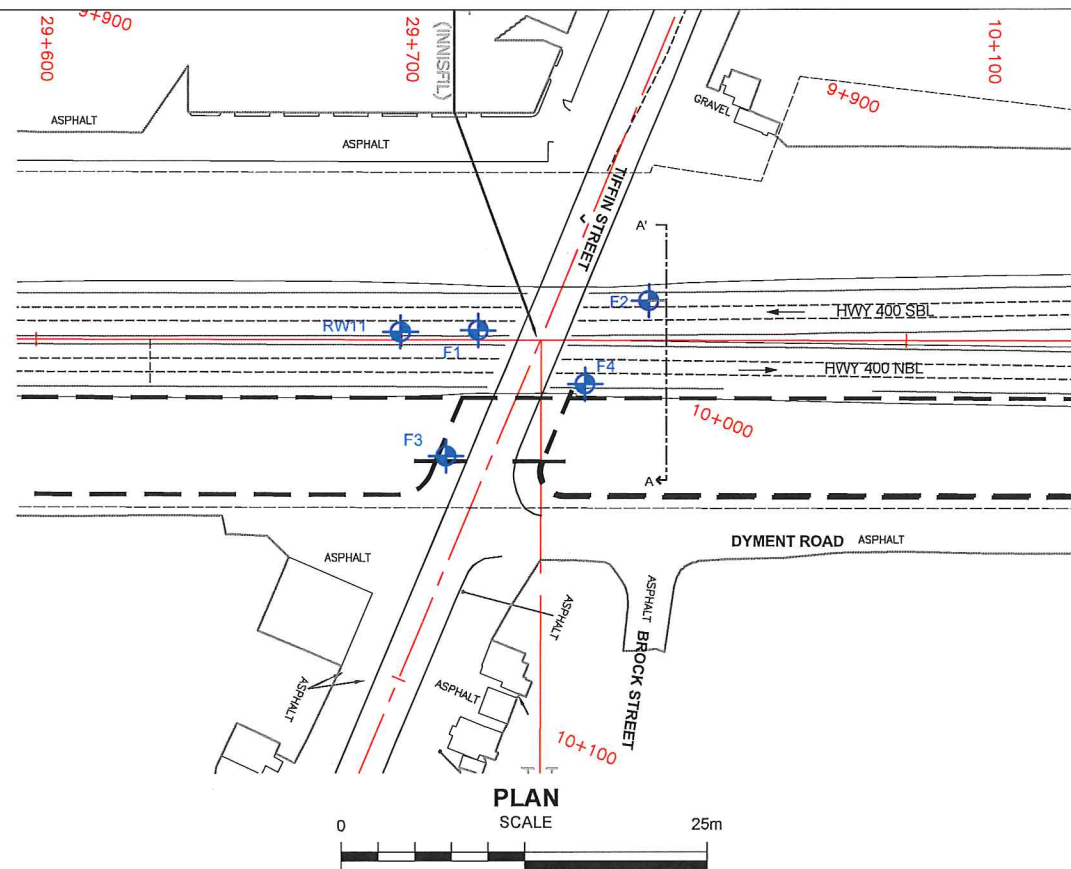
Sanket Shah, P.Eng.
Project Manager, Geotechnical Engineer



Cam Mirza, P.Eng.
MTO Designated Contact, Principal



Drawing



DISTRICT
CONT. No.
WP No. -

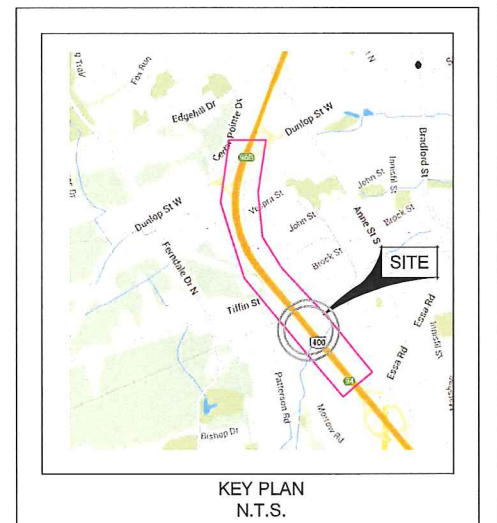
Highway 400/Tiffin Street
Overpass Structures

Borehole Location Plan
and Soil Strata

coffey

METRIC

SHEET



LEGEND

Borehole
 N
 Blows/0.3m (Std. Pen. Test, 475 J/blow)
 Water Level at Time of Investigation
 Water Level in Piezometer
 Piezometer

No.	ELEVATION	NORTHING	EASTING
F1	240.8	288270.5	4914538.5
F2	239.5	288234.3	4914569.2
F3	234.1	288302.4	4914553.7
F4	239.9	288262.9	4914570.4
RW11	241.0	288284.4	4914522.5

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
GEOCRE No. 31D-587 PROJECT No. GEOTETOB22161AA

LICENSED PROFESSIONAL ENGINEER
Feb 11, 15
C.MIRZA
31976012
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
Feb 11, 15
G.H.ROH
100148471
PROVINCE OF ONTARIO

REVISIONS		DESCRIPTION					
DESIGN	GR	CHK	SH	CODE	LOAD	DATE	Dec /14
DESIGN	SSH	CHK	CM	SITE	-	STRUCT	DWG
						DWG	1

Appendix A

**Explanation of Terms Used in Report and
Record of Borehole Sheets**

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS N.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
C_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

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

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F1

1 OF 2

METRIC

GWP 2074-11-00 LOCATION 29+721, 2.6 m Lt C/L (N 4914538.5, E288270.5) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 15/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
240.6	GROUND SURFACE													
240.0	220 mm ASPHALT													
0.2	PAVEMENT GRANULAR FILL: 0.2 m Sand and Gravel 0.4 m Sand, some gravel		1	SS	24		240							
239.8			2	SS	22		239							
0.9	FILL: Silty Sand trace to some gravel trace silty clay lenses grey to brown, compact to loose moist		3	SS	8		238							
			4	SS	3		237							
			5	SS	5		236							
			6	SS	8		235							
			7	SS	13		234							
			8	SS	20		233							
			9	SS	10		232							
233.4	SILTY SAND brown, compact, moist		10	SS	14		231							
7.2			11	SS	29		230							
230.4	SILT trace sand, trace clay brown, compact, moist		12	SS	19		229							
10.2			13	SS	18		228							
228.9	SANDY SILT grey, compact, wet		14	SS	15		227							
11.7							226							
225.6														

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE


GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F1

2 OF 2

METRIC

GWP 2074-11-00 LOCATION 29+721, 2.6 m Lt C/L (N 4914538.5, E288270.5) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 15/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						
225.6 15.0	SANDY SILT grey, compact, wet		15	SS	10		225							
224.8 15.9														
15.9	End of Borehole wet cave-in @10.7 m													

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F2

1 OF 2

METRIC

GWP 2074-11-00 LOCATION 10+030, 10.8 m Lt C/L (N 4914569.2, E 288234.3) ORIGINATED BY JD
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 23/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
239.5	GROUND SURFACE													
0.0	400 mm ASPHALT													
239.1														
0.4	PAVEMENT GRANULAR FILL: 0.3 m Sand and Gravel 0.2 m Sand, some gravel		1	SS	48									
238.6			2	SS	24									
0.9	FILL: Silty Sand brown to grey, dense to compact, moist trace clay		3	SS	27									
			4	SS	24									
			5	SS	29									
			6	SS	11									
			7	SS	24									
234.1			8	SS	27									
5.4	SILTY SAND trace gravel, trace clay brown to grey, compact, moist to wet		9	SS	24									
			10	SS	11									
			11	SS	39									
229.3														
10.2	SILT some sand, trace clay, dilatant grey, compact, wet		12	SS	13									
			13	SS	16									
			14	SS	23									
224.5														

Continued Next Page

+ 3, x 3; Numbers refer to
Sensitivity
20
15
10
(%) STRAIN AT FAILURE

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F2

2 OF 2

METRIC

GWP 2074-11-00 LOCATION 10+030, 10.8 m Lt C/L (N 4914569.2, E 288234.3) ORIGINATED BY JD
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 23/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	10 20 30					
224.5 15.0	SILT some sand, trace clay, dilatant grey, compact, wet														
223.7 15.9			15	SS	20	224									
15.9	End of Borehole cave-in @ 9.1 m Water level @ 9.1 m (not stabilized)* upon completion.														

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F3

1 OF 1

METRIC

GWP 2074-11-00 LOCATION 29+712, 31.6 m Rt C/L (N 4914553.7, E288302.4) ORIGINATED BY LG
DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
DATUM Geodetic DATE 02/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
234.1	GROUND SURFACE													
0.0	0.1 m TOPSOIL FILL: Silty Sand to Sand and Silt trace to some gravel trace rootlets trace organics brown, loose, moist		1	SS	7		234							
			2	SS	4		233							
232.7														
1.4	SILTY SAND trace gravel, trace clay brown, loose to dense, moist to wet		3	SS	23		232							
			4	SS	29		231							
	sand some silt		5	SS	17		230							
			6	SS	34		229							
			7	SS	8		228							
	silt trace sand loose		8	SS	16		227							
			9	SS	17		226							
			10	SS	11		225							
			11	SS	7									
224.4	loose													
9.8	End of Borehole Water level upon completion @ 4.2 m Piezometer installed to 9.1 m. Piezometer water level records : Oct. 31, 2014 4.1 m (El. 230 m)													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F4

1 OF 2

METRIC

GWP 2074-11-00 LOCATION 10+012, 11.9 m Rt C/L (N 4914570.4, E 288262.9) ORIGINATED BY LG
DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
DATUM Geodetic DATE 02/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
239.9	GROUND SURFACE													
239.9	230 mm ASPHALT													
0.2	PAVEMENT GRANULAR FILL: 0.4 m Sand and Gravel 0.3 m Sand, some gravel		1	SS	40		239							
239.0	FILL: Silty Sand trace to some gravel brown to grey, very loose to dense, moist		2	SS	13									
0.9			3	SS	7		238							
			4	SS	8		237							
			5	SS	10		236							
			6	SS	14		235							
			7	SS	3		234							
			8	SS	6		233							
			9	SS	11		232							
232.7	SILTY SAND trace gravel, trace clay brown to grey, compact, moist to wet		10	SS	19		231							
7.2			11	SS	32		230							
			12	SS	26		229							
			13	SS	22		228							
			14	SS	16		227							
224.9							226							
							225							

Continued Next Page

+ 3, x 3; Numbers refer to
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

GEOTETO22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F4

2 OF 2

METRIC

GWP 2074-11-00 LOCATION 10+012, 11.9 m Rt C/L (N 4914570.4, E 288262.9) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 02/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
224.9 15.0	SILTY SAND trace gravel brown to grey, compact, moist to wet		15	SS	20									
224.1 15.9														
	End of Borehole cave-in @ 11.6 m Water level upon completion @ 9.8 m													

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

GEOTETOB22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH RW11

1 OF 1

METRIC

GWP 2074-11-00 LOCATION 29+700, 2.2 m Lt C/L (N 4914522.5, E 288284.4) ORIGINATED BY JD
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP
 DATUM Geodetic DATE 14/10/2014 CHECKED BY SH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
241.0	GROUND SURFACE						241						
240.8	200 mm ASPHALT						240						
0.2	0.4 m gravelly sand		1	SS	25								
	FILL: Silty Sand trace to some gravel brown, compact to dense, moist		2	SS	34								
	sandy gravel		3	SS	20								
	sand		4	SS	35								
			5	SS	22								
			6	SS	32								
			7	SS	36								
			8	SS	22								
			9	SS	23								
233.1	SILTY SAND TO SANDY SILT trace gravel brown to grey, compact to dense moist to wet		10	SS	22		233						
7.9							232						
	silty sand		11	SS	36		231						
	sandy silt		12	SS	47		230						
			13	SS	17		229						
							228						
226.7			14	SS	21		227						
14.3	End of Borehole Water level @ 10.7 m (not stabilized)* upon completion.												

+³, ×³: Numbers refer to
Sensitivity

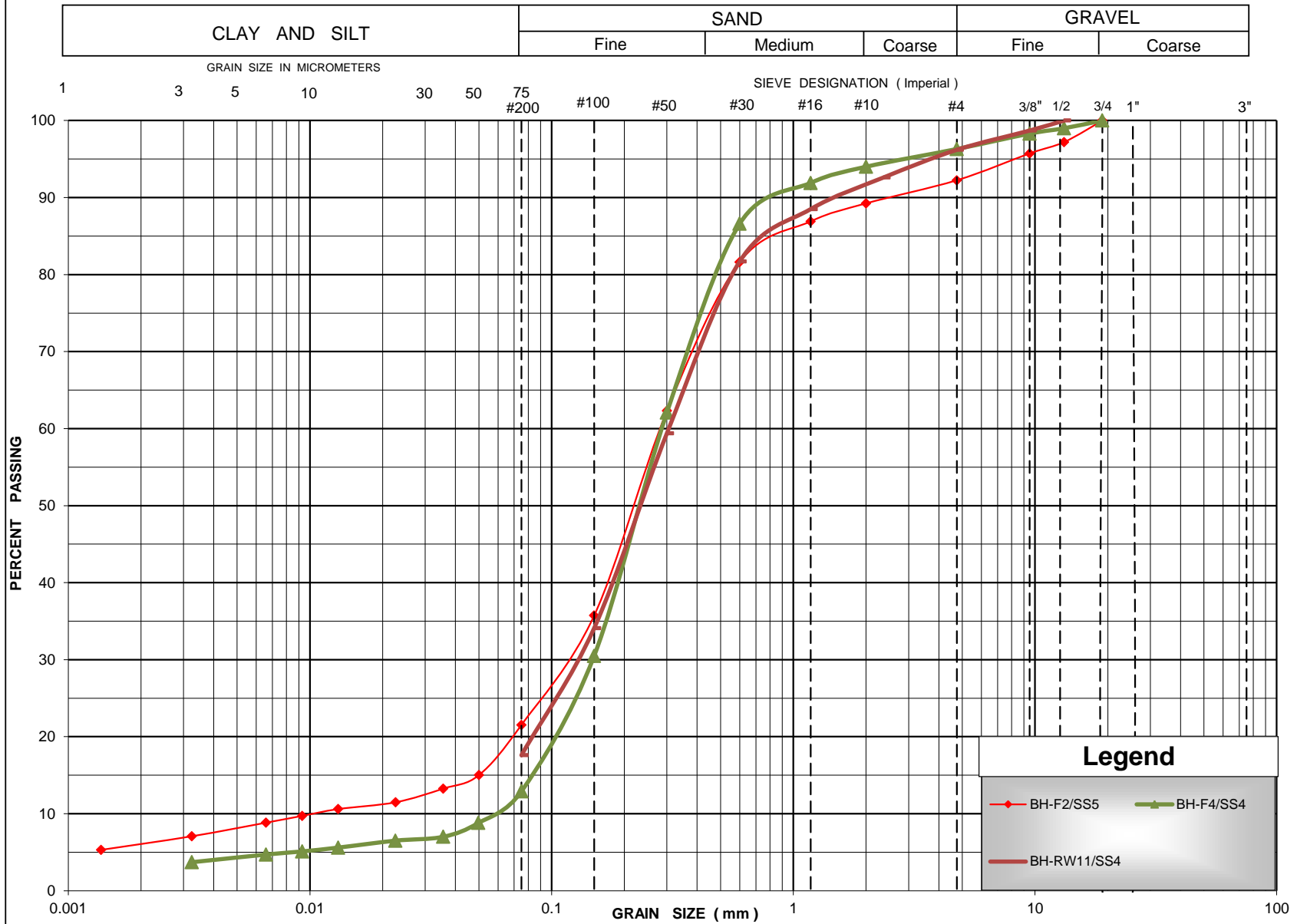
20
15 10 5
(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

LS 702/ ASTM D 422

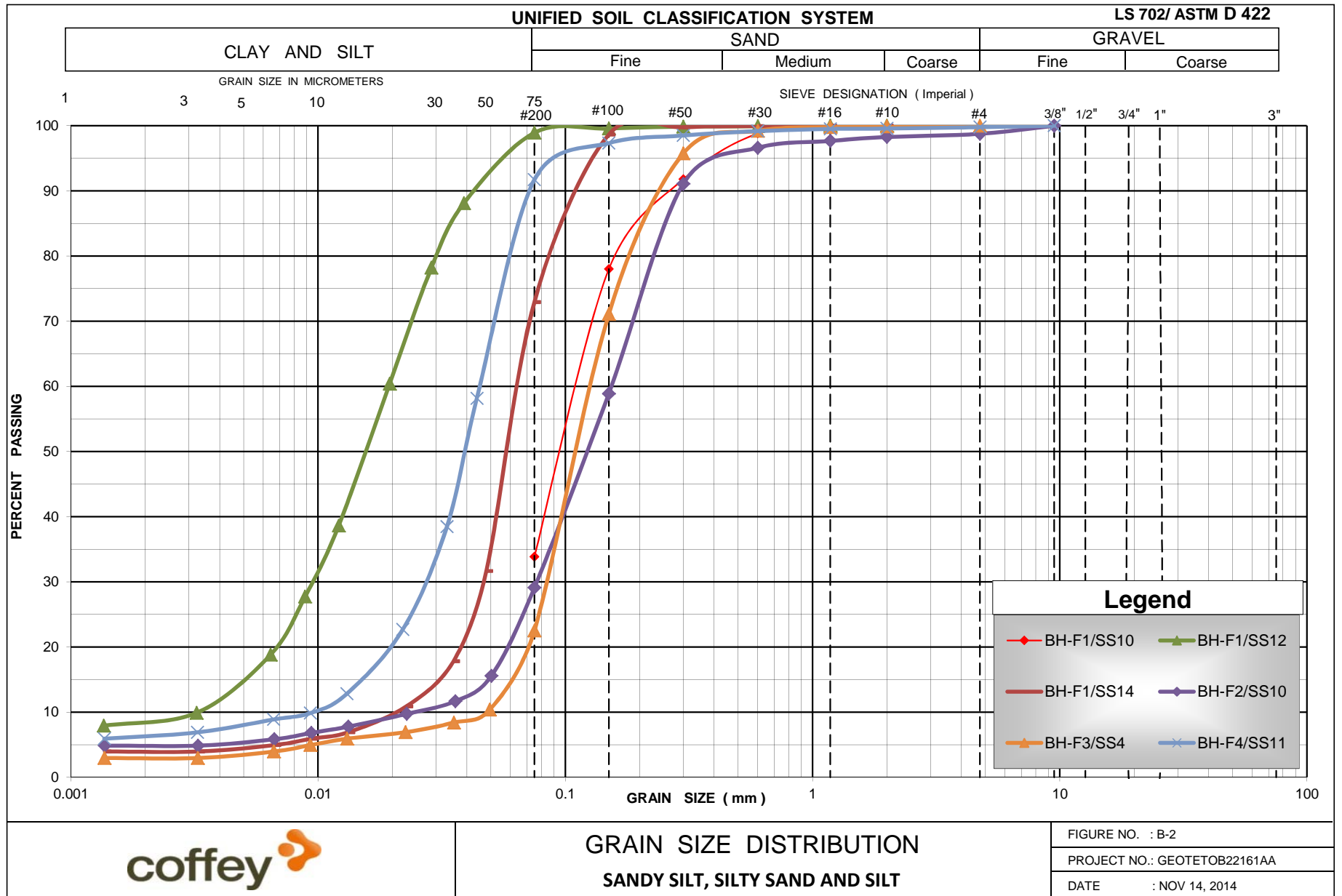


GRAIN SIZE DISTRIBUTION
EMBANKMENT FILL: Silty Sand

FIGURE NO. : B-1

PROJECT NO.: GEOTETO22161AA

DATE : NOV 17, 2014

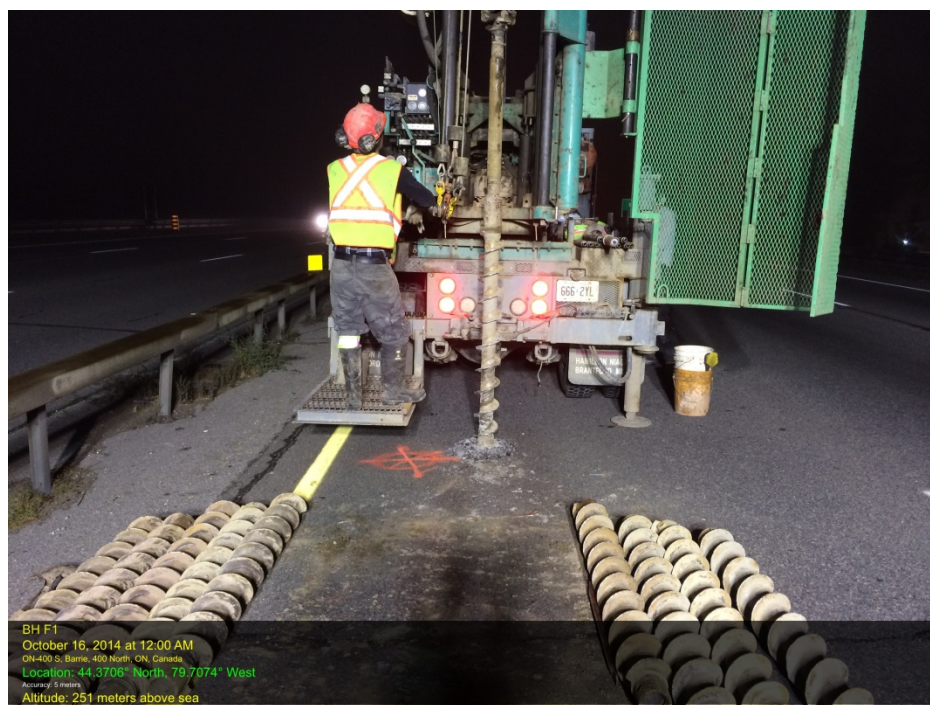


Appendix C

Site Photographs



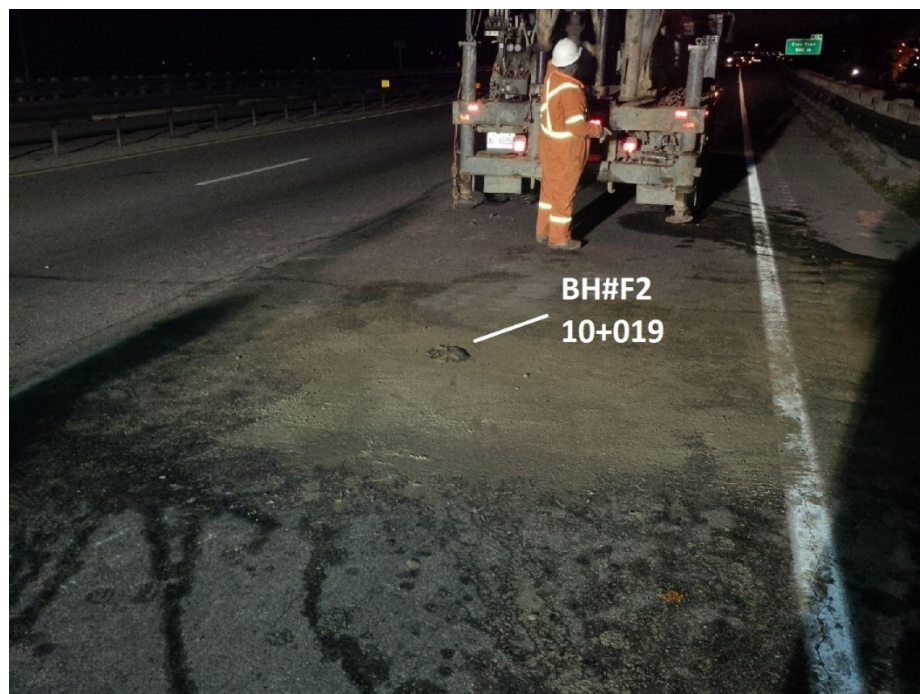
Photograph 1: Borehole F3 @ Station 29+712, Looking West



Photograph 2: Borehole F1 @ Station 29+721, Looking South



Photograph 3: Borehole F4 @ Station 10+012, Looking North

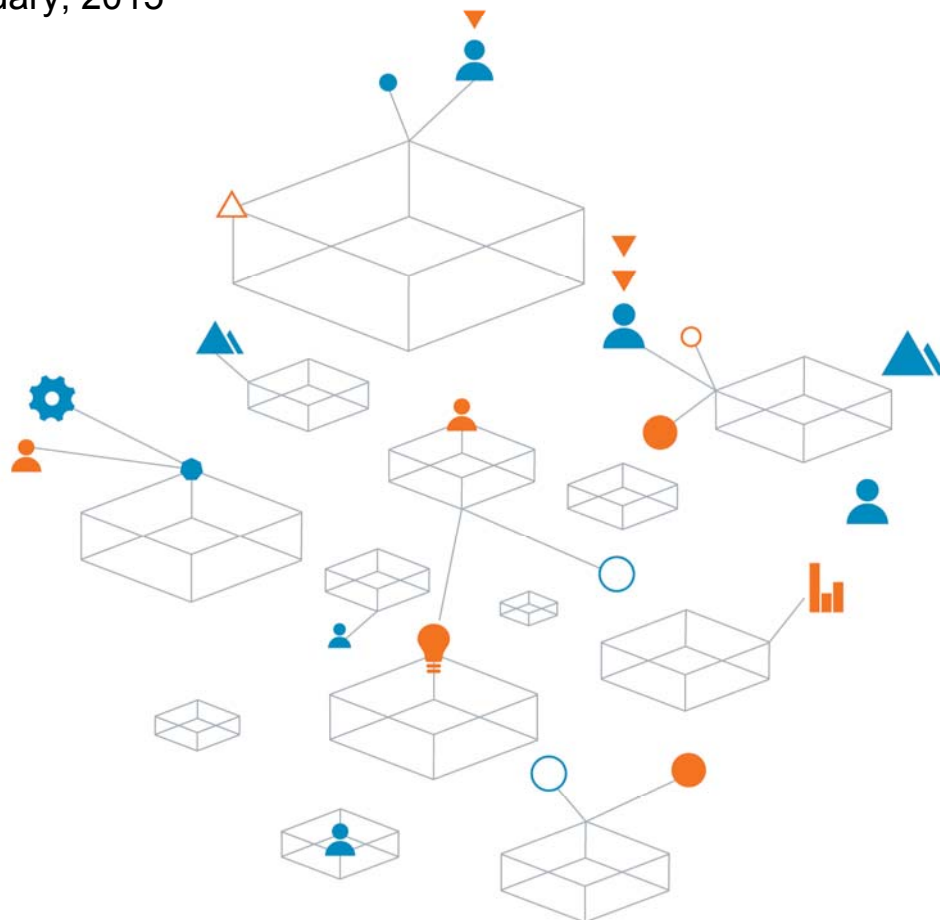


Photograph 4: Borehole F2 @ Station 10+030, Looking South



Preliminary Foundation Design Report

Highway 400/Tiffin Street NB & SB Overpass Structures,
G.W.P. 2074-11-00, Site No's. 30-176/1 & 2,
Design-Build Ready Package, GEOCRES No. 31D-587
GEOTETOB22161AA
11 February, 2015



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of all our
projects

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Appendix D: Tiffin Street Overpass As-built and GA Drawings

Appendix E: Cross-sectional Drawings

Appendix F: MTO Granular 'A' Pad

Appendix G: List of Standard Specifications-OPSSs and OPSDs

Appendix H: NSSPs

Appendix I: Limitations of Report

**PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 400/TIFFIN STREET NB & SB OVERPASS STRUCTURES
G.W.P. 2074-11-00, SITE NO'S. 30-176/1&2,
DESIGN-BUILD READY PACKAGE**

5 Discussions and Recommendations

5.1 General

As part of Highway 400/Tiffin Street Overpass Structure Replacement and Highway 400/Barrie-Collingwood Railway Overhead Structure Rehabilitation, it is proposed to replace the existing Tiffin Street overpass at Station 10+000 (Sta. 29+738 = Sta. 10+000, Hwy. 400 centreline chainage). The project involves realignment of the highway slightly to the east and a grade raise of up to 4.5 m to improve geometrics and safety. The Highway 400 grade raise at Tiffin Street will be in the order of 3.5 m. The proposed staging, that will accommodate the future highway platform widening to 10-12 lanes, is as follows:

- Stage 1 – Construct new NB structure to serve 3 new NB lanes; build a permanent retaining wall along the east ROW for platform widening; construct a temporary retaining wall on the west side of Lane 1 to accommodate the grade raise. Highway 400 NB traffic will be diverted to the new NB structure when completed. SB traffic will be diverted via median cross-overs to the existing NB lanes.
- Stage 2A – Demolish the existing SB overpass; construct new SB structure; provide a temporary retaining wall on the east side of Lane 1 for the grade raise.
- Stage 2B – Demolish the existing NB overpass; construct new retained soil system (RSS) walls to accommodate widening of Tiffin Street to 5 lanes.

As-built drawings (**Appendix D**) show the existing structure and wing walls are supported on spread footings at elev. 233.2 m (assuming 0.6 m thick footings). Tiffin Street grade is at elev. 234.0 m to 234.5 m. The approach fill embankments are 6-7 m in height.

The City of Barrie is planning to widen Tiffin Street from the current one lane in each direction with a middle turning lane to two lanes in each direction with a middle turning lane, bike lanes and sidewalks on both sides. Therefore, the new NB and SB overpasses will have spans of 28 m, and lengths of 18 m. Due to the proposed grade raise, the approach fill heights for both new structures (NB and SB) will be 8-10 m. The new structure sites are underlain by loose to compact stratified sand and silt. The groundwater table is located at about elev. 230.5 m, some 4 m below the ground level adjacent to Tiffin Street.

5.2 Structure Foundations

There are several utilities and services (gas, water, sewer, fibre optic cables, &c.) buried below or adjacent to, or through the approach fills close to the abutments of the existing overpass at Tiffin Street. The new structure foundations will need to avoid or contend with these existing and possibly relocated services. Environmental constraints and concerns (potential TCE contamination just below the groundwater table) have obviated the consideration of a deep foundation alternative for the new structures. Therefore, the new structures will need to be supported on shallow spread footings.

A factored geotechnical resistance of 350 kPa at ULS and 225 kPa at SLS (for 25 mm total settlement and 20 mm differential settlement) are available for foundations placed on the undisturbed natural soil at about elev. 233.2 m, for concentric vertical loading conditions.

If the above-noted ULS and SLS values are insufficient, given 10 m high approach fills and a much longer span for both structures than the existing, it may be necessary to consider other options for shallow foundation support of the new structures. These include, but are not limited to, the following:

- Lightweight fill to reduce stresses from 10 m high embankment fills
- In situ soil treatment (physical and chemical options to increase bearing capacity)
- Soil replacement (e.g., with a compacted granular pad to increase bearing capacity)
- Structure weight reduction

The option selected should not undermine the existing structure in terms of safety and settlement until such time as the new structures have been constructed.

The unfactored horizontal resistance against sliding between poured concrete and the natural silty sand can be estimated using a friction angle of 28 degrees.

5.2.1 Lightweight Fill

Lightweight fill, placed near the top portions of the approach fill, will result in a reduced active pressure moment component on the toe of abutment wall footings and lessen the vertical load component. Extruded polystyrene (EPS) is a demonstrated lightweight fill used behind many bridge abutments where subsoil conditions are unfavourable. However, EPS needs protection against gasoline spills.

5.2.2 In-Situ Soil Treatment

The soil at the new footing locations could be made denser and more competent through physical or chemical means. Physical means include in-place compaction using methods such as dynamic compaction. However, working space limitations and potential adverse impacts on the existing structure rule out this option. Chemical modification may not be permitted owing to concerns already present with respect to potential TCE contamination. Deep soil mixing may not be possible again owing to concerns with disturbing the TCE contamination. The new fairly heavy structures cannot be supported on gravel columns owing to the high water table and risk of collapsing the sandy silt soil owing to dilation during installation and jetting and/or infilling. In addition, any densification or in situ soil treatment will cause imposed stresses to reach deeper into the stratigraphy where the subsurface conditions have not been fully explored.

5.2.3 Soil Replacement

The new structures can be supported at ULS and SLS values higher than stated in Section 5.2 when supported on a compacted granular pad, much the same as many MTO bridge abutments that are currently supported on compacted Granular A pads. For this option, the native soil must be excavated and removed for re-use elsewhere and replaced with compacted Granular A. The pad thickness should be 2.5 m beneath the footing base and 1 m wider than the foundation footprint. An example is illustrated in **Appendix F**.

5.2.4 Structure Weight Reduction

Consideration may be given to reducing the dead weight of the abutments by using a cellular type of footing. Such a footing consists of several open cells surrounded by structural diaphragm walls. The empty cells reduce the mass of the footing, and hence the dead weight of the abutment. .

5.2.5 Recommended Options

From a practical standpoint, soil replacement with compacted Granular A appears to be the most favourable option. For a 2.5 m thick granular pad (Granular A 100% SPMDD), a factored geotechnical resistance / geotechnical reaction of 500 kPa at ULS and 325 kPa at SLS (for 25 mm total settlement and 20 mm differential settlement) are recommended for preliminary foundation design. The granular pad should be placed as per MTO standard (**Appendix F**). These wider and deeper excavations will require temporary roadway protection system for Tiffin Street, highway embankment and structure foundations.

The unfactored horizontal resistance against sliding between poured concrete and compacted granular pad can be calculated using a friction angle of 31 degrees.

The footing excavation base will may be located slightly above the prevailing groundwater table (depending on seasonal weather conditions). The excavation base elevation will be the existing footing base elevation of 233.2 m - 2.5 m = 230.7 m. If 1200 mm thick footings are required (as shown on the latest GA drawings, **Appendix D**), the excavation will extended to elev. 230.1 m and dewatering may be required. If required, groundwater level should be depressed sufficiently below the deepest part of the excavation to maintain basal stability and excavation side slope stability. The Design-Build Contractor should be made aware of his responsibilities which include the following:

- Engaging a dewatering specialist to provide advice on the most appropriate method for dewatering
- Ensuring that dewatering by the method chosen does not disturb the TCE plume below the phreatic surface
- Obtaining the necessary dewatering permits
- Testing of pumped water for disposal in accordance with prevailing regulations for surface disposal
- Municipal approval and testing of pumped water for discharge into a storm sewer system.

A major concern with the granular pad option is the safeguarding of the existing foundations. Excavation sides and bases that lie within 45 degree lines drawn down from the base of the existing footing could be cause for distress to the existing bridge. Where this situation occurs, a non-yielding type of shoring system will be required. The design of such systems is the responsibility of the design-build contractor.

If TCE contamination is not present at this site, consideration should be given to supporting the new structures on a deep foundation. The selection of an appropriate deep foundation support system will depend on the stratigraphy at depth, for which additional subsurface investigations will be required.

5.3 New NB Embankment Construction

A new north bound embankment will be constructed, supported by a permanent retaining wall along the east ROW, with temporary retaining walls required near to the existing NB edge of pavement. Due to the loose to compact soil conditions along the east side of the existing highway and anticipated height of new embankment, the proposed embankment should be constructed in stages to reduce total and post-construction residual settlements. The settlements will be primarily elastic.

5.3.1 Permanent Retaining Walls

The proposed new NB overpass structure construction includes the construction of a permanent retaining wall on the east side. The proposed wall height near the structure location is about 10 m (**Appendix E**). Due to space limitations, vertical walls are proposed. Based on the finding of retaining wall boreholes (presented in a separate foundation report) drilled along the east side of the existing highway, a

conventional concrete retaining wall is not feasible due to the prevailing loose to compact soil conditions. A more settlement insensitive wall, such as RSS, is better suited to the site subsurface conditions. High performance and high appearance RSS walls are recommended for the proposed permanent retaining wall.

Typically, RSS wall facing is supported on a granular bearing pad placed below the frost depth (1.5 m). The same geotechnical resistance and reaction values provided in section 5.2 for shallow foundations may be used for preliminary permanent retaining wall design at Tiffin Street. These geotechnical resistance and reaction values should be verified during detailed site investigation and design phases, with consideration of MTO “*Embankment Settlement Criteria for Design*” issued on July 2010.

For proper abutting between new and existing embankment fills, *OPSD208.010 Benching of Earth Slopes* should be applied.

The RSS supplier and wall designer are responsible for RSS wall internal stability. Highway traffic loads should be considered for the wall design, as applicable. The sliding and overturning of the wall should be checked by the wall designer. Global stability of the RSS wall should be assessed when detailed wall design drawings become available.

5.3.2 Temporary Retaining Walls

The maximum height of a temporary retaining wall will be about 3.5 m at Tiffin Street (**Appendix E**). Conventional cast-in-place concrete walls or RSS walls may be selected to retain the proposed grade raise. The existing highway embankment or newly constructed RSS embankment can support the proposed grade raise and retaining walls. For preliminary design purposes, the ULS and SLS values given in Table 5.3.2.1 may be used.

Table 5.3.2.1

Foundation Soil	ULS (kPa)	SLS (kPa)*
Newly constructed embankment (Granular B Type I)**	225	150
Existing embankment	200	120

*SLS for 50 mm total settlement

**compacted to minimum 95% of SPMD

Close to the bridge abutments, consideration may also be given to extending the roadway protection wall to support the new NB embankment, with additional reinforcement (such as deadmen and anchors), as necessary. Roadway protection should also be designed to support the loading induced by the proposed temporary retaining wall depending on the detail configuration of bridge abutment and approach embankment.

5.4 SB Embankment Reconfiguration

The existing SB bound slope will be widened toward the west ROW and a temporary retaining wall will be placed close to the existing highway centreline.

5.4.1 Embankment Widening

About 10 m embankment widening (see **Appendix E**) is proposed towards the west side of the existing highway without retaining walls. 2H:1V embankment side slopes, similar to the existing embankment, can be used for the proposed widening. Embankment widening should be carried out in accordance with *OPSS.PROV206 Construction Specification of Grading*, *OPSS 501 Construction Specification for Compacting*. The existing embankment side slopes should be benched as per Ontario Provincial Standards (*OPSD208.010 Benching of Earth Slopes*).

Sub-excavation and replacement with approved granular materials will be required where unsuitable subgrade conditions are encountered.

The soil for the widening of the approach embankments should consist of approved, acceptable earth borrow, free of cobbles and boulders, frozen materials, organic soils, etc. The fill should be placed in lift thicknesses not exceeding 200-300 mm before compaction (thicker loose lift for coarser material). Each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The compaction should be not less than 98% of the material's SPMDD within 1 m of pavement subgrade. Settlement below embankment widening will occur concurrently with placement and compaction of new fill. Post-construction settlement of the new fill, relative to the existing fill, is anticipated not to exceed 50 mm within one month of completion of filling, not including self-weight compaction of the new fill. The magnitude of self-compaction settlement will be determined by the type of fill material used, degree of quality control provided during construction and similar variables that cannot be predicted in advance. For preliminary design purposes, the total settlement of 8 m high fills is expected to be 100 mm, of which 75% should be achieved by the time of completion.

Embankments greater than 8 m in total height above the surrounding ground level should be provided with mid-height benches as per *OPSD 202.010 slope flattening using surplus excavated material on earth and rock embankment*. Embankment side slopes should be protected using sodding or seed and cover (OPSSs 571 and 572).

5.4.2 Temporary Retaining Walls

The recommendations given in Table 5.3.2.1 and Section 5.3.2 also apply to these walls.

5.5 Demolition of Existing NB Structure and Retaining Walls

Once both the new NB and SB overpass structures are completed at Tiffin Street, at a higher grade than present, temporary and permanent soil retention will be required for demolition of the existing structures and retention of the high fills.

Consideration can be given to the use of cast-in-place concrete walls or high performance high appearance retained soil system (RSS) walls. Consideration may also be given to using the existing structure foundations to support the proposed retaining walls until such time as Tiffin Street widening occurs.

5.6 Lateral Earth Pressures

Backfill behind structures and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with *OPSD 3101.150*. Free-draining backfill (Granular 'A' or Granular 'B' Type I or Type II, with less than 5-7% fines. The provision of drain pipes and weep holes should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with CAN/CSA-S6-06. For design purposes, the following unfactored static parameters are recommended.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, $\phi = 35^\circ$

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

$K_A = 0.27$

$K_O = 0.43$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$$K_A = 0.31$$

$$K_O = 0.47$$

The effect of compaction should be taken into account in the selection of earth pressure coefficients. The use of vibratory equipment behind abutment walls and retaining structures should be restricted in size as per current MTO policy.

5.7 Seismic Design Consideration

The subsurface conditions encountered at the site are represented by Soil Profile Type II (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design therefore, in accordance with Clause 4.4.6.1, the site coefficient, $S = 1.2$. Table A3.1.1 of CAN/CSA-S6-06 provides that Toronto has a Zonal Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Z_v) of zero. As site coefficient (S) = 1.2, and the zonal acceleration = 0.05, the design zonal acceleration ratio for the site can be taken as $A = 0.06$. This bridge site can be classified as Seismic Performance Zone 2 based on the above values. This should be reviewed by a Structural Engineer.

5.8 Construction Considerations

All excavations, shoring and backfilling should be carried out in conformance with the *Occupational Health and Safety Act (OHSA)*, *Regulation 213/91*, as well as the following specifications.

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures.

Excavations can be expected to extend through embankment fill and native granular soil deposits. These soils can be classified as follows:

Fill	Type 3 soil above water level
Native Granular Soils	Type 3 soil above water level
	Type 4 soil below water level

Temporary shoring will be required to retain the existing embankment during new structure construction and to support excavations below existing foundation levels. The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required Performance Level is 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. Relevant parameters for design are given in Table 5.8.1. All shoring should be in accordance with *OPSS 539*. Due to the height of retaining wall and environmental constraints (TCE contamination), additional reinforcement such as tie-back or deadmen anchors should be considered to meet the performance criterion.

Table 5.8.1 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	Unit weight γ (kN/m ³)
Embankment Fill	0.36	0.53	2.77	19.5
Silty Sand to Sandy Silt, Silt to Sand & Silt	0.36	0.53	2.77	19.5

It should be pointed out that the cobble size particles and even boulders can possibly occur within the embankment fill as well as rock fill. If encountered, these can cause problems during the installation of shoring systems.

As a precaution, it would be prudent to monitor vibrations during the driving of shoring support units (e.g., sheet piling) close to the existing structure and utilities (if any).

For preliminary design purposes, the capacity of pressure grouted anchors used to provide restraint to earth retention walls in excavations may be assumed to be 100 kN/m, in the natural soil above elev. 225 m, based on typical values published by the Federal Highway Administration (FHWA 1984).

5.9 Frost Depth

The design frost protection depth for the Tiffin Street bridge site is 1.5 m.

5.10 Underground Utilities

Existing underground utilities and buried services within the work area should be properly protected during construction. If required, vibration studies and pre- & post-construction surveys should be implemented.

5.11 Instrumentation and Monitoring

Due to the anticipated construction and demolition in the proximity of the existing and newly built structures and embankments, appropriate monitoring programs (embankment and foundation settlement, pore pressure generation rate and magnitude and speed of dissipation, excavation behaviour, movement of temporary roadway protection systems, etc.) should be developed and implemented during, and for two months after completion of, construction.

5.12 Dewatering Impacts

Dewatering at the Tiffin Street bridge site to a permanent static groundwater table that is more than 1.0 m below the existing may result in ground settlements that may be detrimental to the performance of nearby structures. It is recommended that any permanent dewatering scheme be accompanied by preconditions surveys and settlement monitoring.

6 Scope of Work Required for Detailed Design

Due to environmental constraints and the DB nature of the project, this investigation falls short of MTO requirements for both lateral coverage of boreholes and depth of borings for the proposed structures. It may become necessary to drill additional and deeper boreholes to comply with *RFP, Appendix 6.8, Minimum Requirements for Foundations Engineering Applications*, unless waived by the MTO.

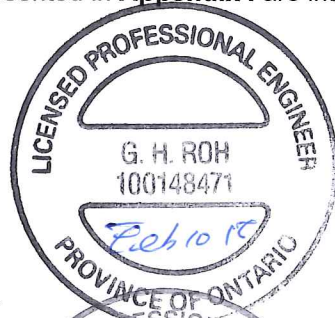
7 Closure

The "Limitations of Report" as presented in **Appendix I** are integral part of the report.

For and on behalf of Coffey.



Gwangha Roh, P.Eng., Ph.D.
Associate Geotechnical Engineer



Sanket Shah, P.Eng.
Project Manager, Geotechnical Engineer



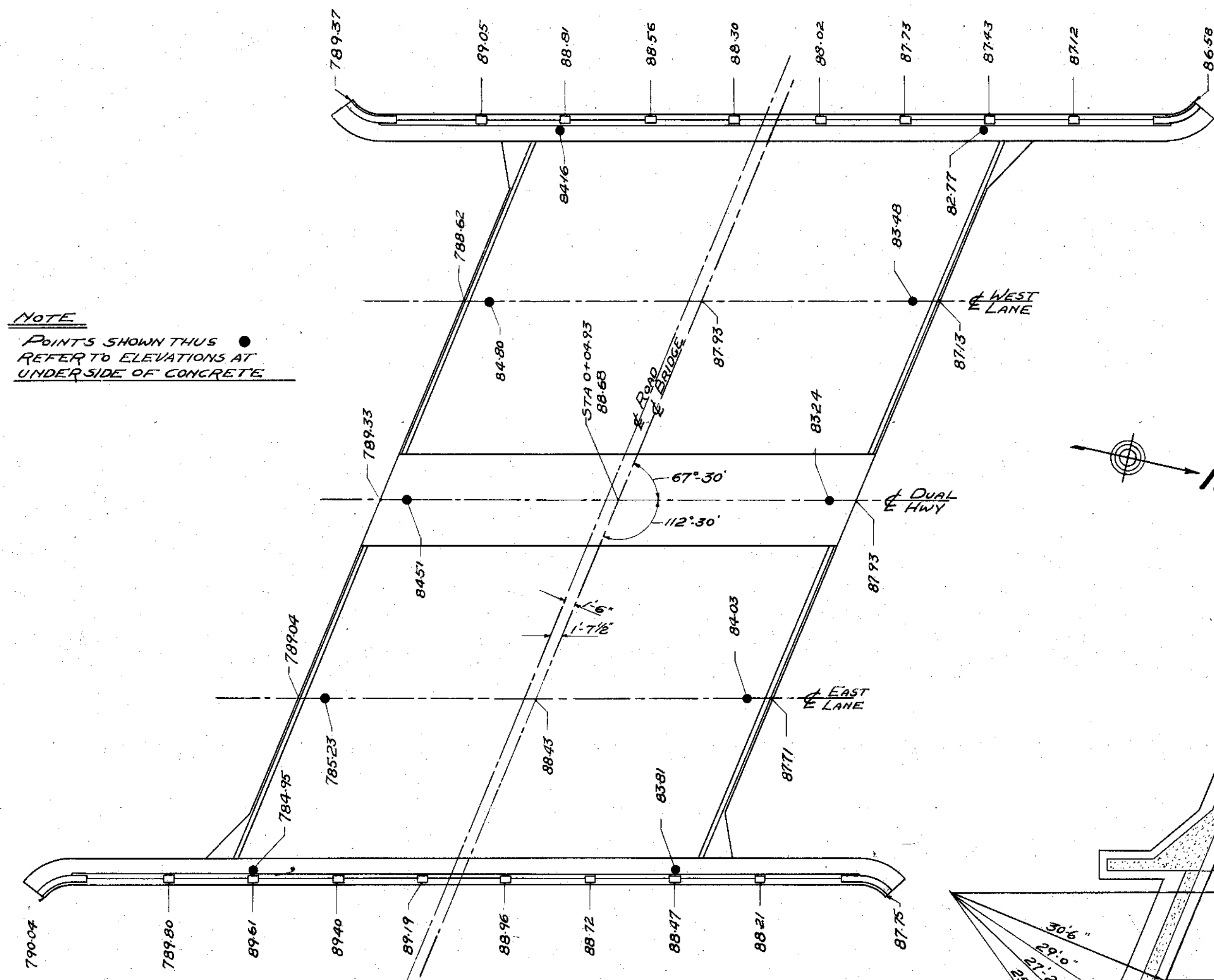
Cam Mirza, P.Eng.
MTO Designated Contact, Principal



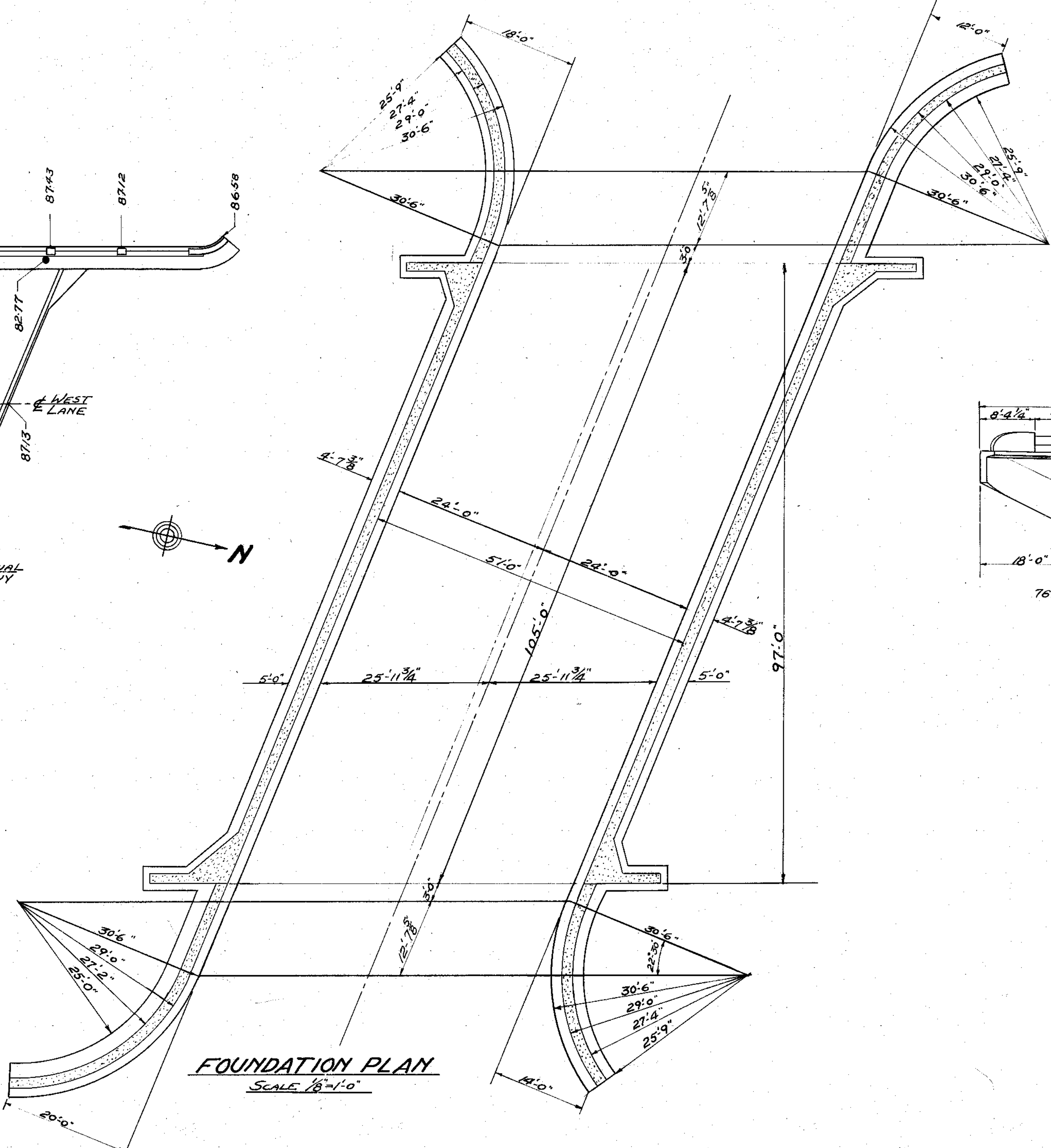
Appendix D

Tiffin Street Overpass As-built and GA Drawings

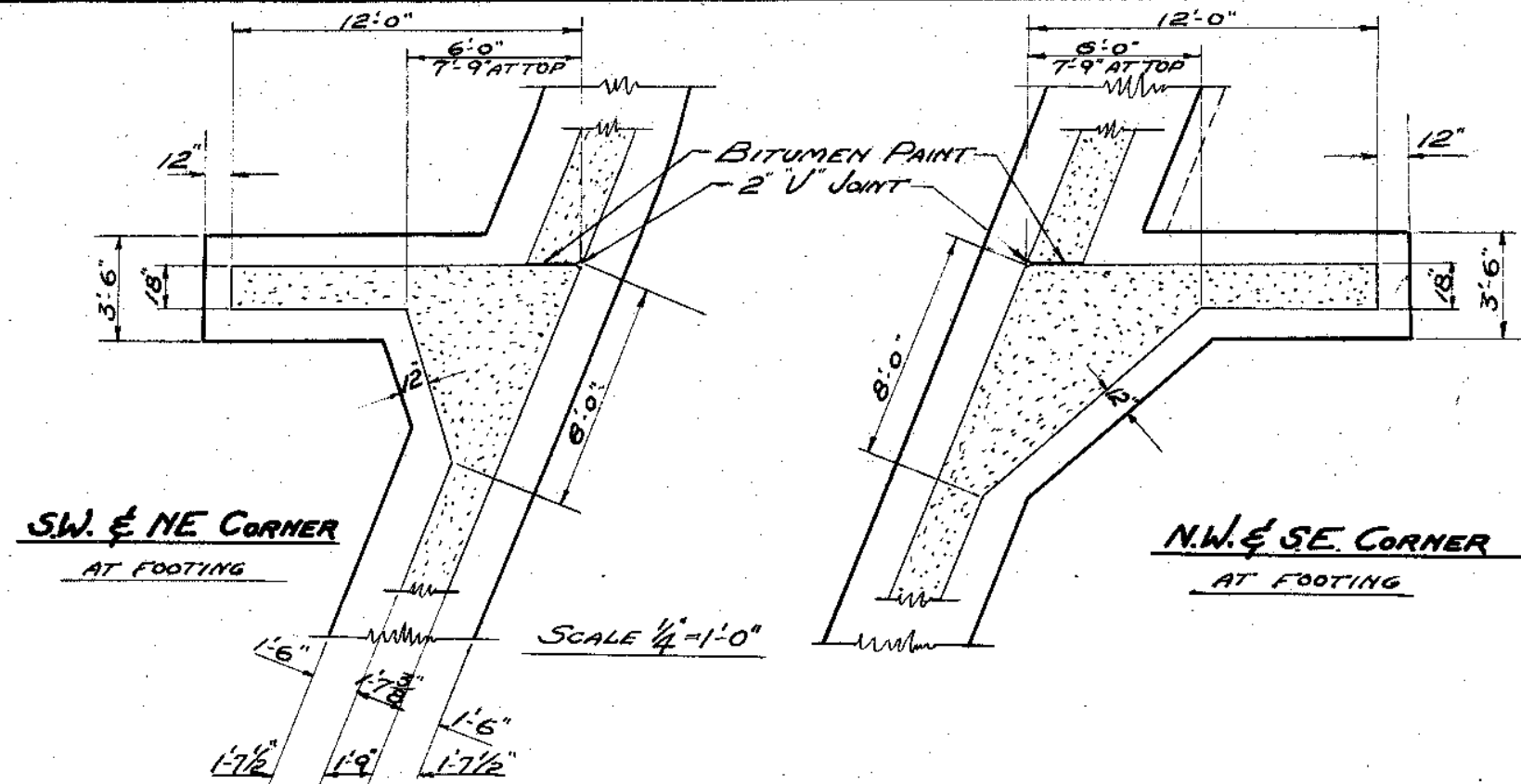
PRINT RECORD		
N.	FOG	DATE
25	Tender	5-3-50
1	Contract	10-2-50
1	Order	8-5-50
7	Order	2-6-50



DECK PLAN
SCALE 1/8" = 1'-0"



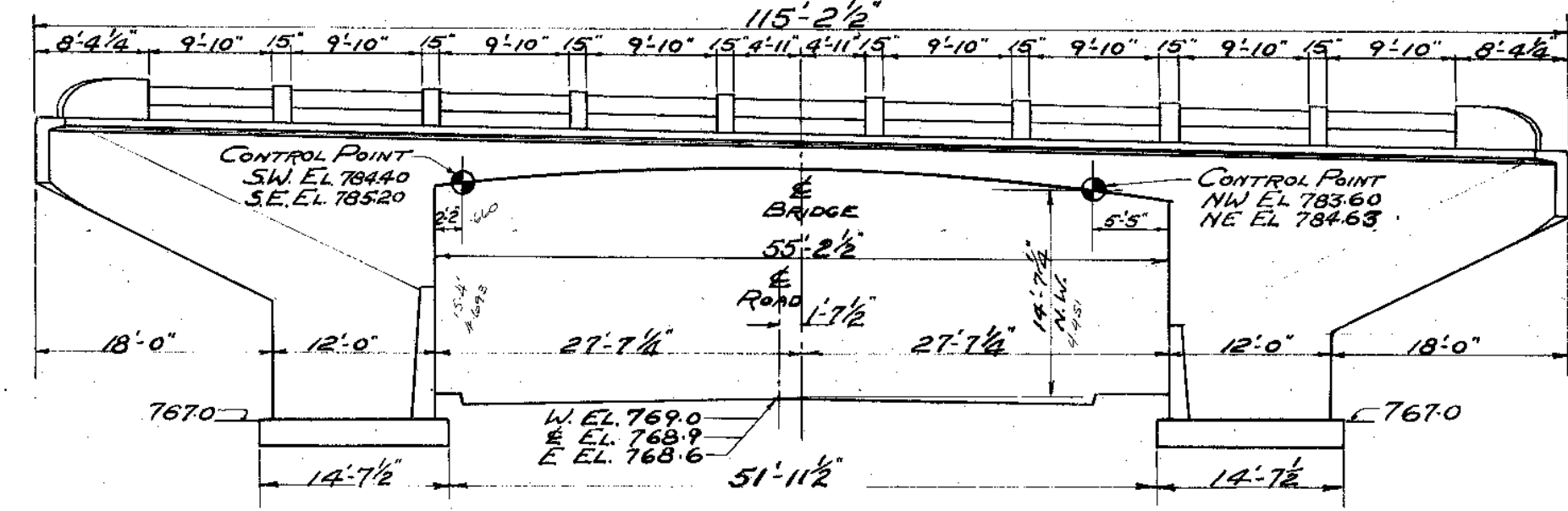
FOUNDATION PLAN
SCALE 1/8" = 1'-0"



SW. & NE CORNER AT FOOTING

NW & SE CORNER AT FOOTING

SCALE 1/4" = 1'-0"



EAST ELEVATION
SCALE 1/8" = 1'-0"

NOTE:-
CONTROL POINT ELEVATIONS MUST BE CHECKED BY THE ENGINEER BEFORE THE CONCRETE IS POURED.

NOTE FOR DIVISION ENGINEER

CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE ENGINEER.

NOTE FOR CONTRACTOR

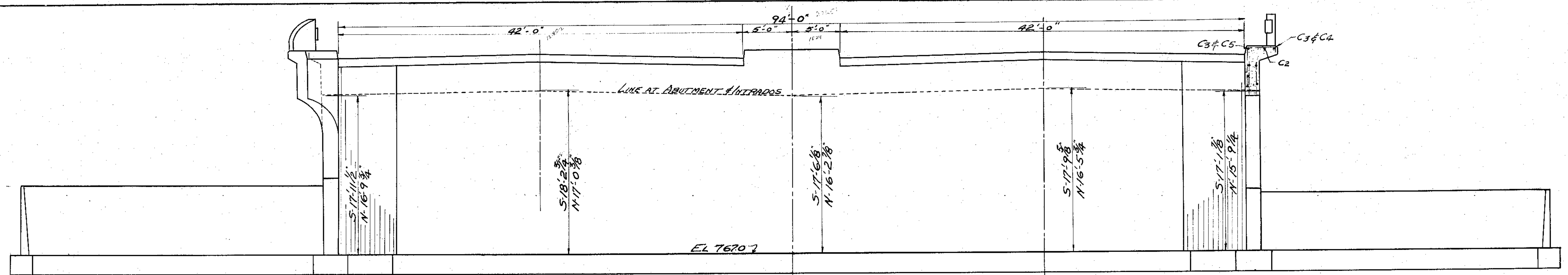
STRUCTURE TO BE BUILT IN ACCORDANCE WITH D.H.O. GENERAL SPECIFICATIONS FOR HIGHWAY BRIDGES FORM NO. 9 AND THE SPECIAL SPECIFICATIONS ATTACHED TO THE "INFORMATION TO BIDDERS" SHEET.

CONCRETE MIX

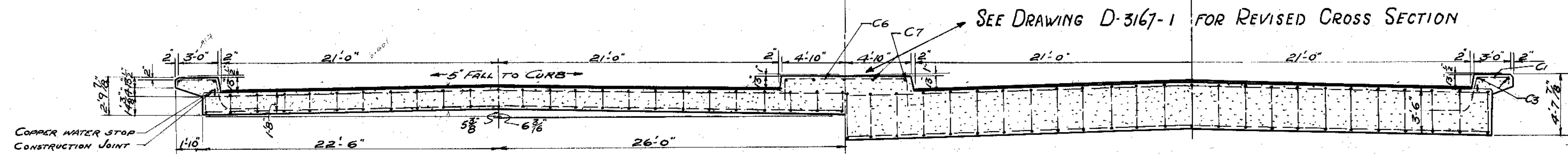
FOOTINGS - CLASS B 1:2:4
ENTIRE STRUCTURE ABOVE FOOTINGS CLASS A 1:1 3/4:3 1/2
ADD 1/2 LB. OF POZZOLITH TO EACH BAG OF CEMENT

SHEW - 22°-30'
SIN - 38268
COS - 92388
TAN - 41421

BRIDGE No. 2	
DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO	
PROPOSED OVERPASS	
BARRIE BY-PASS AT HWY No. 90	
THE KING'S HIGHWAY NO.	DIV. NO. 39
CO.	
TWP. VESPRE	LOT CON.
PLAN & ELEVATION	
APPROVED _____ 30-176	
CHIEF BRIDGE ENGINEER	
DESIGN WMB	CHECK RP
DRAWING WMB	CHECK RP
TRACING WMB	CHECK RP
DATE MARCH 1950	LOADING H20
DRAWING NUMBER 50-10	
DRAWING NUMBER D-3110-1	



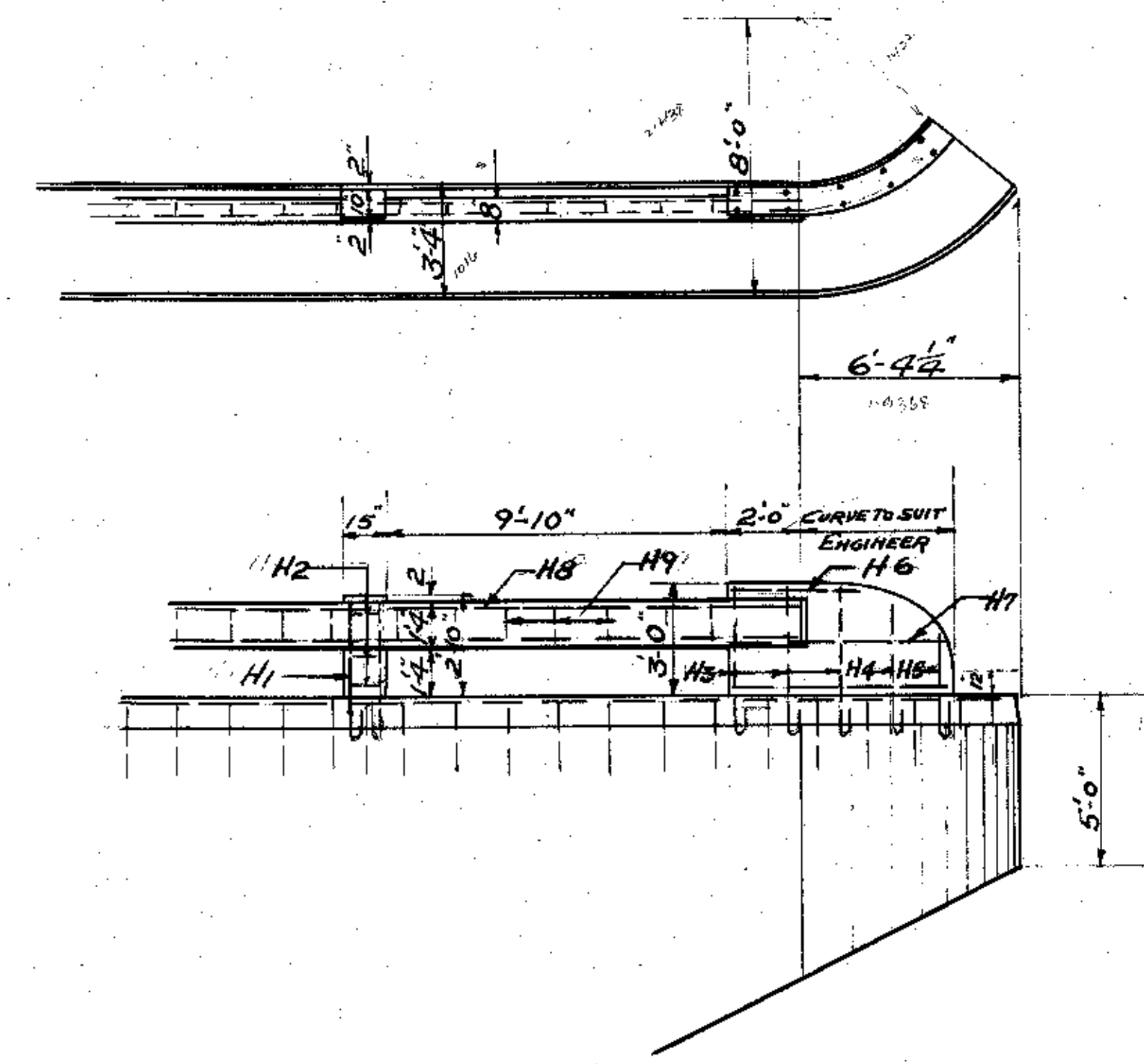
ELEVATION NORMAL TO DUAL HIGHWAY SHOWING SECTION AT N.W. WING WALL



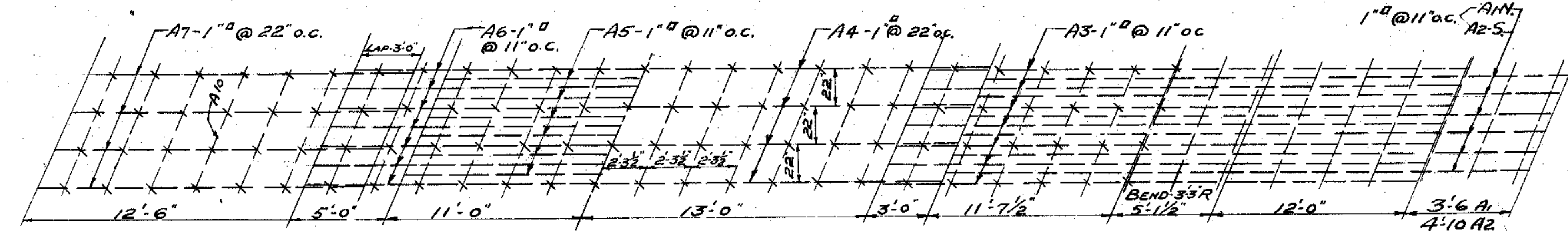
SECTION AT CENTRE NORMAL TO DUAL HIGHWAY

SECTION AT ABUTMENT NORMAL TO DUAL HIGHWAY

NOTE
1" CHAMFER REQUIRED
FOR POST & RAIL

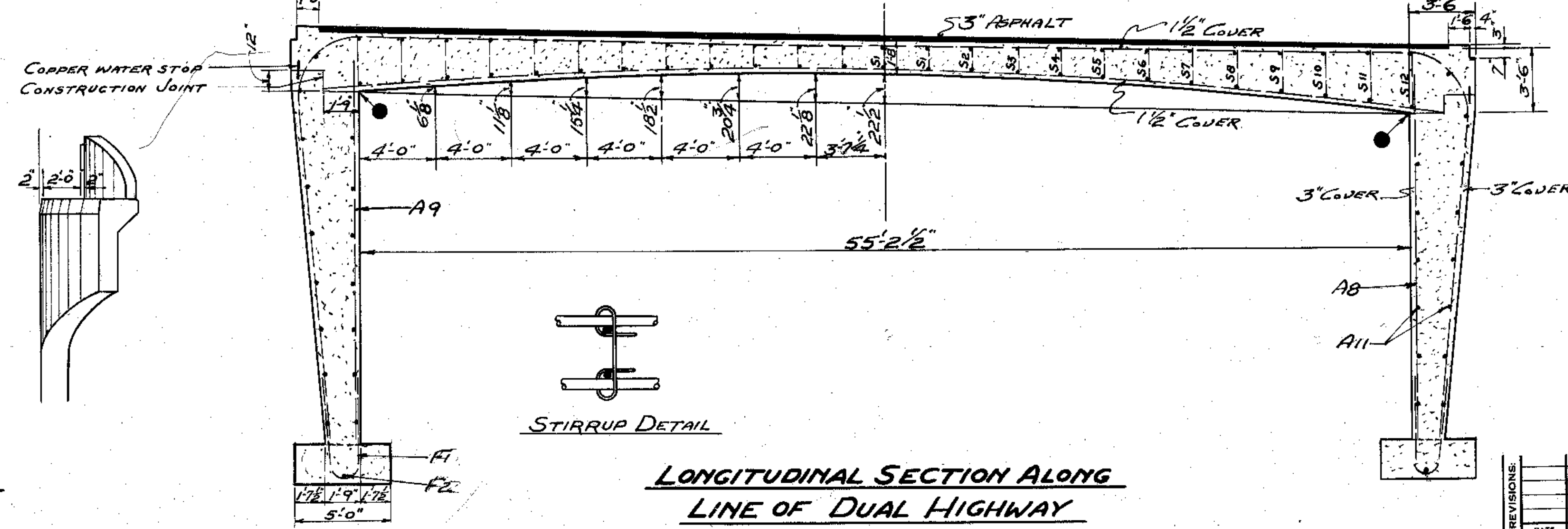


CURB AND HANDRAIL DETAILS

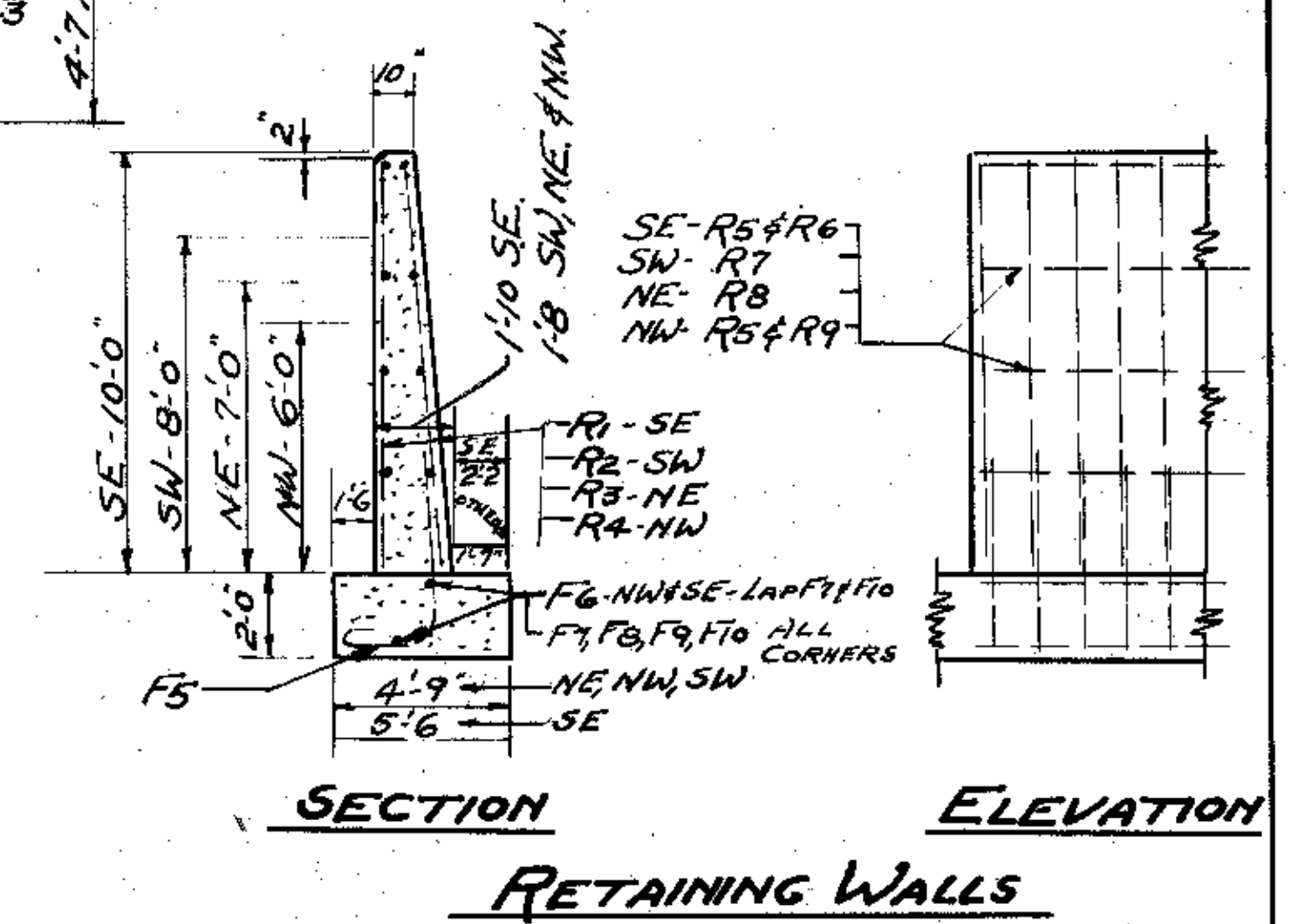


INTRADOS STEEL

EXTRADOS STEEL



LONGITUDINAL SECTION ALONG
LINE OF DUAL HIGHWAY



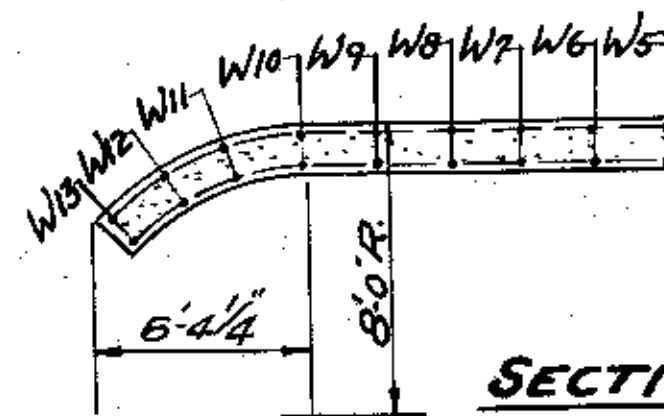
SECTION
RETAINING WALLS

BRIDGE No 2			
DEPARTMENT OF HIGHWAYS-ONTARIO			
BRIDGE OFFICE-TORONTO			
PROPOSED OVERPASS			
BARRIE BY-PASS AT HWY No 90			
THE KING'S HIGHWAY No.		DIV. No. 39	
CO.		CON.	
TWP. VESPRE		LOT	
DETAILS			
APPROVED			
CHIEF BRIDGE ENGINEER		CHIEF ENGINEER	
DESIGN	STP	CHECK	RR
DRAWING	WMB	CHECK	RR
TRACING	WMB	CHECK	RR
DATE	BY	DESCRIPTION	50-10
TWP. 74-156-2-A		78-12-2	

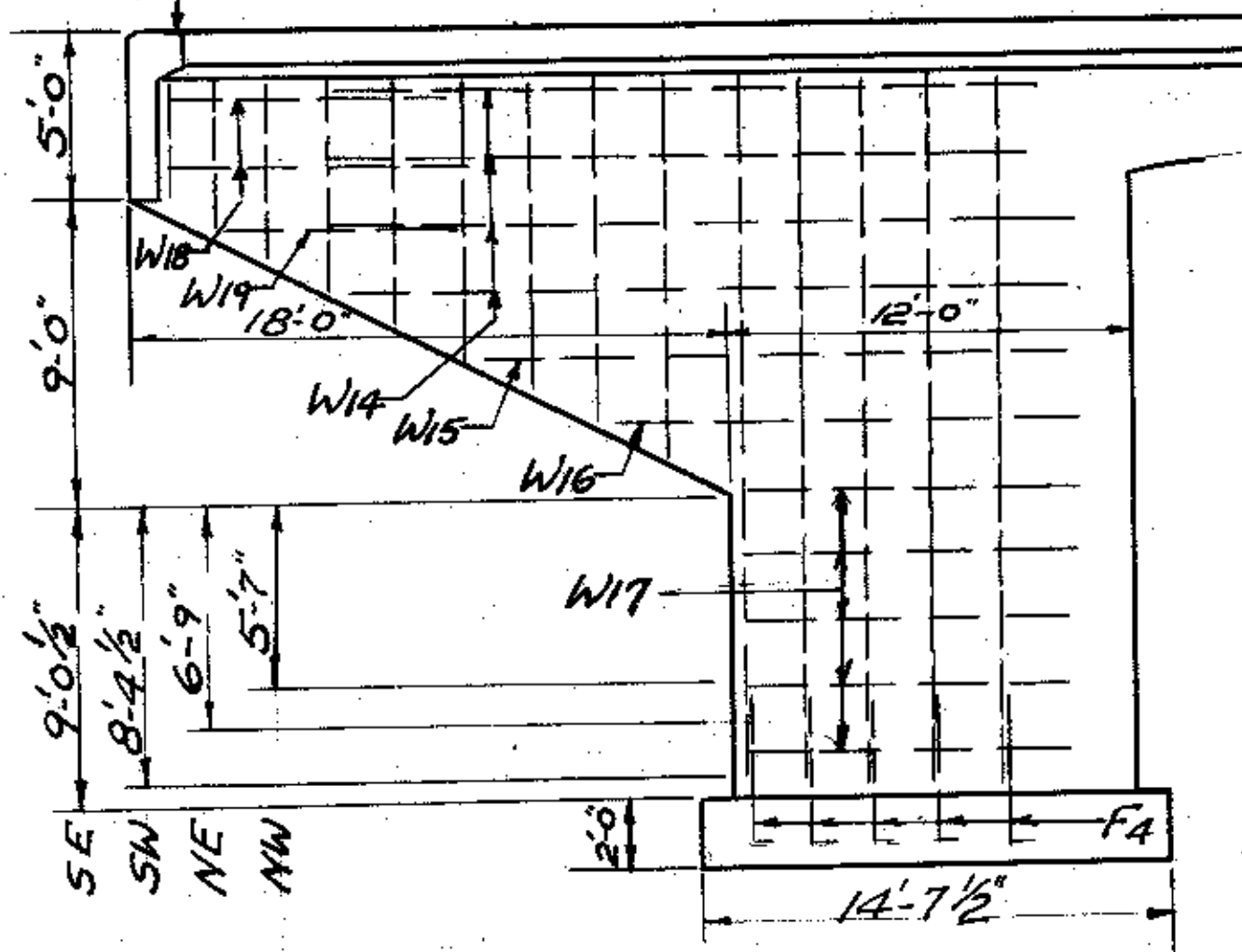
NO.	FOR	DATE
25	Tender	8-3-50
1	CHECK	10-4-50
1	"	8-5-50
7	Q. L. E.	8-5-50

30-150

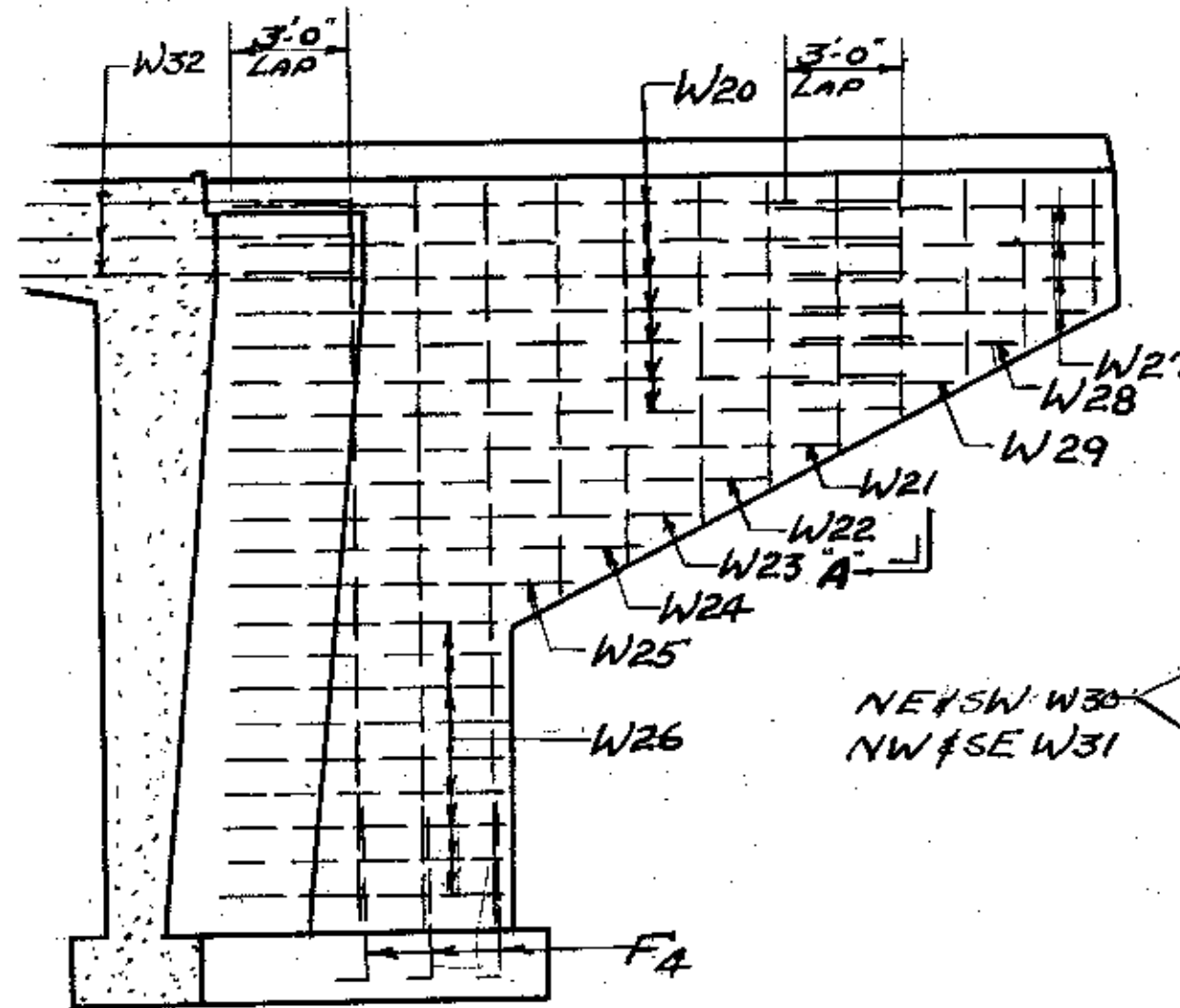
W1-SE
W2-SW
W3-NE
W4-NW



SECTION PLAN

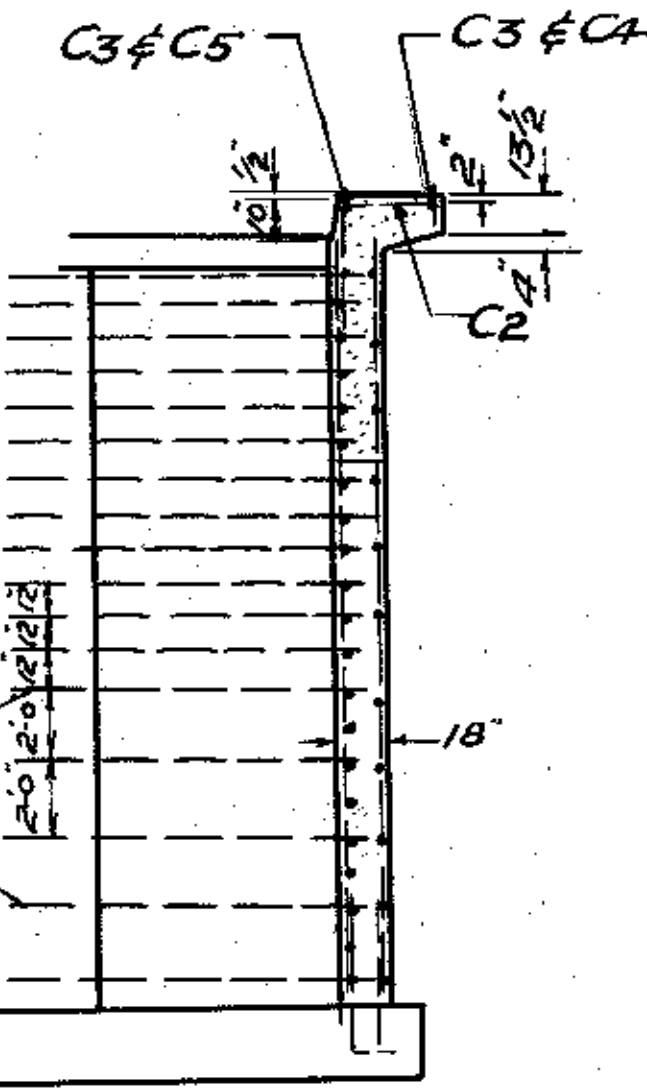


OUTER FACE



INNER FACE

WING WALLS
SCALE $\frac{3}{16} = 1'-0"$



SECTION A-A

PRINT RECORD		
NO.	FOR	DATE
25	Tender	8-3-50
1	CHECK	10-4-50
1	"	8-5-50
7	D.S.L.F.	26.5.50

BRIDGE No 2

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

PROPOSED OVERPASS
BARRIE BY-PASS AT HWY No 90
THE KING'S HIGHWAY No. DIV. No. 39

CO. TWP. VESPRE LOT CON.

WING WALL DETAILS

APPROVED

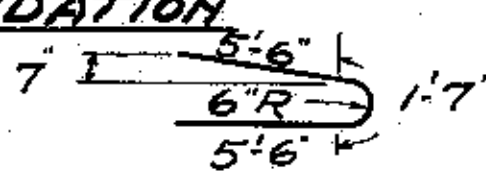
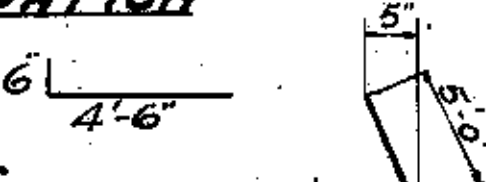


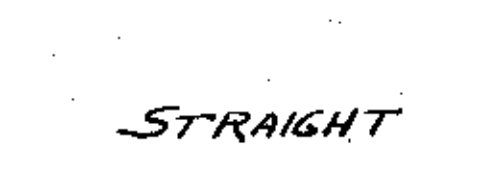
CHIEF BRIDGE ENGINEER CHIEF ENGINEER

DESIGN	WMB	CHECK	R.P.	CONTRACT	NUMBERS	50-10
DRAWING	WMB	CHECK	R.P.	LOADING	NUMBER	D-340-3
TRACING	WMB	CHECK	R.P.	DATE		

REVISIONS:	DATE	BY	DESCRIPTION

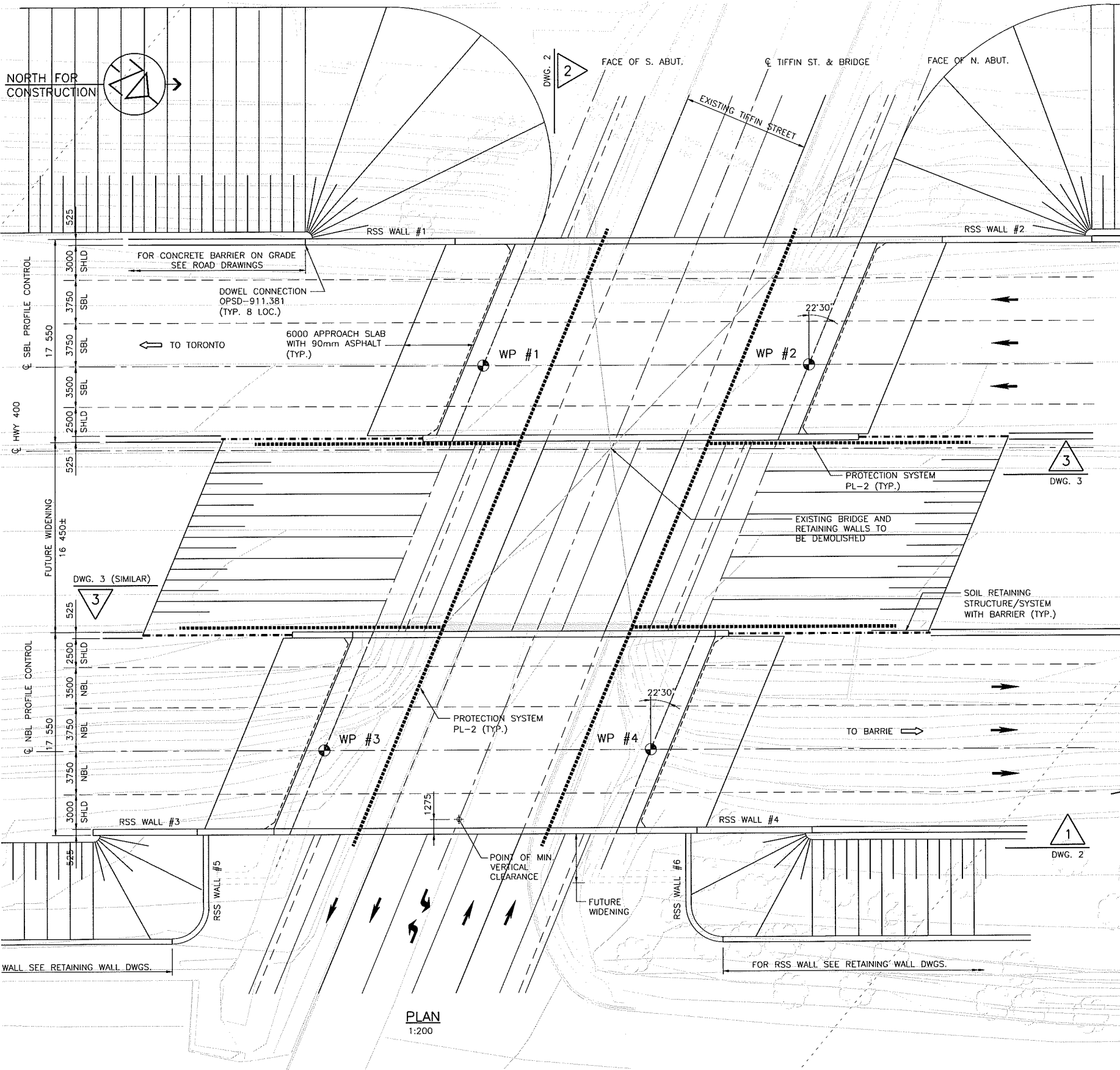
30-176 TWP# 78-176-3-A

78-12-3

MARK	No.	SIZE	LENGTH	DETAILS	REMARKS
ABUTMENT FOUNDATION					
F1	212	3/4"	12'-7"		@ 11" o.c.
F2	8	5/8"	28'-0"	STRAIGHT	2 LINES 4 PER LINE LAP 2'-0"
WING WALL FOUNDATION					
F4	56	5/8"	5'-0"		@ 2'-0" o.c. AS SHOWN
RETAINING WALLS					
F5	161	5/8"	9'-1 1/2"		IN ALL WALLS @ 12" o.c.
F6	4	D	18'-0"	STRAIGHT	IN NW & SE
F7	2	D	37'-0"		IN SE LAP F6 2'-0" ±
F8	2	D	35'-6"		IN SW
F9	2	D	32'-0"		IN NE
F10	2	D	29'-0"		IN NW LAP F6 2'-0" ±
ARCH EXTRADOS					
A1	106	1"	32'-3"		IN NORTH @ 11" o.c.
A2	106	1"	33'-7"	STRAIGHT	IN SOUTH D.
A3	210	1"	31'-9"		
A4	53	1"	32'-0"		
ARCH INTRADOS					
A5	105	1"	22'-0"	STRAIGHT	BOTH SIDES @ 11" o.c.
A6	106	1"	32'-0"	D.	@ 22" o.c.
A7	106	1"	15'-6"	D.	@ 11" o.c.
A8	106	3/4"	17'-3"	D.	D.
A9	106	3/4"	18'-6"	D.	@ 22" o.c.
TIES FOR DECK					
A10	208	5/8"	27'-9"	STRAIGHT	IN NORTH ABUT. @ 11" o.c.
TIES FOR ABUTMENT					
A11	120	5/8"	27'-9"	STRAIGHT	IN SOUTH ABUT. D.
STIRRUPS					
S1	159	3/8"	3'-1"		BEND
S2	106	D	3'-2"		IN
S3	D	D	3'-2 3/4"		FIELD
S4	D	D	3'-3 1/2"		PLACE
S5	D	D	3'-4 1/2"		AS
S6	D	D	3'-6"		SHOWN
S7	D	D	3'-8"		
S8	D	D	3'-10"		
S9	D	D	4'-1"		
S10	D	D	4'-4"		
S11	D	D	4'-7"		
S12	D	D	4'-11"		
WING WALLS VERTICLE					
W1	14	5/8"	21'-6"	STRAIGHT	IN SE. @ 24" o.c.
W2	12	D	20'-10"	D.	IN SW D.
W3	12	D	19'-3"	D.	IN NE D.
W4	14	D	18'-0"	D.	IN NW D.
W5	8	D	12'-0"	D.	IN ALL W.C. WLS
W6	8	D	11'-0"	D.	D.
W7	8	D	10'-0"	D.	D.
W8	8	D	9'-0"	D.	D.
W9	8	D	8'-0"	D.	D.
W10	8	D	7'-0"	D.	D.
W11	8	D	6'-0"	D.	D.

MARK	No.	SIZE	LENGTH	DETAILS	REMARKS
W.G. WLS CONT'D					
W12	8	5/8"	5'-0"	STRAIGHT	IN ALL W.G. WLS
W13	8	D ₆	4'-0"	STRAIGHT	D ₆
WING WALLS HORIZONTAL OUTER FACE					
W14	16	5/8"	22'-0"	STRAIGHT	@ 24" O.C. IN ALL W.G. WLS
W15	4	D ₆	18'-0"	D ₆	D ₆
W16	4	D ₆	14'-0"	D ₆	D ₆
W17	18	D ₆	10'-0"	D ₆	D ₆
W18	8	D ₆	8'-0"		LAP W14-3'-0"
W19	4	D ₆	6'-0"	R-6'8"	D ₆
INNER FACE					
W20	28	1"	20'-0"	STRAIGHT	@ 12" O.C. IN ALL W.G. WLS
W21	4	D ₆	18'-0"	D ₆	D ₆
W22	4	D ₆	16'-0"	D ₆	D ₆
W23	4	D ₆	14'-0"	D ₆	D ₆
W24	4	D ₆	12'-0"	D ₆	D ₆
W25	4	D ₆	10'-0"	D ₆	D ₆
W26	32	5/8"	8'-0"	D ₆	D ₆
W27	16	3/4"	9'-0"		LAP W23 3'-0"
W28	4	D ₆	7'-0"	R 7'-10"	D ₆
W29	4	D ₆	5'-0"		D ₆
HAUNCHES					
W30	33	5/8"	9'-4"		IN NE & S.W.
W31	33	D ₆	13'-4"		IN NW & S.E.
CURBS					
STIRRUPS ON BRIDGE EXT.					
C1	82	5/8"	6'-1/2"		3 NEAR POST AS SHOWN @ 18" O.C. ALONG CURB
STIRRUPS ON WING WALLS					
C2	84	5/8"	6'-4"		D ₆
TIES FOR ABOVE					
C3	16	5/8"	27'-4"	STRAIGHT 4 LINES - 4 PER LINE	LAP 2'-0" ±
C4	4	D ₆	7'-0"		C4 - OUTER LAP C3 2'-0" ±
C5	4	D ₆	9'-0"	C4 - R: 4'-11" C5 - R: 7'-9"	C5 - INNER LAP C3 2'-0" ±
CURB - BRIDGE INTERIOR.					
C6	32	5/8"	11'-8"		@ 2'-0" O.C.
C7	15	D ₆	21'-3"	STRAIGHT 5 LINES - 3 PER LINE	TIES FOR C6 LAP 2'-0" ±
HAND RAIL POSTS - INTERMEDIATE					
H1	64	3/4"	4'-10"	1-4"	4 PER POST
H2	48	3/8"	3'-8"	7"	3 D ₆
END POSTS					
H3	24	3/4"	5'-0"	1-4"	6 PER POST
H4	8	D ₆	4'-4"		2 D ₆
H5	8	D ₆	3'-2"		2 D ₆

MARK	NO.	SIZE	LENGTH	DETAILS	REMARKS
END POSTS CONT'D					
H6	4	1/2"	9'-7 1/2"		PLACE AS SHOWN
H7	8	1/2"	12'-7 1/4"		Do
HAND RAIL					
H8	32	3/4"	28'-7"	STRAIGHT	8 LINES - 4 PER LINE LAP 3'-0"
H9	126	3/8"	3'-4"	5"	7 PER RAIL @ 18" ± oc
RETAINING WALL					
R1	100	5 1/8"	9'-9"	STRAIGHT	IN SE @ 12" oc.
R2	72	Do	7'-9"	Do	IN SW Do
R3	66	Do	6'-9"	Do	IN NE Do
R4	84	Do	5'-9"	Do	IN NW Do
R5	14	Do	18'-0"	STRAIGHT	IN NW & SE
R6	8	Do	37'-0"		IN SE LAP R5 @ 2'-6" oc
R7	6	Do	35'-6"		IN SW Do
R8	6	Do	32'-0"		IN NE Do
R9	6	Do	29'-0"		IN NW LAP R5 Do
W32	12	3/4"	10'-0"		STRAIGHT
BRIDGE No 2					
WEIGHT OF STEEL - 58 TONS					
REQUISITION No. 56373					
DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO					
REINFORCING STEEL TABLE FOR PROPOSED OVERPASS BARRIE BY-PASS AT HWY No 90					
THE KING'S HIGHWAY No. _____ DIV. No. 39					
CO. _____ LOT _____ CON. _____					
TWP. NESPra					
APPROVED _____					
CHIEF BRIDGE ENGINEER					
CONTRACT NUMBER 50-10					
DATE MARCH 1950					
DRAWING W-3 CHECK R.P. DRAWING NUMBER D-3110-4					
NOTE: ALL STEEL TO BE HARD GRADE EXCEPT WHERE NOTED.					
REVISIONS					
DATE BY DESCRIPTION					



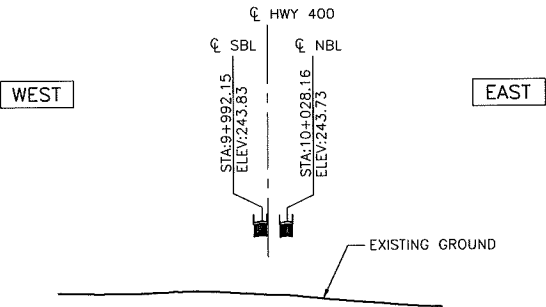
PLAN
1:200

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

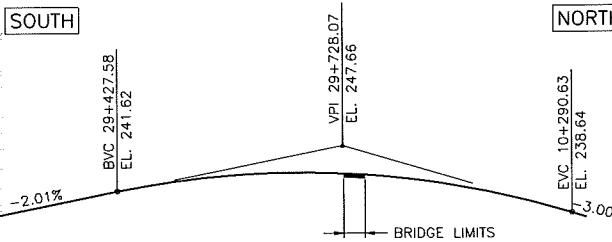
WP#	STATION	CO-ORDINATE		T/P EL.
		NORTH	EAST	
1	XX+XXX.XXX	XXXXXXXXXX	XXXXXXXXXX	XXX.XXX
2	XX+XXX.XXX	XXXXXXXXXX	XXXXXXXXXX	XXX.XXX
3	XX+XXX.XXX	XXXXXXXXXX	XXXXXXXXXX	XXX.XXX
4	XX+XXX.XXX	XXXXXXXXXX	XXXXXXXXXX	XXX.XXX

LEGEND

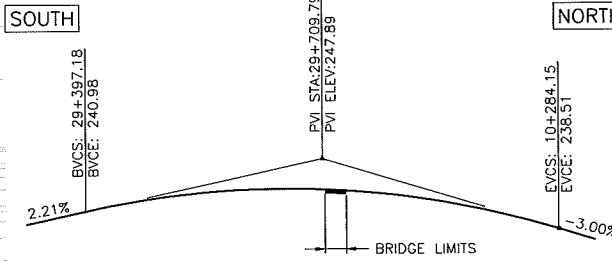
- T/F - TOP OF FOOTING
T/C - TOP OF CONCRETE
T/P - TOP OF PAVEMENT
WP - WORKING POINT
C.L. - CENTRE LINE
S.S. - STAINLESS STEEL
STA. - STATION
O.G. - ORIGINAL GROUND
N.T.S. - NOT TO SCALE
U.N.O. - UNLESS NOTED OTHERWISE
RSS - RETAINED SOIL SYSTEM
----- PROTECTION SYSTEM
- - - - - SOIL RETAINING STRUCTURE/SYSTEM



PROFILE AT TIFFIN STREET
N.T.S.



PROFILE AT SBL HWY 400
N.T.S.



PROFILE AT NBL HWY 400
N.T.S.

DIST. No.
CONT. No.
WP. No.2159-11-00



HWY 400 / TIFFIN STREET OVERPASS
NBL & SBL
STRUCTURE REPLACEMENT
GENERAL ARRANGEMENT - 1

SHEET
S1



GENERAL NOTES:

- DESIGN CODE: CAN/CSA-S6-06 INCLUDING SUPPLEMENTS #1 (S6S1-10), #2 (S6S2-11), AND #3 (S6S3-13).
- CLASS OF CONCRETE:
ALL REINFORCED CONCRETE ----- 30 MPa
CLASS OF CONCRETE FOR PRECAST GIRDERS ARE GIVEN ON PRESTRESSED GIRDER DRAWINGS.
- CLEAR COVER TO REINFORCING STEEL:
FOOTINGS ----- 100 ± 25 mm
DECK----- TOP ----- 70 ± 20 mm
BOTTOM ----- 40 ± 10 mm
REMAINDER ----- 70 ± 20 mm
UNLESS OTHERWISE NOTED.
- REINFORCING STEEL:
• REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
• STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa UNLESS OTHERWISE SPECIFIED.
• BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
• UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
• BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 UNLESS INDICATED OTHERWISE.
- GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE 1 OR GRADE 2 AS SPECIFIED IN THE CONTRACT DRAWINGS. THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS.
- ALL DIMENSIONS OF EXISTING STRUCTURE AS SHOWN ON THIS DRAWING ARE OBTAINED FROM AVAILABLE REFERENCE DRAWINGS AND ARE NOT NECESSARILY THE SAME AS AS-BUILT DIMENSIONS.

RETAINED SOIL SYSTEM (RSS) NOTES:

- RETAINED SOIL SYSTEM WALL SHALL HAVE THE FOLLOWING ATTRIBUTES:
APPLICATION: WALL/SLOPE
GEOMETRY: VERTICAL
PERFORMANCE: HIGH
APPEARANCE: HIGH
- SOIL RETAINING STRUCTURE/SYSTEM WALL SHALL HAVE THE FOLLOWING ATTRIBUTES:
APPLICATION: WALL/SLOPE
GEOMETRY: VERTICAL
PERFORMANCE: HIGH
APPEARANCE: LOW

APPLICABLE STANDARD DRAWINGS

- OPSD- 911.381 GUIDE RAIL SYSTEM, CONCRETE BARRIER PERMANENT TRANSITION INSTALLATION
CONCRETE BARRIER TO STRUCTURE WINGWALL
OPSD-3101.150 WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
OPSD-3120.100 WALLS RETAINING CONCRETE TOE WALL
OPSD-3370.100 DECK WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD-3370.101 DECK WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD-3390.100 DECK DRIP CHANNEL
OPSD-3419.155 BARRIERS AND RAILINGS STEEL DOUBLE RAILING ANCHORAGE
OPSD-3940.151 FIGURES IN CONCRETE WARNING MESSAGE LETTERS

January 9, 2015

PRELIMINARY

REVISIONS		DATE		BY		DESCRIPTION	
DESIGN	MW	CHK.	CL	CODE	CHBDC 2006	LOAD	CL-625-0W1
DRAWN	TLC	CHK.	MW	SITE	30-176		
						DATE	FEB. 2015
						DWG.	1

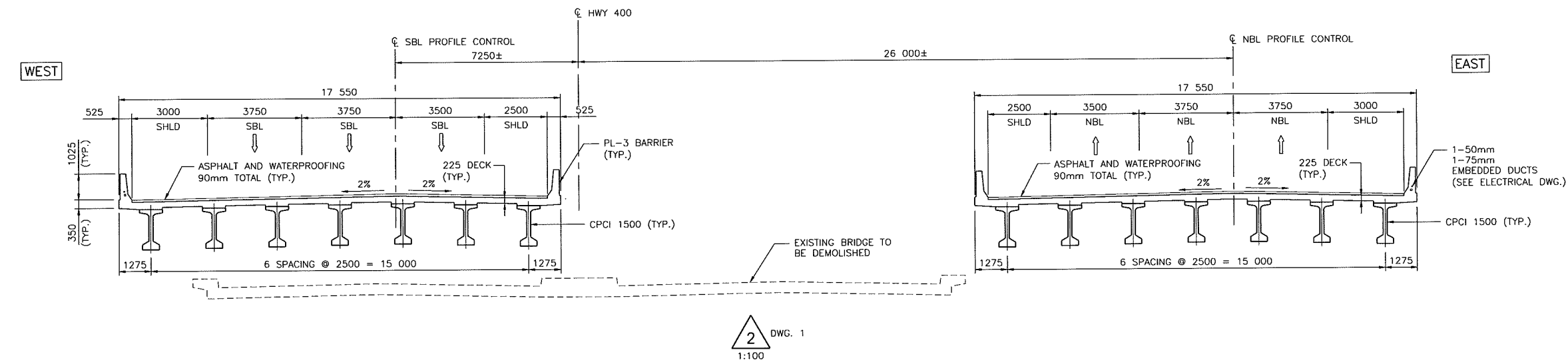
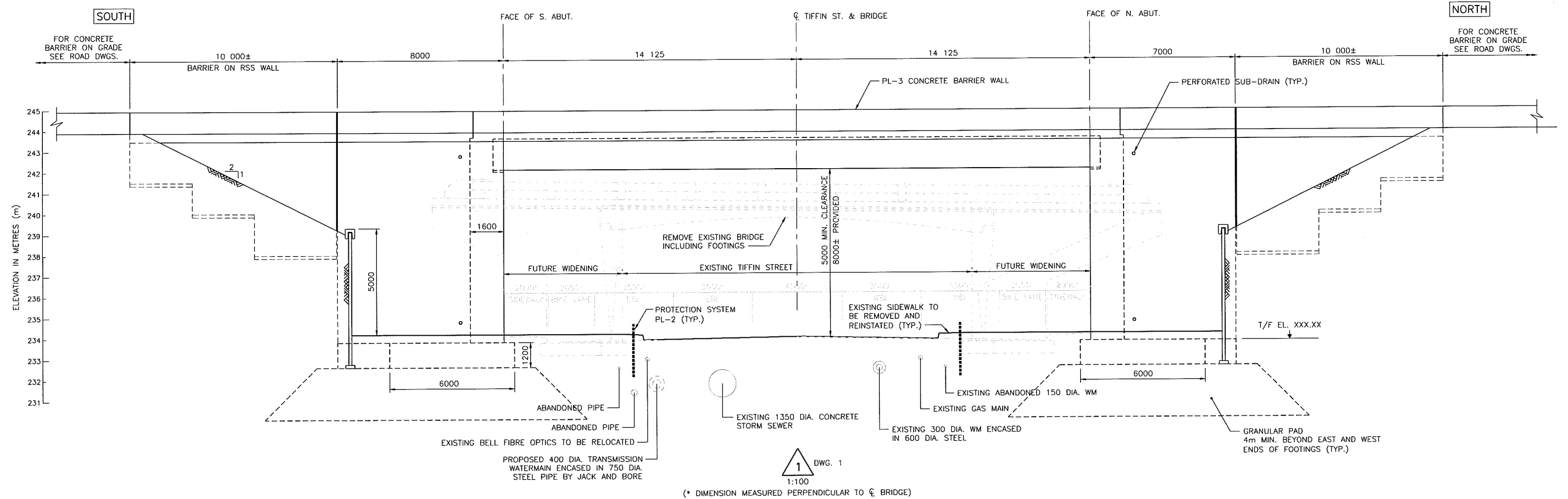
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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

DIST. No.
CONT. No.
WP. No.2159-11-00

HWY 400 / TIFFIN STREET OVERPASS
NBL & SBL
STRUCTURE REPLACEMENT
GENERAL ARRANGEMENT - 2

SHEET
S2



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

January 9, 2015

PRELIMINARY

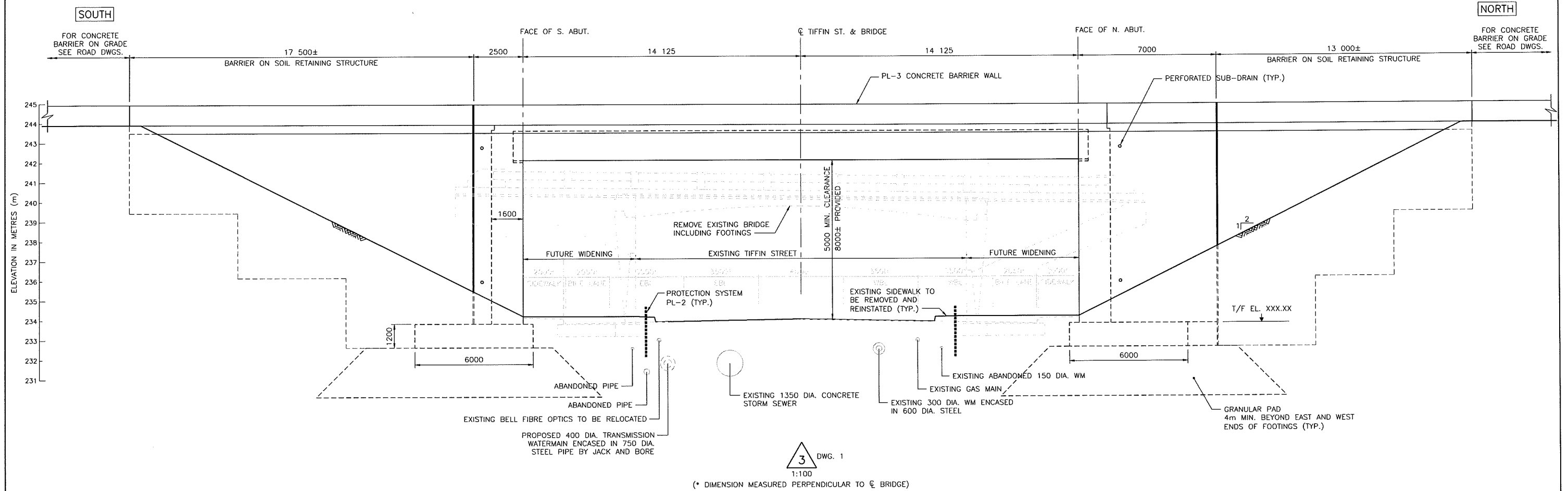
REVISIONS		DATE					BY		DESCRIPTION	
DESIGN	MW	CHK.	CL	CODE	CHBDC	2006	LOAD	CL-625-001	DATE	FEB. 2015
DRAWN	TLC	CHK.	MW	SITE	30-176				DWG.	2

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

DIST. No.
CONT. No.
WP. No.2159-11-00

HWY 400 / TIFFIN STREET OVERPASS
NBL & SBL
STRUCTURE REPLACEMENT
GENERAL ARRANGEMENT - 3

SHEET
S3



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

January 9, 2015

PRELIMINARY

REVISIONS		DESCRIPTION					
DATE	BY	CHK.	CL.	CODE	CHBDC	LOAD	CL-625-ONT
DESIGN	MW	CHK.	CL.	CODE	CHBDC	2006	DATE FEB. 2015
DRAWN	TLC	CHK.	MW	SITE	30-176		DWG. 3

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

DIST. No.
CONT. No.
WP. No.2159-11-00

HWY 400 / TIFFIN STREET OVERPASS
NBL & SBL
STRUCTURE REPLACEMENT
CONSTRUCTION STAGING

SHEET
S4



STAGE 1

1. INSTALL PROTECTION SYSTEM ON THE EAST SIDE OF EXISTING STRUCTURE.
2. EXCAVATE AND CONSTRUCT FOOTINGS OF NEW NB STRUCTURE.
3. CONSTRUCT NEW NB OVERPASS.
4. INSTALL SOIL RETAINING SYSTEM AND ADD FILL BEHIND STRUCTURE.

STAGE 2

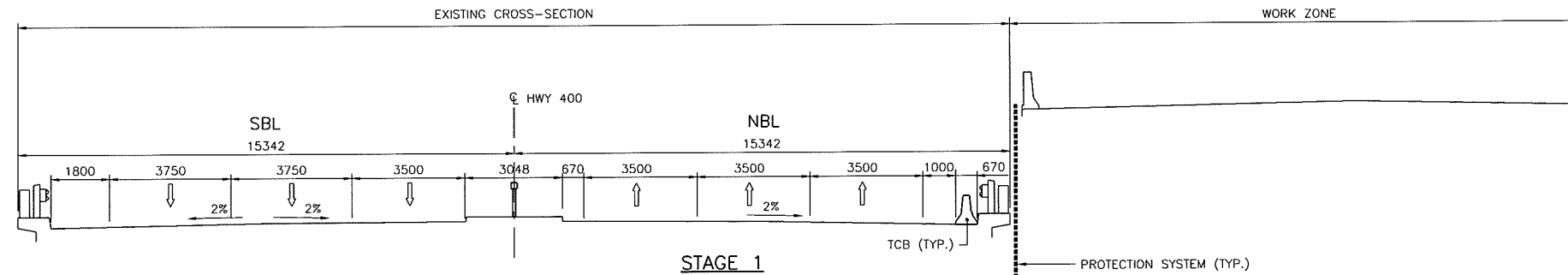
1. INSTALL TRAFFIC CONTROL MEASURES FOR NB AND SB TRAFFIC.
2. SHIFT NB TRAFFIC TO NEW NB STRUCTURE.
3. SHIFT SB TRAFFIC TO EAST PORTION OF EXISTING STRUCTURE.
4. REMOVE WEST PORTION OF EXISTING SUPERSTRUCTURE.
5. INSTALL PROTECTION SYSTEM ALONG CENTERLINE OF EXISTING HWY 400.
5. EXCAVATE AND CONSTRUCT FOOTINGS OF NEW SB STRUCTURE.
6. CONSTRUCT NEW SB OVERPASS.

STAGE 3

1. SHIFT SB TRAFFIC TO NEW SB STRUCTURE.
2. REMOVE EAST PORTION OF EXISTING STRUCTURE.
3. SLOPE SOIL ALONG MEDIAN DOWN TO TIFFIN STREET.
4. REMOVE TRAFFIC CONTROL MEASURES.

WEST

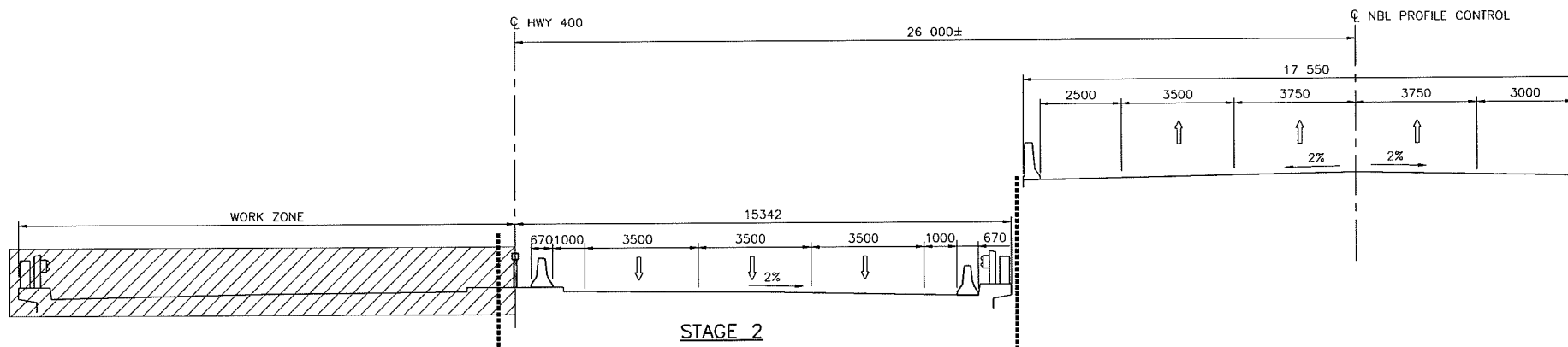
EAST



STAGE 1

WEST

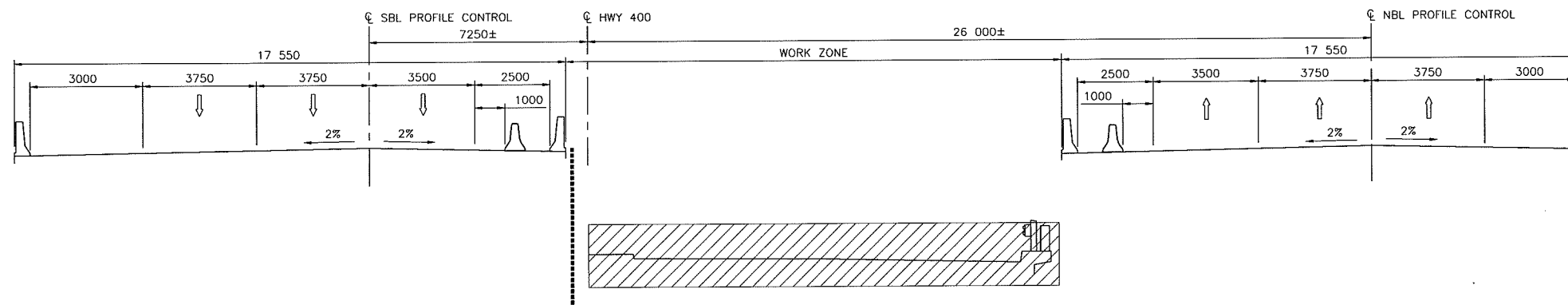
EAST



STAGE 2

WEST

EAST



STAGE 3

WEST

EAST



FINAL

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

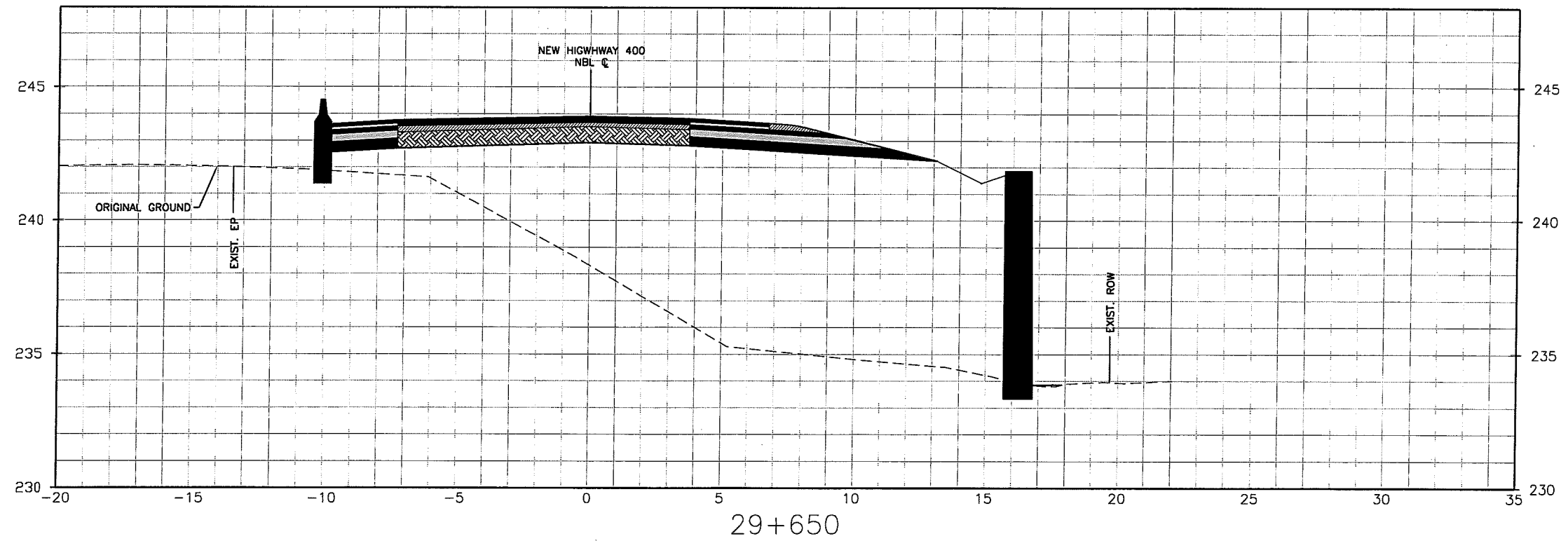
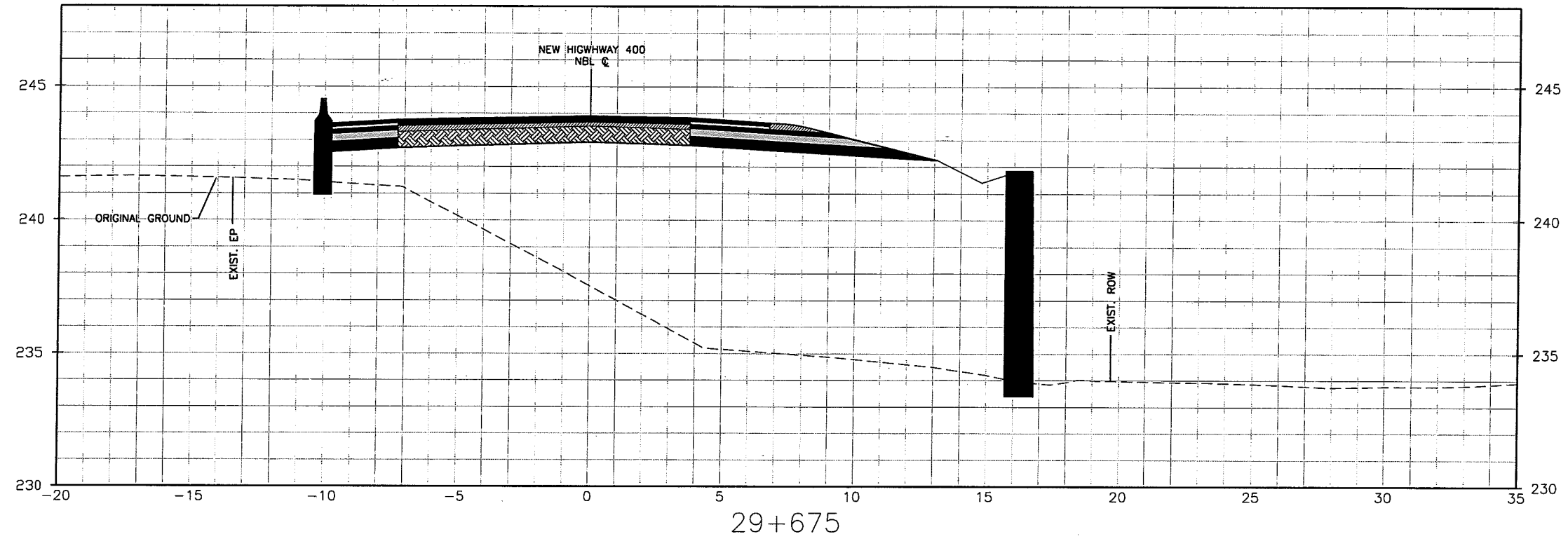
January 9, 2015

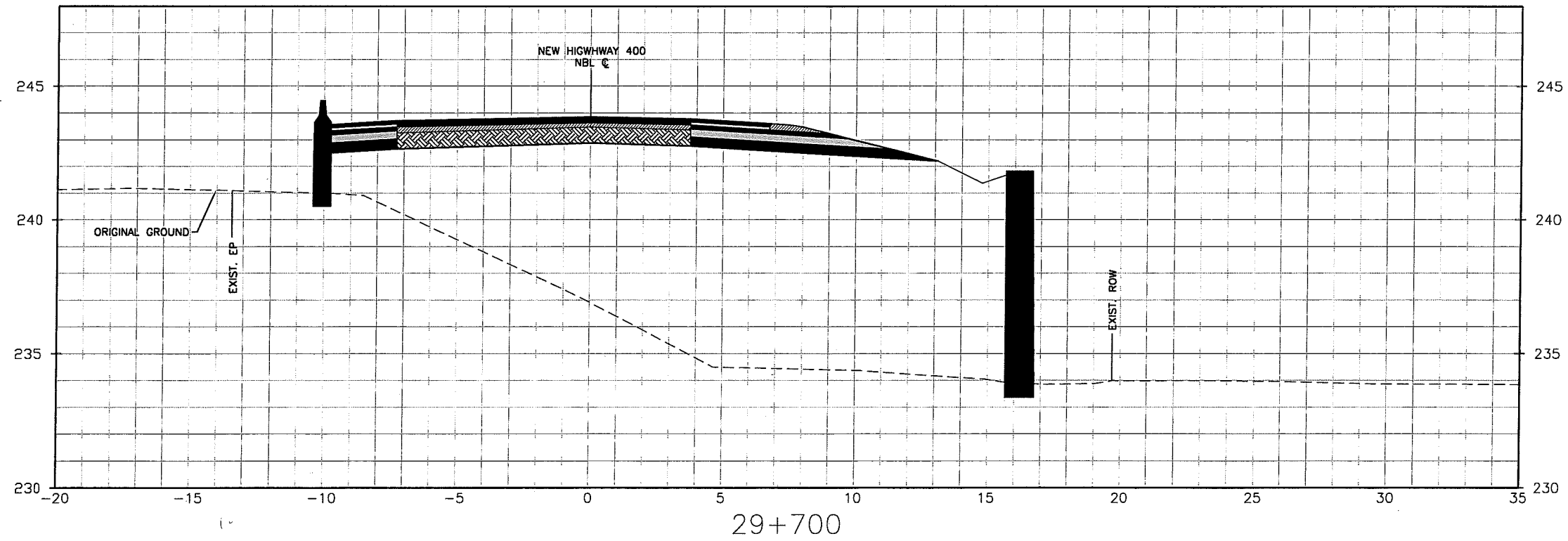
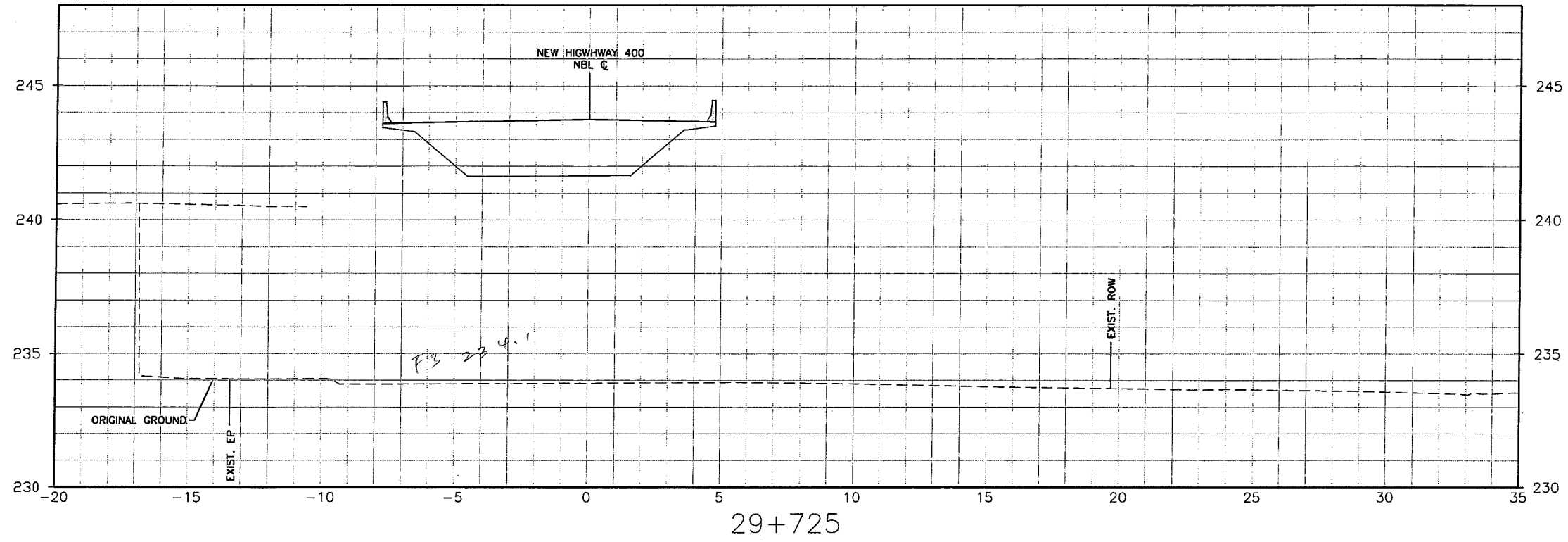
PRELIMINARY

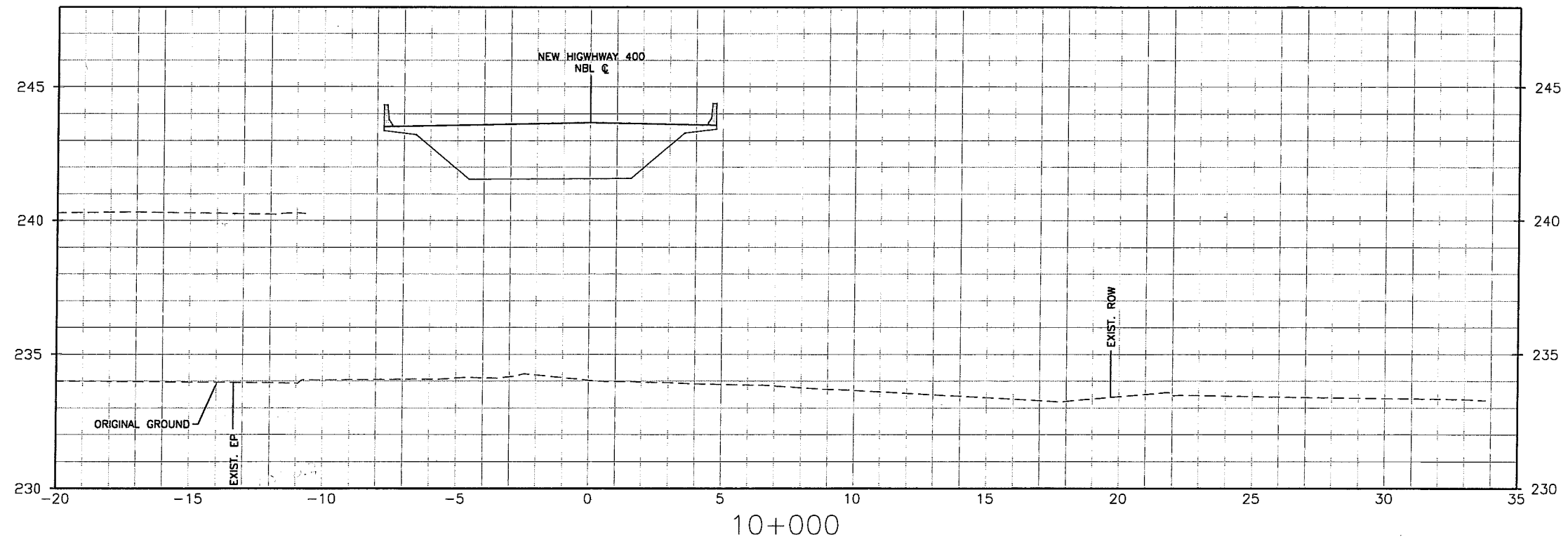
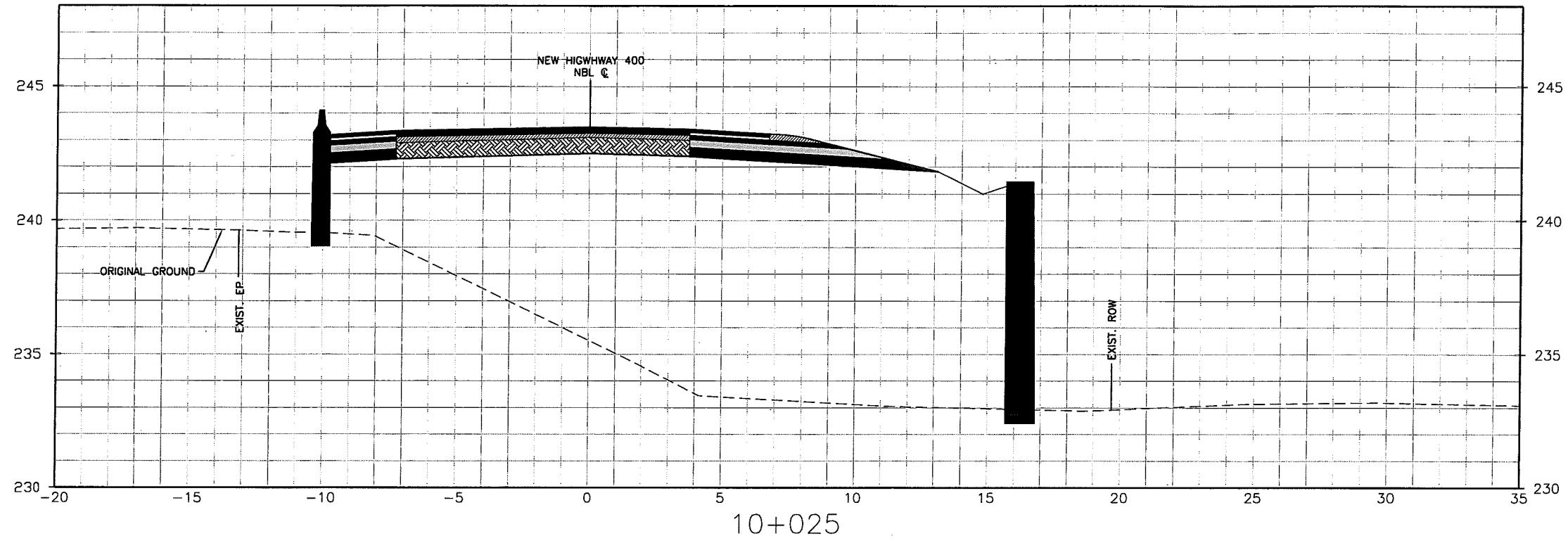
REVISIONS		DATE		BY		DESCRIPTION		DATE	
DESIGN	MW	CHK.	CL	CODE	CHBDC 2006	LOAD	CL-625-ONT	DATE	FEB. 2015
DRAWN	TLC	CHK.	MW	SITE	30-176			DWG.	4

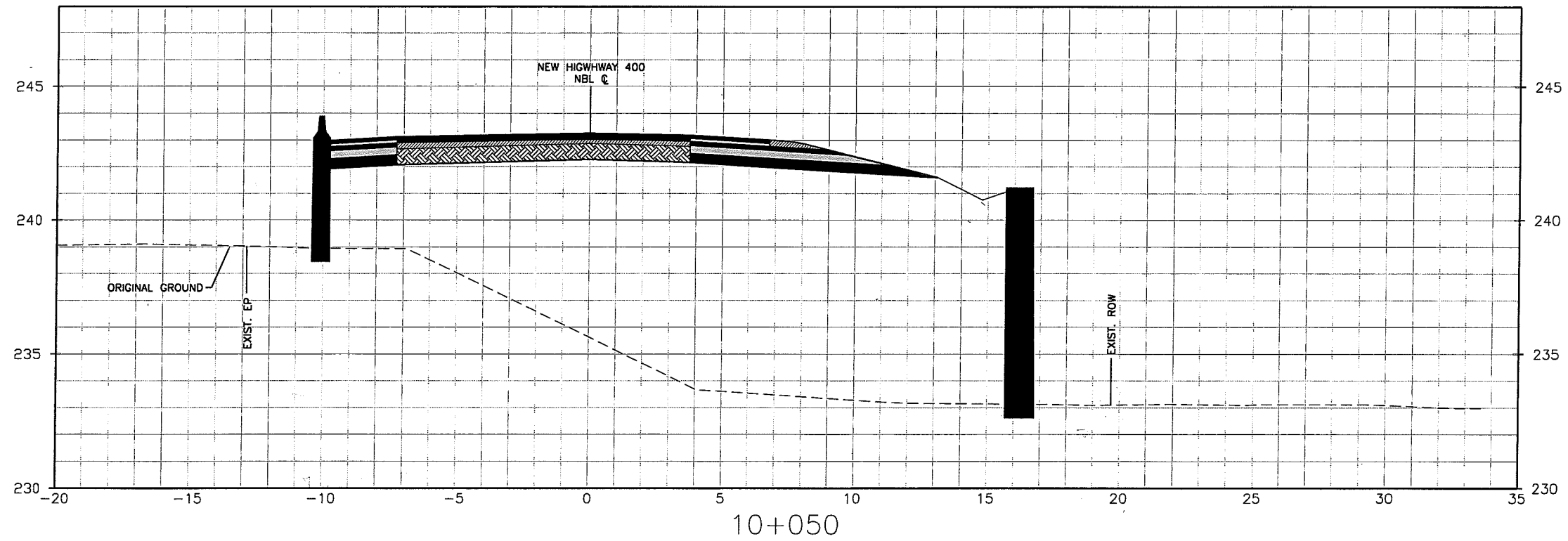
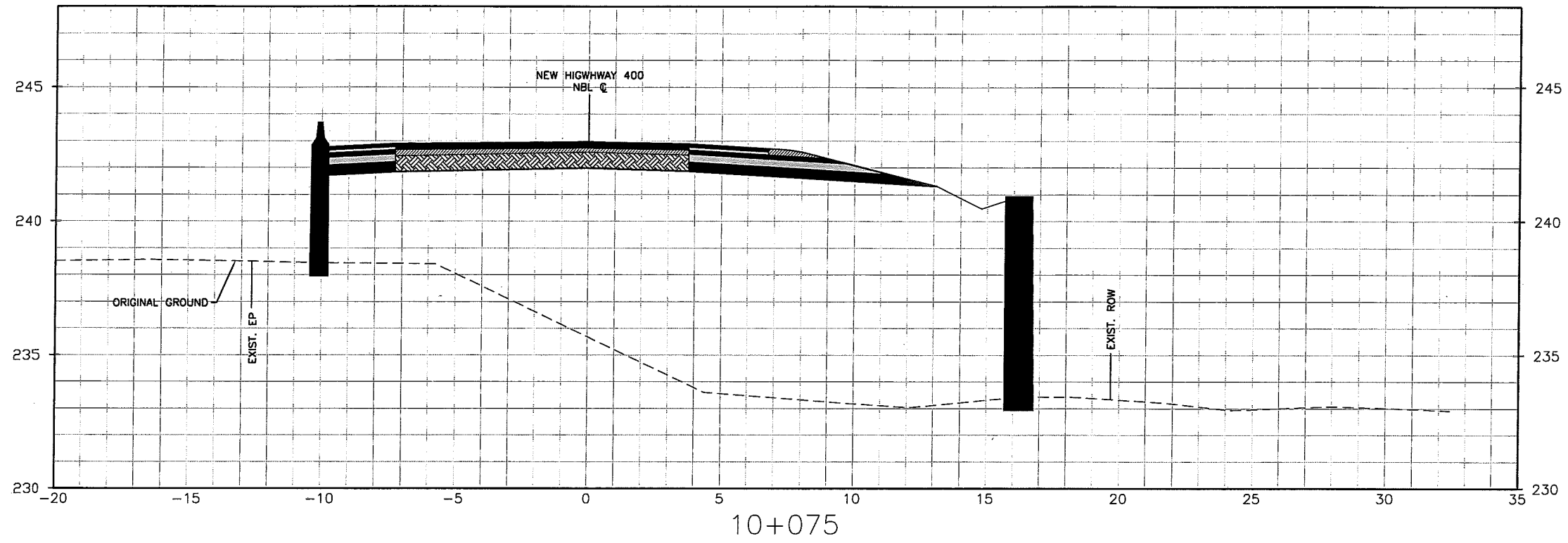
Appendix E

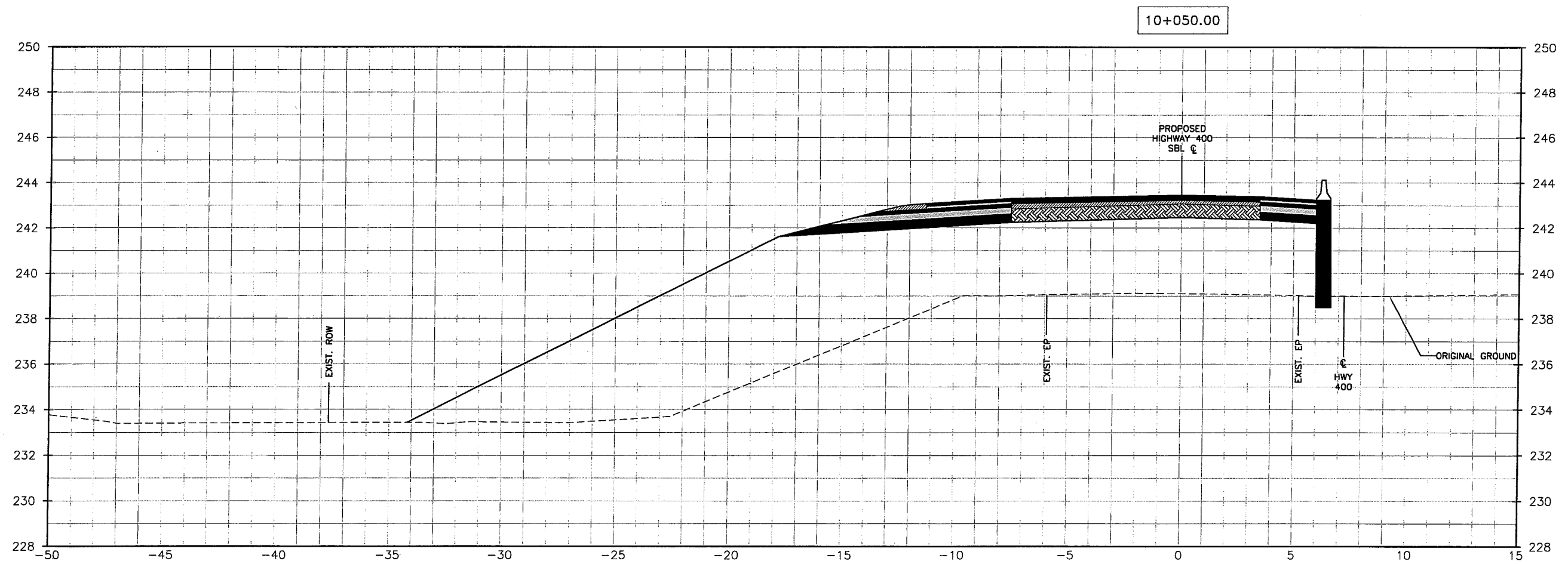
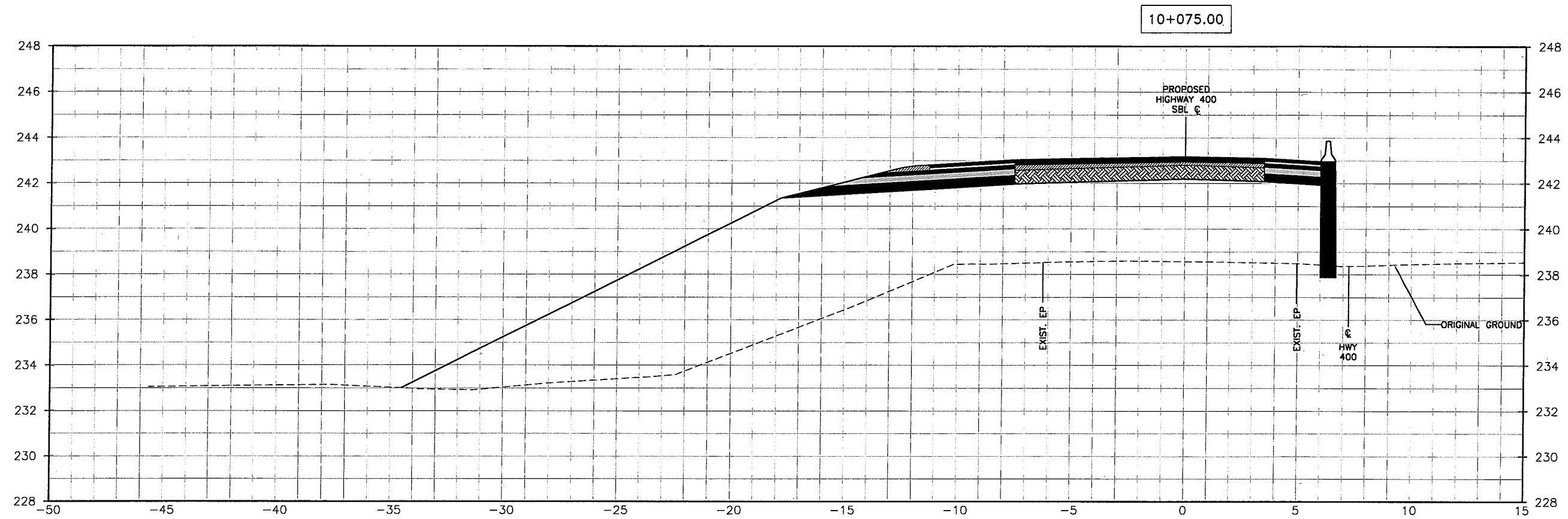
Cross-sectional Drawings







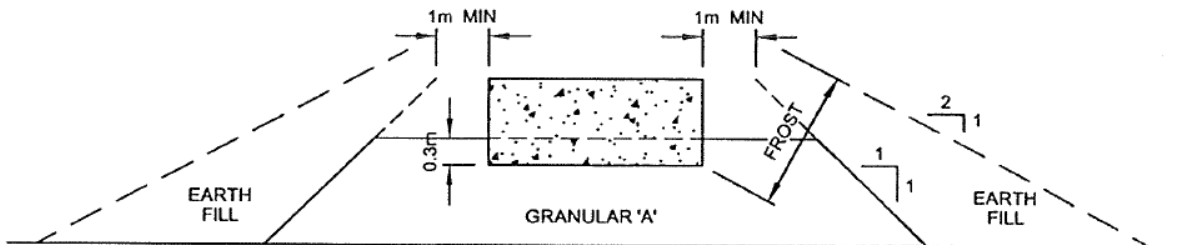




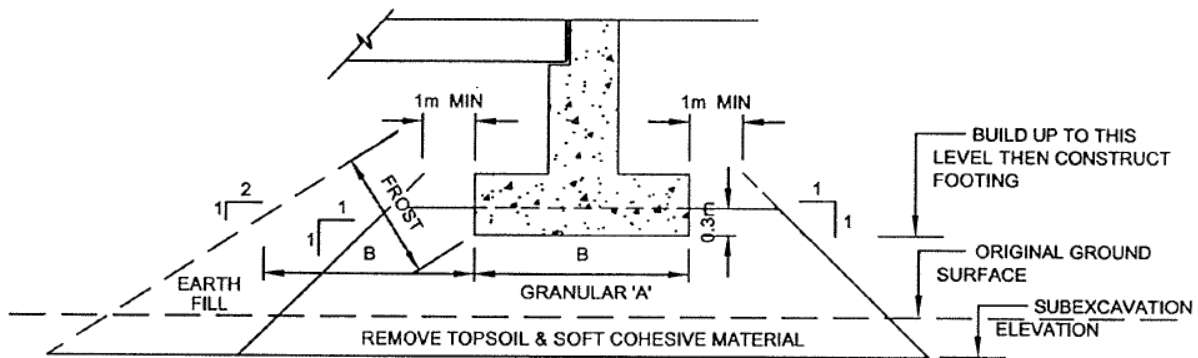
Appendix F

MTO Granular A Pad

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

Notes

1. Remove topsoil &/or soft subsoil under area of compacted Granular 'A' & earth fill.
2. Place Granular 'A' & earth fill to top of footing level, compacted according to current MTO standards.
3. Excavate compacted Granular 'A' pad & earth fill for footing

Appendix G

List of Standard Specifications-OPSDs and OPSSs

OPSSs

OPSS.PROV206 Construction Specification of Grading

OPSS 501 Construction Specification for Compacting.

OPSS 539 - Construction Specification for Temporary Protection Systems

OPSS 571 Construction Specification for Sodding

OPSS 572 Construction Specification for Seed and Cover

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

OPSS 915 - Construction Specification for Sign Support Structures

OPSS 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSD3101.150 Walls, Abutment, Backfill Minimum Granular Requirement

OPSD

OPSD202.010 Slope Flattening using Surplus Excavated Material on Earth and Rock Embankment.

OPSD208.010 Benching of Earth Slopes

Appendix H

NSSP

Vibration Monitoring

Special Provision

The vibration monitoring equipment shall be placed on the existing structure and newly constructed structure such that it will not be disturbed. The location should be as close as possible to the demolition and construction work.

The vibrations at the existing structure shall not exceed 100 mm/s (peak particle velocity).

The monitoring results shall be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specification. The results shall be submitted to the Contract Administrator.

Appendix I

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey at the time of preparation. Unless otherwise agreed in writing by Coffey it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time. Any user of this report specifically denies any right to claims against the Consultant, Sub-Consultants, their officers, agents and employees in excess of the fee paid for professional services.