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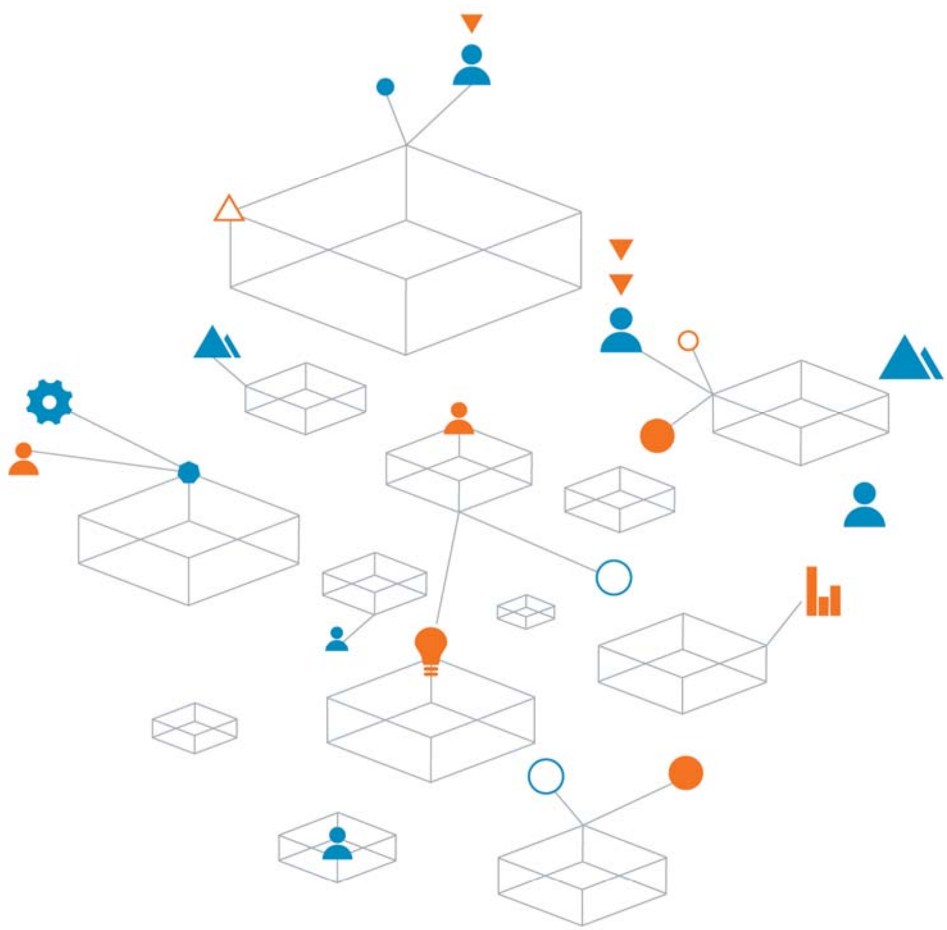
MORRISON HERSHFIELD

Preliminary Foundation Investigation and Design Report

Highway 400/Barrie-Collingwood Railway Overpass Structure
Rehabilitation and New NB Structure Construction, G.W.P. 2074-11-00
Site No's. 30-177/1 & 2, Design-Build Ready Package
GEOTETOB22161AA - DRAFT
15 December 2014



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15 December 2014

Attention: Bruce Dickey, P. Eng., AVS

RE: Draft – Preliminary Foundation Investigation and Design Reports, Highway 400/ Barrie-Collingwood Railway Overpass Structure Rehabilitation and New NB Structure, G.W.P. 2074-11-00, Site No's. 30-177/1&2, Design-Build Ready Package

Coffey is pleased to present our Draft Preliminary Foundation Investigation and Design Reports (for a Design-Build Ready Package) for the above-referenced subject.

Should you have any questions or require clarification on any aspect of these reports, please contact the undersigned at (416) 213-5357.

For and on behalf of Coffey

Draft

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

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MORRISON HERSHFIELD

Preliminary Foundation Investigation Report

Highway 400/Barrie-Collingwood Railway Overpass Structure Rehabilitation and New NB Structure Construction, G.W.P. 2074-11-00

Site No's. 30-177/1 & 2, Design-Build Ready Package

GEOTETOB22161AA - DRAFT

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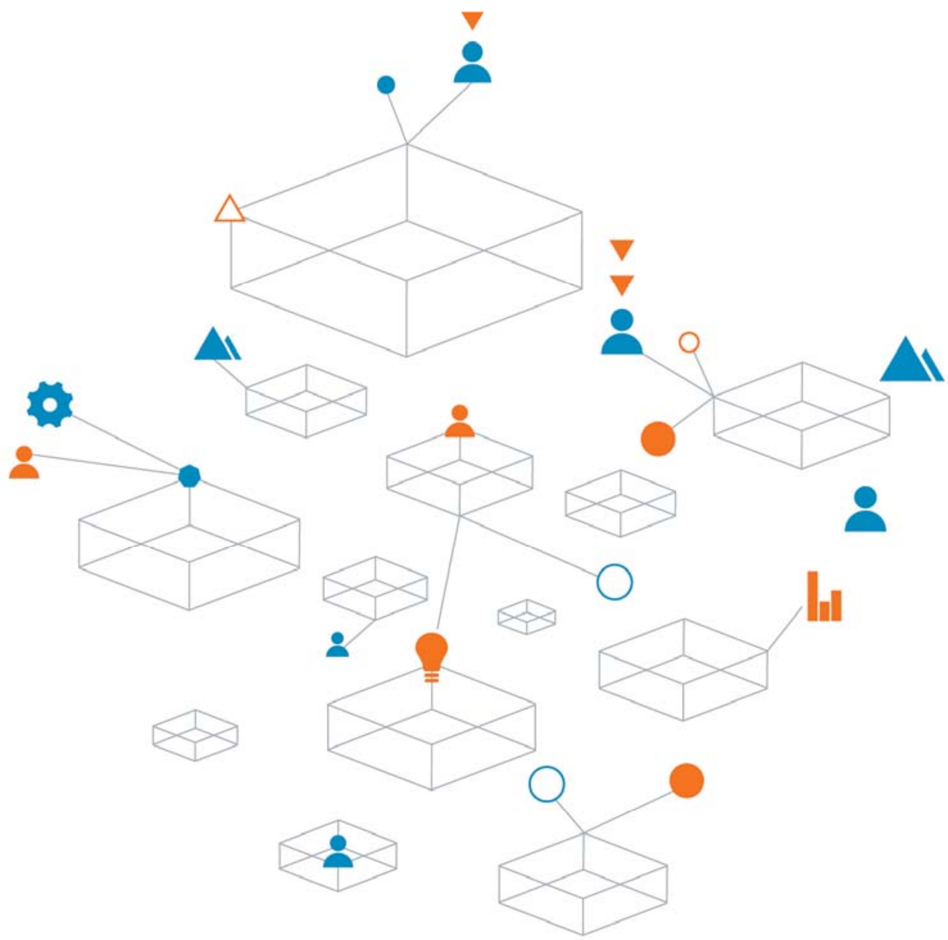

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REHABILITATION AND NEW NB STRUCTURE CONSTRUCTION
G.W.P. 2074-11-00, SITE NO’S 30-177/1&2,
DESIGN-BUILD READY PACKAGE

1 Introduction

Coffey was retained by Morrison Hershfield (herein “MH”) on behalf of the Ministry of Transportation Ontario (herein “MTO”) to provide preliminary foundation investigation and engineering services for a proposed Design-Build ready package (DB) 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 new North Bound (NB) Barrie-Collingwood Railway (BCR) overpass structure and rehabilitation of the existing BCR overpass.

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 known TCE (trichloroethylene) contamination in the project area 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 overpass is a single span rigid frame structure with a span of 10 m and a length of 29 m. It was built in the 1950’s.

The areas on the east and west side of Highway 400 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 the existing trichloroethylene (TCE) contamination within the project limit, borehole depths/elevations were determined by MH environmental specialists to minimize possible environmental issues.

Three (3) boreholes were advanced adjacent to the existing BCR structure (2 BHs for foundation and 1 BH 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 3 to 21, 2014 under observation of Coffey technical personnel. Boreholes F6 and RW10 were drilled from the existing Highway 400 grade during nightly lane closures as directed by MTO COMPASS. Borehole F6 was drilled at railway grade under the guidance of a BCR representative. All field work was performed in a safe manner, with no inconvenience being caused to the traveling public. No property damage occurred. All drilled locations were restored to their former condition.

The first borehole (F6) was drilled from the existing railway crossing grade in the presence of an MH environmental specialist. The subsurface conditions encountered in Borehole F6 established the lowest elevation in which the remaining boreholes in the vicinity of BCR could be drilled and sampled. Borehole F5 was also drilled in the presence of an MH environmental specialist. **Table 3.1** below 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
BCR Existing	F5	29+538, 12 m Rt	243.0	18.9	224.1	-
BCR New NB	F6	29+554, 30 m Rt	234.6	6.1	228.5	Monitoring well
Temporary Retaining Wall	RW10	29+574, 3 m Lt	242.9	15.1	227.8	-

One (1) monitoring well was installed for groundwater sampling and long term groundwater monitoring. Boreholes F5 and RW10 were backfilled and sealed in accordance with MOE Reg. 903. A vapour monitoring well was installed, just above the water table, by MH personnel. Details and observational data from that well can be obtained from MH.

The three (3) 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 (SPT, 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. Some duplicate samples were bagged for head space vapour readings by MH personnel.

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 to sandy silt and silt to sand 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 stratigraphy are presented on **Drawing 1**.

4.1 Pavement Structure

The asphaltic concrete pavement thickness was 225 mm (BH F5) and 260 mm (BH RW10) above 0.6 m thickness of dense (N = 35-38 blows/0.3 m) sand and gravel base and sub-base course.

4.2 Embankment Fill

Under the pavement structure in Boreholes F5 and RW10 the highway embankment fill consists of silty sand, trace to some gravel, down to about elev. 235 m to 233 m.

The gradation of a sample from the embankment fill is included in **Figure B-1.** It shows the following grain size distribution: 3% gravel, 82% sand and 15% silt and clay sized particles.

The natural moisture content of the recovered samples from the embankment fill was 3-18% (average 10%).

Standard Penetration Test N values ranged from 1 to 43 blows/0.3 m (average 17 blows/0.3 m), indicating a very loose to dense condition, the relative density being mostly compact. The variability in N values suggests that 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.

4.3 Sandy Silt to Silty Sand and Silt to Sand and Silt

The native soil beneath and adjacent to the Highway 400 embankment typically consists of sandy silt to silty sand and silt to sand and silt. This stratified deposit contains trace gravel and clay. All boreholes were terminated within this deposit at depths ranging from 6.1 m to 18.9 m below the existing grade (elev. 228.5 to 224.1 m).

The grain size distribution of four (4) samples from the deposit is given in **Figure B-2**. The following grain size distribution ranges were obtained:

Gravel:	0 - 1%
Sand:	2 – 81%
Silt and Clay:	19 - 98% (4 - 8% clay size particles)

The natural moisture content of this deposit was 8-26% (average 19%).SPT N values of 6 to 50 blows/0.3 m indicate a loose to dense condition (generally compact).

4.4 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. A monitoring well was installed in Borehole F6 for long term groundwater monitoring. The groundwater levels observed during and after the investigation are summarized in **Table 4.4.1** and are also presented on the Record of Borehole Sheets in **Appendix A**.

Table 4.4.1: Groundwater Observations

Borehole No.	Ground Elevation (m)	Date	Depth to Water Level (m)	Groundwater Elevation (m)
F5	243.0	Upon Completion	13.7	229.3
F6	234.6	October 31, 2014 (about 4 weeks after well installation)	4.1	230.5
RW10	242.9	Upon Completion	11.6	231.3 (wet spoon at 230.6 m)

Groundwater levels measured on completion are considered not stabilized and therefore do not represent the established long term average groundwater table (phreatic surface).

The observations in **Table 4.4.1** indicate the groundwater table at the site lies typically between elev. 231 m and 230 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

Draft

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

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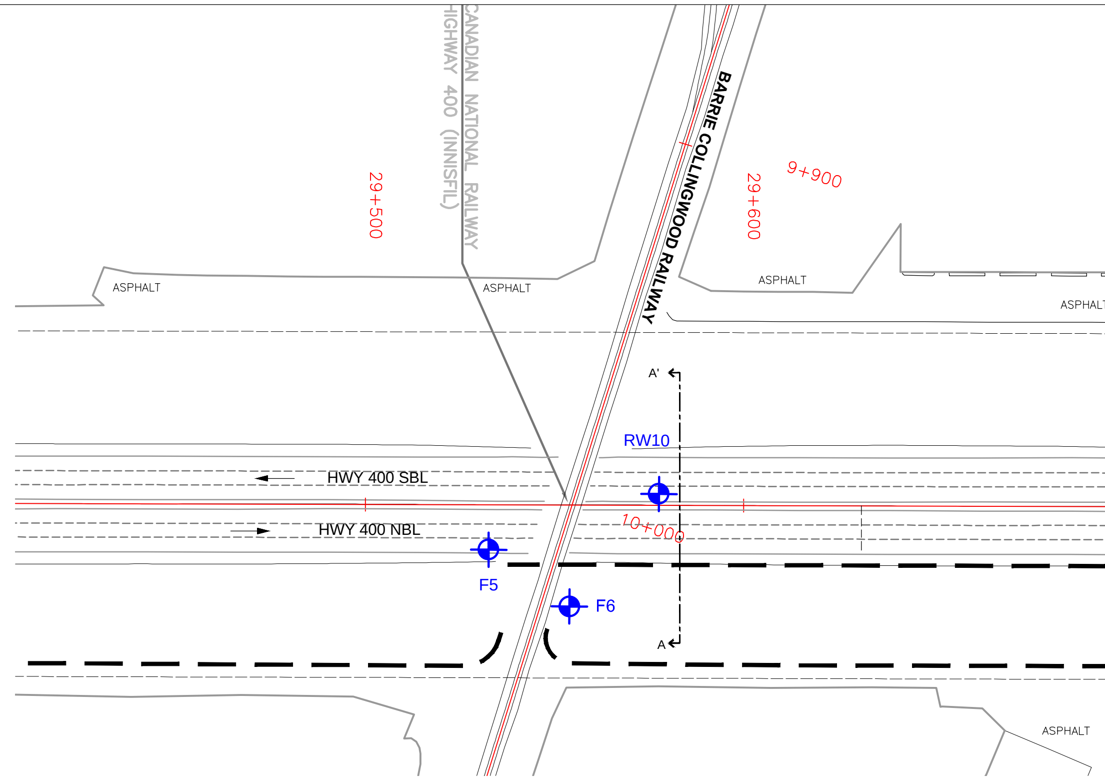
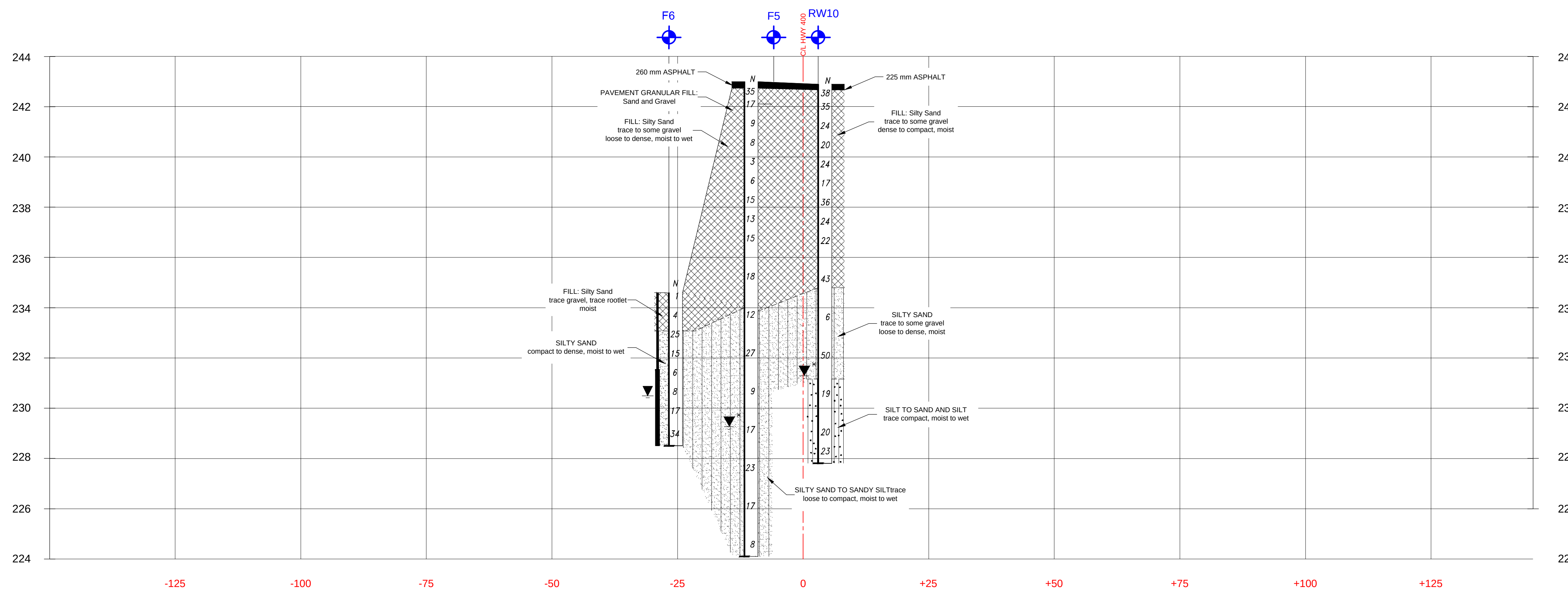
Cam Mirza, P.Eng.
MTO Designated Contact, Principal

Drawing



PLAN

SCALE 0 50 100 m



SECTION A-A'

HORIZONTAL SCALE 0 25 50m

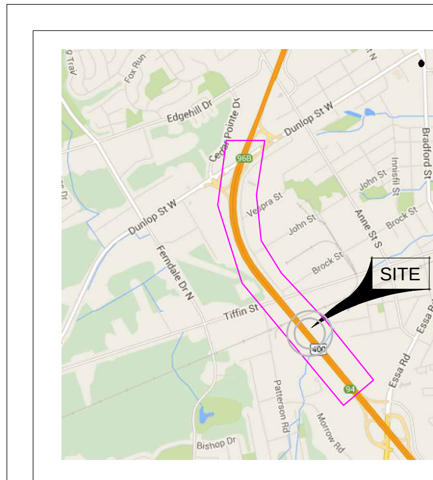
DISTRICT
CONT. No.
WP No. -

Highway 400/Barrie-Collingwood Railway Overpass Structure

Borehole Location Plan and Soil Strata

SHEET

METRIC



KEY PLAN
N.T.S.

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	STATION	OFFSET
F5	243.0	288402.0	4914403.1
F6	234.6	288399.8	4914429.1
RW10	242.9	288361.8	4914428.2

-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.

GEOCRES No. -	PROJECT No. GEOTETOB22161AA

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 $\overline{N}$.

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 COMPOSION AND STRUCUTRAL 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
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	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_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	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 = $(W_L - W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

GEOTETOB22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F5

METRIC

GWP	2074-11-00	LOCATION	29+533, 11.9 m Rt C/L (N 4914403.1, E288402)	ORIGINATED BY	LG
DIST	HWY 400	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	MP
DATUM	Geodetic	DATE	21/10/2014	CHECKED BY	SH

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
243.0	GROUND SURFACE					
242.7 / 0.0	260 mm **ASPHALT**					
0.3	**PAVEMENT GRANULAR FILL:** 0.2 m thick Sand and Gravel, 0.4 m thick Sand, some gravel	1	SS	35		
242.1 / 0.9	**FILL:** Silty Sand, trace to some gravel, brown to grey, loose to dense, moist to wet	2	SS	17		
		3	SS	9		
	loose	4	SS	8		
		5	SS	3		
		6	SS	6		
	silty clay lenses	7	SS	15		
		8	SS	13		
		9	SS	15		
		10	SS	18		
233.9 / 9.1	**SILTY SAND TO SANDY SILT** brown to grey, loost to compact, moist to wet; sand, some silt	11	SS	12		0 81 15 4
		12	SS	27		
	loose	13	SS	9		
		14	SS	17	▼	wet spoon
228.0						

Elevation scale: 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15-○-5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

GEOTETOB22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH F5

METRIC

GWP 2074-11-00 | LOCATION 29+533, 11.9 m Rt C/L (N 4914403.1, E288402) | ORIGINATED BY LG

DIST | HWY 400 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY MP

DATUM Geodetic | DATE 21/10/2014 | CHECKED BY SH

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
228.0 15.0	**SILTY SAND TO SANDY SILT** brown to grey, loost to compact moist to wet						228		added bentonite (quick gel) for further drilling
			15	SS	23			o (~24)	
							227		
			16	SS	17		226	o (~23)	
	loose						225		
224.1			17	SS	8			o (~21)	
18.9	End of Borehole Cave-in @ 13.7 m								

Column headings: SOIL PROFILE; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m3; GR SA SI CL

+3, ×3: Numbers refer to Sensitivity

20 / 15 -o- 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH F6

METRIC

GWP 2074-11-00 LOCATION 29+554, 26.8 m Rt C/L (N 4914429.1, E288399.8) ORIGINATED BY LG

DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP

DATUM Geodetic DATE 03/10/2014 CHECKED BY SH

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.6	GROUND SURFACE					
0.0	**FILL:** Silty Sand, trace gravel, trace rootlet, brown, moist	1	SS	1	234	
		2	SS	4		
233.1 / 1.5	**SILTY SAND** brown, compact to dense, moist to wet	3	SS	25	233	0 53 (47)
		4	SS	15	232	
	silt, loose, trace clay layer	5	SS	6	231	wet spoon
		6	SS	8		0 2 90 8
		7	SS	17	230	
		8	SS	34	229	
228.5 / 6.1	End of Borehole					

End of Borehole
Water level @ 3.9 m (not stabilized)* upon completion.
Piezometer installed to 6.1 m.
Piezometer water level records :
Oct. 31, 2014 4.1 m (El. 230.5 m)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH RW10

METRIC

GWP 2074-11-00	LOCATION 29+578, 3.0 m Lt C/L (N 4914428.2, E288361.8)		ORIGINATED BY JD
DIST	HWY 400	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY MP
DATUM Geodetic	DATE 14/10/2014 15/10/2014		CHECKED BY SH

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
242.9	GROUND SURFACE						
242.0	225 mm **ASPHALT**						
0.2	0.4 m gravelly sand to sand some gravel	1	SS	38			
		2	SS	35		242	
	FILL: Silty Sand trace to some gravel brown, dense to compact, moist	3	SS	24		241	
		4	SS	20		240	
		5	SS	24			3 82 (15)
		6	SS	17		239	
		7	SS	36		238	
		8	SS	24		237	
		9	SS	22		236	
		10	SS	43		235	
234.8 8.1	**SILTY SAND** trace to some gravel brown, loose to dense, moist					234	
		11	SS	6		233	
		12	SS	50		232	
231.2 11.7	**SILT TO SAND AND SILT** brown to grey, compact, moist to wet				▼	231	
		13	SS	19		230	1 16 75 8 wet spoon
		14	SS	20		229	
		15	SS	23		228	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Continued Next Page

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

GEOTETOB22161AA: Hwy 400/ Tiffin Street

RECORD OF BOREHOLE No BH RW10

METRIC

GWP 2074-11-00 LOCATION 29+578, 3.0 m Lt C/L (N 4914428.2, E288361.8) ORIGINATED BY JD

DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MP

DATUM Geodetic DATE 14/10/2014 15/10/2014 CHECKED BY SH

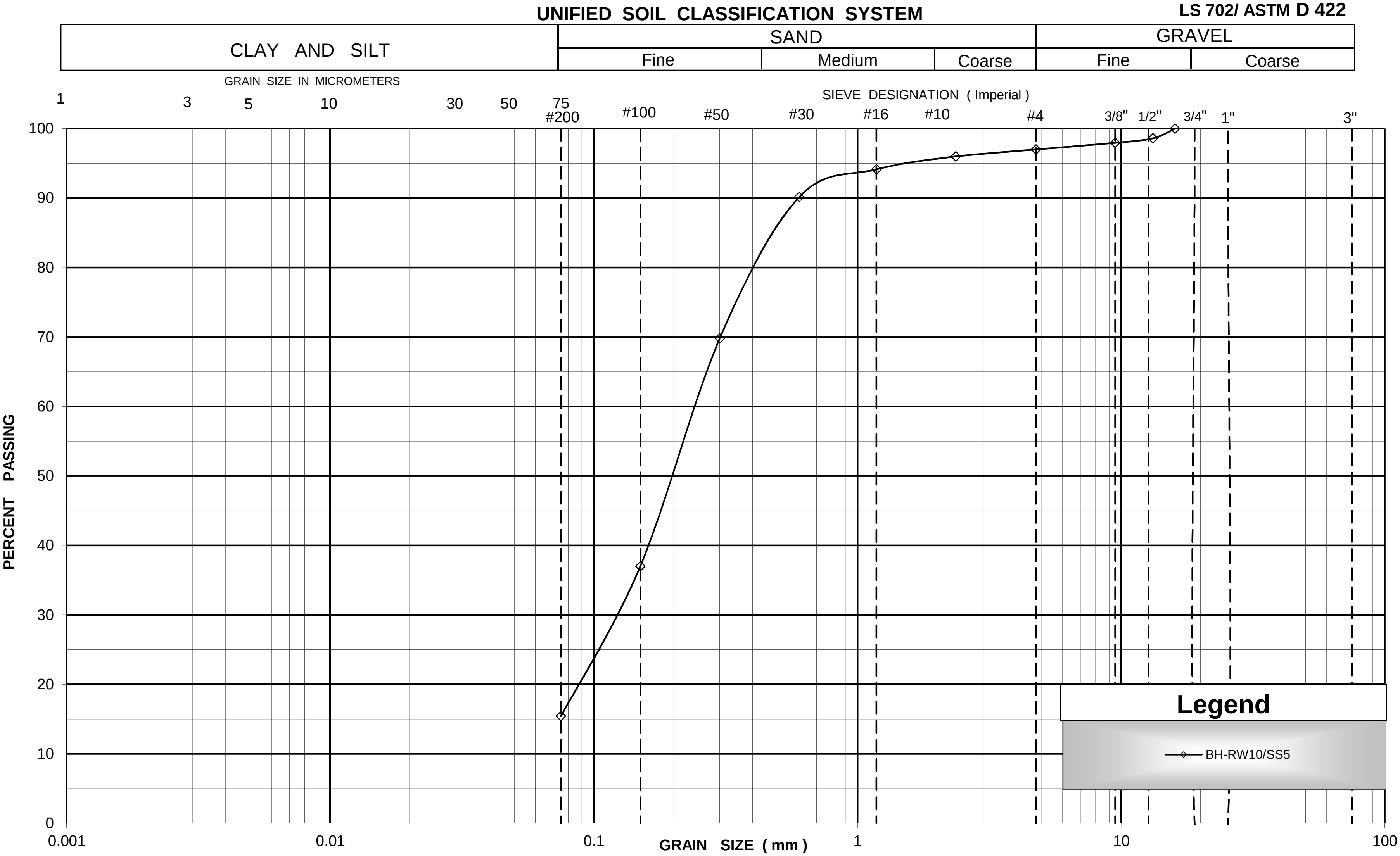
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
227.9											
227.8 15.1	End of Borehole Water level @ 11.6 m (not stabilized)* upon completion.										

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

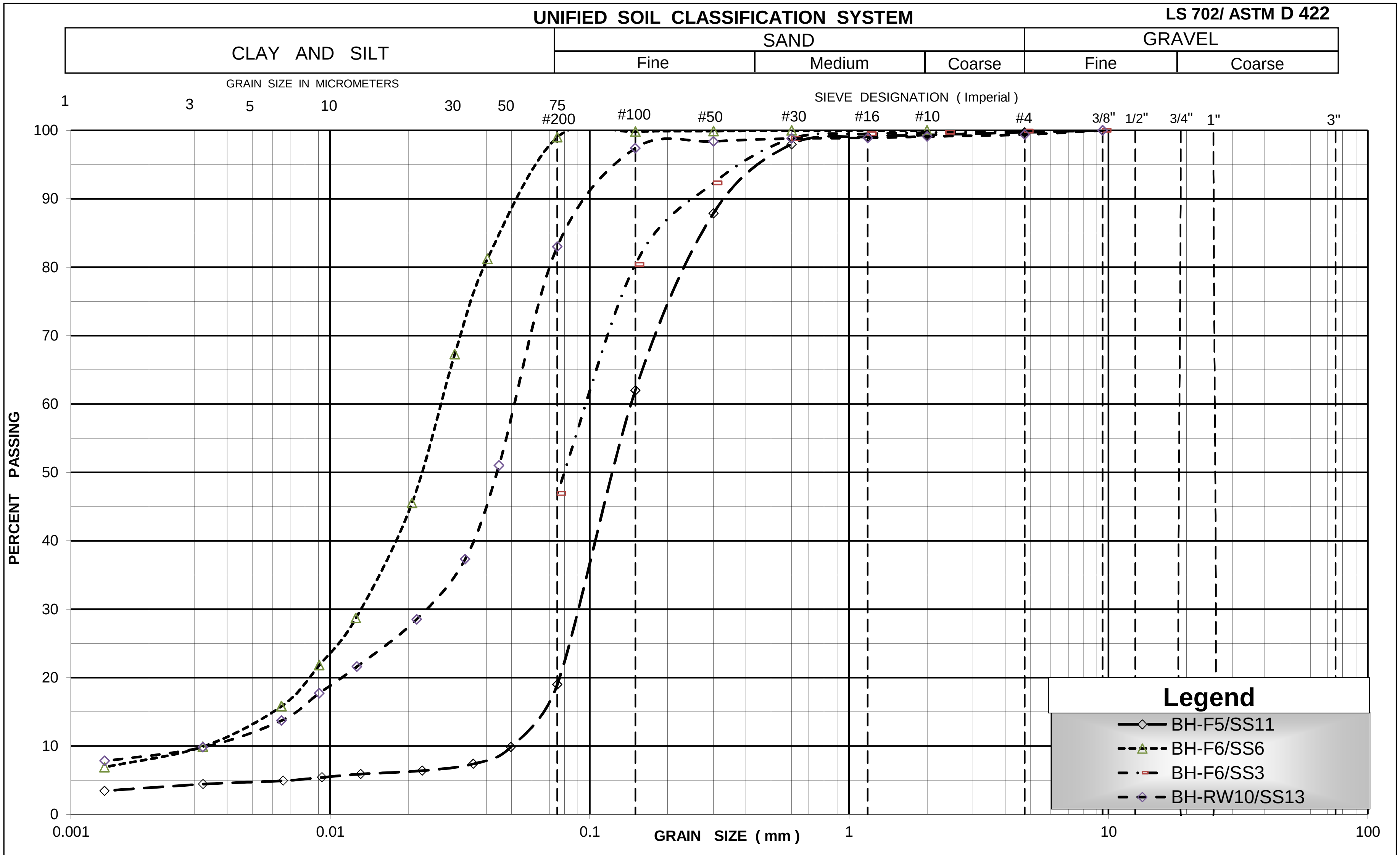


coffey

GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Silty Sand

FIGURE NO.	: B-1
PROJECT NO.:	GEOTETOB22161AA
DATE	: NOV 12, 2014



GRAIN SIZE DISTRIBUTION

Sandy Silt to Silty Sand and Silt to Sand & Silt

FIGURE NO. : B-2

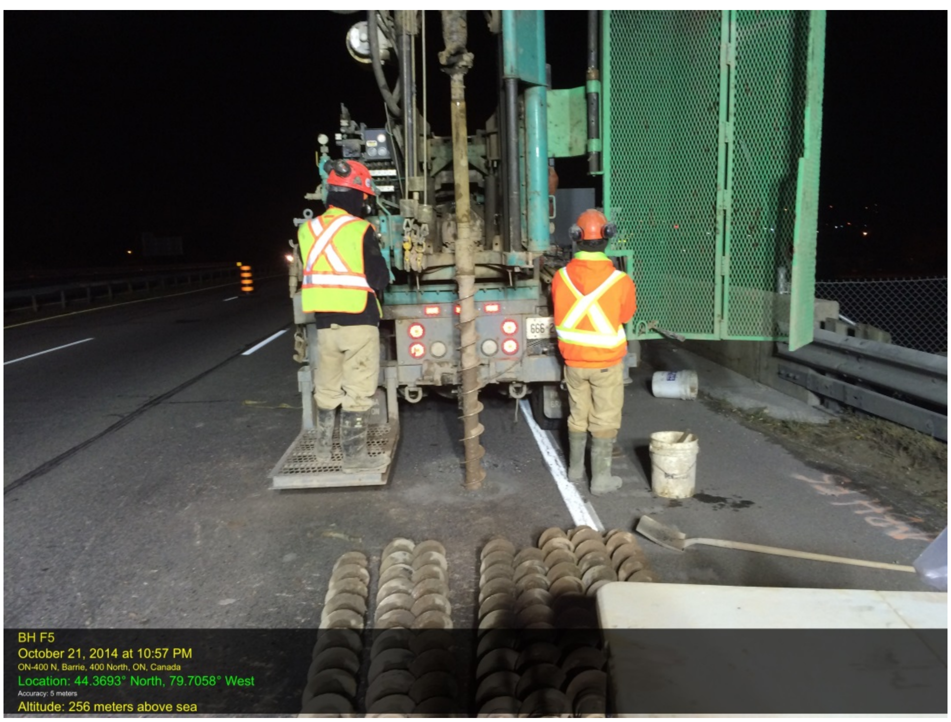
PROJECT NO.: GEOTETOB22161AA

DATE : NOV 19, 2014

Appendix C

Site Photographs

Photograph 1: Borehole F6 @ Station 29+554, Looking South-West



Photograph 2: Borehole F5 @ Station 29+533, Looking North

Photograph 3: Borehole RW10 @ Station 29+578, Looking South

coffey

MORRISON HERSHFIELD

Preliminary Foundation Design Report

Highway 400/Barrie-Collingwood Railway Overpass Structure Rehabilitation and New NB Structure - G.W.P. 2074-11-00

Site No's. 30-177/1 & 2, Design-Build Ready Package

GEOTETOB22161AA - DRAFT

15 December 2014

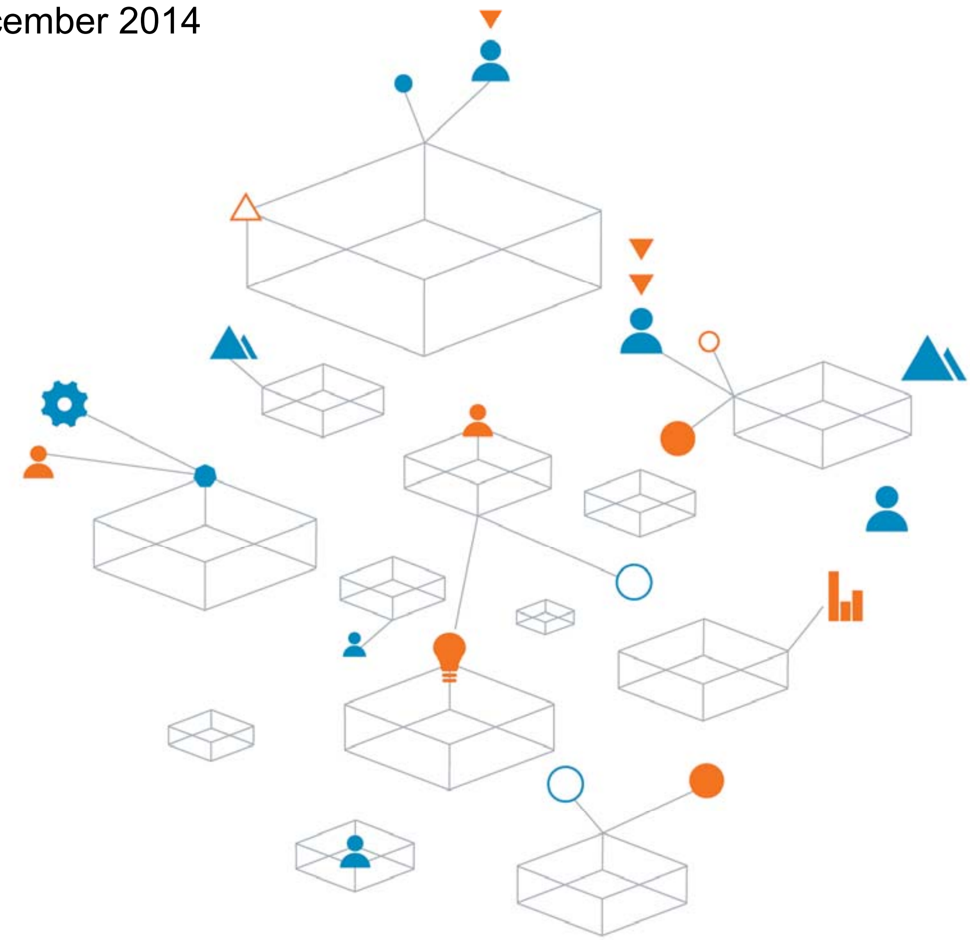


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Table of Contents

Appendices

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 rehabilitate the existing Barrie-Collingwood Railway Overpass at Station 29+550 (Hwy. 400 centreline chainage). A new overpass structure on a re-aligned and grade raised new northbound lanes is also proposed to accommodate a future 10-12 lane highway platform. The re-alignment and grade raise of about 0.5 m at the overpass location is intended to improve geometrics and safety. Information supplied by Morrison Hershfield (herein "MH") indicates the construction of the new NB structure, and rehabilitation of the existing structure will be carried out in two stages:

- Stage 1 – Construction of new NB BCR structure with permanent retaining walls (close to the east ROW) and temporary retaining walls (beyond the outer edge of pavement of existing NB highway) on the east side of the existing highway. NB traffic will be diverted to the new NB structure after construction.

- Stage 2 – Rehabilitation (strengthening) of the existing structure, temporary retaining wall installation, grade raise and embankment widening on the west side of the highway.

Drawings provided by MH (refer to **Appendix D**) show the existing single span rigid frame structure has a span of 10 m span and is 29 m long. The existing BCR overhead is supported on 4.3 m wide spread footings (assuming 0.6 m thickness) founded on the native soil at elev. 233.5 m (east side) and 233.9 m (west side). The existing embankment height at the structure location is 7.5 m.

The foundation investigation, consisting of three (3) boreholes, shows the site is underlain by fine grained non-cohesive soils in a general compact state. The natural soil is classified as sandy silt to silty sand and silt to silt and sand within the exploration depth.

The groundwater table at the site is situated between elev. 231 m and 230 m.

A General Arrangement (GA) drawing was not available for the new structure at the time of preparing this report.

5.2 New Structure Foundation Options

It is assumed that the new structure will be similar to the existing structure but not be made integral with the existing structure. The embankment grade raise will be 0.5 m maximum at both structure locations.

Due to environmental constraints (TCE contamination) deep foundation options for the new structure are not being considered.

The new structure may be supported on shallow spread footings located at the same elevation as the existing or slightly above, to avoid undermining of the existing bridge foundations. New structure footings founded at the same elevation as the existing within the existing compact silty sand deposit may be designed for a factored geotechnical resistance of 350 kPa at ULS and 225 kPa at SLS for a concentric vertical loading condition. The unfactored horizontal resistance against sliding between poured concrete and the native silty sand can be estimated using a friction angle of 28 degrees.

A minimum 1.5 m of earth cover or equivalent insulation should be provided to all shallow foundations for frost protection.

Excavations for the new footing, down to elev. 233.0 m, may not require dewatering, as the excavation base will be located above the groundwater table at about elev. 231 m. However, the existing footing should be protected. A line drawn down at 45 degrees from any part of the existing footing that intersects the new foundation excavation could cause instability of the existing structure. Should this occur, a rigid excavation shoring system will need to be provided to protect the existing footings.

5.3 Existing Structure Rehabilitation

The following geotechnical parameters may be used for structure evaluation purposes:

Embankment Fill (loose to compact silty sand to sandy silt)

- Friction angle, ϕ' = 28 degrees
- Unit weight, γ = 19.5 kN/m^3
- Coefficient of active earth pressure, K_a = 0.36
- Coefficient of at-rest earth pressure, K_0 = 0.53
- Coefficient of passive earth pressure, K_p = 2.77

Abutment rigidity and potential wall deflections should be considered in selecting an appropriate earth pressure coefficient.

Foundation Soil (compact silty sand to sandy silt) modulus of subgrade reaction, K_s = 50 MPa/m.

Structure rehabilitation details were not available at the time of preparing this report. Since the new NB overpass will not be structurally attached to the existing overpass structure, any settlements during and after construction of the new NB structure will not have a significant impact on the existing overpass and existing highway embankment.

5.4 New NB Embankment Construction

A new north bound embankment will be constructed with the support of a permanent retaining wall along the east ROW and a temporary retaining wall near to the existing NB edge of pavement. Since the existing foundation soil is loose to compact the proposed embankment should be constructed in stages to reduce post-construction residual settlement and to permit excess pore water pressures to dissipate.

5.4.1 Permanent Retaining Walls

The proposed new NB BCR structure construction includes the construction of a permanent retaining wall on the east side. The proposed wall height near the structure location is about 8 m (refer to **Appendix E**). Due to space limitations, vertical walls are proposed. The loose to compact soil conditions preclude the use of a conventional rigid concrete cantilever type of wall, owing to settlement and stability concerns. A more settlement insensitive wall, such as RSS, is better suited to the site subsurface conditions.

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 the preliminary retaining wall design. 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 needs to be assessed when detailed wall design drawings become available.

5.4.2 Temporary Retaining Walls

The maximum height of a temporary retaining wall will be 0.5 m at the BCR location (refer to **Appendix E**). Conventional cast-in-place concrete wall or concrete jersey barriers may be selected to retain the proposed grade raise. The existing highway embankment or newly constructed RSS embankment can safely support the proposed grade raise and retaining walls.

5.5 SB Embankment Reconfiguration

The existing SB embankment slope will be widened towards the west. A temporary retaining wall will be required close to the existing highway centreline.

5.5.1 Embankment Widening

About 6 m of embankment widening (refer to **Appendix E**) is proposed towards the west side of the existing highway, without the use of retaining walls. 2H:1V embankment side slope similar to the existing embankment, can be used for the proposed widening. Embankment widening should be carried out in accordance with O*PSS.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 is 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 loose lift thicknesses not exceeding 200 mm to 300 mm (depending on material type - thicker lift for coarser material). Each lift should be uniformly compacted to at least 95 percent of the material's Standard Proctor Maximum Dry Density (SPMDD). This should be increased to not less than 98 percent of the material's SPMDD within 1 m of the pavement subgrade.

Where space is available, mid-height slope benches should be provided as per *OPSD 202.010 slope flattening using surplus excavated material on earth and rock embankment.* Embankment slopes should be protected using sodding or seed and cover (OPSSs 571 and 572).

5.6 Lateral Earth Pressure

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 and 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-S6-00. For design purposes, the following static parameters (unfactored) can be used.

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

Angle of Internal Friction, ϕ = 35° (unfactored)
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressure:
K_A = 0.27
K_O = 0.43

Compacted Granular 'B' Type I

Angle of Internal Friction, ϕ = 32° (unfactored)
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressure:
K_A = 0.31
K_O = 0.47

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

The design of abutment and retaining walls adjacent to the railway tracks should be carried out as per American Railway Engineering and Maintenance-of-Way Association (AREMA) or Canadian National Railway (CNR) design guidelines.

5.7 Frost Depth

The design frost penetration depth for this project is 1.5 m.

5.8 Seismic Design Consideration

The subsurface conditions encountered at the site are represented by Soil Profile Type II (refer to Clause 4.4.6.2 of CHBDC CAN/CSA-S6-00). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S, for the site is 1.2. Table A3.1.1 of the CHBDC provides that Toronto has a Zonal Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Zv) of zero. As site coefficient (S) is 1.2, and the zonal acceleration is 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. These should be reviewed by the Structural Engineer.

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

Excavations will extend through sandy embankment fill and native sandy silt deposits. These soils can be classified as follows:

Fill	Type 3 soil above water level

Native Sandy Silt	Type 3 soil above, Type 4 below, the water table

Temporary shoring may be required to retain the existing embankment during new structure construction and to support the excavation below existing foundation levels (if necessary), due to the proximity of the existing BCR structure foundation. Dewatering may not be required for excavations taken down to the existing foundation level but dewatering may be required if excavations extend deeper. 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. Shoring systems should be designed by Professional Engineers specializing in shoring works. The soil parameters for shoring design are given in Table 5.8.1. The shoring design should satisfy the requirements of *OPSS539* and/or AREMA/CNR design guidelines, whichever is more stringent. Due to the height of the vertical retaining walls and environmental constraints (TCE contamination), additional reinforcement such as earth anchor tie-backs or anchorage into deadmen cast into new fill being placed may be considered to satisfy performance criteria.

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 cobbles and random boulders may be present within the existing Highway 400 embankment fill. Where present, they may cause some problems during the installation of shoring elements, such as vibrated or driven interlocking steel sheet piles.

5.10 Underground Utilities

Existing underground utilities (if any, such as watermain, sewer and gas main) should be properly protected during construction.

5.11 Instrumentation and Monitoring

Instrumentation for vibration and settlement monitoring, including the measurement of pore pressure response to new embankment and foundation loading, is recommended, both to control construction speed and progress and to adapt the design to observational feedback.

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 H** are integral part of this report.

For and on behalf of Coffey

Draft

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

Draft

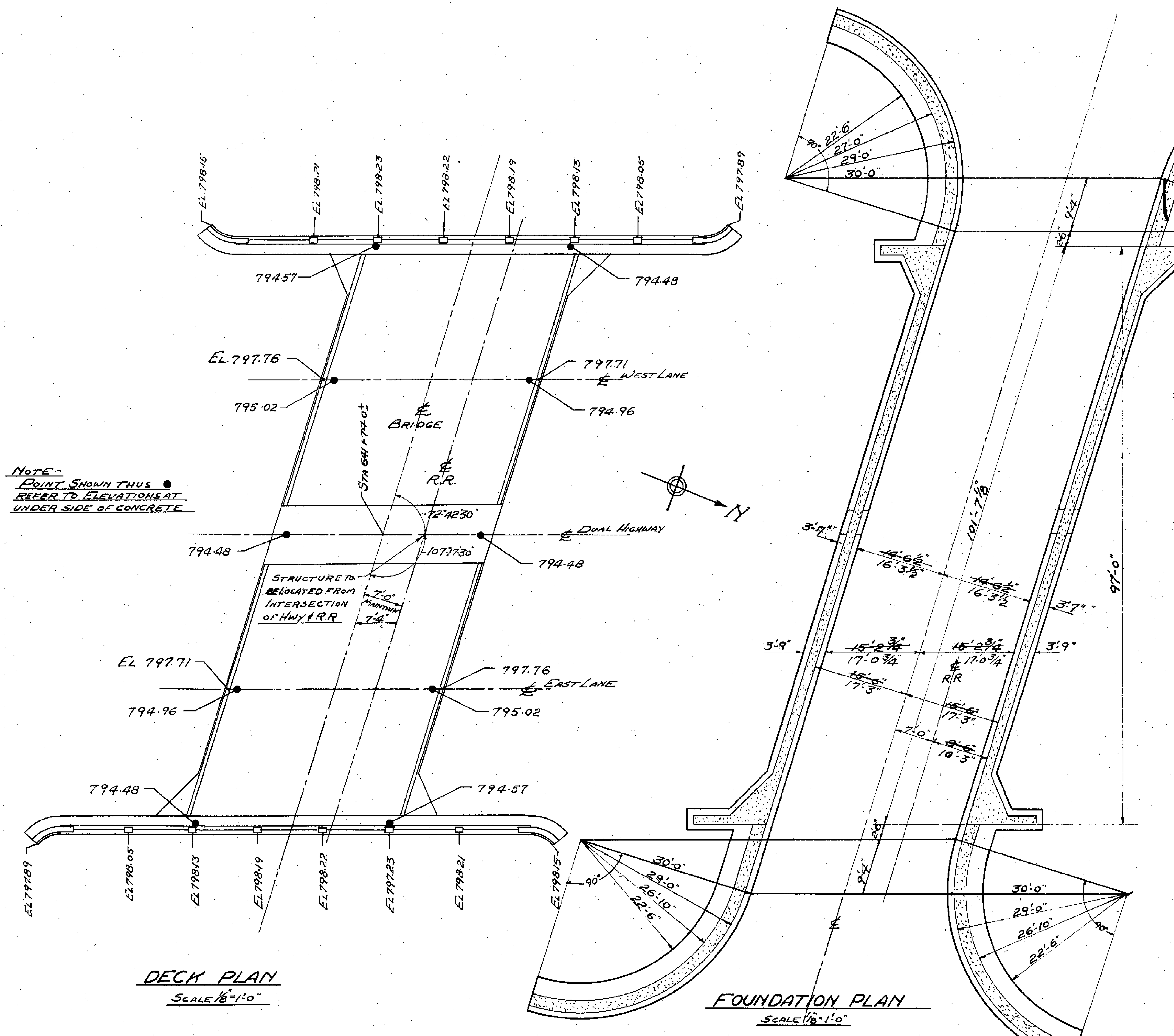
Sanket Shah, P. Eng.
Project Manager, Geotechnical Engineer

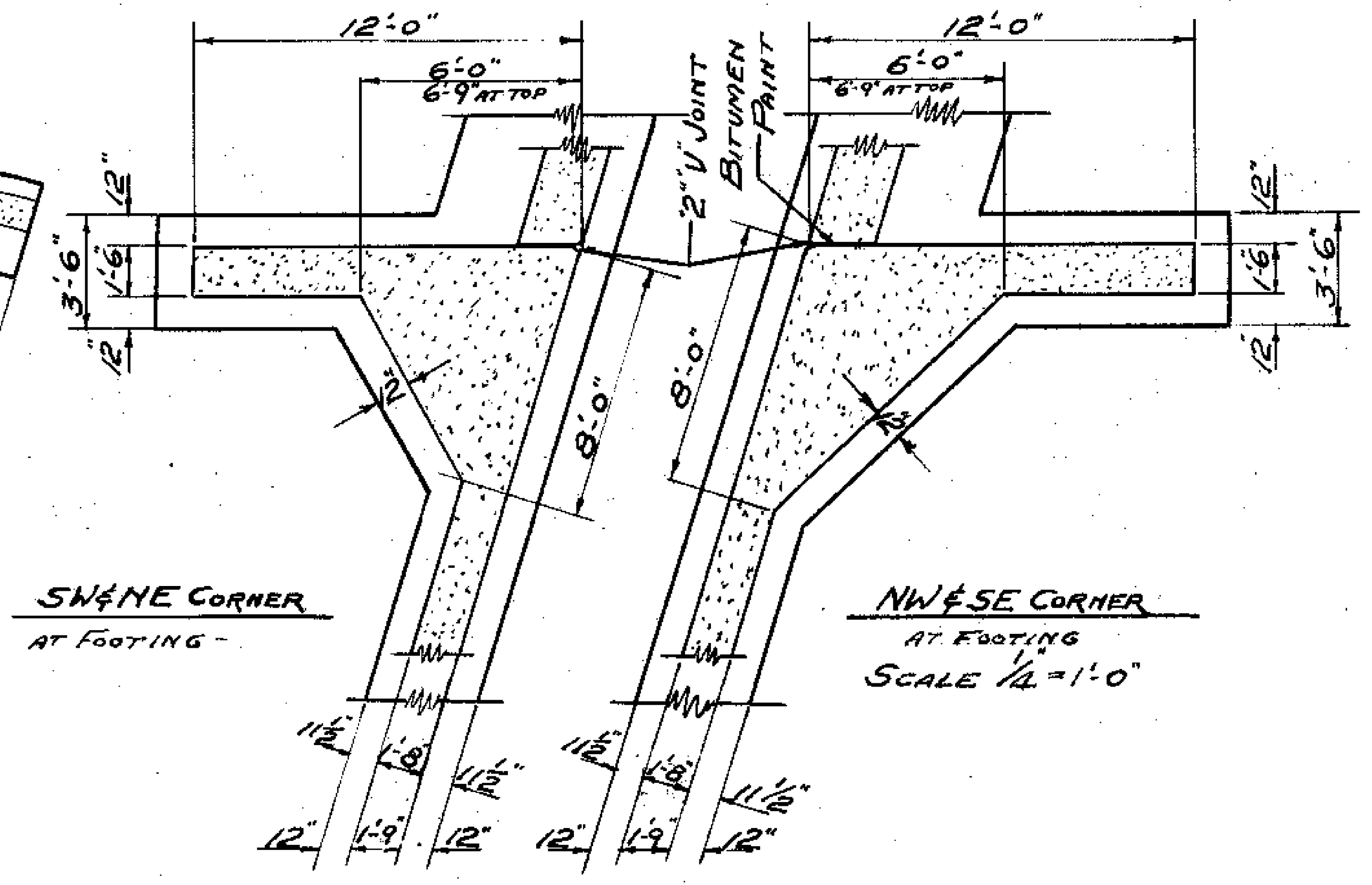
Draft

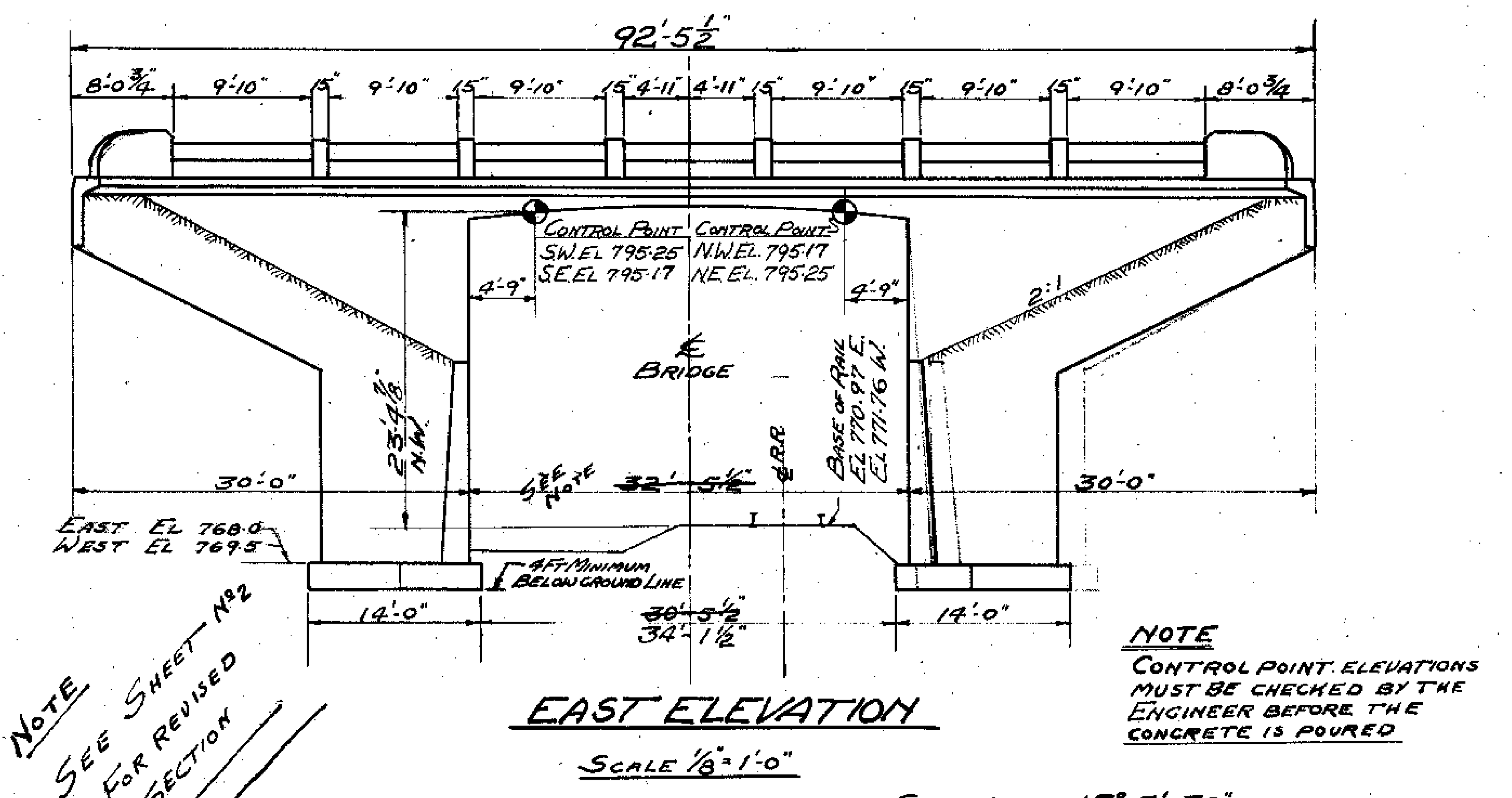
Cam Mirza, P. Eng.
MTO Designated Contact, Principal

Appendix D

BCR Overpass As-built Drawing







NOTE
SEE SHEET Nº2
FOR REVISED
SECTION

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

SKEW	17°17'-30"
SIN	.29724
COS	.95480
TAN	.31131

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

ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER

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

30-177 D3102 1 to 4

BRIDGE Nº 1

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

C.N.R OVERPASS

BARRIE BY-PASS

THE KING'S HIGHWAY No. DIV. No. 39
CO.
TWP INNISFIL LOT CON.

PLAN & ELEVATION

APPROVED
[signature] CHIEF BRIDGE ENGINEER
[signature] CHIEF ENGINEER

DESIGN	W.B	CHECK	CSG	CONTRACT NUMBERS	50-10
DRAWING	W.B	CHECK	CSG		
TRACING	W.B	CHECK	CSG	LOADING H20	DRAWING NUMBER D-3102-1
DATE	FEB 1950				

REVISIONS:

DATE	BY	DESCRIPTION
29-6-50	W.B	FOOTING SPACING WIDENED

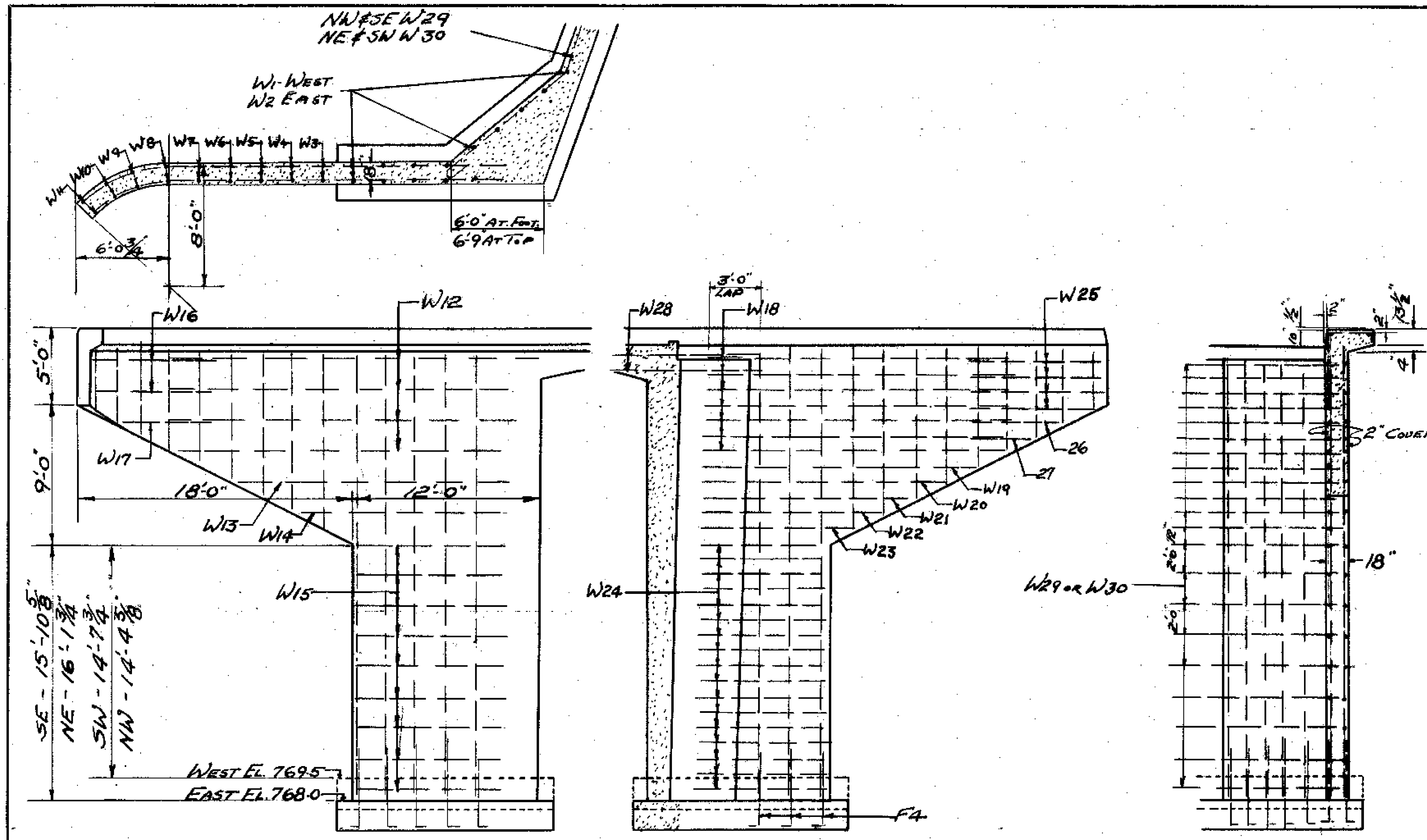
TWP# 77-177-1-A 7713

PRINT RECORD		
NO.	FOR	DATE
1	CHECK	2-8-50
30	Tender	8-3-50
1	CHECK	18-4-50
1	"	4-5-50
14	D.F.L.B7	9.5.50
14	do.	3-7-50
1	F	28-7-50

30 - 177

PRINT RECORD		
No.	FOR	DATE
1	CHECK	2-8-50
30	Tender	8-3-50
1	CHECK	18-4-50
1	"	4-5-50
14	D5 F1 L1 B7	9.5.50
14	do.	3.7.50
1	F	28-7-50
1	D	4-8-50

30-111



OUTER FACE

INNER FACE

WING WALL

SCALE 3/16" = 1'-0"

PRINT RECORD		
NO.	FOR	DATE
1	CHECK	2-8-50
30	Tender	8/3/50
1	CHECK	18-4-50
1	"	4-5-50
14	Def. L. B7	9.5.50

BRIDGE No 1

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

C.N.R. OVERPASS
BARRIE BY-PASS

THE KING'S HIGHWAY No. DIV. No. 39
CO.
TWP. INNISFIL LOT CON.

WING WALL DETAILS

APPROVED: Arthur Sedgwick, CHIEF BRIDGE ENGINEER; A. A. Smith, CHIEF ENGINEER

DESIGN	Wm B	CHECK	CSG	CONTRACT NUMBERS	50-10
DRAWING	Wm B	CHECK	CSG		
TRACING	Wm B	CHECK	CSG	LOADING H20	DRAWING NUMBER D-~~3102-3~~
DATE	FEB. 1950				

REVISIONS: DATE	BY	DESCRIPTION

TWP# 77-177-3-A ~~77-13~~-5

Mark	Number	Size	Length	Detail	Remarks
Abutment Foundation					
F1	232	3/4"φ	11'-5 1/4"	5'-0" 5 1/2 R 1'-5 1/4" 5'-0"	@10" o.c.
F2	4	5/8"φ	25'-0"	Straight	In West Side Lap 2'-0"
F3	4	Do	27'-0"	Do	In East Side Do
Wing Wall - Foundation					
F4	52	5/8"φ	5'-0"	4'-6" 6"	@24" o.c. as shown
Arch Extrados					
A1	116	3/4"φ	11'-3"	Straight	West @10" o.c.
A2	116	Do	12'-9"	Do	East @10" o.c.
A3	230	Do	27'-7 5/8"	A3-8'-3" A4-11'-3" 2'-0"R 97° 1'6"	@10" o.c. Alt. A4
A4	232	Do	30'-7 5/8"	6"	@10" o.c. Lap A1 or A2
A5	58	Do	16'-0"	Straight	Do Lap A4
Arch Extrados					
A6	116	1"φ	15'-0"	Straight	@10" o.c.
A7	58	Do	22'-0"	Do	@20" o.c.
A8	58	Do	34'-0"	Do	@20" o.c. Alt A7
A9	116	3/4"φ	25'-0"	Do	@10" o.c. West
A10	116	Do	26'-6"	Do	@10" o.c. East
Ties For Deck					
A11	144	5/8"φ	26'-5"	Straight	@2'-0" o.c. Lap 1'-6" 36 Lines - 4 Per Line
Ties For Abut					
A12	176	5/8"φ	26'-5"	Straight	@2'-6" o.c. Lap 1'-6" 42 Lines 4 Per Line 4 Lines 2 Per Line - East
Stirrups					
S1	174	3/8"φ	2'-6"		
S2	116	Do	2'-6 1/2"		
S3	116	Do	2'-8"	10" Varies Soft Grade	Bend in Field
S4	116	Do	2'-10 1/2"		
S5	116	Do	3'-0"		Place as Shown
S6	116	Do	3'-3"		
S7	116	Do	3'-7"		
S8	116	Do	3'-10"		
Wing Walls Vertical					
W1	26	5/8"φ	26'-0"	Straight	In West Side @24" o.c. 5'-1/4
W2	26	Do	27'-6"	Do	In East Do Do
W3	8	Do	12'-0"	Do	All W.Ws @24" o.c. 5'-1/4
W4	8	Do	11'-0"	Do	Do
W5	8	Do	10'-0"	Do	Do
W6	8	Do	9'-0"	Do	Do
W7	8	Do	8'-0"	Do	Do
W8	8	Do	7'-0"	Do	Do
W9	8	Do	6'-0"	Do	Do
W10	8	Do	5'-0"	Do	Do
W11	8	Do	4'-0"	Do	Do

1 Check 2-8-50
1 Check 18-4-50
1 " 4-5-50
10 O.I.A. Del'd. 9-5-50

Mark	Number	Size	Length	Detail	Remarks
Wing Walls - Horizontal					
Outer Face					
W12	16	5/8"φ	22'-0"	Straight	@24" o.c.
W13	4	Do	18'-0"	Do	Do
W14	4	Do	14'-0"	Do	Do
W15	34	Do	10'-0"	Do	Do
W16	8	Do	8'-6"	W16-5'-6" W17-3'-3" 3'-0"	Lap W12-3'-0"
W17	4	Do	6'-3"	R-6'-8"	Do
Inner Face					
W18	28	1"□	20'-0"	Straight	@12" o.c.
W19	4	Do	18'-0"	Do	Do
W20	4	Do	16'-0"	Do	Do
W21	4	Do	14'-0"	Do	Do
W22	4	Do	12'-0"	Do	Do
W23	4	Do	10'-0"	Do	Do
W24	66	5/8"φ	8'-0"	Do	Do
W25	16	3/4"φ	9'-6"	6'-6" W25 4'-3" W24 2'-0" W27 3'-0"	Lap W18-3'-0"
W26	4	Do	7'-3"		Do
W27	4	Do	5'-0"	7'-10"	Do
W28	8	3/4"φ	10'-0"	Straight	Lap W18-3'-0"
Haunches					
W29	41	5/8"φ	14'-10"	12" 3R 1'-4" 3'-0" 10'-6"	As shown in NW & SE
W30	41	Do	11'-4"	3R 1'-4" 3'-0" 7'-0" 2'-0"	As shown in NE & SW
Curbs					
Stirrups on Bridge Ext.					
C1	46	5/8"φ	6'-1 1/2"	2'-10" 2 1/2" 6" 4"R 6 1/2" 15"	3 Near Post as Shown @18" o.c. Along Curb
Stirrups on Wing Walls					
C2	96	5/8"φ	6'-4"	2'-10" 6" 3'-0"	3 Near Posts as Shown @18" o.c. Along Curb
Ties For Above					
C3	16	5/8"φ	24'-0"	Straight	4 Lines - 4 Per Line Lap 2'-0"
C4	4	5/8"φ	9'-0"	C4-6'-0" C5-4'-0" 3'-0"	C4 - Inside Lap C3 C5 Outside 2'-0"
C5	4	Do	7'-0"	R C4-7'-8" C5-5'-0"	
Curbs - Bridge Interior					
C6	20	5/8"φ	11'-8"	14" 9'-4" 14"	@2'-0" o.c.
C7	10	Do	19'-6"	Straight	5 Lines - 2 Per Line Lap 2'-0"
Handrail Posts - Intermediate					
H1	48	3/4"φ	4'-10"	1'-4 3R 3'-6"	4 Per Post
H2	36	3/8"φ	3'-8"	7 12 3"	3 Per Post

Mark	Number	Size	Length	Detail	Remarks
End Posts					
H3	24	3/4"φ	5'-2"	1'-4 3R 3'-10" H3 3'-0" H4 1'-10" H5	6 Per Post as Shown
H4	8	Do	4'-4"		2 Do
H5	8	Do	3'-2"		2 Do
H6	8	1/2"φ	11'-4"	1'-9" 9'-3" 3'-10" R-5'-6" R-4'-11"	2 Per Post as Shown
H7	4	Do	7'-10 1/2"	1'-9" 2'-0" 1'-9 1/2" R-5'-6" R-4'-11"	1 Per Post as Shown
Hand Rail					
H9	32	3/4"φ	22'-10"	Straight	8 Lines - 4 Per Line - Lap 3'-0" Run 30" into End Post
H10	98	3/8"φ	3'-4"	5" 12" 3"	7 Per Rail @18"±
Retaining Wall					
Foundation					
R1	40	5/8"φ	24'-0"	R. 26'-0"	Spaced as Shown Lap R2 2'-0" in NW & SE
R2	10	Do	12'-0"	Straight	As Shown in NW & SE
R3	105	5/8"φ	9'-1 1/2"	R3 & R3A 3" 6" 1'-4 3R 2'-0"	@12" o.c. NW & SW
R3A	105	3/4"φ	9'-1 1/2"		@12" o.c. NE & SE
Wall - Vertical - West					
R4	160	5/8"φ	13'-3"	Straight	Lap R3 on Inside at 12" o.c. @24" o.c. on Outside
Vertical East					
R5	160	5/8"φ	14'-9"	Straight	Lap R3A on Inside @12" o.c. @24" o.c. on Outside
Horizontal					
R6	192	5/8"φ	23'-6"	27'-6"R	@1'-3"± o.c. Lap 2'-0" 2 Per Line Lap R7 in NW & SE
R7	48	Do	12'-0"	Straight	@1'-3"± o.c. in NW & SE
R8	210	Do	7'-0"	Do	In Foundation @12" o.c.

Note
All steel hard grade unless noted otherwise

Regn. No. 56345

Bridge No I

Department of Highways - Ontario
Bridge Office - Toronto

Reinforcing Steel
for
C.N.R. Overpass
on
Barrie By-Pass

Dwg. W.B. | Cont. No 50-10
Chkd. C.S.G. | Dwg. No ~~D-3102-4~~

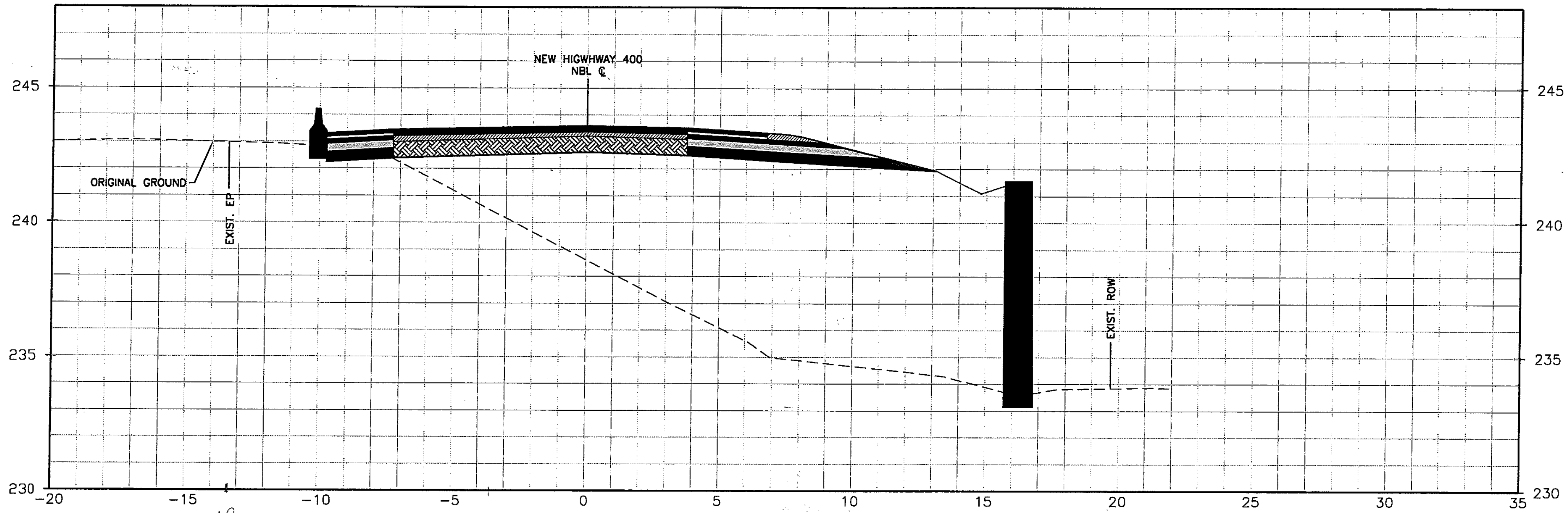
~~77-13-~~6
TWP 77-177-4-A

Appendix E

Cross-sectional Drawings

NBL STAGE 1 PRELIMINART SECTIONS.dwg

18.12.2009



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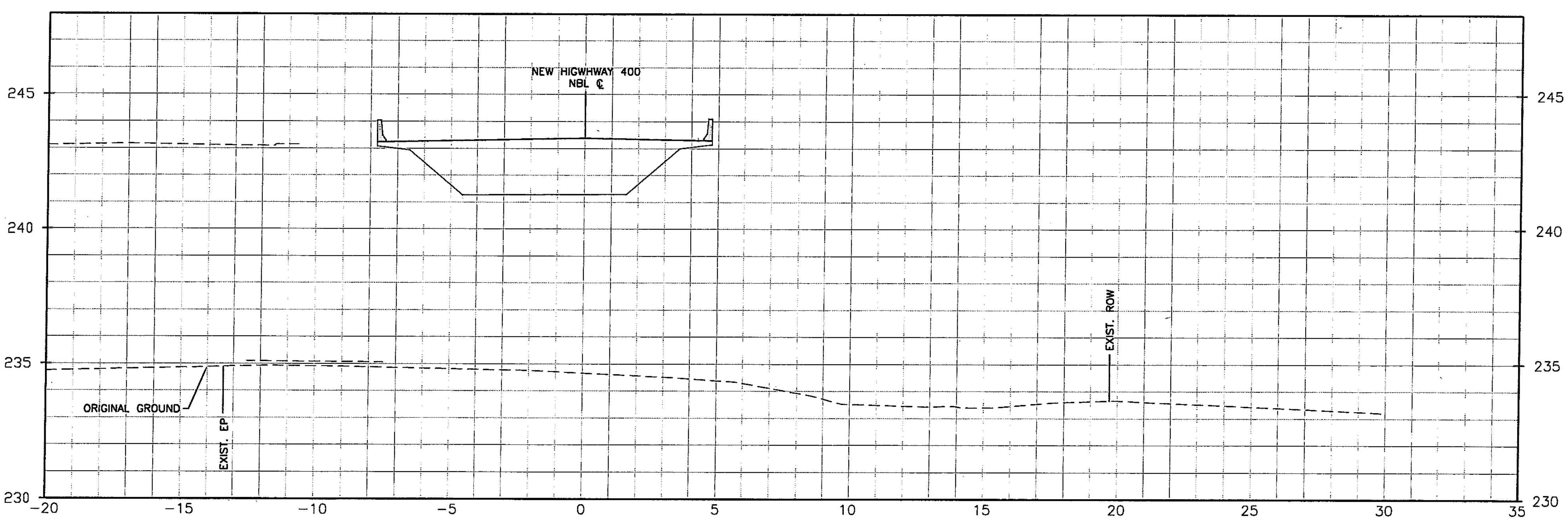

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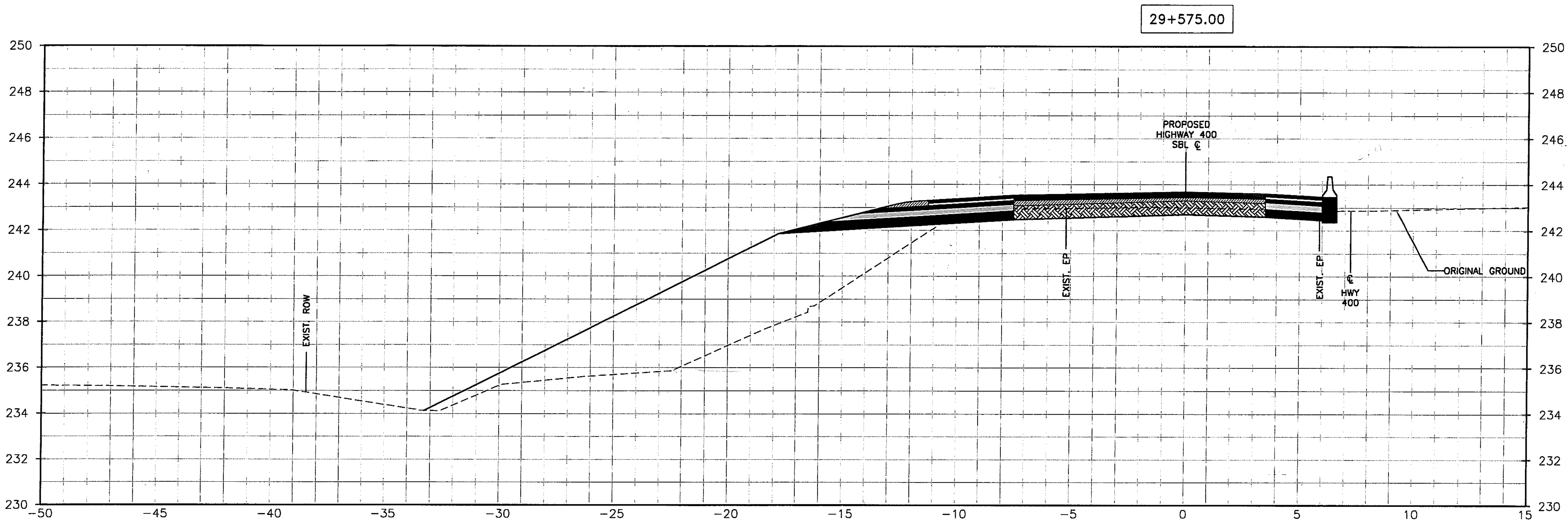
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HIGHWAY 400
SBL ℄
EXIST. ROW
EXIST. EP
EXIST. EP
℄
HWY
400
ORIGINAL GROUND
250
248
246
244
242
240
238
236
234
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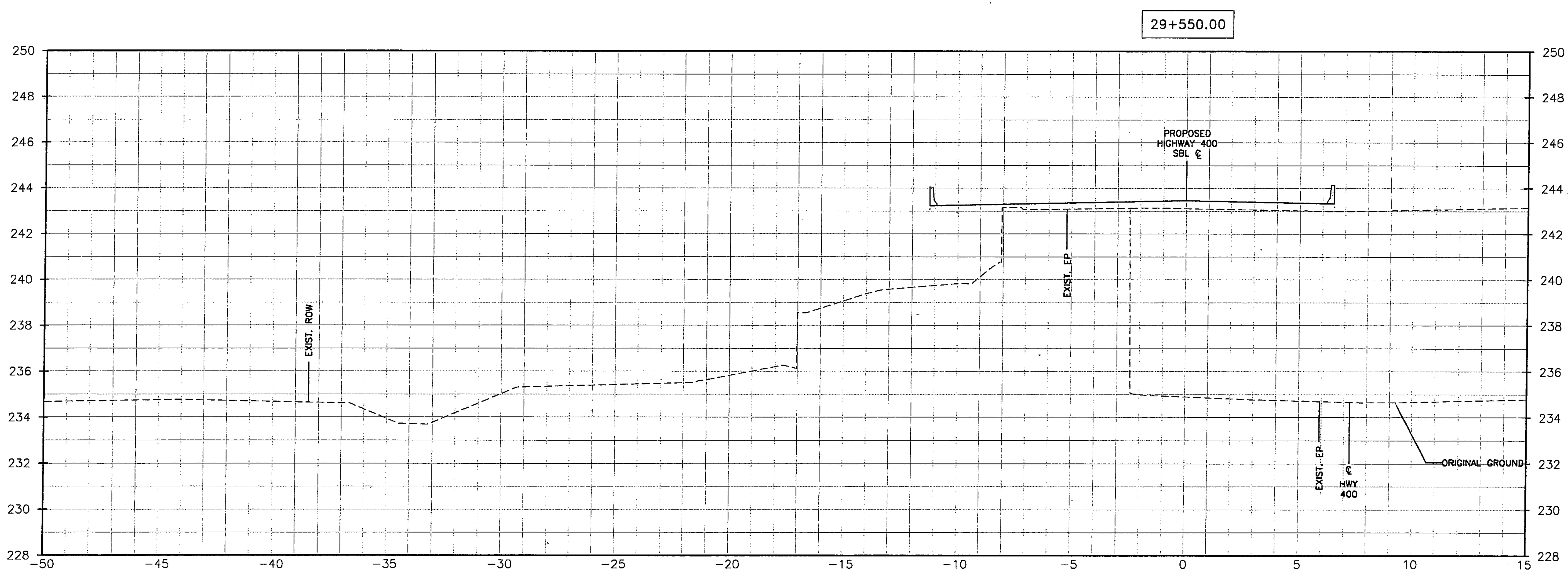
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PROPOSED
HIGHWAY 400
SBL ℄
EXIST. ROW
EXIST. EP
EXIST. EP
℄
HWY
400
ORIGINAL GROUND
250
248
246
244
242
240
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234
232
230
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-5
0
5
10
15

Appendix F

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 G

NSSPs

VIBRATION MONITORING

Special Provision

The vibration monitoring equipment shall be placed on the existing structure such that it will not be disturbed. The location should be as close as possible to the piling works.

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

The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and during seating of the pile onto the bedrock.

The 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 prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the results are acceptable, the Contractor may continue with the remaining piles with readings taken during driving of each pile. Subsequent vibration readings should be taken for each pile during bedrock seating. The results of the subsequent piles should be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specifications. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must alter his driving procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile.

Appendix H

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.