



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
NORTHEAST CORNER RETAINING WALL
FREDERICK STREET UNDERPASS
SITE NO. 33-234
G.W.P. 3110-09-00
CITY OF KITCHENER, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 3 cc: McCormick Rankin Corporation (MRC) for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 3 cc: Foundation Investigation Report only to MRC for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 1 cc: MRC for distribution to MTO, Pavements and
Foundations Section + 1 digital copy (pdf) and
Drawing (AutoCAD)
- 1 cc: Foundation Investigation Report only to MRC for
distribution to MTO, Pavements and Foundations
Section + 1 digital copy (pdf) and Drawing (AutoCAD)
- 2 cc: MRC +1 digital copy (pdf)
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 10KF079C
Index No.: 118FIR and 119FDR
GEOCRES No.: 40P8-199
May 31, 2012



**FOUNDATION INVESTIGATION REPORT
for
NORTHEAST CORNER RETAINING WALL
FREDERICK STREET UNDERPASS
SITE NO. 33-234
G.W.P. 3110-09-00
CITY OF KITCHENER, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 3 cc: McCormick Rankin Corporation (MRC) for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 3 cc: Foundation Investigation Report only to MRC for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 1 cc: MRC for distribution to MTO, Pavements and
Foundations Section + 1 digital copy (pdf) and
Drawing (AutoCAD)
- 1 cc: Foundation Investigation Report only to MRC for
distribution to MTO, Pavements and Foundations
Section + 1 digital copy (pdf) and Drawing (AutoCAD)
- 2 cc: MRC +1 digital copy (pdf)
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 10KF079C
Index No.: 118FIR
GEOCRES No.: 40P8-199
May 31, 2012



TABLE OF CONTENTS

1. INTRODUCTION	1
2. SITE DESCRIPTION AND GEOLOGY	1
3. INVESTIGATION PROCEDURES	2
4. SUMMARIZED SUBSURFACE CONDITIONS	4
4.1 Fill	4
4.2 Sand	5
4.3 Silty Clay	5
4.4 Groundwater	6
5. RETAINING WALL CONDITION SURVEY	6
6. MISCELLANEOUS	8

Table A-1 – List of Atterberg Limits Test Results

Figures RW-GS-1 to RW-GS-7 – Grain Size Distribution Charts

Figures RW-PC-1 and RW-PC-2 – Plasticity Charts

Explanation of Terms Used in Report

Record of Borehole Sheets

Drawing RW-1 – Borehole Locations and Soil Strata

Appendix A – Previous Investigation Report for the Frederick Street Underpass dated July 1966
(GEOCRES No. 40P8-48)

Appendix B – Site Photographs 1 to 10

FOUNDATION INVESTIGATION REPORT
for
Northeast Corner Retaining Wall
Frederick Street Underpass, Site No. 33-234
GWP 3110-09-00
City of Kitchener, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the northeast concrete retaining wall of the Frederick Street Underpass on Highway 7 / 85 (Kitchener-Waterloo Expressway) in the City of Kitchener. Peto MacCallum Ltd. (PML) carried out the investigation to determine the probable causes of the wall movement for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The retaining wall has experienced progressive movement (tilting) towards the highway. The purpose of this report was to summarize the subsurface stratigraphy encountered in the foundation investigation. A previous foundation investigation dated July 1966 for the Frederick Street Underpass was obtained from the MTO library GEOCREs No.40P8-48. A copy of the previous Report with the borehole logs and Foundation Drawing is enclosed in Appendix A for reference.

The inspection of the condition of the retaining wall above and below grade including the foundation and the associated appurtenances is considered a structural facet and was not in the scope of this investigation.

2. SITE DESCRIPTION AND GEOLOGY

The retaining wall is located on the northeast corner of the Frederick Street underpass and along the east side of the Highway 7 speed change lane (to Victoria Street). The wall is approximately 5.4 m high above grade at the bridge abutment and tapers down northerly to approximately 1.8 m height. Further description of the retaining wall are presented in Section 5 of this report.

Photographs of the site and retaining wall taken on July 20 and October 8, 2011 are presented in Appendix B.



Land use in the vicinity of the site includes the existing Highway 7 / 85 transportation corridor including the Frederick Street underpass amidst residential and commercial areas. The local topography of the site is generally flat at Highway 7 / 85. The terrain rises away from the highway and a series of retaining walls exists north and south of the Frederick Street underpass abutments. The ground in front of the retaining wall area includes a paved shoulder and sidewalk. Grasses and bushes cover the ground behind the wall (Photographs 1 to 4). The grade behind the retaining wall slopes up about 2.0 m to the east at an approximate angle of 18°.

Based on the existing construction records and report referenced above, the underpass was constructed by excavating approximately 4.7 m below the previously existing ground level and constructing the Frederick Street structure at its original road grade level.

A double catch basin exists in front of the sidewalk about 14 m north of the south end of the retaining wall, indicating the presence of a storm sewer under the roadway (Photograph 2).

The site is located within in the physiographic region known as the Waterloo Hills that is characterised by sandy hills, sandy till ridges, kames and kames moraines with outwash sandy soils occupying the intervening hollows. The principal surficial soil in the area is fine sand at the hilly regions to more uniform sandy and gravelly materials at the alluvial terraces of the Grand River spillway. Typically, the surficial soils overlay clayey soils/ clay tills.

3. INVESTIGATION PROCEDURES

The condition of the existing retaining wall was surveyed on October 8, 2011 and the results are presented in Section 5 of this report. The subsurface investigation was carried out during the period of April 8 to July 20, 2011. A total of four boreholes (RW-1 to RW-4) were advanced to depths ranging from 6.4 to 9.8 m at the locations shown on Drawing RW-1, appended.

The borehole locations had to be moved away from the retaining wall to clear existing buried utilities. The boreholes located on the Highway 7 / 85 speed change lane were advanced on April 8, 2011 using continuous flight hollow stem augers powered by a truck-mounted CME 55 drill rig. Behind



the retaining wall, the boreholes were drilled in July 2011 with a portable drill rig (Dynamic Ram Sounder). The drill rigs were supplied and operated by specialist drilling contractors, working under the full-time supervision of a PML field supervisor.

Soil samples were recovered from the boreholes drilled on the highway at regular 0.75 and 1.5 m intervals of depth using the standard penetration test method. Continuous sampling was undertaken from the boreholes drilled behind the retaining wall using the standard penetration test. One dynamic cone penetration test was also conducted to assess the strength characteristics of the substrata. Because the shear strength of the clayey soils was too high for using field vane testing, penetrometer tests were carried out on cohesive split spoon soil samples to evaluate their shear strength. Soils were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and where encountered by measuring the groundwater level in the open holes. A total of two 19 and 30 mm diameter PVC standpipe piezometers were installed in boreholes RW-1 and RW-3 for subsequent groundwater level monitoring.

The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guideline and MOE Reg. 903 for borehole abandonment.

The locations of the test holes were laid out by PML as allowed by access and underground utilities and were surveyed by MMM Group Ltd. All elevations in this report are provided in metres.

The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and tactile examination to confirm field classification. The laboratory testing program included the following tests:

- Natural moisture content determinations (32)
- Grain size distribution analyses (19)
- Atterberg limits tests (4)



The laboratory grain size distribution charts are presented in Figures RW-GS-1 to RW-GS-7. The Atterberg plasticity test results are shown on the Figures RW-PC-1 and RW-PC-2.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test results, penetrometer shear strength values, groundwater observations, details of piezometer installations and piezometric level readings. The results of laboratory particle size distributions and Atterberg limits and moisture content determinations are also shown on the Record of Borehole Sheets.

4.1 Fill

A 1.4 m thick fill unit was encountered in boreholes RW-1 and RW-2 drilled on the existing Highway 7 speed change lane in front of the retaining wall. The unit extended to 1.4 m depth (elevation 318.3). The fill layer includes asphalt over granular base and subbase materials including sand and crushed gravel. N values were 3 and 11 indicating very loose to compact relative density. The moisture content results were 15 and 16%.

In addition, a 2.3 m thick fill (backfill material for retaining wall) was present in boreholes RW-3 and RW-4 drilled behind the retaining wall. The unit extended to 2.3 m (elevation 320.0 and 321.2). The fill unit behind the retaining wall is heterogeneous and includes cohesionless silty sand / silt / gravelly sand and cohesive clayey silt. N values ranged from 14 to 27 indicating compact relative density or very stiff consistency.

The results of grain size distribution analysis for fill samples from boreholes RW-3 and RW-4 are included in Figures RW-GS-1 to RW-GS-5. The plasticity chart is presented in Figure RW-PC-1. The fill samples from boreholes RW-3 and RW-4 include 4 to 29% clay, 11 to 54% silt, 20 to 68% sand and 3 to 23% gravel sized materials. Eight grain size analyses were carried out on the fill samples obtained behind the retaining wall. The fines content (total of the silt and clay components) of these samples was in excess of 17% and in 7 of 8 samples ranged from 38 to 71%. The liquid and



plastic limits obtained on a clayey silt fill sample were 22 and 12, respectively with a corresponding plasticity index value of 10. The moisture content determinations varied from 5 to 11%.

4.2 Sand

A cohesionless native sand deposit was encountered below the fill at 2.3 m (elevation 320.0 and 321.2) in boreholes RW-3 and RW-4 drilled behind the wall. The deposit was 2.1 and 3.6 m thick extending to 4.4 and 5.9 m (elevation 317.6 and 317.9). It is noted that the sand deposit contains gravelly to with gravel in borehole RW-4 below elevation 319.7. N values ranged from 9 to 21, typically 13 and 14. The relative density of the sand deposit was compact with local loose layers.

The results of grain size distribution analysis for sand samples are included in Figure RW-GS-6. The moisture content determinations varied from 3 to 20%.

4.3 Silty Clay

A cohesive silty clay stratum was encountered below the fill at 1.4 m (elevation 318.3) in boreholes RW-1 and RW-2 and below the sand at 4.4 and 5.9 m (elevation 317.9 and 317.6) in boreholes RW-3 and RW-4. The stratum was at least 1.1 to 8.4 m thick extending to the borehole termination depths of 6.4 to 9.8 m (elevation 309.9 to 316.5). The boreholes were terminated within the silty clay stratum at 6.4 to 9.8 m (elevation 309.9 to 316.5).

It is noted that cobbles were encountered below elevation 317.0 in boreholes RW-3 and RW-4. N values ranged from 9 to 67 and 50 to 70 for 13 and 15 cm sampler penetration. Pocket penetrometer test results were 175 and 225 kPa. The stratum was very stiff to hard consistency.

The results of grain size distribution analysis for silty clay samples are included in Figure RW-GS-7. The plasticity chart is presented in Figure RW-PC-2. The liquid limits and plastic limits varied from 35 to 45 and 17 to 23, respectively with plasticity index values of 18 and 22. The moisture content determinations varied from 9 to 25%.



4.4 Groundwater

Groundwater was encountered in all of the boreholes. During augering, groundwater was observed at 3.0 and 4.2 m (elevation 319.3) in boreholes RW-3 and RW-4. Upon completion of drilling, groundwater was measured at 7.3 m (elevation 312.4) in borehole RW-2. Cave-in was observed in boreholes RW-1, RW-2 and RW-4 at 5.0 to 8.7 m (elevation 311.0 to 318.5).

A piezometer was installed in each of boreholes RW-1 and RW-3 for subsequent groundwater level monitoring. The piezometric water level readings are tabulated below:

BOREHOLE NO.	GROUND SURFACE ELEVATION	BOREHOLE DEPTH (m)	WATER LEVEL READING IN PIEZOMETER	
			DEPTH (m)	ELEVATION
RW-1	319.7	9.8	2.9 (Apr. 8, 2011)	316.8
RW-3	322.3	6.4	Dry (July 19, 2011)	-
			3.3 (Sept. 23, 2011)	319.0
			3.3 (Oct. 8, 2011)	319.0

The readings taken in the piezometer on April 8, 2011 immediately after installation, showed the water level to be at 2.9 m (elevation 316.8) in borehole RW-1. The flush mounted casing located on the highway pavement was damaged and could not be removed for further piezometer readings. Groundwater was not encountered in the borehole RW-3 piezometer on July 19, 2011. The readings taken in the piezometer RW-3 on September 23 and October 8, 2011 (65 and 81 days after installation) showed the water level to be stabilized at 3.3 m (elevation 319.0). It is noted that the groundwater levels are subject to seasonal fluctuations and precipitation patterns.

5. RETAINING WALL CONDITION SURVEY

Construction of the Frederick Street underpass and associated retaining walls was carried out under the MTO contract No. 68-62 in 1968. The MTO Foundation Investigation and Design Report prepared for the underpass construction has the GEOCREC No. 40P8-48. These original



documents indicate that the previously existing ground was cut under the Frederick Street bridge approximately 4.7 m to allow the Highway 7 / 85 to pass underneath. The earth slopes resulting from the excavations were supported by cast-in-place concrete retaining walls extending to north and south from the bridge abutments.

The subject retaining wall is located at the northeast corner of the Frederick Street underpass. The retaining wall supports the cut slope for Highway 7 / 85 speed change lane and abutment slope. The wall is a cast-in place concrete cantilever wall comprising a total of four panels with 9.1 m (30 ft) each panel. The total length of the retaining wall along Highway 7 / 85 speed change lane is about 36.6 m (120 ft). The retaining wall is about 5.4 m (17 ft 9 in) high at the south end decreasing northerly to 1.8 m (5 ft 10 in) at the north end.

The construction drawings called for the retaining wall to be constructed with a vertical front face and a slightly tapered back face. A review of the inspection report for Frederick Street Underpass, prepared by the Proctor & Redfern Group, dated November 1982 indicated that in 14 years the top of the wall had moved ± 125 mm towards the highway from its original position at the time of construction in 1968. From 1982 to 2011 (29 years), the top of the wall moved an additional ± 105 mm, for an approximate total of ± 230 mm towards the highway from its original position.

The rate of movement was calculated based on the above data to be approximately ± 9 mm/year from its construction to 1982. From 1968 to 2011 the rate of movement was about ± 5 mm/year. The rate of movement is summarized in the following table.

Measurement Years	Top of Wall Total Movement at Abutment		Elapsed Time Since 1968 (Years)	Rate of Movement	
	(mm)	(degrees)		(mm/Year)	(degrees/Year)
1982	125	1.3	14	9	0.09
2011	230	2.4	43	5	0.06

Photographs of the site and retaining wall taken on July 20 and October 8, 2011 are presented in Appendix B.



On October 8, 2011, the measurements of the top of the wall deflection from the vertical were obtained at four locations using a measuring tape and a plumb bob as illustrated in Photographs 6, 8, 9 and 10. The measurements are summarized in the following table.

Location of Section from Bridge Abutment	Retaining Wall Height above Sidewalk (m)	Overhang to Sidewalk Level (Note 1) (mm)	Rotation Angle (degrees)	Remarks
At abutment	4.4	165 (Note 2)	2.4	Measuring tape used
7 m north	4.0	152	2.1	Plumb bob and measuring tape used
14 m north	3.6	114	1.8	Plumb bob and measuring tape used
20 m north	3.1	41	0.8	Plumb bob and measuring tape used

Notes: (1) Overhang is the approximate distance that the top of the retaining wall is currently over the edge of the sidewalk at the retaining wall face.

(2) At this location, measurements were taken to the face of the bridge abutment. Top of wall – 230 mm (9 in.); 1.8 m above sidewalk – 127 mm (5 in.); 0.3 m above sidewalk – 64 mm (2.5 in.).

The measurements indicate a gradual increment of the leaning of the retaining wall toward the highway that becomes more pronounced southerly from 0.8° to 2.4° as the height of the retaining wall increases.

It is understood from MRC that further to the CCTV camera survey of the storm sewer in December 2011, the pipe is in acceptable condition and that sand was not found inside the pipe indicating that a breach has not occurred in the past.

6. MISCELLANEOUS

Messrs. F. Portela, A. Lo, R. Blair carried out the field investigation for this study under the supervision of Mrs. N.S. Balakumaran, P. Eng. and Mr. C. M. P. Nascimento, P. Eng., Project Manager. London Soils and Sonic Drilling. supplied the drill rigs for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.



This report was prepared by Mrs. N. S. Balakumaran, P. Eng. and reviewed by Mr. C. M. P. Nascimento, P. Eng., Project Manager. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



Nesam S. Balakumaran, P.Eng.
Project Engineer



Carlos M.P. Nascimento, P.Eng.
Project Manager



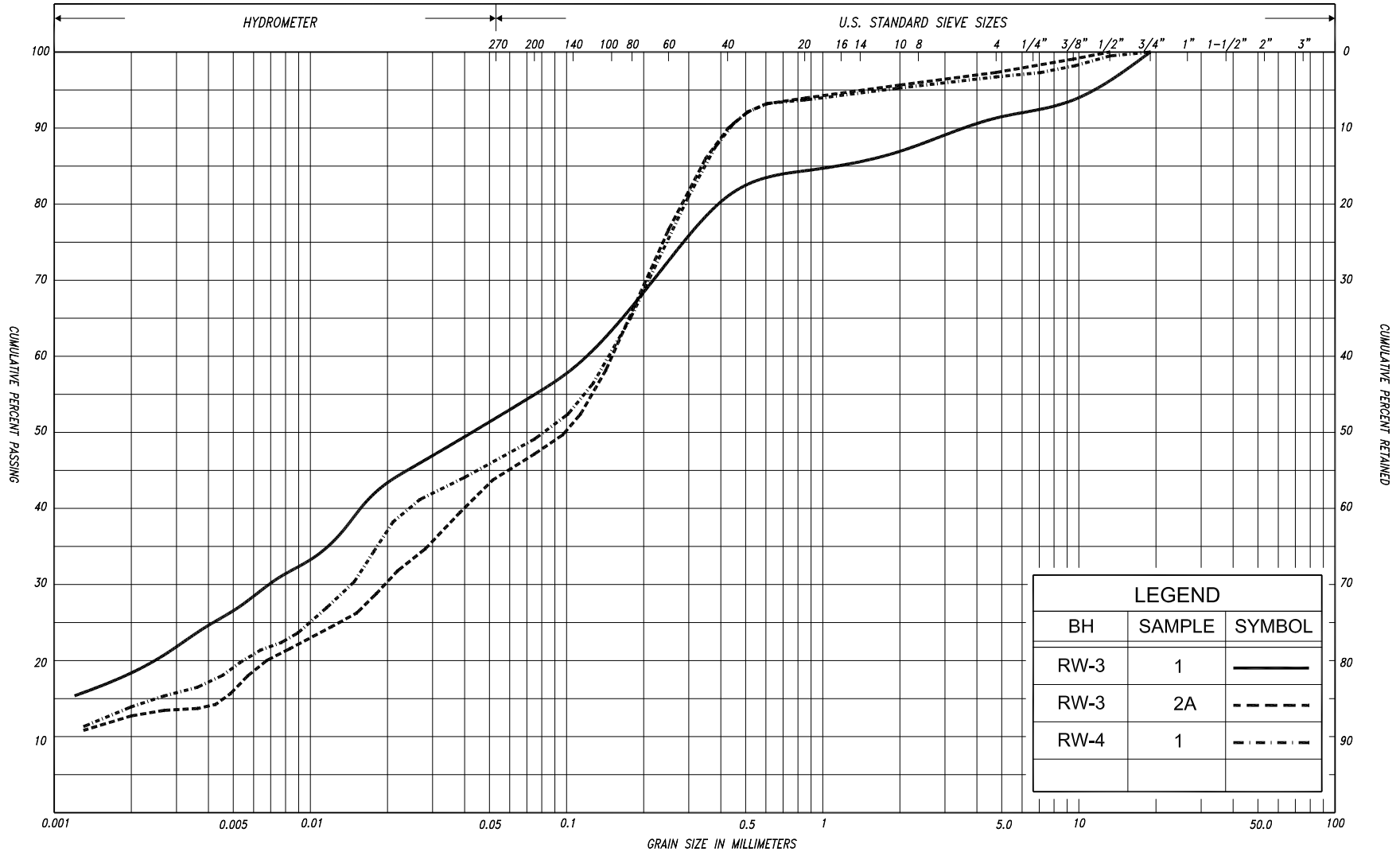
Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

NB/CN/BRG:nb-mi



TABLE A-1
 LIST OF ATTERBERG LIMITS RESULTS

SOIL TYPE	BOREHOLE NO.	SAMPLE NO.	DEPTH / ELEVATION (m)	MOISTURE CONTENT (W %)	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
Clayey Silt Fill	RW-3	3B	2.1 / 320.2	-	22	12	10
Silty Clay	RW-2	3	1.9 / 317.8	19	36	18	18
	RW-2	5	3.3 / 316.3	19	35	17	18
	RW-2	7	6.3 / 313.4	21	45	23	22



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
CLAY	FINE		MEDIUM		COARSE	SAND					GRAVEL			COBBLES	M.I.T.	
	SILT					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	U.S. BUREAU	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					COBBLES	U.S. BUREAU

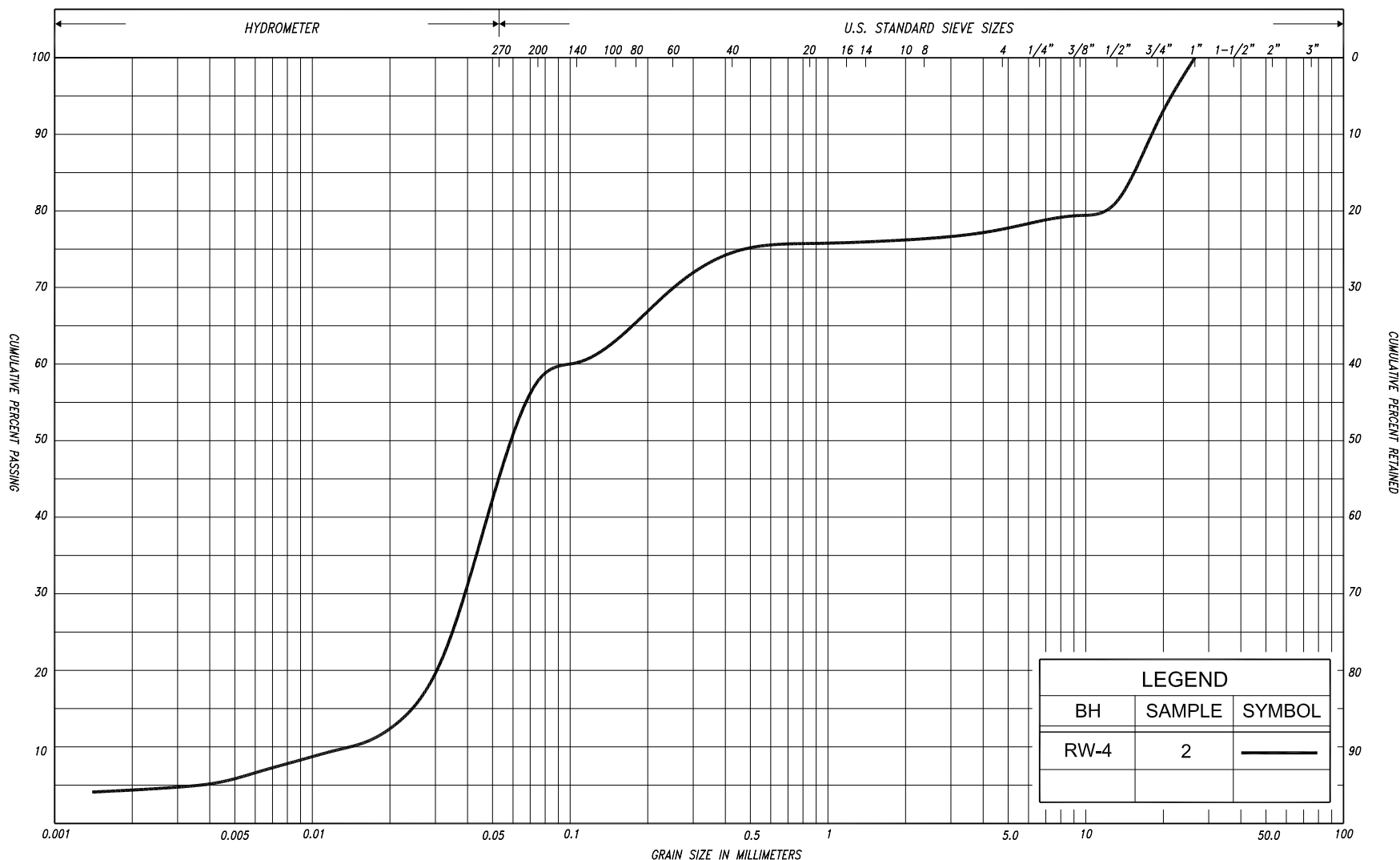
GRAIN SIZE DISTRIBUTION SILTY SAND, some clay, trace gravel (FILL)

FIG No. RW-GS-1

HWY: 7 / 85

G.W.P. No. 3110-09-00





LEGEND		
BH	SAMPLE	SYMBOL
RW-4	2	—

SILT & CLAY				SAND			GRAVEL		COBBLES	UNIFIED
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL		COBBLES	
	SILT			SAND						
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL		U.S. BUREAU

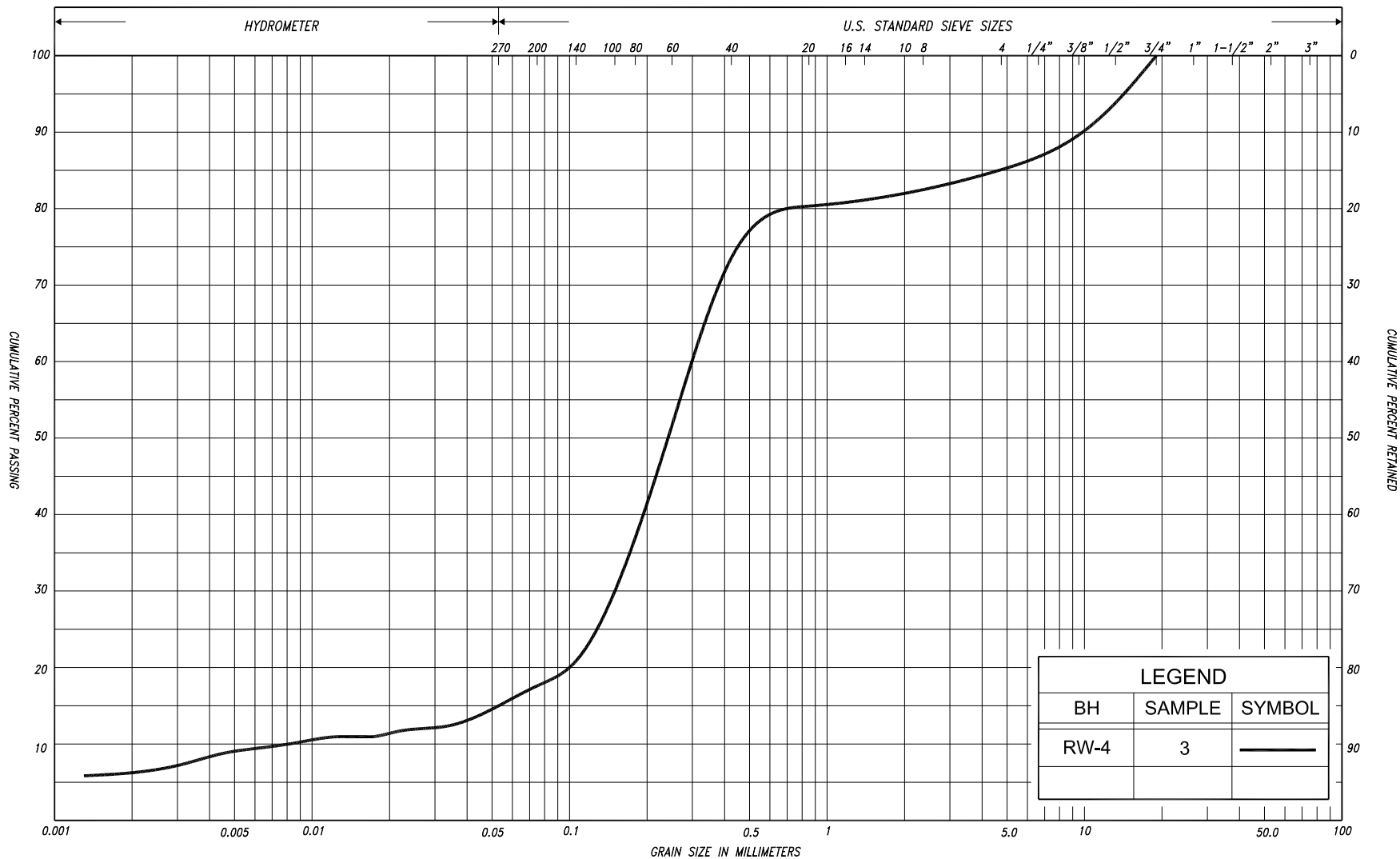


GRAIN SIZE DISTRIBUTION SILT, some sand, some gravel, trace clay (FILL)

FIG No. RW-GS-2

HWY: 7 / 85

G.W.P. No. 3110-09-00

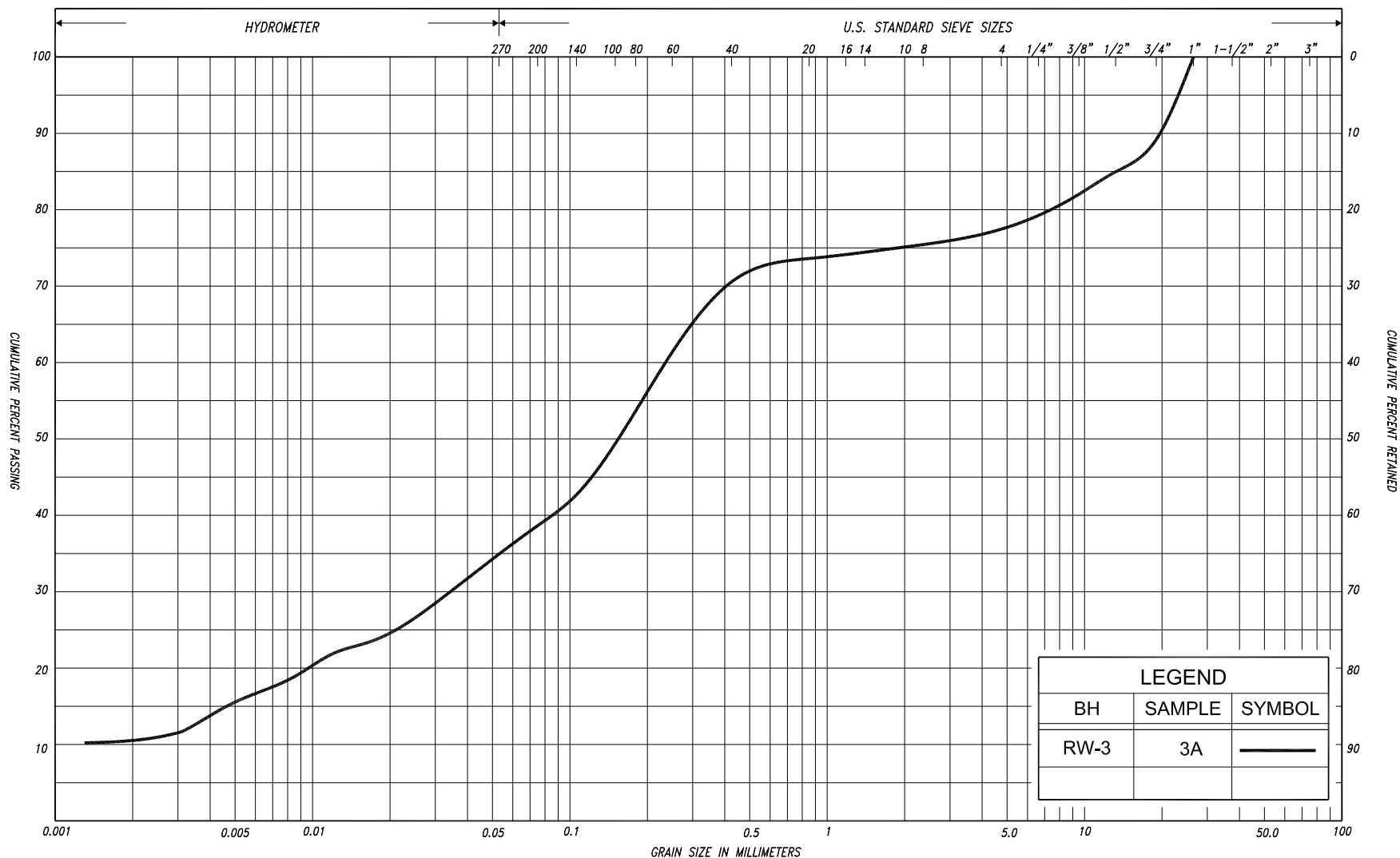


SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COB BLES	UNIFIED		
					SAND											
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.
	SILT						SAND									
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					U.S. BUREAU	
					SAND											

GRAIN SIZE DISTRIBUTION
 SAND, some silt, some gravel, trace clay
 (FILL)

FIG No. RW-GS-3
 HWY: 7 / 85
 G.W.P. No. 3110-09-00



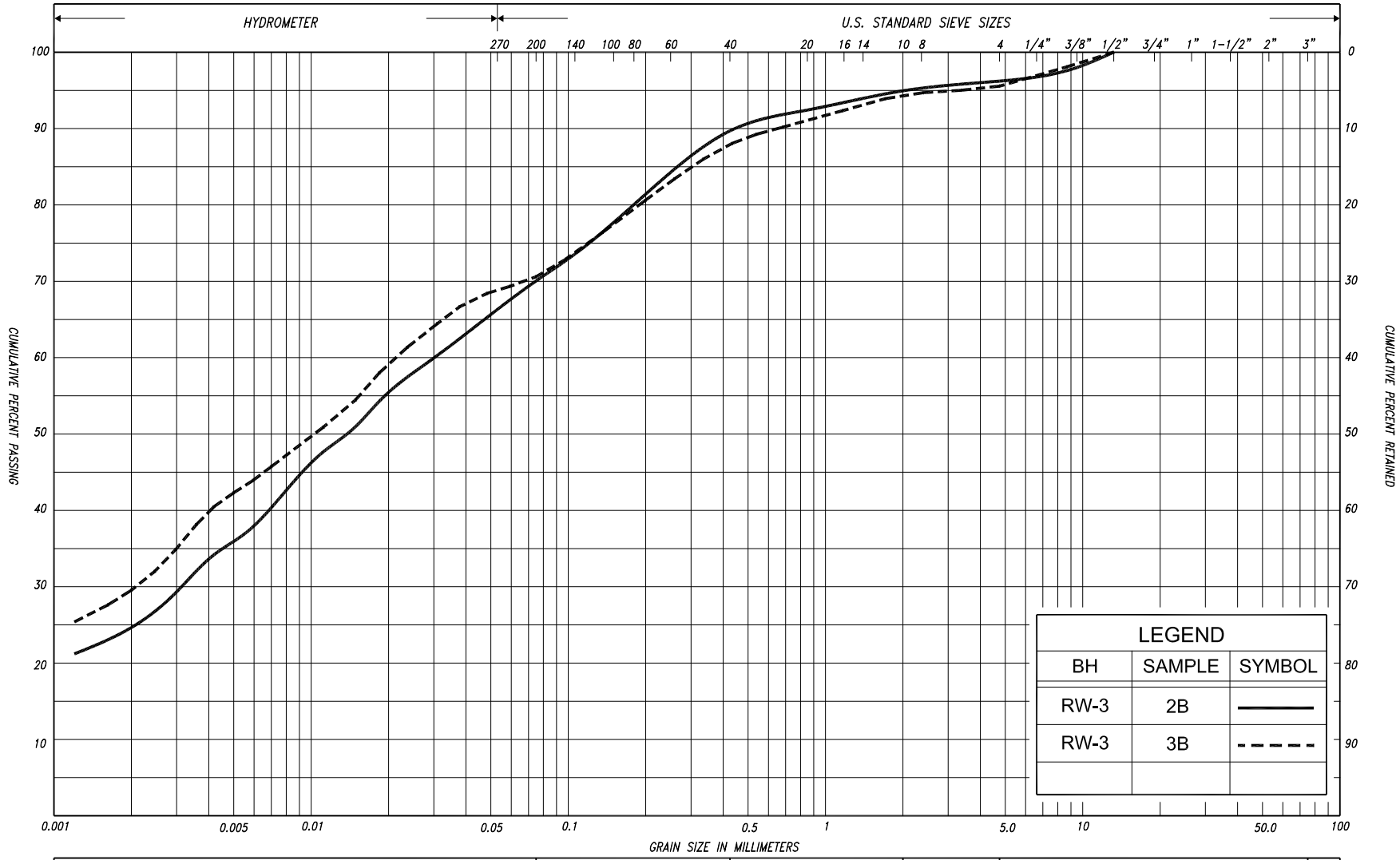


SILT & CLAY				SAND			GRAVEL	COBBLES	UNIFIED
				FINE	MEDIUM	COARSE			
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL	COBBLES	M.I.T.
CLAY	SILT			V. FINE	FINE	MED.	COARSE	GRAVEL	U.S. BUREAU



GRAIN SIZE DISTRIBUTION GRAVELLY SAND, with silt, some clay (FILL)

FIG No. RW-GS-4
HWY: 7 / 85
G.W.P. No. 3110-09-00



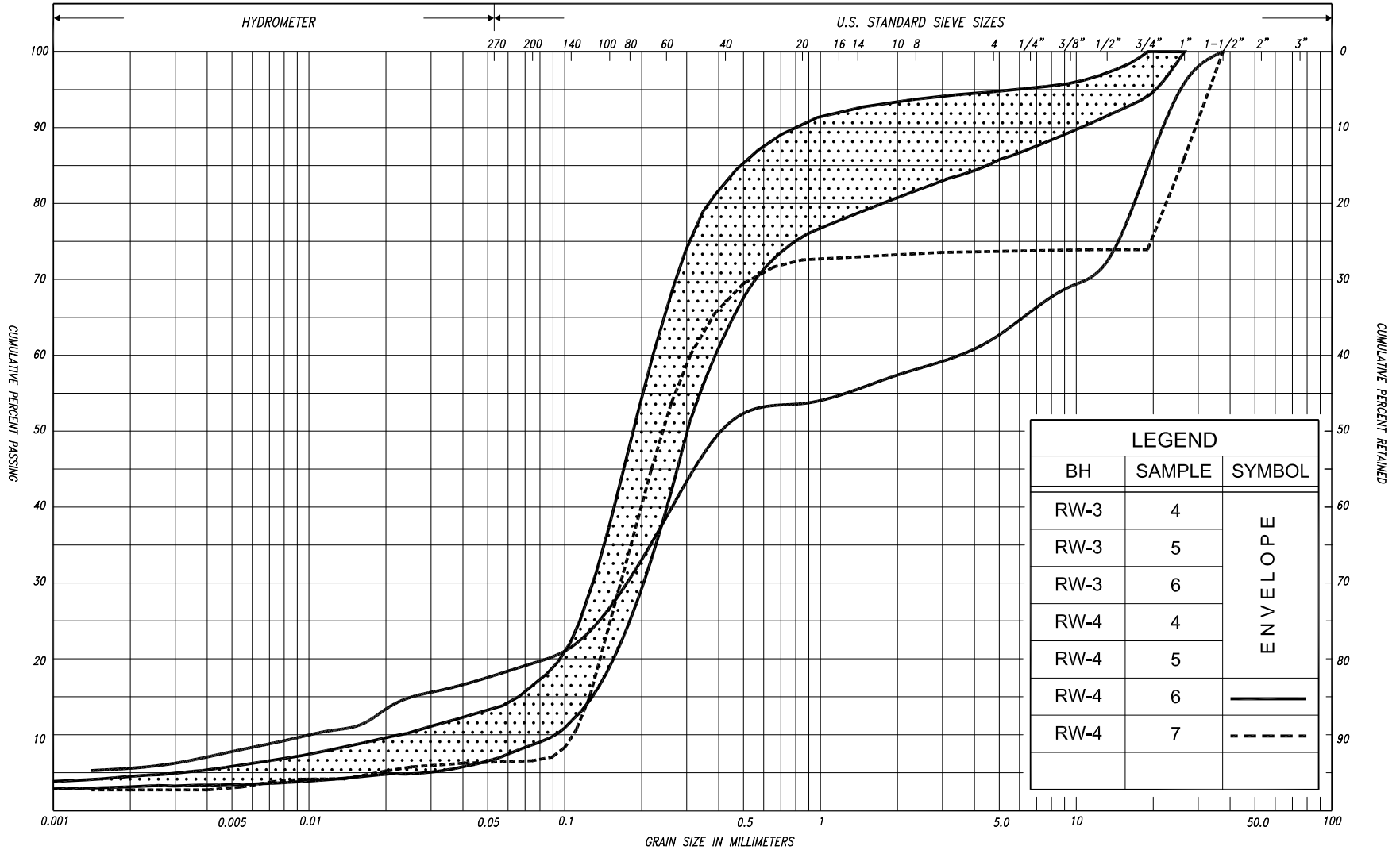
LEGEND		
BH	SAMPLE	SYMBOL
RW-3	2B	————
RW-3	3B	-----

SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COB BLES	UNIFIED		
					SAND											
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.
	SILT					SAND										
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					U.S. BUREAU	
					SAND											



GRAIN SIZE DISTRIBUTION
CLAYEY SILT, with sand, trace gravel (CI)
(FILL)

FIG No. RW-GS-5
HWY: 7 / 85
G.W.P. No. 3110-09-00



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL				COB BLES	UNIFIED		
					SAND													
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL				COBBLES	M.I.T.
	SILT																	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL							U.S. BUREAU	
					SAND													

GRAIN SIZE DISTRIBUTION

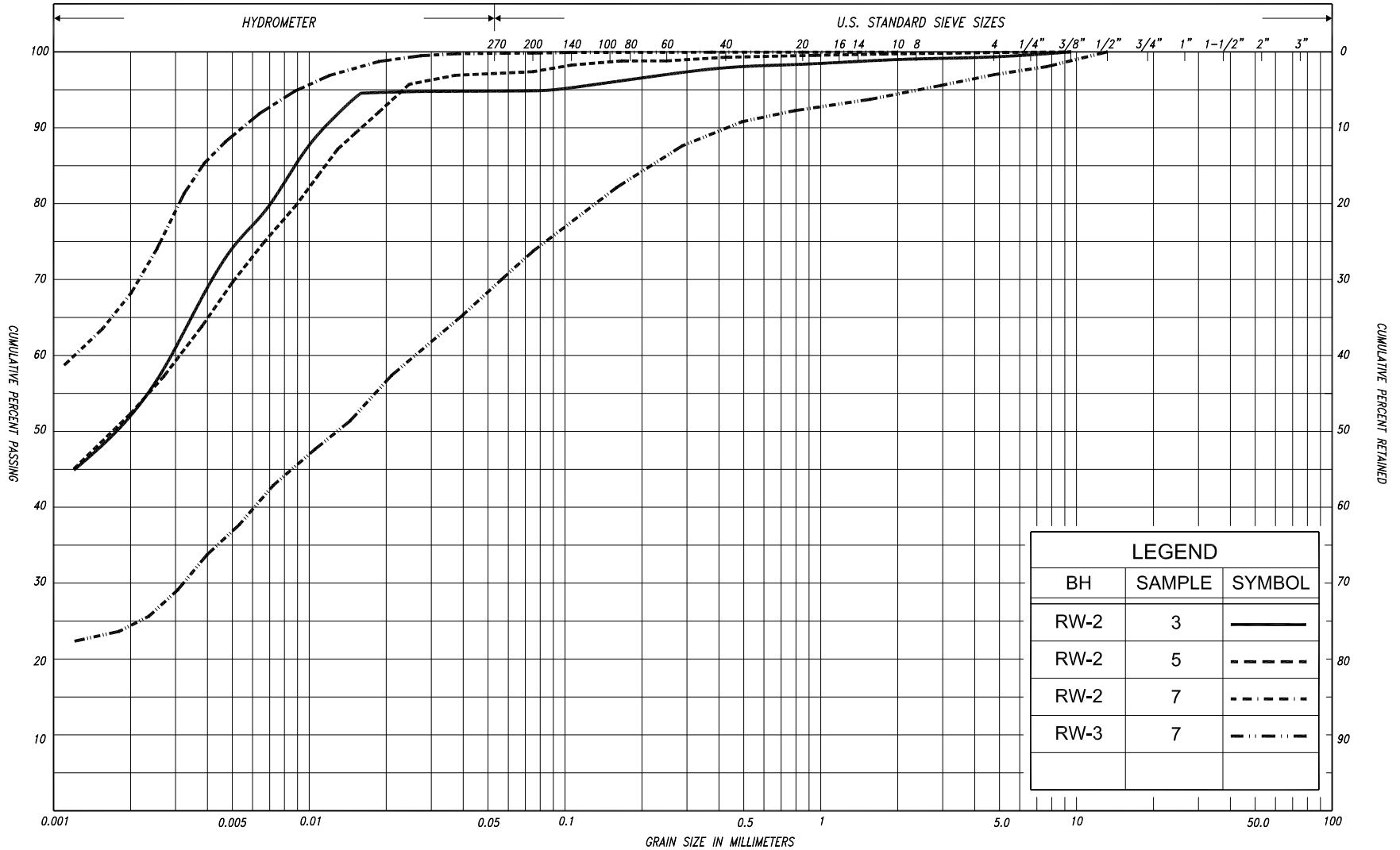
SAND, trace gravel to gravelly, trace to some silt, trace clay

FIG No. RW-GS-6

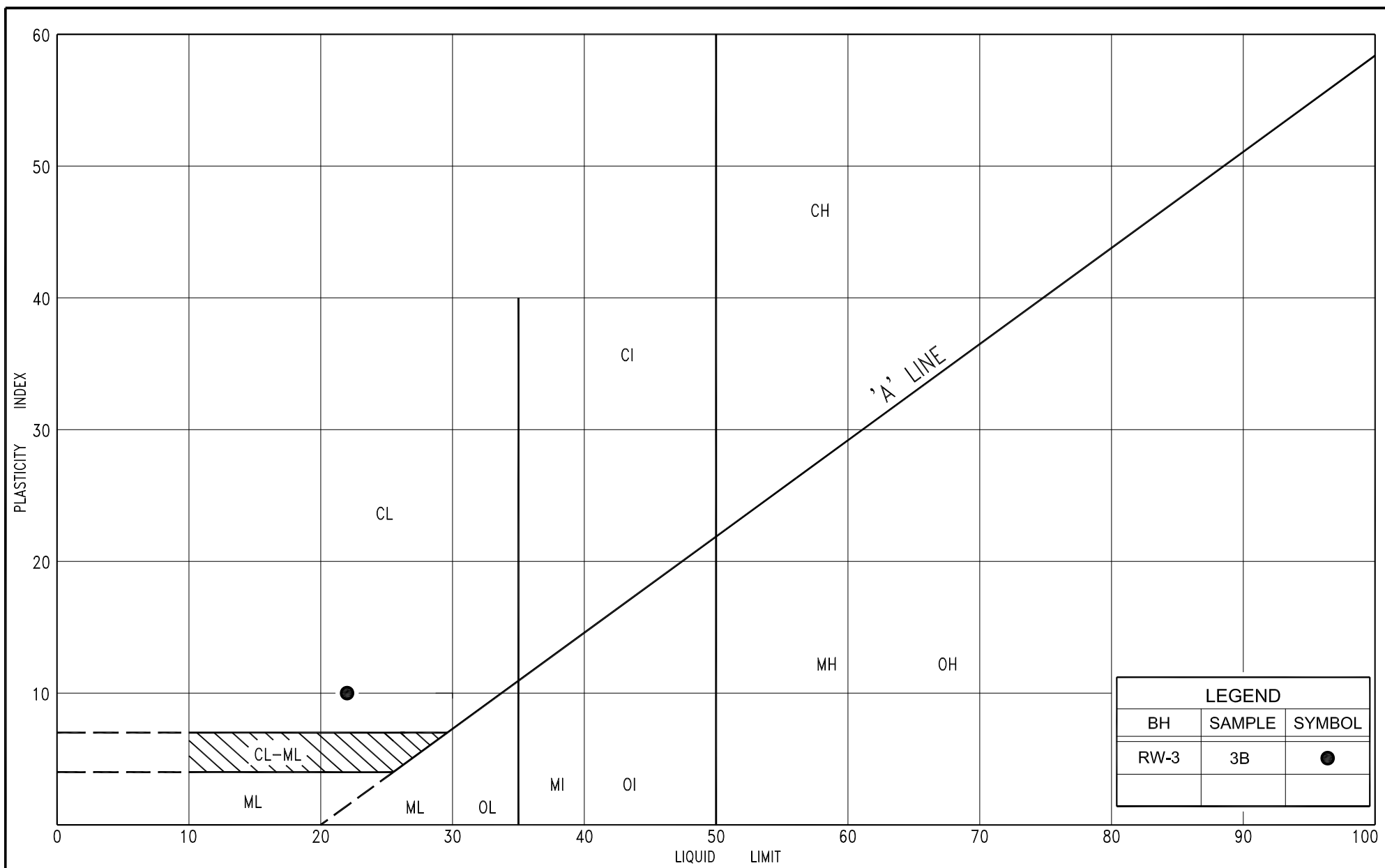
HWY: 7 / 85

G.W.P. No. 3110-09-00



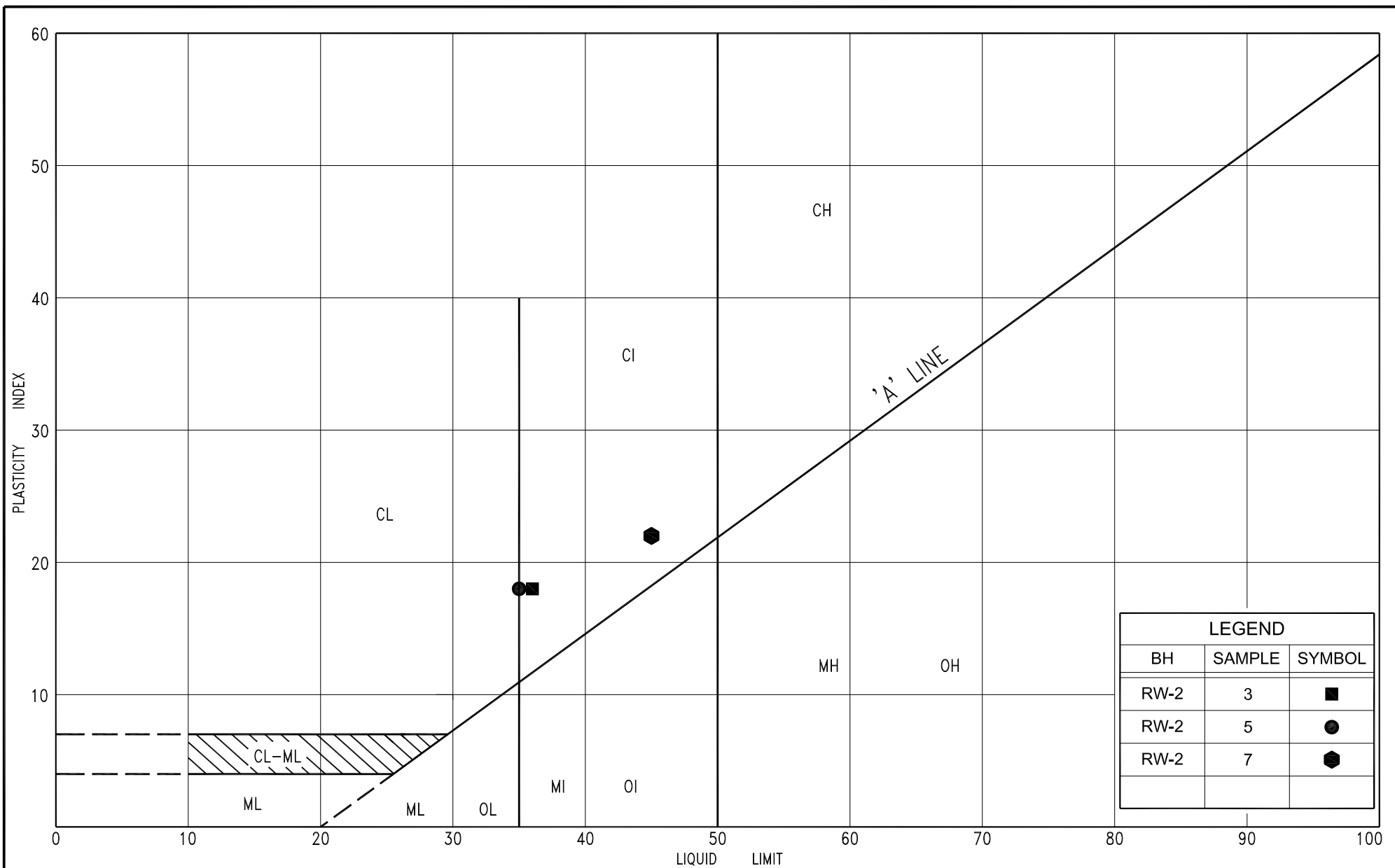


SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL				COB BLES	UNIFIED			
				SAND														
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL				COBBLES	M.I.T.
	SILT																	
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU	
						SAND												



PLASTICITY CHART
 CLAYEY SILT, with sand, trace gravel (CL)
 (FILL)

FIG No. RW-PC-1
 HWY: 7 / 85
 G.W.P. No. 3110-09-00



PLASTICITY CHART

SILTY CLAY, trace to with sand, trace gravel (CI)

FIG No. RW-PC-2

HWY: 7 / 85

G.W.P. No. 3110-09-00

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 63.5kg, 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 \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J 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.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

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

JOINTING 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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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'_{v0}	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_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No RW-1

1 of 1

METRIC

G.W.P. 3110-09-00 **LOCATION** Coords: 4 813 701.9 N; 226 222.6 E **ORIGINATED BY** R.B.
DIST London **HWY** 7/ 85 **BOREHOLE TYPE** C.F.H.S.A. and Dynamic Cone Penetration Test **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** April 08, 2011 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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										WATER CONTENT (%)		
							○ UNCONFINED + FIELD VANE												
							● QUICK TRIAXIAL × LAB VANE												
					20 40 60 80 100					20 40 60									
319.7	Ground Surface					*													
0.0	Asphalt over sand some silt, some gravel Very loose Brown Wet (FILL)		1	AS	-														
318.3			2	SS	3														
1.4	Silty clay, trace sand Very stiff Brown Moist sand layers to 4.9m Hard to Greyish very stiff brown		3	SS	17						225					(**)			
			4	SS	34						225								
			5	SS	25						225								
			6	SS	28						225								
			7	SS	37						225								
			8	SS	31						225								
			9	SS	33						225								
			10	SS	39						225								
309.9	End of borehole																		
9.8																			
	* Borehole dry (**) Base of footing -El.318.2 Note: Borehole cave-in at 8.5m C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers Water Level Readings: Date Depth Elev. Apr. 08,'11 (m) 316.8 Piezometer Legend: Bentonite seal Filter sand 19mm dia. PVC screen Bentonite grout																		

RECORD OF BOREHOLE No RW-2

1 of 1

METRIC

G.W.P. 3110-09-00 **LOCATION** Coords: 4 813 710.4 N; 226 223.0 E **ORIGINATED BY** R.B.
DIST London **HWY** 7/ 85 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** April 08, 2011 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT										PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa					WATER CONTENT (%)									
							○ UNCONFINED + FIELD VANE					○ QUICK TRIAXIAL × LAB VANE									
							20	40	60	80	100	20	40	60							
319.7	Ground Surface																				
0.0	Asphalt over sand and crushed gravel, trace silt Compact Brown Moist 																				

RECORD OF BOREHOLE No RW-3

1 of 1

METRIC

G.W.P. 3110-09-00 **LOCATION** Coords: 4 813 719.3 N; 226 229.5 E **ORIGINATED BY** F.P.
DIST London **HWY** 7/ 85 **BOREHOLE TYPE** Dynamic Ram Sounder **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** July 19, 2011 **CHECKED BY** B.R.G.

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										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
322.3	Ground Surface						20	40	60	80	100									
0.0	Silty sand some clay, trace gravel organic inclusions		1	SS	14											8 37 37 18				
	Compact Grey Moist (FILL)		2	SS	27											3 50 34 13				
	clayey silt layers															4 26 45 25				
	gravelly sand		3	SS	20											23 39 27 11				
320.0	Compact Brown Damp clayey silt layers															4 25 42 29				
2.3	Sand trace to some gravel trace clay		4	SS	21											15 76 6 3				
	Compact Brown Moist to wet		5	SS	18											10 76 10 4				
			6	SS	14											(14*) 73 12 4				
317.9	Silty clay trace sand, trace gravel silty sand and gravelly sand layers, cobbles		7	SS	36											3 23 50 24				
4.4	Hard Grey Moist		8	SS	67															
315.9			9	SS	70/15cm															
6.4	End of borehole																			
	Sample 9: Sampler bouncing																			
	 * 2011 07 19																			
	 ▽ Water level observed during drilling																			
	 (**) Base of footing -El.318.2																			
	 Water Level Readings:																			
	 Date Depth Elev. (m)																			
	July 19,'11 Dry ----																			
	Sept. 23,'11 3.3 319.0																			
	Oct. 08, '11 3.3 319.0																			
	 Piezometer Legend:																			
	 Bentonite seal																			
	 Filter sand																			
	 30mm dia. PVC screen																			
	 Filter bed																			

RECORD OF BOREHOLE No RW-4

1 of 1

METRIC

G.W.P. 3110-09-00 **LOCATION** Coords: 4 813 705.4 N; 226 228.2 E **ORIGINATED BY** A.L.
DIST London **HWY** 7/ 85 **BOREHOLE TYPE** Dynamic Ram Sounder **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** July 20, 2011 **CHECKED BY** B.R.G.

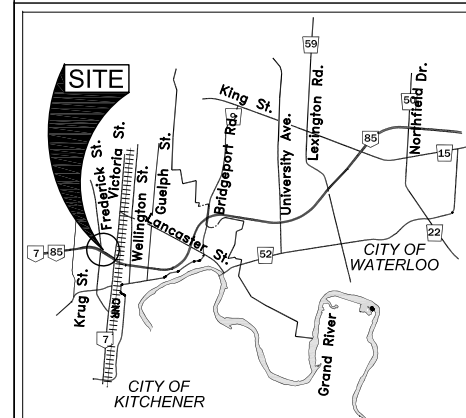
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
323.5	Ground Surface						20	40	60	80	100						GR SA SI CL	
0.0	Silty sand, some clay trace gravel, rootlets		1	SS	21		323										4 47 35 14	
	Compact Brown Moist (FILL)		2	SS	21		322										22 20 54 4	
	Silt with sand, trace gravel		3	SS	21		322										15 68 11 6	
	Compact Grey Sand, some silt some gravel, trace clay						321.2											
2.3	Compact Brown Clayey silt, trace sand																	
317.6	Very stiff Grey		4	SS	20	▽*	321										9 83 (8)	
	Sand trace to some gravel trace to some silt trace clay		5	SS	13		320										11 73 12 4	
	Compact Brown Moist to wet		6	SS	13		319										38 43 13 6	
	Gravelly to with gravel		7	SS	9		318										26 68 3 3	
			8	SS	14												(**)	
5.9	Silty clay, trace gravel cobbles		9	SS	49		317											
316.5	Stiff to Grey Moist hard		10	SS	52/15cm													
			11	SS	50/13cm													
7.0	End of borehole																	
	Samples 10 and 11: Sampler bouncing																	
	 * 2011 07 20																	
	▽ Water level observed during drilling																	
	(**) Base of footing -El.318.2																	
	Note: Borehole cave-in at 5.0m																	

CONT No
GWP No 3110-09-00



FREDERICK STREET UNDERPASS
RETAINING WALL
HIGHWAY 7/85
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 3km

LEGEND

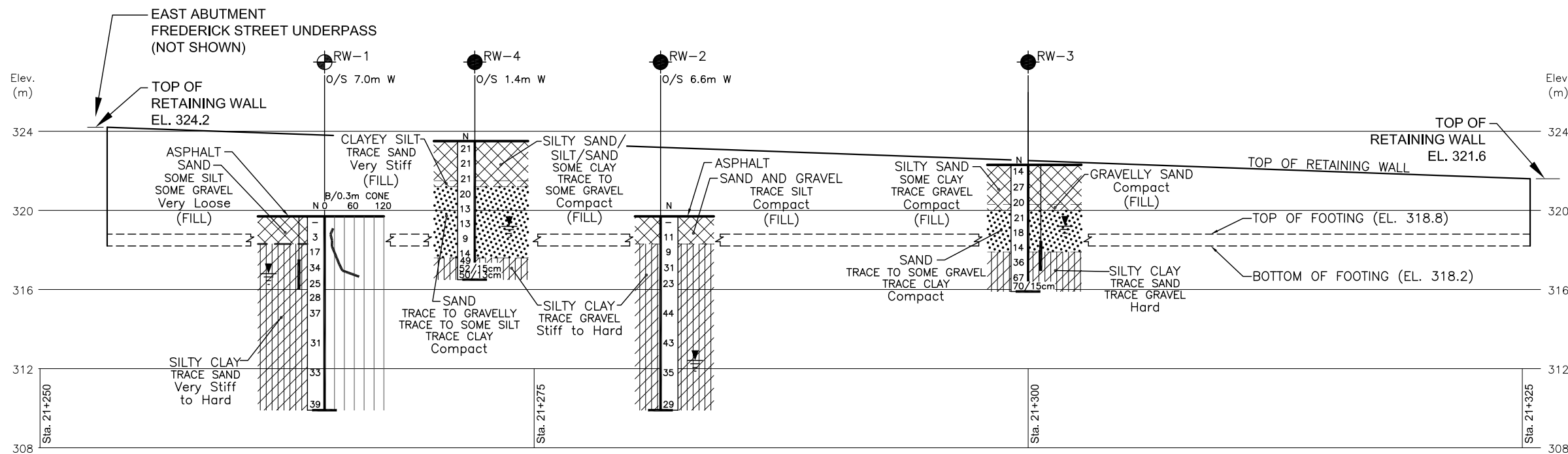
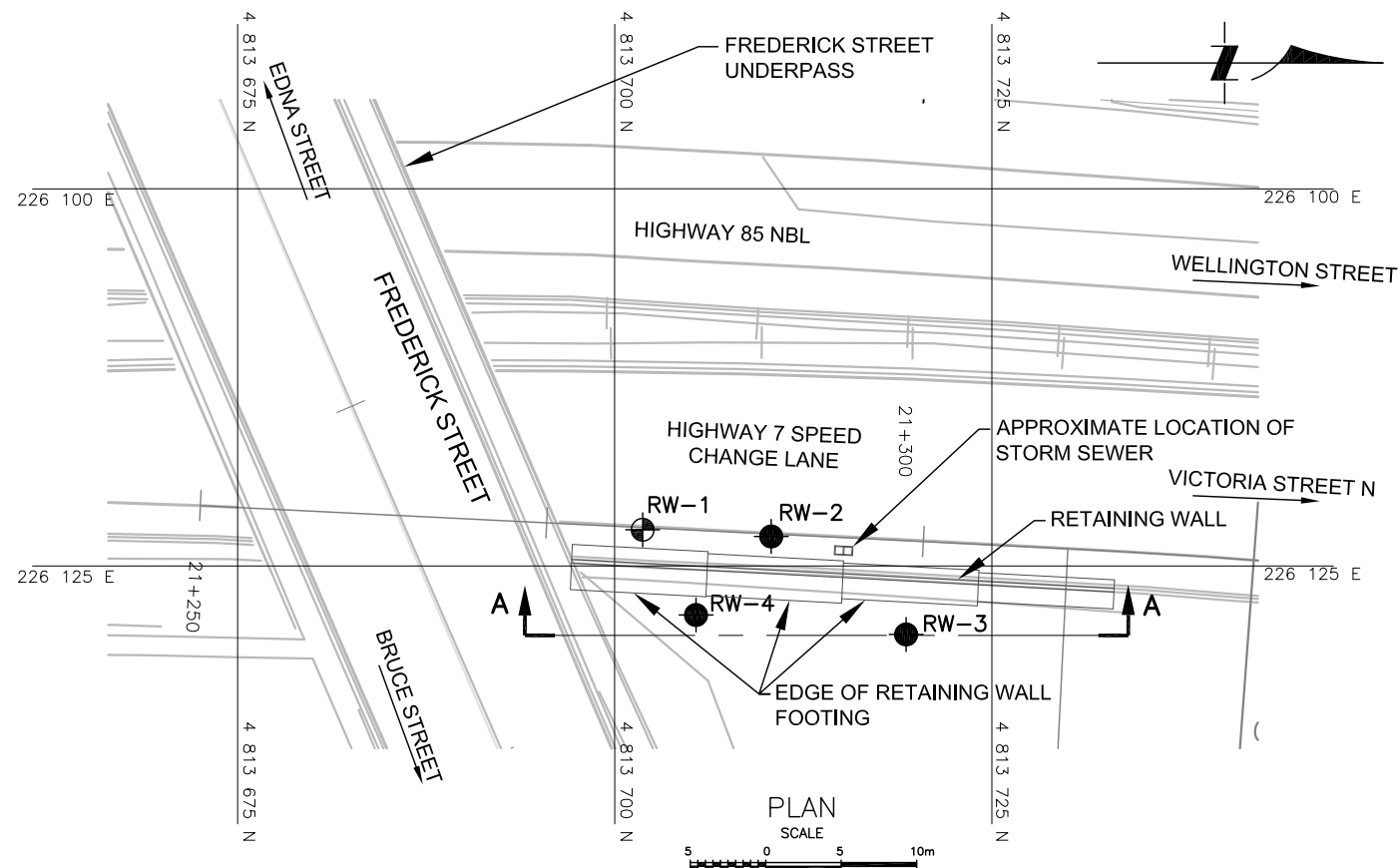
- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation April and July 2011
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
RW-1	319.7	4 813 701.9	226 222.6
RW-2	319.7	4 813 710.4	226 223.0
RW-3	322.3	4 813 719.3	226 229.5
RW-4	323.5	4 813 705.4	226 228.2

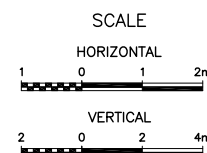
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40P8-199			
HWY No 7 / 85			DIST London
SUBM'D NA	CHECKED NSB	DATE MAY 28, 2012	SITE
DRAWN NA	CHECKED CN	APPROVED BRG	DWG RW-1



PROFILE A - A



NOTES:

- DRAWING RW-1 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND THE RECORD OF LOG OF BOREHOLES.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REF MRC Drawing: 2010362_Alignment.dwg; CONTRACT
DRAWINGS - CONTRACT No. 68-62



APPENDIX A

PREVIOUS INVESTIGATION REPORT FOR
THE FREDERICK STREET UNDERPASS
DATED JULY 1966 (GEOCRES NO. 40P8-48)

66-F-53
W.P. # 634-64
KITCHENER -
WATERLOO
EXPRESSWAY
FREDERICK ST.
UNDERPASS

GEORGE
ADP8-48

FOUNDATION INVESTIGATION REPORT

For
Frederick Street Underpass
Kitchener-Waterloo Expressway
District #4 (Hamilton)
W.J. 66-F-53 -- W.P. 634-64

1. INTRODUCTION:

A request dated April 15, 1966, for a foundation investigation at the site of the proposed crossing of Frederick Street and the Kitchener-Waterloo Expressway, was received by this office from Mr. W. S. Melinyshyn, Regional Bridge Location Engineer.

A field investigation was subsequently carried out by this Section. Presented in this report are the results of this investigation, together with recommendations pertaining to the foundation design for this structure.

2. DESCRIPTION OF SITE:

The site is located 3/4 mile west of the east boundary of Kitchener City Limits on Frederick Street. The surrounding immediate area is partially built up and the topography is generally flat. The centre-line of the proposed expressway at this point passes through a proposed cut of about 17 feet deep at a grade of elevation 1052.0.

Physiographically, the site is located in the region referred to as the "Waterloo Hills." Soils in this region are mainly well drained glacio-fluvial deposits.

3. FIELD WORK AND LABORATORY TESTING:

A total of nine sampled boreholes and sixteen dynamic cone penetration tests was carried out during the course of the

cont'd. /2 ...

3. FIELD WORK AND LABORATORY TESTING: (Cont'd.) ...

field investigation using a conventional diamond drill adapted for soil sampling purposes.

Samples were obtained using a 2" O.D. split-spoon soil sampler advanced by blows of a 140-lb. hammer falling freely a distance of 30" thus imparting an impulse of 350 ft.-lbs./blow.

The locations and elevations of all boreholes were surveyed in the field by personnel from A. D. Margison and Associates, and are shown on Drawing #66-F-53A, which accompanies this report.

Samples were visually examined in the field prior to transportation to the laboratory where they were carefully visually classified. Subsequently, combinations of the following tests were carried out on selected samples:

Atterberg Limits
Moisture Contents
Grain-Size Distributions

The laboratory test results are summarized on the borelog sheets attached to the Appendix of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

Subsoil at the site consists of stratified glacio-fluvial deposits, mostly of fine-grained composition. Detailed descriptions of the soil in each borehole are recorded in borehole logs appended to this report, together with the inferred stratigraphical profile of the area in question, shown on Drawing #66-F-53A.

From ground level downwards, the soil types encountered were as follows:

4.2) Topsoil:

Topsoil of loose sand with traces of organics, from three to six feet in depth, was found to overlies the site area adjacent to the existing road.

cont'd. /3 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Sand with traces of Silt:

The depth of this deposit ranges from about 8 to 22 feet. The material consists mostly of sand (90%) with traces of silt (10%). 'N' values obtained from Standard Penetration tests, ranged from 9 to 47 blows/ft., indicating a loose to dense relative density.

4.4) Clayey Silt:

This deposit of grey clayey silt of low plasticity was found in B.H. No's 3, 6 and 7. 'N' values obtained by Standard Penetration tests, ranged from 16 to 34 blows/ft., indicating a very stiff to hard consistency. Tests for moisture content and Atterberg limits gave the following average values: moisture content = 16%, plastic limit = 15%, liquid limit = 20%. Based on the foregoing, the shear strength of this deposit is estimated to range from 2,000 to 4,500 p.s.f.

4.5) Fine Sandy Silt to Silty Fine Sand:

This deposit has an average moisture content of 17%. Standard Penetration tests gave 'N' values ranging from 29 to 157 blows/ft., indicating a relative density of dense to very dense.

4.6) Clayey Silt with some Sand and Gravel:

This brownish-grey deposit was found in most of the boreholes with a varying thickness from 3 feet in B.H. #10 to 10 feet in B.H. #6. Mechanical analyses indicate the following average grain-size distribution: gravel 10%, sand 27%, silt 45%, clay 17%. Standard Penetration tests gave 'N' values ranging from 13 to 70 blows/ft., indicating a stiff to hard consistency. The results of Atterberg limit tests showed this soil to fall mainly in the classification of CL (clayey silt). The results of these tests are summarized as follows:

cont'd. /4 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.6) Clayey Silt with some Sand and Gravel: (cont'd.) ...

	<u>Minimum</u>	<u>Maximum</u>	<u>Average</u>
Plastic Limit	10.6%	19.8%	14.5%
Liquid Limit	17.7%	49.1%	28.8%
Moisture Content	8.6%	22.4%	14.9%

The shear strength of this deposit is estimated to range from 2,000 to more than 4,000 p.s.f.

4.7) Silty Clay:

This silty clay of medium plasticity, was found in every borehole. The material consists of silty clay, brownish-grey in colour. It is similar to the clayey silt deposit described in Section 4.4, except with a higher plasticity and does not have the sand and gravel content. Average grain-size distribution indicated by mechanical analyses are: sand 1%, silt 48%, clay 51%. 'N' values obtained from Standard Penetration tests, ranged from 41 to 130 blows/ft., indicating a hard consistency. The results of Atterberg limit tests, showed that this deposit is classified as silty clay (CI) on the Plasticity Chart; the summary of the tests are as follows:

	<u>Minimum</u>	<u>Maximum</u>	<u>Average</u>
Plastic Limit	15.4%	22.2%	19.5%
Liquid Limit	32.5%	55.5%	43.7%
Moisture Content	14.5%	26.0%	20.7%

The shear strength of this deposit is estimated to be more than 4,000 p.s.f.

4.8) Silty Fine Sand to Fine Sandy Silt:

This deposit was found to underlie the silty clay deposit in every borehole, having an 'N' value of more than 99 blows/ft.,

cont'd. /5 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.8) Silty Fine Sand to Fine Sandy Silt: (cont'd.) ...

and an average moisture content of 13.7%. These results indicate that this material has a very dense relative density.

4.9) Silty Clay:

This brownish-grey silty clay of medium plasticity, is found below the silty fine sand to fine sandy silt described in Section 4.6, in B.H. #2 and B.H. #4 only. The index properties are: plastic limit = 17.4%, liquid limit = 36.6%, moisture content = 17.5%. The consistency of this material is hard, as indicated by 'N' values ranging from 46 to 130 blows/ft., given by Standard Penetration tests. Based on the foregoing, the shear strength of this deposit is estimated to be more than 4,000 p.s.f.

5. GROUNDWATER:

Groundwater levels at the time of field investigation were found to range from El. 1054.2 in B.H. #14 to El. 1059.4 in B.H. #2.

8. MISCELLANEOUS:

The field work for this project was carried out during the period May 26 to June 6, 1966, under the supervision of Mr. D. T. Wan, Project Foundation Engineer, who also prepared this report. Mr. K. G. Selby, Supervising Foundation Engineer, generally supervised the entire project and reviewed this report. The equipment used was owned and operated by Dominion Soil Investigation Limited.

July 1966

APPENDIX I

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200.887.525, E 210.709.963 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE May 26, 1966. COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring. CHECKED BY K.G.S.

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	BLOWS / 100T	WATER CONTENT	WATER CONTENT %		
1072.4	Groundlevel										
0.0	Sand (topsoil)										
1068.4	Loose		1	SS	8						Sa 80%
4.0			2	SS	24						Si 20%
	Sand		3	SS	20						G. W. L.
	occasional trace of silt.		4	SS	25						$\frac{V}{V} K_1$
	Compact.		5	SS	10						$\frac{V}{V} 1059.4$
			6	SS	30						Sa 96% Si 4%
1052.4			7	SS	28						
20.0	Clayey silt with some sand and gravel		8	SS	15						Gr 14%
	Stiff to hard.		9	SS	57						Sa 27%
1045.4			10	SS	71						Si 44%
27.0	Silty clay		11	SS	44						Cl 15%
	Hard		12	SS	194						
	Brownish grey.		13	SS	88						
1028.4			14	SS	100/11"						
44.0			15	SS	85/74"						
	Fine sandy silt to silty fine sand.										
	Very dense.										
1013.4			16	SS	120						
59.0			17	SS	92						
	Silty clay										
	Hard										
	Brownish grey.										
988.9			18	SS	46						
83.5	End of borehole.		19	SS	130						

FOUNDATION SECTION

ORIGINATED BY D.W.

COMPILED BY D.W.

CHECKED BY K.G.S.

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F.	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — w wp — w — WL WATER CONTENT %	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE					
1069.3 0.0	Groundlevel								
1044.5 24.8	End of borehole.								

FOUNDATION SECTION

CHECKED BY K.G.S. 8/2

[illegible]

FOUNDATION SECTION

Sa 28%
S1 72%

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200.939.749, E 210.847.402 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE June 6, 1966. COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Penetration. CHECKED BY K.G.S.

[illegible]

FOUNDATION SECTION

ORIGINATED BY D.W.

COMPILED BY D.W.

CHECKED BY K.G.S.O.

8a	5%
81	95%

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200,883.088, E 210,947.710 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE May 31, 1966 COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring CHECKED BY K.G.S.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	W P	W L	W		
1068.2	Groundlevel															
0.0	Sand (Topsoil)															
1063.2	Loose		1	SS	6											
5.0	Sand with trace of silt.		2	SS	13											
1057.2	Compact.		3	SS	23											
11.0	Silty fine sand.		4	SS	47											
1054.2	Dense															
14.0	Clayey silt with some sand and gravel.		5	SS	32											
	Hard.		6	SS	41											
1045.2			7	SS	57											
23.0	Fine sandy silt		8	SS	70											
	Very dense		9	SS	100											
1039.2																
29.0			10	SS	132											
			11	SS	131											
	Silty clay.		12	SS	62											
	Hard.		13	SS	64											
	Brownish gray.		14	SS	59											
1009.2																
1007.2	Fine sandy silt. Very dense.		15	SS	62 1/2											
61.0	End of borehole.															

Sa 83%
Si 17%
OML E1.
▽ 1055.7

Sa 25%
Si 75%

Sa 1%
Si 54%
Cl 45%

Sa 26%
Si 74%

50/3 1/2"

FOUNDATION SECTION

CHECKED BY K.G.S.

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F.	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W Wp — WL WATER CONTENT %	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE					
1067.0 0.0	Groundlevel								
1044.4 22.6	End of borehole.					100/7"			

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

JOB 66-P-53 LOCATION N 200.997.938, E 210.999.324 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE June 2, 1966 COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring CHECKED BY K.G.S.

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	20	40	60	80	100	WP	WL		
1067.2	Groundlevel														
0.0	Sand (Topsoil)														
1062.2	Compact		1	SS	11										
5.0			2	SS	14										
	Sand with trace of silt.		3	SS	18										
			4	SS	15										
	Compact.		5	SS	15										
			6	SS	17										
			7	SS	20										
			8	SS	28										
1041.2	Silty clay with trace of sand. Hard.		9	SS	130										
26.0			10	SS	100										
1038.2			11	SS	62										
29.0	Sand with some silt. Very dense.		12	SS	93										
1026.2			13	SS	47										
41.0	Silty clay. Hard.		14	SS	109										
1015.2	Fine sandy silt. Very dense.		15	SS	120										
52.0															
1006.2	Silty clay Hard														
61.0	Brownish gray														
1000.7	End of borehole.														
66.5															

Sa 89%
Si 11%
GWL El.

1054.2

Sa 93%
Si 7%

Gr 2%
Sa 97%
Si 1%

100/11"

321.5

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 16

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 201,116.543, E 210,741.917 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE June 3, 1966 COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring CHECKED BY K.G.S.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — WL PLASTIC LIMIT — WP		BULK DENSITY P.C.F.	REMARKS
E. EV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	WL	
1065.0	Groundlevel													
0.0	Sand					1060								
	Compact		1	SS	17									
1056.0			2	SS	26									
9.0			3	SS	62	1050								
	Clayey silt to silty clay with some sand and gravel.		4	SS	111									
	Very stiff to hard		5	SS	126									
	Brownish grey.		6	SS	87	1040								
1036.0			7	SS	85									
29.0			8	SS	83									
	Silty clay		9	SS	39	1030								
	Hard		10	SS	105									
	Brownish grey.		11	SS	9376	1020								
1024.0														
41.0														
	Silty fine sand.													
	Very dense													
1008.5			12	SS	116	1010								
56.5	End of borehole.					1000								

100/8"

Gr 7%
Sa 28%
S1 46%
Cl 19%

Sa 2%
S1 41%
Cl 57%

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 17

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200,754.293, E 210,935.966 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE June 2, 1966 COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring CHECKED BY K.G.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	20	40	60	80	100	WATER CONTENT %		
1069.6	Groundlevel														
0.0	Sand		1	SS	9										
	Loose to v. dense		2	SS	70	1060									
			3	SS	28										
1054.6	Clayey silt		4	SS	34										
15.0	Very stiff to hard.		5	SS	22	1050									
1049.2	Fine sandy silt, v. dense		6	SS	27										
20.5			7	SS	53										
1046.6	Clayey silt to silty clay.		8	SS	150	7"									
23.0	Hard.		9	SS	60	1040									
	Brownish grey.		10	SS	88										
			11	SS	76	1030									
1024.5	ve sandy silt.		12	SS	68	1020									
4.6	very dense														
1013.8			13	SS	50	1010									
55.8	End of borehole.														

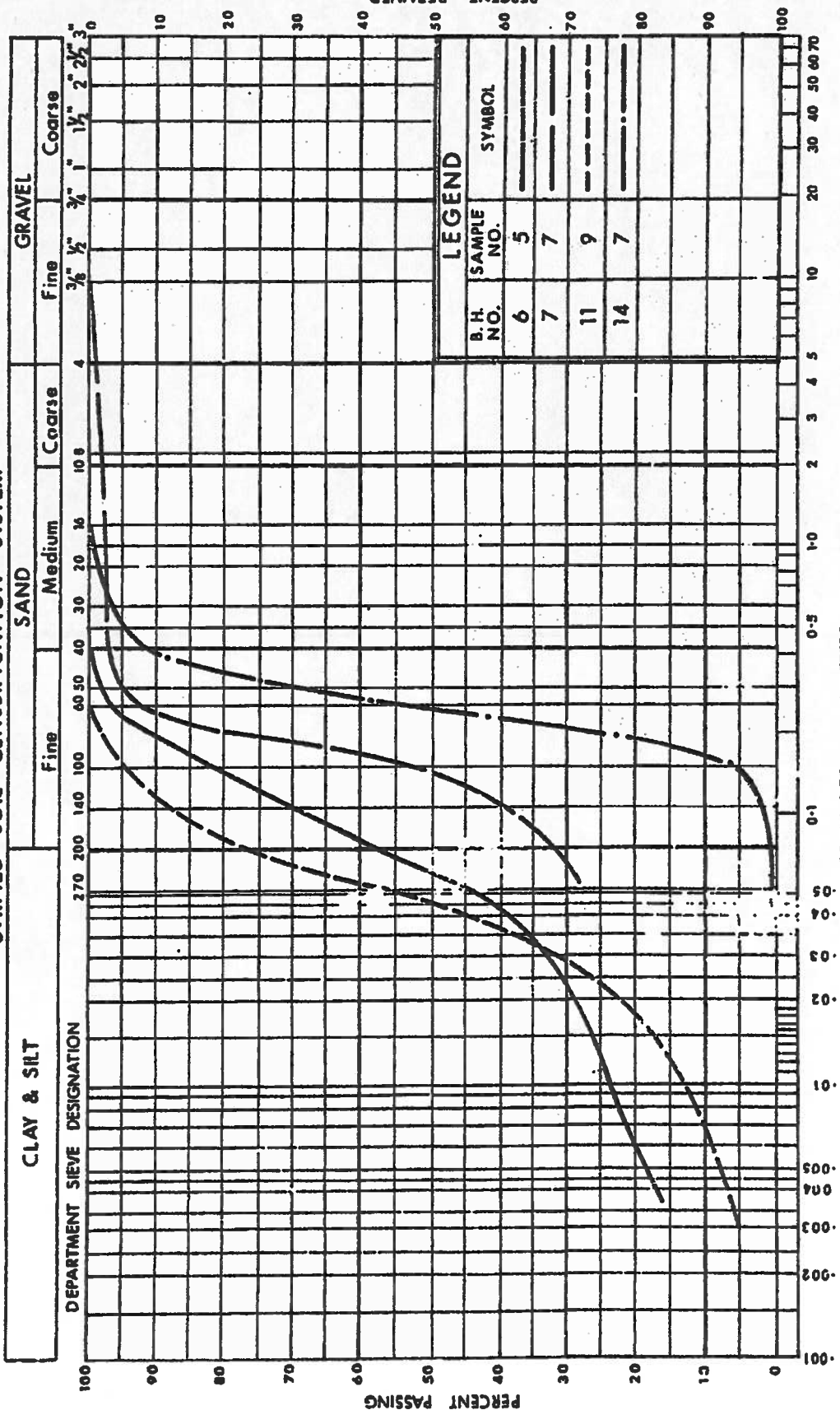
Sa 91%
Si 9%
GWL
V. EL.
1057.8

Sa 1%
Si 80%
Cl 19%

Sa 28%
Si 72%

100/7"

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION



GRAIN SIZE DISTRIBUTION

W.P. No. 634-64

JOB No. 66-F-53

ONTARIO

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta \sigma'}$
C_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_a	APPARENT COHESION
ϕ_a	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_r	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

**A. D. Margison and Associates Limited
Consulting Professional Engineers**

**W.P. 634-64, Kitchener-Waterloo Expressway
Frederick Street Underpass
Project No. 2121**

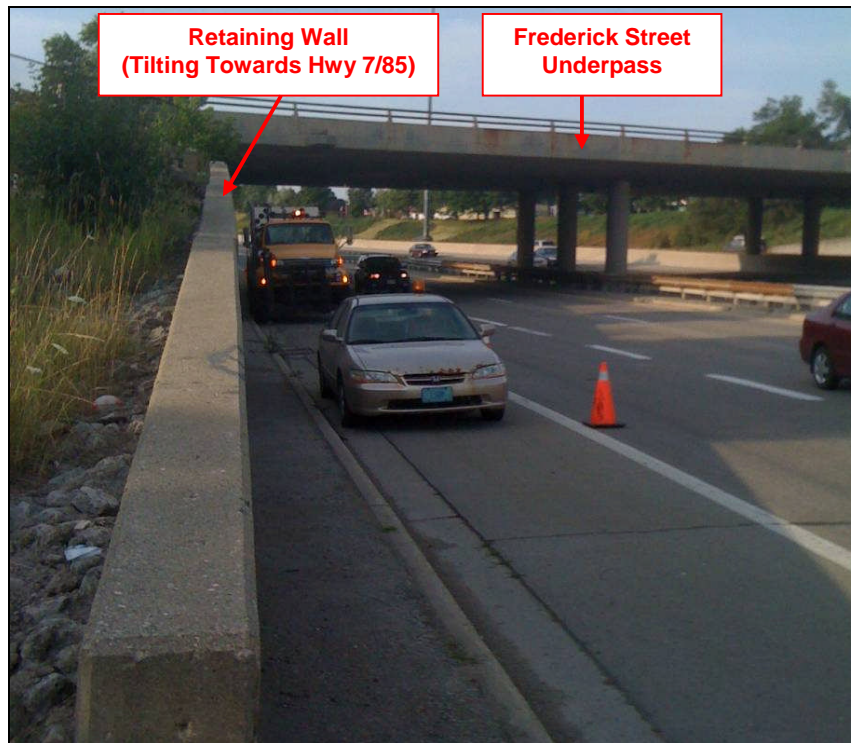
Bore Hole No.	Elevation	North Coordinates	East Coordinates
1	1072.73	200,786.159	210,719.093
3	1071.88	200,803.248	210,761.731
5	1069.78	200,821.407	210,811.176
7	1069.06	200,844.706	210,861.156
9	1069.03	200,873.400	210,912.942
11	1068.15	200,883.088	210,947.710
13	1066.95	200,914.762	211,000.220
17	1069.586	200,754.293	210,935.966

2	1072.41	200,887.529	210,709.963
4	1069.34	200,890.443	210,747.672
6	1067.89	200,919.964	210,802.832
8	1068.09	200,939.749	210,847.402
10	1067.91	200,955.200	210,895.815
12	1067.55	200,967.693	210,949.158
14	1067.23	200,997.938	210,999.324
16	1064.97	201,116.543	210,741.917

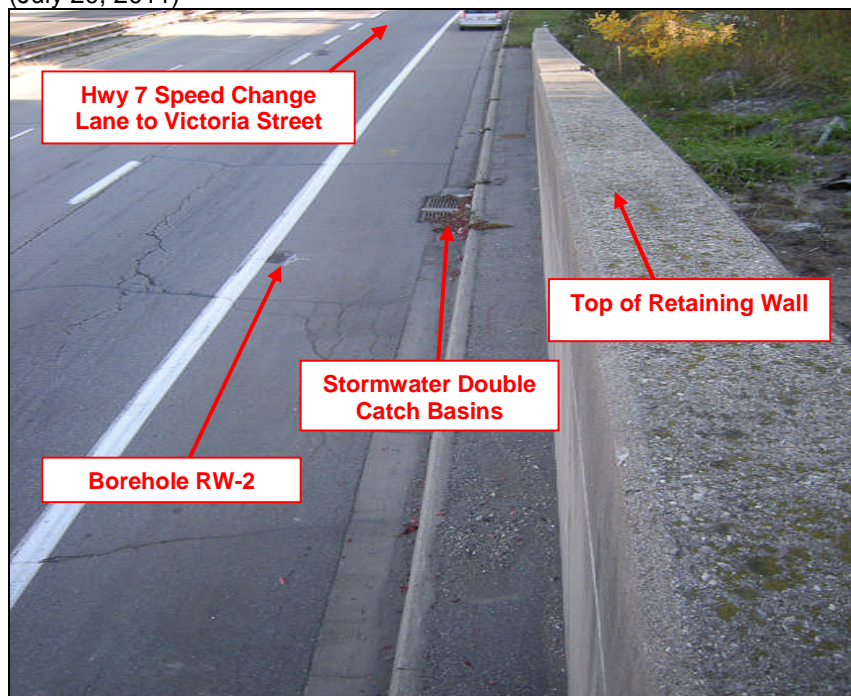


APPENDIX B

SITE PHOTOGRAPHS 1 to 10



Photograph 1: Looking south from north end of the retaining wall.
(July 20, 2011)



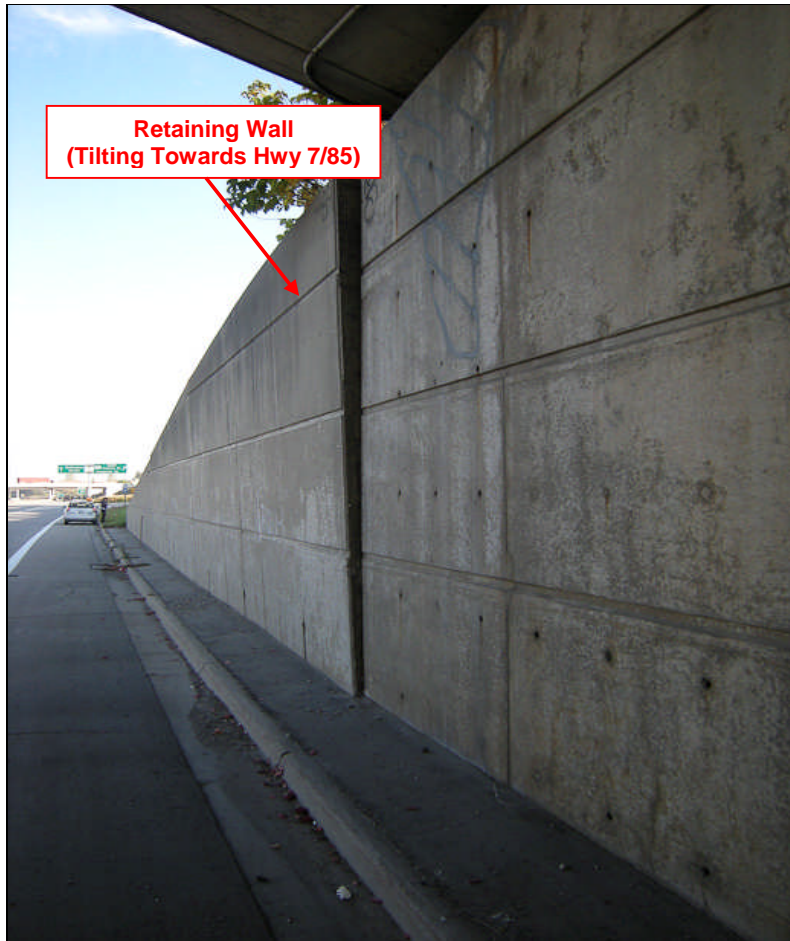
Photograph 2: Looking north from south portion of retaining wall. A double catch basin is located about 14 m north from south end of the retaining wall. (October 8, 2011)



Photograph 3: Looking north from borehole RW-4. Drill rig (Dynamic Ram Sounder) at borehole RW-4. (July 20, 2011)



Photograph 4: Looking south from approximately 3 m north of borehole RW-3. (July 20, 2011)



Photograph 5: Looking north from east shoulder under the Frederick Street underpass. (October 8, 2011)



Photograph 6: Close-up view of south end of retaining wall. Measurement to face of abutment was taken at retaining wall/abutment joint, approximately 64 mm (2.5 in.) at 0.3 m (1 ft.) above ground surface, 127 mm (5 in.) at 1.8 m (6 ft.) and 230 mm (9 in.) at top of retaining wall. (October 8, 2011)



Photograph 7: Close-up view from top of retaining wall at abutment. (October 8, 2011)



Photograph 8: Overhang measurement was taken from approximately 7 m north of south end of the retaining wall and it was approximately 152 mm (6 in.). (October 8, 2011).



Photograph 9: Overhang measurement was taken from approximately (at double catch basin) 14 m north of south end of the retaining wall and it was approximately 114 mm (4.5 in.). (October 8, 2011)



Photograph 10: Overhang measurement was taken from approximately 20 m north of the south end of the retaining wall and it was approximately 41 mm (1 5/8 in.) (October 8, 2011)



**FOUNDATION DESIGN REPORT
for
NORTHEAST CORNER RETAINING WALL
FREDERICK STREET UNDERPASS
SITE NO. 33-234
G.W.P. 3110-09-00
CITY OF KITCHENER, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 3 cc: McCormick Rankin Corporation (MRC) for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 3 cc: Foundation Investigation Report only to MRC for
distribution to MTO Project Manager – West Region
(London) + 1 digital copy (pdf)
- 1 cc: MRC for distribution to MTO, Pavements and
Foundations Section + 1 digital copy (pdf) and
Drawing (AutoCAD)
- 1 cc: Foundation Investigation Report only to MRC for
distribution to MTO, Pavements and Foundations
Section + 1 digital copy (pdf) and Drawing (AutoCAD)
- 2 cc: MRC +1 digital copy (pdf)
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 10KF079C
Index No.: 119FDR
GEOCRES No.: 40P8-199
May 31, 2012



TABLE OF CONTENTS

1. INTRODUCTION	1
2. REVIEW OF CONSTRUCTION DRAWINGS	1
3. SUBSURFACE FOUNDING AND BACKFILL CONDITIONS	2
4. STABILITY ANALYSES	3
4.1 General	3
4.2 Sliding Resistance.....	6
4.3 Overturning Moment	7
4.4 Bearing Resistance	7
4.5 Global Stability	7
5. SUMMARIZED DISCUSSION AND CONCLUSIONS	8
5.1 Sliding Resistance Check.....	8
5.2 Overturning Resistance Check.....	8
5.3 Bearing Resistance Check	9
5.4 Global Stability Check	9
5.5 Loss of Ground Support	9
5.6 Conclusions	10
6. REMEDIAL MEASURES	10
6.1 General Considerations	10
6.2 Remediation Maintaining Tilted Wall Condition	11
6.2.1 Tieback Soil Anchors	12
6.2.2 Horizontal Helical Anchors.....	12
6.2.3 Underpinning with Grout Concrete	13
6.2.4 Underpinning with Helical Piles or Micropiles.....	14
6.3 Remediation with Improved Wall Condition	14
6.4 Monitoring Program.....	17
7. CLOSURE	18



Figure 1 – Slope Stability Diagram

Figure 2 – Underpinning Procedure

Appendix FDR-1 – Excerpt of Construction Drawings, Contract No. 68-62 (Excerpts Only)

Appendix FDR-2 – Computations

Appendix FDR-3 – NSSP for Supply and Installation of Retaining Wall Monitoring Instrumentation and Equipment

Appendix FDR-3A – Portable Digitilt Tiltmeter

Appendix FDR-4 – Geotechnical (Foundation Specialty) Monitoring Plan – Northeast Corner Retaining Wall at Frederick Street Underpass

FOUNDATION DESIGN REPORT
for
Northeast Corner Retaining Wall
Frederick Street Underpass, Site No. 33-234
GWP 3110-09-00
City of Kitchener, Ontario

1. INTRODUCTION

This foundation design report provides our assessment of potential causes of the movement and recommendations for the remediation / stabilization of the Frederick Street Underpass concrete retaining wall located at the northeast corner of the interchange structure. The retaining wall has experienced progressive movement (tilting) towards the Highway 7 / 85 (Kitchener-Waterloo Expressway). This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

This foundation design report does not include conclusions of a structural nature related to the condition of the retaining wall and its footing. This analysis was not within the scope of this investigation and analysis.

The foundation frost penetration depth at the site is 1.4 m according to OPSD 3090.101.

2. REVIEW OF CONSTRUCTION DRAWINGS

A review of construction drawings numbered D-6049-1, D-6049-2, D-6049-12 and D-6049-13 for the 1968 Contract No. 68-62 (attached in Appendix FDR-1) that are relevant to the subject retaining wall design and construction was carried out. The following notes were compiled.

- The south elevation detail on Drawing D-6049-1 indicates that a minimum 4.7 m cut was carried out below the original ground surface for the underpass construction.
- The bottom of footing of the subject northeast retaining wall (Drawing No. D-6049-12) was designed at estimated elevation 318.2.
- A subdrain pipe was designed above the top of the footing with invert at elevation 319.1 at the south and north ends, sloping down to the middle of the wall about 150 mm (0.5 ft.) for a slope of approximately 0.6%.
- A 150 mm (6 in.) diameter CSP was designed to drain the subdrain to a new double catch basin to be placed in front of the retaining wall.
- From Drawings D-6049-12 and D-6049-13, it can be seen that the wall was planned to be constructed in four panels each approximately 9.1 m (30 ft.) long.



3. SUBSURFACE FOUNDING AND BACKFILL CONDITIONS

Based on the results of previous 1966 foundation investigations for the design of Frederick Street Underpass and associated retaining walls, the local stratigraphy includes a deposit of compact cohesionless sand overlying very stiff to hard clayey soils. At the northeast retaining wall location, the sand deposit was about 3 to 6 m thicker (as noted in borehole 14) than other locations at the Frederick Road Underpass and extended to approximately elevation 317.9.

In summary, the current investigations revealed embankment fill to approximately elevation 318.3 over stiff to hard silty clay in front of the retaining wall. Behind the wall, about 2.3 m thick fill (backfill material for retaining wall) overlies 2.1 and 3.6 m thick cohesionless compact sand deposits extending to elevation 317.9 and 317.6. The sands are underlain by stiff to hard silty clay.

Based on current and previous investigation data, it is inferred that the northeast retaining wall appears to be founded on the cohesionless compact sand layer less than 1 m thick overlying very stiff to hard silty clay.

Groundwater was found at about elevation 321.5 in 1966 and was at about elevation 319.1 in 2011. Groundwater is subjected to seasonal fluctuations. It is noted that the current groundwater level behind the retaining wall coincides with the design invert level of the subdrain, indicating that the drainage pipe was operating adequately at the time of the investigation.

The current investigation indicated that the retaining wall was backfilled with heterogeneous materials including silts, silty sands and clayey silts containing typically over 38% of fines (silt and clay particles) extending to about 2.3 m depth, elevation 320.0 and 321.2. These layers are about 1.7 m thicker than the specified 0.6 m thick layer of earth backfill (clayey soils) indicated on Drawing No. D-6049-19.

In view of the amount of fines in excess of the typically higher limit of 8% for non-frost susceptible soils such as OPSS Granular A or Granular B Type I materials, the existing backfill is considered to be frost susceptible. Where moisture is present frost susceptible backfill materials will apply lateral pressure (ice jacking) on the back of the retaining wall during the winter months.



It is understood from MRC further to the CCTV camera survey of the storm sewer in December 2011, that the pipe is in acceptable condition and that sand was not found inside the pipe indicating that a breach has not occurred in the past.

4. STABILITY ANALYSES

4.1 General

The review of the analysis results was carried out for the factored resistances at ULS in view of the excessive magnitude of the retaining wall movement which indicated that the geotechnical resistances at SLS were exceeded.

The stability analysis was carried out for the factored resistances at ULS using a calculation spreadsheet provided by MRC.

Stability analyses were carried out for the retaining wall panel adjacent to the bridge abutment for sliding, overturning and bearing resistance and for the global stability using limit equilibrium methods. The average height of this retaining wall panel was considered for the analysis. It is also considered that the analysis of this selected highest panel is representative of the remaining wall panels.

The lateral earth and water pressure, p (kPa), for the analysis were computed using the equivalent fluid pressures presented in Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC) generally employing the following equation and assuming a triangular pressure distribution.

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p + C_s$$

where p = lateral earth pressure (kPa)
 K = lateral earth pressure coefficient
 γ = unit weight of backfill material above design water level (kN/m³)
 γ' = unit weight of submerged backfill material below design water level (kN/m³)
 = $\gamma - \gamma_w$
 γ_w = unit weight of water
 = 9.8 kN/m³
 h_1 = depth below final grade (m), above design water level
 h_2 = depth below design water level (m)
 q = any surcharge load (kPa)
 C_p = compaction pressure (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where ϕ = angle of internal friction of retained soil (30° for backfill material)
 δ = angle of friction between soil and wall (17° for backfill material)



The following parameters are used /assumed for the analyses:

PARAMETER	BACKFILL MATERIAL
Angle of Internal Friction, degrees	30
Unit Weight, kN/m ³	20.0
Coefficient of Active Earth Pressure (K_a)	0.45 to 0.47
Coefficient of Passive Earth Pressure (K_p)	3.0

The coefficient of Active Earth Pressure (K_a) varies with the angle of tilting of the retaining wall assumed in the analysis. The value of 0.45 was used for the as-built condition (tilt angle of 0°), while 0.47 was computed for the present condition (tilt angle of 2.4°).

For the analysis, the inclination of the cut slope behind the wall was assumed to be 3H:1V (18.4°). A friction factor of 0.65 was considered for the cast-in-place concrete footings. A compaction pressure (C_p) of 12 kPa was utilized for the analysis.

Analysis of earth retaining structures also typically included earth pressure forces (C_s) induced by seismic / earth quake loading. However, since the tilting was progressive and seismic events were not common in the area of the retaining wall, only a static loading was considered in the analyses.

Ice jacking was considered in the analysis due to the frost susceptible backfill materials encountered behind the wall. This ice jacking develops progressively with decreasing temperatures during the winter months. The existing earth slope behind the wall drains toward the top of the wall and supplies moisture to the backfill, increasing the probability of developing ice jacking forces. It is also noted that the backfill includes silty soils which are relatively more pervious than desired to create an impervious barrier. A relatively less pervious clayey silt layer was present but below the silty layers where it is not effective to minimize water infiltration.

The developed ice jacking acts on the retaining wall as an additional resulting force which may not have been considered during the design of the retaining wall. The ice jacking pressures may vary



from 10 to 1800 kPa depending on the lateral restriction of the structure based on published data¹ and ².

To assess the effect of ice jacking on the back of the retaining wall for this report, an ice jacking pressure was assumed at 10 and 20 kPa, respectively with a rectangular distribution from the surface to the 2.3 m depth of frost susceptible materials.

As previously indicated, the retaining wall geometry used for the analyses was taken from construction Drawings D-6049-12 and D-6049-13, Contract No. 68-62.

The analyses were carried out for the 1968 as-built condition of the retaining wall and also considering ice jacking pressures of 10 and 20 kPa. In addition, the analyses were also conducted for actual wall inclination of 2.4° and also with ice jacking pressures and 10 and 20 kPa. Copies of the computations were included in Appendix FDR-2. The results of the analyses are summarized in the following table.

The retaining wall stability was also assessed for further tilting of the wall when the rotation angle increases from the current rotation angle of 2.4° to 2.6°, 2.8° and 3.0° without ice jacking pressure. These results are listed in the Table with the analyses numbers 1B, 1C and 1D, respectively.

Analysis No.	Analysis Conditions	Sliding Resistance Check			Overturning Check			Bearing Resistance at Toe Check		
		Factored ULS			Factored ULS			Factored ULS		
		(a) Acting Force (kN/m)	(b) Resistance Force (kN/m)	Ratio (b)/(a)	(a) Acting Moment (kNm)	(b) Resisting Moment (kNm)	Ratio (b)/(a)	Actual Loading (kPa)	Soil Resistance (kPa)	Ratio (b)/(a)
1	As-built	236	253	1.07	556	731	1.31	365	387	1.06
2	As-built with Ice Jacking Pressure 10 kPa	<u>259</u>	<u>253</u>	0.98	675	731	1.08	<u>439</u>	<u>353</u>	0.80

¹ E. Penner and C.B. Crawford Frost Action and Foundations, Presented at the 27th Annual University of Minnesota Soil Mechanics and Foundation Engineering Conference February 1979 and Published as DBR Paper No. 1090 of the Division of Building Research, Ottawa March 1983.

² Canadian Foundation Engineering Manual 4th Edition.



Analysis No.	Analysis Conditions	Sliding Resistance Check			Overturning Check			Bearing Resistance at Toe Check		
		Factored ULS			Factored ULS			Factored ULS		
		(a) Acting Force (kN/m)	(b) Resistance Force (kN/m)	Ratio (b)/(a)	(a) Acting Moment (kNm)	(b) Resisting Moment (kNm)	Ratio (b)/(a)	Actual Loading (kPa)	Soil Resistance (kPa)	Ratio (b)/(a)
3	As-built with Ice Jacking Pressure 20 kPa	<u>282</u>	<u>253</u>	0.90	<u>794</u>	<u>731</u>	0.92	<u>512</u>	<u>329</u>	0.64
1A	At Actual Wall Rotation Angle of 2.4°	<u>250</u>	<u>256</u>	1.02	592	746	1.26	<u>382</u>	<u>371</u>	0.97
2A	Actual Wall Rotation with Ice Jacking Pressure 10 kPa	<u>273</u>	<u>256</u>	0.94	<u>742</u>	<u>746</u>	1.00	<u>475</u>	<u>339</u>	0.71
3A	Actual Wall Rotation with Ice Jacking Pressure 20 kPa	<u>296</u>	<u>256</u>	0.86	<u>833</u>	<u>746</u>	0.90	<u>530</u>	<u>323</u>	0.61
1B*	Check Stability at Wall Rotation Angle of 2.6°	<u>254</u>	<u>256</u>	1.01	601	750	1.24	<u>386</u>	<u>368</u>	0.95
1C*	Check Stability at Wall Rotation Angle of 2.8°	<u>256</u>	<u>257</u>	1.00	606	751	1.24	<u>388</u>	<u>366</u>	0.94
1D*	Check Stability at Wall Rotation Angle of 3.0°	<u>257</u>	<u>257</u>	1.00	610	753	1.23	<u>390</u>	<u>365</u>	0.94

Notes: Single underlined values denote marginally adequate stability.

Double underlined values denote not adequate stability.

* Ice jacking pressure was not considered for analyses.

4.2 Sliding Resistance

Based on analyses results as summarized in previous Section 4.1, the sliding resistance for the as-built and actual wall inclination conditions without ice jacking pressures is considered to be adequate. When ice jacking pressures of 10 and 20 kPa are applied for the as-built condition and actual wall inclination, sliding resistance is not sufficient.



An assessment of the effect of potential further increases of the rotation angle of the wall to 2.6° , 2.8° and 3.0° indicated that the sliding resistance remains at the marginally adequate levels without ice jacking forces.

4.3 Overturning Moment

Based on analyses results as summarized in previous Section 4.1, the overturning moment check for the as-built and actual wall inclination conditions without ice jacking pressures and with ice jacking pressure of 10 kPa is considered to be adequate. When ice jacking pressures of 20 kPa are applied for as-built condition and actual wall inclination, resisting moment for overturning check is not sufficient.

The assessment of potential further increases in the rotation angle of the wall to 2.6° , 2.8° and 3.0° indicated that the overturning resistance remained at adequate levels when no frost jacking was included.

4.4 Bearing Resistance

Based on the analyses results as summarized in previous Section 4.1, the total applied pressures from the loading on the toe of the retaining wall footing is considered to be adequate for the as-built condition and marginally sufficient for the actual wall inclination without ice jacking pressures.

When ice jacking pressures of 10 and 20 kPa considered for analyses, the total applied pressures are exceeded for the as-built and actual wall conditions. The effect of potential further increases in the rotation angle of the wall to 2.6° , 2.8° and 3.0° will likely cause unsatisfactory conditions when the rotation angle exceeds 2.8° . In this case, the ratio of the factored geotechnical soil resistances at ULS to the factored actual pressures at the toe of the wall will be 0.94 and 0.93.

4.5 Global Stability

For the global stability of the retaining wall, the possibility of slope failure occurring along a surface passing at some depth below the wall and well behind the backfill were analyzed by means of the limit equilibrium method utilizing the Slope/W software developed by Geo-Slope International Ltd. The software analyses numerous potential failure surfaces and establishes a



minimum safety factor aided by user's input. The following soil parameters were used in the slope stability analyses (effective stress condition).

SOIL TYPE	UNIT WEIGHT (kN/m ³)	SHEAR STRENGTH (kPa)	INTERNAL FRICTION (Degrees)
Heterogeneous Backfill (Silt/Sandy Silt/Clayey Silt)	20	3	30
Sand (native and backfill)	20	0	30
Silty Clay (stiff to hard)	19	10	32

Based on the above selected parameters, the analyses indicated that a factor of safety of 1.8 was obtained indicating adequate global stability as shown in Figure 1. A factor of safety of 1.5 is normally considered to be sufficient.

5. SUMMARIZED DISCUSSION AND CONCLUSIONS

Based on analyses, the following probable causes were identified and investigated, the conclusions are summarized and discussed in the following sections.

5.1 Sliding Resistance Check

The factored sliding resistance at ULS was adequate to marginal without ice jacking pressures but considered to be marginal to insufficient when ice jacking pressures are applied.

Future rotation of the wall to a total of 3.0° without ice jacking pressures will not exceed the sliding resistance at ULS.

The analysis also indicated that the factored sliding resistance at ULS will be insufficient without the support from passive resistance in front of the wall. In view of these results, the materials in front of the wall should not be excavated without removing some of the loading behind wall or otherwise supporting the wall for sliding resistance.

5.2 Overturning Resistance Check

The factored overturning resistance at ULS of the retaining wall was adequate to marginally adequate for the as-built and present conditions and with ice jacking pressures of 10 kPa. It was,



however found to be inadequate when ice jacking pressures of 20 kPa were applied in the analysis.

Potential increases in the wall rotation angle from the current 2.4° to 3.0° values without ice jacking pressures will not cause the computed factored loads to exceed the factored resistance at ULS for the overturning moments.

5.3 Bearing Resistance Check

The analysis showed that the factored total applied pressure at ULS under the toe zone of the footing is adequate to marginally supported in the dry summer months (without ice jacking pressure). During the wet periods of late fall/early spring when ice jacking from frost in the frost susceptible soils behind the wall may occur, the estimated factored loading exceeded the factored geotechnical resistance at ULS of the foundation subgrade.

Potential increases in the wall rotation angle from the current 2.4° to 3.0° values without ice jacking forces will likely cause the factored applied pressures at the toe of the wall to exceed the factored resistance at ULS when the angle exceeds 2.6° , causing further tilting of the wall. This condition will tend to cause the wall to lean forward by creep of the toe area into the founding sand subgrade.

5.4 Global Stability Check

The results of the global stability analysis indicated adequate conditions with factor of safety over 1.5.

5.5 Loss of Ground Support

A potential ground loss condition could occur when the existing storm sewer has become cracked/damaged and sand from the subgrade is washed into the sewer. However, based on the results of a CCTV camera survey of the existing storm sewer, this possible cause was discounted because the pipe was found in acceptable conditions.



5.6 Conclusions

It is concluded that the causes of the wall movement are a combination of marginally sufficient resistances at ULS to sliding, overturning and geotechnical bearing resistance that is aggravated by ice jacking forces during the winter months and in the wet periods of seasonal freeze-thaw cycles in the late fall/early spring. These ice jacking forces appear to originate from the 2.3 m thick frost susceptible backfill materials to the wall and presence of a buried clayey silt layer that restricts the vertical drainage.

It is noted that the stability of the retaining wall becomes less adequate as the wall progressively tilts forward towards the highway. It is also concluded, based on the current analyses and assumptions, in particular Analysis No. 1B (overturning check) that the retaining wall may only tilt forward an additional 20 mm (0.2°) towards the highway measured at the abutment before progressively excessive movement occurs due to the increasing large loads applied under the toe of the wall footing. Based on historical data, the limit may be reached within the next 3 to 5 years, subject to monitoring of wall movement. In addition, if the wall is allowed to tilt further, a gap will likely develop at the abutment with consequent loss of backfill behind the wall. These conclusions were based on the analyses of the subsurface data from the boreholes and the 1968 construction drawings. It is recommended that the actual founding conditions at the subgrade level as well as the structural condition of the structure be confirmed and investigated in test pits dug to expose the existing footing and subgrade material.

6. REMEDIAL MEASURES

6.1 General Considerations

As indicated previously, the "Do-Nothing" approach is not recommended unless the wall will be replaced in less than 3 years, subject to satisfactory wall monitoring.

If the wall can be left in its present out-of-vertical position, it may be possible to stop further movement by reducing the driving forces and/or increasing the resisting forces on the retaining wall, subject to a structural review of the wall condition. Should the long-term safety or aesthetics of the wall need to be improved, a new retaining wall should be constructed, or the retaining wall may be jacked back into its original position.



6.2 Remediation Maintaining Tilted Wall Condition

To reduce the driving forces on the retaining wall, the existing 2.3 m thick layer of frost-susceptible fill material behind the wall should be replaced with non-frost susceptible material (OPSS Granular A or Granular B Type 1) to reduce ice-jacking effects. Backfilling behind the wall should be carried out in conformance with Ontario Provincial Standards Drawings for granular soils OPSD 3121.150. Lightweight backfill fill such as water-cooled furnace slag or tire derived aggregate (TDA) (ASTM D6270-08) may also be used to further reduce the driving forces, however these materials will also reduce the resisting forces of gravity retaining wall structures. The upper 0.6 m of the backfill should be constructed using clayey soils as indicated in the existing construction drawing D-6214-19 detail to minimize infiltration of storm water behind the retaining wall. In addition, the surface run-off over the slope behind the wall should be directed by mid-height and top of slope swales away from the top of retaining wall area.

This material replacement should be constructed without temporarily increasing the loading on the retaining wall with added loadings from stockpiles or heavy equipment. In view of the required up to 2.3 m excavation depth, shoring or the use of a trench box will likely be required to support the existing slope. An alternative method of construction may be proposed by the contractor that will satisfy the requirements of the Occupational Health and Safety Act. Based on the previous contract Drawings D-6049-6 and D-6049-8 temporary shoring or underpinning should be implemented at the northeast wing wall to avoid structural problems from unbalanced horizontal loading on the wing wall and/or undermining of the existing backfill to the abutment. The excavation and replacement of the existing retaining wall backfill should not extend to within approximately 5 m of the east abutment during the construction of the shoring or underpinning.

Several alternatives may be considered in addition to the replacement of the backfill materials to increase the resisting forces as outlined below, subject to structural analysis and possible reinforcement, such as tieback or helical anchors. Deadman anchors require a separate active and passive zones of influence behind the retaining wall and in front of the deadman anchor that would exceed the existing working space behind the retaining wall. Consequently, deadman anchors are not considered feasible at this site.



6.2.1 Tieback Soil Anchors

Tieback soil anchors installed near horizontally through the wall are considered to be feasible to increase the resisting forces. Vertical anchors through heel of the retaining wall are not considered to be practical within the highest panel 12 because a very deep excavation would be required to expose the existing footing.

For design purposes, it is recommended that soil anchors be fixed within the compact sand. The ultimate load holding capacity of anchorages (R) can be estimated using the following equation:

$$R = \pi D L \tau_{ult}$$

Where

- D = diameter of fixed anchor, m
- L = length of fixed anchor, m
- τ_{ult} = ultimate bond or skin friction at soil/grout interface, kPa
= 300 kPa compact sand

A resistance factor of 0.4 should be applied to the computed anchor capacity to determine the factored ULS resistance. It is recommended that the anchors be prestressed. The anchors should extend at least 30 bar diameters into compact sand and be spaced at a distance of at least four times the diameter of the anchor hole. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length. Design, installation and testing of the anchors subjected to tensile stresses should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC.

If anchors are employed, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the anchors.

6.2.2 Horizontal Helical Anchors

Horizontal helical anchors installed through the wall are considered to be feasible to increase the resisting forces.

The helical anchor systems are considered to be proprietary products and should be designed by the supplier.



For preliminary design and cost estimate purposes, it is recommended that soil anchors be fixed within the native compact sand. The ultimate load bearing capacity of multi-helix anchors (Q_{ult}) for the cohesionless soils can be estimated using the following equation:

$$Q_{ult} = A_H \gamma D N_q$$

Where

- A_H - projected helix area, m^2
- γ - effective unit weight of the soil, kN/m^3
- D - depth of helix plate, m
- N_q - bearing capacity factor for non-cohesive component of soil, based on internal angle of friction

A resistance factor of 0.4 should be applied to the computed anchor capacity to determine the factored ULS resistance. The torque required to install a helical anchor is empirically related to its ultimate capacity and provided in the following equation:

$$Q_{ult} = K_t T$$

Where

- Q_{ult} - Ultimate capacity, kN
- K_t - Empirical Torque factor, m^{-1}
- T - Installation Torque (kNm)

As indicated in Section 6.2.1, the design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC.

6.2.3 Underpinning with Grout Concrete

Underpinning the toe area of the wall footing with concrete grout to increase the local bearing resistance may be considered.

The underpinning should be carried out in short panels as shown in Figure 2. A maximum panel width of 1.5 m is recommended. At all times, at least two intact panels must be left between open panels.

Underpinning may be done by pouring concrete panels up to approximately 80 mm below the underside of the existing footing. Once the concrete has cured, the remaining space must be filled tightly with dry-pack grout.



Care must be taken to avoid loss of soil below the retaining wall footing and any voids must be grouted. Underpinning operations should be inspected by geotechnical personnel.

6.2.4 Underpinning with Helical Piles or Micropiles

Underpinning the retaining wall footing at the toe area using helical piles or micropiles is also considered feasible to increase the geotechnical resisting forces.

Any underpinning will require that the excavations in front of the retaining wall be carried out after partial removal of the wall backfill in view of the low safety reserve for the actual stability condition of the wall.

6.3 Remediation with Improved Wall Condition

Should the safety and/or aesthetics of the retaining wall need to be improved, the retaining wall should be reconstructed. Considering that the existing wall is approximately 44 years old and upon a structural condition review, the construction of new replacement wall may be considered to be appropriate. Alternatively, it may be possible to jack the wall back into nearly its original position using pressure grouting under the foundation. The pressure will need to be designed to avoid structural damage to the wall footing and is typically a proprietary procedure. It is anticipated, however that extensive excavation and replacement of the soil behind the wall will be required to accomplish this effect.

A new retaining wall may be designed as a concrete cantilever wall or RSS at this site. Recommendations for the reconstruction of the retaining wall were not within the scope of this assignment.

The following table presents an overview assessment of the advantages and disadvantages, including relative costs and risk/consequences of the remedial alternatives from the foundation perspective.



ALTERNATIVES	ADVANTAGES	DISADVANTAGES	RELATIVE COST	RISKS/CONSEQUENCES
Reduce Driving Forces				
Replace 2.3 m thick layer of frost susceptible backfill material behind the wall with non-frost susceptible material (Granular A or Granular B Type 1)	<ul style="list-style-type: none"> • Small construction equipment is adequate • Moderate risk to abutment foundations • Minimized disruption to Highway 7 traffic 	<ul style="list-style-type: none"> • May require temporary shoring for excavation of existing fill • Will require monitoring of wall movement • Will require partial fill replacement adjacent to the existing abutment 	<ul style="list-style-type: none"> • Lower cost than increasing resistance forces or constructing a new wall 	<ul style="list-style-type: none"> • May undermine the fill to the abutment under the existing wing wall • Loss or potential fill settlement may affect the existing approach slabs
Increase Resisting Forces				
Tieback soil anchors	<ul style="list-style-type: none"> • Tieback anchors can be installed near horizontally to increase resistance forces • No excavation or groundwater control required • Lower risk to abutment foundations • Shorter construction period 	<ul style="list-style-type: none"> • May require additional property beyond east property line • Larger construction equipment is required than for soil replacement • Will require associated construction to reduce driving forces due to ice jacking by replacing the frost-susceptible backfill • May require structural reinforcement of retaining wall • Will require lane closures of the Highway for installation of tieback anchors 	<ul style="list-style-type: none"> • Higher cost than replacement of frost susceptible soil 	<ul style="list-style-type: none"> • Possible obstructions to anchor hole drilling may increase costs • Possible cost escalation due to undocumented retaining wall condition requiring reinforcement • Extended traffic lane closures may increase local accident rate
Horizontal Helical Anchors	<ul style="list-style-type: none"> • Helical anchors can be installed near horizontally to increase resistance forces • No excavation or groundwater control required • Lower risk to abutment foundations • Shorter construction period 	<ul style="list-style-type: none"> • May require additional property beyond east property line • May require structural reinforcement of retaining wall • Will require associated construction to reduce driving forces due to ice jacking by replacing the frost-susceptible backfill • May require traffic lane restrictions 	<ul style="list-style-type: none"> • Lower cost than tieback soil anchors 	<ul style="list-style-type: none"> • Possible obstructions to anchor hole drilling may increase costs • Possible cost escalation due to undocumented retaining wall condition requiring reinforcement • Restricted traffic lanes may increase local accident rate.



ALTERNATIVES	ADVANTAGES	DISADVANTAGES	RELATIVE COST	RISKS/CONSEQUENCES
Underpinning toe of retaining wall footing with (grout) concrete	<ul style="list-style-type: none"> Smaller construction equipment is required Suitable due to shallow foundation zone Requires a narrower construction zone than for anchors 	<ul style="list-style-type: none"> May affect the existing abutment foundations Groundwater control will be required Installation will be hindered by existing underground utilities 	<ul style="list-style-type: none"> Higher cost than replacement of frost susceptible soil 	<ul style="list-style-type: none"> High risk of damage to existing sewers will require temporary diversion Excavations for underpinning temporarily reduce the stability of the retaining wall
Underpinning toe of retaining wall footing with Helical piles or micropiles	<ul style="list-style-type: none"> Smaller construction equipment is required than for tieback anchors No groundwater control required 	<ul style="list-style-type: none"> Needs to be installed below the abutment foundations and adjacent utilities Test section and design will be required Will require exposing the toe of the retaining wall footing and temporarily unloading of the wall backfill 	<ul style="list-style-type: none"> Higher cost than replacement of frost susceptible soil Lower cost than for underpinning option with grout concrete 	<ul style="list-style-type: none"> Risk of inadequate structural resistance of the toe of the retaining wall footing may require a redesign of the support system
Replace with New Retaining Wall				
Demolish existing retaining wall and construct a new retaining wall	<ul style="list-style-type: none"> New design wall may consider several options including concrete, cantilever and RSS walls etc. Low risk Long-term solution 	<ul style="list-style-type: none"> Additional foundation investigation may be required for full length replacement Utilities relocation may be required Roadway protection (shoring) will be required adjacent to abutment to support fill below wing wall Larger construction equipment is required Longer construction period Extended lane closure period will be required during construction 	<ul style="list-style-type: none"> Higher cost than other options 	<ul style="list-style-type: none"> Lowest risk of all options in the long term Will require roadway protection for excavations below the wing wall level Inadequate roadway protection would cause loss of backfill below approach slab



It is recommended that the frost susceptible backfill behind the retaining wall should be excavated and replaced with non-frost susceptible materials to effectively reduce the driving forces acting on the wall due to ice jacking forces. It is considered that attempting to only increase the resisting forces would augment the ice jacking forces by restricting the expansion of ice lenses in the frost susceptible backfill materials. These ice jacking forces could damage the wall and/or tilting of the wall would likely continue.

Although the replacement of the retaining wall backfill material would result in a more stable wall than the present condition, it is recommended that further stabilization measures should also be provided unless the retaining wall is to be replaced within the next 3 years. Based on the assessment of the advantages and disadvantages of the various alternatives, it is recommended that soil anchors (preferably horizontal helical or tieback anchors) be installed to augment the resisting forces, providing that it will be acceptable to maintain the wall in its presently tilted position. The soil anchors will not require that the toe of the retaining wall footing be exposed, reducing the passive resistances in front of the wall. The selection of the type of soil anchor will depend on the relative costs and constructability issues at the site.

6.4 Monitoring Program

It is recommended that a monitoring program of the retaining wall movement be implemented prior to the start of the remedial measures. The NSSP for the monitoring program is attached in Appendices FDR-3 and FDR-3A.

The Geotechnical (Foundation Specialty) Monitoring Plan for the monitoring program during and after remedial measures is also required and is attached in Appendix FDR-4.



7. CLOSURE

This Foundation Design Report was prepared by Mrs. N. S. Balakumaran, P. Eng., and reviewed by Mr. C. M. P. Nascimento, P. Eng., Project Manager. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



Nesam S. Balakumaran, P.Eng.
Project Engineer



Carlos M.P. Nascimento, P.Eng.
Project Manager



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

NB/CN/BRG:nb-mi-nk

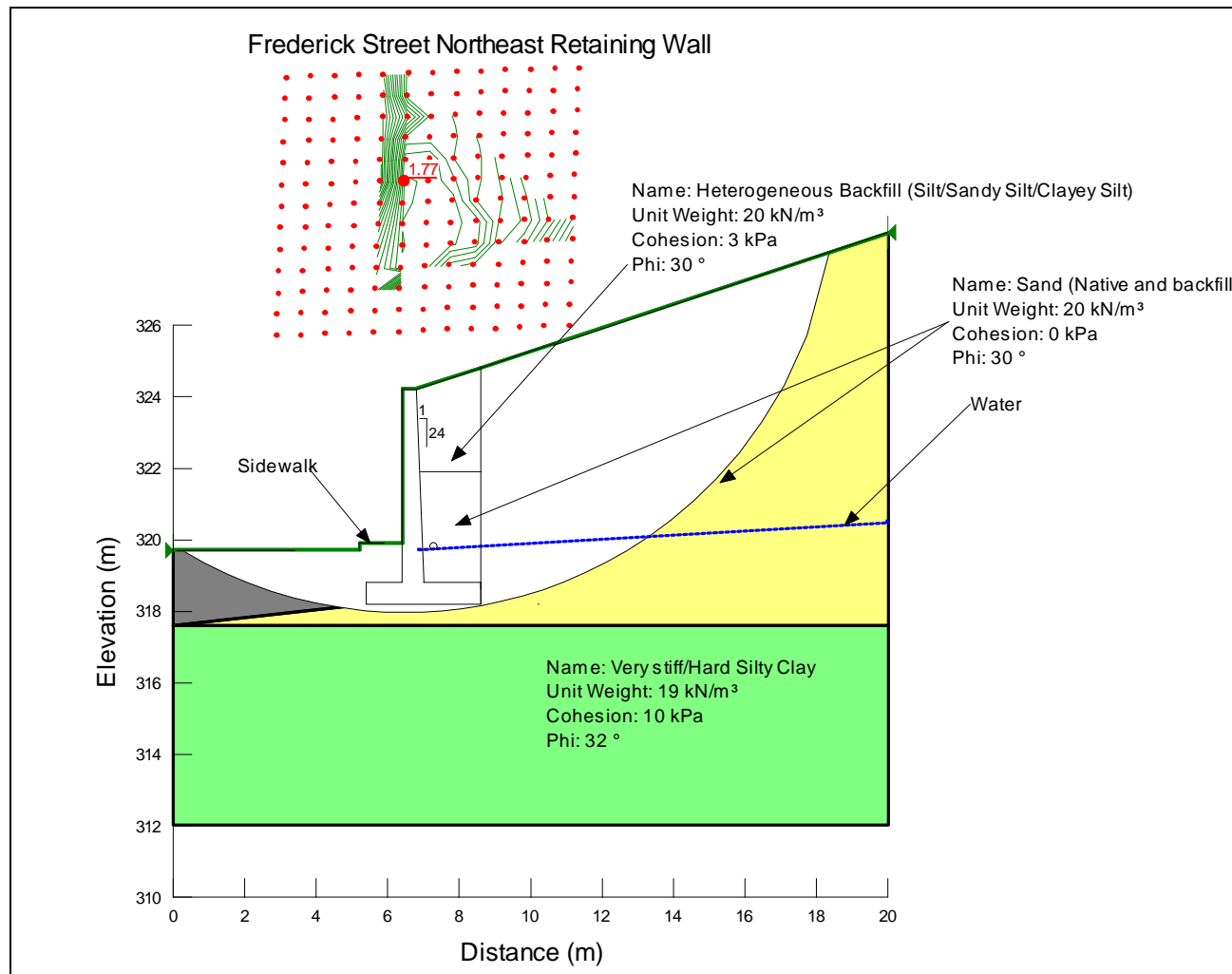
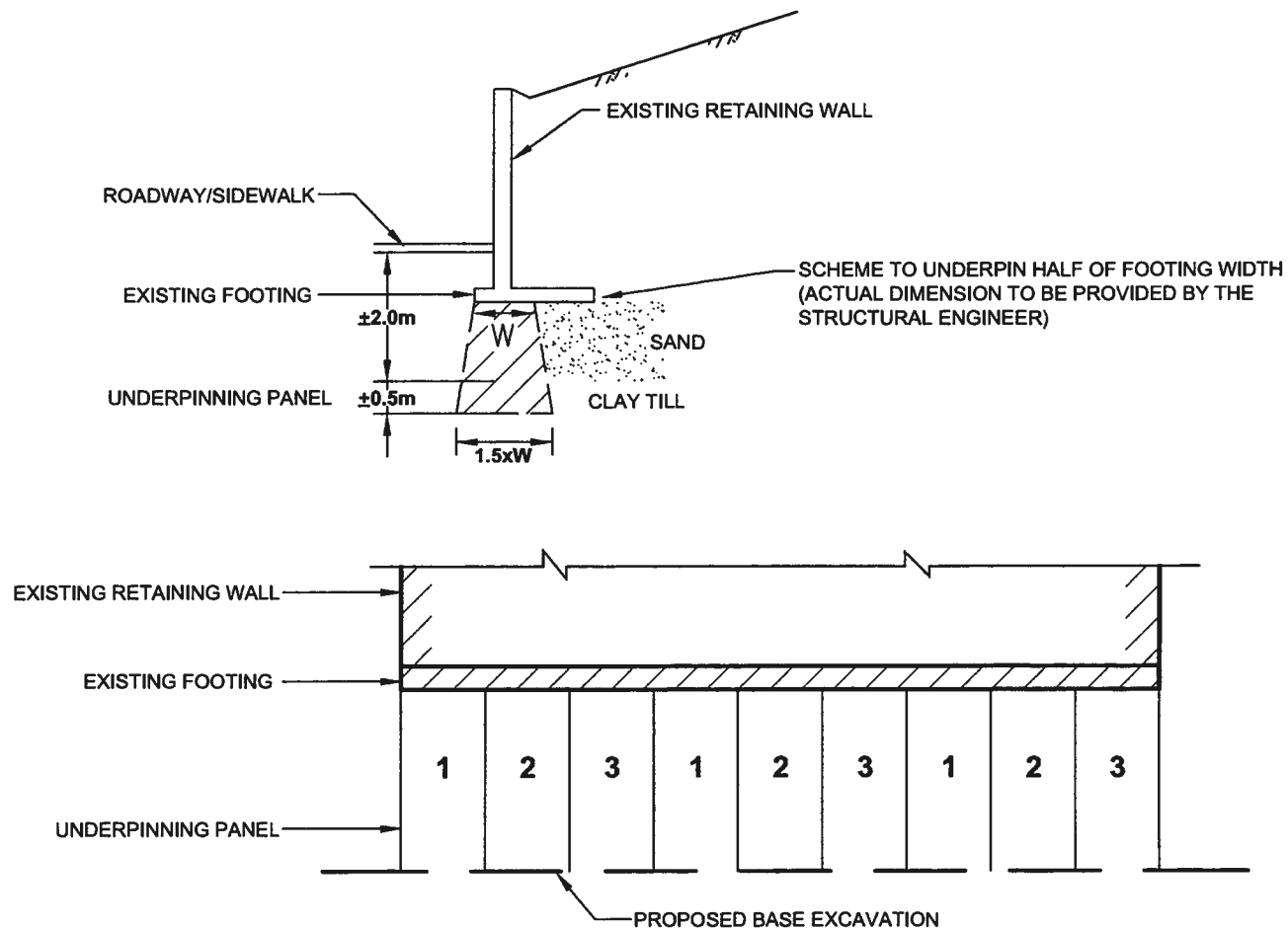


FIGURE 1-SLOPE STABILITY DIAGRAM



STANDARD DRAWING

UNDERPINNING PROCEDURE

FREDERICK STREET UNDERPASS RETAINING WALL



Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN:	N.A.	DATE	SCALE	JOB NO.	FIGURE NO.
CHECKED:	N.B.	MAY 2012	N.T.S.	10KF079C	2
APPROVED:	C.N.				



APPENDIX FDR-1

EXCERPT OF CONSTRUCTION DRAWINGS, CONTRACT NO. 68-62
(EXCERPTS ONLY)

PROVINCE OF ONTARIO

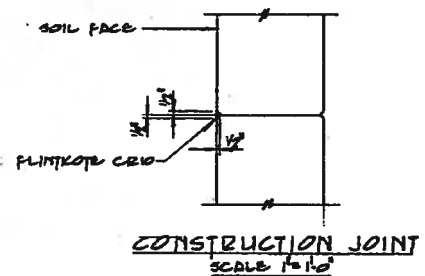
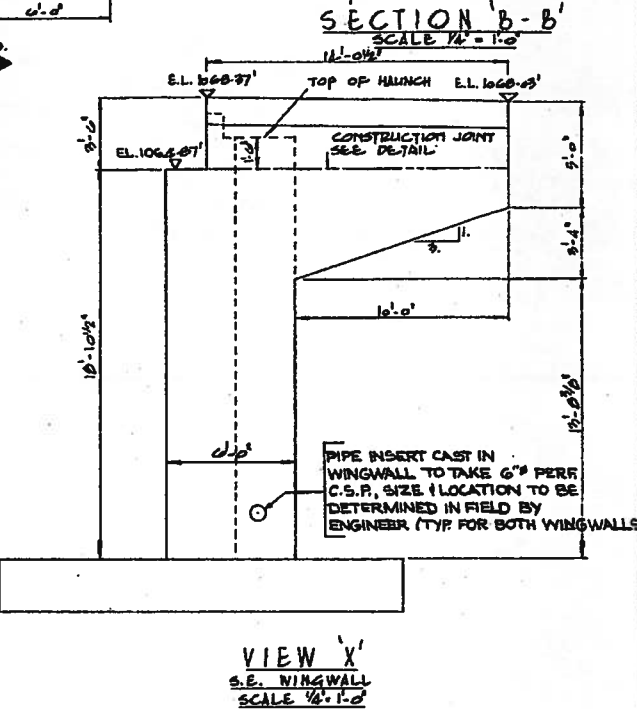
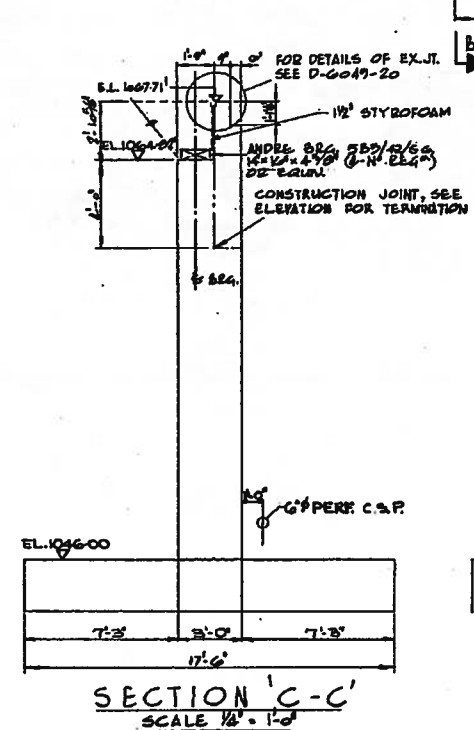
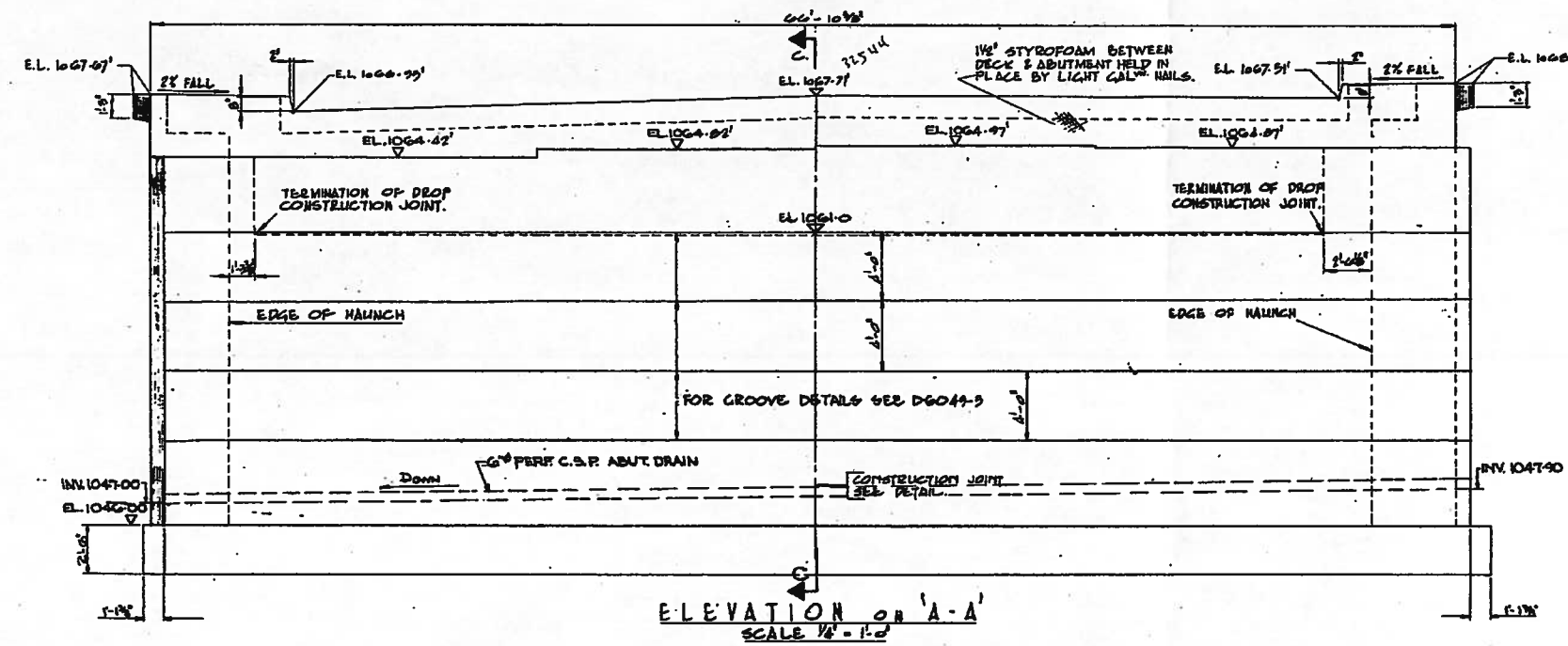
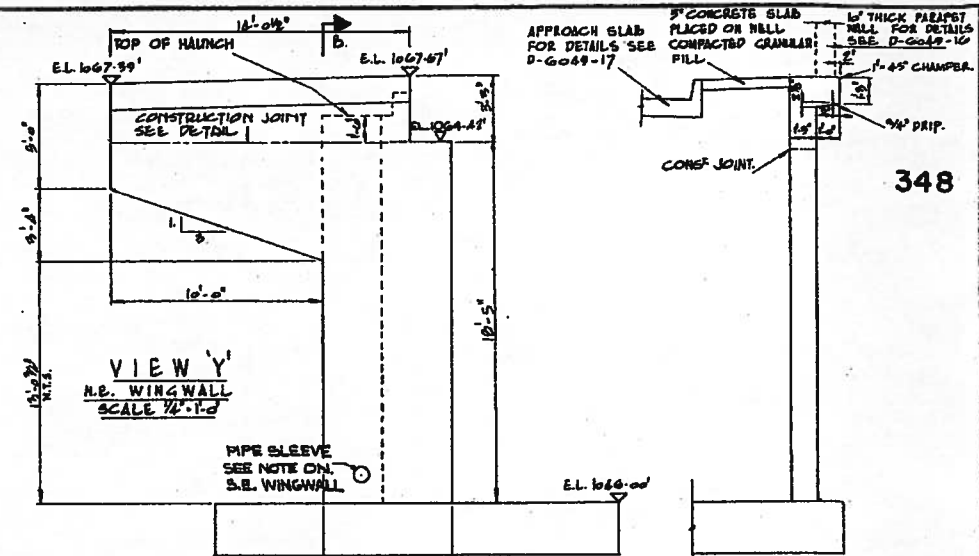
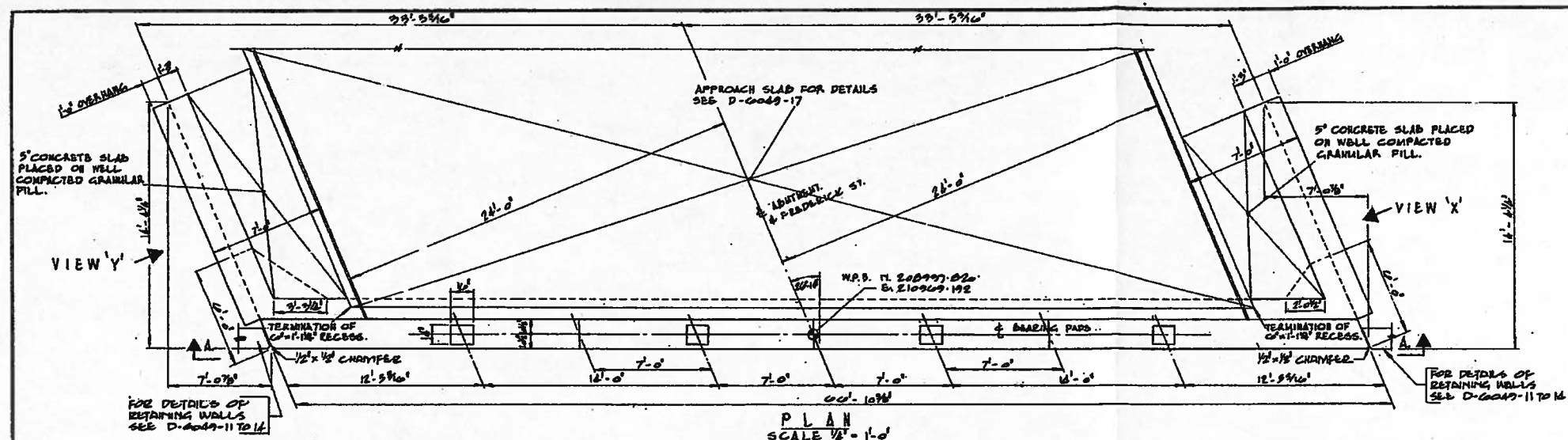


DEPARTMENT OF HIGHWAYS

CONTRACT No. 68-62

CONTRACT DRAWINGS

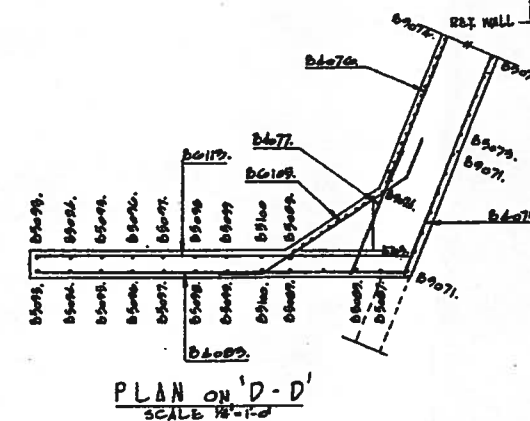
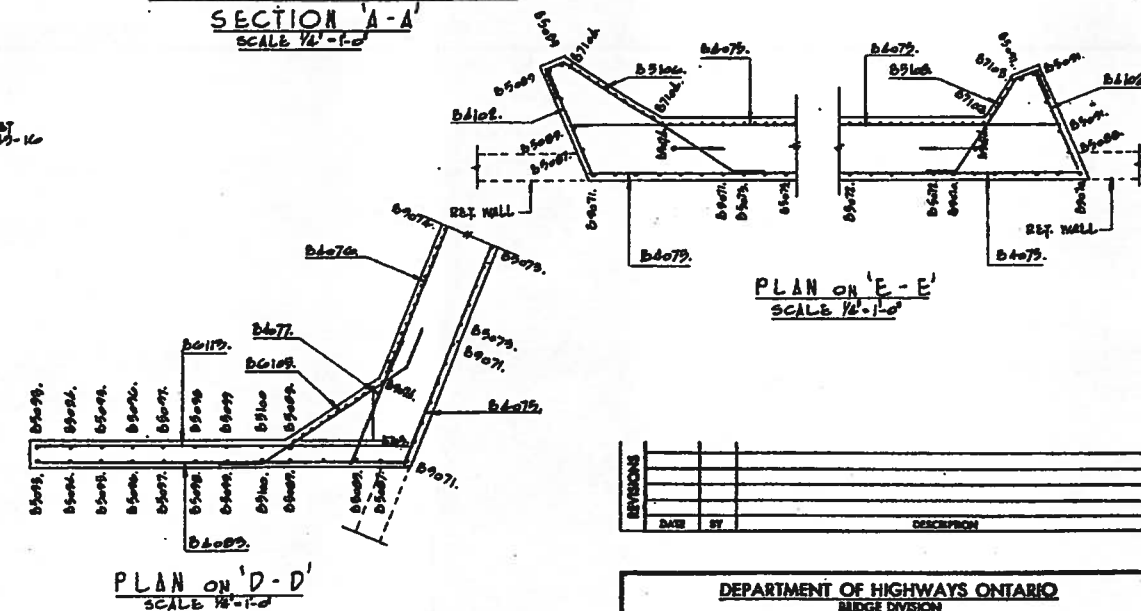
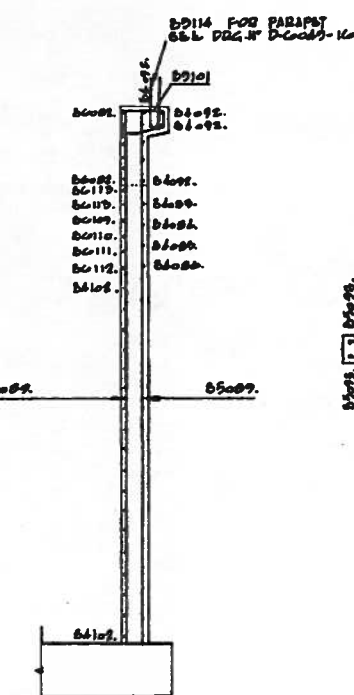
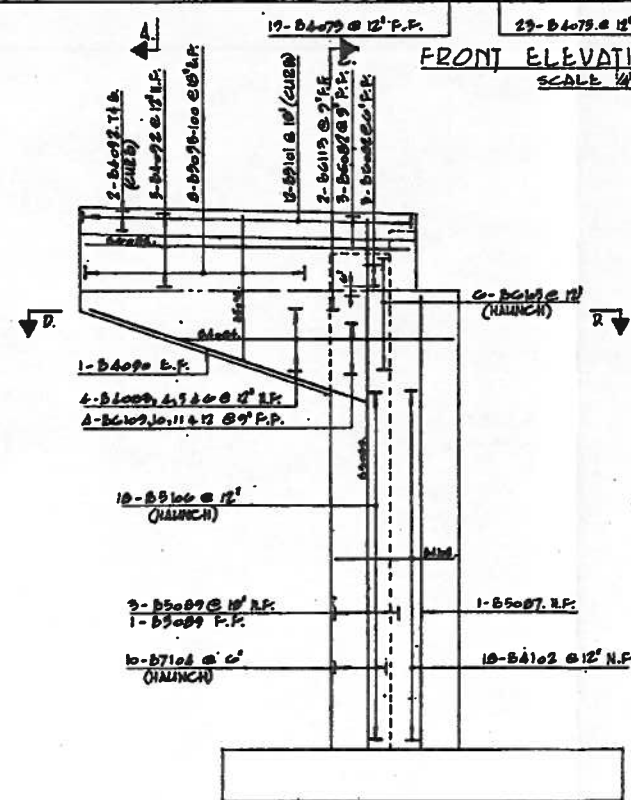
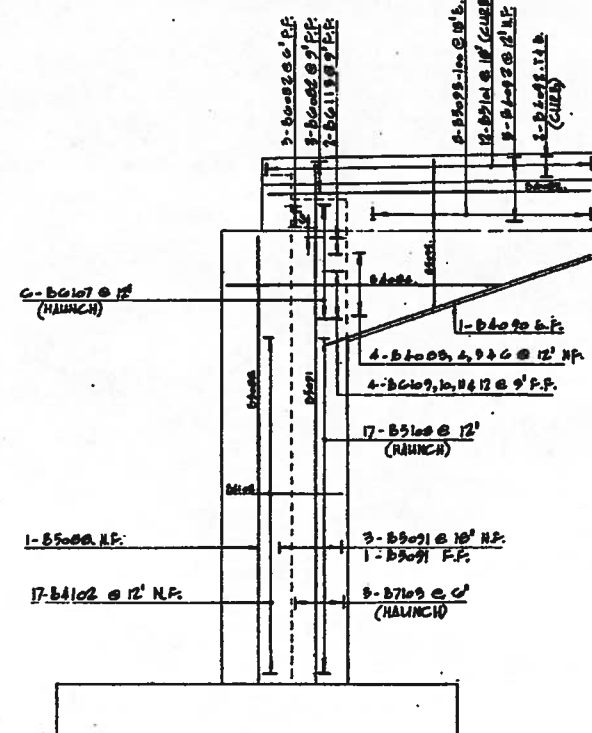
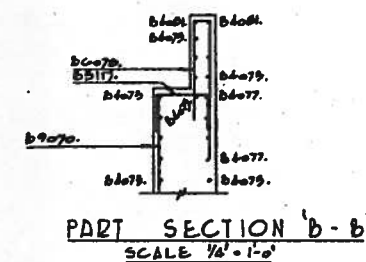
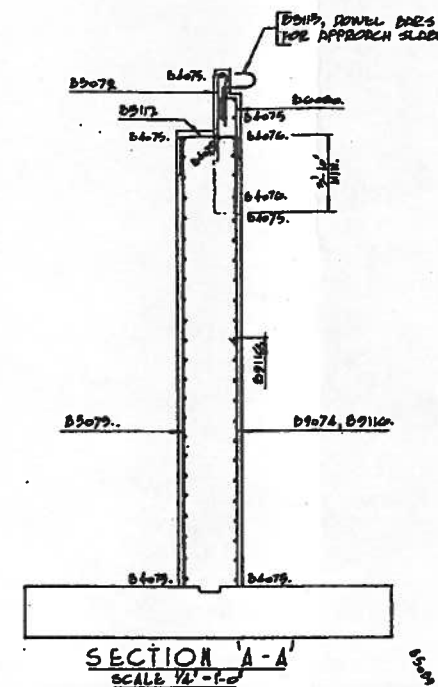
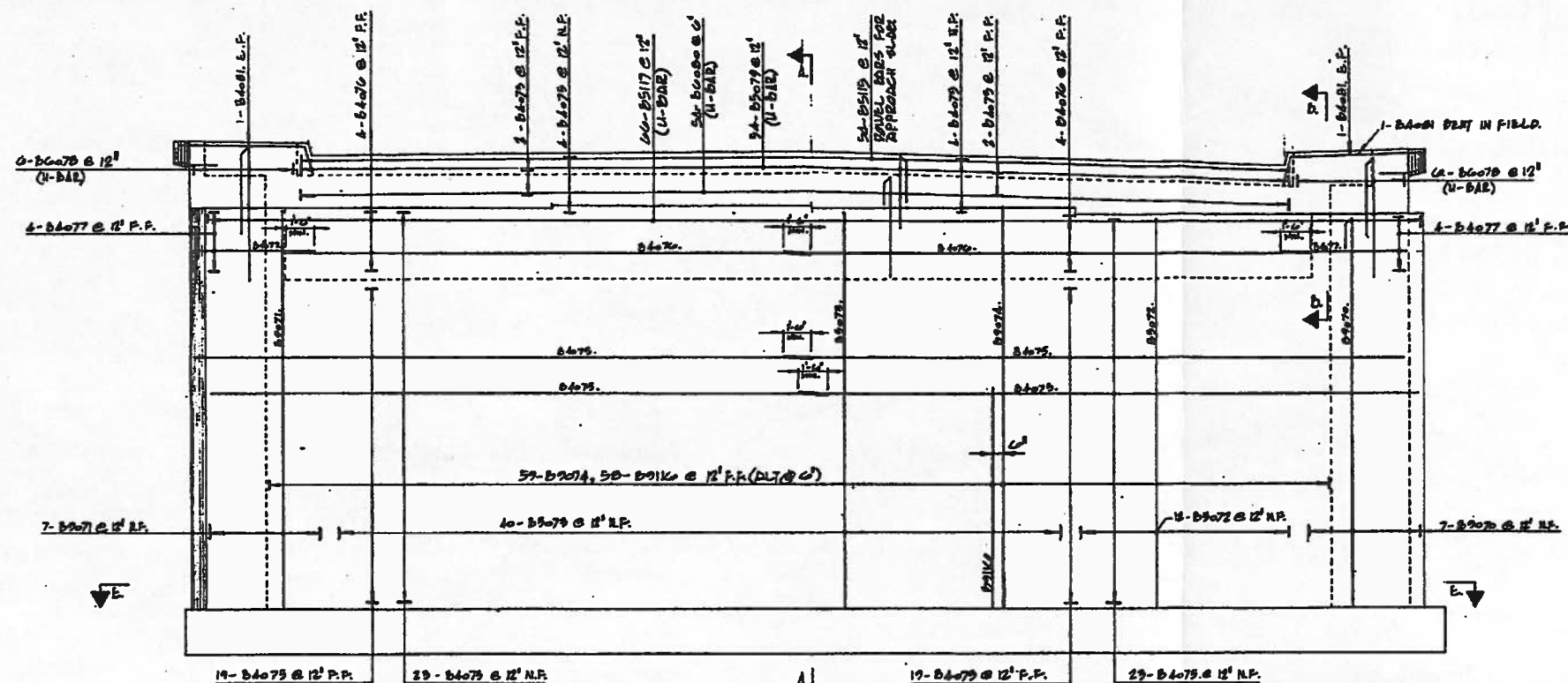
BOOK 1 OF 5



REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO			
BRIDGE DIVISION			
A. D. MARGISON AND ASSOCIATES LIMITED			
CONSULTING PROFESSIONAL ENGINEERS			
KITCHENER-WATERLOO EXPRESSWAY			
FREDERICK STREET UNDERPASS			
KING'S HIGHWAY DRYDEN BLVD.		DIST. No. 4	
CO. WATERLOO			
CITY OF KITCHENER		CON.	
LAYOUT OF EAST ABUTMENT & WINGWALLS			
APPROVED	DESIGN	SITE No.	W.P. No.
		33-234	634-64
		CONTRACT	
		No.	63-62
DRAWN	CHECK	DATE	LOADING
K.E.J.	P.E.M.		1980-44
		DRAWING	D-6049-6

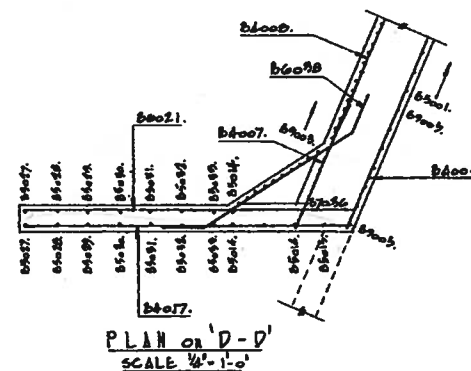
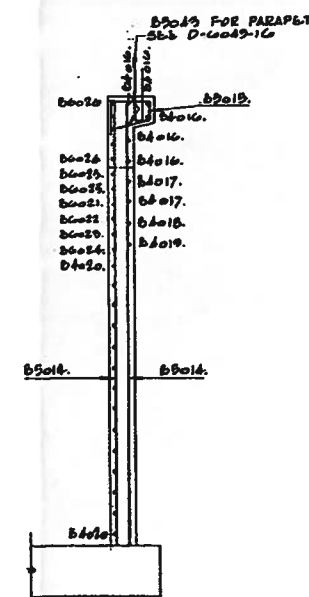
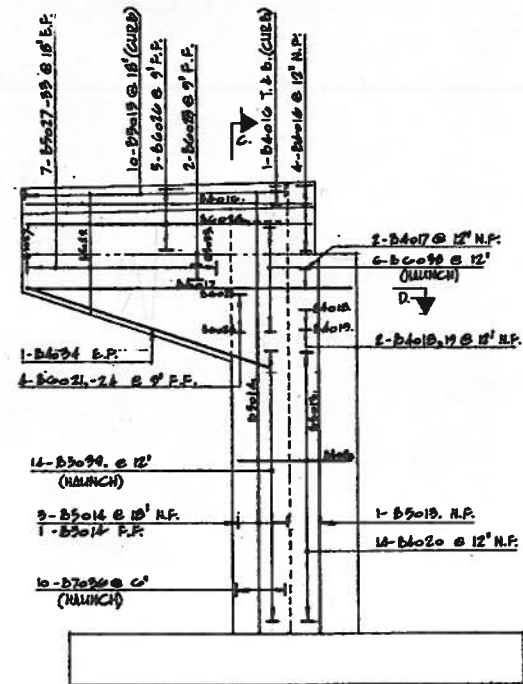
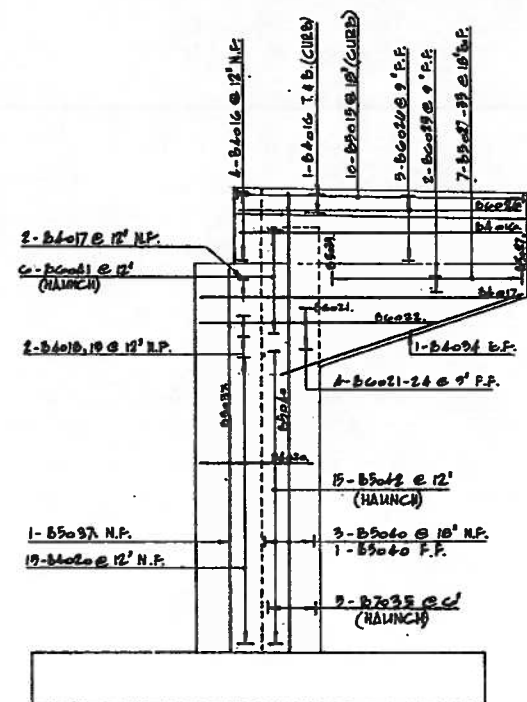
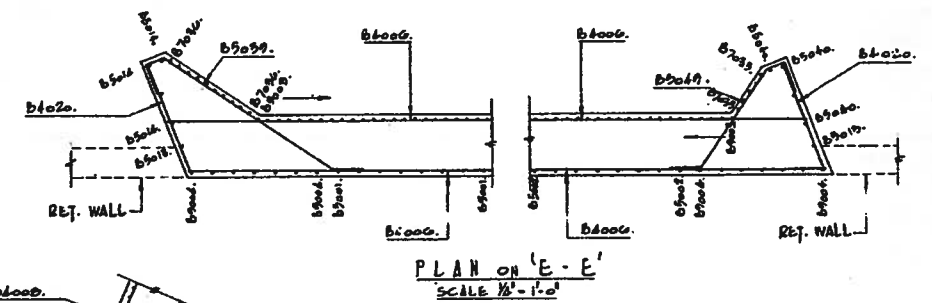
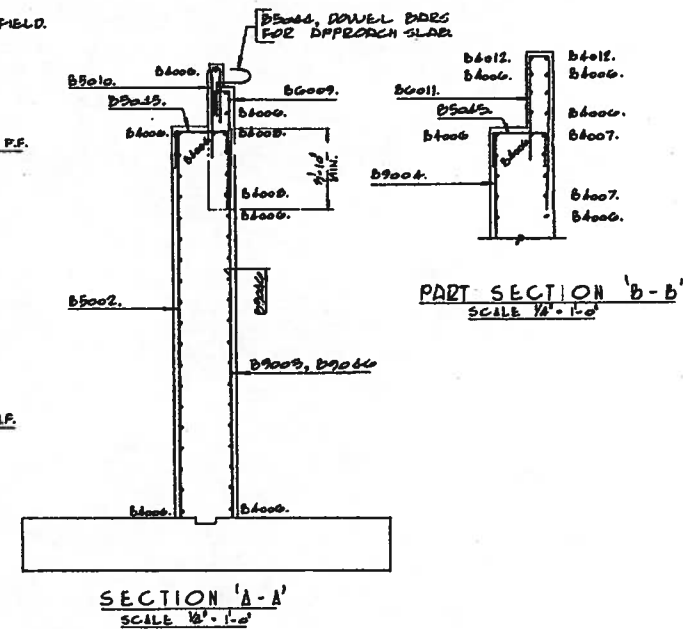
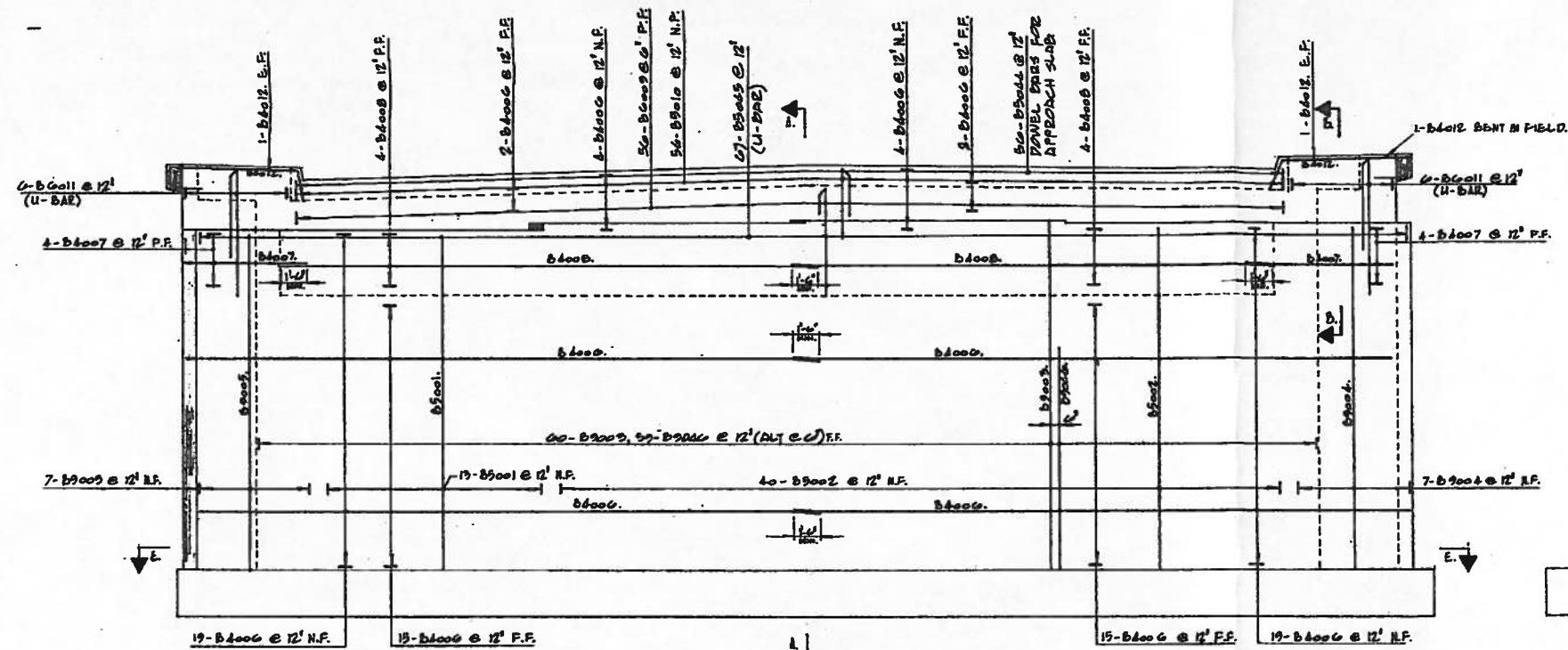




NOTE
N.F. = NEAR FACE
F.F. = FAR FACE
E.F. = EACH FACE



DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION							
A. D. MARGISON AND ASSOCIATES LIMITED CONSULTING PROFESSIONAL ENGINEERS.							
KITCHENER-WATERLOO EXPRESSWAY FREDERICK STREET UNDERPASS							
KING'S HIGHWAY <u>DRYDEN BLVD.</u>	Dist. No. 4						
CO. WATERLOO							
CITY OF KITCHENER	LOT <u>CON.</u>						
REINFORCEMENT OF WEST ABUTMENT & WINGWALLS							
APPROVED <u><i>[Signature]</i></u> <small>SEAL OF ENGINEER</small>							
SITE No. <u>33-234</u>	W.P. No. <u>634-6</u>						
CONTRACT No. <u>686</u>	DRAWING No. <u>D-6049-7</u>						
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 50%; padding: 2px;">DESIGN <u>H. G.</u></td> <td style="width: 50%; padding: 2px;">CHECK <u>C.</u></td> </tr> <tr> <td style="padding: 2px;">DRAWING <u>K. E. J.</u></td> <td style="padding: 2px;">CHECK <u>P. E. M.</u></td> </tr> <tr> <td style="padding: 2px;">DATE <u>JUNE 62</u></td> <td style="padding: 2px;">LOADING <u>H20-44</u></td> </tr> </table>	DESIGN <u>H. G.</u>	CHECK <u>C.</u>	DRAWING <u>K. E. J.</u>	CHECK <u>P. E. M.</u>	DATE <u>JUNE 62</u>	LOADING <u>H20-44</u>	
DESIGN <u>H. G.</u>	CHECK <u>C.</u>						
DRAWING <u>K. E. J.</u>	CHECK <u>P. E. M.</u>						
DATE <u>JUNE 62</u>	LOADING <u>H20-44</u>						

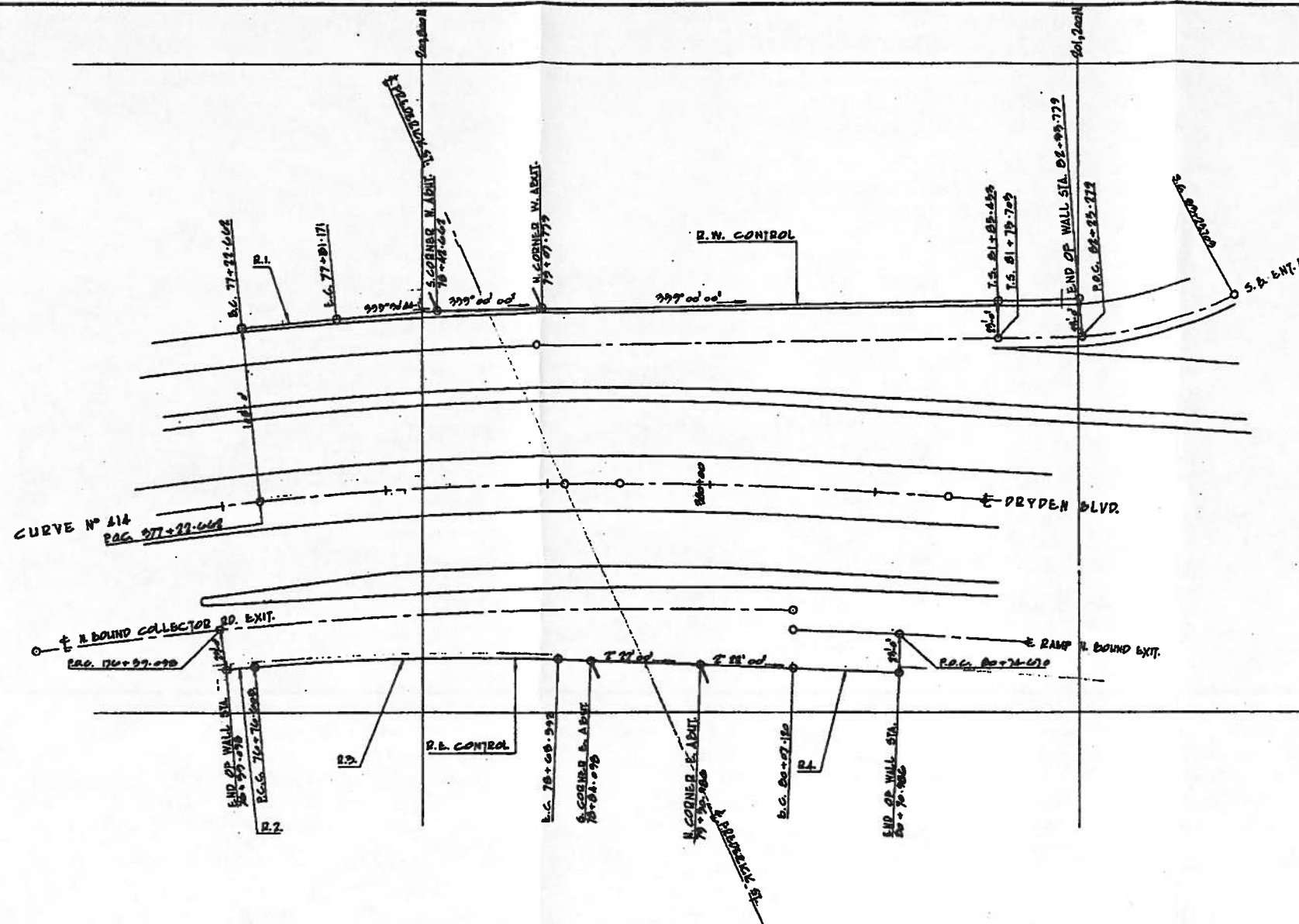


NOTE
N.F. = NEAR FACE
F.F. = FAR FACE
E.F. = EACH FACE

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION		
A.D. MARGISON AND ASSOCIATES LIMITED CONSULTING PROFESSIONAL ENGINEERS		
KITCHENER-WATERLOO EXPRESSWAY FREDERICK STREET UNDERPASS		
KING'S HIGHWAY <u>DRYDEN BLVD.</u>	DIST. No. 4	
CD. WATERLOO		
CITY OF KITCHENER	LOT	CON.

REINFORCEMENT OF EAST ABUTMENT & WINGWALLS		
<div style="display: flex; justify-content: space-between; align-items: center;"> <div style="width: 40%;"> APPROVED </div> <div style="width: 30%;"> SITE No. <u>33-234</u> </div> <div style="width: 30%;"> W.P. No. <u>634-64</u> </div> </div> <div style="display: flex; justify-content: space-between; align-items: center; margin-top: 10px;"> <div style="width: 40%;"> CONTRACT No. </div> <div style="width: 30%;"></div> <div style="width: 30%; text-align: right;"> <u>68-62</u> </div> </div> <div style="display: flex; justify-content: space-between; align-items: center; margin-top: 10px;"> <div style="width: 40%;"> DESIGN <u>R.G.</u> CHECK <u>L.</u> DRAWN <u>R.G.</u> CHECK <u>F.B.M.</u> </div> <div style="width: 30%;"></div> <div style="width: 30%;"> DRAWING No. <u>D-6049-8</u> </div> </div> <div style="display: flex; justify-content: space-between; align-items: center; margin-top: 10px;"> <div style="width: 40%;"> DATE <u>JUNE 27</u> LOADING <u>HS20-44</u> </div> <div style="width: 30%;"></div> <div style="width: 30%;"></div> </div>		



CIRCULAR CURVE DATA				
	R1	R2	R3	R4
Δ	1° 33' 40.8"	0° 21' 10.4"	7° 32' 15"	1° 17' 19.6"
Δ	2° 30' 22"	2° 02' 00"	8° 01' 51.4"	2° 02' 00"
Δ	207.000'	1041.700'	1421.900'	1041.700'
Δ	20.290'	0.750'	93.600'	31.730'
Δ	30.300'	17.500'	180.900'	60.000'
Δ	0.212'	0.010'	3.074'	.170'

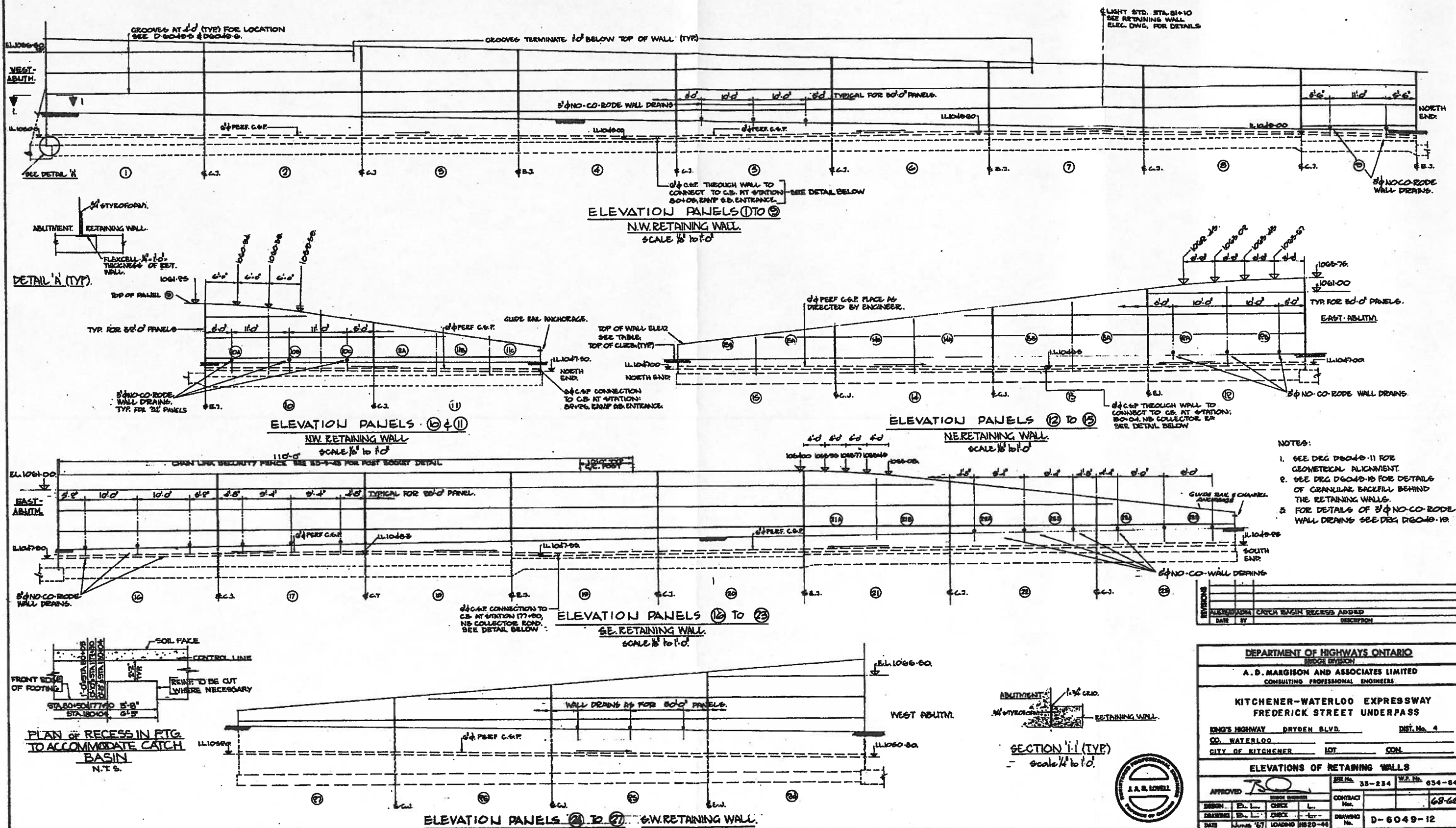
WEST RETAINING WALLS				
CONTROL	CURVE	POINT	STATION	COORDINATES
				NORTH EAST
E. DRYDEN BLVD.	414	P.C.	77+22.660	200700.911 210467.330
		P.T.	77+22.660	200697.939 210766.180
		P.L.	77+51.720	200719.144 210797.043
		E.C.	77+01.171	200767.378 210750.794
		E.C.	78+01.608	200808.094 210791.260
		E.C.	79+07.779	200874.401 210790.000
		E.C.	80+08.433	201171.000 210749.777
		E.C.	82+08.779	201200.420 210744.230
RAMP S.B. ENTRANCE	T-1	P.C.	82+29.870	201201.700 210767.180

EAST RETAINING WALL				
CONTROL	CURVE	POINT	STATION	COORDINATES
				NORTH EAST
R. BOUND COLLECTOR RD. EXIT	P.C.	P.C.	76+39.000	201607.070 210467.330
		P.T.	76+39.000	201607.287 210776.164
		P.L.	76+67.300	201608.000 210771.431
		P.C.	76+70.400	201609.720 210770.044
		P.L.	77+70.293	201709.775 210764.203
		E.C.	78+08.593	201809.499 210760.078
		E.C.	79+08.078	201909.000 210760.000
		E.C.	79+30.704	201970.000 210760.000
		E.C.	80+07.120	201970.000 210760.000
		P.L.	80+07.120	201970.000 210760.000
		P.C.	80+78.400	201971.000 210760.000

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
 KITCHENER-WATERLOO EXPRESSWAY
 FREDERICK STREET UNDERPASS
 KING'S HIGHWAY DRYDEN BLVD. DIST. No. 4
 CO. WATERLOO
 CITY OF KITCHENER LOT CON.
 GEOMETRICAL ALIGNMENT OF RETAINING WALLS
 APPROVED [Signature] SITE No. 33-234 W.P. No. 634-64
 DRAWING [Signature] CHECK [Signature] CONTRACT No. 63-62
 DATE JUNE 67 DRAWING No. D-6049-11

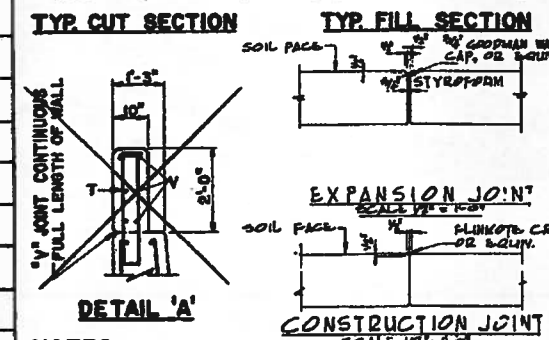
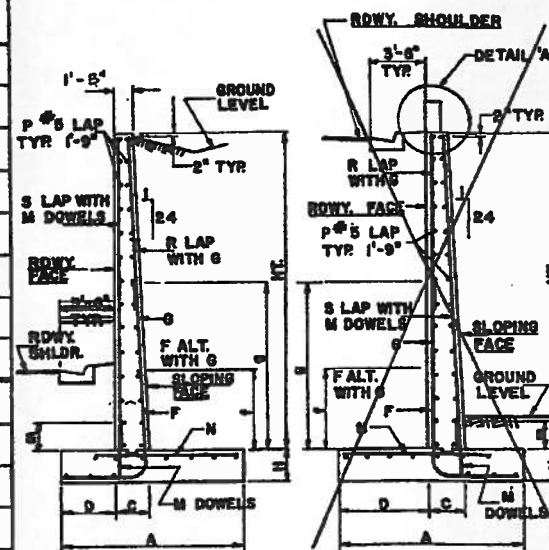




REVISIONS	DATE	BY	DESCRIPTION
1			CATCH BASIN RECESS ADDED

DEPARTMENT OF HIGHWAYS ONTARIO			
A.D. MARGISON AND ASSOCIATES LIMITED CONSULTING PROFESSIONAL ENGINEERS			
KITCHENER-WATERLOO EXPRESSWAY FREDERICK STREET UNDERPASS			
KING'S HIGHWAY DRYDEN BLVD.		DIST. No. 4	
CO. WATERLOO			
CITY OF KITCHENER		LOT CON.	
ELEVATIONS OF RETAINING WALLS			
APPROVED	DESIGN	CHECK	DATE
	B.L.	L.	June '67
DESIGNED	CHECKED	DATE	
B.L.	L.	June '67	
CONTRACT No.		DRAWING No.	
33-234		D-6049-12	

REINFORCING STEEL FOR PANEL														PANEL DESCRIPTION			DATA AT STATIONS					SECTION DIMENSIONS AT STATIONS										
F		G		M		N		P - HOR.		S PER LINE		R		S		PARAPET WALL		PANEL NUMBER	LENGTH OF PANEL	TYPE OF SECTION CUT (FILL)	STATION	TOP OF FOOTING ELEV.	TOP OF WALL ELEV.	WALL HEIGHT	JOINT CENTER	C	A	D	H	I	G	M
VERT.	VERT.	DOWELS	TOP OF FTS.	TOP OF FTS.	BOT. OF FTS.	SLOPING FACE	ROADWAY FACE	VERT.	VERT.	VERT.	VERT.	HOR. 2/LINE	V	HOR.	VERT.																	
20-4 2005	27-4 2013	21-4 2004	27-4 1120	7-4 2001	2-4 2001	15-4 2003	15-4 2003	27-4 2028	21-4 2005	20-4 2002		1	80'-0"	X	70+07.770	1048.50	1060.25	20'-0"	X	2'-1 1/2'	12'-0"	5'-6"	2'-0"	5'-0"	11'-6"	3'-0"	5'-0"	11'-6"	3'-0"	5'-0"	11'-6"	3'-0"
10-4 2006	27-4 2014	20-4 2005	27-4 1021	DO	DO	14-4 2003	14-4 2003	DO	21-4 2020	DO		2	DO		77+770	DO	1060.25	15'-10"	X	2'-1'	11'-5"	5'-6"	DO	DO	10'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 1021	DO	DO	14-4 2003	14-4 2003	DO	21-4 2020	DO		3	DO		77+770	DO	1060.25	15'-11"	X	2'-0 1/2'	10'-10"	5'-6"	DO	DO	5'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 1021	DO	DO	14-4 2003	14-4 2003	DO	21-4 2020	DO		4	DO		77+770	DO	1060.25	15'-0"	X	2'-0'	10'-3"	5'-6"	DO	DO	5'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		5	DO		80+27.770	DO	1060.27	17'-0"	X	1'-1 1/2'	9'-9"	5'-0"	DO	DO	4'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		6	DO		77+770	DO	1060.25	16'-2"	X	1'-1 1/2'	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		7	DO		77+770	DO	1060.25	15'-2 1/2"	X	1'-10 1/2"	8'-8"	2'-10"	DO	DO	6'-10"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		8	DO		84+77.770	DO	1060.26	14'-4 1/2"	X	1'-6 1/2"	8'-8"	2'-8"	DO	DO	4'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		9	DO		47+770	DO	1061.25	13'-5 1/2"	X	1'-9 1/2"	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		10A	12'-0"		47+770	DO	1061.25	14'-3"	X	1'-10 1/2"	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		10B	10'-0"		81+770	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		10C	DO		91+770	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		11A	12'-0"		90+01.770	DO	1057.20	10'-8 1/2"	X	1'-8 1/2"	6'-9"	2'-0"	DO	5'-6"	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		11B	10'-0"		15+770	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		11C	DO		83+770	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		12A	16'-0"		133+770	DO	1052.20	5'-6"		1'-5 1/2"	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		12B	DO		70+50.280	DO	1062.75	17'-9"	X	1'-10 1/2"	10'-3"	5'-9"	DO	5'-0"	5'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		13A	DO		80+280	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		13B	DO		80+280	DO	1061.28	15'-10 1/2"	X	1'-11'	6'-2"	2'-10"	DO	4'-6"	6'-6"	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		14A	DO		90+280	DO	1058.28	12'-6 1/2"	X	1'-2 1/2"	7'-9"	4'-4"	DO	5'-6"	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		4B	DO		125+280	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		15A	DO		10+280	DO	1053.18	9'-2 1/2"	X	1'-7 1/2"	6'-4"	1'-10"	DO	4'-6"	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO		15B	DO	X	135+280	DO			NO JOINT	DO	DO	DO	DO	DO	DO	DO						
10-4 2006	27-4 2014	DO	27-4 2025	DO	DO	DO	DO	DO	21-4 2028	DO					70+280	DO	1051.23	5'-10"		1'-6"	DO	DO	DO	DO	DO	DO						



NOTES

1. R.S.M AND T BARS TO START AND FINISH 3' FROM JOINTS OR ENDS OF WALL.

2. HORIZONTAL STEEL IS NOT CONTINUOUS THROUGH EXPANSION AND CONSTRUCTION JOINTS.

3. R.S. TO HAVE MIN. 1'-9" LAP WITH G.B.M.

4. FOR LOCATION OF PARAPET RAIL AND SECURITY FENCE SEE ELEVATIONS.

5. FOOTINGS TO BE POURED AGAINST UNDISTURBED GROUND.

6. HORIZONTAL BARS P FANED AT TOP OF WALL WHEN SLOPED.

7. STYROFOAM TO BE HELD IN POSITION BY LIGHT GALV. NAILS.

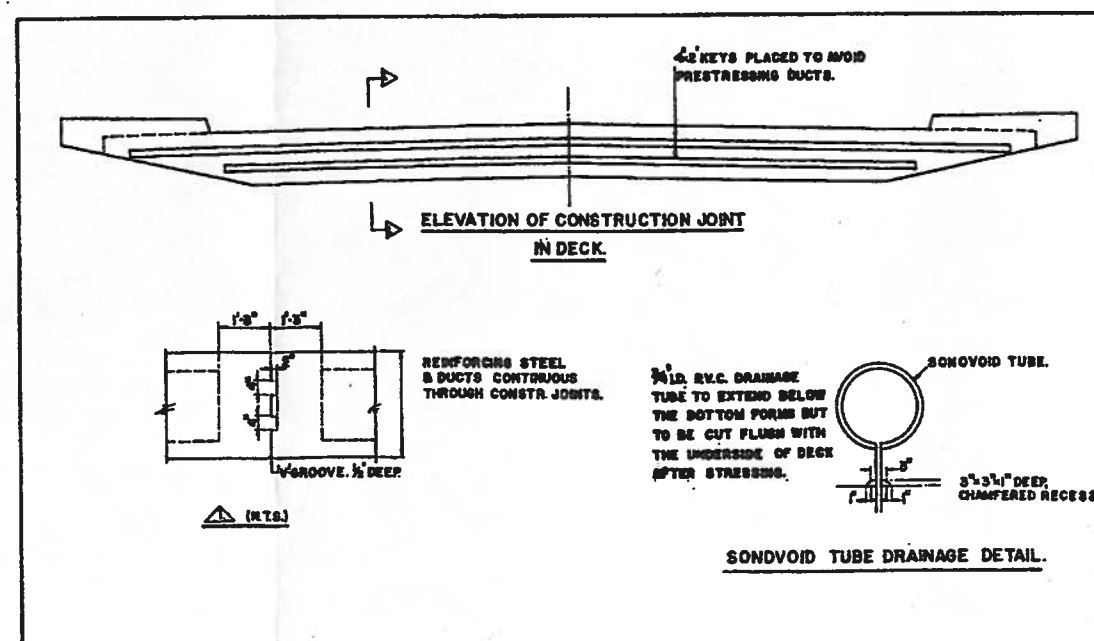
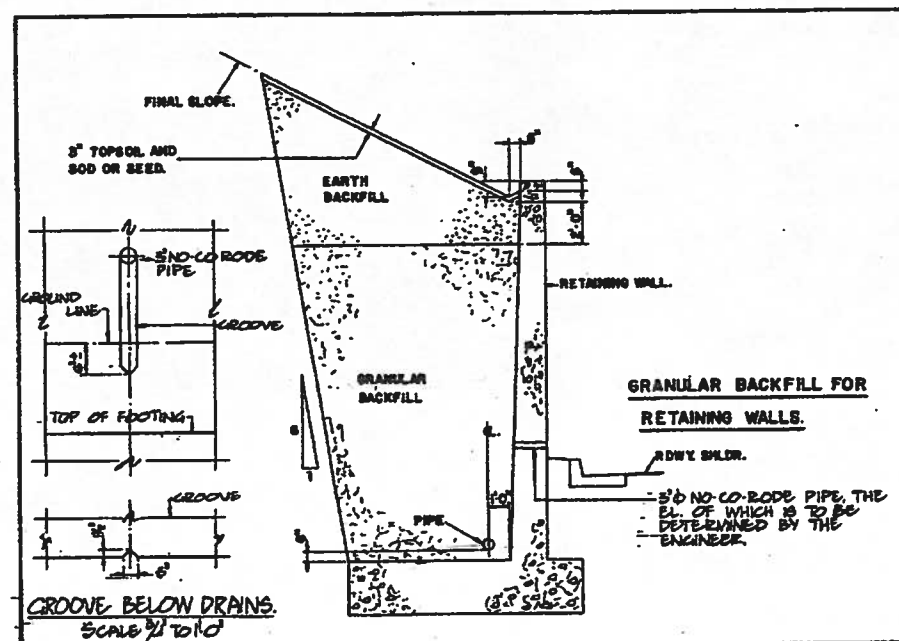
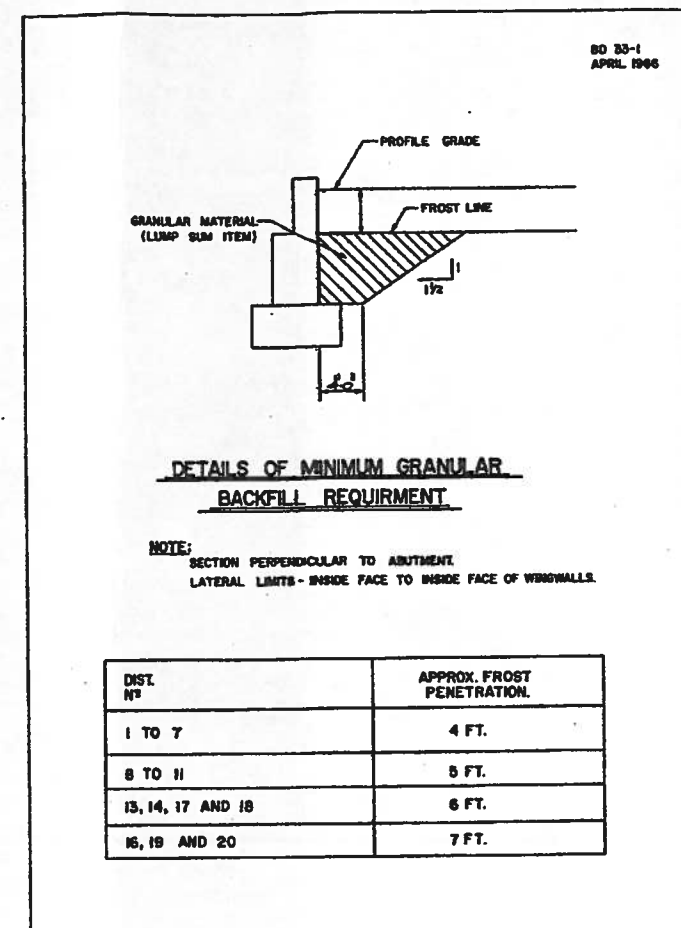
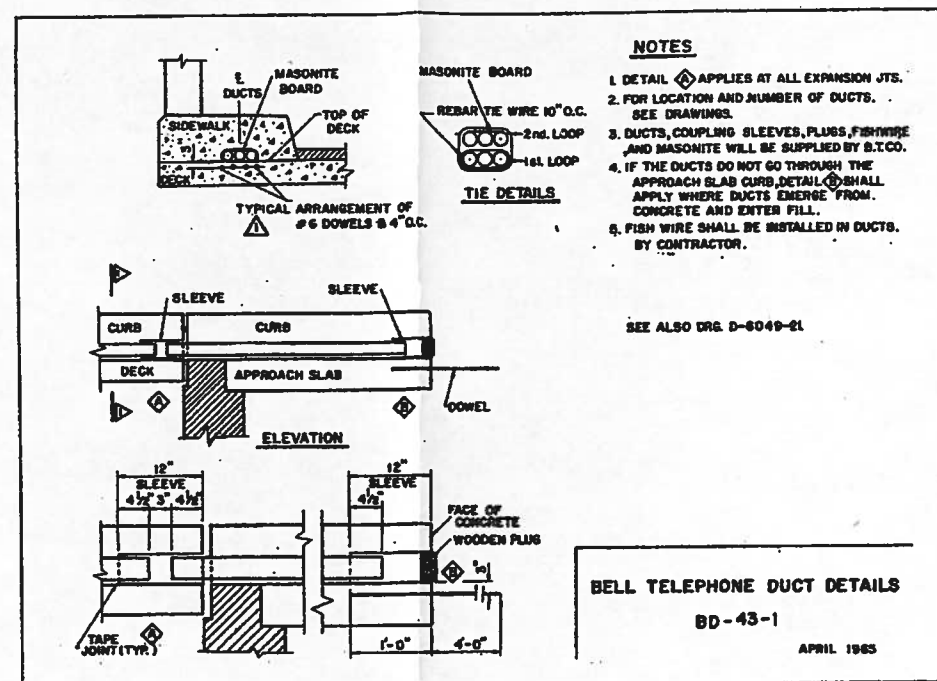
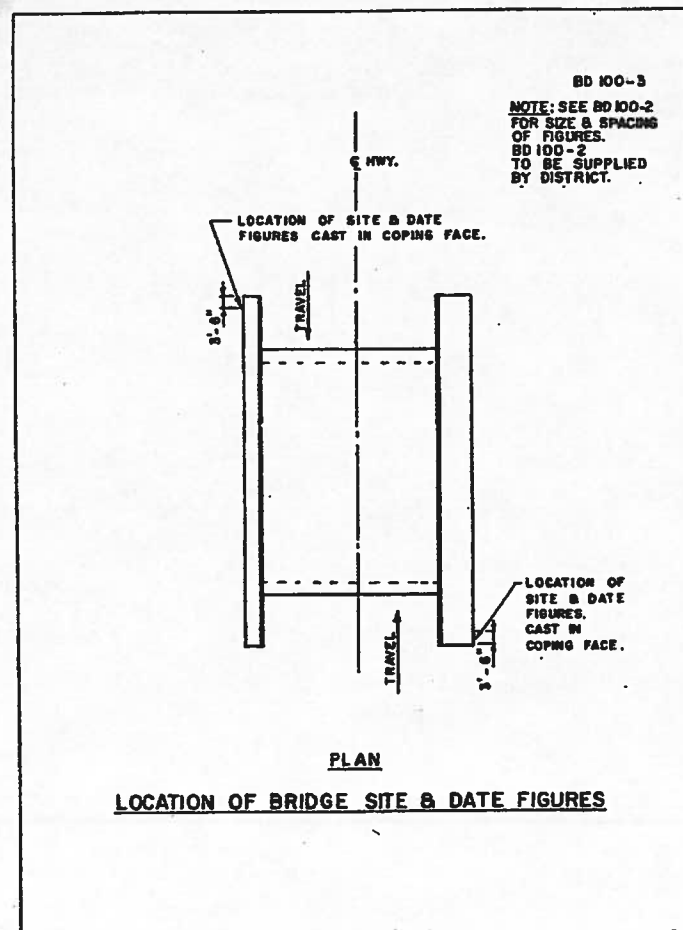
8. STATIONS FOR N.E. RETAINING WALL ARE ON R.E. ALIGNMENT CONTROL.

9. STATIONS FOR N.W. RETAINING WALL ARE ON R.W. ALIGNMENT CONTROL.

10. SEE DEC 2049-11 FOR DETAILS OF SOIL SHEAR KEY.

REVISION	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO			
A.D. MARGISON AND ASSOCIATES LIMITED CONSULTING PROFESSIONAL ENGINEERS			
KITCHENER-WATERLOO EXPRESSWAY FREDERICK STREET UNDERPASS			
RD. 10 HIGHWAY No. DRYDEN BLVD.		REV. No. 4	
CO. WATERLOO		CON.	
CONSTRUCTION TABLE #1			
APPROVED	DESIGNED	CHECKED	DATE
70	33-234	72.12.83	634-64
DESIGNED	CHECKED	DATE	CONTRACT No.
70	72.12.83	634-64	68-62
DATE	DESIGNED	CHECKED	DRAWING No.
70	72.12.83	634-64	D-8049-13



REVISIONS	
DATE	BY

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION
A. D. MARSHON AND ASSOCIATES LIMITED
CONSULTING PROFESSIONAL ENGINEERS

KITCHENER - WATERLOO EXPRESSWAY
FREDERICK STREET UNDERPASS

KING'S HIGHWAY DRYDEN BLVD. DIST. No. 4
CO. WATERLOO
CITY OF KITCHENER NOT CON.

STANDARD DETAILS

DESIGN	ADAPTED	CHECK	DATE	LOADING	REVISION

APPROVED

CONTRACT No.	DATE	REVISION

0-6049-19



APPENDIX FDR-2

COMPUTATIONS

ANALYSIS #1

[illegible]

$$K_a = \frac{\sin^2(\Theta + \varphi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\varphi \sin(\varphi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.20056}{0.99912 \times 0.96112} \right)} \right]^2} = 0.449$$

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} = 3$$

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.449 \times 20 \times 6.32^2 / 2 \\ &= 179.087 \text{ kN/m} \end{aligned}$$

$$P_{av} = P_a \sin (90 - \Theta + \delta)$$

$$= 38.456 \text{ kN / m}$$

$$P_p = K_a \gamma H^2 / 2$$

$$= 3 \times 20 \times 2.31^2 / 2$$

$$= 160.083 \text{ kN / m}$$

Effective passive depth = 2.31

$$P_s = q H = 0 \times 20 \times 1.55$$
$$= 0 \text{ kN/m}$$

$$P_{sh} = 0.449 \times 0 \times 20 \times 5.188$$

$$= 0.00 \text{ kN/m}$$

$$P_c = 0.5 \times 12 \times 2.3 = 13.800 \text{ kN/m}$$

Pih = 0.00 kN / m Factored
Piv = 0 kN / m Unfactored

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =				106.512		145.915	
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.51752	0.804	20	16.08	2.598	41.768
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =				188.420		434.495	

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L \\
 &= 1.25 \times 174.909 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / 9.14 \\
 &= 235.887 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 188.420 + 38.456 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 253.403 \text{ kN / m} > Ph = 235.887 \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L \\
 &= 1 \times 174.909 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 \\
 &= 188.709 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 188.420 + 1 \times 38.456 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 376.785 \text{ kN / m} > 188.709 \quad \text{O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L \\
 &= 1.25 \times 174.909 / 3 \times 6.31552 + 1.25 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.25 \times 13.8 \times (6.32 - 2.3 / 3) + 1 \times 0.00 \times (6.31552 + 0.9) \\
 &= 555.99 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 38.456 \times 3.115 + 145.9 + 434.495 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 730.93 \text{ kN-m} > Ma = 555.99 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L \\
 &= 1.0 \times 174.909 / 3 \times 6.31552 + 1 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.00 \times 13.8 \times (6.32 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.31552 + 0.9) \\
 &= 444.788 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 730.93 \text{ kN-m} > Ma = 444.79 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 188.420 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 368.665 \text{ kN} \\
 Mr &= 730.927 \text{ kN-m} \quad Ma = 555.99 \text{ kN-m} \quad Ma = 555.99 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$\begin{aligned}
 q_u &= c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma \\
 &= 1.5 \times 30.1 \times 1.06 \times 0.407 + 46.20 \times 18.4 \times 1.05 \times 0.407 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.96 \times 0.008 \\
 &= 387.46 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{m \tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.06$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.05$$

$$s_\gamma = 1 - 0.4 (B'/L') = 0.96$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (235.887 / 368.665) = 32.61 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.407$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.008$$

a. Yielding soil condition

Eccentricity limit

$$\begin{aligned}
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V & e \text{ (without vehicle impact)} &= 1.08 \\
 &= 3.115 / 2 - (730.927 - 555.99) / 368.665 \\
 &= 1.083 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}
 \end{aligned}$$

$$\begin{aligned}
 B' &= B - 2e = 3.115 - 2 \times 1.083 = 0.949 \text{ m} & B' \text{ (for footing design)} &= 0.949 \text{ m} \\
 Q &= (235.887^2 + 368.665^2) = 437.672 \\
 q &= \Sigma V / B' L' & \Sigma V &= 368.665 \\
 &= 368.665 / 0.949 \\
 &= 388.409 \text{ kPa} > 387.46 \quad \text{N.G} & Q \text{ (without vehicle impact)} &= 388.45 \text{ kPa}
 \end{aligned}$$

$$\begin{aligned}
 D/B' &= 2.31 / 0.94917 = 2.434 > 0.125 \\
 \Sigma H / \Sigma V &= 235.887 / 368.665 = 0.64 > 0.55 \\
 \text{Reduction factor} &= 1 \times 750 = 750
 \end{aligned}$$

b. Linear elastic non yielding soil

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{368.665}{3.115} \pm \frac{6 \times 368.665 \times 1.083}{3.115^2} \\
 P_b &= \frac{365.204}{-128.51} < 387.46 \quad \text{O.K} & \text{for footing design } P_a &= 365.217
 \end{aligned}$$

SLS

$$\begin{aligned}
 \Sigma V &= 1 \times 106.512 + 1.00 \times 188.420 + 0 + 0 \\
 &= 294.932 \text{ kN} \\
 M_r &= 730.927 \text{ kN-m} & M_a &= 444.788 \text{ kN-m} \\
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{294.932}{3.11517} \pm \frac{6 \times 294.932 \times 0.587}{3.11517^2} \\
 P_b &= \frac{201.717}{-12.364} > 193.73 \quad \text{N.G} \\
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V \\
 &= 3.11517 / 2 - (730.927 - 444.788) / 294.932 \\
 &= 0.587 > B/6 = 3.115 / 6 = 0.519
 \end{aligned}$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 120.850 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188 \\
 &\quad + 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9) \\
 &= 337.505 \text{ kN-m}
 \end{aligned}$$

As Built

At half of wall $t = 0.489 \text{ mm}$

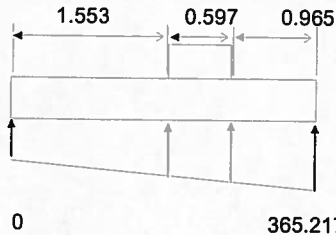
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times H/2 + 1.25 P_s \times \frac{1}{2} \times H/2 + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 30.212 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 64.176 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.949 \text{ m}$

$$M_f = 388.45 \times 0.95 \times 0.95 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 168.125 \text{ kN-m}$$

Heel design : Non yielding soil

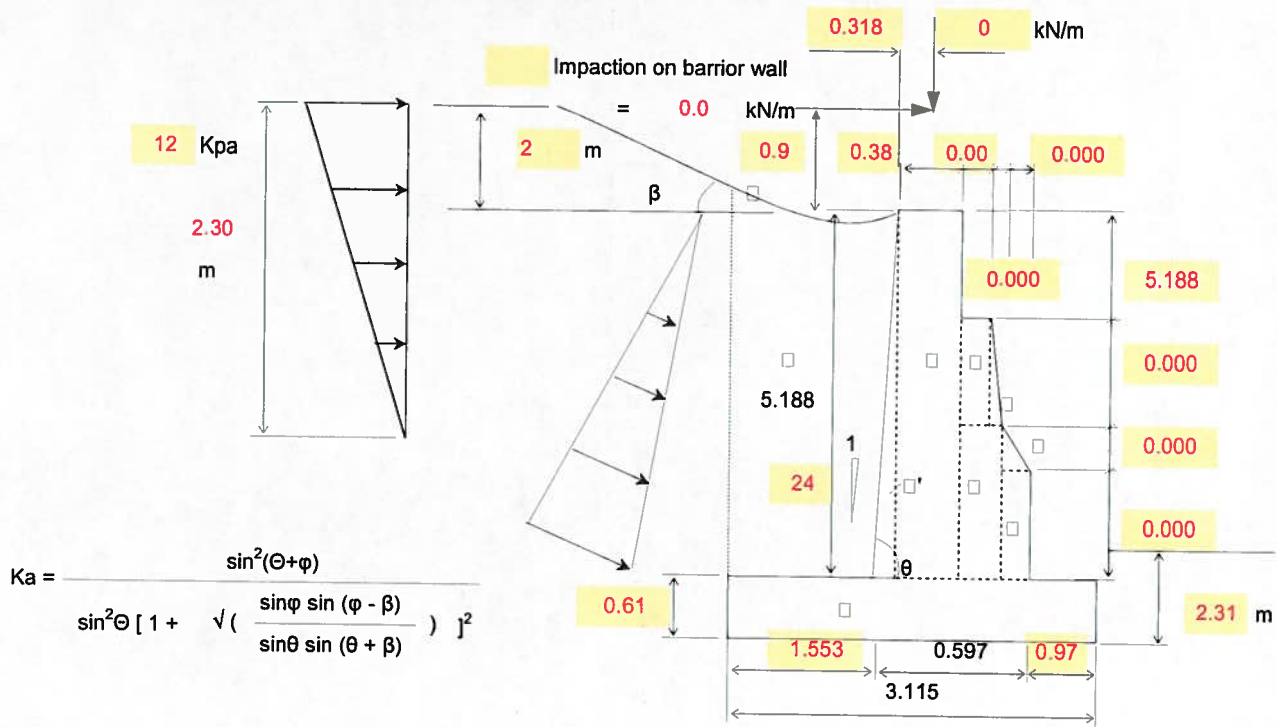


$$\begin{aligned}
 M_f &= (182.071 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 16.08 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 73.234 - 122.695 - 15.490 - 0.000 + 0.808 \\
 &= -64.142 \text{ kN-m}
 \end{aligned}$$

$$M_f = 168.125 \text{ kN-m}$$

ANALYSIS #2

Retaining wall- As Built with Ice Jacking Pressure 10 kPa

a. Load evaluation **Geotechnical design : Do not include compact load and linear distribution for bearing resistance**

$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.20056}{0.99912 \times 0.96112} \right)^2} \right]} = 0.449$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.449 \times 20 \times 6.32^2 / 2 \\ &= 179.087 \text{ kN/m} \\ P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 174.909 \text{ kN/m} \\ P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 38.456 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \\ \text{Effective passive depth} &= 2.31 \end{aligned}$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.449 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =				106.512		145.915	
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.51752	0.804	20	16.08	2.598	41.768
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =				188.420		434.495	

6. Icejacking Pressure assumed 10 kPa
and from top 2.3 m frost susceptible soils
Ice jacking force = 23 kN/m

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 P_h &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L + P_{icejacking} \\
 &= 1.25 \times 174.909 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / \quad 9.14 + 23 \\
 &= 258.887 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= 0.8 \times (106.512 + 188.420 + 38.456 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 253.403 \text{ kN/m} < P_h = 258.887 \quad \text{N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 P_h &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L + P_{icejacking} \\
 &= 1 \times 174.909 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 + 23 \\
 &= 211.709 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= (1 \times 106.512 + 1 \times 188.420 + 1 \times 38.456 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 376.785 \text{ kN/m} > 211.709 \quad \text{O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L + P_{icejacking} \times (H - 2.3/2) \\
 &= 1.25 \times 174.909 / 3 \times 6.31552 + 1.25 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.25 \times 13.8 \times (6.32 - 2.3 / 3) + 1 \times 0.00 \times (6.31552 + 0.9) + 23 \times (H - 2.3/2) \\
 &= 674.79 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 38.456 \times 3.115 + 145.9 + 434.495 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 730.93 \text{ kN-m} > Ma = 674.79 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L + P_{icejacking} \times (H - 2.3/2) \\
 &= 1.0 \times 174.909 / 3 \times 6.31552 + 1 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.00 \times 13.8 \times (6.32 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.31552 + 0.9) + 23 \times (H - 2.3/2) \\
 &= 563.595 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 730.93 \text{ kN-m} > Ma = 563.60 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 188.420 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 368.665 \text{ kN} \\
 Mr &= 730.927 \text{ kN-m} \quad Ma = 674.79 \text{ kN-m} \quad Ma = 674.79 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$q_u = c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma$$

$$\begin{aligned}
 &= 1.5 \times 30.1 \times 1.02 \times 0.372 + 46.20 \times 18.4 \times 1.02 \times 0.372 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.99 \times 0.029 \\
 &= 352.604 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.02$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.02$$

$$s_\gamma = 1 - 0.4 (B'/L') = 0.99$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (258.887 / 368.665) = 35.08 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.372$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.029$$

a. Yielding soil condition

Eccentricity limit

$$e = B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V$$

$$e \text{ (without vehicle impact)} = 1.41$$

$$= 3.115 / 2 - (730.927 - 674.79) / 368.665$$

$$= 1.405 \text{ m} > 0.3B = 0.93 \text{ m} \text{ N.G. MTO Seminar 1 and 2 Section 6 Foundation}$$

$$B' = B - 2e = 3.115 - 2 \times 1.405 = 0.305 \text{ m} \quad B' \text{ (for footing design)} = 0.305 \text{ m}$$

$$Q = (258.887^2 + 368.665^2) = 450.484$$

$$q = \Sigma V / B' L' \quad \Sigma V = 368.665$$

$$= 368.665 / 0.305$$

$$= 1208.08 \text{ kPa} > 352.604 \text{ N.G.} \quad Q \text{ (without vehicle impact)} = 1210.60 \text{ kPa}$$

$$D / B' = 2.31 / 0.30517 = 7.570 > 0.125$$

$$\Sigma H / \Sigma V = 258.887 / 368.665 = 0.70 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{368.665}{3.115} \pm \frac{6 \times 368.665 \times 1.405}{3.115^2}$$

$$P_b = \frac{438.601}{-201.91} > 352.604 \text{ N.G.} \quad \text{for footing design } P_a = 438.673$$

SLS

$$\Sigma V = 1 \times 106.512 + 1.00 \times 188.420 + 0 + 0$$

$$= 294.932 \text{ kN}$$

$$M_r = 730.927 \text{ kN-m} \quad M_a = 563.595 \text{ kN-m}$$

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{294.932}{3.11517} \pm \frac{6 \times 294.932 \times 0.99}{3.11517^2}$$

$$P_b = \frac{275.205}{-85.852} > 176.302 \text{ N.G.}$$

$$e = B/2 - (\Sigma M_r - \Sigma M_o) / \Sigma V$$

$$= 3.11517 / 2 - (730.927 - 563.595) / 294.932$$

$$= 0.99 > B/6 = 3.115 / 6 = 0.519$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$M_f = 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h)$$

$$= 1.25 \times 120.850 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188$$

$$+ 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9)$$

$$= 337.505 \text{ kN-m}$$

At half of wall $t = 0.489 \text{ mm}$

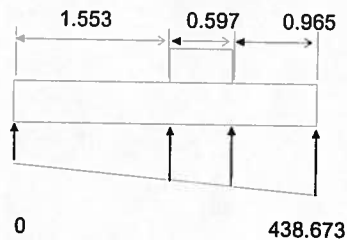
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times H/2 + 1.25 P_s \times \frac{1}{2} \times H/2 + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 30.212 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 64.176 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.305 \text{ m}$

$$M_f = 1210.60 \times 0.3 \times 0.3 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 49.318 \text{ kN-m}$$

Heel design : Non yielding soil

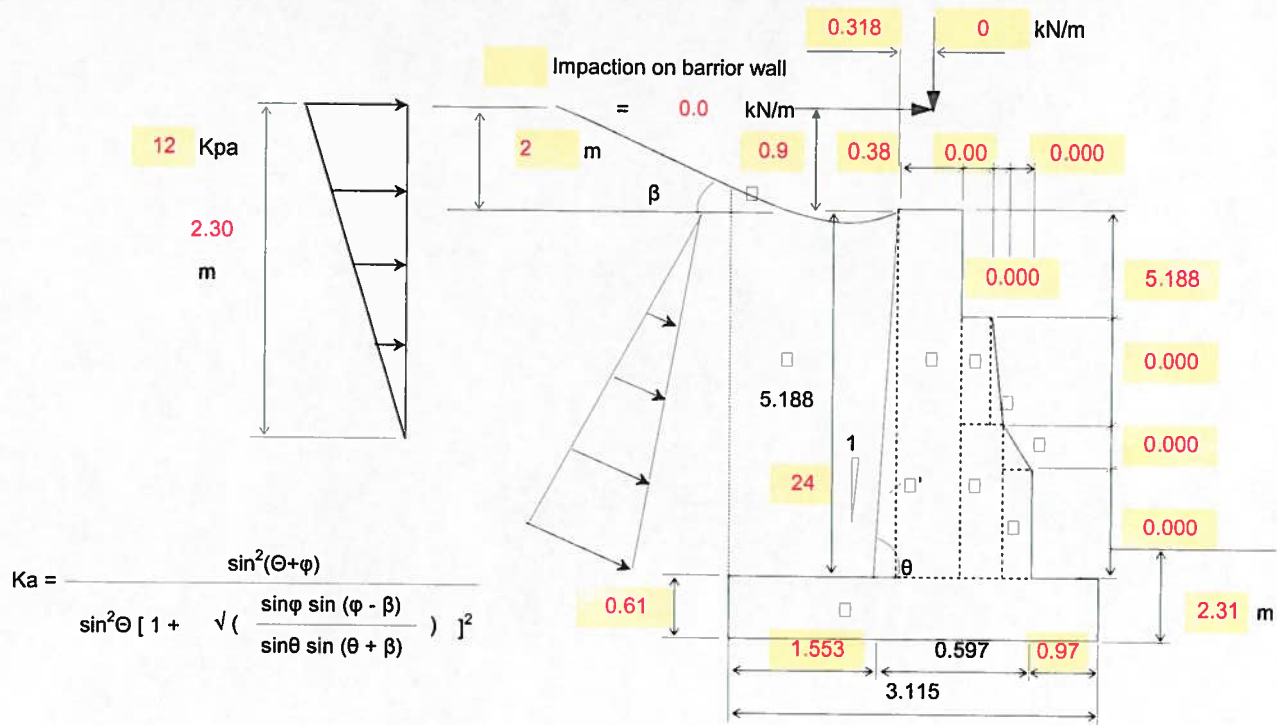


$$\begin{aligned}
 M_f &= (218.691 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 16.08 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 87.964 - 122.695 - 15.490 - 0.000 + 0.808 \\
 &= -49.412 \text{ kN-m}
 \end{aligned}$$

$$M_f = 49.318 \text{ kN-m}$$

ANALYSIS #3

Retaining wall- As Built with Ice Jacking Pressure 20 kPa

a. Load evaluation **Geotechnical design : Do not include compact load and linear distribution for bearing resistance**

$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.20056}{0.99912 \times 0.96112} \right)^2} \right]} = 0.449$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.449 \times 20 \times 6.32^2 / 2 \\ &= 179.087 \text{ kN / m} \\ P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 174.909 \text{ kN / m} \\ P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 38.456 \text{ kN / m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN / m} \\ \text{Effective passive depth} &= 2.31 \end{aligned}$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN / m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.449 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN / m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN / m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN / m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN / m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =					106.512		145.915
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.51752	0.804	20	16.08	2.598	41.768
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =					188.420		434.495

6. Icejacking Pressure assumed 20 kPa

and from top 2.3 m frost susceptible soils

Ice jacking force = 46 kN/m

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 P_h &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L + P_{icejacking} \\
 &= 1.25 \times 174.909 + 1.25 \times 0.000 + 1.25 \times 13.800 + 9.14 + 46 \\
 &= 281.887 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= 0.8 \times (106.512 + 188.420 + 38.456 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 253.403 \text{ kN / m} < P_h = 281.887 \text{ N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 P_h &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L + P_{icejacking} \\
 &= 1 \times 174.909 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 + 46 \\
 &= 234.709 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= (1 \times 106.512 + 1 \times 188.420 + 1 \times 38.456 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 376.785 \text{ kN / m} > 234.709 \text{ O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L + P_{icejacking} \times (H - 2.3/2) \\
 &= 1.25 \times 174.909 / 3 \times 6.31552 + 1.25 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.25 \times 13.8 \times (6.32 - 2.3 / 3) + 1 \times 0.00 \times (6.31552 + 0.9) + 46 \times (H - 2.3/2) \\
 &= 793.60 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 38.456 \times 3.115 + 145.9 + 434.495 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 730.93 \text{ kN-m} < Ma = 793.60 \text{ kN-m} \quad \text{N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L + P_{icejacking} \times (H - 2.3/2) \\
 &= 1.0 \times 174.909 / 3 \times 6.31552 + 1 \times 0.000 / 2 \times 6.31552 \\
 &\quad + 1.00 \times 13.8 \times (6.32 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.31552 + 0.9) + 46 \times (H - 2.3/2) \\
 &= 682.402 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 730.93 \text{ kN-m} > Ma = 682.40 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 188.420 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 368.665 \text{ kN} \\
 Mr &= 730.927 \text{ kN-m} \quad Ma = 793.60 \text{ kN-m} \quad Ma = 793.60 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$q_u = c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma$$

$$\begin{aligned}
 &= 1.5 \times 30.1 \times 0.98 \times 0.342 + 46.20 \times 18.4 \times 0.98 \times 0.342 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 1.01 \times 0.061 \\
 &= 328.736 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{m \tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 0.98$$

$$s_q = 1 + (B'/L') \tan \Phi' = 0.98$$

$$s_\gamma = 1 - 0.4 (B'/L') = 1.01$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (281.887 / 368.665) = 37.40 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.342$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.061$$

a. Yielding soil condition

Eccentricity limit

$$e = B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V \quad e \text{ (without vehicle impact)} = 1.73$$

$$= 3.115 / 2 - (730.927 - 793.60) / 368.665$$

$$= 1.728 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G.} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}$$

$$B' = B - 2e = 3.115 - 2 \times 1.728 = -0.341 \text{ m} \quad B' \text{ (for footing design)} = -0.340 \text{ m}$$

$$Q = (281.887^2 + 368.665^2) = 464.084$$

$$q = \Sigma V / B' L' \quad \Sigma V = 368.665$$

$$= 368.665 / -0.341$$

$$= -1081.7 \text{ kPa} < 328.736 \quad \text{O.K.} \quad Q \text{ (without vehicle impact)} = -1084.33 \text{ kPa}$$

$$D / B' = 2.31 / -0.3408 = -6.778 < 0.125$$

$$\Sigma H / \Sigma V = 281.887 / 368.665 = 0.76 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{368.665}{3.115} \pm \frac{6 \times 368.665 \times 1.728}{3.115^2}$$

$$P_b = \frac{512.226}{-275.54} > 328.736 \quad \text{N.G.} \quad \text{for footing design} \quad P_a = 512.13$$

SLS

$$\Sigma V = 1 \times 106.512 + 1.00 \times 188.420 + 0 + 0$$

$$= 294.932 \text{ kN}$$

$$M_r = 730.927 \text{ kN-m} \quad M_a = 682.402 \text{ kN-m}$$

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{294.932}{3.11517} \pm \frac{6 \times 294.932 \times 1.393}{3.11517^2}$$

$$P_b = \frac{348.693}{-159.34} > 164.368 \quad \text{N.G.}$$

$$e = B/2 - (\Sigma M_r - \Sigma M_o) / \Sigma V$$

$$= 3.11517 / 2 - (730.927 - 682.402) / 294.932$$

$$= 1.393 > B/6 = 3.115 / 6 = 0.519$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$M_f = 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h)$$

$$= 1.25 \times 120.850 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188$$

$$+ 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9)$$

$$= 337.505 \text{ kN-m}$$

At half of wall $t = 0.489$ mm

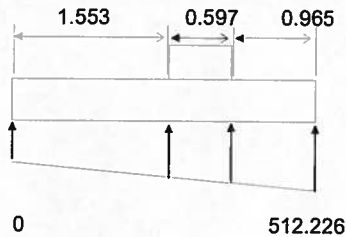
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times \frac{H}{2} + 1.25 P_s \times \frac{1}{2} \times \frac{H}{2} + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 30.212 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 64.176 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = -0.341$ m

$$M_f = -1081.66 \times -0.34 \times -0.34 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = -69.643 \text{ kN-m}$$

Heel design : Non yielding soil



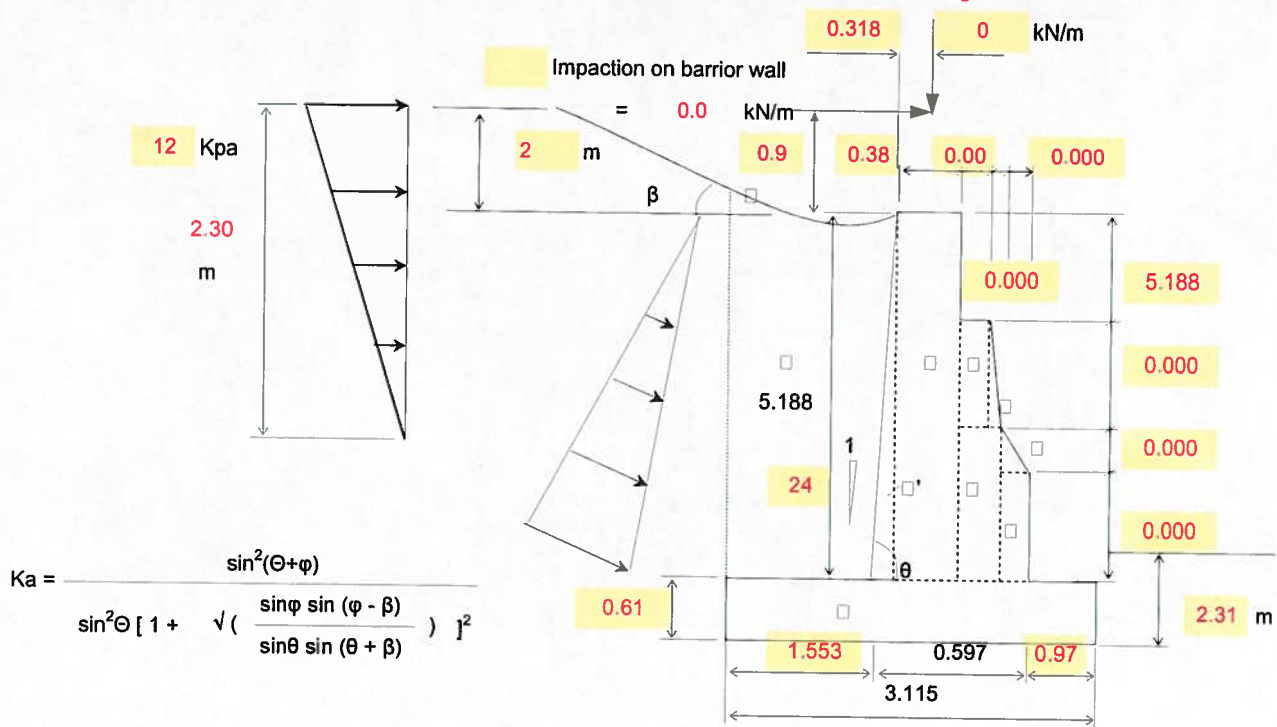
$$\begin{aligned}
 M_f &= (255.359 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 16.08 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 102.712 - 122.695 - 15.490 - 0.000 + 0.808 \\
 &= -34.663 \text{ kN-m}
 \end{aligned}$$

$$M_f = -34.663 \text{ kN-m}$$

ANALYSIS #1A

Retaining wall- Actual Wall Inclination with No Ice Jacking Pressure

a. Load evaluation **Geotechnical design : Do not include compact load and linear distribution for bearing resistance**



$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.16505}{0.99912 \times 0.95052} \right)^2} \right]} = 0.469$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.469 \times 20 \times 6.38^2 / 2 \\ &= 190.822 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 186.371 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 40.976 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \end{aligned}$$

$$\text{Effective passive depth} = 2.31$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.469 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =				106.512		145.915	
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.58064	0.902	20	18.04	2.598	46.859
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =				190.380		439.586	

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L \\
 &= 1.25 \times 186.371 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / 9.14 \\
 &= 250.213 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 190.380 + 40.976 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 255.733 \text{ kN/m} > Ph = 250.213 \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L \\
 &= 1 \times 186.371 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 \\
 &= 200.171 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 190.380 + 1 \times 40.976 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 379.697 \text{ kN/m} > 200.171 \quad \text{O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L \\
 &= 1.25 \times 186.371 / 3 \times 6.37864 + 1.25 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.25 \times 13.8 \times (6.38 - 2.3 / 3) + 1 \times 0.00 \times (6.37864 + 0.9) \\
 &= 592.14 \text{ kN-m}
 \end{aligned}$$

$$\begin{aligned}
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 40.976 \times 3.115 + 145.9 + 439.586 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 745.83 \text{ kN-m} > Ma = 592.14 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L \\
 &= 1.0 \times 186.371 / 3 \times 6.37864 + 1 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.00 \times 13.8 \times (6.38 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.37864 + 0.9) \\
 &= 473.709 \text{ kN-m}
 \end{aligned}$$

$$\begin{aligned}
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 745.83 \text{ kN-m} > Ma = 473.71 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 190.380 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 371.115 \text{ kN}
 \end{aligned}$$

$$Mr = 745.831 \text{ kN-m} \quad Ma = 592.14 \text{ kN-m} \quad Ma = 592.14 \text{ kN-m (For footing design)}$$

$$q_u = c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma$$

$$\begin{aligned}
 &= 1.5 \times 30.1 \times 1.05 \times 0.387 + 46.20 \times 18.4 \times 1.05 \times 0.387 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.97 \times 0.018 \\
 &= 371.454 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.05$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.05$$

$$s_\gamma = 1 - 0.4(B'/L') = 0.97$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (250.213 / 371.115) = 33.99 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.387$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.018$$

a. Yielding soil condition

Eccentricity limit

$$\begin{aligned}
 e &= B/2 - (\Sigma Mr - \Sigma Ma) / \Sigma V & e \text{ (without vehicle impact)} &= 1.14 \\
 &= 3.115 / 2 - (745.831 - 592.14) / 371.115 \\
 &= 1.143 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}
 \end{aligned}$$

$$B' = B - 2e = 3.115 - 2 \times 1.143 = 0.829 \text{ m} \quad B' \text{ (for footing design)} = 0.828 \text{ m}$$

$$Q = (250.213^2 + 371.115^2) = 447.586$$

$$\begin{aligned}
 q &= \Sigma V / B' L' & \Sigma V &= 371.115 \\
 &= 371.115 / 0.829 \\
 &= 447.576 \text{ kPa} > 371.454 \quad \text{N.G} & Q \text{ (without vehicle impact)} &= 448.05 \text{ kPa}
 \end{aligned}$$

$$D / B' = 2.31 / 0.82917 = 2.786 > 0.125$$

$$\Sigma H / \Sigma V = 250.213 / 371.115 = 0.67 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{371.115}{3.115} \pm \frac{6 \times 371.115 \times 1.143}{3.115^2} \\
 P_b &= \frac{381.399}{-143.14} > 371.454 \quad \text{N.G} & \text{for footing design } P_a &= 381.5
 \end{aligned}$$

SLS

$$\begin{aligned}
 \Sigma V &= 1 \times 106.512 + 1.00 \times 190.380 + 0 + 0 \\
 &= 296.892 \text{ kN}
 \end{aligned}$$

$$Mr = 745.831 \text{ kN-m} \quad Ma = 473.709 \text{ kN-m}$$

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{296.892}{3.11517} \pm \frac{6 \times 296.892 \times 0.641}{3.11517^2} \\
 P_b &= \frac{212.97}{-22.359} > 185.727 \quad \text{N.G}
 \end{aligned}$$

$$\begin{aligned}
 e &= B/2 - (\Sigma Mr - \Sigma Mo) / \Sigma V \\
 &= 3.11517 / 2 - (745.831 - 473.709) / 296.892 \\
 &= 0.641 > B/6 = 3.115 / 6 = 0.519
 \end{aligned}$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 126.233 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188 \\
 &\quad + 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9) \\
 &= 349.142 \text{ kN-m}
 \end{aligned}$$

At half of wall $t = 0.489 \text{ mm}$

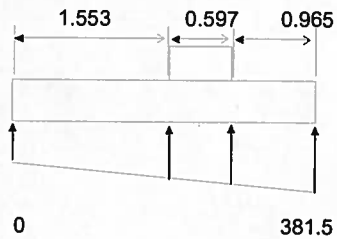
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times H/2 + 1.25 P_s \times \frac{1}{2} \times H/2 + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 31.558 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 65.631 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.828 \text{ m}$

$$M_f = 448.05 \times 0.83 \times 0.83 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 146.878 \text{ kN-m}$$

Heel design : Non yielding soil



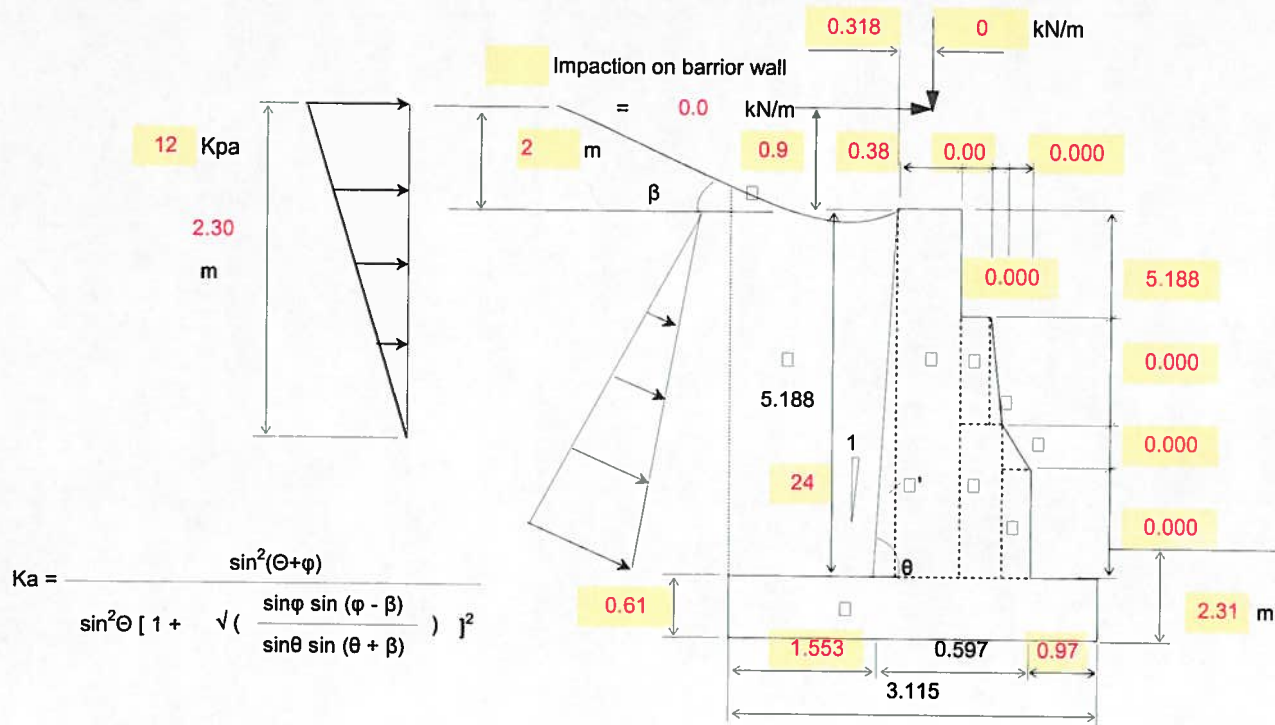
$$\begin{aligned}
 M_f &= (190.189 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.04 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 76.499 - 122.695 - 17.378 - 0.000 + 0.808 \\
 &= -62.765 \text{ kN-m}
 \end{aligned}$$

$$M_f = 146.878 \text{ kN-m}$$

ANALYSIS #2A

Retaining wall - Actual Wall Inclination with 10 kPa Ice Jacking Pressure

a. Load evaluation **Geotechnical design : Do not include compact load and linear distribution for bearing resistance**



$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.16505}{0.99912 \times 0.95052} \right)^2} \right]} = 0.469$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.469 \times 20 \times 6.38^2 / 2 \\ &= 190.822 \text{ kN/m} \\ P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 186.371 \text{ kN/m} \\ P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 40.976 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \\ \text{Effective passive depth} &= 2.31 \end{aligned}$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.469 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =					106.512		145.915
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.58064	0.902	20	18.04	2.598	46.859
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =					190.380		439.586

6. Icejacking Pressure assumed 10 kPa
and from top 2.3 m frost susceptible soils
Ice jacking force = 23 kN/m

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 P_h &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L + P_{icejacking} \\
 &= 1.25 \times 186.371 + 1.25 \times 0.000 + 1.25 \times 13.800 + 9.14 + 23 \\
 &= 273.213 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= 0.8 \times (106.512 + 190.380 + 40.976 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 255.733 \text{ kN/m} < P_h = 273.213 \text{ N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 P_h &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L + P_{icejacking} \\
 &= 1 \times 186.371 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 + 23 \\
 &= 223.171 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 P_{rh} &= (1 \times 106.512 + 1 \times 190.380 + 1 \times 40.976 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 379.697 \text{ kN/m} > 223.171 \text{ O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L + 1.25 P_{icejacking} \times (H - \\
 &= 1.25 \times 186.371 / 3 \times 6.37864 + 1.25 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.25 \times 13.8 \times (6.38 - 2.3 / 3) + 1 \times 0.00 \times (6.37864 + 0.9) + 1.25 \times 23 \times (H - 2.3 / 2) \\
 &= 742.46 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 40.976 \times 3.115 + 145.9 + 439.586 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 745.83 \text{ kN-m} > Ma = 742.46 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L + 1.0 P_{icejacking} \times (H - 2.3 / 2) \\
 &= 1.0 \times 186.371 / 3 \times 6.37864 + 1 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.00 \times 13.8 \times (6.38 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.37864 + 0.9) + 23 \times (H - 2.3 / 2) \\
 &= 593.968 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 745.83 \text{ kN-m} > Ma = 593.97 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 190.380 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 371.115 \text{ kN} \\
 Mr &= 745.831 \text{ kN-m} \quad Ma = 742.46 \text{ kN-m} \quad Ma = 742.46 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$q_u = c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma$$

$$\begin{aligned}
 &= 1.5 \times 30.1 \times 1.00 \times 0.355 + 46.20 \times 18.4 \times 1.00 \times 0.355 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 1.00 \times 0.045 \\
 &= 339.438 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{m \tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.00$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.00$$

$$s_\gamma = 1 - 0.4 (B'/L') = 1.00$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (273.213 / 371.115) = 36.36 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.355$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.045$$

a. Yielding soil condition

Eccentricity limit

$$e = B/2 - (\Sigma Mr - \Sigma Ma) / \Sigma V \quad e \text{ (without vehicle impact)} = 1.55$$

$$= 3.115 / 2 - (745.831 - 742.46) / 371.115$$

$$= 1.549 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}$$

$$B' = B - 2e = 3.115 - 2 \times 1.549 = 0.017 \text{ m} \quad B' \text{ (for footing design)} = 0.018 \text{ m}$$

$$Q = (273.213^2 + 371.115^2) = 460.838$$

$$q = \Sigma V / B' L' \quad \Sigma V = 371.115$$

$$= 371.115 / 0.017$$

$$= 21618.3 \text{ kPa} > 339.438 \quad \text{N.G} \quad Q \text{ (without vehicle impact)} 20428.38 \text{ kPa}$$

$$D/B' = 2.31 / 0.01717 = 134.563 > 0.125$$

$$\Sigma H / \Sigma V = 273.213 / 371.115 = 0.74 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$Pa = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{371.115}{3.115} \pm \frac{6 \times 371.115 \times 1.549}{3.115^2}$$

$$Pb = \frac{474.557}{-236.29} > 339.438 \quad \text{N.G} \quad \text{for footing design} \quad Pa = 474.442$$

SLS

$$\Sigma V = 1 \times 106.512 + 1.00 \times 190.380 + 0 + 0$$

$$= 296.892 \text{ kN}$$

$$Mr = 745.831 \text{ kN-m} \quad Ma = 593.968 \text{ kN-m}$$

$$Pa = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{296.892}{3.11517} \pm \frac{6 \times 296.892 \times 1.046}{3.11517^2}$$

$$Pb = \frac{287.313}{-96.702} > 169.719 \quad \text{N.G}$$

$$e = B/2 - (\Sigma Mr - \Sigma Mo) / \Sigma V$$

$$= 3.11517 / 2 - (745.831 - 593.968) / 296.892$$

$$= 1.046 > B/6 = 3.115 / 6 = 0.519$$

c. Structural design

At the top of footing $t = 0.597 \text{ mm}$

$$M_f = 1.25 P_{ah} \times 1/3 \times H + 1.25 P_{s} \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h)$$

$$= 1.25 \times 126.233 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188$$

$$+ 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9)$$

$$= 349.142 \text{ kN-m}$$

At half of wall $t = 0.489 \text{ mm}$

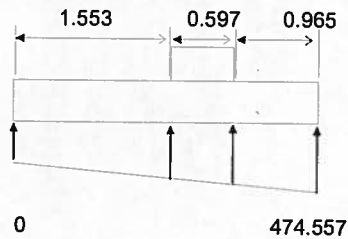
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times H/2 + 1.25 P_s \times \frac{1}{2} \times H/2 + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 31.558 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 65.631 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.017 \text{ m}$

$$M_f = 21618.3 \times 0.02 \times 0.02 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = -3.631 \text{ kN-m}$$

Heel design : Non yielding soil



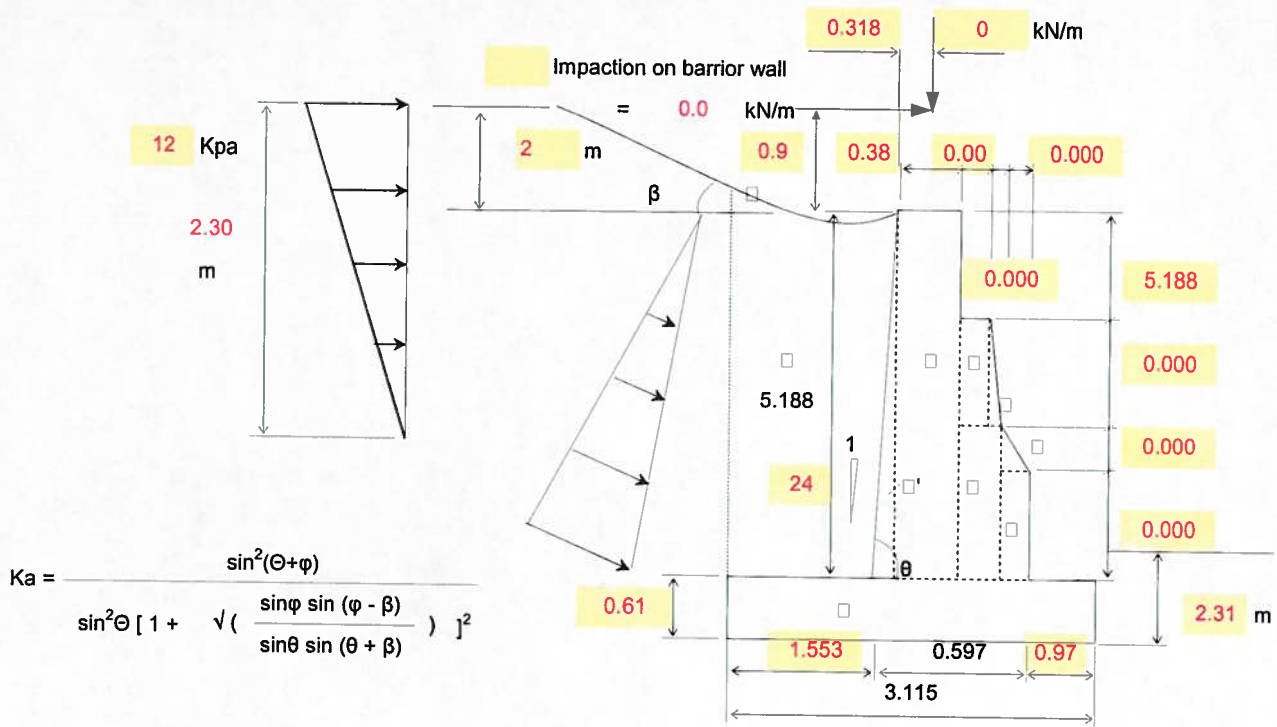
$$\begin{aligned}
 M_f &= (236.58 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.04 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 95.159 - 122.695 - 17.378 - 0.000 + 0.808 \\
 &= -44.105 \text{ kN-m}
 \end{aligned}$$

$$M_f = -3.631 \text{ kN-m}$$

ANALYSIS #3A

Retaining wall- Actual Wall Inclination with Ice Jacking Pressure 20 kPa

a. Load evaluation Geotechnical design : Do not include compact load and linear distribution for bearing resistance



$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.16505}{0.99912 \times 0.95052} \right)^2} \right]} = 0.469$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.469 \times 20 \times 6.38^2 / 2 \\ &= 190.822 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 186.371 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 40.976 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \end{aligned}$$

Effective passive depth = 2.31

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.469 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =	106.512					145.915	
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.58064	0.902	20	18.04	2.598	46.859
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =	190.380					439.586	

6. Icejacking Pressure assumed 20 kPa
and from top 2.3 m frost susceptible soils
Ice jacking force = 46 kN/m

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L + P_{icejacking} \\
 &= 1.25 \times 186.371 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / \quad 9.14 + 46 \\
 &= 296.213 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 190.380 + 40.976 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 255.733 \text{ kN/m} < Ph = 296.213 \text{ N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L + P_{icejacking} \\
 &= 1 \times 186.371 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 + 46 \\
 &= 246.171 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 190.380 + 1 \times 40.976 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 379.697 \text{ kN/m} > 246.171 \text{ O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L + 1.25 P_{icejacking} \times (H - 2.3/2) \\
 &= 1.25 \times 186.371 / 3 \times 6.37864 + 1.25 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.25 \times 13.8 \times (6.38 - 2.3 / 3) + 1 \times 0.00 \times (6.37864 + 0.9) + 46 \times (H - 2.3/2) \\
 &= 832.65 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 40.976 \times 3.115 + 145.9 + 439.586 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 745.83 \text{ kN-m} < Ma = 832.65 \text{ kN-m} \quad \text{N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L + 1.0 P_{icejacking} \times (H - 2.3/2) \\
 &= 1.0 \times 186.371 / 3 \times 6.37864 + 1 \times 0.000 / 2 \times 6.37864 \\
 &\quad + 1.00 \times 13.8 \times (6.38 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.37864 + 0.9) + 46 \times (H - 2.3/2) \\
 &= 714.227 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 745.83 \text{ kN-m} > Ma = 714.23 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 190.380 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 371.115 \text{ kN} \\
 Mr &= 745.831 \text{ kN-m} \quad Ma = 832.65 \text{ kN-m} \quad Ma = 832.65 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$\begin{aligned}
 q_u &= c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma \\
 &= 1.5 \times 30.1 \times 0.97 \times 0.326 + 46.20 \times 18.4 \times 0.97 \times 0.326 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 1.02 \times 0.082 \\
 &= 323.414 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 0.97$$

$$s_q = 1 + (B'/L') \tan \Phi' = 0.97$$

$$s_\gamma = 1 - 0.4 (B'/L') = 1.02$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (296.213 / 371.115) = 38.60 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.326$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.082$$

a. Yielding soil condition

Eccentricity limit

$$\begin{aligned}
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V & e \text{ (without vehicle impact)} &= 1.79 \\
 &= 3.115 / 2 - (745.831 - 832.65) / 371.115 \\
 &= 1.792 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}
 \end{aligned}$$

$$B' = B - 2e = 3.115 - 2 \times 1.792 = -0.469 \text{ m} \quad B' \text{ (for footing design)} = -0.468 \text{ m}$$

$$Q = (296.213^2 + 371.115^2) = 474.835$$

$$\begin{aligned}
 q &= \Sigma V / B' L' & \Sigma V &= 371.115 \\
 &= 371.115 / -0.469 \\
 &= -791.57 \text{ kPa} < 323.414 \quad \text{O.K} & Q \text{ (without vehicle impact)} &= -793.14 \text{ kPa}
 \end{aligned}$$

$$D / B' = 2.31 / -0.4688 = -4.927 < 0.125$$

$$\Sigma H / \Sigma V = 296.213 / 371.115 = 0.80 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{371.115}{3.115} \pm \frac{6 \times 371.115 \times 1.792}{3.115^2} \\
 P_b &= \frac{530.315}{-292.05} > 323.414 \quad \text{N.G} & \text{for footing design } P_a &= 530.208
 \end{aligned}$$

SLS

$$\begin{aligned}
 \Sigma V &= 1 \times 106.512 + 1.00 \times 190.380 + 0 + 0 \\
 &= 296.892 \text{ kN}
 \end{aligned}$$

$$M_r = 745.831 \text{ kN-m} \quad M_a = 714.227 \text{ kN-m}$$

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{296.892}{3.11517} \pm \frac{6 \times 296.892 \times 1.451}{3.11517^2} \\
 P_b &= \frac{361.656}{-171.05} > 161.707 \quad \text{N.G}
 \end{aligned}$$

$$\begin{aligned}
 e &= B/2 - (\Sigma M_r - \Sigma M_o) / \Sigma V \\
 &= 3.11517 / 2 - (745.831 - 714.227) / 296.892 \\
 &= 1.451 > B/6 = 3.115 / 6 = 0.519
 \end{aligned}$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_{sH} \times 1/2 \times H + 1.25 P_{cH} \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 126.233 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188 \\
 &\quad + 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9) \\
 &= 349.142 \text{ kN-m}
 \end{aligned}$$

At half of wall $t = 0.489 \text{ mm}$

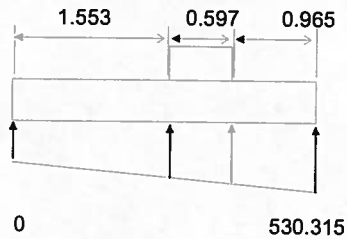
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times \frac{H}{2} + 1.25 P_s \times \frac{1}{2} \times \frac{H}{2} + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 31.558 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 65.631 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = -0.469 \text{ m}$

$$M_f = -791.57 \times -0.47 \times -0.47 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = -93.812 \text{ kN-m}$$

Heel design : Non yielding soil



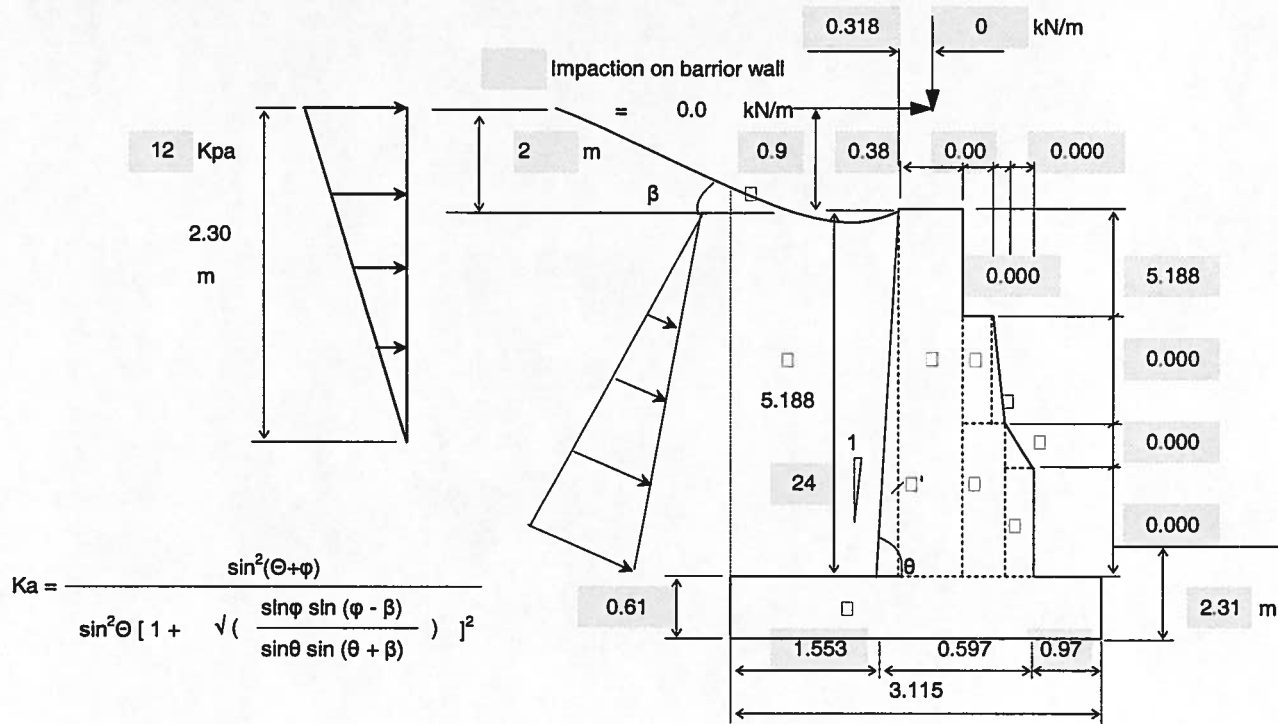
$$\begin{aligned}
 M_f &= (264.377 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.04 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 106.340 - 122.695 - 17.378 - 0.000 + 0.808 \\
 &= -32.924 \text{ kN-m}
 \end{aligned}$$

$$M_f = -32.924 \text{ kN-m}$$

ANALYSIS #1B

Retaining wall - Check stability at wall rotation angle of 2.6 deg. With No Ice Jacking Pressure

a. Load evaluation Geotechnical design : Do not include compact load and linear distribution for bearing resistance



$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.15643}{0.99912 \times 0.94777} \right)^2} \right]} = 0.474$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.474 \times 20 \times 6.39^2 / 2 \\ &= 193.795 \text{ kN/m} \\ P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 189.274 \text{ kN/m} \\ P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 41.615 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \\ \text{Effective passive depth} &= 2.31 \end{aligned}$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.474 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} \quad \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} \quad \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =					106.512		145.915
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.59614	0.926	20	18.52	2.598	48.106
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =					190.860		440.833

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L \\
 &= 1.25 \times 189.274 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / \quad 9.14 \\
 &= 253.843 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 190.860 + 41.615 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 256.315 \text{ kN/m} > Ph = 253.843 \text{ O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L \\
 &= 1 \times 189.274 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 \\
 &= 203.074 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 190.860 + 1 \times 41.615 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 380.424 \text{ kN/m} > 203.074 \text{ O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L \\
 &= 1.25 \times 189.274 / 3 \times 6.39414 + 1.25 \times 0.000 / 2 \times 6.39414 \\
 &\quad + 1.25 \times 13.8 \times (6.39 - 2.3 / 3) + 1 \times 0.00 \times (6.39414 + 0.9) \\
 &= 601.34 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 41.615 \times 3.115 + 145.9 + 440.833 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 749.56 \text{ kN-m} > Ma = 601.34 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L \\
 &= 1.0 \times 189.274 / 3 \times 6.39414 + 1 \times 0.000 / 2 \times 6.39414 \\
 &\quad + 1.00 \times 13.8 \times (6.39 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.39414 + 0.9) \\
 &= 481.074 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 749.56 \text{ kN-m} > Ma = 481.07 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 190.860 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 371.715 \text{ kN} \\
 Mr &= 749.564 \text{ kN-m} \quad Ma = 601.34 \text{ kN-m} \quad Ma = 601.34 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$\begin{aligned}
 q_u &= c N_{cs} i_c + q N_{qs} i_q + 0.5 \gamma' B N_{\gamma} s_{\gamma} i_{\gamma} \\
 &= 1.5 \times 30.1 \times 1.05 \times 0.383 + 46.20 \times 18.4 \times 1.05 \times 0.383 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.97 \times 0.021 \\
 &= 367.856 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_{\gamma} = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D\gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.05$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.05$$

$$s_{\gamma} = 1 - 0.4(B'/L') = 0.97$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (253.843 / 371.715) = 34.33 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.383$$

$$i_{\gamma} = (1 - \delta_i / \Phi')^2 = 0.021$$

a. Yielding soil condition

Eccentricity limit

$$\begin{aligned}
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V & e \text{ (without vehicle impact)} &= 1.16 \\
 &= 3.115 / 2 - (749.564 - 601.34) / 371.715 \\
 &= 1.159 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}
 \end{aligned}$$

$$\begin{aligned}
 B' &= B - 2e = 3.115 - 2 \times 1.159 = 0.797 \text{ m} & B' \text{ (for footing design)} &= 0.797 \text{ m} \\
 Q &= (253.843^2 + 371.715^2) = 450.12 \\
 q &= \Sigma V / B' L' & \Sigma V &= 371.715 \\
 &= 371.715 / 0.797 \\
 &= 466.295 \text{ kPa} > 367.856 \quad \text{N.G} & Q \text{ (without vehicle impact)} &= 466.10 \text{ kPa}
 \end{aligned}$$

$$\begin{aligned}
 D/B' &= 2.31 / 0.79717 = 2.898 > 0.125 \\
 \Sigma H / \Sigma V &= 253.843 / 371.715 = 0.68 > 0.55 \\
 \text{Reduction factor} &= 1 \times 750 = 750
 \end{aligned}$$

b. Linear elastic non yielding soil

$$\begin{aligned}
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{371.715}{3.115} \pm \frac{6 \times 371.715 \times 1.159}{3.115^2} \\
 P_b &= \frac{385.692}{-147.04} > 367.856 \quad \text{N.G} & \text{for footing design } P_a &= 385.655
 \end{aligned}$$

SLS

$$\begin{aligned}
 \Sigma V &= 1 \times 106.512 + 1.00 \times 190.860 + 0 + 0 \\
 &= 297.372 \text{ kN} \\
 M_r &= 749.564 \text{ kN-m} & M_a &= 481.074 \text{ kN-m} \\
 P_a &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{297.372}{3.11517} \pm \frac{6 \times 297.372 \times 0.655}{3.11517^2} \\
 P_b &= \frac{215.888}{-24.969} > 183.928 \quad \text{N.G} \\
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V \\
 &= 3.11517 / 2 - (749.564 - 481.074) / 297.372 \\
 &= 0.655 > B/6 = 3.115 / 6 = 0.519
 \end{aligned}$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 127.579 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188 \\
 &\quad + 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9) \\
 &= 352.051 \text{ kN-m}
 \end{aligned}$$

At half of wall $t = 0.489 \text{ mm}$

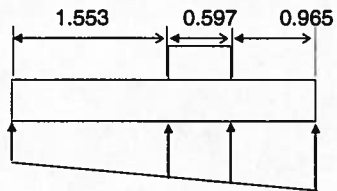
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H/2 + 1.25 P_s \times 1/2 \times H/2 + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 31.895 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 65.994 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.797 \text{ m}$

$$M_f = 466.30 \times 0.8 \times 0.8 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 141.343 \text{ kN-m}$$

Heel design : Non yielding soil



0

385.692

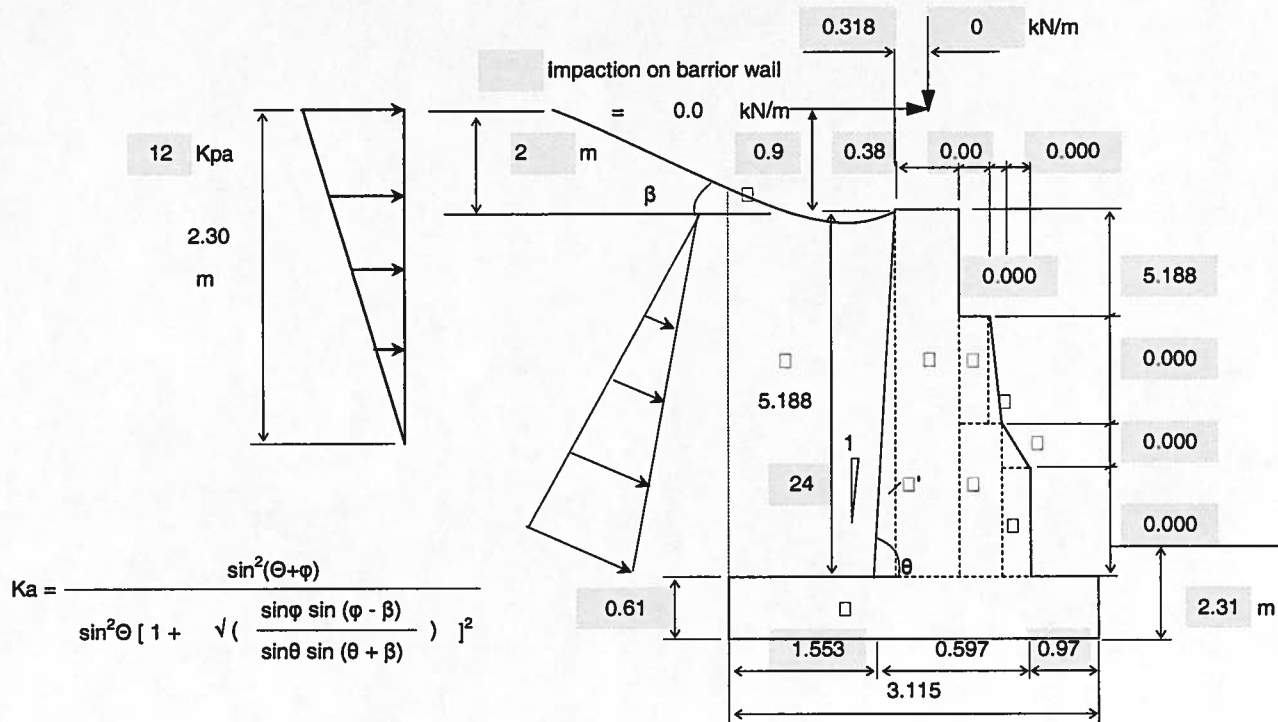
$$\begin{aligned}
 M_f &= (192.279 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.52 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 77.340 - 122.695 - 17.840 - 0.000 + 0.808 \\
 &= -62.386 \text{ kN-m}
 \end{aligned}$$

$$M_f = 141.343 \text{ kN-m}$$

ANALYSIS #1C

Retaining wall - Check stability at wall rotation angle of 2.8 deg. With No Ice Jacking Pressure

a. Load evaluation Geotechnical design : Do not include compact load and linear distribution for bearing resistance



$$K_a = \frac{\sin^2(\Theta + \phi)}{\sin^2\Theta \left[1 + \sqrt{\left(\frac{\sin\phi \sin(\phi - \beta)}{\sin\theta \sin(\theta + \beta)} \right)^2} \right]}$$

$$K_a = \frac{0.785}{1.00 \left[1 + \sqrt{\left(\frac{0.5 \times 0.15299}{0.99912 \times 0.94665} \right)^2} \right]} = 0.477$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3$$

1. Soil pressure

$$\begin{aligned} P_a &= K_a \gamma H^2 / 2 \\ &= 0.477 \times 20 \times 6.4^2 / 2 \\ &= 195.402 \text{ kN/m} \\ P_{ah} &= P_a \cos(90 - \Theta + \delta) \\ &= 190.844 \text{ kN/m} \\ P_{av} &= P_a \sin(90 - \Theta + \delta) \\ &= 41.960 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_p &= K_p \gamma H^2 / 2 \\ &= 3 \times 20 \times 2.31^2 / 2 \\ &= 160.083 \text{ kN/m} \\ \text{Effective passive depth} &= 2.31 \end{aligned}$$

2. Surcharge

$$\begin{aligned} P_s &= q H = 0 \times 20 \times 1.55 \\ &= 0 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} P_{sh} &= 0.477 \times 0 \times 20 \times 5.188 \\ &= 0.00 \text{ kN/m} \end{aligned}$$

3. Compaction

$$\begin{aligned} P_c &= 0.5 \times 12 \times 2.3 \\ &= 13.800 \text{ kN/m} \end{aligned}$$

4. Impaction

$$\begin{aligned} P_{ih} &= 0.00 \text{ kN/m} & \text{Factored} \\ P_{iv} &= 0 \text{ kN/m} & \text{Unfactored} \end{aligned}$$

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =					106.512		145.915
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.60237	0.935	20	18.7	2.598	48.573
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =					191.040		441.3

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L \\
 &= 1.25 \times 190.844 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / \quad 9.14 \\
 &= 255.805 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 191.040 + 41.960 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 256.588 \text{ kN / m} > Ph = 255.805 \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L \\
 &= 1 \times 190.844 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 \\
 &= 204.644 \text{ kN / m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 191.040 + 1 \times 41.960 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 380.766 \text{ kN / m} > 204.644 \quad \text{O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L \\
 &= 1.25 \times 190.844 / 3 \times 6.40037 + 1.25 \times 0.000 / 2 \times 6.40037 \\
 &\quad + 1.25 \times 13.8 \times (6.4 - 2.3 / 3) + 1 \times 0.00 \times (6.40037 + 0.9) \\
 &= 606.13 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 41.960 \times 3.115 + 145.9 + 441.3 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 751.37 \text{ kN-m} > Ma = 606.13 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L \\
 &= 1.0 \times 190.844 / 3 \times 6.40037 + 1 \times 0.000 / 2 \times 6.40037 \\
 &\quad + 1.00 \times 13.8 \times (6.4 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.40037 + 0.9) \\
 &= 484.902 \text{ kN-m} \\
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 751.37 \text{ kN-m} > Ma = 484.90 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 191.040 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 371.940 \text{ kN} \\
 Mr &= 751.374 \text{ kN-m} \quad Ma = 606.13 \text{ kN-m} \quad Ma = 606.13 \text{ kN-m (For footing design)}
 \end{aligned}$$

$$\begin{aligned}
 q_u &= c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma \\
 &= 1.5 \times 30.1 \times 1.05 \times 0.380 + 46.20 \times 18.4 \times 1.05 \times 0.380 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.97 \times 0.023 \\
 &= 365.958 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\pi \tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D\gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.05$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.05$$

$$s_\gamma = 1 - 0.4(B'/L') = 0.97$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (255.805 / 371.940) = 34.52 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.380$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.023$$

a. Yielding soil condition

Eccentricity limit

$$\begin{aligned}
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V & e \text{ (without vehicle impact)} &= 1.17 \\
 &= 3.115 / 2 - (751.374 - 606.13) / 371.940 \\
 &= 1.167 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}
 \end{aligned}$$

$$\begin{aligned}
 B' &= B - 2e = 3.115 - 2 \times 1.167 = 0.781 \text{ m} & B' \text{ (for footing design)} &= 0.781 \text{ m} \\
 Q &= (255.805^2 + 371.940^2) = 451.415 \\
 q &= \Sigma V / B' L' & \Sigma V &= 371.940 \\
 &= 371.940 / 0.781 \\
 &= 476.134 \text{ kPa} > 365.958 \quad \text{N.G} & Q \text{ (without vehicle impact)} &= 476.22 \text{ kPa}
 \end{aligned}$$

$$\begin{aligned}
 D/B' &= 2.31 / 0.78117 = 2.957 > 0.125 \\
 \Sigma H / \Sigma V &= 255.805 / 371.940 = 0.69 > 0.55 \\
 \text{Reduction factor} &= 1 \times 750 = 750
 \end{aligned}$$

b. Linear elastic non yielding soil

$$\begin{aligned}
 \frac{P_a}{P_b} &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{371.940}{3.115} \pm \frac{6 \times 371.940 \times 1.167}{3.115^2} \\
 &= \frac{387.766}{-148.97} > 365.958 \quad \text{N.G} & \text{for footing design } P_a &= 387.782
 \end{aligned}$$

SLS

$$\begin{aligned}
 \Sigma V &= 1 \times 106.512 + 1.00 \times 191.040 + 0 + 0 \\
 &= 297.552 \text{ kN} \\
 M_r &= 751.374 \text{ kN-m} & M_a &= 484.902 \text{ kN-m} \\
 \frac{P_a}{P_b} &= \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{297.552}{3.11517} \pm \frac{6 \times 297.552 \times 0.662}{3.11517^2} \\
 &= \frac{217.307}{-26.272} > 182.979 \quad \text{N.G} \\
 e &= B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V \\
 &= 3.11517 / 2 - (751.374 - 484.902) / 297.552 \\
 &= 0.662 > B/6 = 3.115 / 6 = 0.519
 \end{aligned}$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 128.386 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188 \\
 &\quad + 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9) \\
 &= 353.796 \text{ kN-m}
 \end{aligned}$$

total failure 2.8 deg

At half of wall $t = 0.489 \text{ mm}$

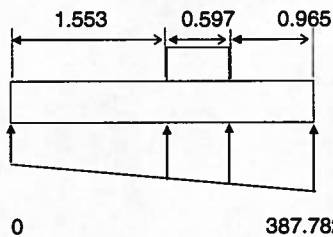
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times \frac{H}{2} + 1.25 P_s \times \frac{1}{2} \times \frac{H}{2} + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 32.097 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 66.213 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.781 \text{ m}$

$$M_f = 476.22 \times 0.78 \times 0.78 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 138.431 \text{ kN-m}$$

Heel design : Non yielding soil



$$\begin{aligned}
 M_f &= (193.32 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.7 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 77.759 - 122.695 - 18.013 - 0.000 + 0.808 \\
 &= -62.141 \text{ kN-m}
 \end{aligned}$$

$$M_f = 138.431 \text{ kN-m}$$

ANALYSIS #1D

5. Self weight

	B	H	A	w (kN/m ³)	Aw	x	Mx
□	3.115	0.610	1.9	24	45.6	1.558	71.026
□	0.381	5.188	1.977	24	47.448	1.156	54.826
□'	0.216	5.188	0.561	24	13.464	1.490	20.063
□	0.000	0.000	0	24	0	1.562	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
□	0.000	0.000	0	24	0	0.965	0
Sum =				106.512		145.915	
□	1.769	5.188	9.178	20	183.56	2.231	409.446
□	1.553	0.60861	0.945	20	18.9	2.598	49.093
- □'	0.216	5.188	0.561	20	11.22	1.490	16.719
Sum =				191.240		441.82	

b. Stability check

Horizontal resistance check

ULS

$$\begin{aligned}
 Ph &= 1.25P_{ah} + 1.25P_s + 1.25P_c + P_{ih} / L \\
 &= 1.25 \times 192.018 + 1.25 \times 0.000 + 1.25 \times 13.800 + \quad / \quad 9.14 \\
 &= 257.272 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= 0.8 \times (106.512 + 191.240 + 42.218 + 0 + 0) \times 0.65 + 0.5 \times 160.083 \\
 &= 256.826 \text{ kN/m} < Ph = 257.272 \quad \text{N.G}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ph &= 1.0P_{ah} + 1.0P_s + 1.0P_c + P_{ih} / 1.7 / L \\
 &= 1 \times 192.018 + 1 \times 0.000 + 1 \times 13.800 + 1 \times 0.00 / 1.7 / 9.14 \\
 &= 205.818 \text{ kN/m}
 \end{aligned}$$

$$\begin{aligned}
 Pr_h &= (1 \times 106.512 + 1 \times 191.240 + 1 \times 42.218 + 0 + 0.000) \times 0.65 + 160.083 \\
 &= 381.063 \text{ kN/m} > 205.818 \quad \text{O.K}
 \end{aligned}$$

Overturning check**ULS**

$$\begin{aligned}
 Ma &= 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / L \\
 &= 1.25 \times 192.018 / 3 \times 6.40661 + 1.25 \times 0.000 / 2 \times 6.40661 \\
 &\quad + 1.25 \times 13.8 \times (6.41 - 2.3 / 3) + 1 \times 0.00 \times (6.40661 + 0.9) \\
 &= 609.87 \text{ kN-m}
 \end{aligned}$$

$$\begin{aligned}
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 1.25 \times 42.218 \times 3.115 + 145.9 + 441.82 + 0 \times 1.028 + 1.25 \times 0.00 \times 2.339 \\
 &\quad + 0.5 \times 1.54 \\
 &= 752.90 \text{ kN-m} > Ma = 609.87 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

SLS

$$\begin{aligned}
 Ma &= 1.0 P_{ah} \times 1/3 \times H + 1.0 P_s \times 1/2 \times H + 1.0 \times P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h) / 1.7 / L \\
 &= 1.0 \times 192.018 / 3 \times 6.40661 + 1 \times 0.000 / 2 \times 6.40661 \\
 &\quad + 1.00 \times 13.8 \times (6.41 - 2.3 / 3) + 1.00 \times 0.00 / 1.7 \times (6.40661 + 0.9) \\
 &= 487.892 \text{ kN-m}
 \end{aligned}$$

$$\begin{aligned}
 Mr &= P_v \times d_h + M_{con} + M_{fill} + M_{barrier} + M_{surcharge} + M_{passive} \\
 &= 752.90 \text{ kN-m} > Ma = 487.89 \text{ kN-m} \quad \text{O.K}
 \end{aligned}$$

Geotechnical resistance check**ULS**

$$\begin{aligned}
 \Sigma V &= 1.25 \times 106.512 + 1.25 \times 191.240 + 1.25 \times 0 + 1.25 \times 0.000 \\
 &= 372.190 \text{ kN}
 \end{aligned}$$

$$Mr = 752.899 \text{ kN-m} \quad Ma = 609.87 \text{ kN-m} \quad Ma = 609.87 \text{ kN-m (For footing design)}$$

$$q_u = c N_c s_c i_c + q N_q s_q i_q + 0.5 \gamma' B N_\gamma s_\gamma i_\gamma$$

$$\begin{aligned}
 &= 1.5 \times 30.1 \times 1.05 \times 0.378 + 46.20 \times 18.4 \times 1.04 \times 0.378 \\
 &\quad + 0.50 \times 20.0 \times 3.12 \times 15.070 \times 0.97 \times 0.024 \\
 &= 364.624 \text{ kPa}
 \end{aligned}$$

$$N_q = (e^{\pi \tan \Phi'}) (1 + \sin \Phi') / (1 + \sin \Phi') = 18.4$$

$$N_c = (N_q - 1) \cot \Phi' = 30.1$$

$$N_\gamma = 3/2 (N_q - 1) \tan \Phi' = 15.1$$

$$q' = D \gamma' = 46.2 \text{ kPa}$$

$$s_c = 1 + (B'/L') (N_q / N_c) = 1.05$$

$$s_q = 1 + (B'/L') \tan \Phi' = 1.04$$

$$s_\gamma = 1 - 0.4(B'/L') = 0.97$$

$$c = 1.5$$

$$\delta_i = \tan^{-1} (\text{horizontal force} / \text{vertical force}) = \tan^{-1} (257.272 / 372.190) = 34.65 \text{ degree}$$

$$i_c = i_q = (1 - \delta_i / 90^\circ)^2 = 0.378$$

$$i_\gamma = (1 - \delta_i / \Phi')^2 = 0.024$$

a. Yielding soil condition

Eccentricity limit

$$e = B/2 - (\Sigma M_r - \Sigma M_a) / \Sigma V \quad e \text{ (without vehicle impact)} = 1.17$$

$$= 3.115 / 2 - (752.899 - 609.87) / 372.190$$

$$= 1.173 \text{ m} > 0.3B = 0.93 \text{ m} \quad \text{N.G} \quad \text{MTO Seminar 1 and 2 Section 6 Foundation}$$

$$B' = B - 2e = 3.115 - 2 \times 1.173 = 0.769 \text{ m} \quad B' \text{ (for footing design)} = 0.769 \text{ m}$$

$$Q = (257.272^2 + 372.190^2) = 452.454$$

$$q = \Sigma V / B' L' \quad \Sigma V = 372.190$$

$$= 372.190 / 0.769$$

$$= 483.887 \text{ kPa} > 364.624 \quad \text{N.G} \quad Q \text{ (without vehicle impact)} = 484.24 \text{ kPa}$$

$$D / B' = 2.31 / 0.76917 = 3.003 > 0.125$$

$$\Sigma H / \Sigma V = 257.272 / 372.190 = 0.69 > 0.55$$

$$\text{Reduction factor} = 1 \times 750 = 750$$

b. Linear elastic non yielding soil

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{372.190}{3.115} \pm \frac{6 \times 372.190 \times 1.173}{3.115^2}$$

$$P_b = \frac{389.407}{-150.45} > 364.624 \quad \text{N.G} \quad \text{for footing design} \quad P_a = 389.471$$

SLS

$$\Sigma V = 1 \times 106.512 + 1.00 \times 191.240 + 0 + 0$$

$$= 297.752 \text{ kN}$$

$$M_r = 752.899 \text{ kN-m} \quad M_a = 487.892 \text{ kN-m}$$

$$P_a = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V e}{B^2} = \frac{297.752}{3.11517} \pm \frac{6 \times 297.752 \times 0.668}{3.11517^2}$$

$$P_b = \frac{218.557}{-27.394} > 182.312 \quad \text{N.G}$$

$$e = B/2 - (\Sigma M_r - \Sigma M_o) / \Sigma V$$

$$= 3.11517 / 2 - (752.899 - 487.892) / 297.752$$

$$= 0.668 > B/6 = 3.115 / 6 = 0.519$$

c. Structural designAt the top of footing $t = 0.597 \text{ mm}$

$$M_f = 1.25 P_{ah} \times 1/3 \times H + 1.25 P_s \times 1/2 \times H + 1.25 P_c \times (H - 1/3 \times 2.3) + P_{ih} \times (H + h)$$

$$= 1.25 \times 128.924 / 3 \times 5.188 + 1.25 \times 0.000 / 2 \times 5.188$$

$$+ 1.25 \times 13.8 \times (5.19 - 2.3 / 3) + 0.00 \times (5.188 + 0.9)$$

$$= 354.960 \text{ kN-m}$$

total failure

At half of wall $t = 0.489 \text{ mm}$

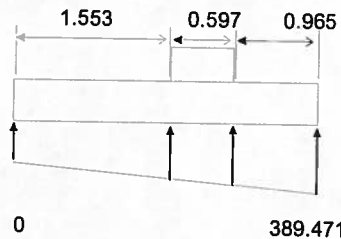
$$\begin{aligned}
 M_f &= 1.25 P_{ah} \times \frac{1}{3} \times \frac{H}{2} + 1.25 P_s \times \frac{1}{2} \times \frac{H}{2} + 1.25 P_c \times (H - \frac{1}{3} \times 2.3) + P_{ih} \times (H + h) \\
 &= 1.25 \times 32.231 / 3 \times 2.594 + 1.25 \times 0.000 / 2 \times 2.594 \\
 &\quad + 1.25 \times 13.8 \times (2.59 - 2.3 / 3) + 0.00 \times (2.594 + 0.9) \\
 &= 66.358 \text{ kN-m}
 \end{aligned}$$

Footing

Toe design :Yielding soil $B' = 0.769 \text{ m}$

$$M_f = 484.24 \times 0.77 \times 0.77 / 2 - 0.61 \times 0.97^2 / 2 \times 24 = 136.217 \text{ kN-m}$$

Heel design : Non yielding soil



$$\begin{aligned}
 M_f &= (194.163 + 0) / 2 \times 1.553 \times 0.518 \\
 &\quad - \{ 183.56 \times (1.769 / 2 - 0.216) \} \\
 &\quad - 18.9 \times (1.769 \times 2 / 3 - 0.216) \\
 &\quad - 1.25 \times 0.000 \times 1.553^2 / 2 \\
 &\quad + 11.22 \times 0.216 / 3 \\
 &= 78.097 - 122.695 - 18.206 - 0.000 + 0.808 \\
 &= -61.995 \text{ kN-m}
 \end{aligned}$$

$$M_f = 136.217 \text{ kN-m}$$



APPENDIX FDR-3

NSSP FOR SUPPLY AND INSTALLATION OF RETAINING WALL MONITORING INSTRUMENTATION AND EQUIPMENT



SUPPLY AND INSTALLATION OF RETAINING WALL MONITORING INSTRUMENTATION AND EQUIPMENT – Item No.

Non Standard Special Provision

April 2012

-
- 1.0 SCOPE**
 - 2.0 REFERENCES**
 - 3.0 DEFINITIONS**
 - 4.0 DESIGN AND SUBMISSION REQUIREMENTS**
 - 5.0 MATERIALS**
 - 6.0 EQUIPMENT**
 - 7.0 CONSTRUCTION**
 - 8.0 MEASUREMENT FOR PAYMENT**
 - 9.0 BASIS OF PAYMENT**

1.0 SCOPE

This Special Provision contains the requirements for the supply and installation of Displacement Pins (DP) and Tilt Plates (TP) to monitor lateral displacement and tilting during replacement of the backfill material behind the northeast retaining wall located at Highway 7/85 Frederick Street underpass.

The purpose of the Displacement Pins is to directly monitor lateral displacement of the top of the retaining wall. Displacement is measured by survey of the top of the pin with reference to stable, non-settling benchmarks.

The purpose of the Tilt Plates is to directly monitor tilt and relative lateral displacement of the top of the retaining wall. Tilt measurements (magnitude and rate) shall help to identify earlier failure movements.

The rate of fill removal and placement and the timing for construction of the retaining wall remediation shall be controlled by the instrumentation readings.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete



Ontario Provincial Standards Specifications, Material

OPSS1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS PROV 1350 Concrete – Materials and Production

3.0 DEFINITIONS

Geotechnical Engineering Consultant means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - High Complexity”, to undertake the supply and installation of geotechnical instruments.

Benchmark means a non-yielding, deep-seated survey reference point.

Monitoring Program means the monitoring readings conducted by others as part of the Contract Administration Assignment.

Displacement Pin means a bolt or specialized survey marker permanently embedded in a concrete plug for purposes of displacement monitoring. The pin should be readable to survey instruments capable of level 1 survey (accuracy of 1 mm).

Tilt Plate means a plate installed at the defined location for purpose of tilt monitoring. An example of this instrumentation is enclosed in Appendix FDR-3A.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

4.01.01 Survey Benchmarks

The Contractor shall provide non-yielding temporary benchmarks relative to the existing Benchmark on site as necessary such that direct sighting is possible from all Displacement Pins (DP) to at least one temporary benchmark or the Benchmark.

The locations of the temporary benchmarks are to be approved by the Contract Administrator prior to installation of the monitoring instruments.

4.01.02 Marking and Labelling

The location of monitoring fixture shall be made clearly visible to construction traffic before, during and after replacement of backfill material behind the retaining wall.



4.01.03 Protection of Instruments

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Contract Administrator.

4.02 Submission Requirements

4.02.01 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

4.02.02 Installation Methods

The Contractor shall submit details of proposed installation methods, including location and types of survey equipment, monitoring enclosure if required, temporary benchmarks and installation schedule, to the Contract Administrator a minimum of 15 days before the start of instrument installation.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise. The installations shall not affect the structural integrity of the retaining wall.

5.02 Displacement Pins (DP)

5.02.01 Grout or Concrete

Grout or epoxy products that will permanently bond the displacement pin to the top of the wall shall be used as required by the type and design of the selected displacement pin type. These materials shall have non-shrink characteristics and be capable of adequately withstanding winter and summer temperatures.

If required, concrete (OPSS PROV 1350) for anchoring the displacement pin shall be minimum 25 MPa compressive strength and set time sufficient to secure the pin within two days of pouring.

5.02.02 Displacement Pin

The displacement pin shall be capable of being surveyed by a level survey instrument to a minimum horizontal displacement accuracy of 1 mm.

5.03 Tilt Plates (TP)

5.03.01 Tilt Plates

The tilt plates shall be:

- Slope Indicator model 50307300; or



- Equal.

The TPs shall be compatible with the Slope Indicator Portable Tiltmeter, model 50304410, or equal. All TPs shall be of the same make/supplier.

5.03.02 Data Acquisition System (Portable Tiltmeters)

One Portable Tiltmeter shall be supplied for this project. The data acquisition system shall be from the same supplier as the TPs and shall consist of:

- Slope Indicator Model 50304410; or
- Equal

Calibration Certificate for tilt meter shall be supplied for this project.

6.0 EQUIPMENT

6.01 Equipment Operation and Weather Conditions

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring will be conducted year round as part of the Monitoring Program.

7.0 CONSTRUCTION

7.01 Instrumentation Installation

All instruments shall be placed on the top of the retaining wall as specified in Table 1.

7.01.01 Instrument Locations

The quantity and location of instruments are shown in Table 1 below.

Table 1 – Instrument Quantities and Locations

Retaining Wall Panel No. (*)	Location of Instrument from South end of Each Panel (m)	Number of Instruments	
		DP	TP
12	0.3	–	1
	0.9	1	–
	8.2	1	–
	8.8	–	1
13	0.3	–	1
	0.9	1	–
	8.2	1	–
	8.8	–	1



Retaining Wall Panel No. (*)	Location of Instrument from South end of Each Panel (m)	Number of Instruments	
		DP	TP
14	0.3	—	1
	0.9	1	—
	8.2	1	—
	8.8	—	1
15	0.3	—	1
	0.9	1	—
	8.2	1	—
	8.8	—	1

(*) Retaining wall panel number increases south to north direction from abutment. Panel numbers are noted in the previous Contract Drawing D-6049-12.

The Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

The locations of the monitoring instruments should be adjusted in the field such that they will not be damaged by the excavation procedures for the fill replacement, by highway maintenance equipment on the existing highway, or by earth moving equipment.

7.01.02 Installation Program

Instrument installation shall be completed a minimum of 3 weeks before the construction. Excavation and backfilling behind the retaining wall may not proceed until the instrument is fully installed and baselined.

7.01.03 Displacement Pins

7.01.03.01 General

The elevation, easting and northing of the top of the pins shall be surveyed after installation.

The locations of the Displacement Pins are shown in the Contract Documents and in Table 2 below.

Table 2 - Approximate Displacement Pin Locations

Retaining Wall Panel No.	Location of Displacement Pin from South end of each Panel	Instrument Number
12	0.9	DP#1
	8.2	DP#2
13	0.9	DP#3
	8.2	DP#4



Retaining Wall Panel No.	Location of Displacement Pin from South end of each Panel	Instrument Number
14	0.9	DP#5
	8.2	DP#6
15	0.9	DP#7
	8.2	DP#8

(*) Retaining wall panel number increases south to north direction from abutment. Panel numbers are noted in the previous Contract Drawing D-6049-12.

7.01.03.02 Displacement Pins

The Displacement Pins shall be cast into a concrete hole placed in a hole at the top of the retaining wall.

7.01.04 Tilt Plates

7.01.04.01 General

Installation of the Tilt Plates shall be as per the manufacture's recommendations in addition what is stated below.

The elevation, easting and northing of the top of the tilt plates shall be surveyed after installation.

The locations of the tilt plates are shown in Table 3 below.

Table 3 - Approximate Tilt Plate Locations

Retaining Wall Panel No.	Location of Tilt Plate from South end of each Panel	Instrument Number
12	0.3	TP#1
	8.8	TP#2
13	0.3	TP#3
	8.8	TP#4
14	0.3	TP#5
	8.8	TP#6
15	0.3	TP#7
	8.8	TP#8

(*) Retaining wall panel number increases south to north direction from abutment. Panel numbers are noted in the previous Contract Drawing D-6049-12.

The Tilt Plates shall be installed on top of the retaining wall.



7.02 Coordination with Monitoring Program

7.02.01 Notification

The Contractor shall notify the Contract Administrator no later than 3 days after the completion of installation of Displacement Pins and Tilt Plates.

7.02.02 Reporting

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within 3 days of completion of installation of each instrument.

7.02.02.01 Displacement Pins (DP)

- DP location, northing and easting (NAD83 MTM coordinates);
- Geodetic elevation of top of pin;
- Dates of installation; and
- Installation notes / sketches.

7.02.02.02 Tilt Plates (TP)

- TP location, northing and easting (NAD83 MTM coordinates);
- Geodetic elevation of top of plate;
- Dates of installation;
- Installation notes / sketches; and
- Model, make and serial numbers of TPs and readout unit

7.02.03 Monitoring

7.02.03.01 Displacement Pins

Monitoring of the Displacement Pins shall be done by others working for the Contract Administrator/MTO. Monitoring shall be conducted during and after excavation and backfilling behind the retaining wall. The Contractor shall provide installation information as specified above and provide access to the Displacement Pins for monitoring.

7.02.03.02 Tilt Plates

Monitoring of the Tilt Plates shall be done by others for the Contract Administrator/MTO. Monitoring shall be conducted during and after the construction. The Contractor shall provide installation information as specified above and provide access to the Tilt Plates for monitoring, including, but not limited to a scaffolding platform and ladder if required and snow clearing in the winter. The contractor shall provide electric power and general area lighting as needed for reading the instruments.



8.0 MEASUREMENT FOR PAYMENT

Measurement will be made of the number of units of Displacement Pins and Tilt Plates.

9.0 BASIS OF PAYMENT

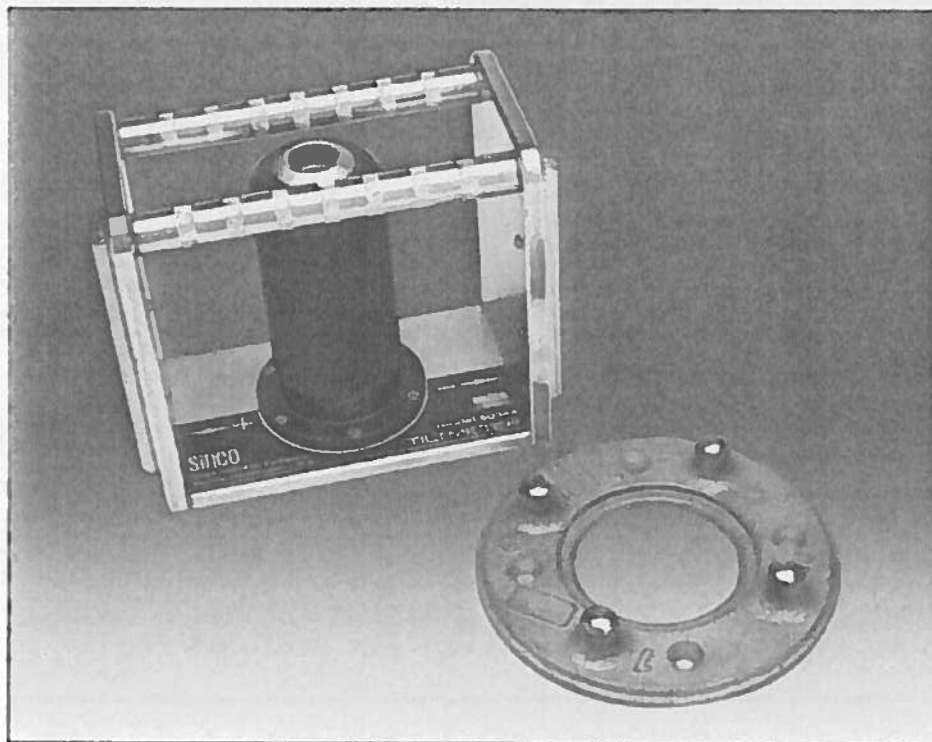
Payment at the Lump Sum price for this tender item shall be full compensation for all labour, supply of monitoring instruments and equipment and material to complete the work.



APPENDIX FDR-3A

PORTABLE DIGITILT TILTMETER

Portable Digitilt Tiltmeter



Applications

The portable tiltmeter is used to monitor changes in the inclination of a structure.

- Monitoring rotation caused by mining, tunneling, soil compaction, or excavation.
- Monitoring rotation of retaining walls, piers, and piles.

Operation

A tiltmeter system includes tilt plates, the portable tiltmeter, and a readout unit.

The tilt plates are mounted on the structure in specified locations. They are typically bonded to the structure, but may also be screwed to the surface.

To obtain readings, the operator connects the tiltmeter to the readout unit and positions the tiltmeter on the tilt plate. The bottom surface of the tiltmeter is used with horizontal tilt plates, and the side surfaces of the tiltmeter are used with vertical tilt plates.

After noting the displayed reading, the operator rotates the tiltmeter 180° and obtains a second reading. The two readings are later combined.

Changes in tilt are found by comparing the current reading to the initial reading and converting the results to angles or displacements.

Advantages

Economical: One tiltmeter can be used to monitor any number of inexpensive tilt plates.

Easy to Install: Bronze tilt plates can be bonded or screwed to the structure.

Easy to Use: Tilt readings are obtained quickly and easily by a single operator.

Rugged, Reliable, and Accurate: The tiltmeter uses the same proven force-balanced, servo-accelerometer technology used in the Digitilt inclinometer probe.

PORTABLE TILTMETERS**Metric Tiltmeter 50304410**

Includes case and jumper cable (3 m) for connecting to readout.

Sensor: Digitilt uniaxial force-balanced servo-accelerometer.

Range: $\pm 53^\circ$ from vertical.

Resolution: 8 arc seconds.

System Repeatability: ± 50 arc seconds.

Materials: Stainless steel frame, anodized aluminum housing.

Dimensions: 152 x 89 x 178 mm.

Weight: 4.5 kg.

English Tiltmeter 50304400

Includes case and jumper cable (10') for connecting to readout.

Sensor: Digitilt uniaxial force-balanced servo-accelerometer.

Range: $\pm 35^\circ$ from vertical.

Resolution: 10 arc seconds.

System Repeatability: ± 50 arc seconds.

Materials: Stainless steel frame, anodized aluminum housing.

Dimensions: 7 x 10 x 7 inch.

Weight: 10 lb.

TILT PLATES**Tilt Plate 50307300**

Mounting Method: Epoxy bonding compound or screws.

Material: Bronze.

Diameter: 140 mm (5.5").

Height: 24 mm (0.95").

Center Hole: 63 mm (2.5").

Weight: 0.68 kg (1.5 lb).

Tilt Plate Cover 50307350

Includes anchors.

Epoxy Bonding Compound 50305500

0.45 kg (1 lb). For mounting up to five tilt plates.

Requires ambient temperature above 4.5°C (40°F) for curing.

READOUTS

Compatible readouts include the Digitilt Data-Mate and the Digitilt 09. See separate data sheet for features and specifications.





APPENDIX FDR-4

GEOTECHNICAL (FOUNDATION SPECIALTY) MONITORING PLAN – NORTHEAST CORNER RETAINING WALL AT FREDERICK STREET UNDERPASS



Geotechnical (Foundation Speciality) Monitoring Plan – Northeast Corner Retaining Wall at Frederick Street underpass, Site No. 33-234, City of Kitchener

1.0 GENERAL

Requirements specified for Specialist Qualifications; Services, Deliverables and Records; and the Retaining Wall Monitoring Plan apply to all the Instrumentation Monitoring. Instrumentation monitoring is required for the following geotechnical instruments:

- Displacement Pins (DPs); and
- Tilt Plates (TPs);

The instrumentation monitoring services include: data collection, data reduction and reporting; adherence to criteria used to assess the retaining wall failure movements based on the monitoring data collected from the instrumentation installed by others.

1.0.1 Specialist Qualifications

The Foundation Engineering Consultant services required for this assignment have been categorized as Geotechnical Specialty - High Complexity.

The Foundation Engineering Consultants that are registered in MTO's Consultant Registry Appraisals and Qualifications System (RAQS) at the complexity rating in the required specialty that meets the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Consultant shall not be the same Instrument Installation Consultant retained by the Contractor for the supply and installation of retaining wall monitoring equipment.

1.0.2 Services, Deliverables and Records

The Foundation Engineering Consultant shall:

- Review the monitoring program and, if deemed necessary, submit in writing to the Contract Administrator recommendations for modifications to the Monitoring Program;
- Meet with the Contractor in order to receive reports with details about installation of instruments installed by the Contractor and calibration certificates, as specified in the Special Provision titled "Supply and Installation of Retaining Wall Monitoring Instrumentation and Equipment" and included in the contract documents;
- The Foundation Engineering Consultant is required on site to establish the baseline readings. The Contract Administrator staff may take all other required readings provided they are immediately forwarded to the Foundation Engineering Consultant.
- Supply all materials and equipment that are required for the Monitoring Program;
- Calibrate and maintain monitoring equipment as required;
- Take instrument readings, reduce data, prepare reports;
- Provide transmittal of instrumentation readings and reports to the Contract Administrator;



- Interpret instrumentation readings as needed for the purpose of ongoing construction;
- Notify the Contract Administrator of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making correlations between instrumentation data and specific construction activities; and
- Notify the Contract Administrator if critical instrument readings, as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s), and submit a plan of actions, to prevent the critical instrument readings (i.e. Review/Alert levels) from being exceeded.

Progress Reports shall be submitted to the Contract Administrator, the Ministry's Contract Control Officer and Foundations Engineer. Weekly reports shall be issued from the beginning of construction monitoring to the end of the construction period (fill replacement behind the wall). Thereafter, one report shall be submitted after each set of readings is taken. The Progress Reports shall discuss the Contractor's operations with respect to the installation of instrumentation and/or a summary of the monitoring that was completed.

The Foundation Engineering Consultant (in co-operation with the Contract Administrator, if applicable) shall maintain a Retaining Wall Monitoring diary. The diary shall document original conditions, work in progress, including any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions.

1.0.3 Submission of Retaining Wall Monitoring Plan

The Foundation Engineering Consultant shall, in a brief narrative, discuss the applicable experience and qualifications of specialist staff, the role each will play in administration of the Contract, the authority to be assumed, and the reporting relationships with the construction administration staff.

The Consultant shall also complete the Retaining Wall Monitoring Plan table in the format provided below.

Retaining Wall Monitoring Plan		
<i>Major Inspection Tasks</i>	<i>Level of Inspection</i>	<i>Deliverable Record(s)</i>
List major inspection tasks associated with retaining wall monitoring.	State frequency/level of inspection.	List associated Deliverable Records for each task.

1.0.4 Purpose

The purpose of the Displacement Pins is to directly monitor displacement of the top of the retaining wall. Displacement is measured by survey of the top of the pin with reference to stable, non-settling benchmarks.



The purpose of the Tilt Plates is to directly monitor tilt and lateral displacement of the top of the retaining wall. Tilt measurements (magnitude and rate) shall help to identify earlier failure movements.

The rate of fill removal and placement and the timing for construction of the retaining wall remediation shall be controlled by the instrumentation readings.

1.0.5 Subsurface Conditions

The subsurface conditions at the site are described in the following report:

- Foundation Investigation Report-Northeast Corner Retaining Wall, Frederick Street Underpass, Site No. 33-234, City of Kitchener, GWP 3110-09-00, Geocres No. 40P8-199, dated May 31, 2012 by Peto MacCallum Ltd.

1.0.6 Equipment Operation

- Monitoring shall be conducted year round. All monitoring equipment shall be maintained and rendered operational throughout the monitoring period.
- Any equipment malfunction shall be investigated and attempts shall be made to remedy the malfunction. Notification of any equipment malfunction and equipment that cannot be repaired shall be made to the Contract Administrator. Documentation of the possible causes and suggested remedial measures shall be forwarded to the Contract Administrator / MTO Pavements and Foundations Section.

1.0.7 Reading Schedule and Frequency

- The Foundation Engineering Consultant shall save and archive raw data in electronic and hard copy format.
- Monitoring shall commence immediately after the Instrumentation Installation Consultant has confirmed proper functioning of the instrumentation (i.e. has satisfied himself that a proper baseline has been established). Monitoring is to be carried out during backfilling replacement (i.e. DPs and TPs). The actual length of the monitoring period depends on the construction schedule and on the results of the monitoring. Termination of the monitoring program shall be as directed by the MTO Pavements and Foundations Section.
- The minimum monitoring frequencies along with the anticipated number of readings for the monitoring areas in this contract are given in Tables 1a. Instruments shall be read more or less frequently if judged to be required by the Contract Administrator / MTO Pavements and Foundations Section.
- It should be noted that the number of readings given in Table 1a are estimates and may vary depending on the actual construction schedule.



Table 1a - Minimum Monitoring Frequency of DPs and TPs

STAGE	FREQUENCY	ANTICIPATED NO. OF SITE VISITS (**)
Baseline readings (*) (after installation)	Once a day for 3 consecutive days following completion of installation	3
During backfill replacement	Twice a day	28
After backfill replacement	Daily – For 2 weeks Weekly – For 6 weeks Monthly – For 1 year Bi-monthly – Afterwards	14 6 12 varies

(*) Baseline readings: Value of instrument readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the retaining wall. The instruments shall be completely and firmly set prior to starting the readings.
 (**) Number of site visits to obtain readings may vary.

2.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

2.0.1 Displacement Pins

Surveying

The horizontal displacement of the Displacement pins (DPs) shall be surveyed to an accuracy of plus/minus 1 mm, or better, and shall be reported to the nearest millimetre.

Surveying for displacement monitoring shall be conducted by a surveyor with appropriate equipment and experience to meet the required accuracy requirements. The surveyor shall be capable of surveying in type of Displacement pin installed on the retaining wall.

Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator/MTO Pavements and Foundations Section within one (1) working day during construction and on a weekly basis thereafter, after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum, the following shall be submitted to the Contract Administrator/MTO Pavements and Foundations Section in the Progress Reports for each Displacement pin based on the readings collected from the DPs.

- A plot of level of the backfill during replacement versus time;



- A plot of displacement versus time;
- Review and alert levels on DPs plots; and
- Plan view, cross-section and profile sketches showing the backfill replacement while the DPs are being surveyed

Review Levels and Alert Levels

Review and Alert Levels are provided below for the monitoring of the retaining wall movement during construction. For this project it is assumed that the construction will be completed within two (2) weeks. The Foundation Design Engineer should review and adjust, if required, the Review and Alert Levels based on the actual construction schedule.

Typically, retaining wall failures result in acceleration of displacements during excavation and after placement of fill and during spring/winter seasons. If such a condition is observed or the maximum displacement measured exceeds the respective Review Level in Table 2a, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and discuss response action(s) to prevent the Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached. In addition, after backfill replacement (construction) in case the maximum measured displacement exceeds the respective Review Levels indicated in Table 2a during or after construction, the Foundation Engineering Consultant shall immediately inform the MTO Pavements and Foundations Section engineer.

If the maximum displacements/tilts measured exceed the respective Alert Level in Table 2a, the Foundation Engineering consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities. No construction shall take place until all the following conditions are satisfied.

- The cause of the accelerated displacement has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.
- After construction, any corrective action/mitigation measures including roadway protection measures deemed to be required by the Foundation Design Engineer and approved by MTO Pavements and Foundations Section should be implemented.

Table 2a – Review Levels and Alert Levels for Displacement Pins

Retaining Wall Panel No. (*)	Location of Displacement Pin from South end of each Panel	Instrument Number	Displacement Response Levels (mm)	
			Review	Alert
12	0.9	DP#1	2	5
	8.2	DP#2	2	5



Retaining Wall Panel No. (*)	Location of Displacement Pin from South end of each Panel	Instrument Number	Displacement Response Levels (mm)	
			Review	Alert
13	0.9	DP#3	2	5
	8.2	DP#4	2	5
14	0.9	DP#5	2	5
	8.2	DP#6	2	5
15	0.9	DP#7	2	5
	8.2	DP#8	2	5

(*) Retaining wall panel number increases south to north direction from abutment. Panel numbers are noted in the previous Contract Drawing D-6049-12.

Note: If the measured rate of Displacement is greater than that predicted, notify the Contract Administrator / MTO Pavements and Foundations Section.

2.0.2 Tilt Plates

Readout Unit

The Tilt Plates (TPs) shall be read using the readout supplied by the Contractor.

Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within one (1) working day during construction and on a weekly basis thereafter, after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum the following shall be submitted to the Contract Administrator in the Progress Reports:

- Plots of tilts versus time for TPs and the respective retaining wall excavation and backfill reference;
- Plan view, cross section and profile sketches showing the top of fill location while the TPs readings were being taken;
- Completed fill replacement and date of completion on the tilt plots; and
- Review and Alert Levels on the TP plots.

Review Levels and Alert Levels

If the maximum tilt measured exceeds the respective Alert Level in Table 2b, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment until all the following conditions are satisfied. In addition, after backfill replacement



(construction) maximum tilt measured exceeds the respective Review Levels indicated in Table 2b after construction, the Foundation Engineering Consultant shall immediately inform the MTO Pavements and Foundations Section. :

- The cause of the excess tilt has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed;
- After construction, any corrective action/mitigation measures including roadway protection measures deemed to be required by the Foundation Design Engineer and approved by MTO Pavements and Foundations Section should be implemented.

Table 2b - Review Levels and Alert Levels for tilts for TPs

Retaining Wall Panel No. (*)	Location of Tilt Plate from South end of each Panel	Instrument Number	Tilt Response Levels (°)	
			Review	Alert
12	0.3	TP#1	0.1	0.2
	8.8	TP#2	0.1	0.2
13	0.3	TP#3	0.1	0.2
	8.8	TP#4	0.1	0.2
14	0.3	TP#5	0.1	0.2
	8.8	TP#6	0.1	0.2
15	0.3	TP#7	0.1	0.2
	8.8	TP#8	0.1	0.2

(*) Retaining wall panel number increases south to north direction from abutment. Panel numbers are noted in the previous Drawing D-6049-12.

Note: If the measured rate of Displacement is greater than that predicted, notify the Contract Administrator / MTO Pavements and Foundations Section.

3.0 CONTROL MONITORING LEVELS

3.0.1 General

The monitoring program will provide input to identify earlier failure movements during backfill replacement and after construction.

4.0 FINAL REPORT

At the completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator and/or MTO Pavements and Foundations Section, as applicable. The monitoring results shall be presented in tabular and graphical form as described above for each instrument type. An interpretation of the monitoring readings shall be included in the report.